Islamic Republic of Afghanistan

Ministry of Rural Rehabilitation and Development

Low Volume Rural Roads Guideline and Standards

Volume 1
Pavement Design

July 2020

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ISBN: 978-9936-8080-0-3

Printed by: TAMADON Photography, Digital & Offset Printing
Kot-e-Sangi, Dehboori Road, Atifaq Market
Kabul
Afghanistan

Layout: Infra Africa (Pty) Ltd

Foreword

This Pavement Design Guideline for Low Volume Rural Roads (LVRRs) applies to Tertiary Roads and minor Secondary Roads. The effective management of this important component of the classified road network in Afghanistan 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 Guideline 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 planning and designing unpaved roads or undertaking their upgrading to a paved standard. The Guideline takes account of best practice developments in low volume roads technology that have evolved both regionally and internationally in the past few decades. In so doing, it retains those aspects of the existing MRRD documents that are relevant to LVRRs and addresses the gaps that have been identified from their evaluation to produce a new self-standing Guideline that will replace the current guidelines.

The Ministry of Rural Rehabilitation and Development (MRRD), therefore, expects all practitioners in the roads sector to adhere to the standards set out in the Guideline. This will ensure that a consistent, harmonized approach is followed in the design of low volume roads in the country.

The development of the Guideline was overseen by a Technical Steering Committee comprising representatives from MRRD. By its very nature, the Guideline will require periodic updating to take account of the dynamic developments in low volume roads technology. MRRD would, therefore, welcome comments and suggestions from any stakeholders as feedback on all aspects of the Guideline during its implementation. All feedback will be carefully considered by professionals and experts in future updates of the Guideline.

On behalf of the MRRD, I would like to thank UK Aid through the United Kingdom Department for International Development (DFID) for its support of the development of the Guideline. 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, l would commend all the road sector stakeholders who contributed their time, knowledge, and effort during the development of the Guideline.

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

Assistant Professor, Mujeeb Rahman Karimi
Minister, Ministry of Rural Rehabilitation and Development (MRRD)

Acknowledgements

The Ministry of Rural Rehabilitation and Development (MRRD) wishes to acknowledge the support that was provided by the United Kingdom Department for International Development (DfID) for the preparation of the Pavement Design Guideline for Low Volume Rural Roads. The project was carried out under the aegis of the Research for Community Access Partnership (ReCAP) – a DFID-funded research program that promotes safe and sustainable access for rural communities in Africa and Asia.

The development of the Guideline was guided by a Technical Steering Committee comprising the following representatives from the MRRD.

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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 Guideline.

Pavement

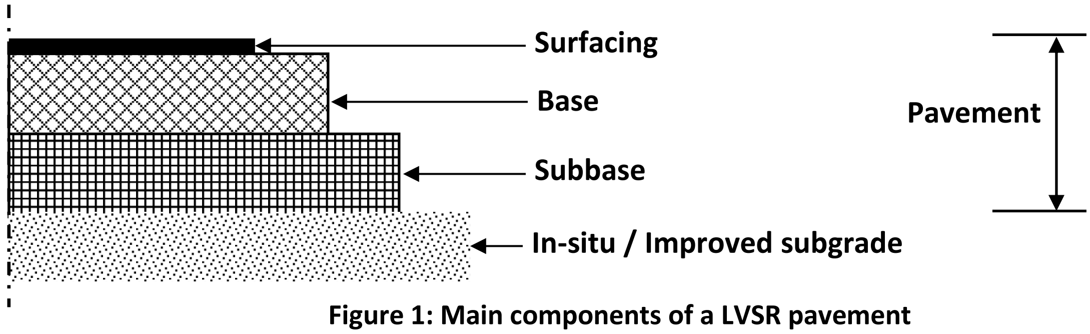


Figure 1: Main components of a LVSR pavement

Table 1: Purpose of main components of a LVRR 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 the 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.

Cross Section

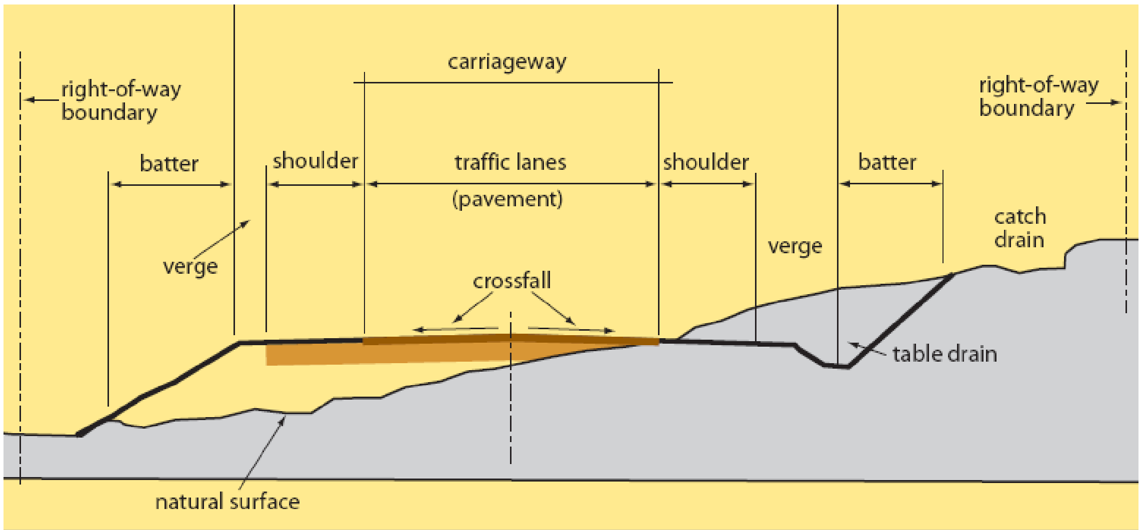


Figure 2: Cross section elements

Drainage Elements

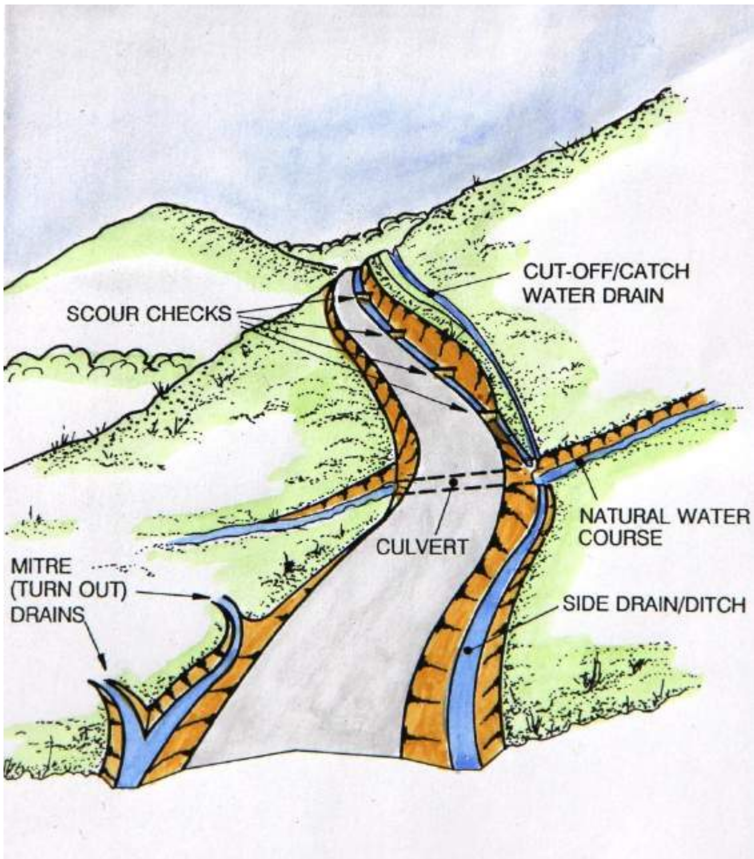


Figure 3: Main drainage elements

List of Abbreviations, Acronyms and Initialisms

AADT	Average Annual Daily Traffic
ASTM	American Society for Testing 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
GVC	Gross Vehicle Weight

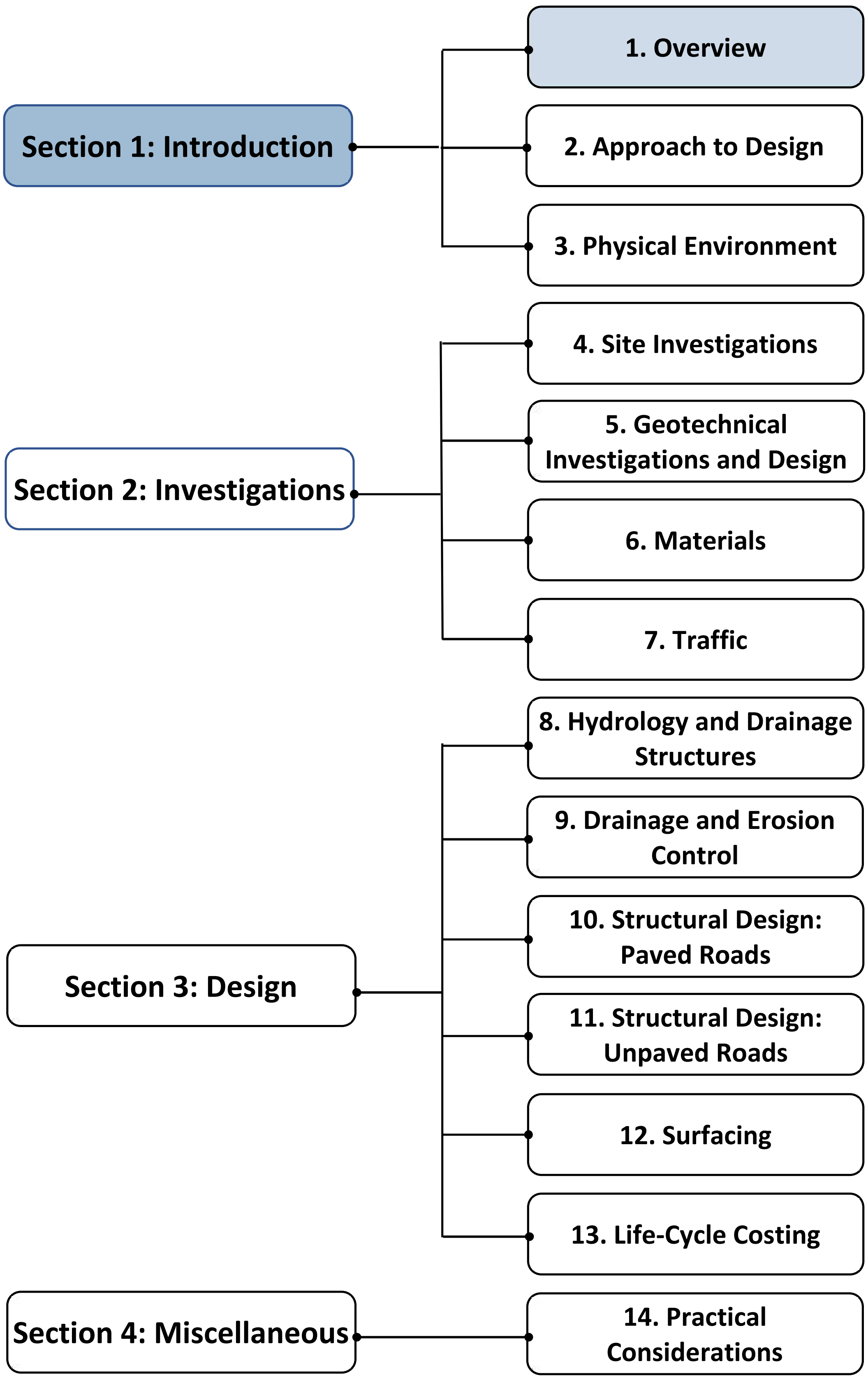
HDM-4	Highway Development and Management Model - 4
HGV	Heavy Goods Vehicle
HVR	High Volume Road
HPS	Hand Packed Stone
LAA	Los Angeles Abrasion Value
LBM	Labour Based Methods
LCC	Life Cycle Cost
LGV	Light Goods Vehicle
LL	Liquid Limit
LVR	Low Volume Road
LVSR	Low Volume Sealed Road
MAASHTO	Modified AASHTO
MC	Moisture Content
MC	Medium Curing
MDD	Maximum Dry Density
MESA	Million Equivalent Standard Axles
MGV	Medium Goods Vehicle
MRRD	Ministry of Rural Rehabilitation and Development
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	United Kingdom Aid for International. Development
URC	Unreinforced Concrete
VEF	Vehicle Equivalence Factor
VHGV	Very Heavy Goods Vehicle
VOC	Vehicle Operating Costs
vpd	Vehicles per day

Section 1

Introduction

Low Volume Rural Roads Guideline and Standards
Volume 1 – Pavement Design



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1.1 Background

As a landlocked mountainous country, Afghanistan depends mainly on road transport, which serves as the backbone of the development of the country. In this context, the Government of the Islamic Republic of Afghanistan faces a massive challenge in fulfilling one of its key developmental goals – reducing poverty in the rural areas of the country by facilitating improved access to economic and social services for rural communities.

Low volume rural roads (LVRRs) compromise approximately 85% of Afghanistan's classified road network, and are viewed by the Government as a key driver for improving rural well-being, economic development, and food security for the majority of Afghans (> 70%) who live in the rural areas of the country. The attainment of this goal depends critically on the existence of sound rural road infrastructure. It is therefore important that the Ministry of Rural Rehabilitation and Development (MRRD) adopts appropriate, economical design standards and practices that are tailored to the diverse physical environment of the country.

There are currently a number of technical guidelines used for the planning and design of LVRRs in Afghanistan. However, in a number of aspects, they do not reflect the latest developments in LVRR technology. This has led to a need to develop new Guidelines and Standards for LVRRs that are tailored to the needs of the country and take account of the many advances in LVRR technology that have taken place in recent times in the Asian region and internationally.

1.2 Purpose

The main purpose of the Guideline 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 LVRRs in Afghanistan.

The focus of the Guideline is on the provision of low-cost solutions for the design of LVRRs. Such roads typically have traffic levels below 400 vpd vehicles per day and include all those classified as Tertiary Roads and some of those classified as Secondary Roads.

Figure 1-1: Typical LVRR in Afghanistan

The design of the LVRRs is aimed at minimizing the life-cycle costs of their provision by taking account of the many locally prevailing road environment factors that impact on the design process. In so doing, the developmental goal is to:

- enhance socio-economic growth, development and poverty alleviation in rural areas;
- improve the connectivity of all villages to each other and to main roads and urban centers;
- improve access to basic services, such as markets, health care and education centers;
- provide reliable, lower-cost movement of people and goods from rural to urban areas;
- reduce the depletion of finite materials resources.

The Guideline 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 this research provide a wealth of performance-based information that has advanced previous knowledge on various aspects of LVRR technology. This has allowed state-of-the-art guidance to be provided in the Guideline which is expected to serve as a nationally recognized document, the application of which will harmonize approaches to the provision of LVRRs in Afghanistan. The Guideline is intended for use by all road agencies and organizations in the country's roads sector.

1.3 Scope

The Guideline caters for a range of road types, from basic earth tracks to bituminous sealed roads, that are typically found in Afghanistan. The environmentally optimized approach to the design of such roads is a key feature of the Guideline 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, the 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 LVRR provision.

1.4 Development

The development of the Guideline was overseen by a Technical Steering Committee (TSC) comprising seven members of the MRRD, covering a range of disciplines within the organization.

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

1.5 Structure

The Guideline is divided into four separate sections, each comprising chapters on various topics related to the design of LVRRs, as presented in Table 1.

Table 1-1: Structure and content of the Guideline

Section	Chapter
A. Introduction	1. Overview 2. Approach to Design 3. Physical Environment
B. Investigations	4. Site Investigations 5. Geotechnical Investigations and Design 6. Materials 7. Traffic
C. Design	8. Hydrology and Drainage Structures 9. Drainage and Erosion Control 10. Structural Design: Paved Roads 11. Structural Design: Unpaved Roads 12. Surfacing 13. Life-Cycle Costing
D. Miscellaneous	14. Practical Considerations
Appendix	Glossary of Terms

1.6 Benefits of Using the Guideline

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

- Are less expensive, in economic terms, to build and to maintain through the adoption of more appropriate LVRR technology and design/construction techniques that are better suited to local conditions.
- Minimize 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 labor-based methods where feasible.
- Incorporate road safety measures to minimize road accidents.
- Take a better account of the needs of all stakeholders, particularly the local communities served by such roads.
- Ultimately, facilitate the longer-term goal of socio-economic growth, development and poverty alleviation in Afghanistan.

1.7 Sources of Information

In addition to providing general information and guidance, the Guideline 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 Guideline. Where the sources of any tables or figures are not specifically indicated, they are attributed to the authors.

1.8 Updating of the Guideline

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

- Any proposed amendments should be sent to the Regional Director, MRRD, motivating the need for the change and indicating the proposed amendment.
- Any agreed changes to the Guideline will be approved by the Regional Director, MRRD, 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 Guideline. An example of a Departure from Standard could be the use of a material specification that may be outside the limits given in the Guideline. Where the designer departs from a standard, he/she must obtain written approval and authorization from the MRRD. The designer shall submit the following information to the Regional Director, MRRD:

- 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 given approval by the Regional Director, MRRD.

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  - 2. Approach to Design
  - 3. Physical Environment
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  - 7. Traffic
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  - 8. Hydrology and Drainage Structures
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  - 10. Structural Design: Paved Roads
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  - 12. Surfacing
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2.1 Introduction

2.1.1 Background

The traditional approaches to the provision of low volume rural roads (LVRRs) in many countries 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 Afghanistan’s rural road network. Thus, in facing the challenges of improving and expanding the country’s LVRR network, more appropriate approaches need to be considered.

The approach to the design of LVRRs 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 to provide designs that will meet with the multiple social, economic and environmental requirements of Afghanistan 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 LVRRs. To this end, the chapter addresses the following topics:

- The particular characteristics of LVRRs.
- The LVRR design philosophy.
- Various implementation considerations.

2.2 Definition of Low Volume Rural Roads

For pavement design purposes LVRRs are defined as those roads that have a base year average annual daily traffic (AADT) of up to about 300 motorized, 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. However, this figure does not provide a complete picture of the unique characteristics of LVRRs in that there are many other aspects that need to be considered in their design, as discussed below.

2.3 Road Classification System

Afghanistan’s road network is classified on the basis of function and connectivity. The type of LVRRs being catered for in this Guideline comprise the following:

1. **Secondary roads:** Only those secondary roads that comply with the definition of a LVRR as defined above and include:
   - Roads that link:
     - Provincial capital to District capital
     - District capital to National or State Highways
     - District capital to District capital
2. **Tertiary roads:** All roads that are classified as Tertiary roads and include:
   - Roads that link:
     - Village to Provincial capital
     - Village to Main Road
     - Village to District capital
     - Village to village

Chapter 2 – Approach to Design

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

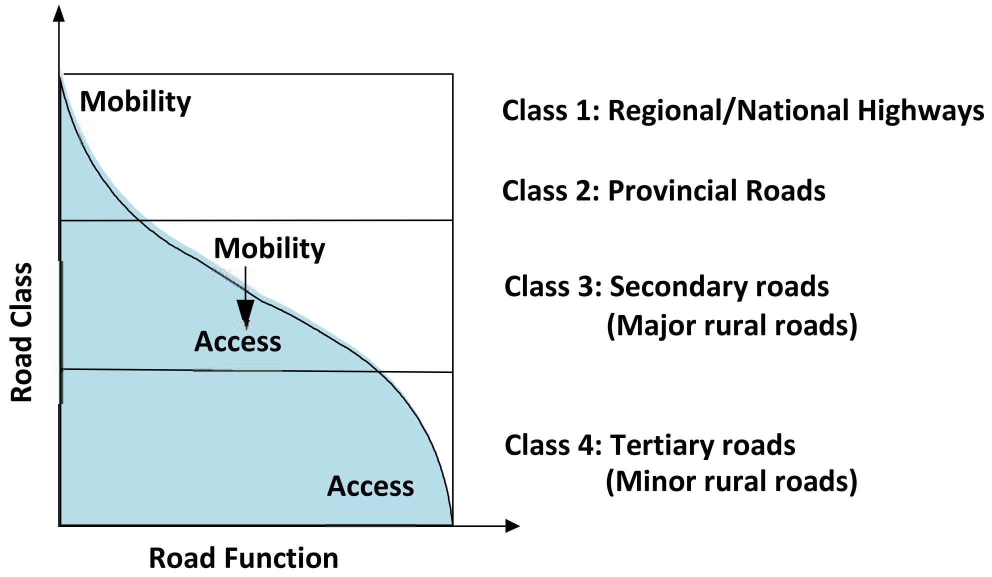

Figure 2-1: Relationship between Road Class and Road Function

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 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 a division of administrative responsibilities, which may be based on other criteria.

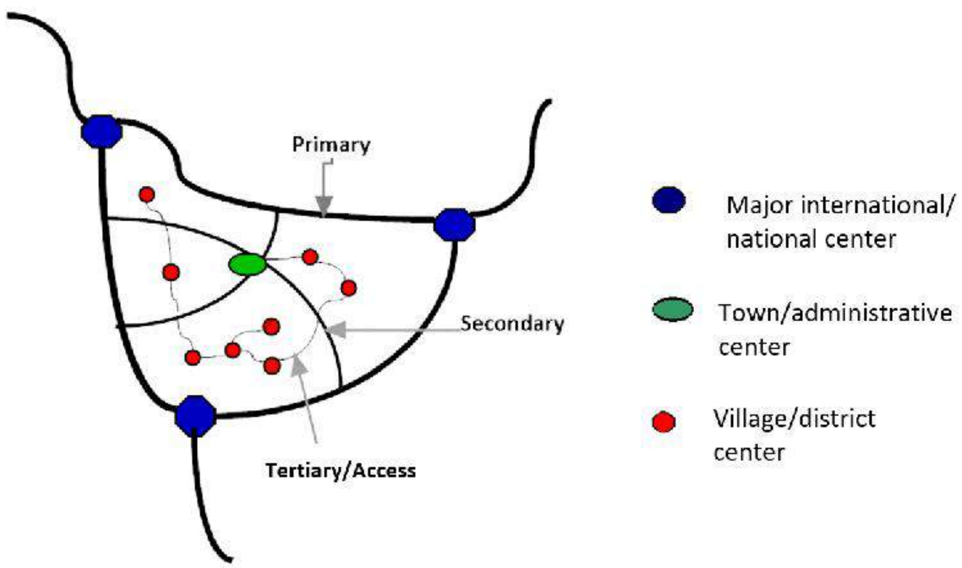

Figure 2-2: Road hierarchy and functions

Based on the specific function and typical characteristics of LVRRs described above, it would be apparent that roads that do not generally fulfill these attributes would not fall under the heading of LVRRs. 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 LVRR. This is because the level of serviceability that it would be expected to provide would be dictated by its function as characterized by a relatively high design speed and corresponding geometric design requirements, which would be economically unjustifiable to apply to a LVRR. Nonetheless, its pavement could be designed in accordance with the LVRR design philosophy presented in this Guideline.

2.4 Characteristics of Low Volume Rural Roads

The following specific characteristics of LVRRs 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 special 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 traveling 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 LVRRs 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 LVRRs, 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 Guideline.

2.5 Design Considerations

2.5.1 General

Whilst the approach to the design of LVRRs follows the general principles of any good road design practice, the level of attention and engineering judgment 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:

- 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 LVRR pavement, it is 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 that apply.

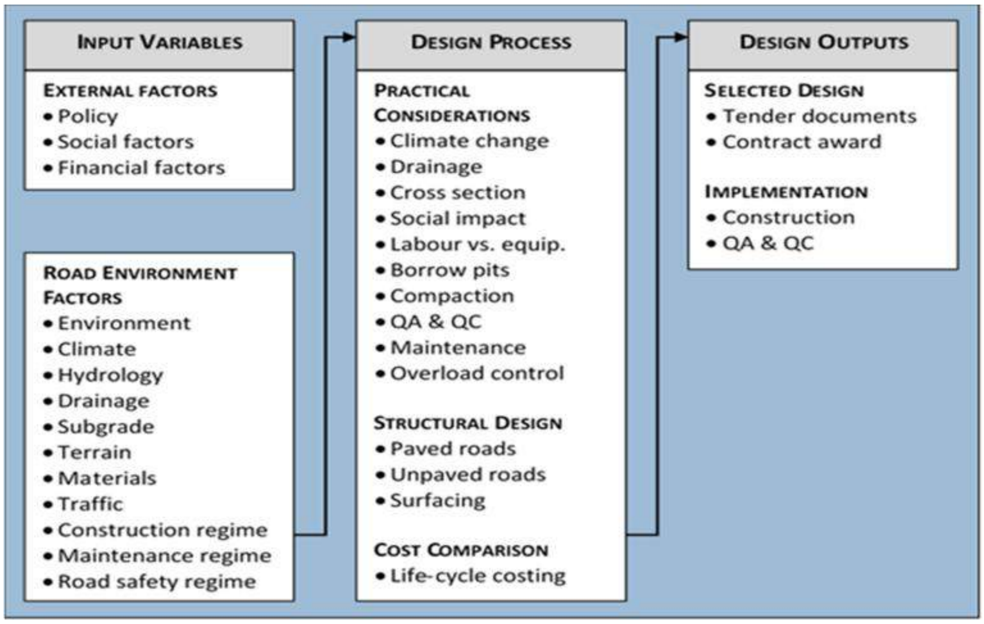

Figure 2-3: Pavement design system

2.5.2 Input Variables

Road environment factors

The term "road environment" is all-encompassing and includes both the natural or biophysical 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 negative 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 of 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.

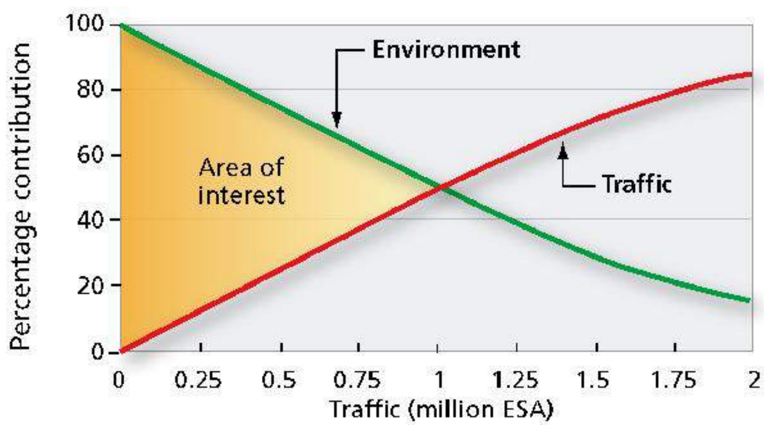

Figure 2-4: Traffic loading versus dominant mechanism of pavement distress (schematic only)

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 LVRR design. This includes, for example, environmental controls, road safety legislation and promotion, where appropriate, of the use of labor-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 and construction issues, as discussed below.

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 labor-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 labor rather than plant.
- The occurrence of problem soils, such as expansive, collapsible, or salt-laden soils, all of which require specialized design solutions, as discussed in *Chapter 6: Materials.*
- The constructability of the pavement layers in relation, for example, to the use of stabilized or unstabilized 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 non-bituminous surfacings, such as concrete or block paving, on steep grades.
    - 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 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 certain 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 issues 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 that 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 LVRRs, including the DCP-DN and DCP-CBR methods, as discussed in *Chapter 10: Structural Design: Paved Roads* and *Chapter 11: Structural Design: Unpaved Roads.* The design catalogs 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 catalog 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 the 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 that influence maintenance, safety, and road users. These constraints may need to override cost considerations and are discussed in *Chapter 13 – 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 a number of 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 14 – 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

To obtain optimal results from investments in road infrastructure in Afghanistan, it is important 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 LVRRs 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:

- **Task-based:** LVRRs 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.

- **Environmentally compatible:** Suitable for, and where necessary, adapted to the local road environment factors.
- **Local resource-based:** The design of the LVRR 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: LVRR implementation within an EOD context

EOD is a strategy for utilizing 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. To be successful and sustainable, LVRR technology needs to be implemented within the framework of an EOD strategy. Moreover, if the LVRR 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.
- The maximum use of local labor and skills.
- The maximum use of materials that are locally available or those that can be processed locally.
- In-built maintenance considerations in the design, such as the provision of adequate drainage, resistance to soil erosion along the side slopes, adequate lateral support from shoulders, etc. 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.

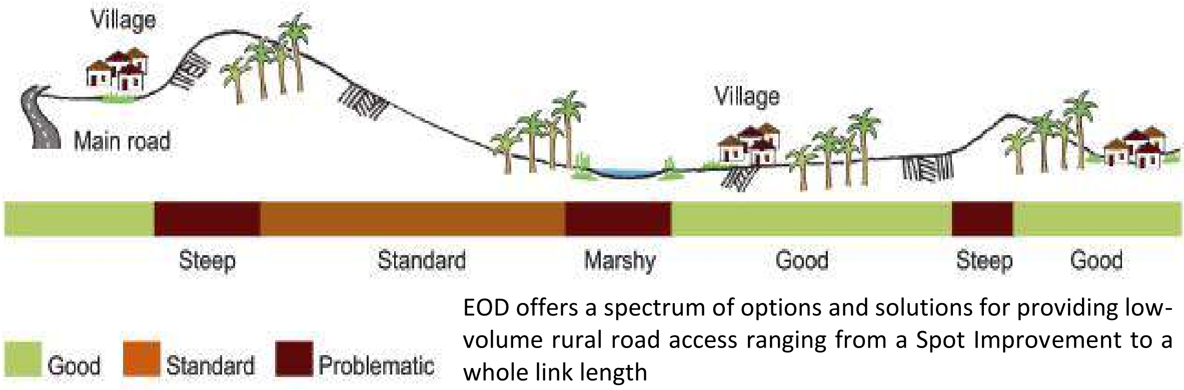

Figure 2-6: Environmentally optimized and spot improvement approach

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 LVRR. 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 LVRRs. These aspects of the LVRR design philosophy are addressed in *Chapter 6 – Materials* and *Chapter 10 – 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 LVRRs. 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 12 – 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 LVRR 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 LVRRs. A life-cycle cost analysis, as discussed in *Chapter 13 – Life-Cycle Costing*, should be undertaken to determine when to upgrade from one standard to a higher one.

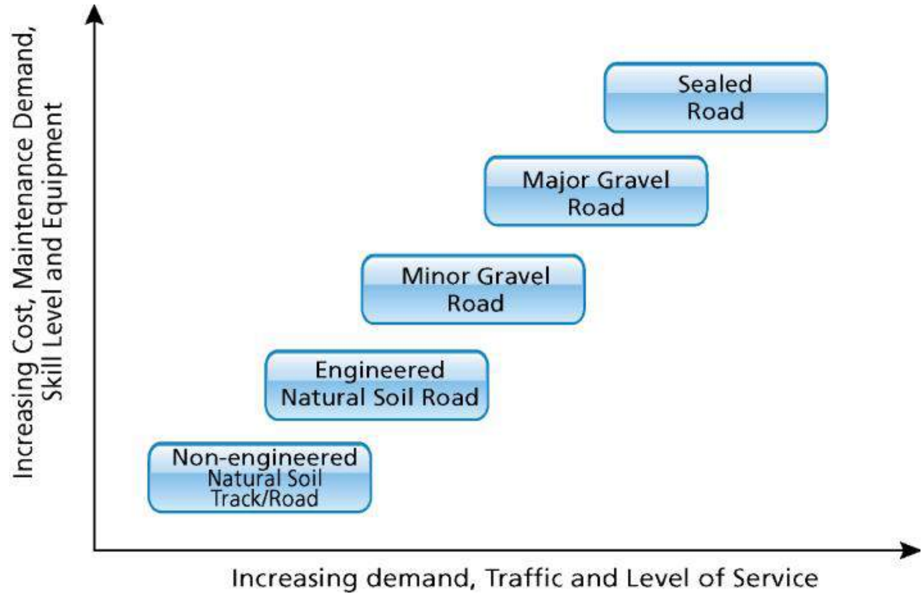

Figure 2-7: Upgrading stages of low volume rural roads

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 LVRR. However, such a risk should be a calculated one 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 LVRRs 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 Guideline, 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 14 – Practical Considerations.*

2.6 Project Implementation

2.6.1 Level of Influence of Key Activities

The four major phases of LVRR provision are as follows:

- Planning
- Design
- Construction
- Maintenance

The above phases 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 form, 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.

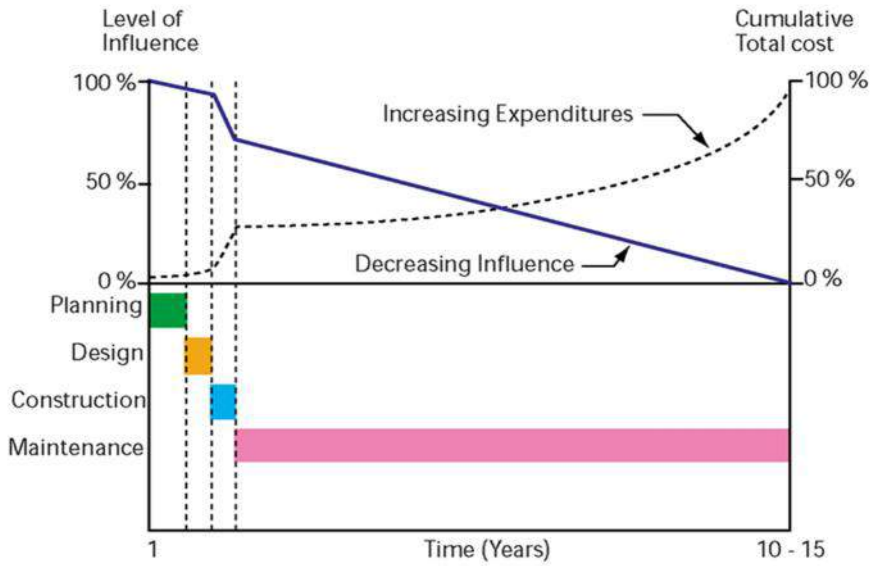

Figure 2-8: Level of influence of activity in relation to life-cycle cost of LVRR provision

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 % influence) in determining future expenditures. The key challenge is how to ensure that the key components of LVRR provision are optimized so as to minimize the life-cycle cost of LVRR 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 LVRR'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 greatly 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 LVRR 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.

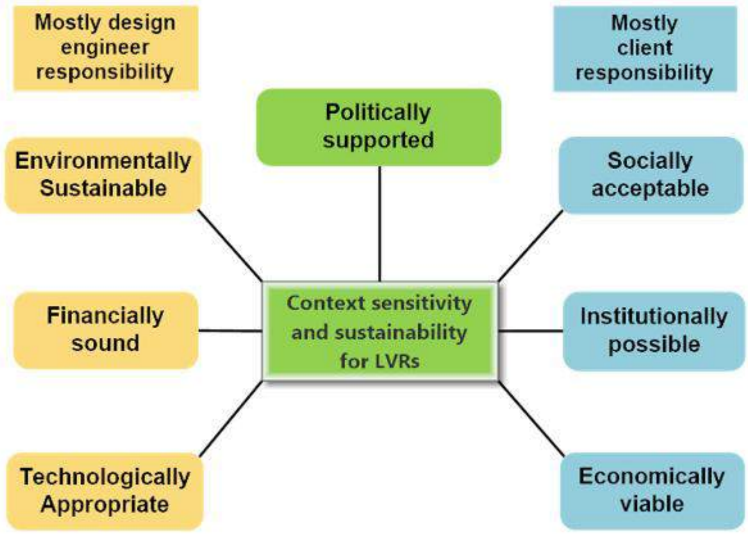

Figure 2-9: Framework for sustainable provision of LVRRs

2.7.2 Strategy for Ensuring Sustainability

In terms of ensuring sustainability in LVRR 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 LVRR provision and its role in underpinning other sectoral development strategies and poverty alleviation programs 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 LVRR 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 emphasize the provision of LVRRs 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 Rural Roads Guideline and Standards
Volume 1 – Pavement Design

- **Section 1: Introduction**
  - 1. Overview
  - 2. Approach to Design
  - 3. Physical Environment
- **Section 2: Investigations**
  - 4. Site Investigations
  - 5. Geotechnical Investigations and Design
  - 6. Materials
  - 7. Traffic
- **Section 3: Design**
  - 8. Hydrology and Drainage Structures
  - 9. Drainage and Erosion Control
  - 10. Structural Design: Paved Roads
  - 11. Structural Design: Unpaved Roads
  - 12. Surfacing
  - 13. Life-Cycle Costing
- **Section 4: Miscellaneous**
  - 14. 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 Low Volume Rural Roads (LVRR). Thus, for a designer, it is essential to have a comprehensive understanding of the various factors that make up the physical environment to cater to them appropriately in the design of the LVRR. 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 LVRR. The chapter discusses both the physical features and climate of Afghanistan 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

Afghanistan has a highly variable topography as a result of its location at the "corner" of the Eurasian, Indian and Arabian plates. The Hindu Kush mountains, which run from the northeast to the southwest, generally dominate the topography, with almost 75% of the country being classified as mountainous. The country can, however, be divided into eleven different zones, each with distinctive geology and geomorphological features as summarized below and illustrated in Figure 3-1:

(1) *Wāḵān corridor and Pamir knot*: More than three-quarters of the area is above 3,000 m in altitude, and numerous peaks rise above 6,000 m. Snowfields and glaciers are common in the area. Several wide, flat valleys between the mountains provide limited access and sites for human habitation.

(2) *Badaḵšān*: This high rugged portion of north-eastern Afghanistan is characterized by peaks, gorges, and alpine landscapes. The highest peaks are over 6,000 m, but most are around 4,000-5,000 m. Glaciers occur in both the north and south and lakes, mostly of glacial origin, abound.

(3) *Central mountains*: The main axis of the Hindu Kush spreads out in the center of the country in a broad, fan-shaped arrangement from northeast to southwest. Numerous near-parallel valleys also fan outwards from this massif. The highest peaks range between 4,000 and 5,000 m.

(4) *Eastern mountains*: These mountains are a topographically complex area of high-altitude peaks (about 6,000 m) and large valleys. Four main valley systems occur in this region which contains thick, late Cenozoic sedimentary valley-fill sequences. The Rīg-e Ravān is an anomalous area of sand dunes south of Čārīkār.

(5) *Southern mountains and foothills*: Many low (about 2,500-3,000 m), northeast-southwest trending mountain ranges occur here. Extensive valley fills and broad alluvial plains extend between the ranges, and ephemeral stream channels are common. Some alluvial fans and basins of interior drainage occur, together with a few minor areas of sand dunes.

(6) *Northern mountains and foothills*: This is a broad zone of mountain plateaus and foothills, with some peaks over 3,000 m.

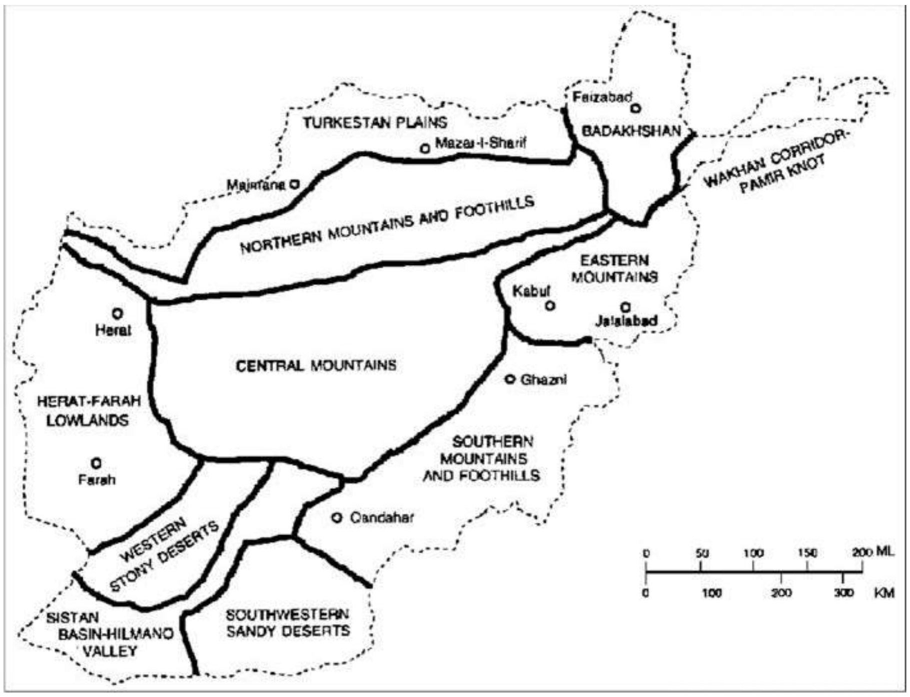
(7) *Turkestan plains*: The northern foothills decrease in altitude and pass into stony plains of 300-400 m. Sand drifts and dunes abound; loess deposits (wind-blown dust) and salt pans occur. This dry desert area is commonly separated by marshy, alluvial terraces from the level floodplain of the Āmū Daryā river, which forms the northern border.

(8) *Herat-Farāh lowlands*: This area is a relatively low-lying complex of broad arid alluvial plains, playa basins, and low hills and mountain ranges. Numerous alluvial fans and dry desert washes occur. The general elevation is about 1,000 m, and the regional slope is to the west and southwest.

(9) *Helmand valley-Sīstān basin*: The Helmand river system, which rises in the section of the central mountains, passes through the center of this area and ends as a series of marshes and connecting lakes. In exceptionally wet years, it empties into the Gowd-e Zereh, which is an ephemeral brackish lake. The area as a whole is an alluvial plain of about 500-600 m altitude and is characterized by surrounding sandy and rocky desert.

(10) *Western stony deserts*: These are waterless, barren, alluvial wastes north of the great arc of the Helmand as it swings from south to north. The Dašt-e Ḵaš and Dašt-e Mārgō deserts are characterized by a desert pavement of stones left where finer sediments were blown away by the wind. The altitudes average about 700 m.

(11) *Southwestern sandy deserts*: This area (Rēgestān, Dašt-e Pōḡdar, Dašt-e Arbū) is similar to the above but has many more fixed and mobile sand dunes with some moist, sandy, clay, interdune areas similar to playas.

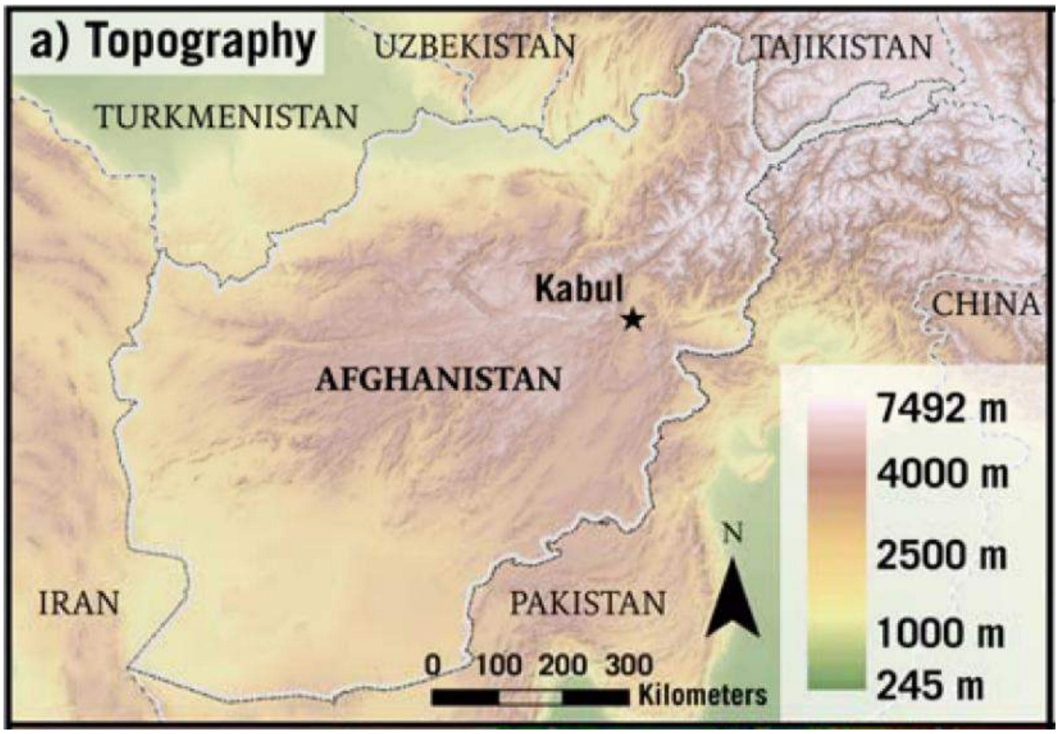


Source: Dupree, 1980. Available from https://bawar.net/data0/books/59f4e009e3dc1/pdf/Afghanistan%20by%20Dupree%20s.pdf

Figure 3-1: Topographic regions of Afghanistan

Detailed topographic mapping is available for Afghanistan at a scale of 1:250 000, downloadable from https://afghanistan.cr.usgs.gov/afghan_geol with a contour interval of 100 m.

A simplified topographic map of Afghanistan is shown in Figure 3-2, which shows the typical altitudes in the various regions of the country.



Source: Aich et al, 2017. Available from: https://www.mdpi.com/2225-1154/5/2/38/htm#

Figure 3-2: Simplified topographic map of Afghanistan

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

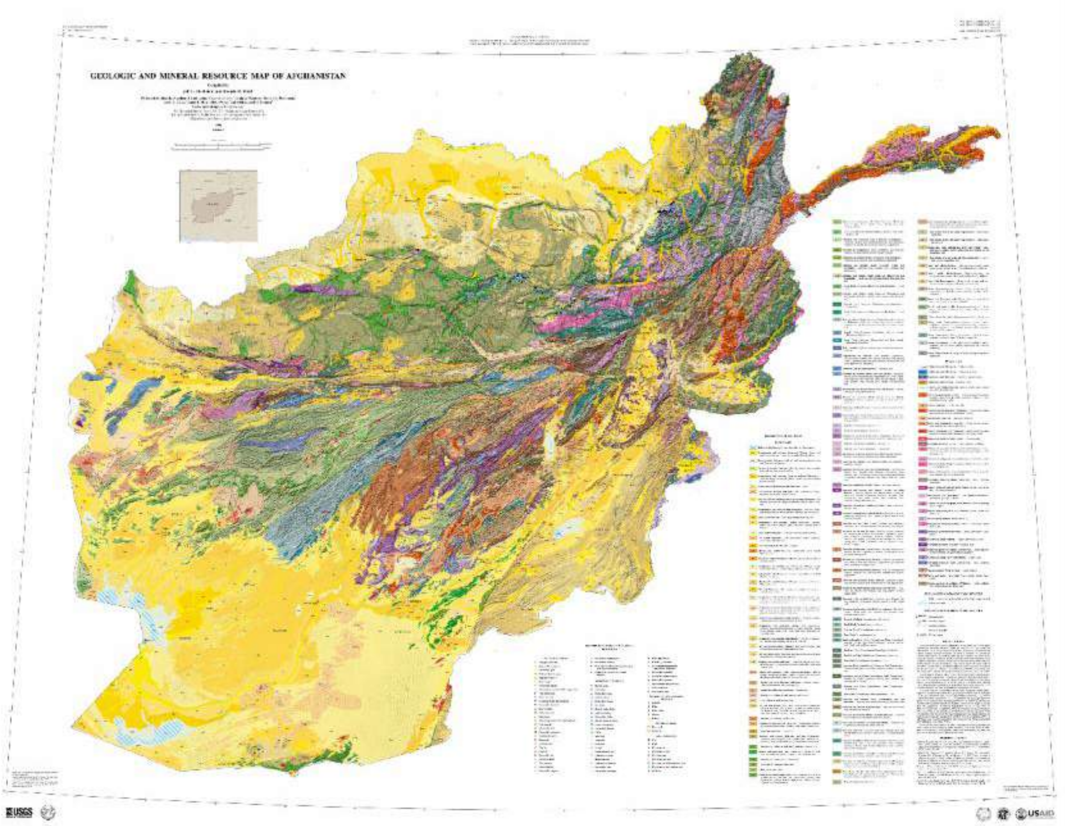
- geometric design in terms of horizontal, vertical alignment and road cross section;
- drainage and anti-erosion measures;
- traffic safety measures; and
- choice of road surfacing.

3.2.3 Geology

Afghanistan has varied and complex geology evolved through the collision of the Eurasian, Indian and Arabian plates. Although none of the plate boundaries actually occur in Afghanistan, they are close enough to have had a major influence on the topography. The Hindu Kush is essentially the western portion of the Himalayas and formed as part of their development during the collision of the Eurasian and Arabian plates. It remains geologically active and is still rising and is prone to earthquakes. This is mostly surrounded by sediments derived from the mountains overlying metamorphic rocks (gneiss, schists, phyllites) ranging from more than a billion years old. Widespread marine transgressions and deposition occurred during the Paleozoic (250 -540 million years ago) and Mesozoic (60 – 250 million years ago), which continued into the Cenozoic (last 60 million years) with the uplift of the Hindu Kush mountains. These resulted in the placement of thick deposits of sedimentary rocks such as limestones, dolomites, sandstones, shales and quartzites. A wide range of geological materials is thus present in Afghanistan, as shown in the geological map in Figure 3-3 below. High-resolution copies of this map can be downloaded from https://pubs.usgs.gov/of/2006/1038/.

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. However, most of Afghanistan is covered by transported soils from the mountains, and the underlying geology thus plays only a minor part in the overlying materials. Hard rock aggregate can, therefore, only be expected in mountainous areas and in areas where the transported soils are thin.

Geological maps, with as large a scale as possible, should be obtained for the relevant project area as a basis for planning site investigations and materials prospecting.



Source: Aich et al, 2017. Available from: https://www.mdpi.com/2225-1154/5/2/38/htm#

Figure 3-3: Geological Map of Afghanistan

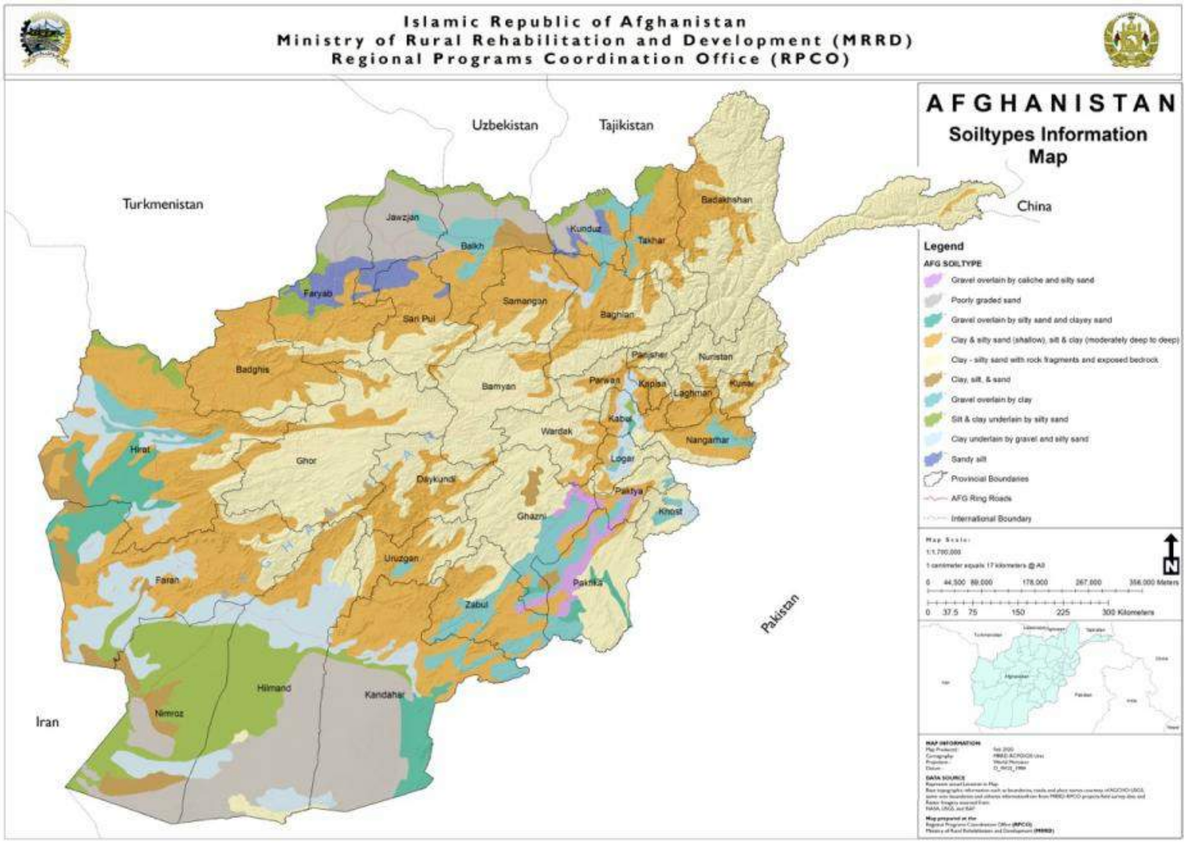
3.2.4 Soils

As for the geology of Afghanistan, the soil varieties are also wide and complex. A study of the Genesis of Afghanistan soils indicates that these are developed predominantly from the influence of climate. Since moisture is low, physical weathering is more pronounced than chemical weathering and soil formation has proceeded rather slowly. Most of the products of weathering are retained within the soil itself.

Calcification is the most dominant soil-forming process in dry conditions. In areas of low rainfall, there is a tendency towards evaporation from the surface and replacement from the water table below. The groundwater, which is drawn up, often contains large amounts of dissolved calcium bicarbonate and, on evaporation, the calcium carbonate is deposited within the soil-body resulting in an accumulation of this substance. Leaching of the soluble materials from the surface downwards is minimal because of the absence of percolating water.

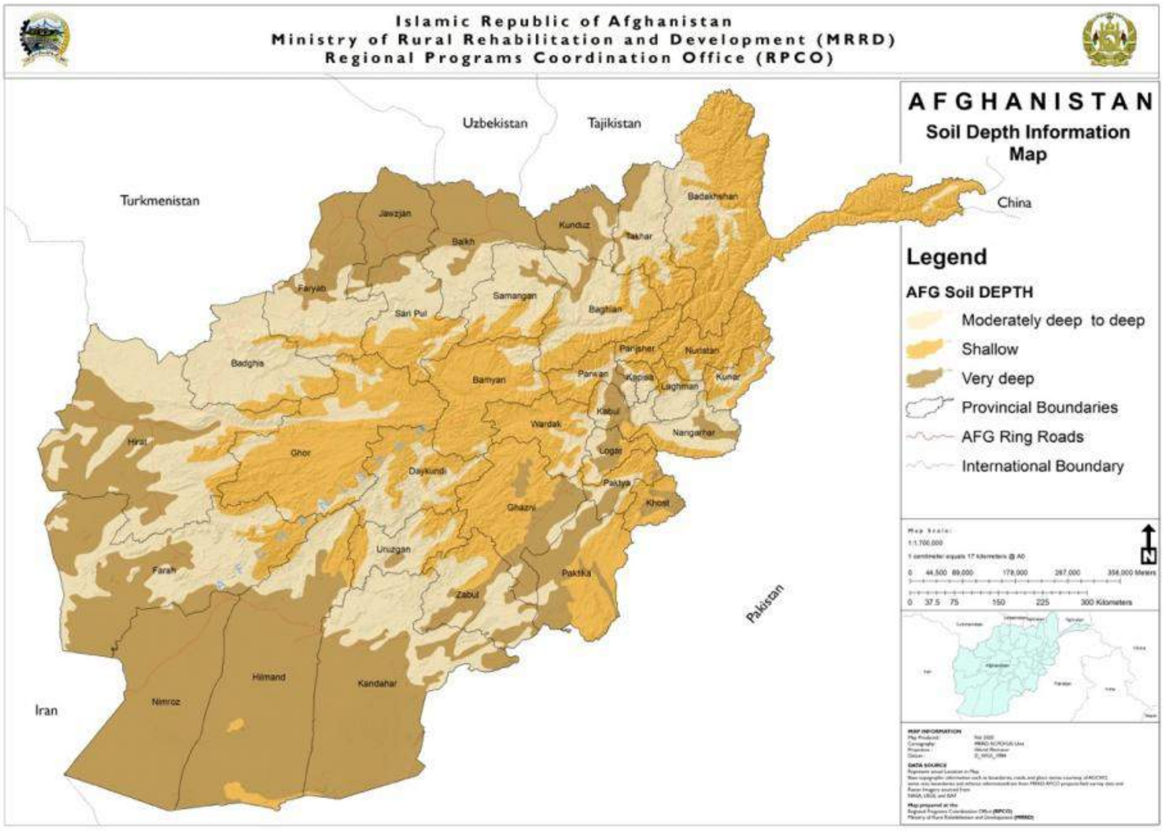
The soils of Afghanistan are alkaline in reaction. Most of the samples studied have a pH much higher than 7.0. Nearly 50% of the soils of Afghanistan are estimated to have a pH between 8 and 8.5, about 35% between 8.5 and 9.0 and about 10% between 9 and 9.5. Soils with a pH of 8.0 to 8.5 are also known to be generally rich in alkaline earth carbonates, except for the soils of Panjshir Ghorband Valley. All the soils studied so far have a calcium carbonate content of 10% or more; soils of the Ghazani area have in general 10 to 12% calcium carbonate, Kunduz soils show 10 to 15% Kabul and Katawaz 10 to 20%, Hari-Rud 15 to 20%, and Farah, Logar and Adras Kand areas show 20 to 25% calcium carbonate in the surface samples. Occasionally in some areas $CaCO_3$, content is more than 40%. The calcium carbonate is mostly in the form of “calcite” and includes calcareous soils, nodules and hardpan covered by the blanket term “calcareous soils.” Conventionally, a material with more than 50% carbonate would be classified as a calcrete (South Africa), caliche (USA), or kunkar (India).

Figure 3-4 shows a soil map of Afghanistan, while Figures 3-5 shows typical soil depths.



Source: MRRD Regional Programs Coordination Office (RPCO)

Figure 3-4: Simplified Soil Map of Afghanistan



Source: MRRD Regional Programs Coordination Office (RPCO)

Figure 3-5: Soil Depth Map of Afghanistan

3.2.5 Hydrology

The hydrological systems in Afghanistan are all related to the Hindu Kush mountains. Although the annual rainfall is generally low in Afghanistan (< 300 mm), most rivers are fed well from the winter snows and glaciers in the mountains.

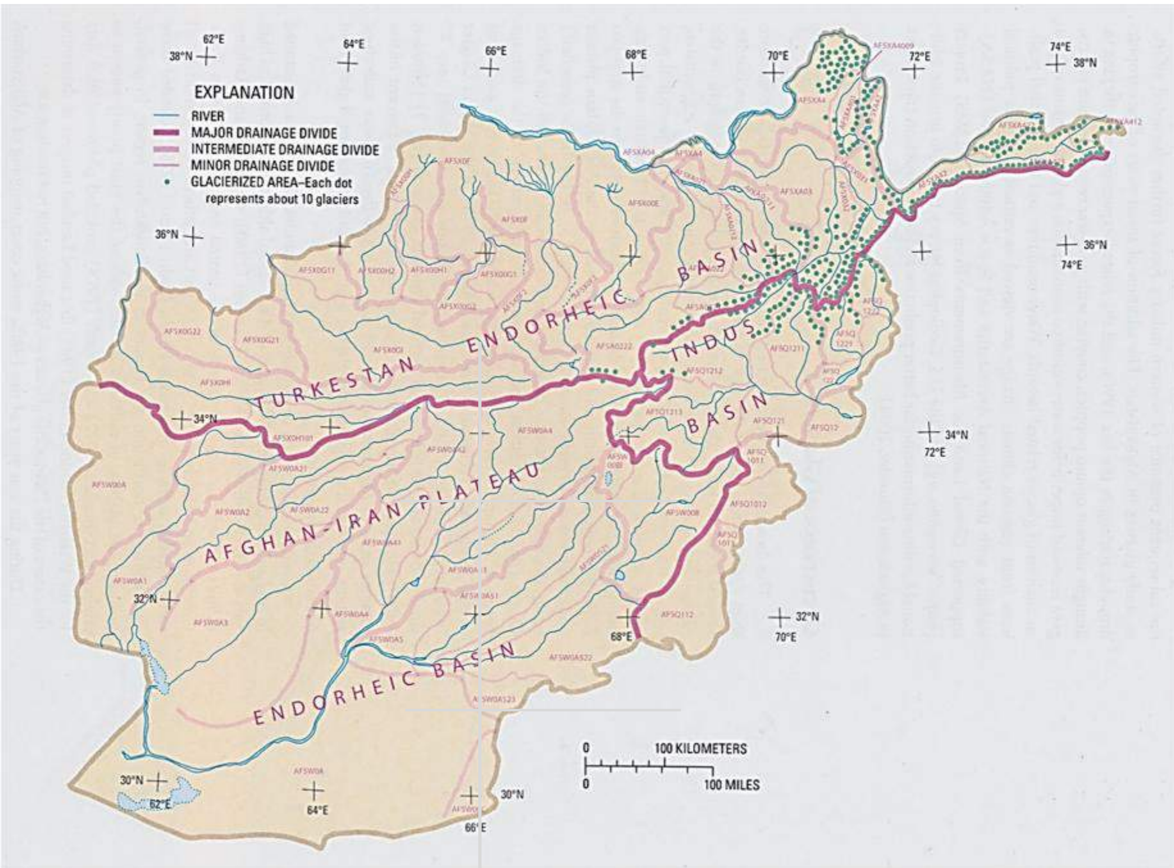
The rivers of Afghanistan generally reflect three major drainage controls:

- The north and north-westward flow into the Central Asian depressions of Turkmenistan, Tajikistan and Uzbekistan;
- The strong west and south-westward flow, largely structurally controlled, into other basins, particularly the Sīstān depression; and
- The south-eastward flow into the Indus system.

Most of the drainage rises in the higher and wetter eastern half of the country. Many of the rivers have steep gradients and run through slender and commonly precipitous valleys in the higher elevations. They carry large silt loads, particularly during peak runoff in the spring and early summer storm-and-melt season, when many disastrous floods occur. About ten river systems or major subsystems exist, although it is difficult to group many of these watercourses, which disappear into deserts or swampy areas.

Few large lakes exist in Afghanistan because of the general aridity and lack of suitable depressions. However, those that do occur tend to have high salinities because of their inland nature, and as they are the end-points of the internal drainage courses. Many small glacial lakes do occur, however, high in the mountains of the northeast, and these have better quality water.

The main river systems and catchment areas of Afghanistan are shown in Figure 3-6.



Source: University of Nebraska Omaha, undated. Available from https://www.unomaha.edu/international-studies-and-programs/center-for-afghanistan-studies/academics/transboundary-water-research/DLM3/Figure%203.1C-.jpg

Figure 3-6: Map of main river courses and catchments in Afghanistan

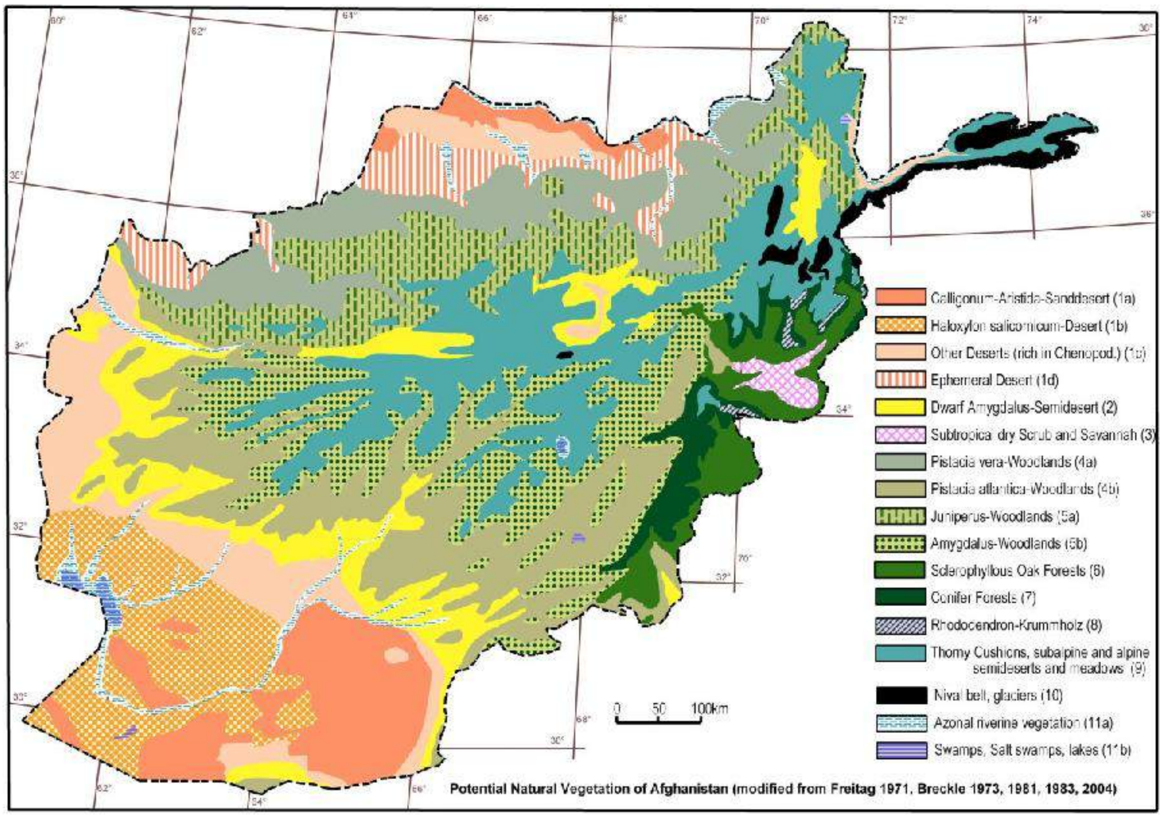
3.2.6 Vegetation

The wide variation in physiography and climate has given rise to a large variety of vegetation types in Afghanistan. Human beings have severely influenced vegetation all over Afghanistan, and only a few high mountains and very dry desert areas retain a quasi-natural vegetation cover. In most parts, the vegetation depends on the winter rain, which is often irregular. Rainfall increases to the north and east, resulting in better vegetation conditions in these parts. The eastern parts also receive some monsoon rains in summer.

There is no single dominant vegetation type, and the vegetation has been classified as follows:

1. Calligonum-Aristida-Sanddesert (1a) Haloxylon salicornicum-Desert (1b) Other Deserts (rich in Chenopod.) (1c) Ephemeral Desert (1d)
2. Dwarf Amygdalus-Semidesert (2)
3. Subtropical dry Scrub and Savannah (3)
4. Pistacia vera-Woodlands (4a)
5. Pistacia atlantica-Woodlands (4b)
6. Juniperus-Woodlands (5a)
7. Amygdalus-Woodlands (5b)
8. Sclerophyllous Oak Forests (6)
9. Conifer Forests (7)
10. Rhododendron-Krummholz (8)
11. Thorny Cushions, subalpine and alpine semideserts and meadows (9)
12. Nival belt, glaciers (10)
13. Azonal riverine vegetation (11a)
14. Swamps, Salt swamps, lakes (11b)

These vegetation zones of Afghanistan are shown in Figure 3-7.



Source: Breckle, 2007. Available from https://pdfs.semanticscholar.org/bcdb/3f33ba37c494a03436251cfb276eae294ee9.pdf

Figure 3-7: Map of vegetation types and distribution in Afghanistan

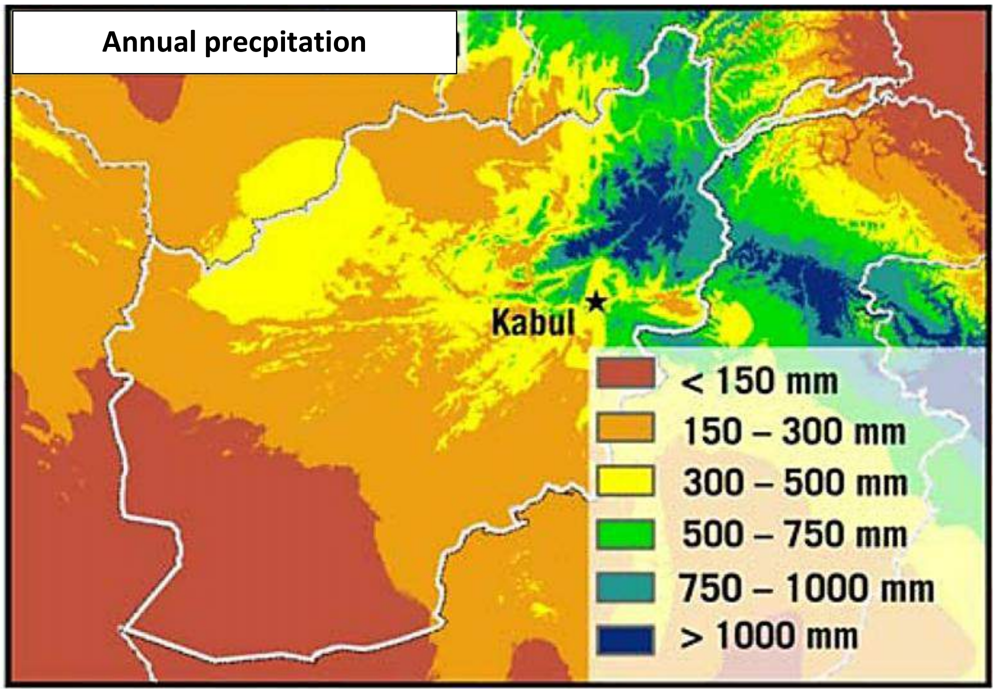
3.3 Climate

3.3.1 General

Afghanistan has a continental climate influenced by the country's landlocked and mountainous nature. Generally, the climate is arid with low rainfall and particularly harsh temperatures in the mountainous regions in winter.

3.3.2 Rainfall

The rainfall in Afghanistan varies considerably across the country. The wet season is mostly in winter and spring (February to April) with an annual average rainfall of about 75 mm in Farah to 1 170 mm in south Salang. At higher elevations, precipitation falls in the form of snow that ensures river flows in summer. From June to October, Afghanistan receives hardly any precipitation. The average annual rainfall is shown in Figure 3-9.



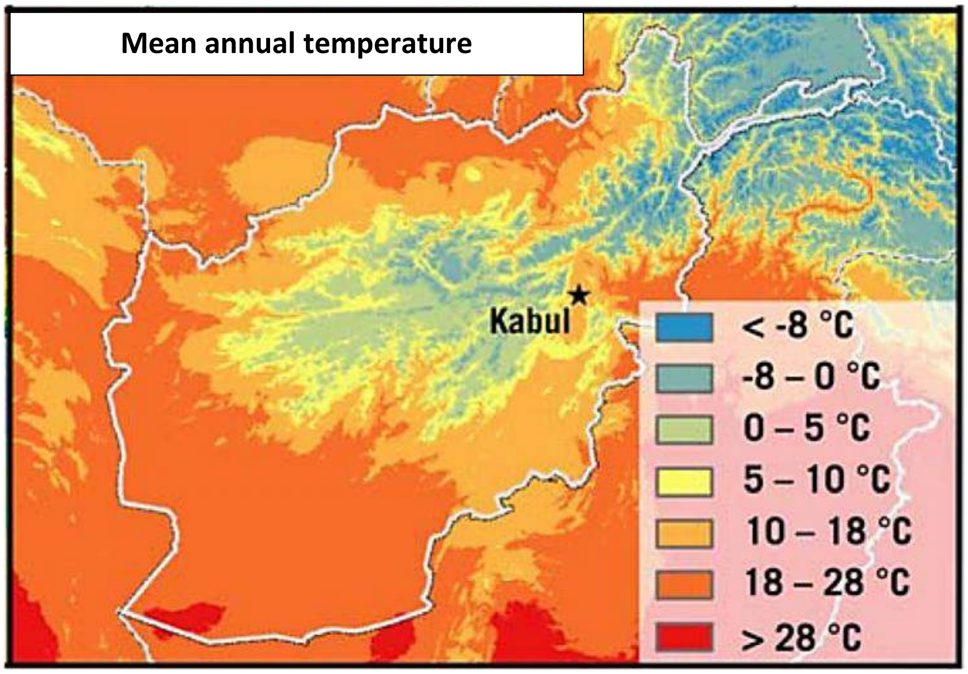
Source: Aich et al, 2017. Available from: https://www.mdpi.com/2225-1154/5/2/38/htm#

Figure 3-8: Mean annual rainfall in Afghanistan

3.3.3 Temperatures

Temperatures in Afghanistan range from above 30°C in summer to below -20°C in winter. The mean annual temperatures are shown in Figure 3-10.

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



Source: Aich et al, 2017. Available from: https://www.mdpi.com/2225-1154/5/2/38/htm#

Figure 3-9: Mean annual temperatures in Afghanistan

3.3.4 Climatic zones

As can be seen from Figure 3-9, about 80% of Afghanistan is in the dry climatic zone with a mean annual rainfall below 500 mm, as shown in Table 3-1. Only the mountainous region in the north-eastern corner of the country is in the Moderate to Wet climatic zone with the mean annual rainfall above 500 mm.

Knowledge of the climatic zone for the project area is important for the pavement design, as discussed in *Chapter 10 – Structural Design: Paved Roads*.

Table 3-1: Guideline for selection of climatic zone

Description		Weinert N value	Thornthwaite 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

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

3.3.5 Climate Change

The change in the climate of Afghanistan during the 21st century is predicted to show significant temperature changes, but minimal rainfall changes based on the World Bank Knowledge Portal projections.

Temperature

The average annual temperature is projected to increase between 1.4° C and 4.0° C by the 2060s and between 2.0° C and 6.2° C by the 2090s. Spring and summer are projected to experience the fastest rate of warming under these projections with uniform warming over the country's regions.

The frequency of 'hot' days and nights per year is projected to increase throughout the middle and late 21st century. 'Hot' days are projected to increase and occur on 14-25% of days by the 2060s and 16-32% of days by the 2090s, while 'hot' nights are projected to increase and occur on 16-26% of nights by the 2060s and 19-36% of nights by the 2090s. Both 'hot' days and nights are projected to increase most rapidly in the summer months of June-August.

‘Cold’ days and nights are projected to decrease in frequency and become exceedingly uncommon, with projections for the 2090s indicating that they will occur on 0-6% of days per. ‘Heatwave duration’ is projected to increase over Afghanistan throughout the middle and late 21st century under a range of expected emission scenarios.

‘Hot’ days or nights are defined as the temperature exceeded on 10 percent of days or nights in the current climate of that region and season per year. ‘Cold’ nights are defined as the temperature below which 10 percent of days or nights are recorded in the current climate of that region or season per year.

The eastern regions of Afghanistan are projected to see the largest change in heatwave duration.

The annual number of ‘frost’ days is projected to decrease under high, medium, and low emissions scenarios for the middle and late 21st century. Mountainous areas of eastern and north-eastern Afghanistan are projected to see the largest decreases in ‘frost’ days.

Precipitation

Annual precipitation projections from the Fifth Assessment Report of the Intergovernmental Panel on Climate Change indicate that there will be little or no change in precipitation over Afghanistan throughout the 21st century.

Projections for maximum 1- and 5-day rainfall indicate small increases in every season but March through May.

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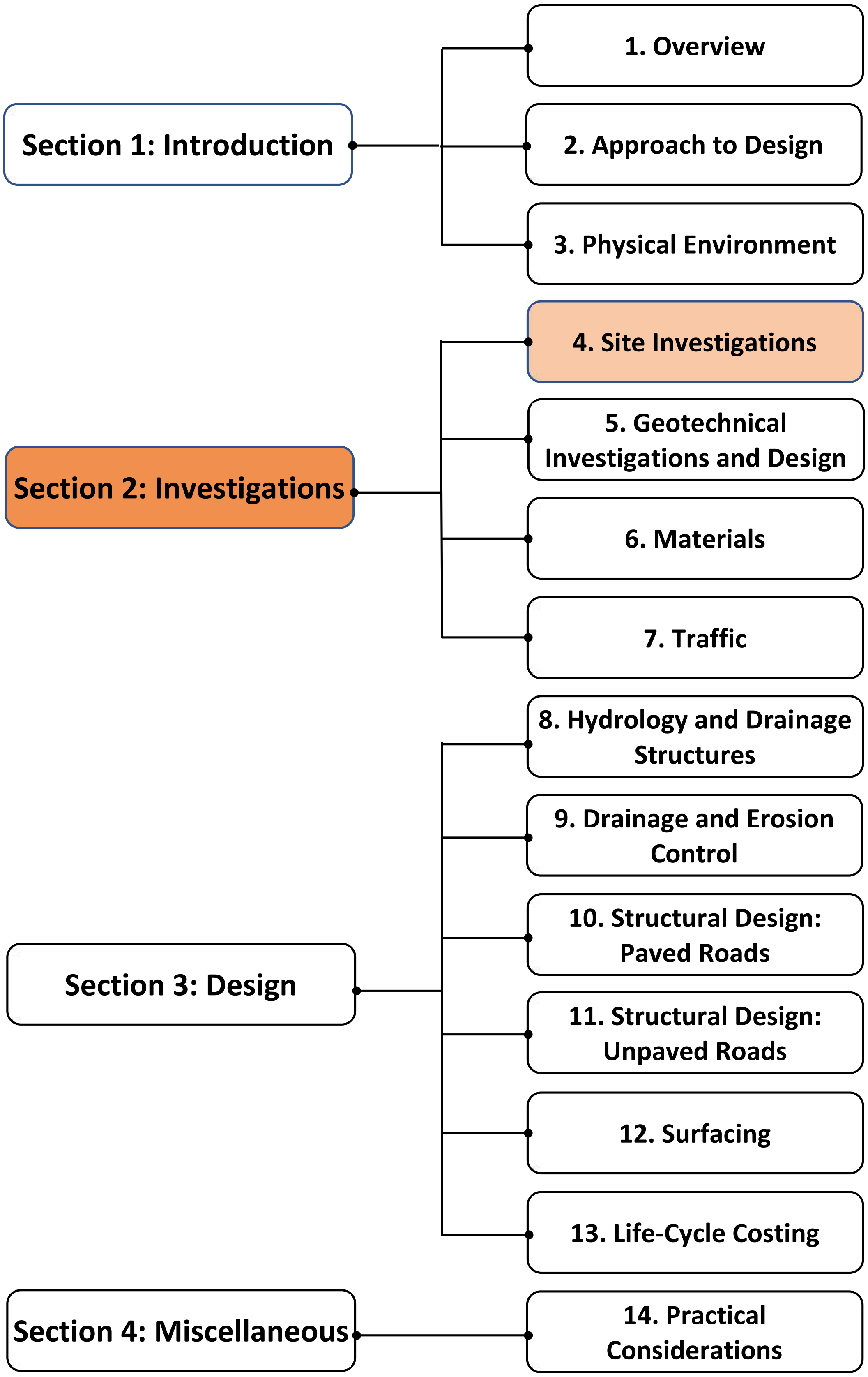
Section 2 Investigations

Low Volume Rural Roads Guideline and Standards
Volume 1 – Pavement Design



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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 LVRR 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 LVRRs in Afghanistan, 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.

This chapter covers those factors related to pavement provision, while specific geotechnical issues, such as earthworks and roadside stabilization, are addressed in *Chapter 5 – Geotechnical Investigations and Design.*

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 the design of LVRRs. It focusses on ‘engineering’ or, more precisely, ‘geotechnical engineering’ investigations related to subgrade conditions. However, other information, as described below, is also required from what can generally be described as site surveys.

- Community consultations to discuss with stakeholders the details of the project.
- Hydrological surveys to estimate the water flows that determine the drainage design of the road and the design of water crossings (refer to *Chapter 8 – Hydrology and Drainage Structures*).

- Traffic surveys to estimate future traffic (including non-motorized road users, number and type of vehicles, and traffic loading) that will use the road over its design life (refer to *Chapter 7 - Traffic*).
- Environmental impact surveys to evaluate impacts and determine mitigation measures.

This chapter provides practitioners with the necessary tools to develop suitable site investigation programs 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 Tertiary or Secondary roads will be constructed on completely new alignments. Where this is the case, the approach to route selection is addressed in *Volume 2: Part A–Geometric Design, Chapter 3–Route Selection and Fundamental Design Considerations*.

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 and scope of site investigation methods are determined by the type of road project, the road class (as defined in *Chapter 2 – Approach to Design*) practical problems arising from site conditions, terrain and climate. Only techniques appropriate for LVRRs 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 for identifying areas requiring specialized input.

For LVRRs, 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 basic properties (index tests) 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 assistance from a materials specialist or geotechnical engineer.

It is the responsibility of the design engineer to determine the frequency and type of tests required for the specific road projects and to assess when samples should be taken for laboratory testing in accordance with the appropriate standard.

Minor realignments

As indicated above, it is not likely that entirely new Tertiary or Secondary roads will be constructed on entirely new alignments. However, short sections of existing roads may be re-aligned for various reasons (e.g. technical or safety). For "greenfield" sections, more emphasis should be placed on the investigation of the subgrade conditions as there will be little visual data to collect in terms of inventory and condition. The techniques that will be used will, however, be the same as for existing roads.

Existing roads

Where a road already exists and needs to be upgraded to a higher class, for example, from a gravel to a paved road, route selection is not required, although some minor realignments may be required, as discussed above. Standard inventory and road condition data must be collected, in addition to subgrade materials testing, as described in Section 4.4. This data will be essential to inform the design as described in *Chapter 10 – Structural Design: Paved Roads* and *Chapter 11 – 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 the final 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 all existing data are reliable. In particular, data might be out of date (e.g., traffic data) or incomplete (e.g., hydrological data).

Sources of information typically include:

- 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 "critical points" 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.;
- 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 depend 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. As much reference as possible should be made to Departmental Geographic Information Systems (GIS), which would normally include a lot of the necessary data.

Initial Survey

An initial site visit and assessment are essential for planning a cost-effective detailed 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 material sources and labor/skills resources, any seasonal accessibility problems, and areas susceptible to slope instability.

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 (see *Chapter 5 – Geotechnical Investigations and Design).*

Furthermore, sources of materials used 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 can affect drainage. It is, therefore, 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 proposed 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	Aspect	Activity
Desk Study	Technical	Review of topographical, geological, soils mapping, including satellite imagery. A preliminary review of alignment and identification of potential problems (materials, 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 LVRR projects, it is vital that communities that will be directly affected by the project are involved at an early stage in the planning process. This ensures that their needs are catered for in the final design and engenders 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 in the choice of technology, i.e., the use of labor-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 perceptions of access problems are often different from men's perceptions. 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 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:

- Subgrade rutting.
- Shear deformation.
- Potholes (structural and not surface).
- Oversize material (if the road is surfaced with gravel or bedrock is exposed).

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 areas. 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 more cost-effectively 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 pavement 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 projected 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 far as possible. Removing, altering, or adjusting existing structures will increase the costs of projects. 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 investigation and analysis.

Geometric design and road safety

The 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, etc. 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 LVRR 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 water-crossings, and through the riverbed using topographic survey instruments to ensure that adequate grades are maintained.

Materials

An assessment must be made of the sources 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 minimize haulage costs.

Traffic assessment

Although the 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, as this will often be carried out by a different team.

Climate

Characteristics of the current climate, such as historical annual rainfall data, should be obtained. The rainfall data required partly depends on the method to be used for designing the drainage (see *Chapter 8 – Hydrology and Drainage Structures*). The projected future climatic conditions and expected changes should be analyzed, based on the best modeling available in the area (see Chapter 3 – Physical Environment).

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 summarizes the activities to be considered during the site visit, while Section 4-3 provides more detail on the activities required for the final design.

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 the project area, climatic data, historical flood events, 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 potential slope stability and 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. • Initial assessment of traffic (observation).
Environmental	Many common environmental issues associated with major roads are unlikely to be significant for LVRRs, 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, Afghanistan's laws and regulations must be complied with. See *Chapter 14 – 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 LVRR project, these activities may also be combined in one site investigation to reduce costs. 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 road structure in order to maximize 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 the 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:

- Surface drainage:
  - Standing water due to rutting, surface unevenness, etc.
- Drainage channels:
  - Effectiveness of side drains (shape of the drain, ponding, silting, scour, erosion)
  - Catchwater drains and cut-off drains (shape of drain, ponding, silting, erosion)
  - Mitre drains (frequency, effectiveness, shape of the drain, ponding, silting, erosion)
- Down chutes (condition, erosion)
- Culverts:
  - Adequacy of opening (size, flooding, length of culvert)
  - Inlet and outlet conditions (ponding, silting, erosion, headwalls, protection works)
  - Structural strength (condition of concrete or other materials)
- Low-level structures (causeways, drifts, small bridges, etc.):
  - Flood levels and duration of closures
  - Adequacy of the existing structure to cope with floods
  - Structural condition
  - Width
  - Erosion
- Bridges (if any)
  - Flood levels
  - Adequacy of the existing structure to cope with floods
  - Structural condition
  - Width
  - Erosion and protection works.

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 (side drains) and borrow areas, leading to the 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 design and construction and to ensure that the materials meet the relevant specifications (refer to *Chapter 6 - Materials*).

4.4 Site Investigation Methods

4.4.1 General

The engineering design requires sufficient data for the 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 reduced simply because the road is classified as an LVRR. This chapter describes and summarizes the principal methods available.

4.4.2 Characterization of Subgrade and In-situ Materials

Dynamic Cone Penetrometer (DCP) Survey

General - All the design methods described in this Guideline makes use of the DCP for characterizing the in situ strength of the subgrade or existing pavement layers. The advantages of using the DCP to characterize 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 (oversize permitting). It is important that the DCP survey is carried out correctly (ASTM D6951) and in a consistent manner by trained personnel to obtain useful and reliable data. It is equally important that the designer participates in the DCP survey to develop a "feel" for the ground conditions and to get first-hand views of the materials involved. This enables a better interpretation of the data once back in the office.

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

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 behavior in the new road. 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 the 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. better compaction, importation of stronger materials, improving drainage or chemical or mechanical stabilization) or the road alignment must be altered to avoid such areas completely.

DCP Survey on existing roads – For upgrading an existing road, it is important to determine the characteristics of the existing pavement layers because these should be utilized in the new pavement structure as far as possible to capitalize on existing traffic compaction and drainage. An adequate structural survey of the existing road is, therefore, essential.

The DCP is light and portable, and DCP tests are quick and simple to carry out, making it highly suitable for the subgrade investigation. The advantage of the DCP is that information can be gathered without unduly disturbing the in-situ material. Using this device, the strength characteristics and thickness of the subsurface material layers at field moisture and density conditions can be obtained directly. The DCP is also useful for identifying sample locations and depths as well as for quality control testing during construction, as discussed later in this Guideline.

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 unless refusal is reached before this depth.

The frequency of the DCP measurements depends on the variability in road/subgrade conditions and the level of confidence required. Where obvious changes in surface conditions occur, the frequency of the testing should be increased in the vicinity of the locations where the changes occur. Similarly, where surface conditions are uniform, the frequency of testing may be reduced. A guideline for the minimum frequency of testing for upgrading an existing track or unpaved road to a paved standard is shown in Table 4-3.

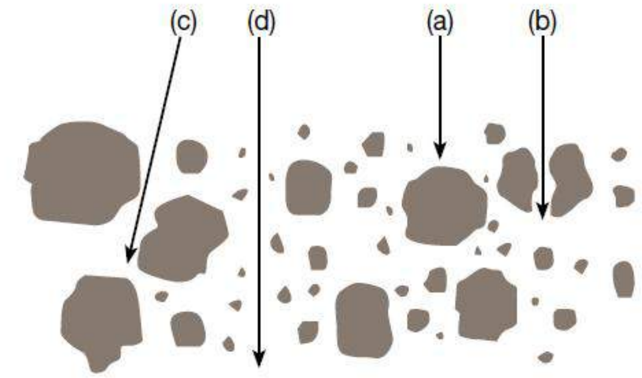
Table 4-3: Frequency of DCP testing

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 the width of the road at outer wheel-tracks (left and right) and the centerline. However, the variability of the road subgrade strength will only become fully apparent when the tests have been carried out. In order to ensure statistical reliability, at least 10 tests must be taken in each uniform section. Hence additional tests may be required after analyzing 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, particularly when there is excessive oversize material. However, in these cases, the in situ strength is likely to be considerably higher than similar materials without the oversize material, and this will usually result in stronger 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 program for the DCP-DN method and UK DCP program for the DCP-CBR method described in Chapter 10).

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 judgment on how to deal with the localized situation to provide sound and cost-effective design solutions. The most common problem found in practice is the occurrence of subsurface moisture due to seepage from higher ground or simply because of inadequate drainage of the existing road or track. The former may require installation of subsurface drainage (expensive) while the latter can often be remedied simply by ensuring that the level of the pavement is raised (i.e., by importing selected fill material or constructing a thicker pavement), and that proper side-drains are constructed.

For the analysis of the DCP test results with any of the design methods, it is 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 10 - 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 Guideline, the DCP is used to characterize 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 a 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 proposed 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 catalog to ensure that all the layers comply with the specifications of the respective design method. This will allow designers to go straight to design catalogs 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 final subgrade level. This may give rise to contractual problems which should be provided for with appropriate clauses and pay items in the contract documentation. The exposed "subgrade" in deeper cuttings is usually partly weathered or fresh rock, and this does not require DCP testing as it can invariably be considered as a suitable subgrade with an adequate penetration rate.

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

The determination of uniform sections, as described in *Chapter 10 - Structural Design: Paved Roads*, statistically limits the variability of the subgrade. For gravel or earth roads in a reasonable condition, where the DCP tests indicate fairly uniform subgrades with depth, three test pits per uniform section for the purpose of material sampling and description in uniform materials to a depth of 450 mm below the surface, are deemed to be sufficient for the determination of the design subgrade strength (keeping in mind that the in-situ moisture content normally increases after construction, resulting in higher DN values with depth in uniform subgrade material).

Engineering judgment must be used to locate the uniform sections at points deemed to be representative of the section.

If there is reason to suspect the occurrence of problematic subgrade soils (see section 4.4.3), in particular for "greenfield" projects, i.e., on sections where the horizontal alignment is new or 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 soil or suitable weathered 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 the pavement layers. This is especially true in areas where a thick layer of problem soils or soft deposits exist and where the road alignment passes through landslide zones, solution cavities and unconsolidated soils.

Sampling and testing

The contribution of a thin layer of 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 the addition of more gravel to form a full base- or subbase layer, in which case the properties and strength of the gravel wearing course must be determined in the laboratory.

The subgrade design strength is determined through the laboratory DN test 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.

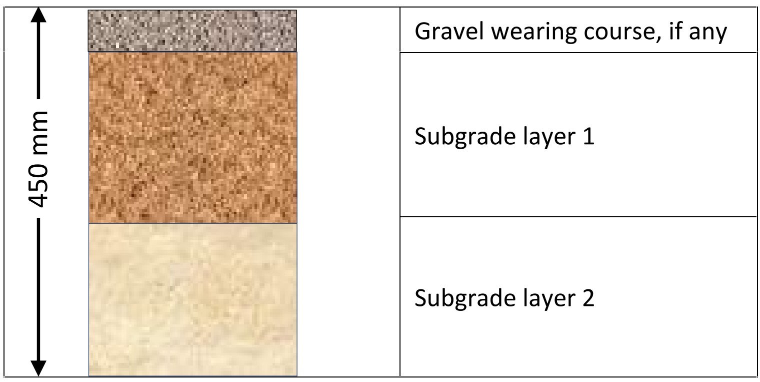


Figure 4-3: Typical soil profile in test pit

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 Test Methods
Soil Profile:		
Overburden		
Layer/horizon thickness		
Visual description		
In-situ moisture content	ASTM D2216	BS 1377 – 4
In-situ density	ASTM D1566 or D6031	BS 1377 – 4
Index tests	ASTM D6913 & D4318	BS 1377 – 2
Compaction (Density/Moisture relationship)	ASTM D588	BS 1377 – 4
Strength	Laboratory DN	CBR & Swell (BS 1377-4)

The soil profile shall be described, and index and compaction tests shall 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 10 – Structural Design: Paved Roads.*

Figure 4-4: Logging of subgrade profile

4.4.3 Identification and Treatment of Problem Soils

General

Any soils which can cause foundation problems, and adversely affect the performance of roads, must be timeously identified and the necessary precautions are undertaken. These could cover extensive areas such as in the case of saline or expansive soils or be localized in the case of dispersive soils. 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 and remedial measures included in the design. Those areas with particularly problematic soils often require detailed investigations and testing by a specialist materials or geotechnical engineer. Failure to recognize 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.

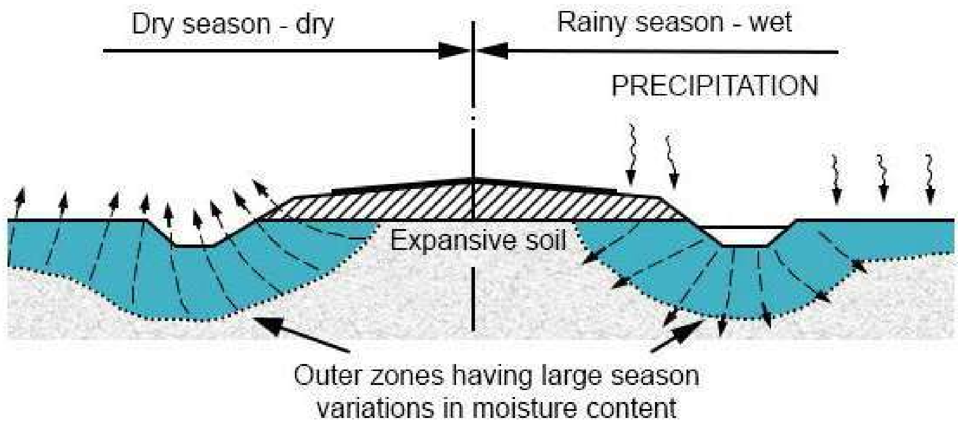
In assessing the appropriateness of the measures available for dealing with the above problem soils, a careful balance has to be struck between the cost of the measures and the benefits to be derived. This would normally require that a life-cycle cost analysis be carried out to determine whether the costs of the measures would be at least off-set by the benefits (see *Chapter 13 – Life Cycle Costing*).

Expansive soils

Causes – Expansive soils are those that contain smectite (generally 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, ophiolites and dolerites in tropical and sub-tropical areas (current and past climates). 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 soil can be related to many factors, it is primarily controlled by the quantity and type of active clay minerals (e.g., smectites).

Recognition – Expansive soils are those that 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 the fluctuating movement of the water table or surface water. 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 movements. It is these movements 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 (leaking) culvert sites.

Volume changes in expansive soils are confined to the upper few meters 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.



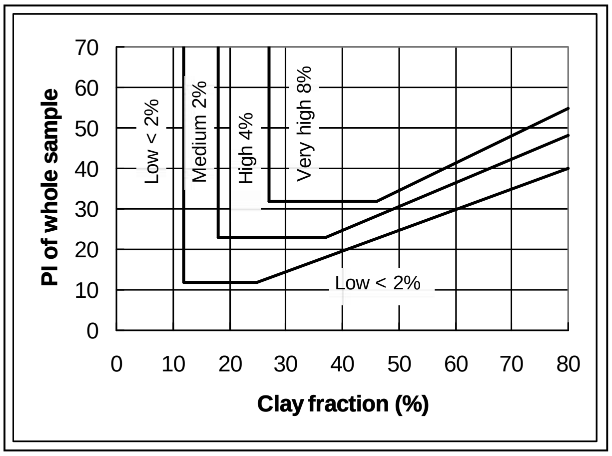
Source: Weston, 1980

Figure 4-5: 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 4-6. 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 determined using X-ray diffraction analyses.

Figure 4-6: 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 4-7, 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 Merve, 1976

Figure 4-7: 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 (or vertisols) will always have expansive characteristics, while any dark (grey or black) soils with a high base status (or eutrophic) and high 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 (expansiveness) determined from Figure 4-7.

If the calculated potential heave exceeds 25 mm to 50 mm (i.e. the product of % expansiveness and thickness of layer), 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 localized drainage differences, the countermeasures will need to take this into account to avoid localized sections of the 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 undisturbed specimens cut from block samples. The 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 LVRRs over expansive clays include:

1. Flattening of embankment side slopes (between 1V: 4H and 1V:6H).
2. Remove expansive soil and replace it with inert material (between 0.6 m and 1 m or more depending on the 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 deep 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, which are costly).

Figure 4-8 provides a preliminary indication of possible countermeasure options (numbered as above) as a function of potential expansiveness and road class. 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.

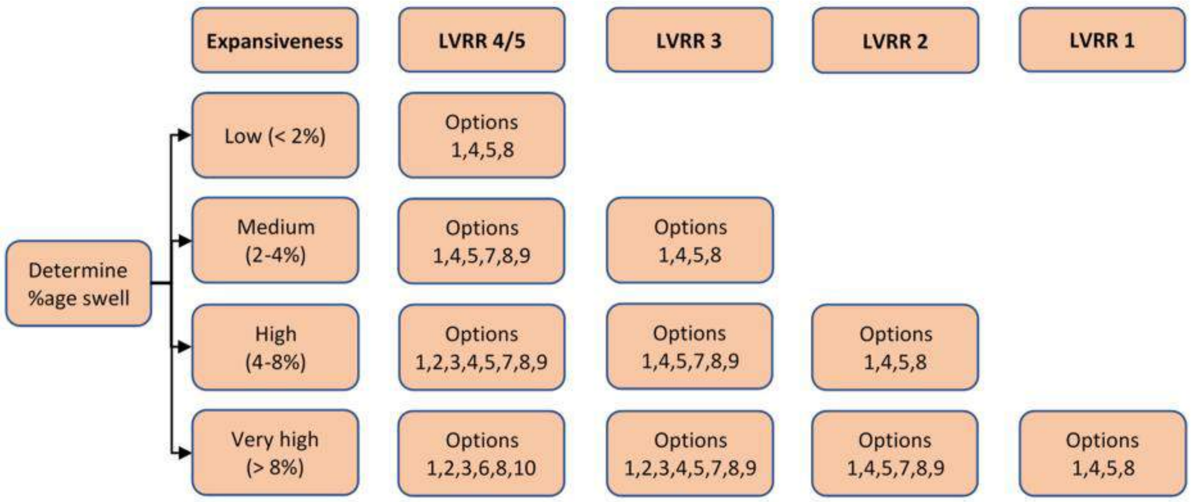


Figure 4-8: Possible solutions for the construction of roads on active clays

For the lower LVRR classes, it may be more economical to retain the road as a gravel road over the expansive clay subgrade sections and apply the necessary maintenance.

One of the most important considerations is to try and minimize the zone of seasonal moisture movement beneath the road, as shown in Figure 4-9, and to widen the zone of moisture equilibrium. A combination of slope flattening, material replacement, sealed shoulders, and lined side drains, as shown in Figure 4-10 is usually the most cost-effective means of achieving this, but the design of countermeasures needs to be specific to any situation.

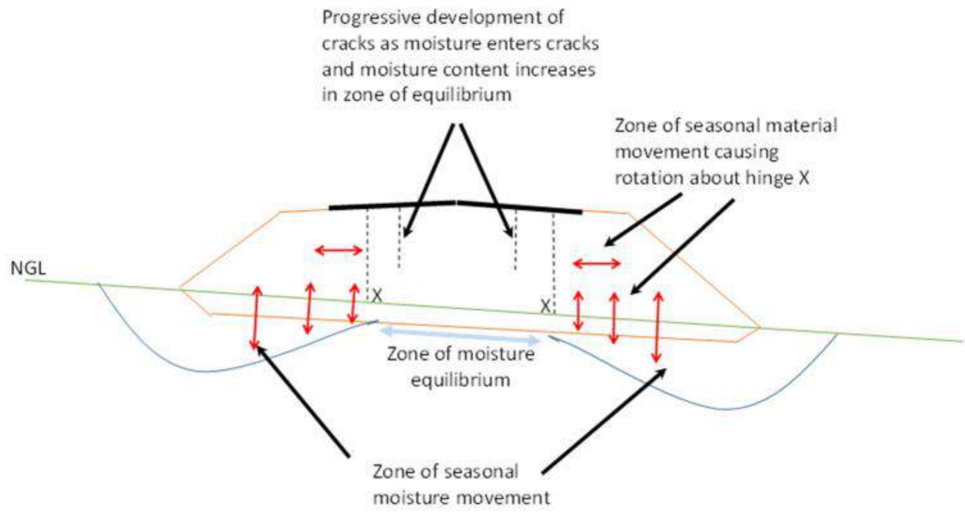


Figure 4-9: Typical moisture movement regime under roads on expansive clays

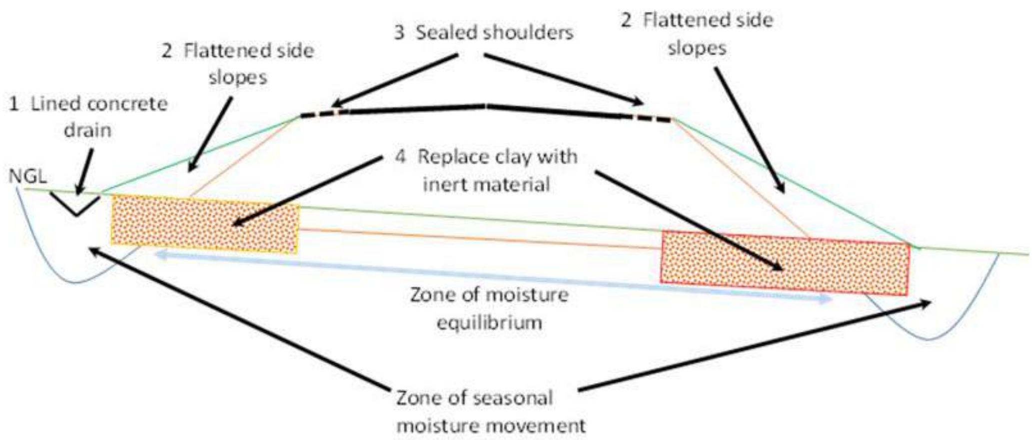


Figure 4-10: Some countermeasures used to increase zone of moisture equilibrium

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 imported material. Unfortunately, this is often impracticable or uneconomic for LVRRs, unless the problem is localized. 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.

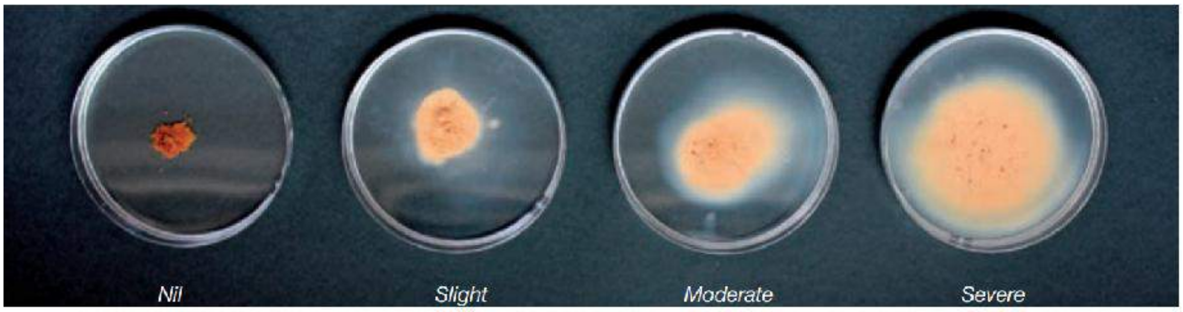
The recommended, and probably most economical solution specifically for LVRRs showing high to very high potential swell, is to partially remove the clay from the subgrade and replace it with a less active material, increase the fill height using inactive material to provide a greater load on the clay, 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 and less critical fill slopes rather than in the stiffer pavement structure.

Particular attention should be paid to culverts. The clay beneath them must be replaced with an inert material, and 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 (caused by adsorbed sodium ions) that exceed the attractive forces. This results in the colloidal fraction going into suspension and in still water staying in suspension. In flowing 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 characterized by an annual rainfall of less than 850 mm. The saline materials and arid nature of southern Afghanistan make this area particularly conducive to dispersive soils.



Source: Davies and Lacey, 2009

Figure 4-11: Degrees of soil dispersion

Dispersive, erodible and slaking soils are similar in their field appearance (highly eroded, gullied and channeled 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 **4-12** shows a typical dispersive soil with definite evidence of piping (or tunneling).

Figure 4-12: 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.

Slaking soils disintegrate in water to silt, sand and fine-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.

Figure 4-13: Gulley formation in erodible soil

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 gain access to the subgrade or fill. Problems are thus mostly associated with poor culvert and drainage design and poor protection works. The inclusion of dispersive soils in the subgrade or fill, on the other hand, has been seen to lead to significant failures through piping, tunneling, and the formation of cavities in the structure. It is, therefore, essential to identify dispersive soils timeously.

Recognition – The testing and recognition of dispersive soils require 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 definitive. Dispersive soils tend to produce a colloidal suspension or cloudiness over the crumb/lump during the test (Figure 4-11), without the material necessarily disintegrating fully. The disintegration of the crumb in slaking soils is very rapid and forms a heap of silt, sand and fine 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 low sodium components have also occasionally 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 stabilization with its associated cracking, allowing water to move through the cracks and accelerate problems.

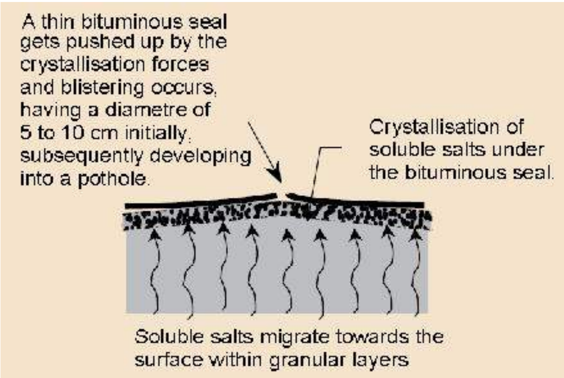
It is also important that the material is compacted at 2% to 3% above optimum moisture content to as high a density as possible.

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

Saline soils

4.4.3.1 **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 (Source: *Roads Department, 2001*

Figure 4-14) leads to weakening of the upper base and blistering and disintegration of the surfacings. Soluble salts, particularly sulfates, and their acids can also have a seriously detrimental effect on the stability/durability of chemically stabilized materials and concrete.



Source: Roads Department, 2001

Figure 4-14: 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 and particularly inland drainage areas such as those common in Afghanistan resulting in seasonal flows of water are particularly prone to the accumulation of salts leached from the surrounding areas. In other flat areas, the capillary rise of groundwater and precipitation in saline soils can result in the upward migration of salts to or near the soil surface. The calcareous nature of soils common in Afghanistan is often associated with saline conditions, and testing, as described below, should be implemented when calcareous soils are being investigated for use in roads.

Recognition – In some cases, the visible presence of crystallized 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.

Figure 4-15: Salt deposition on the soil surface

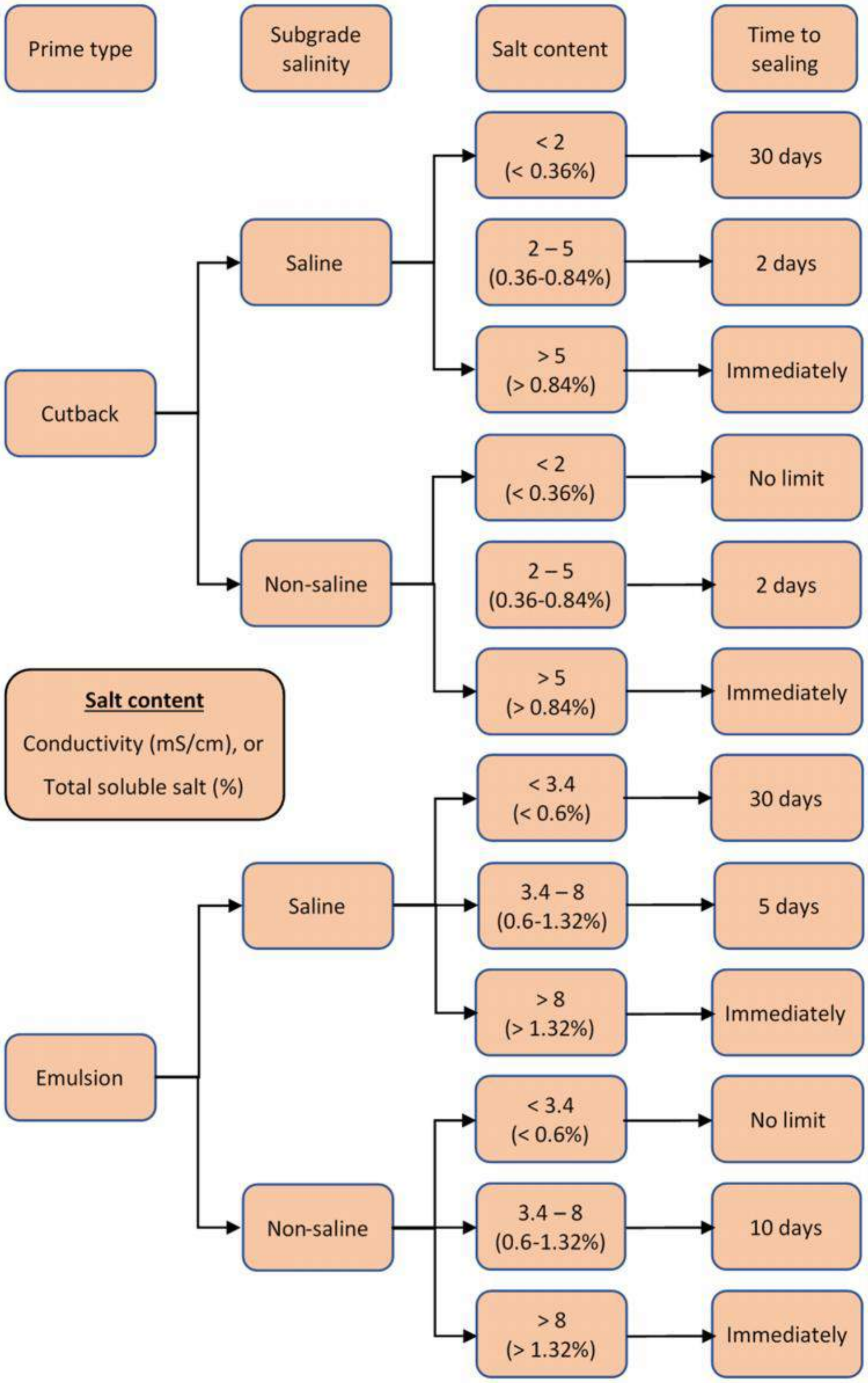
In the conventional road engineering context, the identification of possible soluble salt problems is based on the pH and conductivity of the materials. It should be noted that the results of the electrical conductivity and pH tests can vary significantly depending on the material pre-treatment, the 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 electrical resistance of less than 200 Ω on the minus 2 mm fraction) should raise a 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.

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, minimization of salts in the pavement layers and subgrade should be attempted.
- If the surfacing is sufficiently impermeable (coefficient of permeability, k in nanometer/ second)/ surfacing thickness, T in mm or k/T < 30 (μsec)-1) to avoid water vapor passing through it, crystallization will not occur beneath the surfacing.
- Construction should proceed as fast as possible to minimize the migration of salts through the layers. Only impermeable primes should be used, e.g., bitumen emulsions. Figure 4-16 provides an indication of the allowable delay between priming and sealing for various material subgrade salinity.
- The addition of lime to increase the pH in excess of 10.0 will also suppress the solubility of the more soluble salts.

Even for the lowest classes of road (LVRR1 and LVRR2), 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.



Source: Roads Department, 2001.

Figure 4-16: Permissible intervals between priming and surfacing

Micaceous soils

Causes – Micaceous soils contain large quantities of mica (usually muscovite and biotite) and occur in such materials as weathered granite, gneiss, mica schist and phyllite. Such soils belong to a group of minerals characterized by their extreme platy (flake-like) cleavage and can be distinguished by the presence of individual grains/flakes 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 4-17: Micaceous soil with large flakes of muscovite

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 (XRD) analysis to determine the quantity of mica present in the different fractions of the sample. A simple laboratory test that can be used is to compact the material into CBR molds and determine the density. Without soaking, measure the swell characteristics over a 4-day period. If the material increases in volume, the potential for compaction problems exists, and use of the material should be avoided or corrective measures taken as discussed below.

Countermeasures – The typical upper limit 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 behavior and the determination of their properties. In an arid environment such as Afghanistan, such materials would be localized and unlikely to cause widespread problems.

The shear strength of these clays would normally be between 10 kPa and 40 kPa, making them impossible to or at least 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/stability 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 need careful control during their construction to avoid stability failures as pore water pressures increase rapidly 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 level of performance of the road.

The use of a wide range of geosynthetic products as separation layers, and to facilitate and accelerate drainage (e.g., wick drains), 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.

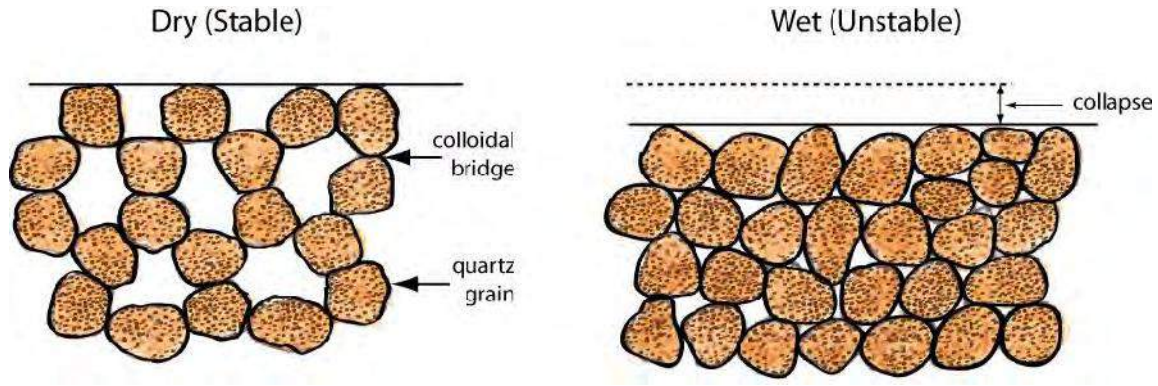


Figure 4-18: Graphical illustration of mechanism of collapse

Collapsible soils can occur on both residual and transported materials. Loessic soils are particularly susceptible to collapse as are residual/deeply weathered feldspathic sandstones and granites. The latter, when weathered, result in the feldspar altering to kaolinite with the quartz particles staying intact. This forms the 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 the 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/m^3 (mostly in the range 1000 kg/m^3 to 1585 kg/m^3).
- The presence of "pinholing" or voiding observed during 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, however, will seldom have a major impact on their performance.

The result of the 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 – Despite the arid nature of Afghanistan, 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 recognize potential wet conditions, which are characterized by areas of standing water, specific types of vegetation (reeds, papyrus grasses, etc.), localized muddy conditions. Site investigations during the rainy season will also provide ready information.

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.

Figure 4-19: Typical road condition in a wet/marshy area

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

The installation of subsoil drainage systems is seldom justified, on economic grounds, for use on the lower classes of LVRRs (LVRR1 to LVRR3) because of the cost of installation and the ongoing need to maintain them properly. However, for the higher classes of LVRRs, they may be justified.

Sinkholes

In areas with carbonate rocks (dolomites and limestones), the potential for dissolution of the rock material by rainwater (often a weak acid) 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 4-20: Sinkholes

4.4.4 Location and Characteristics of Construction Materials

General

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

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, usually entailing thicker layers or material improvement (i.e., stabilization), both of which can be costly.

The investigation of construction materials usually requires an extensive program 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 summarized below and discussed in more detail in *Chapter 6 – 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., rockfill 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 regraveling of unpaved roads 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 regular re-graveling is normally needed to replace material lost from the surface. Sources of good materials could be depleted, resulting in increased haul distances and subsequent costs in the future. 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 4-21: 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 the 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, possible flow of surface water into the excavation ground and its potential use for construction; 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 pose few problems. Exceptions include organic soils and clays with high liquid limit and plasticity. Problems may also exist in lacustrine (stratified deposits at the bottom of a lake) and flood plain deposits where very fine materials are abundant. Where possible, fill should be taken from within the road alignment (balanced cut-fill operations) or by the excavation of the side drains (where materials meet the requirements). Borrow pits producing fills should be avoided as far as possible due to cost implications, and special consideration should be given to avoid winning fill in agriculturally productive areas where land expropriation impacts can be high.

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 (capping layer) may be necessary. As far as possible, the requirement to import material from borrow areas should be avoided due to the additional haulage costs. However, the import of strong (CBR>9) subgrade materials can provide economies because pavement thickness design can be reduced (refer to *Chapters 10 and 11 – Structural Design*). Where improvement is necessary or unavoidable, mechanical and chemical stabilization 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 treatment may be required to improve the quality to the required standard. This often requires the use of special equipment and processing plants that are relatively immobile or static. In such cases, the borrow pits for base and subbase materials are usually spaced widely.

The main sources of base and subbase materials are rocky hillsides and cliffs, high steep hills, river banks and naturally-occurring residual soil deposits and pedocretes, e.g. calcretes and laterites. Base and subbase materials are expected to meet the specifications for the design method being used (*Chapter 6 - 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 meter. However, thinner horizons could also be exploited if there are no alternatives. The absolute minimum depends on material availability, the thickness of the overburden, and the methods of excavation. If there is no overburden, as may be the case in arid areas, horizons as thin as 300 mm may be excavated.

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

Figure 4-22: Variable degrees of weathering in quarry

Locating and testing construction materials
Field surveys and associated laboratory testing programs 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 concrete or surfacing).

Projects involving significant cut and fill requirements will require mass-haul diagrams to help designers understand where gross material imbalances occur as a basis for comparing alternative designs, e.g., by modifying the material through stabilziation or 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 materials should be done by traditional methods using established, full laboratory facilities.

Table 4-5: Minimum 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 heterogeneous the material, the more 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 LVRRs, 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 (usually the four corners and the center) 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 the 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 and estimate 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 analyzed for salinity (to assess the concentration of chlorides and sulfates), which could be deleterious to the performance of concrete and bituminous materials.

4.4.5 Geotechnical Investigations

General

The geotechnical investigation involving aspects such as the types and extent of excavations, the foundation works, and the assessment of problem subgrades is a more sophisticated exercise than the activities described previously in this chapter. 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) (see following chapter); and
- Identify areas of potentially problematic soils requiring additional investigation and treatment.

In many cases, especially smaller projects, the preliminary 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 will usually be required.

Geotechnical investigations are progressive, 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 far as possible. More sophisticated and expensive techniques should only be employed when a significant geotechnical problem is encountered with potentially severe consequences should a failure occur. Under such circumstances, it is necessary 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.

Figure 4-23 outlines the key objectives of ground investigations which may be undertaken using a variety of sampling and testing techniques, outlined in Table 4-6.

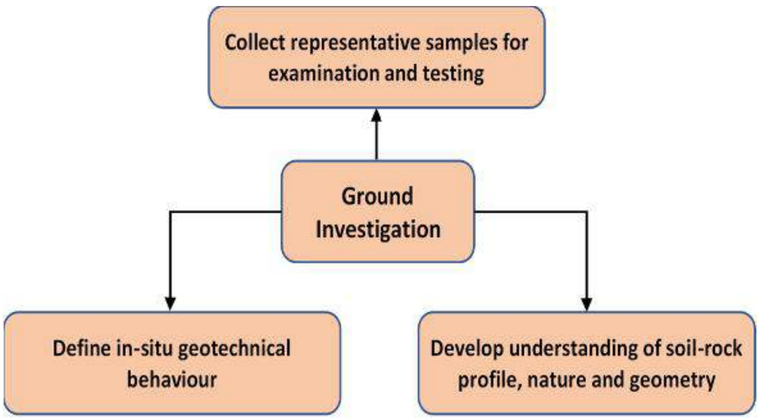


Figure 4-23: Objectives of ground investigation

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 is quick and simple.	A minimum of 5-20 DCP tests/km should be used for LVRRs.
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 geo-phones provide a cheap option.	Can use for key areas where rock head is uncertain and critical for design.

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;
- 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 localized loads are expected, (e.g., bridge abutments and piers). The soil must have a high enough bearing capacity to support these loads. Where the bearing capacity of the soils is deficient, 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 4-7 gives a guide to the minimum 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 given only 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 4-7: Guideline for minimum number of trial pits for structure foundations

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

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 approach 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 4-24. Damage due to scour is one of the most likely causes of structural failure. Minimizing 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:

Figure 4-24: Typical bridge scour

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 in the flow characteristics of the water depend on the actual design as well as the stream channel geometry and water flow rates. Simplified empirical methods for determining scour at water crossings are presented in *Chapter 8 – Hydrology and Drainage Structures.* However, the geotechnical investigation should provide the engineer with a basic knowledge of the scour characteristics of the materials.

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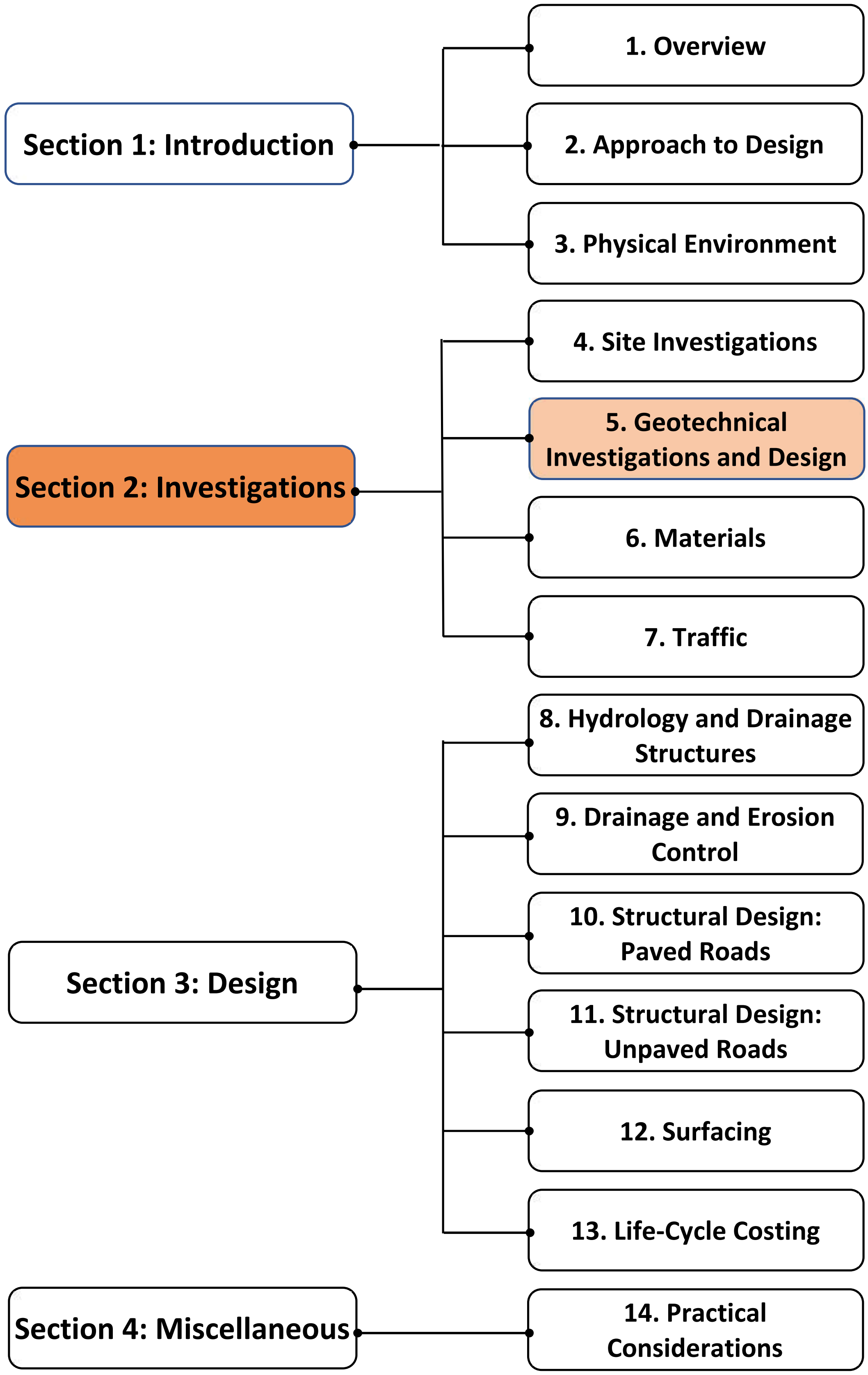

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5.1 Introduction

5.1.1 Background

Much of Afghanistan is mountainous with steep terrain, weathered and closely-jointed rocks, complex geological structure, high exposure to earthquakes, extensive deposits of transported soils and a climate regime in which snowmelt and short-duration intense rainfall form the conditioning and triggering factors for landslides, rockfalls and related geohazards. Roads constructed in this terrain are not only potentially exposed to these geohazards, but their construction can significantly contribute to levels of instability through excavation, spoil disposal and drainage disturbance. These factors are related to the geology and geomorphology of the country, as discussed below and addressed in general terms in *Chapter 3 – Physical Environment*.

5.1.2 Purpose and Scope

The purpose of this chapter is to provide guidance on the design of earthworks for LVRs in this terrain and, in particular, the design of cross-sections and cut slopes most suitable for the topography and underlying geology. It also provides guidance on slope stabilization measures, concentrating on low-cost techniques, but also providing some discussion on the more expensive solutions that might be necessary for high-risk situations.

Issues concerning route selection and, in particular, the need to select alignments with the least exposure to landslide hazards are discussed in *Volume 2 – Geometric Design, Section 3-2*, while techniques of bio-engineering for slope protection are discussed in *Chapter 9 – Drainage and Erosion Control,* Section 8.6.5.

5.2 Geology and Geomorphology

The rocks of Afghanistan have varying strengths and structural patterns and pose varying levels of hazard to road infrastructure. Argillaceous rocks, such as shales, claystones, mudstones and clayey siltstones, are typically low-strength and can be especially problematic when encountered in roadside slopes. While strong rocks, such as granites, limestones and quartzites are much stronger, their stability is usually controlled by the orientation of their jointing pattern, and they can pose significant rockfall and rock slide hazards.

Over thousands of years, landslides and similar processes have deposited extensive thicknesses of material referred to as colluvium. These are transported soils that are prone to instability when excavated. Glaciation during previous ice ages has been responsible for the deposition of till and outwash sediments and loess deposits (windblown sands and silts) are particularly common in some parts. These soils are prone to liquefaction when saturated and can result in earth flows that travel significant distances. Finally, alluvial fans and river terrace sediments occupy many parts of the lower ground. Many rivers have cut down through recent sediments to create deep canyons formed in unconsolidated deposits, forming steep and unstable side slopes. River scour, especially on meander bends, is responsible for the loss of toe support to hillsides above, triggering a failure.

High seasonal rainfall is responsible for temporary saturation of soils and water pressures in rock joints causing slopes to fail, particularly where over-steepened by roadside excavations. These are the geological and geomorphological conditions that should be borne in mind when developing slope management strategies for roads located in the hilly and mountainous parts of the country.

5.3 Earthworks

5.3.1 General

In order to comply with horizontal or vertical geometric guidelines, and thus permit reasonable access for users, LVRR alignments in hilly or mountainous areas inevitably require the construction of cuts whilst 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 minimize 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 LVRR route alignment and the geometry or instability of the natural slopes may be such that an earthworks solution alone is neither economical nor feasible. In particular, excavation volumes would be prohibitively high in steep terrain if cut slopes were designed to be unsupported. Thus, retaining walls, revetments and other measures might need to be considered in order to avoid excessive cut back. However, in resource-constrained situations, it is almost always necessary to find a suitable compromise between risk level, risk reduction and investment. For example, if temporary road closures and debris clearance can be tolerated from time to time, then a steeper slope without expensive stabilization and protection may be more economical.

A particular difficulty in steep terrain is the disposal of excess excavation material (spoil). Whether a balanced cut and fill can be achieved will depend on the detail of the topography and the suitability of excavated material as fill. In mountainous terrain, this is not possible because the ground slope is too steep (usually greater than the designed stable slope angle (1 Vertical (V):1.5 Horizontal (H)) or the natural angle of repose of most granular materials) for unsupported fill. Accordingly, it is important to identify suitable stable areas for the disposal of spoil material prior to any earthworks. The selection of these areas must also be compliant with the country's environmental law and usually requires close consultation with local stakeholders. If improperly disposed of, the spoil can cause slopes to fail by overloading, can strip slopes of any protective vegetation cover, can alter drainage patterns leading to erosion, and can pollute watercourses below with debris. During maintenance operations, the clearance of landslide and other debris will also require careful selection of disposal sites.

5.3.2 Investigations

Prior to designing the earthworks of a new road, or an improved and widened road, it will be necessary to carry out some field investigations to determine the likely materials to be encountered. In the case of new roads, surface mapping of exposed soils and rocks combined with trial pitting is the usual way of deriving this information. Usually, in undulating to hilly terrain, most cut slopes will be formed predominantly in soil, i.e., alluvium, colluvium or soil derived from in situ weathering. Field observations and trial pitting will allow the soil profiles to be assessed for design purposes. High embankments may be required in some circumstances, and it is normal practice to carry out trial pitting, soil classification and in situ density tests for embankment loading purposes.

In steep terrain, an indication of soil depth, soil type and underlying rock type can often be provided through field observation without the need for trial pits. However, where retaining walls are required to be constructed below the road (see below), trial pitting in advance of design and construction will be necessary to determine foundation conditions, the depth to rock as a potential founding stratum, and possible groundwater conditions. In situations where roads are to be widened as part of an improvement program, then exposures in the existing cut slopes will indicate what is likely to be encountered when widened.

5.3.3 Choice of Cross-section

In flat to undulating terrain, there will be little to no requirement for cutting, and the majority of the road will be either at grade or on an embankment. In hilly terrain, a combination of cut and fill will be necessary, and the imperative will be to design an alignment with cross-sections that yield a balance between these quantities. Excessive cuts can destabilize slopes above and pose problems for spoil disposal. Inevitably, there will be an excess of excavation material that is required to be disposed of, and it is important to identify safe disposal sites as part of the earthworks design, i.e., prior to construction. The term 'safe' is used to include sites that are stable geotechnically and sites where dumping of spoil will not trigger slope instability and erosion. 'Safe' is also used in an environmental context, because land use, water supply and drainage patterns must also be protected.

In mountainous terrain, where natural slopes are usually above 30°, there may be very little opportunity to achieve a balance between cut and fill because the fill cannot be placed on the steeply sloping ground, as discussed previously. The cross-section will usually comprise full cut or predominantly cut with fill supported by retaining walls. Retaining walls can be very useful for spanning sections of alignment that would otherwise be difficult to build due to topographical constraints (Figure 5-1).

Source: (Hearn 2011, reproduced with permission of the Geological Society, London)

Figure 5-1: The use of retaining walls to span rock embayments

Table 5-1 summarizes some of the issues that should be considered when designing road cross-sections in mountain areas. Consideration of the underlying geology is paramount, as illustrated in Figure 5-2.

Table 5-1: Considerations for road cross-section in mountainous areas

Type of section	Advantages	Disadvantages
Full cut	- Road formation requires minimum compaction because it is formed entirely in natural ground. - No requirement for fill slope placement or compaction. - Potential source of fill material for use elsewhere along the road. - Potential source of rock, if present, for masonry, aggregate and drainage backfill. - Usually the only practical solution if existing ground slope > 50°.	- Greater height of cut may lead to greater instability and/or erosion. - May result in large volumes of spoil requiring safe disposal.
Part cut and part fill	- Volume of spoil minimized if balanced cut/fill can be obtained. - Minimum impact on landscape.	- Requirement for fill placement and compaction. - May require below-road retaining wall or reinforced fill to avoid excessive area of fill if existing ground slope > 30°.
Full fill (including wall-retained fill)	- Usually only practical solution when traversing re-entrants or water courses. - Usually only practical solution (with fill retaining structure) on steep rock slopes if jointing is adverse to stability.	- Requirement for significant fill import, ground preparation (including benching), placement and compaction. - Will require below-road retaining wall or reinforced fill to avoid excessive fill area if existing ground slope > 25°. - Impracticable if existing ground slope > 40°.

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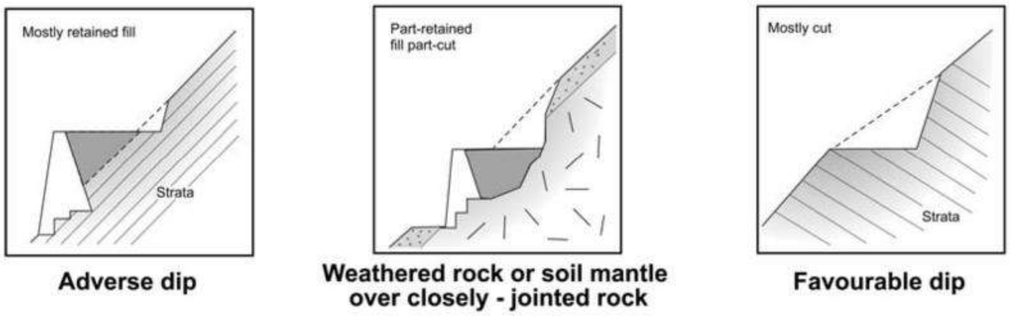

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Figure 5-2: Selecting cross-section based on underlying geology

5.3.4 Cut-slopes and Embankments

General

Although the construction of high fills and deep cuts should be minimized for low volume roads on cost grounds, there may be certain areas with variable topography such as in the highland areas that require larger earthworks. Where possible, it is better to have sharper bends and short steeper grades (suitably sign-posted) for low volume roads than to spend large amounts of money constructing high fills and blasting deep cuttings through rocky hillsides.

Localized areas where heavier earthworks are necessary need to be identified and inspected during the site investigation so that the necessary geotechnical investigations can be planned and budgeted for.

These investigations should consider:

- the types of materials in the cut;
- slope stability; and
- the different types of movements that may occur.

Instability may be evidenced by one or more of the following features:

- Past slope movements (manifested by scars, anomalous bulges, odd outcrops, broken contours, ridge top trenches, fissures, terraced slopes, bent trees and misaligned fences, abrupt changes in slope or in stream direction, springs or seepage zones);
- The presence of tension cracks;
- The nature of the rock mass and the rock material;
- Presence of water (manifested by excessive vegetation growth or vegetation out of season);
- Road surface damage from falling material.

These areas often require carefully designed drainage structures (usually in conjunction with the cuts and fills) in order to convey concentrated water flows down to lower altitudes and these flow paths need to be identified. Evidence of active erosion and the transportation of large boulders usually make such watercourses obvious. Changes in future precipitation events arising from climate change should be considered in determining the capacities of the drainage structures.

Natural slopes, road cuts, and existing embankment fills in the vicinity of the planned project provide preliminary evidence of expected ground stability and likely requirement for detailed surface and subsurface investigations.

Initial investigations for cuts and excavations should concentrate on identifying those medium to high-risk areas where additional review and specialist investigation are necessary. A simple flow-chart can be used for this purpose, as shown in Figure 5-3. Based on various slope parameters shown in the chart, it can be decided whether sufficient information is available to design the slope on precedent (past experience, neighboring slopes that are similar, standard tables, etc.) without additional review or whether to carry out a detailed geotechnical investigation (review).

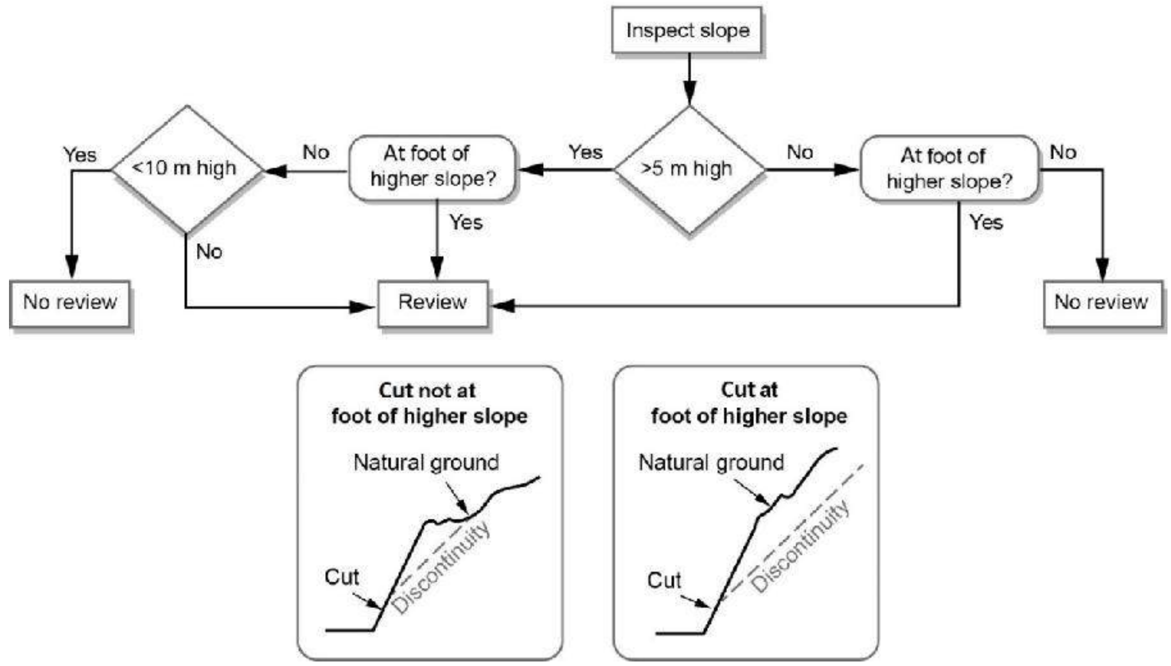

Figure 5-3: Decision chart for design of road cuttings

The question as to whether the cut is at the foot of a higher slope is extremely important. If the cut does not undercut a higher slope, failure will have minimal consequences. However, where the slope continues above a cut, failure of the cut will usually result in large quantities of material higher up the slope becoming unstable and moving onto the road.

5.3.5 Ground Investigations

It is usually necessary to inspect the material to be excavated in order to assess the depth of weathering, material types and the inclination of the strata. Trenches are preferable to pits to inspect cuts because of their dimension. Depending on the geology and degree of weathering, up to five trenches are normally enough to investigate a 100 m long slope cut. The trenches should be located at points where material changes are expected and range between 1 m and 3 m in depth. For safety reasons, great care needs to be taken by personnel accessing trenches or pits greater than 1 m in depth. Adequate support to pit or trench walls is essential, and any investigators entering the pits or trenches must be accompanied by surface safety supervision.

During the investigation for cuts, the material that will be removed should be assessed for use as fill or even pavement construction materials. This will entail sampling and laboratory testing. However, as large excavations usually involve a wide range of material qualities (and even different materials), this sampling needs to be carefully planned in order to get accurate assessments of the quantities of different materials for later use.

The investigation for fills (or embankments) is somewhat easier than for cuts as the fill material itself is selected and constructed to specified standards ensuring adequate shear strength to avoid failure within the fill. This leaves only the underlying subgrade and support areas to be investigated.

The problems that are likely to be encountered are essentially the settlement of the fills or shear failure, both of these being influenced by the properties of the underlying material. The aim of the geotechnical investigation for fills is thus to determine whether the thickness and compressibility of the underlying material are significant enough to cause excessive settlement and whether the shear strength of the underlying material is sufficient to avoid shear failure. Both of these properties are strongly related to the moisture content, and this needs to be taken into account in the testing and catered for in the design. The design issues will usually require the collection of undisturbed soil samples for laboratory strength and consolidation testing. The vane shear test can also provide valuable in-situ strength data, particularly in soft clays.

Typically, test pits or trenches would be the first investigation requirement. These are normally excavated to about 3 m and the materials in the pits classified and described. Any soft or wet cohesive materials less than 2 m thick are likely to result in settlement of the fill and/or possible shear failure of the base. Such materials should be removed or treated, or pre-loading in stages should be planned to accommodate the settlement and dissipation of pore water pressures. Where soft or wet cohesive materials extend deeper than about 2 m, they should be considered as potentially problematic, and specialist geotechnical investigations should be carried out. These should aim at providing quantitative estimates of the amount of potential settlement and its rate as well as providing sufficient data to carry out stability analyses. For fills higher than 3 m, if there is any doubt in the investigators' mind following the site investigation and preliminary geotechnical investigation, specialist assistance should be sought, as failures can have significant consequences.

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 during the geotechnical investigation.

5.3.6 Cut-slopes

Where possible, LVRR cut-slopes are generally designed based on past experience with similar soil and rock materials. Cut slopes greater than 5 m – 10 m in height may require a more detailed engineering geological assessment depending on the complexity of the ground conditions (Figure 5-3). This would include an assessment of the type of the soil-rock materials and their mass structure.

Recommended cut-slope angles in different types of material are presented in Table 5-2.

The cut-slope angles given in Table 5-2 are for rock masses without adverse jointing or bedding. Where discontinuities dip adversely out of the slope, excavations will need to be reduced to accommodate them (e.g., Figure 5-4 and 5-5). In steep terrain, this would require large volumes of excavation, which might prove prohibitively expensive and will result in large volumes of material to be disposed of. An assessment of the site condition will lead to a decision being made over a) reduced cutting angles, b) structural support using retaining walls and other measures, or c) an acceptable level of risk from slope failure. What is important is the need to derive a solution that yields an acceptable level of safety for traffic and minimizes potential road closures. Fig. 5-6 illustrates the case where the dip of bedding appears to be favorable to slope stability, thus allowing steeper excavations than might otherwise be the case.

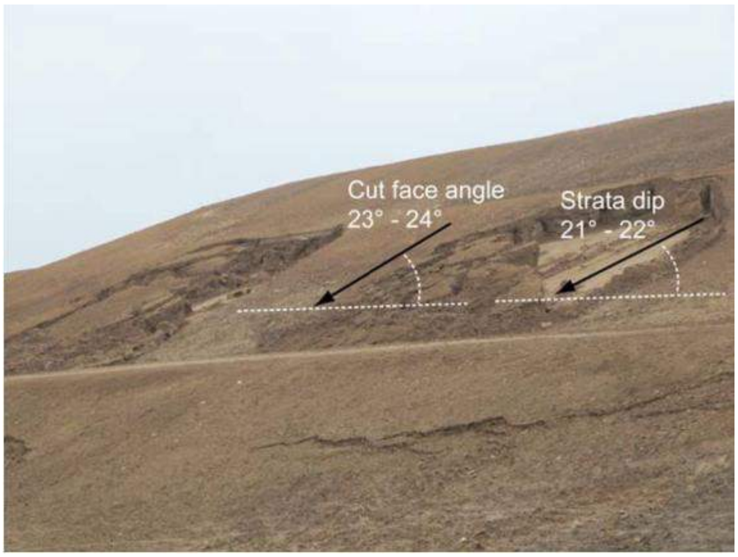

Figure 5-4: Bedding plane failure in sandy clays

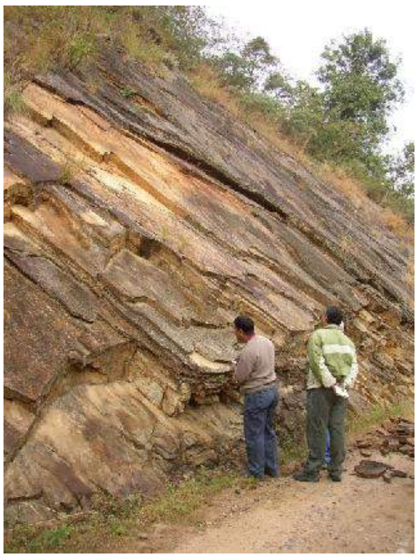

Figure 5-5: Joint-controlled stability in cut slopes

Figure 5-6: Siltstone or fine-grained sandstone exposed in the cut slope

Figure 5-7: Unstable rock mass, cut too steeply with blast damage

Table 5-2: Recommended indicative cutting angles for various rock and soil types

General class of material	Typical rock and soil types	Indicative cut slope angle related to cut slope height (V:H)		
		< 5m	5m to 10m	> 10m
Strong blocky rock mass with widely-spaced joints and bedding planes	Fresh to slightly weathered granite, basalt, limestone, well-cemented, strong sandstone	5:1 – 10:1		5:1
	Moderately weathered granite, basalt, limestone, well-cemented, strong sandstone	3:1 – 5:1		2:1
Weak or partially-disturbed rock mass with moderate to closely-spaced joints	Slightly weathered basalt and volcaniclastic rocks.	2.5:1	2:1	1.7:1
	Moderately weathered basalt, limestone			
	Slightly weathered interbedded limestone and marl, mudstone and shale, and poorly-cemented sandstone	2:1	1.5:1	1:1
Very weak and/or fragmented rock	Moderately weathered mudstone and marl and closely jointed and open jointed basalt, limestone	1:1	1:1.25	1:1.5
	Highly weathered mudstone and marl	1:1	1:1.5	1:1.75
Transported soils (colluvium, taluvium, river terrace gravels)	Compact/dense	1:1	1:1.25	1:1.5
	Uncompact/loose	1:1.5	1:1.75	1:2
Cohesive soil	Completely weathered argillaceous rocks (mudstone, shale etc.)	1:1 – 1:2		NA

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Any cut slope where failure would result in large rehabilitation costs or would threaten traffic and public safety should be designed using more rigorous techniques. Situations that warrant more in-depth analysis include:

- deep cuts;
- cuts with complex geological structure (especially if weak zones are present);
- cuts where high groundwater or seepage pressures are likely;
- cuts involving soils with low strength;
- cuts in landslide debris; and
- cuts in formations susceptible to landslides.

A further consideration is the effects of blasting on rock mass strength and slope stability. It is not uncommon to find that road maintenance engineers are required to manage road side slopes that have been significantly affected (damaged) by excessive blasting, as illustrated in Figure 5-7. In this case, the rock is strong, but the blast damage, combined with the naturally-occurring adverse jointing in the rock, is particularly hazardous to traffic. Pre-split blasting should be considered in such cases.

Slope profiles

For deep excavations in rock (perhaps deeper than 5 m - 6 m), it is common to bench the cut slope. Table 5-3 lists some of the advantages and disadvantages of benched profiles.

Table 5-3: Advantages and disadvantages of benched cut slopes

Advantages	Disadvantages
• Benches slow down the rate of surface runoff, and therefore reduce surface erosion. • Benches permit the construction of mid-slope longitudinal drains much more easily, and these can form part of an overall slope drainage system. • Where excavation is to be undertaken in softer materials, such as weathered rock, benching can help prevent long erosion furrows from developing by interrupting and controlling the flow of surface runoff. • Shallow failures are usually limited to one bench at a time. • Shallow failures are usually contained on the bench below and are thus often prevented from reaching the road. • Benches offer advantages in terms of access for drilling equipment and excavation plant. • Benches permit access to the slope face for maintenance purposes.	• The cut faces in a benched slope profile will be steeper than a continuous slope cut to the same overall angle. This may encourage localized failures to occur in soft materials and may create conditions of instability in adversely-jointed rock that might otherwise not occur. • Conversely, if the risers of a benched cut slope are cut to the same slope as a continuous cut, the overall height of cut will usually be greater. • Vegetation is less easy to establish on a benched slope profile to the same overall angle (i.e., where steeper risers are required between benches). • Defective bench drainage systems due to erosion or blockage can lead to uncontrolled rainfall runoff and concentrated erosion that ultimately leads to slope failure. • Benches are nearly always inadequately maintained on low-cost roads as they are not of primary concern to road maintenance crews. This can quickly lead to drainage failures.

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The maximum bench height and the minimum bench width should normally be 6 m and 2 m, respectively, although narrower benches (minimum 1 m width) are sometimes used. The advantages of a benched rock slope include the ability to control drainage on the slope (an inward gradient of 3-5% is usually provided) and to confine small failures to individual benches, thus reducing the quantities of debris that reach road level. In hard rock effective drainage can be achieved via an excavated channel at the back of each step.

In permeable and erodible rock, a lined channel (mortared masonry or concrete) is required. The runoff will need to be conveyed to the road level via chutes and cascades. Any debris that blocks these drains or any deformation or cracking to the drains themselves can result in the uncontrolled runoff, seepage, erosion and localized slope failure.

If benches are used, they must be regularly inspected and maintained. However, they are not recommended for very weak rocks, colluvium and other debris slopes. Cut slopes in residual soils can be benched if they are provided with sufficient drainage to prevent the softening of the soil. Outward-sloping benches are not recommended in residual soils as they will tend to concentrate runoff on the cut face below, encouraging erosion. 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 minimize excess spoil in hilly terrain.
- 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 5-4.

Table 5-4: Recommended embankment slope gradients

Fill materials	**Embankment side-slope (V:H) for various heights**			
	< 5 m	**5 -10 m**	**10-15 m**	**15-20 m**
Well graded sand, gravels, sandy or silty gravels	1:1.5 – 1:1.8		1:1.8 – 1:2.0	
Poorly graded sand	1:1.8 – 1:2.0			
Weathered rock spoil	1:1.5 – 1:1.8			1:1.8 – 1:2.0
Sandy soils, hard clayey soil and hard clay	1:1.5 – 1:1.8		1:1.8 – 1:2.0	--
Soft clayey soils (not recommended)	1:1.8 – 1:2.0		-	--

Source: Tanzania Low Volume Roads Manual (2016)

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 realignment to avoid soft soil areas should also be considered. Where these approaches are not feasible, a detailed geotechnical analysis will be required. In most areas, pre-loading over a period of time may be necessary.

Cut-fill cross sections

Cut-fill cross sections are a combination of excavation into the 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 failure unless adequate design and construction precautions are adopted, as shown in Figure 5-8.

Before constructing the fill formation, it is necessary to remove all vegetation and organic material. The formation should be benched or stepped in order to provide a key and 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 mass beneath the fill are dipping parallel to the ground slope, or where the groundwater table is at or very close to the surface.

All placed fill must be constructed with materials that conform to the specification and be adequately compacted. Wherever possible, water should be prevented from seeping into the fill slope, as this may give rise to instability or internal erosion of the materials.

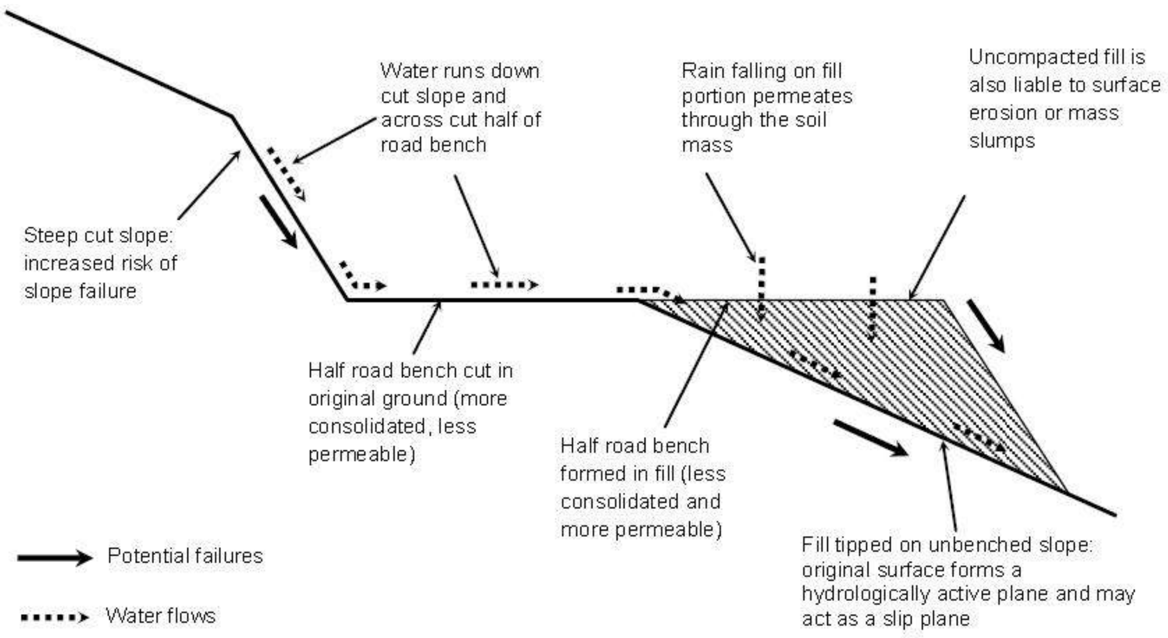

Figure 5-8: Cut-fill situations

Key requirements for achieving an adequate design are:

- Suitable cut slope excavation, as shown in Figure 5-8.
- 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.

5.4 Roadside Slope Stabilization

5.4.1 General

Roadside slope stabilization is the practice of stabilizing slopes adjacent to roads. There are a very large number of road slope stabilization techniques used internationally, ranging from allowing native grass to re-establish on a disturbed slope to building a more complex engineered wall. The treatment measure depends on the affected area, cost and feasibility. This requires a toolbox approach that considers the level of effectiveness and acceptability of the treatment. Site conditions and constraints can vary greatly, and a “one-size-fits-all” approach is unlikely to work. Instead, the right tools have to be selected for the specific project in light of its unique erosion and slope stabilization problems. In high-risk situations, where a landslide may endanger lives, a professional landslide expert should always be consulted. Thus, recourse to specialist texts on these subjects is required (see bibliography), and this section provides only broad guidance on the topic of road side slope stabilization.

5.4.2 Slope Failure Types and Responses

Figure 5-9 shows a schematic model of the common types of slope failure that affect mountain roads. By far the most frequent are Failure Type 1 involving the failure of cut slopes. These can quickly work their way upslope so that the natural slope above the cutting also becomes unstable, creating Failure Type 3. Fill slope failures, shown in Figure 5-9 as Failure Type 2, are also fairly common, and usually result where fill has been placed too steeply and/or where water has been allowed to ingress the slope, causing it to fail. In addition, toe erosion by streams and rivers can remove support to the adjacent valley side slopes, causing them to fail. These can, in turn, remove support to the road, as illustrated in Failure Type 4. Finally, whole hillsides can be unstable, causing the entire road to fail. All of these failure types are prevalent in Afghanistan, with the worst case, Failure Type 5, illustrated in Figure 5-9.

Table 5-6 outlines some typical slope management responses to these five principal failure types. As far as Failure Type 4 and especially Type 5 are concerned, these should be avoided through judicious route selection, which is addressed in *Volume 2 – Geometric Design, Chapter 3 –Route Selection and Fundamental Design Considerations.* However, as illustrated in Figure 5-9, these problems often do not emerge until after construction and road maintenance and rehabilitation are required to deal with them as they occur along the existing road network

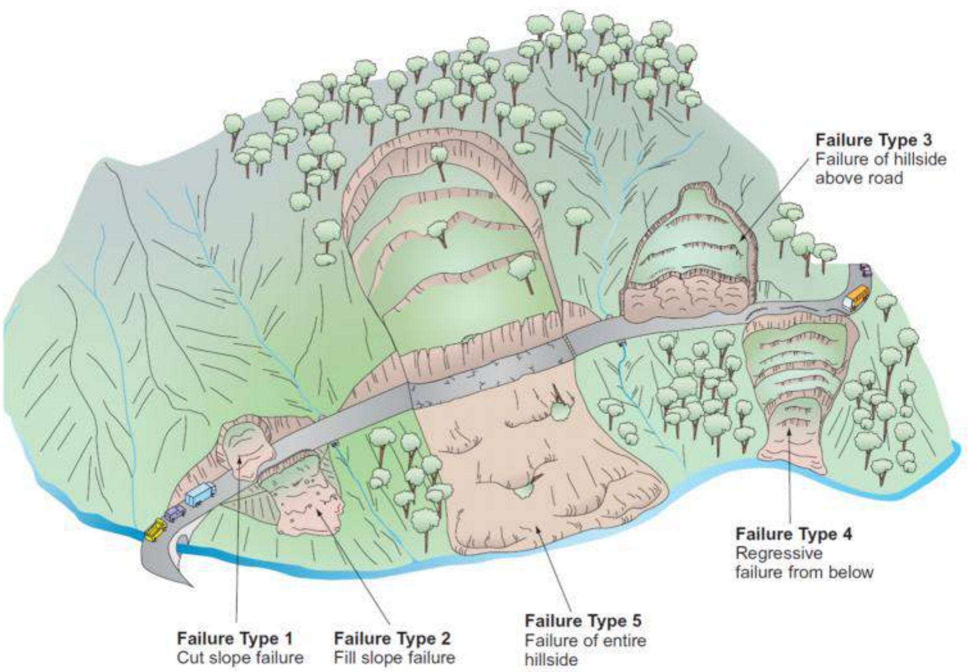

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Figure 5-9: Schematic model of soil slope failure types

Table 5-5: Engineering management options to accommodate the slope failure types.

Failure type	Engineering Management				
	Avoid	Remove	Stabilise	Protect	Accept
1	These failures are often triggered as a result of slope excavation and therefore avoidance during route selection is usually not an option.	Removal of slipped debris is an option in the case of the smaller failures if the remaining slope is stable and can be protected against erosion.	Can be achieved usually through earthworks, drainage and retaining structures.	Catchwalls or fences may be provided to protect road from falling debris.	Small failures onto the road can be accepted if source removal, stabilisation or protection is too difficult or costly compared to the damage inflicted.
2	Shift road into hillside to avoid unstable fill slope below. However, this may initiate type 1 and 3 failures.	Not usually practicable where large fill slopes are involved. Also, either partial or complete removal will result in loss of road width.	This is usually achieved through excavation and recompaction, improved drainage or by the use of retaining structures founded beneath failure surfaces.	Construction of road edge and road fill retaining walls founded beneath failure surfaces may isolate the road from the failing fill slope.	Ongoing movements that cause progressive loss of all or part of the road cannot usually be accepted without loss of road function.
3	These failures are often caused by slope excavation. Therefore, avoidance during route selection is usually not an option. In the worst cases, where landslides frequently cause road blockage, realignment might become cost-effective in the longer term, if a suitable alternative exists.	Not usually practicable given large volumes, access difficulties and uncertainties over the stability of the remaining slope.	May not be practicable or economically feasible to achieve stabilisation in the case of the larger slope failures, though improvements can be achieved through earthworks, drainage and retaining structures.	Catchwalls or fences may be provided to protect road from rock fall debris, but these are unlikely to be appropriate for soil slope failures.	It is usually only feasible to accept ongoing movements if these are slow and can be accommodated by maintenance.
4	Avoid through alignment selection (new roads) or realignment (existing roads) if a suitable alternative exists. Roads are often shifted into the hillside to avoid developing problems below. However, this may initiate type 1 and 3 failures.		If the slope failure is local to the road, then stabilisation by retaining structures and drainage may be possible, though unlikely at low cost.	Construction of road edge and road fill retaining walls founded beneath failure surfaces may isolate the road from the slope failure below.	Ongoing movements that result in the progressive loss of all or part of the road cannot usually be accepted without loss of road function.
5	Avoid through alignment selection (new roads) or realignment (existing roads) if a suitable alternative exists.		Stabilisation of large landslides is usually beyond the scope of LVRs.	Road cannot usually be protected against ground movements.	It is usually only feasible to accept ongoing movements if these are slow and can be accommodated by road surface and drainage repairs.

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5.4.3 Slopes Above the Road

Soil slopes

For existing roads, the three right-hand columns of Table 5-6 apply (stabilization, protection or accept). Stabilization techniques comprise earthworks, drainage and slope support, usually using free-standing retaining walls. For LVRRs, the priority is to find low-cost solutions, maximizing the use of earthworks and surface drainage as well as the use of locally-available materials (Figure 5-10).

Figure 5-10: Use of locally available materials to support the road edge

If a cut slope or natural hillside above the road fails, there are several measures that can be taken. If access and land availability permit, the debris can be removed, and the slope cut back to a reduced angle, thus increasing its stability. If this is not possible, drainage measures can be introduced to reduce the water table, although drainage measures on their own may be insufficient. If this is the case, retaining walls and revetment walls will need to be considered. These can be designed to either retain the soil mass or to provide protection against erosion and shallow failure. It is important to know the depth and mechanisms of instability before designing walls to support or protect a failed or eroding slope. It is normal practice to combine retaining structures with slope drainage if groundwater or surface soil saturation is one of the principal causes of slope instability.

Observations on the use of drainage and retaining walls to stabilize slopes are given in the following sections. Note in Table 5-6 that the option to accept the hazard, i.e., to resort to debris clearance alone, should only be taken when the cost of stabilization or protection to traffic is prohibitive compared with the risk posed by the hazard itself. For example, a slow-moving landslide on a LVRR may be easily manageable by refilling and regrading compared to the cost of attempting to stabilize it. The incurred costs, in terms of additional vehicle operating costs and travel time, are minimal on most LVRRs.

Drainage as a slope stabilization measure

Although drainage can be used to help stabilize both soil and rock slopes, its application is normally greatest on soil slopes along LVRRs. Several surface drainage systems can be considered on earthworks slopes and natural slopes, depending on slope geometry, materials and potential failure mechanisms. Some of these are illustrated schematically in Figure 5-11. Some typical details for surface drains are provided in Figure 5-12. Discussion on the application and performance of these different drainage systems is provided in Table 5-6. Note that

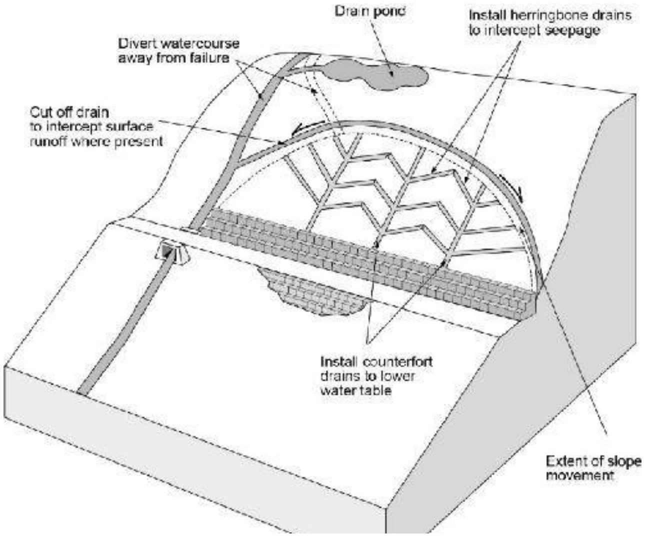

Figure 5-11: Typical drainage measures to improve slope stability

deeper drainage systems involving counterfort drains and horizontal drains are rarely used on LVRRs due to their high cost and the need for ground investigation for their effective design. In many circumstances, it might prove cost-effective to consider off-road drainage systems to reduce the amount of water that reaches the road. Ordinarily, this would be a proactive rather than a reactive measure, but off-road drainage might prove beneficial in controlling individual landslides. The use of earth channels to divert slope runoff into neighboring streams would be a good example of such measures. These measures are discussed further in *Chapter 9 – Drainage and Erosion Control.*

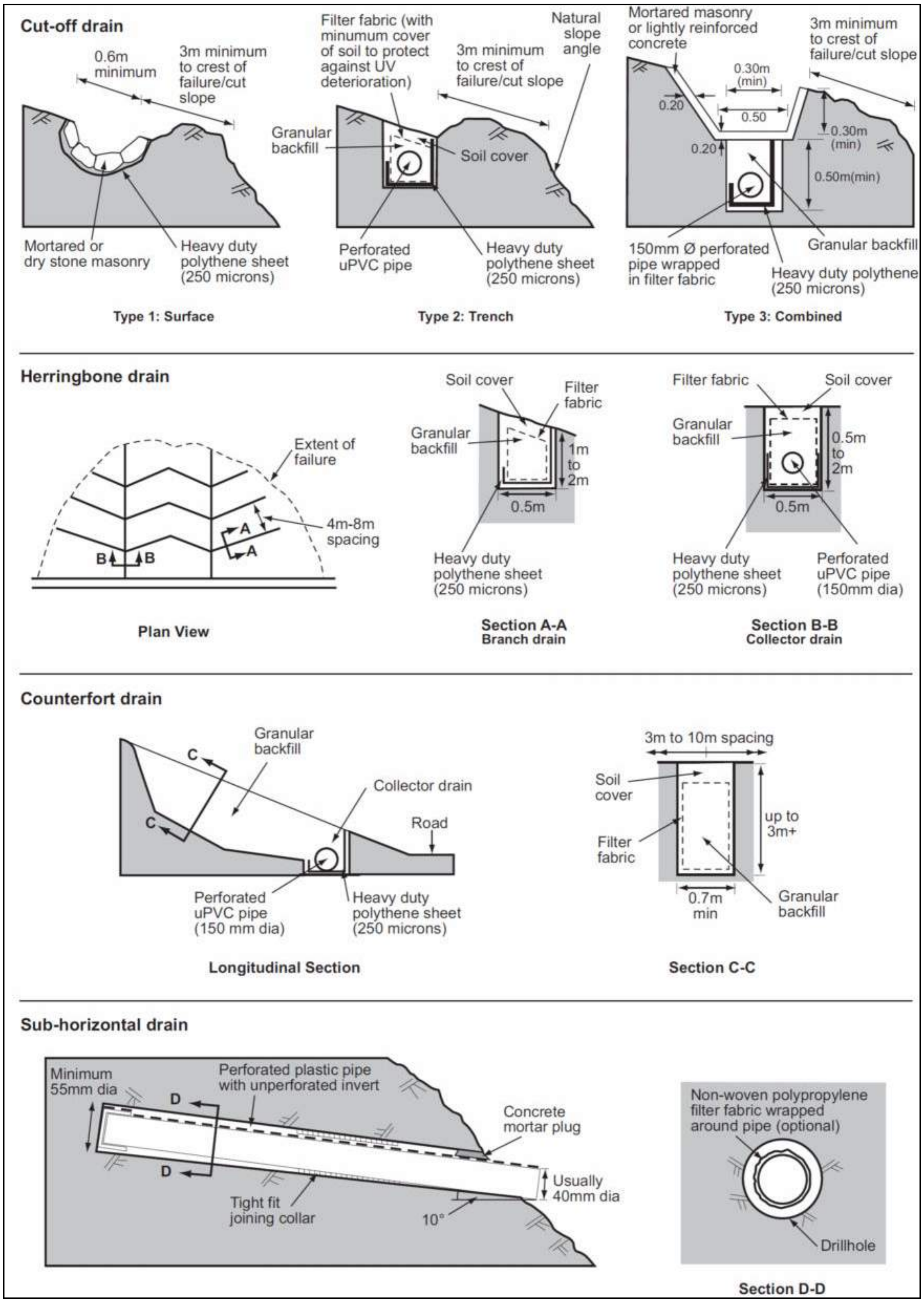

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Figure 5-12: Typical slope drainage details

Table 5-6: Common techniques of drainage to improve slope stability

Function	Type	Advantage	Limitation
Interception of surface runoff	Unlined cut-off drain (open ditch)	Cheap, can in some instances prevent ingress of surface runoff into landslide masses	Prone to leakage and erosion; may act as incipient tension crack beyond slope crest; requires frequent inspection for damage/blockage; access may be difficult for maintenance
	Lined cut-off drain (type 1 on Figure 5-12	As above, though less prone to erosion and leakage	Requires frequent inspection for damage/blockage; access may be difficult for maintenance. Concentrates flow and erosion if ruptured by ground movement
	Sub-soil drain (type 2 on Figure 5-12)	Usually not prone to erosion and leakage and can tolerate some ground movements while continuing to function	May become clogged with silt; can be surcharged during large surface flows; may encourage water to enter the slope if not constructed properly or where excessive ground movements create 'sags' in vertical alignment or tears in the polythene; may be difficult to access for maintenance
	Lined cut-off drain with subsoil drain (type 3 on Figure 5-12)	Combines surface and sub-surface drainage. Can accommodate large surface flows	Requires frequent inspection for damage/blockage; access may be difficult for maintenance
	Bench drain on cut slopes	Collects and discharges surface runoff from one bench or berm to the next. Reduces the tendency for large quantities of water to pond and seep into the slope material	Will crack and dislocate following any ground movements, may become blocked by falling debris or silt if not properly maintained
	Berm drain on fill slopes		
Reduction of shallow sub-surface water and drainage of seepages	Herringbone drain	Depending on depth, usually able to intercept water up to 1.5m below slope face; can be used to drain seepage areas; can accommodate some slope movement; can be used to help stabilize shallow slope failures up to 2m deep	May have very limited effect on overall stability of deep-seated failures. May create shallow instability during construction, hence preference to minimize branch length
	Counterfort or trench drain	Able to intercept water up to 3m depth below slope face; can act as a 'buttress' if base is below slip surface	Usually needs to be machine dug; difficult to construct in boulder material
Interception of deep water table	Drilled sub-horizontal drain	Only feasible method of intercepting groundwater at depth	Relatively high cost; drilling equipment required; may not always be successful

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Rock slopes

Preferably, unstable rock slopes should be stabilized using earthworks approaches. These include the removal of unstable overhangs, scaling of loose rock blocks, and the cutting back of the unstable slope to a shallower angle. However, this is not always possible due to topographic constraints (i.e., the slope of the rockfall or rock slide source is too steep) and practicality (i.e., access may be difficult and unsafe). In these situations, there may be little option but to use expensive retaining structures and anchoring systems or measures to protect the road from falling and falling rock, such as nets and catch walls.

Figure 5-13 shows a range of measures commonly used to stabilize or mitigate rockfall and rock slide hazards along mountain roads. For LVRRs, prescriptive codes 11-16 (right-hand column) may be the only practical options based on cost considerations. Rock shelters and large rock trap structures have been used on sections of the A76 Salang Pass Road, but these are unsuitable for

LVRRs. Instead, if sufficient space exists, large earth mounds can be used to contain debris before it gets to the road. Gabion walls can also be used in such circumstances as they are able to absorb some impact force without failure.

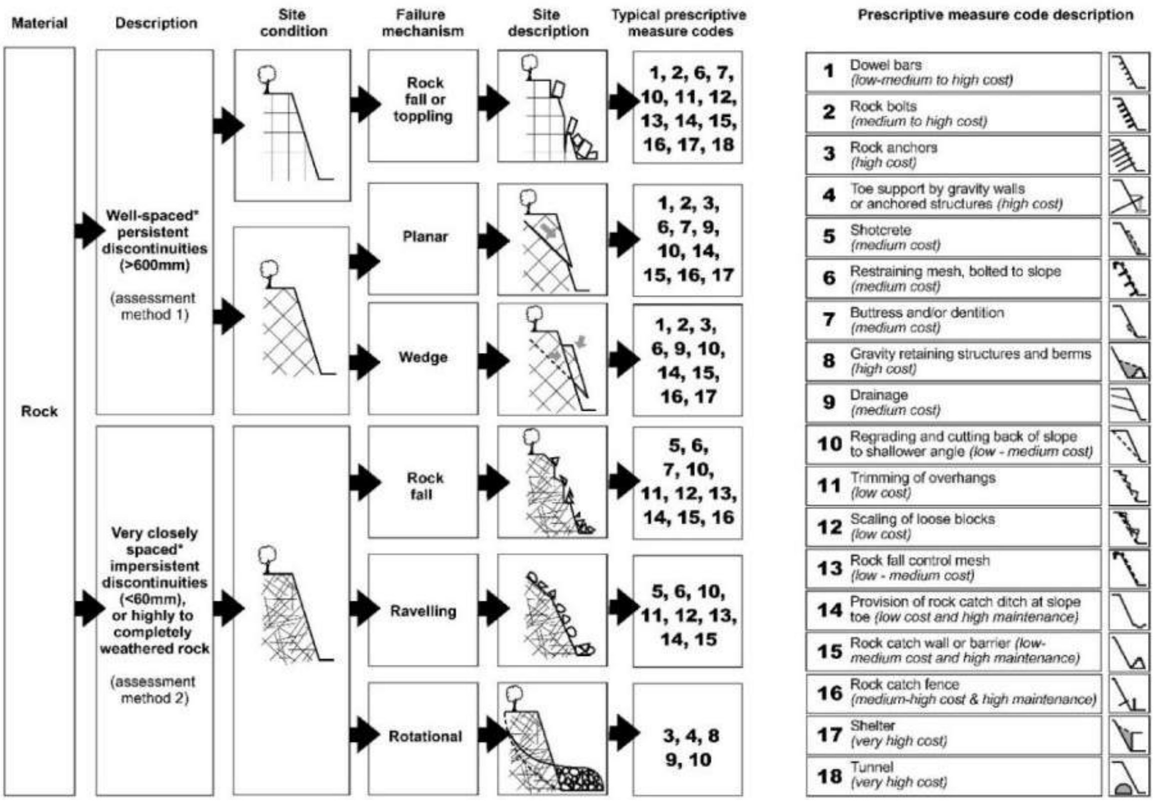

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Figure 5-13: Rock slope materials, failure mechanisms and remedial measures

Some general guidelines are given below. Note that each case will need to be assessed on its own merits, as there will be issues concerning the availability of space, cost and practicality to consider. Further considerations on rock slope stabilization and road protection are given in Table 5-7.

Where the hazard is from falling rock, the following measures should be considered:

- Remove the unstable rock or realign the centerline away from the cut slope to create more space for rocks to fall into (ditches) without impacting the carriageway.
- In combination with the above, or alone, construct a rock trap wall. If space permits, gabions are the best option as they absorb impact without failure. If space does not permit, a reinforced concrete rock trap wall may be a suitable alternative, if affordable.
- Hang wire netting over the slope, anchored into the natural ground above the cutting to control the fall of rock debris. This can be expensive but may be justified if there is a significant risk to traffic.
- Protect the rock face using shotcrete (wire mesh and concrete) bolted into the rock face. Again, this may be expensive, but locally justifiable.

Where the hazard is from sliding rock, the following measures should be considered:

- Construct a retaining wall, sufficient to retain the rock mass. The practicality of this will depend on the volume of the slide materials, the space available to build the wall and cost considerations
- Use rock dowels or bolts to effectively reinforce the rock mass by anchoring surface rock layers into underlying stable rock layers. Again, issues of cost and risk will need to be considered and optimized, and the design must be based on a geotechnical assessment.

In some circumstances, and where the risk posed to traffic is high, it may be easier and cheaper to carry out local realignment. If this is not possible, expert guidance should be sought before deciding on the best course of action. Until a decision is made, it may be necessary to close the road during and immediately after heavy rain and prevent any road widening through additional excavation into the hillside.

Table 5-7: Rock slope stabilization and road protection techniques

Requirement	Technique	Where?	Limitation
Stabilization - Reinforcement	Rock bolting	Any potentially unstable block that can be bolted and tensioned back to stable material	High cost; installation using specialist equipment; long term corrosion/creep problems
	Dowels	Any potentially unstable block that can be kept in place by passive dowel	Use usually restricted to blocks 1-2m thick
	Tied-back walls	Where multiple rock bolting is required to provide load spread	Same as for rock bolting
	Shotcrete	Closely fractured or degradable rock face	Specialist equipment required
	Buttresses	Cavity on rock face	Potential access problems
	Drainage	Any rock face where water pressures in fissures create instability	Drilling equipment necessary for drain holes
Stabilization – Removal	Regrading	Instability at crest of rock face	Potential access problems
	Trimming	Overhangs	Controlled blasting techniques required
	Scaling	Loose rock on surface	Labor-intensive; potential access and safety problems
Protection	Catch Ditch	Base of slope where space permits	Shape of ditch dependent on height and slope of rock face
	Mesh	Loose/weak rock on surface	Will not retain major blocks; good anchorage required at top of face
	Barrier	Base of slope where space permits	Needs to be robust to halt movement onto road
	Shelter	At base of high unstable face where other measures not feasible	High cost
	Tunnel	If relocation only solution	High cost

Source: Hunt et al (2008)

5.4.4 Slopes Below the Road

Table 5-8 provides some slope stabilization and protection options appropriate for fill and valley slopes below the road. As with slopes above the road, each situation will require careful investigation prior to deciding on the most appropriate course of action. It is usually the case that fill slope failures on steeply-sloping ground can only be reliably remedied through the construction of retaining walls with foundation levels below the zone of movement, and preferably on rock, or on in situ soils where allowable bearing pressures permit. In cases where active horizontal earth pressures are high, such retaining structures may need to be anchored to deeper competent rock.

Table 5-8: Stabilization options for slope problems below the road

Instability	Stabilization options
Failures in fill slope (Failure Type 2 on Fig 5-9)	• 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; • Use bioengineering to protect the fill slope from erosion • On steep ground, a road retaining wall may be the only option, properly founded; • Ensure road side drainage is controlled
Failure of the valley slope below the road (Failure Type 4 on Fig 5-9)	• Realign road into the hillside to allow the slope below the be regraded and to provide an increased margin of safety. This may require stabilization and protection to the excavation above and may therefore not be a cost-effective option in the long term; • Depending on the depth of slope failure a retaining wall, founded in suitable strata beneath the failure surface, would provide support to the road. If the depth of failure is much deeper than 5m then the use of retaining walls to support the road may become impracticable and costly on LVRs; • May need extensive scour protections and river training works if toe erosion is significant.
Failure of the entire valley side (Failure Type 5 on Fig 5-9)	• If rates of movement are fast, or access is lost, consider realigning the road if a suitable alternative exists; • If rates of movement are slow then it may be cost-effective to maintain access by the use of fill whenever the road surface subsides; • Reduce the extent of road runoff entering the unstable mass by diverting drainage either side; • Slope stabilization measures might be considered, but these may require extensive scour protection if toe erosion is significant, major earthworks, deep drainage and retaining structures. Such measures would be prohibitively expensive on most LVRs

Source: Modified from Hunt et al, 2008. Reproduced with permission of the Geological Society, London

5.4.5 Retaining Walls

General

The use of retaining walls on LVRRs is relatively uncommon, with preference usually being given to less costly earthworks-based designs. Detailed horizontal and vertical alignments can also sometimes be adjusted to avoid or minimize the need for retaining walls. However, the use of retaining walls is sometimes unavoidable, for example, to a) support road fill on steep slopes, b) retain slipped material, or c) support cut slopes that have to be cut more steeply than the exposed material can stand unsupported. In rarer cases, retaining walls are used to support the road where it crosses unstable ground. Likely areas for the use of retaining walls are:

a) When the slope below is steeper than about 33°. This is the approximate angle of repose of uncompacted granular material, which would thus slide down any steeper slope under gravity.

b) When the road is to be constructed between rock promontories or spurs beneath the road and where there is a requirement to have a robust foundation

c) Where there is a landslide below the road, and the wall can be constructed beneath the zone of movement, thereby separating the road from the slope movement below. Any fill used in this situation would simply slide and add load to the landslide in any case, or

d) Where the slopes below the road might be subject to river scour, i.e., where a fill slope would be washed away and where a sufficiently founded retaining wall would remain intact.

Revetments
Revetments are sometimes used to protect embankment slopes from erosion due to surface runoff or river scour when embankments are constructed in riverine environments. Revetments are commonly constructed in gabion, concrete, or geosynthetic materials. Although revetments can provide support against shallow instability within the embankment, they should not be used to facilitate the steepening of the embankment slope, as this will fail unless supported by a retaining wall.

Dry stone walls
The least expensive form of retaining wall, but often the most effective, is one constructed of interlocking rocks or boulders (Figure 5-14). Such types of walls are fairly easy to construct if the necessary rock is locally available. The stability of the wall is provided by the interlocking of the stone blocks. The advantage of dry stone walls is that they are free-draining and are built using local materials. They are often used in combination with low-cost bioengineering measures (see *Chapter 9 – Drainage and Erosion Control,* Sections 9-5 and 9-6) to help control water and retain small volumes of soil. Their durability depends on the soundness of the stone used and the quality of construction. Any differential movement of the foundation will lead to loss of strength and possible failure.

Figure 5-14: Typical retaining and revetment wall constructed of interlocking boulders

Dry stone revetments, 1 m - 2 m in height (sometimes referred to as 'breast' walls), can provide important protection to the toe of a slope from shallow movement in surface soils and erosion. Rock rip rap can also be used as a means of toe support if placed as an embankment against the toe of the slope.

Mortared masonry walls
Mortared masonry walls are brittle and cannot tolerate differential settlement. They are suited to uneven founding levels but perform equally well on a flat foundation. They are used as cut slope and fill slope retaining walls but also as a form of revetment or 'breast' wall in a slope protection capacity. Mortared masonry walls tend to be more expensive than gabion walls, although there are exceptions to this.

If the wall foundation is stepped along its length, movement joints should be provided at each change in wall height so that any settlement does not cause uncontrolled cracking in the wall. Mortared masonry walls require the construction of weep holes to prevent the build-up of water pressure behind the wall. Weep holes should be 75 mm diameter and placed at 1.5 m centers with a slope of at least 2% towards the front of the wall. A geotextile filter should be placed at the back of the weep holes to permit free drainage of water but prevent migration of the backfill.

Masonry walls constructed on rock are usually provided with a concrete leveling foundation. Masonry walls constructed onto anything other than rock are usually provided with a reinforced concrete foundation. However, there are many cases where masonry has been constructed directly onto the subgrade. If the subgrade is sufficiently strong and of consistent bearing capacity without the potential for seepage erosion, then such walls can perform as required. However, there are many cases where walls have failed because of poor subgrade preparation and inadequate wall foundation. Generally, masonry walls should not be founded on clay soils. On weak subgrades, their cross-section should be widened to distribute bearing pressures over a wider wall footprint.

Gabion walls
Gabion walls are built from gabion baskets wired together. A gabion basket is made of steel wire mesh in the shape of a rectangular box. The wire should be galvanized, and sometimes PVC-coated for greater durability and protection from sunlight. The baskets usually have a double twisted, appropriately sized,

hexagonal mesh, which allows the gabion wall to deform slightly without the box breaking or losing its strength. The manufacturer's specifications for mesh size, galvanizing, wire diameter, panel frames, basket connectors and the twisted connections (usually minimum three half turns) need to be adhered to. Stone fill should be dressed block-shaped with a dimension at least twice the size of the mesh size and should be of sound rock. Rounded river stone should be avoided wherever possible.

Gabion walls are cost-effective because they employ mainly locally available rock and low-skilled labor. Gabion baskets are commonly used to construct walls up to 6 m in height, although greater heights have been constructed. Gabion walls are usually the preferred option where the foundation conditions are variable and where clay soils form the foundation material. In such situations, the base of the wall should be made as wide as possible to spread the load and reduce bearing pressures. Good drainage and free-draining backfill are essential, along with the use of filter fabric to prevent the migration of fines. Where slope movements and differential settlement are anticipated, gabion walls are likely to perform better than other wall types because they can accommodate some deformation without structural failure. Care should be taken to locate the base of the wall on a good foundation beneath the zone of ground movement. Spraying of Gabion walls with concrete can assist in preserving the gabion mesh and increasing their supporting capacity.

Additional guidance on the advantages of different wall types, including the more expensive forms, such as concrete, piled and soil reinforced walls, are given in Table 5-9.

Table 5-9: Features of retaining structures

System	Function	Type	Advantages	Limitations
Externally stabilized	Gravity walls	Masonry	Technique well known	Unable to accommodate movement without distress
		Mass Concrete	Simple to construct	Large quantities of concrete required
		Reinforce Concrete - Cantilever	Generally occupies less width	Requires reinforced concrete construction; good foundations; generally uneconomic above 8m height
		Reinforced Concrete - Counterfort	As above	As above, but can be constructed to greater heights
		Gabion	Technique well-known; can accommodate limited movement without distress; permeable	Moderate durability; not recommended as retaining walls below and immediately adjacent to paved road surface due to flexibility
		Crib	Attractive, environmentally-friendly appearance	Possible problems of durability if timber cribs are used
	In-situ walls	Sheet pile	Occupies very limited space, no temporary excavation works required	High cost; requires specialist installation equipment; impermeability may create problems
		Slurry walls		
		Bored-in-place piles		High cost; requires specialist installation equipment
Internally stabilized	Reinforced soil	Strips and grids	Can accommodate limited movement without distress; easy to construct	Occupies large space behind wall face
		Soil nailing	Used extensively when steepening existing cut slopes	Requires specialist installation equipment

Source: Hunt et al (2008)

The properties of the concrete (and grout) need to be designed for each individual situation, depending on their use as a gravity structure (lower strength) or retaining structure (higher strength) or even requiring reinforced concrete in some cases.

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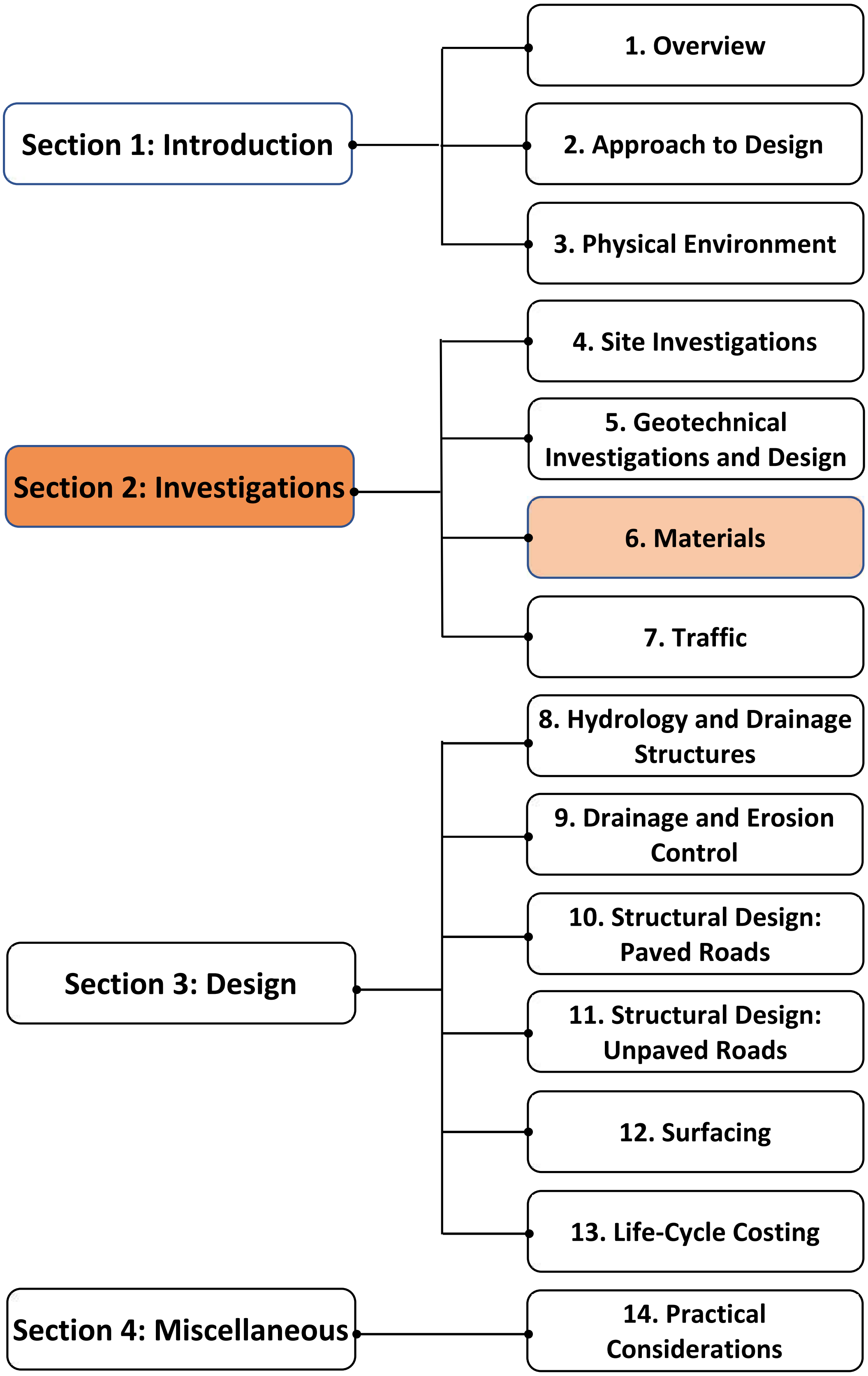
Low Volume Rural Roads Guideline and Standards
Volume 1 – Pavement Design



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6.1 Introduction

6.1.1 General

Naturally occurring soils, gravel soil mixtures and gravels occur extensively in many parts of Afghanistan. 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 behavior but also of the traffic loading, the physical environment and their interactions.

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

6.1.2 Purpose and Scope

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

The chapter covers the range of construction materials required for all classes of LVRRs. 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-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.

6.2 Material Types

6.2.1 General

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

6.2.2 Materials in Afghanistan

Afghanistan 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 is characterized by Precambrian metamorphic and igneous rocks mostly associated with the Hindu Kush and mostly sedimentary rocks surrounding this. Arid weathering (disintegration mostly) and erosion have produced a great variety of soils in the plain areas which, together with the "inland" drainage and widespread occurrence of high carbonate rocks, have led to significant calcification of the soils. Paleo-climates have also led to local deposits of bauxitic (aluminum-rich) and lateritic (iron-rich) soils.

6.2.3 Weathered and Residual Materials

Chemical weathering of rocks causes the alteration of the minerals in the rocks (except quartz, which is relatively resistant) to form different minerals, mostly clays, and changes the hard rock to a residual material that could be used as natural gravel for road construction. However, the current arid conditions in Afghanistan lead to minimal chemical degradation and significant physical deterioration of the rocks caused by temperature changes and cyclical wetting and drying and freezing and thawing. Both of these processes can produce materials suitable for road construction as materials of different quality and with different properties are required for different applications in roads. These materials are of particular interest for use in LVRRs as they can be easily worked without requiring expensive blasting or heavy equipment for ripping. However, the quality and durability of borrow materials and crushed stones can be greatly affected by the weathering or alteration processes.

Weathering effects generally decrease with depth, although zones of differential weathering can occur in many outcrops. Of equal importance, however, is the presence of thick deposits of transported materials in the foothills of the mountains, which can make good construction gravels, although they often require some single-stage crushing.

The properties of the final residual material will, however, depend on the mineralogy of the parent rock. With the diverse range of rock types present in Afghanistan, a wide range of natural gravels can thus be expected. These can vary from non-plastic quarzitic gravels through to relatively inert limestones and dolomites to highly plastic and expansive swelling clays. Only some of these are suitable as selected road construction materials, as discussed in *Section 6.4 – Materials Prospecting.*

6.2.4 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 suitable for the construction of roads. Such transported materials are found in the foothills of the mountains, in river valleys, in desert and inland deposition areas, and can occur almost everywhere except on steep slopes. Many of these can make suitable construction gravels, although they often require some single-stage crushing.

Figure 6-1: Typical deposit of transported, sandy material

Transported soils of a uniform type are often localized, 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.

6.2.5 Pedogenic Materials

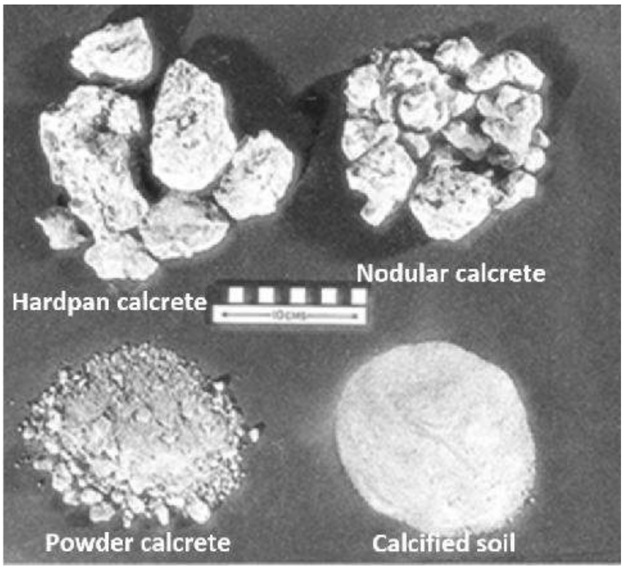
Pedogenic materials, collectively known as pedocretes or pedogenic soils, is a unique type of soil in which the existing material is "fully or partially cemented" by certain minerals. Typical cementing materials include iron and aluminum oxides, calcium carbonate and, to a lesser extent, silica. These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

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

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

Calcrete

Calcrete is the dominant pedocrete in Afghanistan. This material is formed in place either by cementation or replacement – sometimes both – of pre-existing soils, usually by calcium carbonate and, to a lesser extent, by magnesium carbonate precipitated from the soil or groundwater. This results in the original material being transformed into a new one - calcrete - comprising varying quantities of CaCO3 whose properties vary from an almost pure, very hard, massive limestone, in which there is hardly any trace of the host material, to a very loose material consisting largely of the host material (Figure 6-2).



Lionjanga et al, 1987

Figure 6-2: Typical types of calcrete

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

Laterite

Bauxite and laterite are two other pedocretes that are found in Afghanistan. Laterite, in particular, is generally a suitable construction material for all layers up to the base course, if the material properties are tested correctly and understood. A large number of factors control how a particular type of laterite is developed, and the material tends to exhibit both vertical and lateral variability within an often deep and irregular weathering profile. Two typical types of laterite are illustrated in Figures 6-3 and 6-4 below.

Figure 6-3: Nodular laterite stockpile

Figure 6-4: Hardpan laterite in quarry

The behavior of lateritic materials in pavement structures depends mainly on their iron and aluminum oxide (sesquioxide) contents, particle size characteristics, the nature and strength of the gravel-sized particles, the degree of compaction as well as traffic and environmental conditions. The most important requirements for a laterite pavement to perform well are that the material is well graded with a high content of hard particles and 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 requirements for selection and use of lateritic gravels for pavement layers are different from those typically specified for other natural gravels, and this needs to be taken into account during their testing. Conventional testing using oven drying, for instance, can have a major effect on the test results. Other aspects, such as mixing times for the Atterberg limits, can also affect the results. For these reasons, assessing the qualities of laterites as road building materials in terms of conventional specifications must be undertaken carefully.

6.2.6 Summary of Typical Material Properties

Table 6-1 summarizes the typical properties of the residual gravels obtained from the chemical weathering of various rock types. This should only be seen as a guide, as many local conditions (e.g., perched water tables, good drainage conditions) could affect the actual individual properties.

Table 6-1: Typical properties of residual materials derived from various rock types

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

6.3 The Use of Locally Available Materials

6.3.1 General

Making maximum use of naturally occurring, unprocessed materials is a central pillar of the LVRR 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 favor of more expensive crushed rock or other processed materials. However, recent research has shown quite clearly that so-called "non-standard" materials can often be used successfully and cost-effectively in LVRR 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 utilizing 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.

6.3.2 Optimum Utilization of Local Materials

The potential benefits of innovatively using natural and alternative (by-products from industry or mining) can be large.

Alternative materials have already been processed to varying degrees (e.g., crushed, burnt, screened, etc.), and their use will thus have considerable energy savings. In addition, many of these are stored in vast quantities on valuable land and can create aesthetic, dust, pollution, or other environmental problems. Products that have been burnt often have pozzolanic properties, and their use can enhance the properties of other materials. As "waste" materials, they can also often be obtained at low costs as a way of removing them from the producer's property and thus reducing the cost of their storage.

By adopting materials specifications that are suited to LVRRs, more of the naturally-occurring materials can be used for road construction. By so doing, higher-quality materials that are required for the construction of HVRs can be reserved for that purpose. This may also reduce the cost of construction materials and will result in the potential to build more roads.

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 6.8.2, 6.8.3 and 6.8.4:

- Modifying the material (e.g., mechanical or chemical stabilization).
- 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.

Many material specifications 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 LVRRs, it is the strength (or ideally stiffness) that is critical and provided that this is mobilized (and retained under the expected prevailing conditions in the short and long term), there is no reason why the material should not perform satisfactorily.

Figure 6-5: Coarse river gravels can be crushed for concrete or surfacing aggregate

6.3.3 Beneficial Characteristics of Local Materials

Despite the many differences that exist among local materials, some dominant characteristics affect pavement performance that should be appreciated in order to design and construct LVRRs using such materials with confidence. These characteristics depend on whether the materials are used in an unbound or bound state, which affects how 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 (stabilization) forces.

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 the 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 6-6.

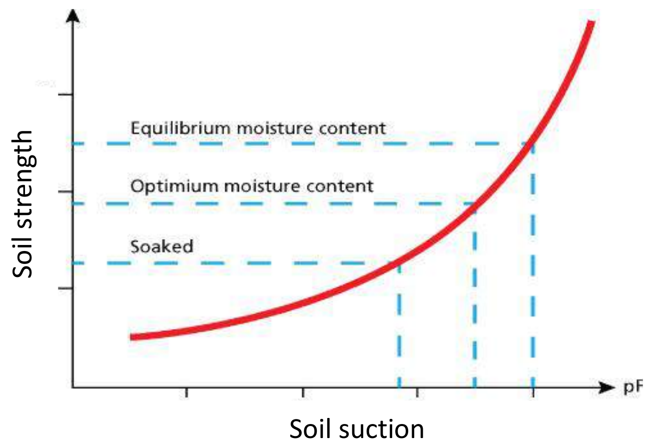


Figure 6-6: Illustrative soil strength – soil suction relationship

Table 6-2 summarizes the typical relative characteristics of unbound and bound materials that critically affect how they can be incorporated into a pavement in relation to their properties and the prevailing conditions of traffic, climate, economics and risk.

Table 6-2: Pavement material types and characteristics

Parameter	Pavement Type			
	Unbound			Bound
	Unprocessed	Moderately Processed	Highly processed	Very highly processed
Material types	Category 1 As-dug gravel	Category 2 Screened gravel	Category 3 Crushed rock	Category 4 Stabilized gravel
Variability	High	Decreases		Low
Plasticity Modulus	High	Decreases		Low
Development of shear strength	Cohesion and suction.	Cohesion, suction & some particle interlock.	Particle interlock	Particle interlock & chemical bonding
Susceptibility to moisture	High	Decreases		Low
Design philosophy	Material strength maintained only in a dry state	Selection criteria reduce the volume of moisture sensitive, soft and poorly graded gravels		Material strength maintained even in a wetter state
Appropriate use	Low traffic loading in a very dry environment	Traffic loading increases, environment becomes wetter		High traffic loading in wetter environments
Cost	Low	Increases	High	High
Maintenance requirement	High	Decreases		Low
	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 the development of shear resistance. The typical deficiency in hard, durable particles prevents reliance on inter-particle friction. Thus, even modest levels of moisture, typically approaching 60% saturation, may be enough to reduce confining forces sufficiently to cause distress and failure.
- **Category 2** materials have a moderate dependency on all forms of shear resistance; friction, suction forces and cohesion. Because these materials have rather limited strength potential, concentrations of moisture, typically 60-80% saturation, may be enough to reduce the strength contribution from suction or cohesion sufficiently to cause distress and failure. This would occur at moisture contents lower than those necessary to generate pore pressures.
- **Category 3** materials have an only minor dependency on suction and cohesion forces but have a much greater reliance on internal friction, which is maximized when the aggregate is hard, durable and well graded. Very high levels of saturation, typically 80-100%, 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 minimizing the entry of moisture into a LVRR pavement to ensure that it operates as much as possible at an unsaturated moisture content. The beneficial effect of doing so is illustrated in Table 6-3, which shows the variation of a material's strength (CBR or DN) with moisture content.

Table 6-3: 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

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.

6.3.4 Materials Selection

General selection criteria

The criteria used for selecting road materials for incorporation in a LVRR need to take account of the following factors:

- Knowledge of the fundamental engineering properties of the material.
- The required performance 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 several attributes that must be satisfied concerning their selection for LVRRs. In addition to the physical properties governing performance, other aspects also need to be satisfied, as presented in Table 6-4:

Table 6-4: 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 the 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 generally 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 the 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 LVRR.

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.

Soil Permeability

Permeability is defined as the property of soil, which permits the flow of water through it. Soil is highly pervious when water can flow through it easily, e.g., in gravels. In contrast, in impervious soil, permeability is very low, and water cannot easily flow through it, e.g., in clays.

Soil permeability is affected by several factors, including the following:

- Particle size
- Impurities in the water
- Void ratio
- Degree of saturation
- Adsorbed water
- Entrapped air
- Organic material

Table 6-5 shows the coefficient of permeability for a range of soil types.

Table 6-5: Coefficient of permeability for various soil types

<table>
<tr><th colspan="13">Coefficient of permeability k (cm/sec)</th></tr>
<tr><td></td><td>10^{2}</td><td>10^{1}</td><td>1.0</td><td>10^{-1}</td><td>10^{-2}</td><td>10^{-3}</td><td>10^{-4}</td><td>10^{-5}</td><td>10^{-6}</td><td>10^{-7}</td><td>10^{-8}</td><td>10^{-9}</td></tr>
<tr><td rowspan="2">Type of soil</td><td colspan="2" rowspan="2">Clean gravel</td><td colspan="3" rowspan="2">Clean sands, clean sand and gravel mixtures</td><td colspan="4">Very fine sands, organic and inorganic slits, mixtures of sand, silt and clay, glacial till, stratified clay deposits</td><td colspan="3" rowspan="2">Impervious soils, e.g. homogeneous clays below zone of weathering</td></tr>
<tr><td colspan="4">Fine grained soils, normally ‘impervious’ but modified by effect of vegetation and weathering (in situ) or dry compaction (fill)</td></tr>
</table>

Source: Standards Association of Australia, 1978

Knowledge of soil permeability is important as this factor affects the rate of ingress of water through the surfacing and road shoulders as well as water entry into the pavement. These aspects of LVRR design are considered in *Chapter 9 – Drainage and Erosion Control.*

6.4 Materials Prospecting

6.4.1 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 the interpretation of maps and other information is a crucial aspect of the process.

6.4.2 Stages in Prospecting

The various stages in the materials prospecting process are shown in the flow chart in Figure 6-7.

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 agency, local authorities, or by road design consultants and contractors. These, and any other materials-related reports, should be consulted to assist with material location.

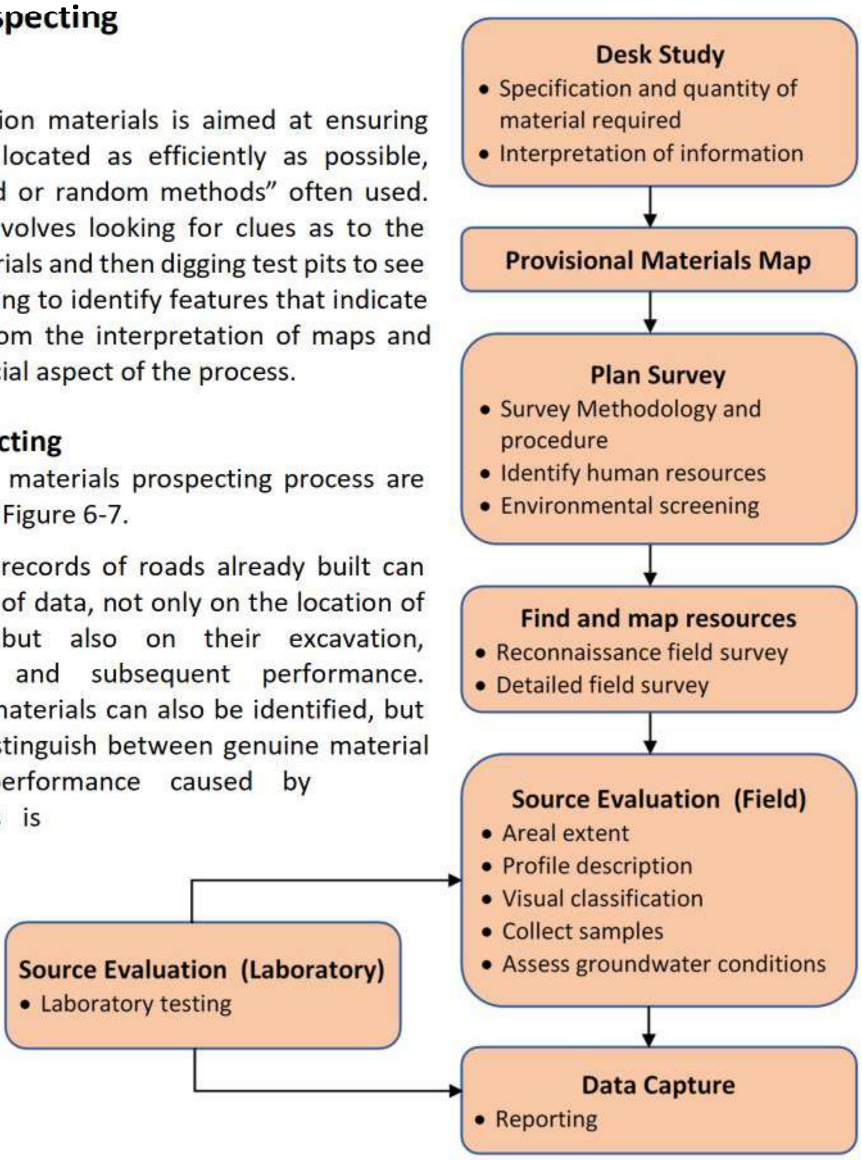


Figure 6-7: Flowchart of prospecting procedure

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 programs. The field investigation program may be undertaken by specialized 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 utilized. 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 identification, for instance, of indigenous halophytic plants of Afghanistan will indicate soils with high salt contents, which should be avoided in road construction.

The geomorphology is also a strong indicator of the potential quality and quantity of materials. Figure 6-8 shows variable thick and thin residual soils in cuttings and transported gravels in a stream. 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.

Figure 6-8: Variable deposits of residual soils visible in cut-slopes

6.4.3 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 providing the required quality and quantity of material, the borrow pits and quarries should ideally be:

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

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

6.5 Materials Sampling and Testing

6.5.1 General

A high proportion of the LVRR 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 the successful construction of LVRRs is, therefore, that a materials sampling and testing program is designed and implemented as appropriate to the circumstances for each project.

6.5.2 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 6-6 gives a guide to the required sample sizes for the most common soil tests applicable to LVRRs. The relevant test protocol should be consulted before the collection of samples from the field.

Table 6-6: General guide for sample size requirements for common soil tests

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 (per mould)	6.00	6.00	12.00
	Compaction (Heavy 4.5 kg / Light 2.5 kg, CBR mould)		80.00	
	Compaction (Heavy 4.5 kg / Light 2.5 kg, 1 ltr mould)		25.00	
	Vibrating Hammer		80.00	
Aggregate strength	Aggregate Crushing Value (ACV)		2.00	
	Aggregate Impact Value (AIV)		2.00	
	Los Angeles Abrasion (LAA)		5.00-10.00	

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

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

Table 6-7: 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 6-7 per DN or CBR test.

6.5.3 Testing Program

The quality of the testing program 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 materials or 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 LVRR.

The laboratory testing program should be part of a comprehensive program to provide all of the information needed to adequately define the nature, use and volumes available of construction materials. The early phases of the program will generally concentrate on gaining clues to unusual soil behavior, 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 programs will be based on relatively simple classification tests that can be done quite quickly. More sophisticated tests will only be used if necessary.

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

6.5.4 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 South African National Standards (SANS) test protocol applies. However, for the DCP-CBR method, the BS test protocol must be applied since the material specifications are based on the various BS tests.

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 that are 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. In all other cases, wet sieving should be mandatory.
- CBRs determined by AASHTO methods can be up to about 20% lower than for the same material tested by the BS method.

It should be noted that the DCP CBR method is based on BS test methods. The DCP DN design method is not affected by the soil test method as long as the proportion of material retained on the 20 mm screen is handled in the standard method.

6.5.5 Standard Tests

The testing program 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 these Guidelines except for the method for the determination of material strength.

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.

Compaction: 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 6-9.

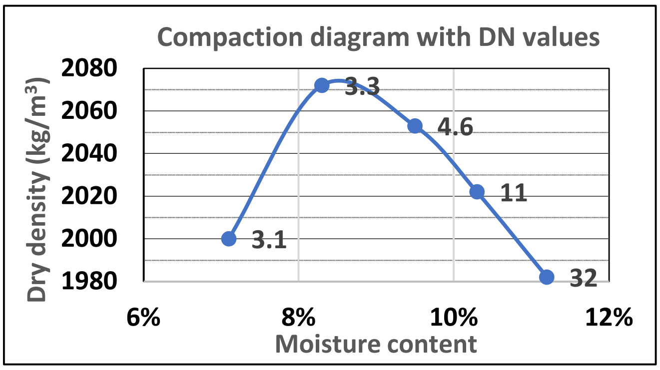


Figure 6-9: Dry density curve with DN values (mm/blow) determined on each mold

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

- For the DCP-DN method, the material strength is determined through the Laboratory DN test, as described in Chapter 10 – Structural Design: Paved Roads, Section 10.2.5, 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).

6.5.6 Specialized Tests

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

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

The use of residual soils derived from a basic igneous rock is potentially problematic if the limits stated above are exceeded. The risk of using the material in Afghanistan, however, is quite low due to the arid nature and low temperatures of the country but can be minimized 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 (including wind-blown sands) exhibit high load-bearing capacities and can be successfully used as a pavement material for LVRs. Experience has shown that desert-type sands which fulfil specific Soil Constant requirements ($5 < BLS_{0.075}/\text{ø}_{mean} < 10$ 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.*

6.6 Construction Material Requirements

6.6.1 General

The different types of road construction materials required are:

- Common embankment fill.
- Imported (selected) subgrade.
- Subbase 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 minimize the 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 capitalized on, and as little additional structure as possible should be constructed. Other aspects such as shape, drainage and the repair of localized 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.

6.6.2 Common Embankment Fill

In general, the location and selection of fill materials for LVRRs pose 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 the 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 1.0 m, 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 (rockfill layers at their base, with or without geosynthetic layers), but the material quality above this would not need to be any higher than for the low fills described previously.

6.6.3 Imported (Selected) Subgrade

Where subgrade soils are weak or problematic, the importation of higher quality selected subgrade (capping) 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, the importation 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 9 – Drainage and Erosion Control*. Where material improvement is necessary or unavoidable, mechanical and chemical stabilization 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, i.e., using the DCP CBR method, the subgrade strength for design is assigned to one of five strength classes reflecting the sensitivity of thickness design to subgrade strength. However, the DCP DN method will take the in-situ subgrade conditions into account directly in the pavement design.

It would generally 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.

6.6.4 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 calcareous, lateritic and quartzitic gravels, river gravels and other transported gravels, or granular residual materials resulting from weathering or disintegration of rocks can be used successfully as a 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. The main sources of sub-base and base materials are rocky hillsides, high steep hills, and riverbanks.

The minimum thickness of a deposit usually considered workable for excavation for materials for subbase and base is of the order of 1.0 m. 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-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.

6.6.5 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 summarized in Table 6-8.

Table 6-8: 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 raveling 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, also, to the formation of sodium, potassium and aluminum 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 following 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 that contain 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 (PSV) results than fresh, un-weathered rocks. There is, therefore, an inverse relationship between polishing resistance and abrasion resistance.

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

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

6.6.7 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 labor-intensive in their construction. Problems due to poor construction or insufficient support can be easily maintained with only the localized 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.

6.6.8 Aggregates for Structural Concrete

Concrete aggregate comprises two components – a coarse fraction and a fine fraction. The fine aggregate is usually naturally occurring sand, with particles up to about 2 mm in size. The coarse aggregate can be natural gravel or, more commonly, crushed or hand-broken stone with a range of sizes from about 5 mm to 20 mm (or sometimes larger). In areas without hard stone resources, but 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.

6.6.9 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. The basic requirements for filter materials are shown in Table 6-9.

Table 6-9: 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 a 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	

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 6-8.

6.6.10 Special Materials

Naturally occurring materials

It is often necessary to produce larger rock particles to fill gabion baskets, for rockfill 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 6-10.

Table 6-10: 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 stabilizers, 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 stabilizers to low volume roads requires careful consideration and design. Their use is seldom cost-effective and should be avoided as far as possible unless significant benefits can be generated during independent laboratory and field testing.

6.7 Material Improvement and Processing

6.7.1 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 stabilization (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.

6.7.2 Reducing Oversize

There are various measures for reducing oversize, including the use of labor, mobile crushers, grid rollers or rock crushers. For very low percentages of oversize material (< 5 or 6%), 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 knapping**: This is quite feasible, especially on relatively small, labor-based projects where material can either be hand-screened and/or broken down to various sizes and stockpiled in advance of construction.

- **Mobile crusher:** 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. Modern mobile plants include impact and cone crushers, which are highly effective in producing well-shaped crushed products.
- **Modified dozer**: Equipped wheels designed to breakdown oversize aggregates, as shown in Figure 6-10. It is essential that the material is re-mixed after breaking down oversize to separate broken aggregates before compaction commences.
- **Rock crusher:** The "Rockbuster" is a patented plant item, which is basically a tractor-towed hammer mill, as shown in Figure 6-11. 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 6-10: Dozer equipped to break down oversize aggregates

Figure 6-11: Rock buster

6.7.3 Mechanical stabilization

Where materials with a suitable grading and/or plasticity are unavailable locally, granular mechanical stabilization 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 stabilization 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 6-8, has been developed for determining the optimal mix ratio for blending two or more materials to meet the required grading specification for unpaved roads but could be applied to improve the grading of any material.

The optimum grading for a gravel wearing course, based on the limits of the Grading Coefficient corresponding to Zones E and F in *Chapter 11 – Structural Design: Unpaved Roads, Figure 11-8*, is demarcated by the shaded area.

The points A and B in Figure 6-12 demarcate the grading of two typical soils, the gradings of which are summarized in Table 6-11.

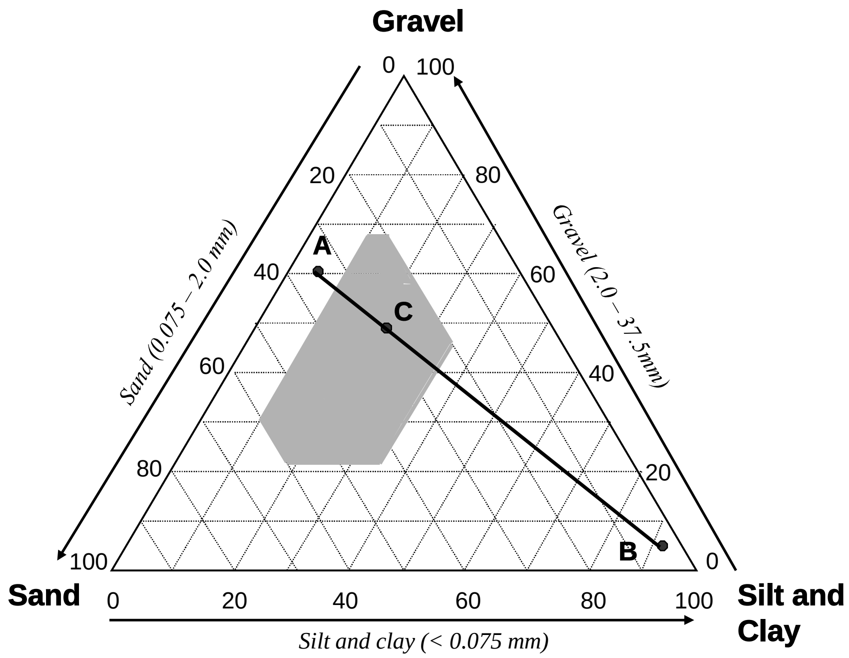


Figure 6-12: Use of ternary diagram for determining proportions during material blending

Table 6-11: Gradings of materials used for blending in Figure 6-12

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 % 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 % 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 5-14).
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 center 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., four loads of material A will be blended with one load of material B). This can be equated to a number of truckloads and related 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 (140-365 or 140-240, 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.

Loose wind-blown sands can present a particular challenge for mechanical stabilization as some of these deposits are basically single-sized, with all the material passing the 2 mm sieve and retained on the 0.425 mm sieve. In these situations, finding naturally-occurring materials for blending to produce an appropriate final grading for use as a wearing course can be nearly impossible. These materials can be effectively compacted as a subgrade and as pavement layers, as discussed in section 5.5.6. Imported materials are required for wearing course.

6.7.4 Chemical Stabilization

Where naturally occurring materials that meet the specified requirements cannot be located within an economic haul distance from the project site, and where mechanical stabilization methods cannot be applied to address the deficiencies, chemical stabilization should be considered. This section provides only general guidance on alternative methods of chemical stabilization; more detailed information may be found in the texts cited in the bibliography.

Stabilization of gravel with cement or lime is more commonly applied to road bases. The suitability of materials for stabilization with either cement or lime can be determined from Table 6-12.

Table 6-12: Guide to selection of stabilization methods

Form of stabilization	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
Mechanical						
Cement						
Lime						
Bitumen						
PP (Plasticity Product) = PI x % passing 75 µm						
Key:		Usually suitable		Doubtful		Usually not suitable

Source: Austroads, 1988

The most common type of cement used for stabilization is CEM-II, i.e. cement with a mixture of Ordinary Portland cement (OPC) and additives such as fly ash, slag or limestone, up to a maximum additive content of 35%. When using lime, hydrated lime is preferred.

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

As an alternative to cement or lime stabilization, bitumen emulsion stabilization could be considered. However, this is a costly option that is hardly applicable to LVRRs.

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7.1 Introduction

7.1.1 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 summarized 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 LVRRs, 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 (carriageway and shoulders). The geometric design standards (refer to *Volume 2* of this Guideline) cater adequately for the traffic volumes expected on LVRRs and are modified based on the characteristics of the traffic using the road, such as different traffic mixes including numbers large vehicles, motorcycles, non-motorized 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 (baseline) and future traffic volumes is required to undertake the design of the road in an appropriate manner.

7.1.2 Purpose and Scope

The purpose of this chapter is to provide details for counting current traffic, predicting future traffic and obtaining traffic loading (axle loading) data for structural design.

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 LVRRs. Simplified methods of accomplishing this are also described.

7.2 Surveys

7.2.1 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

7.2.2 Traffic Surveys

A classified traffic count is one of the most important sources of data for both geometric and pavement structural design, as well as for planning purposes in terms of evaluating economic benefits derived from the construction of LVRRs. 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-motorized vehicles and pedestrians.

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

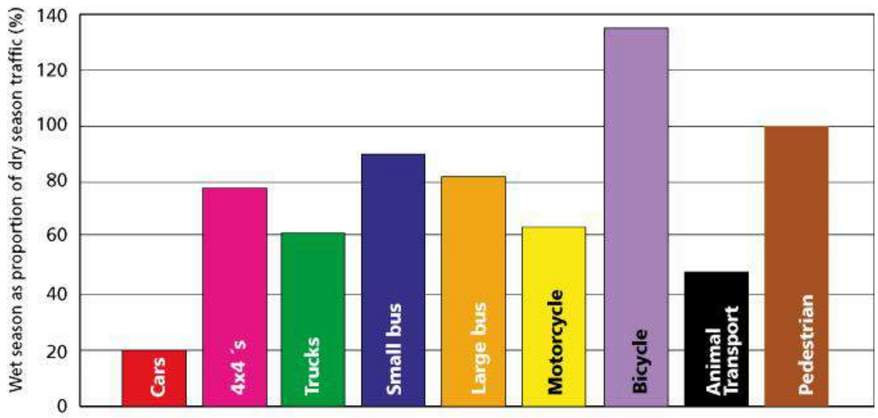
- Manual Traffic Survey
- Automatic Traffic Surveys
- Moving Observer Methods

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 which is primarily to obtain an estimate of the Annual Average Daily Traffic (AADT) using the road, disaggregated by vehicle type. Establishing reliable traffic data is notoriously difficult, especially where the roads serve a predominantly developmental or social function, and when the traffic level is low. Thus, although the AADT is more accurate, since it is based on a continuous count for at least 365 days, for LVRRs it is sufficient to rely on the less accurate Average Daily Traffic (ADT) obtained from seasonal, classified traffic counts, as discussed below.

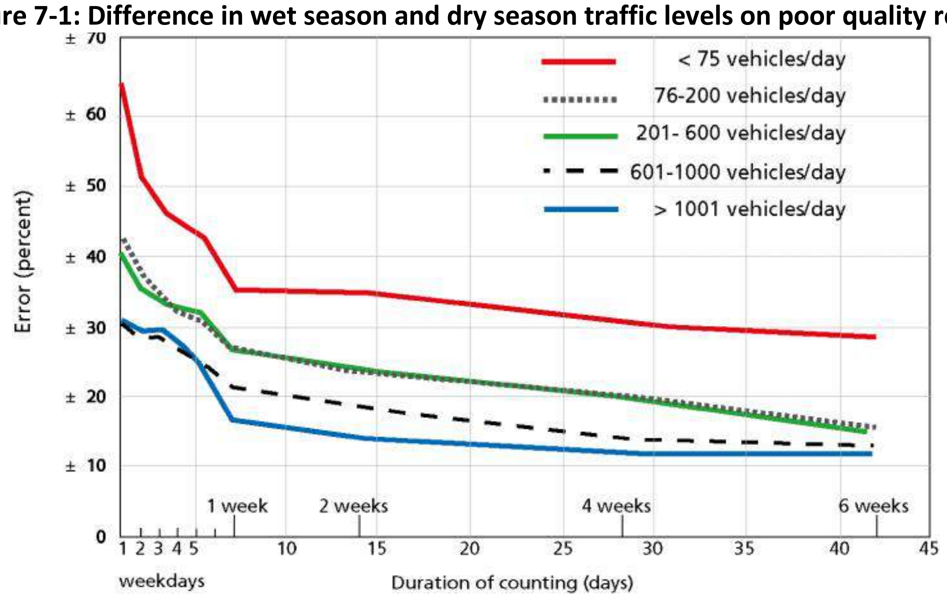
Usually, motorized traffic volumes will decrease in the wet season to, typically, 80% of their dry season level. However, on poor quality roads, this difference can be even more marked, and the wet season traffic can decrease to as much as 35% of dry season traffic levels, as shown in Figure 7-1. For the purposes of this Guideline, 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 7-2, short duration traffic counts in low traffic situations can lead to large errors in traffic estimation.



Source: Parsley and Ellis, 2003

Figure 7-1: Difference in wet season and dry season traffic levels on poor quality roads



Source: Howe 1972

Figure 7-2: Possible errors in ADT estimates from random counts of varying duration

Reducing errors in estimating traffic for LVRRs.

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, 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 market places 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 7-3.

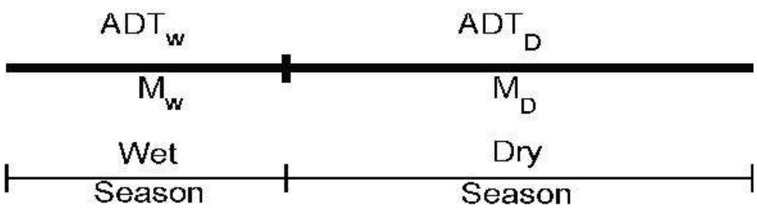


Figure 7-3: Basis for traffic count adjustment in relation to seasonal characteristics

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

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

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

Vehicle Classification

Table 7-1 shows an internationally-based vehicle classification system that can be used for compiling the results of the traffic survey described above.

Table 7-1: Vehicle classification system

Class	Type	Axles	Description	Use
A	Car	2	Passenger cars and taxis	Capacity analysis for geometric design
B	Pick-up/4-wheel drive	2	Pick-up, minibus, Land Rovers, Land Cruisers	
C	Small bus	2	≤ 25 seats	Capacity analysis for geometric design and axle load analysis for pavement design
D	Large bus/coach	2	> 25 seats	
E	Light Goods Vehicle	2	≤ 3.5 tonnes empty weight	
F	Medium Goods Vehicle (MGV)	2	>3.5 tonnes empty weight	
G	Heavy Goods Vehicle (HGV)	3/4	>3.5 tonnes empty weight	
H	Very Heavy Goods Vehicle VHGV)	≥4	>3.5 tonnes empty weight	
I	2-axled trailer	2	Trailers towed by MGVs HGVs or VHGVs.	
J	3-axled trailer	3		
K	4-axled trailer	4		
L	Tractor	2		
M	Motorcycles, motorcycle taxis			Capacity analysis for geometric design
N	Bicycles			
O	Other NMT			
P	Pedestrians			

7.2.3 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 7-5.

7.2.4 Axle Load Surveys

Axle load surveys provide critical and essential information that is required for both cost-effective pavement design as well as the preservation of existing roads. The importance of this parameter is highlighted by the well-known "fourth power law" which exponentially relates increases in axle load 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 LVRR 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 the 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 is influenced by 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 Road Development Agency should be followed.

Simplified axle load surveys

Assuming that a full axle load survey is not being carried out, information about the vehicle loading can be obtained by observation during the traffic counting survey. The enumerator merely records, for every heavy vehicle in the heavy vehicle classes, the state of loading (full, partial or empty) and the type of load (heavy or light). The number of VEFs (ESAs per vehicle) can then be estimated based on Table 7-2 and using guidance on types of load in Table 7-3.

Table 7-2 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 7-2: Indicative average vehicle equivalency factors (VEF) 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/4-wheel drive	2		
C	Small bus	2	0.3	0.15
D	Large bus/coach	2	2.4	1.2
E	Light Goods Vehicle	2	1.5	0.75
F	Medium Goods Vehicle (MGV)	2	4	2
G	Heavy Goods Vehicle (HGV)	3	4.5	2.25
H	Very Heavy Goods Vehicle (VHGV)	≥4	7	3.5
I	2-axled trailer + haulage truck	2	8	4
J	3-axled trailer + haulage truck	3	10	5
K	4-axled trailer + haulage truck	4	12	6

The VEFs in Table 7-2 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 MRRD.

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 7-3: Types of loads affecting choice of VEF in Table 6.2

Dense goods for which the average VEFs in Column 1 in Table 7.2 apply	Light goods for which the average VEFs in Column 2 in Table 7.2 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	

7.3 Determination of Design Traffic

7.3.1 General

The procedure for determining the traffic loading for pavement design purposes is summarized in Figure 7-4, and each step is explained thereafter. The traffic analysis for pavement design cannot be separated from the analysis for geometric design since the geometric design requirements, and ultimately the selection of road class and cross-section width will influence the traffic 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.

7.3.2 Procedure

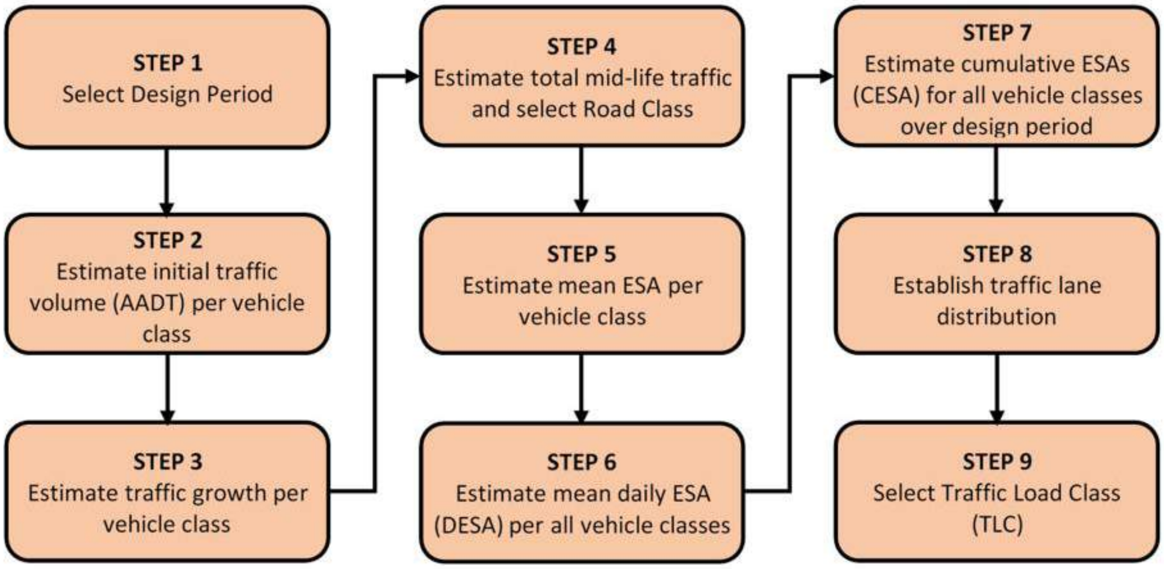


Figure 7-4: Procedure for establishing design traffic class

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. The terminal serviceability is expressed in terms of the International Roughness Index (IRI) or as a Present Serviceability Rating (PSR).

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 7-4 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 7-4: 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 7.2.2, 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 7-2) 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-motorized 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 a forecast of traffic growth in each vehicle class. To forecast such growth, it is first necessary to sort traffic in terms of the following categories (refer to Figure 7-5):

- **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 a judgment of the traffic on nearby roads that could benefit from a shorter or more comfortable route.
- **Generated traffic:** Additional traffic that occurs in response to the new or improved road. This traffic is essentially 'suppressed' traffic that does not currently exist because of the poor state of the existing road. Local historical precedent can sometimes assist in estimating this; otherwise, generated traffic can be assumed to be about 20% of the existing traffic.

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 traffic growth, including the following:

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

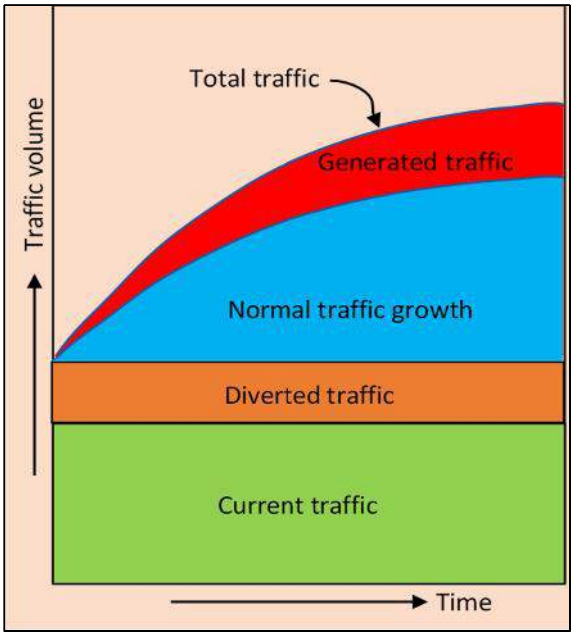


Figure 7-5: Traffic development on an improved road

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 7-6 which shows the multiplier for the AADT in the first year of analysis to obtain the AADT in any other year.

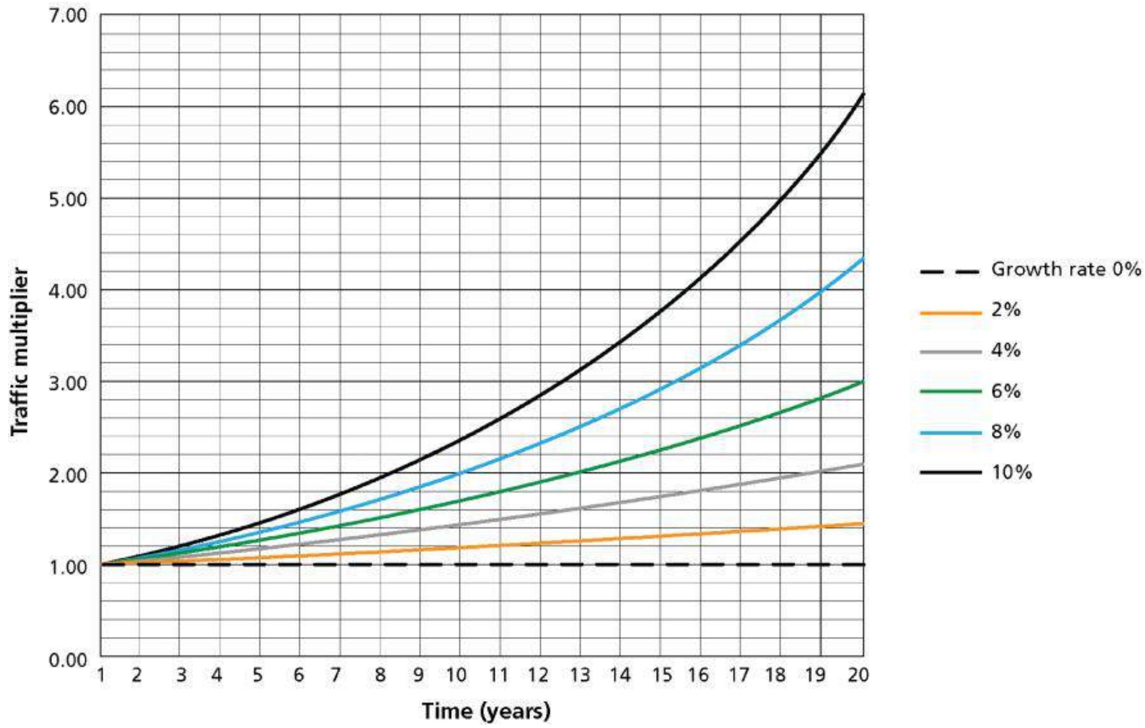


Figure 7-6: Multiplier to obtain AADT in any year for different growth rates

Step 4: Estimate the total mid-life traffic and select Road Class
This is the last step for geometric design purposes (refer to *Volume 2: Chapter 4 – Traffic*). The Road Class will determine the cross-section width and influence the traffic lane distribution, as shown in Table 7-5.

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 LVRRs, in which case reliance will need to be placed on existing information and visual surveys.

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

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

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

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

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

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

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

n = power exponent (typically 4.0 – 4.5 applied by road agencies. Recent research evidence suggests that n = 3.0 – 4.0 may be more appropriate for LVRRs 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 7-2. 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 7-7 shows the multiplier for the CESA in the first year to calculate the CESA after any other number of years up to year 20.

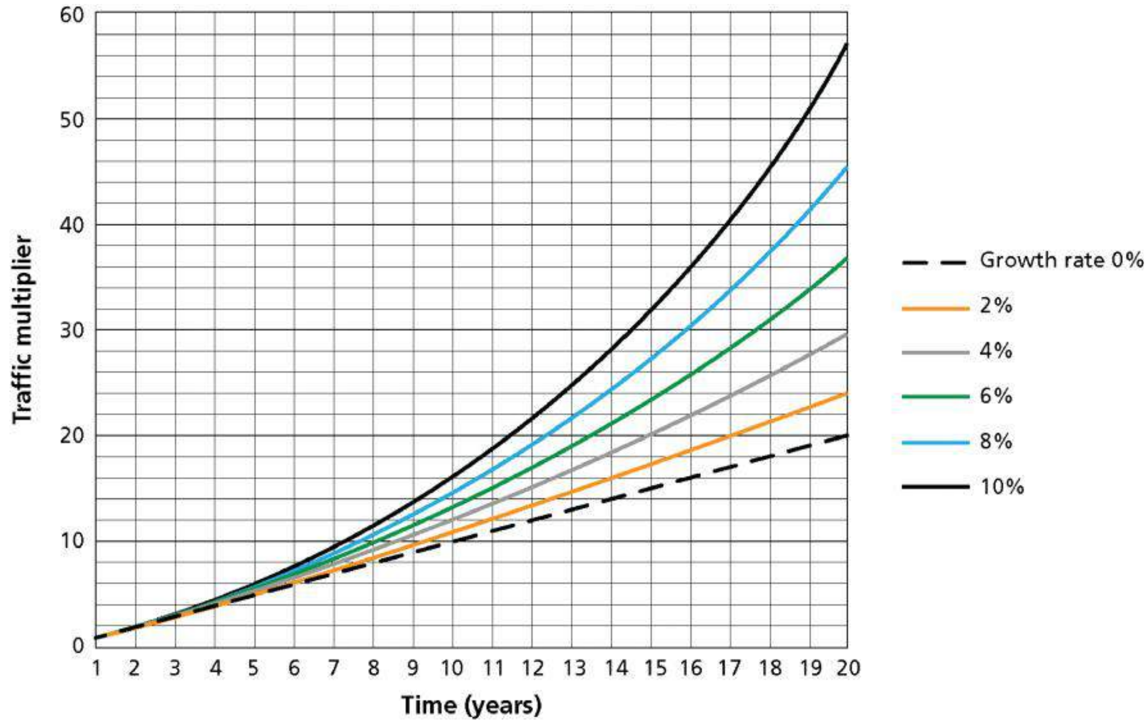


Figure 7-7: Multiplier for the first year CESA to calculate the CESA after any number of years

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 7-5.

Table 7-5: Pavement width adjustment factors for design traffic loading

Cross section	Paved width	Corrected design traffic loading (ESA)	Explanatory notes
Single carriageway.	< 3.5m.	Double the sum of ESAs in both directions.	The driving pattern on this cross-section is very channelized.
	Min. 3m but less than 4.5m.	The sum of ESAs in both directions.	Traffic in both directions uses the same lane, but not all in the same wheel tracks as for the narrower road.
	Min. 4.5m but less than 6m.	80% of the ESAs in both directions.	To allow for overlap in the center section of the road.
	6m or wider.	Total ESAs in the heaviest loaded direction.	Minimal traffic overlap in the center section of the road.
More than one lane in each direction.		90% of the total ESAs in the studied direction.	The majority of vehicles use one lane in each direction.

Note that for a carriageway width of 6.0 m and above, the pavement 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 in Table 7-6. Note that the TLCs are slightly different for the two design methods presented in this Guideline.

Table 7-6: Traffic Load Classes for structural design

DCP-DN Design Method		DCP-CBR Design Method	
Traffic Load Class	**Cumulative traffic load during design life (MESAS)**	**Traffic Load Class**	**Cumulative traffic load during design life (MESAS)**
TLC 1.0	0.7 – 1.0	TLC 1.0	0.5 – 1.0
TLC 0.7	0.3 – 0.7	TLC 0.5	0.3 – 0.5
TLC 0.3	0.1 – 0.3	TLC 0.3	0.1 – 0.3
TLC 0.1	0.01 – 0.1	TLC 0.1	0.01 – 0.1
TLC 0.01	< 0.01	TLC 0.01	< 0.01

7.3.3 Sensitivity Analysis

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

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

If the estimated design traffic loading is close to the upper boundary of the TLC, and the sensitivity analysis indicates that the upper boundary may be exceeded, it may be prudent to assume the next higher TLC and to 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 A: 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 7-7.

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

Traffic/Vehicle Classification	Current Number of units	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 motorized traffic is as shown in Table 7-8.

Table 7-8: Computation of mid-life motorized 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, LVRR3 from Table 5-1 in *Volume 2, Chapter 5 – Cross Section*, 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 7-9.

Table 7-9: Example of calculation of Car Equivalents (CE)

Traffic Classification	Current Number	Car Equivalent 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 the 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

r 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-motorized traffic at the mid-life of the design period can now be computed as shown in Table 7-10.

Table 7-10: Computation of mid-life Car Equivalents of non-motorized traffic

Traffic 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 motorized 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 7-11: 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 total mid-life traffic and select road class

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

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

Table 7-12: 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 6: Estimate Mean Daily ESA for all Vehicle Classes

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

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

Total ESA/day = 15.6 (direction 1)

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

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

Total ESA/day = 9.0 (direction 2)

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

The design CESA can be computed from the following equation:

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

$$= 365 \times (15.6+9.0) \times [(1 + 0.045)^{15} – 1]/0.045$$

$$= 365 \times 24.6 \times [(1.045)^{15} – 1]/0.045$$

$$= 365 \times 24.6 \times [1.935- 1]/0.045$$

$$= 365 \times 24.6 \times 20.78$$

$$= 186{,}583 \text{ ESA}$$

Step 8: Determine traffic load distribution

From Table 7-5, the traffic loading for a design for a 5 m carriageway is 80% of the ESAs in both directions.

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

= 0.15 MESA

Step 9: Select traffic load class

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 7-13: 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 (Example only. See discussion under Step 5, Page 7-9)

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 7-14.

Table 7-14: 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 Front	EFs 1st rear	EFs 2nd rear	VEF	Avg. VEF (East)	Avg. VEF (West)
Bus/Coach	East	2400	4400		0.01	0.08		0.09	0.07	0.05
Bus/Coach	East	1900	3700		0.00	0.04		0.05		
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		0.46	0.29	0.14
2-axle truck	East	3200	6070		0.02	0.31		0.33		
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		0.21		
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	2.39	1.50	1.84
3-axle truck	East	5000	8100	8000	0.14	0.97	0.92	2.04		
3-axle truck	East	4900	8000	7000	0.13	0.92	0.54	1.60		
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	0.48	1.06		
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 LVRRs, has been used for the calculation of the VEF. As shown in Table 7-15, 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 7-15 should be used.

Table 7-15: 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 the type of subbase material".

** The higher values of the range usually refer 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.

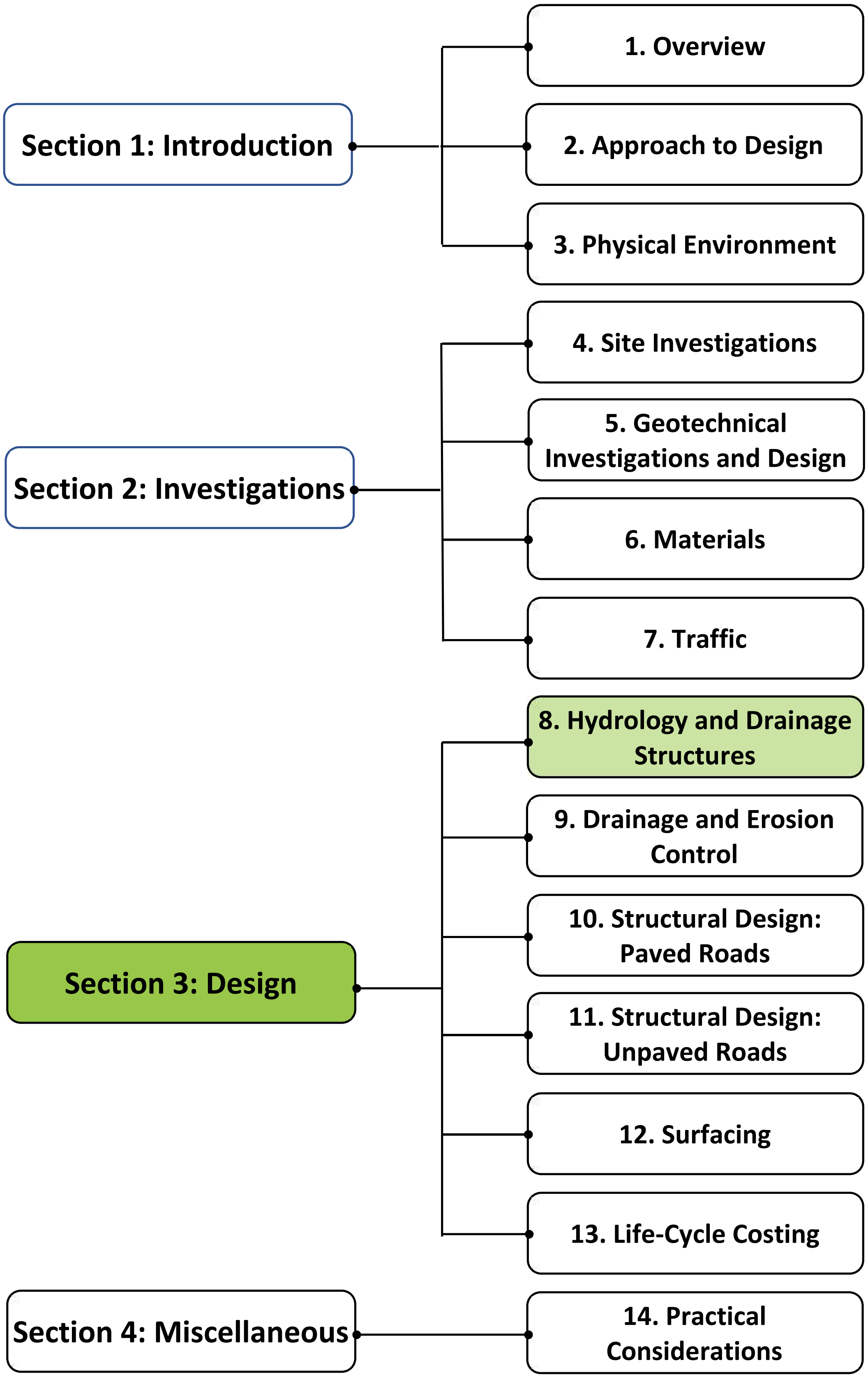
Appendix B: Traffic Count Form

Traffic count form

Road name:				Direction from:		to:			
Date:		Counting period:		Enumerator:		Supervisor:			
	Traffic class	Empty weight No of axles	Two-hourly counts						Totals for counting period
Motorised traffic	Cars								
	Pick-ups/4WD								
	Small Buses	≤ 25 seats							
	Total light vehicles								
	Large Buses	>25 seats							
	LGV	2- axle < 3.5 ton							
	MGV	2-axle > 3.5 ton							
	HGV	3/4-axle > 3.5 ton							
	VHGV	> 4-axle > 3.5 ton							
	Trailers	2-axles							
		3-axles							
		4-axles							
	Tractors								
	Total heavy vehicles								
	Total light & heavy vehicles								
	Motor-cycles								
	Tuc-tucs								
NMT	Bicycles								
	Animal carts								
	Pedestrians								
	Total NMT								

Section 3
Design

Low Volume Rural Roads Guideline and Standards
Volume 1 – Pavement Design



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8.1 Introduction

8.1.1 Background

Hydrology and hydraulic analysis for road drainage design may be defined as the estimation of flood run-off from a catchment for a specified but rare storm and the design of drainage structures of appropriate capacity to pass that design flood discharge. It includes the assessment of risks associated with such rare climatic events. The level of risk that is acceptable depends on the consequences of a failure of the drainage structure. For trunk roads with generally high traffic flows and for costly structures such as bridges, only limited risk can be tolerated. For LVRRs with lower traffic flows and where the consequences of disruptions to traffic and the cost of repair of the drainage structures are less, higher risks can be tolerated. The challenge for the engineer is to choose an appropriate standard of structure and related cost that is commensurate with the class of road and an optimum level of service.

8.1.2 Purpose and Scope

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

The chapter also provides an overview of various drainage structures that are particularly suitable for LVRRs, such as culverts, drifts, vented drifts and small span bridges. More detailed information on the design and construction of such structures may be found in other guidelines, such as ***Small Structures for Rural Roads: A Practical Planning, Design, Construction and Maintenance Guide*** (gTKP, 2010) and ***Building Rural Roads*** (ILO, 2008).

The chapter does not cover aspects of road drainage such as side and miter drains, sub-surface drainage and erosion control measures, which are addressed in *Chapter 9 – Drainage and Erosion Control*.

8.2 Design Storm

8.2.1 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 historical rainfall data.

Historic Rainfall data for the period between the early 50s to early 80s are available for most rainfall stations (located almost in all provinces) are available in Afghanistan. The records were discontinued from the 1980s until 2004. However, since then, most of the data from the hydrological and meteorological stations has been restored and is available with the Water Resources Department under the Ministry of Energy and Water (MEW). Where the gaps have been filled statistically, and continuous daily rainfall records of 50 years or so are available, they can be used to compute design rainfall intensity.

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 8-1 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 8-1: 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

8.2.2 Design Storm for different Structures

The indicative design storm return period for different structures is shown in Table 8-2. 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 judgment and local consultations. Table 8-3 should be used when less risk can be tolerated.

Table 8-2: Indicative storm design return period (years) for different structures

Type of drainage structure	Geometric design standard			
	LVRR 5/4	LVRR 3	LVRR 2	LVRR 1
Gutters and inlets	2	2	2	1
Side ditches	10	5	5	2
Drift or vented drift (1)	10	5	5	2
Culvert diameter/span <2 m	15	10	10	5
Large culvert span >2 m	25	15	10	5
Gabion abutment bridge	25	20	15	-
Short length bridge <10 m	25	25	15	-
Masonry arch bridge	50	25	25	-
Medium length bridge (10 m-50 m)	50	50	25	-
Long bridge >50 m	100	100	50	-

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

Table 8-3: Storm design return period (years) for severe risk situations

Type of drainage structure	Geometric design standard			
	LVRR5/4	LVRR3	LVRR2	LVRR1
Gutters and inlets	5	5	5	2
Side ditches	15	10	10	5
Drift or vented drift	15	10	10	5
Culvert diameter/span <2 m	25	20	20	10
Large culvert span >2 m	50	25	20	10
Gabion abutment bridge	50	25	20	-
Short length bridge <10 m	50	50	25	-
Masonry arch bridge	50	50	25	-
Medium length bridge (15 m-50 m)	100	100	50	-
Long bridge >50 m	100	100	100	-

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, which of course will depend on the terrain and nature of the location site and calls for careful engineering judgment

8.3 Runoff Estimation Methods

8.3.1 General

Water crossing structures must be designed to have a capacity equal to or greater than the design flood discharge that is expected in the watercourse. Structures not designed for overtopping must have an acceptable freeboard. This design flood needs to be determined based on the characteristics of the storm itself, namely the intensity, duration and frequency of the rainfall, and the characteristic of the ground, or catchment.

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.
- The Soil Conservation Service (SCS) Unit Hydrograph Method.

8.3.2 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. Several people should be questioned as memories 'fade' over time. It may be possible to ask people individually how high the most significant 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 on 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.

8.3.3 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 simply on the catchment area using the same relationships.

8.3.4 The Rational Method

This method is used for estimating peak discharges for small drainage areas up to about 100 hectares. Larger catchments can be considered using either the modified Rational Method, for example, by using the Areal Reduction Factor (ARF) shown in Equation 8-3, or the SCS Method described in Section 8.3.5 below.

The flow of water in a channel, Q, is calculated from the following equation:

$Q = 0.278 \times C \times I \times A$ (m^3/s)……………………………………………………………………………….Equation 8-1

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 Runoff Coefficient "C" is obtained from Table 8-4.

Table 8-4: Run-off coefficient

Type of Cover	Soil Type	Run-off Coefficient		
		Flat	Rolling (2% - 10%)	Mountainous (over 10%)
Buildings and roofs		0.90	0.90	0.90
Concrete paved Surfaces		0.80	0.90	0.95
Asphalt paved surfaces		0.70	0.80	0.90
Earth embankments	Bare and compacted	0.60	0.60	0.60
Gravel road shoulders		0.50	0.55	0.60
Sidewalks		0.80	0.82	0.85
Grassed areas	Sandy	0.10	0.15	0.20
Grassed areas	Clay	0.15	0.20	0.30
Farmed land	Sand and gravel	0.25	0.30	0.35
Farmed land	Clay and loam	0.50	0.55	0.60
Steppe forest	Sandy	0.10	0.15	0.20
Semi desert land	Bare and loose	0.10	0.20	0.30

Source: ASCE-AED, 2009

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 locally derived charts should ideally be used. In many situations, these will 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.

US Army Corps of Engineers, Afghanistan Engineer District (ASCE-AED, 2009) has prepared Intensity – Duration – Frequency (IDF) curves for some selected cities of Afghanistan. The one most closely matching the project area may be selected for the design purpose. These IDF curves are presented in Annex 8-1. AED report also recommended the following equation to calculate rainfall intensity I:

$$I \ \mathrm{X} \ \frac{R}{24} * \frac{24}{T_c}^{\,K}$$ ………………………………………………………………………………………… Equation 8-2

where:

I = rainfall intensity (mm/hr)

R = daily rainfall of design frequency (10 year return period for LVRR)

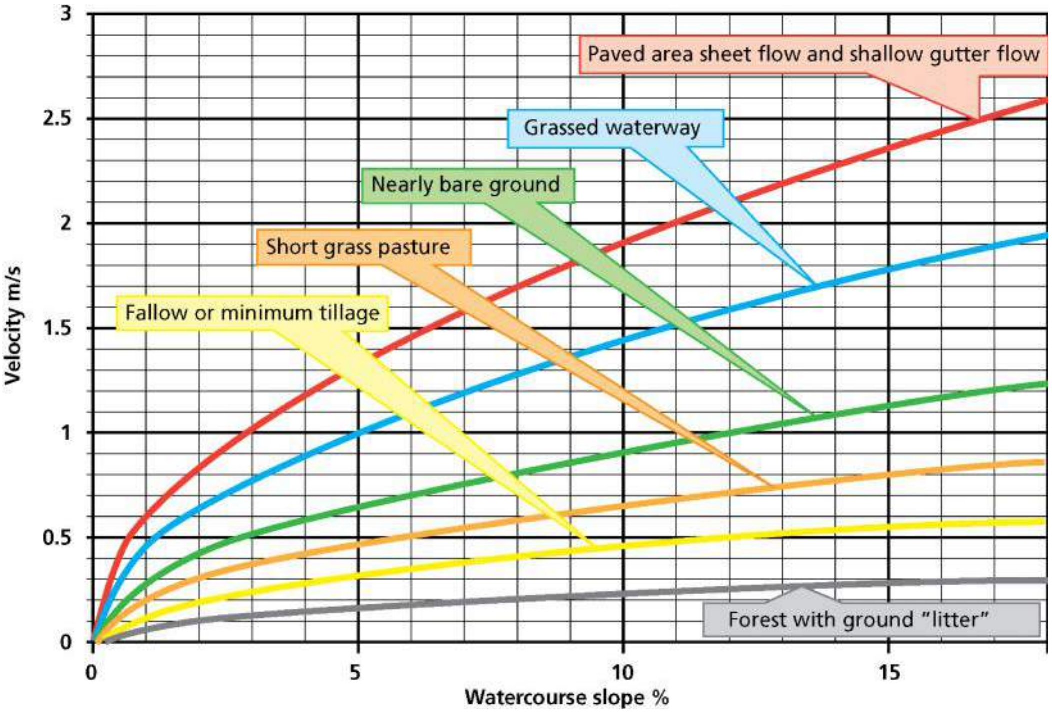
T_c = Time of concentration (hour)

K = a regional coefficient, approximately 0.722 for a 10-year storm in Afghanistan.

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 and is estimated from Figure 8-1.



Source: FHWA. Hydraulic Engineering Circular No. 19 (1984)

Figure 8-1: Velocity of flow for varying surface cover

The storm design return period is taken from Table 8-2 or Table 8-3.

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 ....Equation 8-3

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 runoff caused by a larger catchment area.

The steps in computing the design discharge for a cross drainage structure using the Rational formula can be summarized as follows:

1. Delineate the catchment area in a printed topographical map or on a digital map.
2. Measure the catchment area by planimeter on a printed map or digitally using AutoCAD or GIS software in km^2.
3. If applicable, use the ARF using Equation 8-3.
4. Decide the value of runoff coefficient C from Table 8-4 using careful engineering judgment.
5. Find T_c with the help of Figure 8-1 or using the following Kirpich formula:

$$T_c \, X 0.0078 \, \frac{L^{0.77}}{S_{0.385}}$$ .... Equation 8-4

where:

T_c = Time of concentration in minutes (to be taken as design storm duration).

L = Flow length in meters.

S = Average slope of the flow channel.

6. Find the design rainfall intensity from the appropriate IDF curve from Appendix 8-1 or use Equation 8-2.
7. Put all these values into Equation 8-1 and find the value of Q, which is the design discharge.

8.3.5 The SCS Method

General

The United States Soil Conservation Service method for calculating rates of runoff requires much of the same basic data as the Rational Method, namely catchment area, a runoff factor, time of concentration, and rainfall. Equation 6.1 indicates the critical importance of both the intensity of rainfall and the run-off coefficient. The SCS method uses detailed methods for obtaining an accurate value of the run-off coefficient by considering the initial rainfall losses to interception and storage in the soil and an infiltration rate that decreases during a storm. However, it also recognizes that the most likely local rainfall data to be available is 24-hour precipitation records rather than IDF data (intensity duration frequency data); therefore, broader assumptions concerning intensity and duration are made. Nevertheless, it is potentially more accurate than the Rational Method and is applicable when the catchment area is larger than 50 hectares (0.5 km2).

Catchment area

The catchment area is determined from topographic maps and field surveys. For large catchment areas, it might be necessary to divide the area into sub-catchment areas to account for major land-use changes.

Rainfall

The SCS method is based on a 24-hour storm event. The characteristics of storms are defined in terms of the relationship between the percentage of the total storm rainfall that has fallen as a function of time. Three basic types of storm are defined for three levels of maximum intensity, Type I being the least intense and Type III the most intense. Type III should be used in Afghanistan.

A relationship between accumulated rainfall and accumulated runoff has been derived for numerous hydrologic and vegetative cover conditions. The storm data included the total amount of rainfall in a calendar day but not its distribution with respect to time. The SCS runoff equation is, therefore, a method of estimating direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$Q = (P - I_a)2 / [(P - I_a) + S]$..Equation 8-5

where:

Q = accumulated direct runoff, mm.

P = accumulated rainfall (i.e., the potential maximum runoff), mm

I_a = initial abstraction, including surface storage, interception and infiltration prior to runoff, mm.

S = potential maximum retention, mm.

S is related to the soil and cover conditions of the catchment area through the Curve Numbers, CN, described below.

S = 254 (100/CN – 1) ……………………………………………………………………………………..Equation 8-6

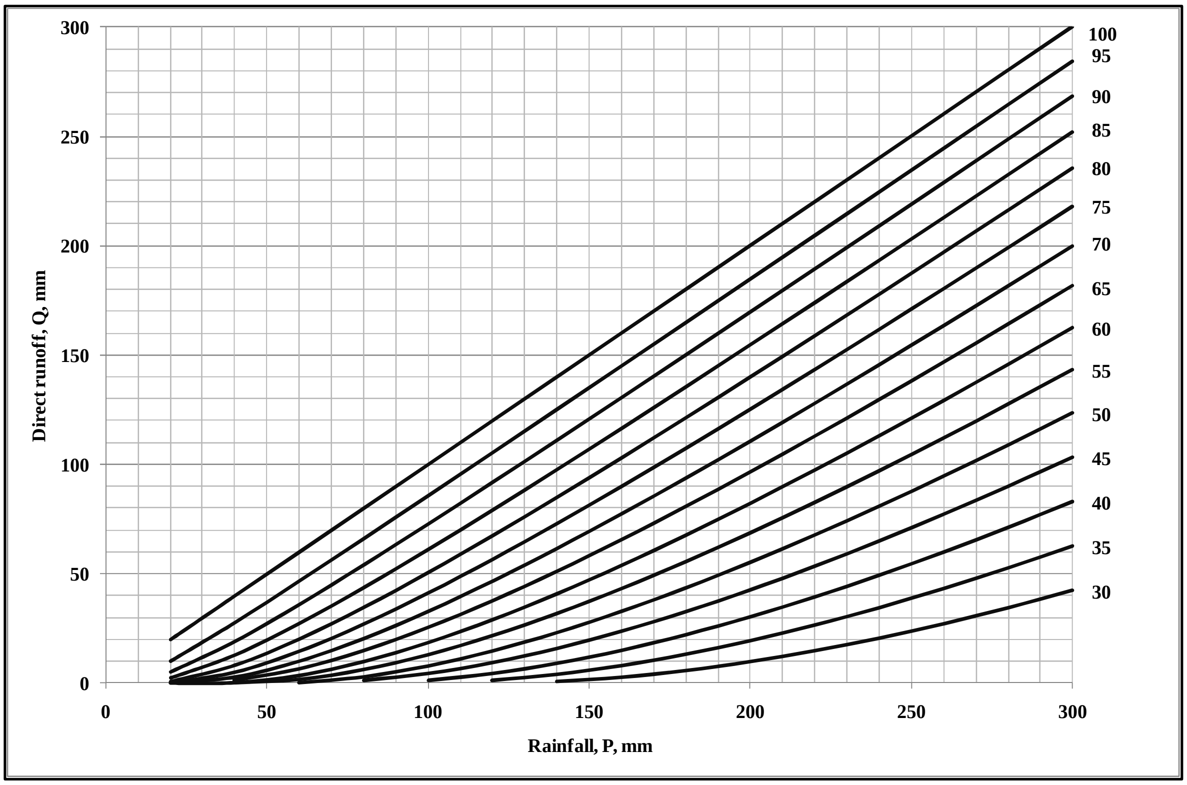
The relationship between I_a and S was found to be:

I_a = 0.2S ……….Equation 8-7

By substituting I_a into Equation 8-5, the equation becomes:

$Q = (P - 0.2S)^2/(P + 0.8S)$ ……………………………………………………………………………… Equation 8-8

Figure 8-3 shows a graphical solution that enables Q, the direct runoff from a storm, to be obtained if the total rainfall and catchment area curve number are known.



Source: Adapted from USDA TR55 (1986)

Figure 8-2: Relationship between precipitation P, direct run off Q and CN

Runoff and curve numbers

The physical catchment area characteristics affecting the relationship between rainfall and runoff (i.e., the CN values) are land use, land treatment, soil types and land slope. Land use is the catchment area cover, and it includes agricultural characteristics, type of vegetation, water surfaces, roads, roofs, etc. Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops. The SCS method uses a combination of soil conditions and land use to assign a runoff factor to an area. These runoff factors or curve numbers (CN) indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential.

Soils are divided into four hydrologic groups (Groups A, B, C, and D) based on infiltration rates, as shown in Table 8-6.

Table 8-5: Hydrological characteristics of soil groups

Soil Group	General Description	
A	Well-drained, sandy	High infiltration, low runoff
B	Sandy loam, low plasticity	
C	Clay loam, medium plasticity	
D	Highly plastic clay	Low infiltration, high run off

Runoff curve numbers also vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the higher the antecedent rainfall, the more direct runoff there is from a given storm. A five-day period is used as the minimum for estimating antecedent moisture conditions.

Table 8-7 gives runoff curve numbers for various land uses and soil groups. This table is based on an average antecedent moisture condition (i.e., soils that are neither very wet nor very dry when the design storm begins). Table 8-8 gives conversion factors to convert average curve numbers to wet and dry conditions.

Table 8-6: Runoff curve number (CN)

Land use			A	B	C	D
Cultivated land	Without conservation treatment		72	81	88	91
	With conservation treatment		62	71	78	81
Pasture land	Poor condition		68	79	86	89
	Good condition		39	61	74	80
Meadow			30	58	71	78
Wood or forest	Thin stand, poor cover, no mulch		45	66	77	83
	Good cover		25	55	70	77
Open spaces, lawns, parks	Good condition, grass cover >75% of area		39	61	74	80
	Fair condition, grass on 50-75%		49	69	79	84
Urban districts	Commercial and business areas, 85% impervious		89	92	94	95
	Industrial districts, 70% impervious		81	88	91	93
Residential	Average lot size	Average % impervious				
	< 0.05 hectares	65	77	85	90	92
	0.1 hectares	38	61	75	83	87
	0.2 hectares	25	54	70	80	85
	0.4 hectares	20	51	68	79	84
	0.8 hectares	12	46	65	77	82
Paved roads with curbs, storm drains, paved parking areas, roofs.			98	98	98	98
Gravel roads			76	85	89	91
Earth roads			72	82	87	89
Open water			0	0	0	0

Source: Adapted from USDA TR55 (1986)

Table 8-7: Conversion from average to wet and dry antecedent moisture conditions

CN values		
Average conditions	**Dry**	**Wet**
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50

Source: Adapted from FHWA. Hydraulic Engineering Circular No. 19 (1984)

Time of concentration

The next step in the SCS Method is to determine the Time of Concentration. This is the time it takes water to flow from the edge of the catchment area to the point of interest. It is a combination of three values:

A - Sheet flow

B - Shallow concentrated flow

C - Open channel flow

The type of flow that occurs is a function of the conveyance system and is determined by field inspection. It is often a combination of A, B and C. Thus, the total travel time is the sum of the time taken for the water to pass through all of the segments of the catchment.

Travel time is the ratio of flow length to flow velocity:

$T = L/(3600V)$ ……………………………………………………………………………………………….Equation 8-9

where:

T = travel time, hr.

L = flow length, m

V = average velocity, m/s

3600 = conversion factor from seconds to hours.

Sheet flow: Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.03 m. Table 6-8 gives Manning's n values for sheet flow for various surface conditions.

Table 8-8: Manning's roughness coefficient n for sheet flow

Surface	n
Smooth surfaces: concrete, asphalt, gravel or bare soil	0.011
Fallow (no residue)	0.05
Cultivated soils Residue cover < 20% Residue cover > 20%	 0.06 0.17
Short grass Dense grass	0.15 0.24
Range	0.13
Woods [1] Light underbrush Dense underbrush	 0.4 0.8
Note 1: Consider cover to a height of 30 mm. This is the only part of the light and dense underbrush that will affect sheet flow	

Source: Adapted from FHWA (1984)

For sheet flow of less than 100 m, Manning's kinematic solution should be used to compute the travel time T:

$T = [0.091(n \times L)^{0.8}/(P_2^{0.5} \times S^{0.4})]$ ……………………………………………………………………………Equation 8-10

where:

T = travel time, hr

n = Manning's roughness coefficient (Table 6 8)

L = flow length, m

P_2 = 2-year, 24-hour rainfall, mm

S = slope of hydraulic grade line (land slope), m/m

Shallow concentrated flow: After a maximum of 100 m, sheet flow usually becomes a shallow concentrated flow. The average velocity for this flow can be determined from the following equations in which average velocity is a function of watercourse slope and type of channel.

Unlined (unpaved) $V = 4.918 (S)^{0.5}$ ……………………………………………………….Equation 8-11

Lined (paved) $V = 6.196 (S)^{0.5}$ ……………………………………………………….Equation 8-12

where:

V = average velocity, m/s

S = slope of hydraulic grade line (watercourse slope), m/m

After determining average velocity, the travel time for the shallow concentrated flow segment is calculated from Equation 8-12.

Open channel flow: Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on topographic maps (1:50,000). Average flow velocity is usually determined for bank-full elevation. Manning's equation or water surface profile information can be used to estimate average flow velocity. When the channel section and roughness coefficient (Manning's n) are available, then the velocity can be computed using the Manning Equation:

$V = (R^{2/3} \times S^{1/2})/n$ ……………………………………………………………………………………….Equation 8-13

where:

V = average velocity, m/s
R = hydraulic radius, m (equal to A/P_w) (See Figure 8-2)
A = cross sectional flow area, m^2
P_w = wetted perimeter, m
S = slope of the hydraulic grade line, m/m
n = Manning's roughness coefficient (Table 8-9)

After the average velocity is computed, the travel time for the segment can be calculated using Equation 8-12.

Reservoir or lake: Sometimes, it is necessary to compute the time of concentration for a catchment area having a relatively large body of water in the flow path. The travel time is computed using the equation:

$V_w = (g \times D_m)^{0.5}$ ……………………………………………………………………………………….Equation 8-14

where:

V_w = the wave velocity across the water, m/s
g = 9.81 m/s^2
D_m = mean depth of lake or reservoir, m

This equation only deals with the travel time across the lake, not the time at the inflow or outflow channels. The times for these are generally very much longer and must be added to the travel time across the lake. Equation 8-16 can be used for swamps with much open water, but where the vegetation or debris is relatively thick (less than about 25 percent open water), Manning's equation is more appropriate.

Steps in the SCS Procedure

The steps in using the SCS method are as follows:

1) Determine the catchment area, A, and its soil and land use characteristics
2) Determine the 'curve runoff number, CN, from Table 8-6 (soil characteristics) Table 8-7 (land use) and any adjustment based on the likely antecedent soil moisture conditions (Table 8-8)
3) Calculate the value of I_a from Equation 8-9.
4) Choose the appropriate design storm recurrence frequency from table 8-2 or 8-3. This is based on the class of road and the drainage structure being designed.
5) For the recurrence frequency chosen, determine the 24-hour rainfall (P) for the appropriate rainfall region (to be obtained locally).
6) Determine the direct runoff (Q) for the rainfall (P) and curve number (CN) obtained in steps 3 and 5 from Figure 8-3.
7) The catchment must be divided into uniform areas to determine the Time of Concentration, Tc. The flow lengths for sheet flow, shallow concentrated flow, and channel flow must be determined using the equations in Section 8.5.5 to calculate the total time of concentration.

   At this stage, the following data have been obtained:

   I_a – the initial abstraction based on the curve number CN (step 3)

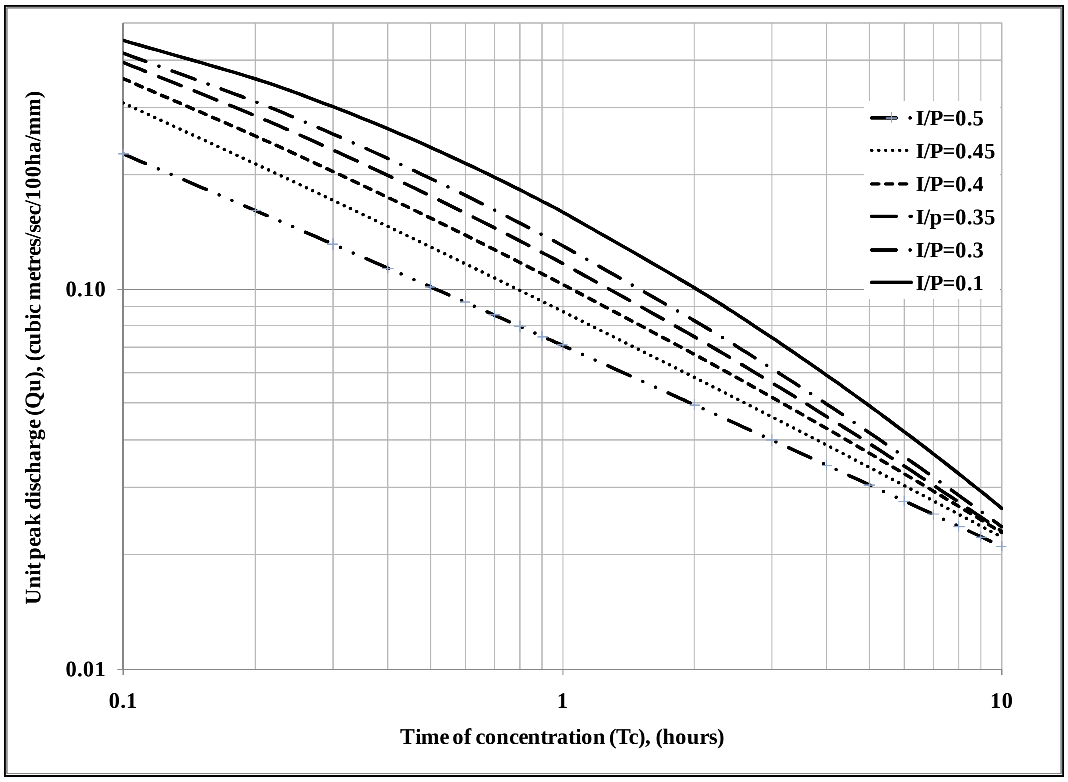
   P – the design storm precipitation (steps 4 and 5)

   Q – the accumulated direct runoff (step 6)

   Tc – the Time of Concentration (step 7)
8) The next step is to determine the unit peak discharge, Qu. This is achieved using Figure 8-4, the value of I_a/P and the Time of Concentration.
9) The final step is to compute the actual peak discharge from the unit value as follows:

   Design peak discharge = Qu x Q x A ……………………………………………………..Equation 8-15

   where Q is in mm and A is in units of 100 hectares.



Source: Adapted from USDA TR55 (1986)

Figure 8-3: Unit peak discharge (Type II rainfall)

8.3.6 Flow Velocity

General

It is also important to determine the velocity of the water flow during peak flows because this affects the amount of scouring that can be expected around the structure and hence the protective measures that may be required. The velocity can be measured directly by using a piece of precision equipment such as a current meter or roughly by a float method or can be indirectly calculated using Manning's equation, as explained below.

Direct measurement by a Float Method

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 a known distance downstream (e.g., about 100m) should be measured. The velocity can then be calculated by dividing the distance the floating object has traveled by the time taken. This exercise should be repeated at least three times, but preferably five 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 very limited. This surface velocity should be multiplied by a factor of 0.75 to get the average velocity at that reach.

Manning's equation

Design discharge 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:

$Q = A \times V$ ....Equation 8-16

The flow velocity is calculated from the Manning equation:

$V = 1/n \times R^{2/3} \times S^{1/2}$ ....Equation 8-17

where:

V = cross-sectional average velocity in m/s
A = cross-sectional area of water (m^2)
R = hydraulic depth (area for the streamflow divided by the wetted perimeter) (Figure 8-2).
S = hydraulic gradient (slope of the drain or watercourse)
n = roughness coefficient (Table 8-5)
Q = discharge volume flow rate (m^3/sec)

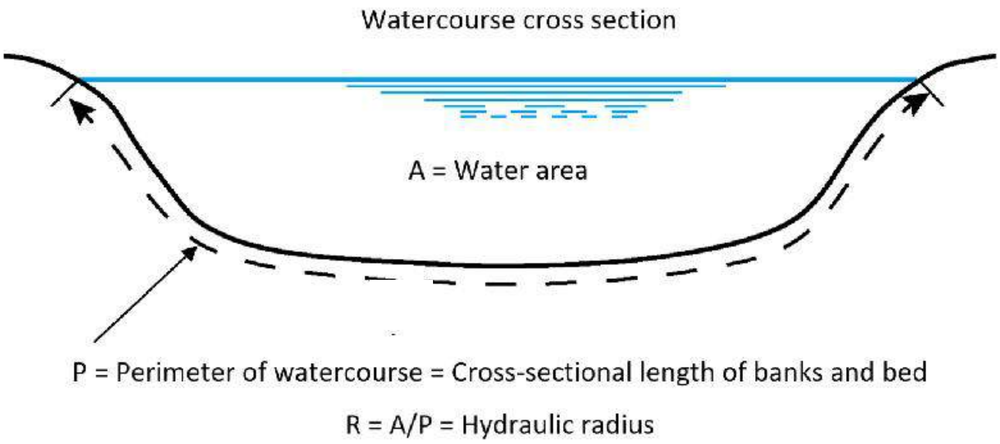


Figure 8-4: Definition of hydraulic radius in Manning's Formula

Table 8-9: 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

8.4 Water Crossings and Associated Structures

8.4.1 General

Although the main focus of this Guideline is on the upgrading and provision of LVRR infrastructure in terms of appropriate pavement design and surfacings, many of these LVRRs will remain impassable unless appropriate water crossing structures, commensurate with the level of service provided by a LVRR, are also provided. The cost of such structures will depend on the topography and may account for a significant proportion of the total construction costs. Once a road has been constructed, the passability and maintenance costs are closely linked to the type and quality of the drainage structure provided.

8.4.2 Type of Structure

The choice of structure type will influence its costs and must, therefore, be given careful consideration. This section considers different water crossing options, from drifts and culverts to small bridges with lengths of <10 m, and describes the characteristics of each, the conditions suitable for their use, and their advantages and disadvantages.

The design of each structure to cope with the design stormwater 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.

On LVRRs it may not be necessary to provide structures that are passable at all times, even during short periods of peak flows. The distinguishing feature between the main types of structure that are suitable for LVRRs is whether they are:

1. **Submersible:** Designed to be overtopped without damage to the structure and be passable except during short periods of peak flow, such as drifts, vented drifts/causeways and submersible bridges.
2. **Non-submersible:** Designed to channel all water under/through the structure and be passable at all times, such as culverts, arch culverts/bridges, embanked crossings and small span bridges.

Due to the relatively low traffic flow and associated disruption to traffic on LVRRs, particularly on the lower classes LVRR 1-3, structures in the first category are generally the most appropriate if the water flow cannot be accommodated in simple cross culverts.

The different types of water crossing structures are presented individually in Sections 8.6 to 8.12. In some instances, particularly for wide alluvial plains, a combination of a number of structures might be the most cost-effective means of spanning the watercourse.

8.4.3 Road 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 that a low-level structure may not be passable. It may be necessary to raise the running surface of the structure, such as a vented drift, 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 method for obtaining information on seasonal flood levels and flows, will be through consultations with the local communities.

In the absence of any local information and data, suggested upper and lower bounds for closure times, are shown in Table 8-10.

Table 8-10: Suggested closure times

Criteria	Low-level structure most favorable	Low-level structure least favorable
ADT	< 5 vpd	>200 vpd
Average annual flooding	< twice per year	> 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 embanked crossings, i.e., vented drifts 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 bed material.

8.4.4 Site Selection and Appraisal

General

Careful site selection is essential to ensure ease of construction and to minimize the life-cycle cost of the structure. Poor site selection can result in a longer, wider or higher structure than is necessary. Poor siting can also lead to excessively high maintenance costs, and in extreme cases a high risk of destruction of the structure.

For minor structures such as drifts or culverts on existing routes, there may be little choice available in site selection. Changing the existing road alignment could incur substantial additional road works costs.

For relief drifts or culverts, necessary to allow the build-up of water in side drains to cross the road alignment, there is usually some flexibility in location. Normally side drains will require to be relieved by a turnout or cross structure after a maximum length of about 200 m to avoid exceeding capacity or causing erosion. Ideal outfall sites are at field boundaries or where there is vegetation or stable ground to minimize the risk of damage or erosion downstream.

For larger structures and watercourses, the location of the site requires more attention. Adjustment of the road alignment is often justifiable to minimize the cost of structures and risk of damage or erosion.

Location of structure

Regardless of the type of structure to be constructed, the following criteria should ideally be met when determining a site for a water crossing (other than at side drain relief, drift and culvert crossings):

- The crossing should be located away from horizontal curves in the watercourse, as these areas are unstable, with the line of the watercourse tending to move towards the outside of the bend with time.
- The crossing should be located at a section of a uniform watercourse gradient. If the gradient is steepening, there is a greater possibility of scour and erosion occurring. However, if the gradient is reducing, there is the potential for silt and other debris to be deposited near or inside the structure.
- The crossing should ideally be located on a section of the channel with a non-erodible bed. These areas have reduced scour potential, reducing the amount of watercourse protection required.
- The road should cross the watercourse at a point with well-defined banks, where the stream will generally be narrower.
- The watercourse should not be prone to flooding at the crossing point.

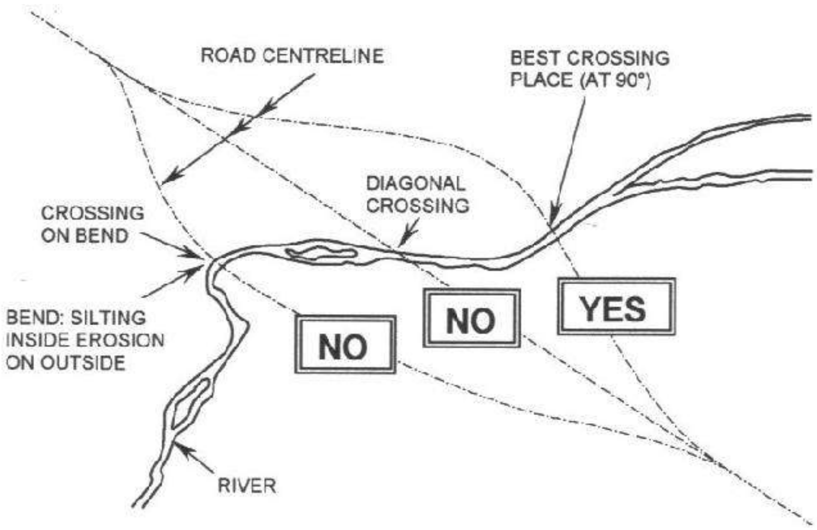


Figure 8-5: Suitable crossing points for large structures

Further to the above, the following criteria should be considered for determining the most appropriate location of water crossing structures:

- A site with a natural narrow channel width rather than a wide one should be used.
- The crossing should be located at a straight stretch of river or watercourse, rather than at a curved one where the stream is likely to cause erosion of the bank on the outside of the curve.
- The road alignment should be at right angles to the water flow to avoid additional scouring.
- A skew crossing may channel the water towards one of the river banks. This channeling may erode the approach way and/or the bank, eventually resulting in the river flowing around the bridge rather than under it.
- The approach roads should preferably be straight on each side to ensure sufficient sight distances and prevent traffic hazards.

It is very rare that all the criteria above can be satisfied for each crossing. Thus, a balanced consideration of the various factors listed above is required. It is necessary to establish the most cost-effective solution for each structure, depending on individual circumstances.

Road alignment

In addition to the watercourse requirements noted above, the road should:

- Cross the watercourse at 90 degrees to minimize the span length of the bridge or pipe, as illustrated in Figure 8-6.

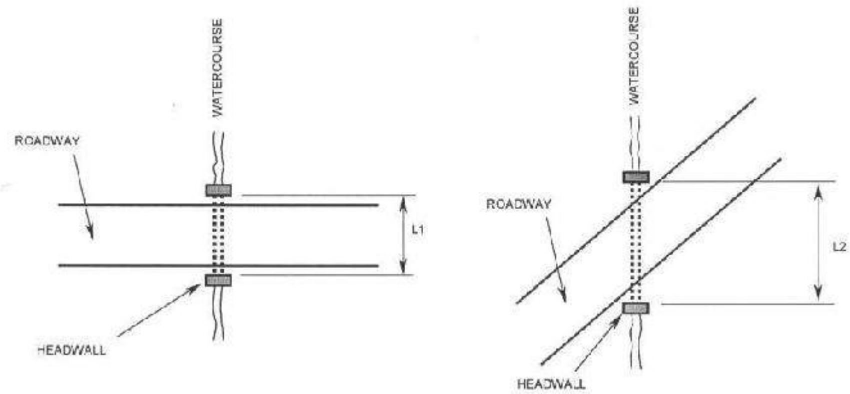


Figure 8-6: Length of structure at 90 degree and skew crossing

- Cross on a straight length of the road, rather than in a curve, to reduce the width of a bridge or the length of a culvert.
- Be fixed vertically at the minimum elevation necessary to pass above the design flood flow (not required for drifts and vented drifts). If the road alignment is fixed too high, unnecessary costs will be incurred in abutment/wingwall/ headwall construction and approach embankments.
- Be centered above the centerline of the substructure.

8.4.5 Selection of Water Crossing Structure

General

The objective of selecting a water crossing structure is to choose the most appropriate design for each location. This selection should be based on the factors outlined in Figure 8-7.

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 drift, a causeway and a large span 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, seasonal variations, and the duration of road closures that can be tolerated.

The flow diagram in Figure 8-7 shows in more detail the questions that should be considered when choosing a structure. Factors affecting the choice of a structure are different for each location and require several issues to be addressed. The flow diagram highlights only the key issues influencing the selection of water crossing structures and should be used as a preliminary guide when selecting the most appropriate structure. The final decision on the choice of water crossing structure should be based on a more detailed investigation of the characteristics of the watercourse, as discussed in Section 8.5.3.

The flow diagram also asks questions regarding the permissible closure time for a road during floods. Each case will have to be assessed separately, depending on its particular circumstances.

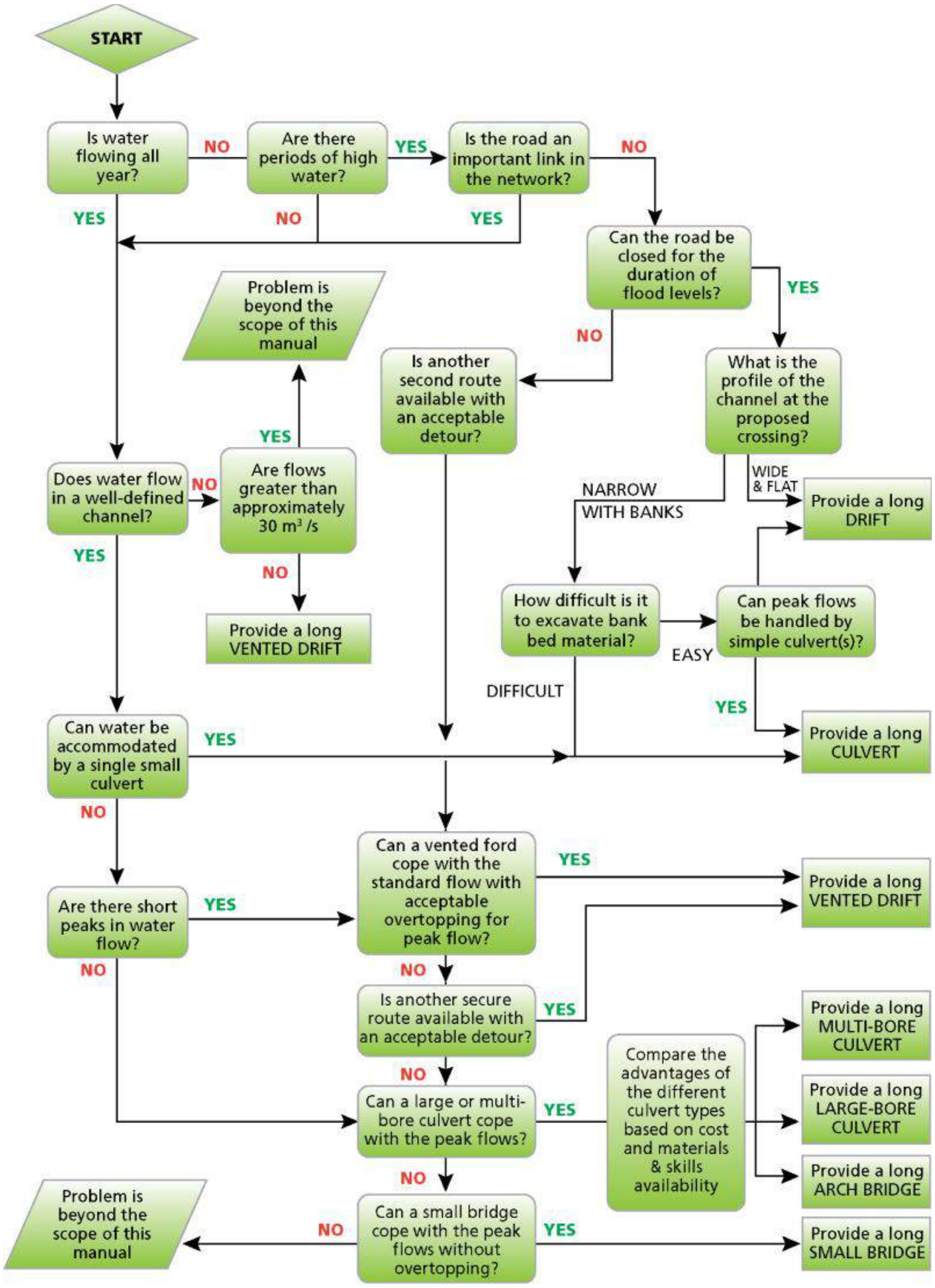


Figure 8-7: Route map for selection of water crossing structure

Width of the structure

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 LVRR 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 a significant proportion of the overall cost of a LVRR, 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 traveling 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 outside 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, as illustrated in Figure 8-8. 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 simultaneously motorcycles as well as a vehicle, a minimum width of 4.5 m is required.

Figure 8-8: Example of a restricted width on water crossing structure

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 the passage of heavy goods 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.

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 historical data on which hydrological analysis and hydraulic design rely may lead to an underestimation 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 8-3, 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 include:

- 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 on 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.
- Providing better surface drainage so that water is dispersed off the road more frequently.
- Reducing water concentration by means of additional cross drains and miter drains to lower the volume of water that each one needs to deal with (*Chapter 9 – Drainage and Erosion Control*).

Erosion is a serious problem in many areas and adverse climate change and deforestation will make matters worse, as discussed in *Chapter 9 - 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 5 – Geotechnical Investigations and Design*.

Ensuring that the drainage system is working correctly is primarily 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.

8.5 Submersible Water Crossings

8.5.1 General

A submersible 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. Drifts are often dry and cater to 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 8.7.4. 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.

8.5.2 Drifts

General

At the most basic level, a simple drift can be constructed 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.

A more substantial drift consists of a flat concrete 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 and may also be referred to as fords.

Drifts are suitable for shallow watercourses 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 drifts, namely:

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 miter drains. It is an alternative to a relief culvert.

Small watercourse (or stream) drifts: Where stream flows are very low with a normal water depth of less than 200 mm) drifts may be used to allow the stream to cross the road, as illustrated in Figure 8-9.

Key features of drifts

The key features of drifts are:

- Stream drifts are structures that 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 the 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 a 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 undercutting 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.

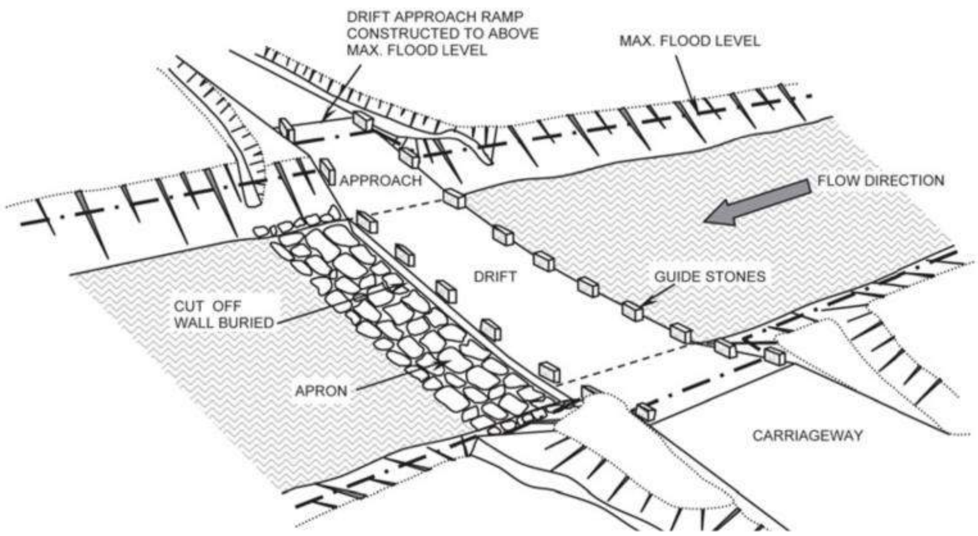


Figure 8-9: Key features of a stream drift

Figure 8-10: A simple stream drift

Figure 8-11: A drift crossing a wide floodplain

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 watercourse (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 watercourse. The slab should be at the same level as the bed of the watercourse, 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 watercourse 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 meter beyond the required high-water point. The cost of a drift is normally estimated per meter 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. Construction joints should be provided so that each slab is no more than 5 m 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 graveled or provided with an improved surface for 50 m 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).

Advantages and disadvantages of drifts

The advantages and disadvantages of drifts are summarized in Table 8-11.

Table 8-11: Advantages and disadvantages of drifts

Advantages	Disadvantages
• Low cost: at the most basic level - can be constructed and easily maintained entirely with local labor 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.

8.5.3 Splash

The simplest form of a drift is a splash, which is a low-cost solution to crossroad drainage and provides an inexpensive alternative to culverts. While drifts are used to cross natural streams and rivers, the splash is mainly used to lead surface water across and away from the road.

This drainage structure usually is only applied to the lowest LVRR classes, i.e., LVRR 1 and 2, with very low traffic volumes.

Figure 8-12: Example of a splash

As with culverts, the main purpose of the splash is to lead water from the hillside drain across the roadway to be discharged on the downstream side of the road. The main design element when building a splash is, therefore, to secure a surface on which the water can flow without creating any erosion or reducing the bearing capacity of the road. In exceptional cases, where there is a limited flow of water, good natural soils and little traffic, the splash can be built using gravel. However, in most cases, it is necessary to install a more durable surface made from materials such as stone or concrete.

Similar to drifts, the splash is designed at a lower level than the road with descending approaches on both sides. The splash needs to be deep enough to cater to the highest flow of water, without reaching the unprotected road sections on each side of the water crossing.

As with all drainage structures, the frequency and capacity of a splash need to be assessed in relation to the overall drainage system of the road. As a general rule, the provision of frequent means for water to be discharged from the side drains will minimize the chances of erosion and silting.

Downstream from the splash, it is often necessary to protect the area where the water is discharged. Packed stone or gabions can provide a solid but inexpensive surface for this purpose.

8.5.4 Vented Drifts

General

A vented drift, also called causeway, is a combination of a culvert and a drift. They are suitable for carrying roads across watercourses 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.

Key features of vented drifts

A typical example of a vented drift is shown in Figure 8-9 (schematic) and 8-10 (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 watercourse.
- 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 center 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.

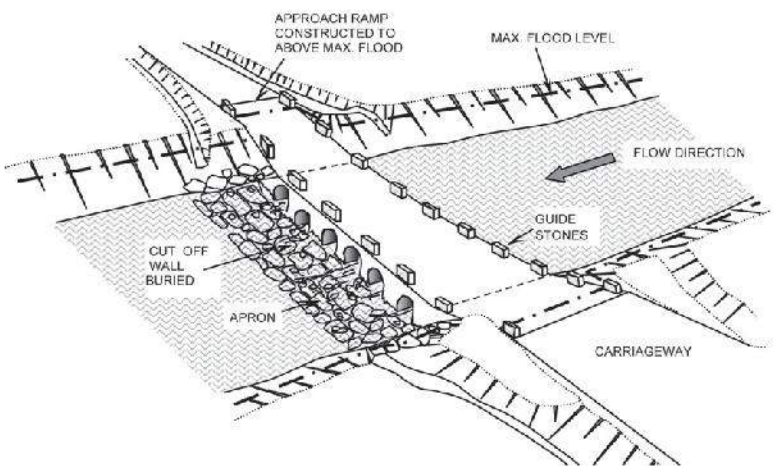


Figure 8-13: A typical vented drift/causeway (schematic)

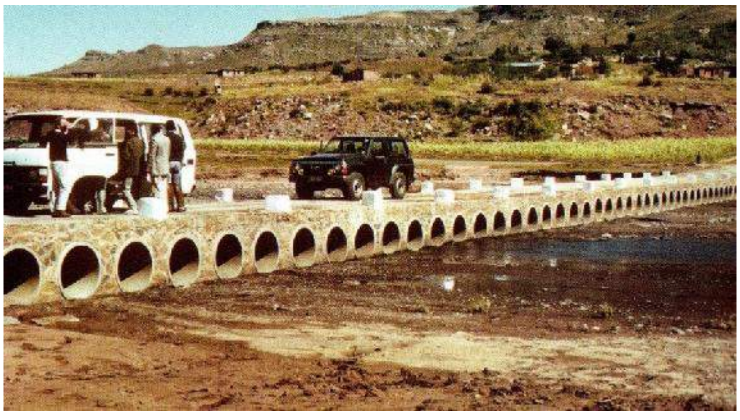


Figure 8-14: A typical Vented drift/Causeway

Advantages and disadvantages of vented drifts/causeways

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

Table 8-12: 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. - The 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 labor 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.

Design of vented drifts/causeways

Vented drifts fall into two categories—low vent-area ratio (VAR) and high VAR—each of which affects stream channels differently. Vented drifts with pipe culverts that are small relative to the full channel area have a low VAR. Conversely, a high VAR can be achieved with box culverts having a vent opening that approximates or exceeds the size of the full channel.

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.

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 meter above the highest water level observed during heavy rain. Gravel or an improved surface should also be placed on the road for 50 m in each direction.

To facilitate the safe passing of vehicles 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. Blocks should also be constructed along the sides of the vented drift to guide vehicles and help pedestrians to pass when water is flowing.

8.5.7 Submersible Bridge

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. These structures allow roads to cross watercourses, which have large flows for most of the year and very large flows lasting for up to a week during and after heavy rains.

Figure 8-15: Example of submersible bridge

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.

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.

8.5.8 Length of Submersible Water Crossings

The length of a submersible water crossing has, in principle, no lower or upper limits. The length is determined by the shape and width of the watercourse and the length of the approach slabs, which are required to prevent the water, at the highest water level, from scouring around the slabs and eventually undermining the whole structure.

The principle for determining the length of the structure is the same for all submersible water crossings, as illustrated in Figure 8-16.

The water crossing should be at least the same width as the natural watercourse, and no attempt should be made to constrain the flow of the water. It is, thus, important to establish with the best possible accuracy, the highest flood water level.

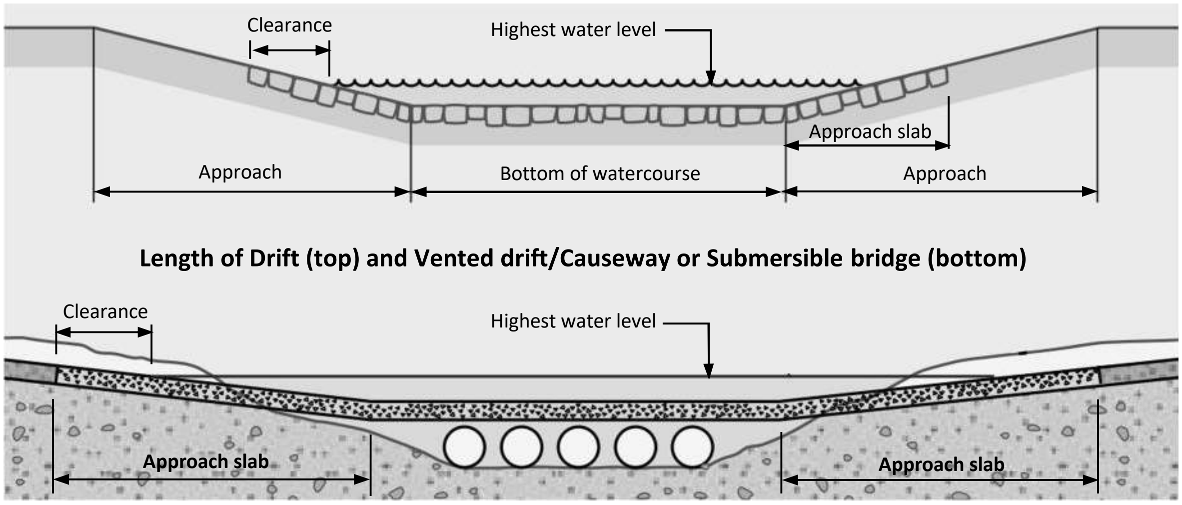


Figure 8-16: Illustration of the required length of submersible drainage structures
Source: Adapted from Building Rural Roads (ILO, 2008)

8.6 Non-submersible water crossings

8.6.1 Culverts

General

Culverts are the most common form of drainage or water crossing structure on LVRRs and perform two basic functions:

- Relief of water in the side drains.
- Channeling water in relatively narrow, well-defined watercourses under the road.

Culverts can be made from precast or cast in-situ circular concrete pipes or corrugated steel pipes (elliptical or circular) or be constructed in-situ as rectangular box culverts with stone masonry/concrete walls and a concrete top slab.

Relief culverts: These are placed at low points in the road alignment where there is no definable stream or along long downhill gradients where regular relief of the side drains is required. A relief culvert should be located at the point where the high volume of water starts to cause erosion or to overtop the pavement. Relief culverts should be used only when relief of the side drains through miter drains is not possible.

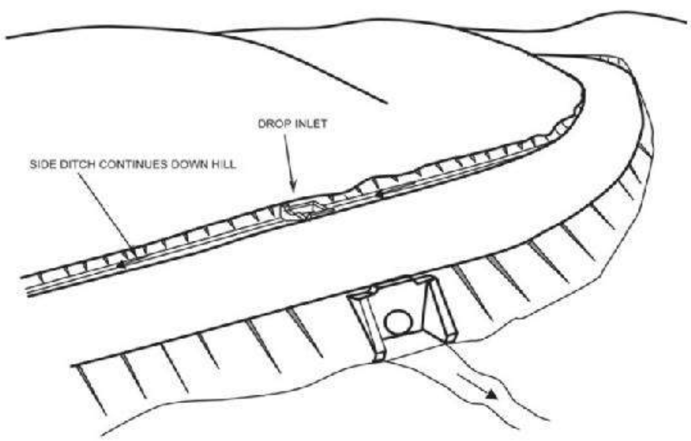


Figure 8-17: Key features of a relief culvert

The number of relief culverts can vary from about two per kilometer 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.

Table 8-13 shows recommended minimum intervals between relief culverts on long gradients.

Table 8-13: Recommended minimum relief culvert spacing on long gradients

Road gradient %	Culvert interval (m)
12	40
10	80
8	120
6	160
4	200
2	> 200

Pre-cast circular concrete pipes are most commonly used for relief culverts. The minimum dimension should be 750 mm to allow for ease of cleaning. However, due to their circular shape and the minimum cover that is required to prevent damage to the pipes, concrete pipes will often require lifting of the alignment over the pipes and/or long and deep outlet channels, depending on the terrain, to ensure adequate flow through the pipes and the outlets.

A rectangular stone masonry box culvert with a height equal to the diameter of a circular pipe will have a larger hydraulic capacity and will require less cover to withstand the forces from traffic. These also do not require transport, sometimes over long distances with a risk of breakage, and are thus often the preferred solution, particularly in remote areas.

Stream culverts: These allow a watercourse to pass under the roadway and may have single- or multiple openings depending on the volume of water they have to accommodate.

Stream culverts can be constructed in several ways, as illustrated in Figure 8-18.

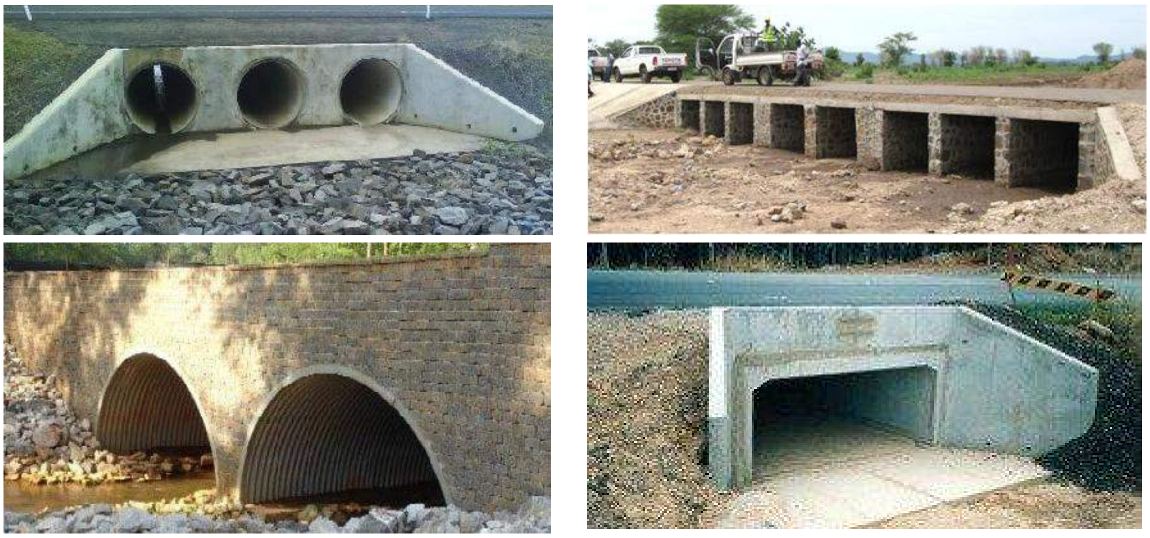


Figure 8-18: Different types of stream culverts

Key features of culverts

The key features of culverts are:

- 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 Guideline.
- 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 a higher gradient or extra maintenance may be required.

Advantages and disadvantages of culverts

The advantages and disadvantages of culverts are summarized in Table 8-14.

Table 8-14: 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 labor 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.

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 adjustments 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.

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 on the road verge or embankment, as shown in Figure 8-19.

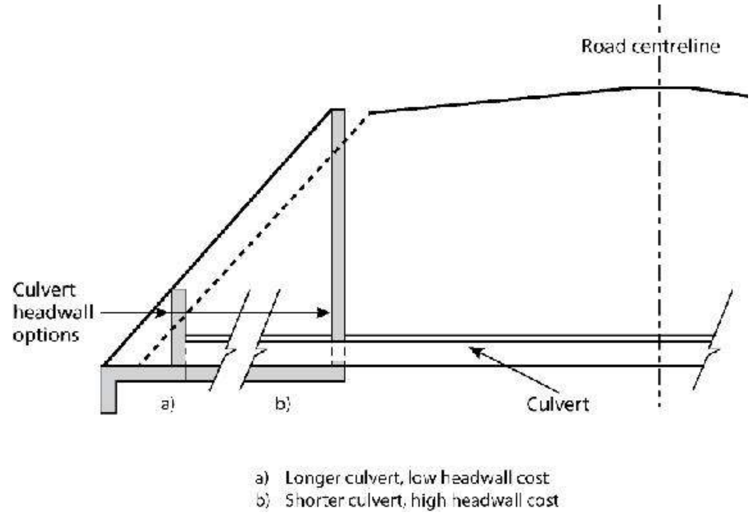
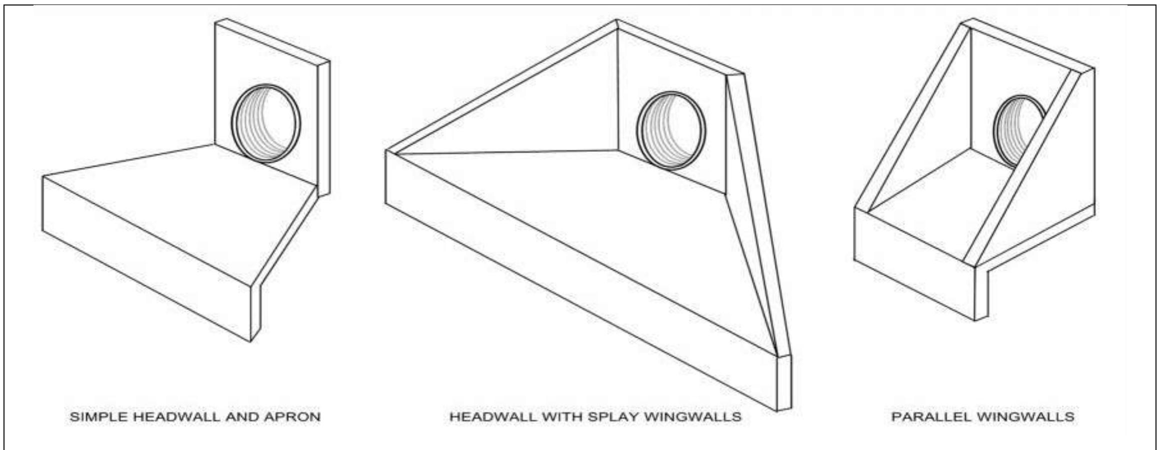


Figure 8-19: Possible culvert headwall positions

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 8-20.



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 8-20: Headwall and wingwall arrangements

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 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.

Pipes transferring large water volumes: One of the most important design rules when constructing a drainage or water crossing 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 scouring. 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.

Protection of culvert outfalls

The location of cross drainage structures on LVRRs is complicated by the fact that they can cause severe erosion 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.

Various methods for erosion protection of culvert outfalls are illustrated in Figure 8-21. Scour and downstream protection of water crossing structures, in general, are dealt with in Sections 8.7 and 8.8, respectively.

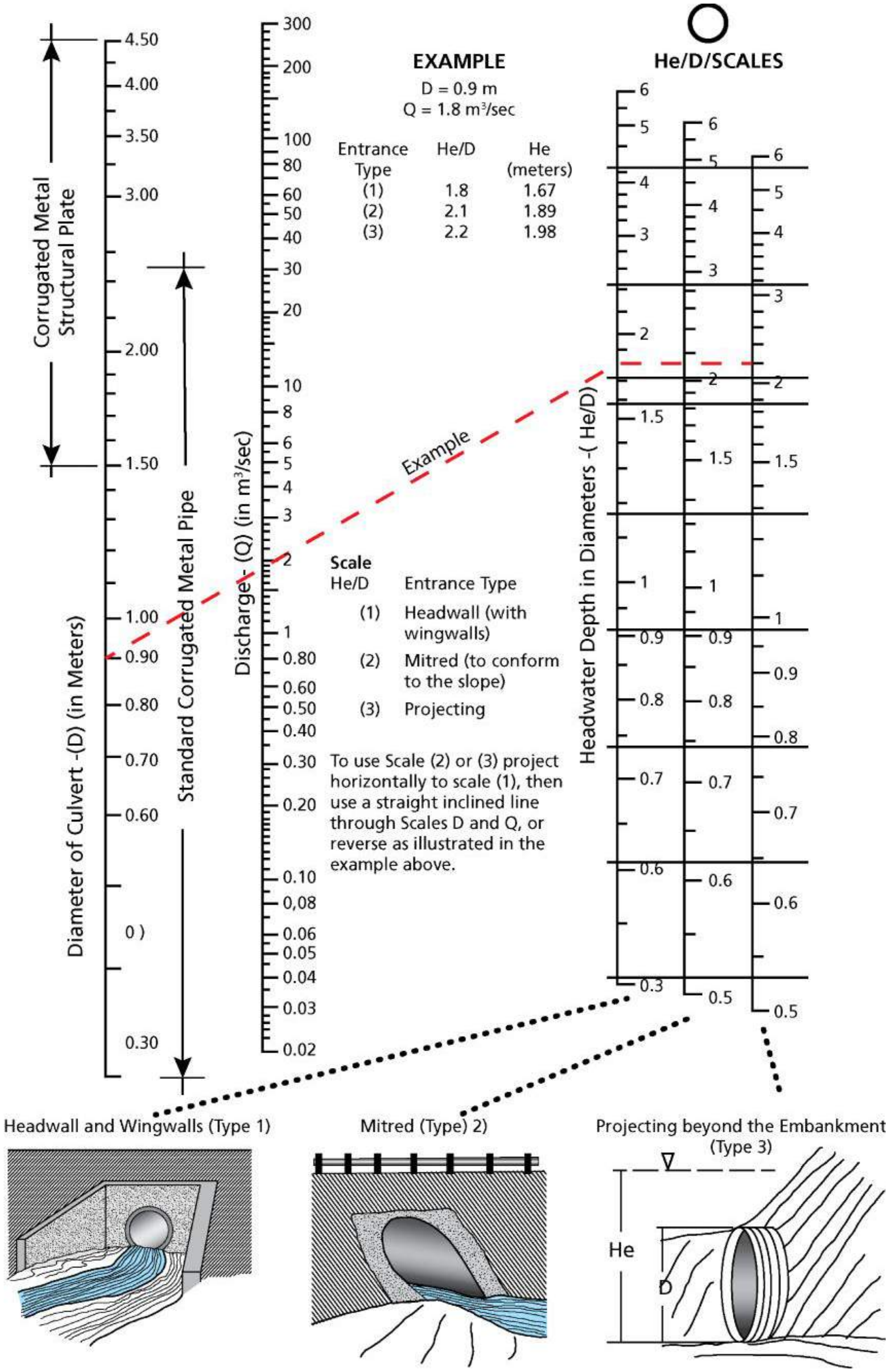
Figure 8-21: Protection of culvert outfall

Estimation of culvert capacity

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 8.3 above and the nomographs in Figures 8-22 (corrugated steel pipes), Figure 8-23 (concrete pipes) or Figure 8-24 (concrete box culverts).

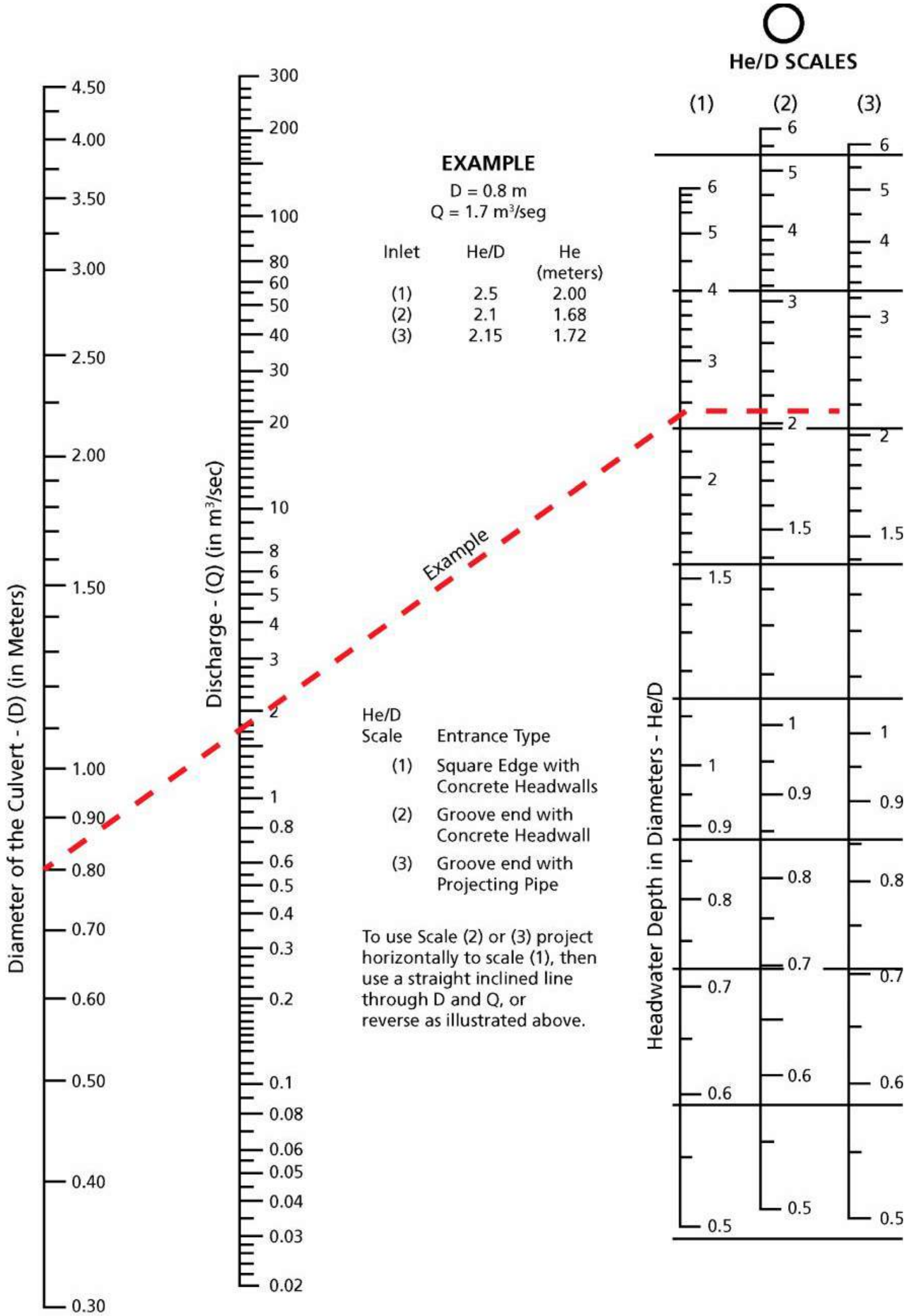
These figures apply to culverts with inlet control where there is no restriction on the downstream flow of the water. Culverts with outlet control, i.e., flow through the culvert is limited by friction between the flowing water and the culvert barrel, are not recommended for use on LVRRs. This is because outlet control will occur only when the culvert is laid on a hydraulically mild slope or if there is some control at tailwater to raise the tailwater level higher than the normal depth. It is recommended to lay the culvert barrel at least at 5% slope, which will always result in an inlet control situation.

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 labeled 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 labeled 'Diameter of Culvert (D) in meters. The culvert size obtained from the nomograph will probably not be one of the standard sizes that are available; hence, the next larger available size should be chosen or the nearest available size if the difference is small (<10 % in diameter).



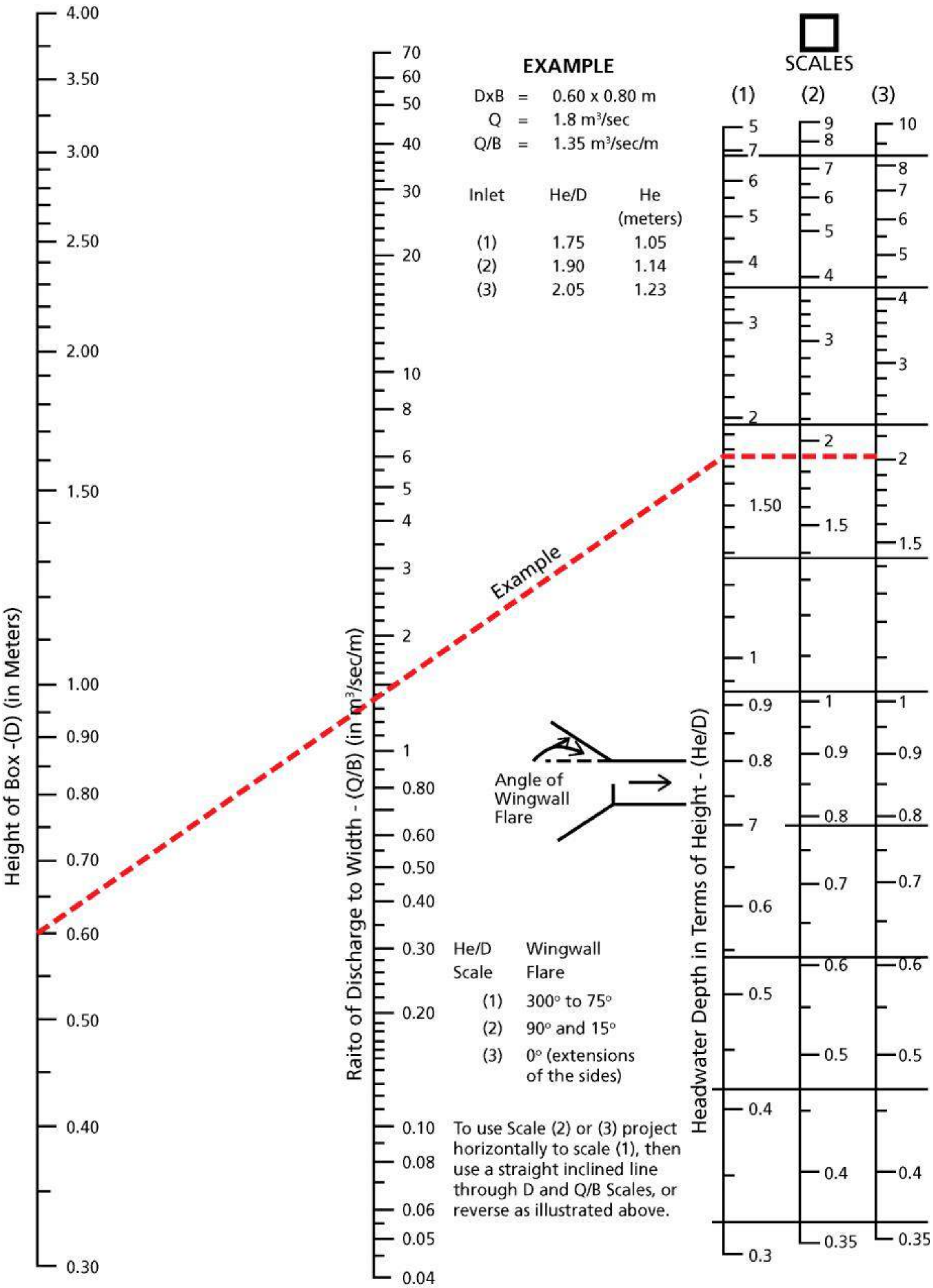
Source: FHWA (2012).

Figure 8-22: Headwater depth and capacity for corrugated pipe culverts with inlet control.



Source: FHWA (2012).

Figure 8-23: Headwater depth and capacity for concrete pipe culverts with inlet control



Source: FHWA (2012)

Figure 8-24: Headwater depth and capacity for concrete box culverts with inlet control

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

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 constructions of culverts 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.
- Arches constructed in wet masonry using a variety of temporary supports.
- Concrete pipes constructed alongside the site using a collapsible mold or bought from local suppliers if the quality is acceptable.
- Metal arches or pipes.

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.

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.

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.

Figure 8-25: Siltation problems at culvert outlet

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.

8.6.2 Masonry Arch Culverts

General

For high water flows, a masonry arch culvert provides greater capacity than a pipe culvert and is sometimes a more appropriate option than a small bridge. A masonry arch culvert is illustrated in Figures 8-26 and 8-27. Their capacity is designed in a similar way to small bridges. For guidance on such structures, refer to *TRL Overseas Road Note 9 – A Design Manual for Small Bridges.*

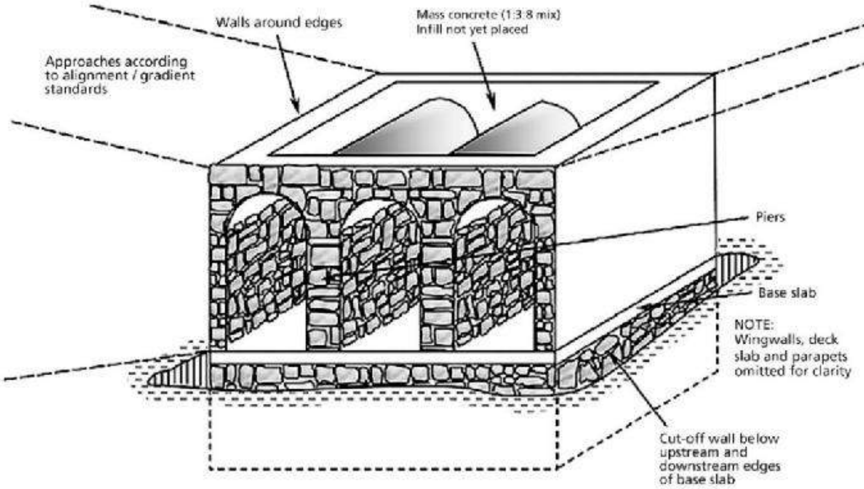


Figure 8-26: Key features of a masonry arch culvert

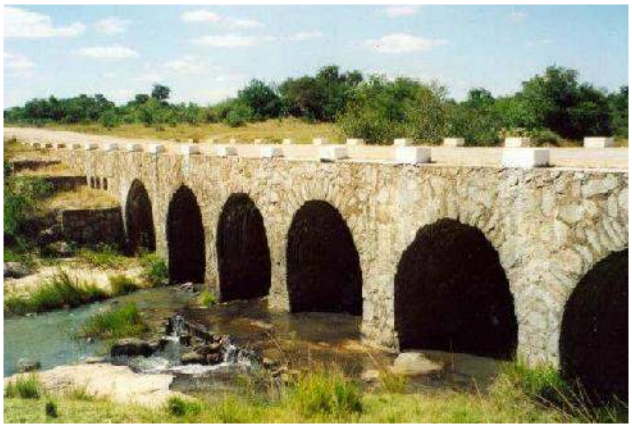


Figure 8-27: Example of a masonry arch culvert

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 channeled into the watercourse away from the structure to prevent erosion of the bank or scour of the culvert structure.

Advantages and disadvantages of masonry arch culverts

The advantages and disadvantages of large arch culverts are summarized in Table 8-15.

Table 8-15: 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 labor 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 Guideline does not cover this type of structure. For guidance on such structures, refer to TRL Overseas Road Note 9.

8.6.3 Embanked Crossings

General

In completely flat terrain where a watercourse 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 8.7 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 conditions 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 8-21 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).

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. Riprap 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 watercourse 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 watercourse 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 one meter 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.

8.6.4 Bridges

General

These are generally the costliest drainage structures. This section covers the arch type, which is the simplest form of bridge, and simply supported bridge types. The Guideline does not cover large or multiple span bridges, which may be simply supported or continuous over piers. For such structures and bridges with lengths more than 10 m, refer to the Overseas Road Note 9.

The key features of a simply supported bridge deck are illustrated in Figure 8-28.

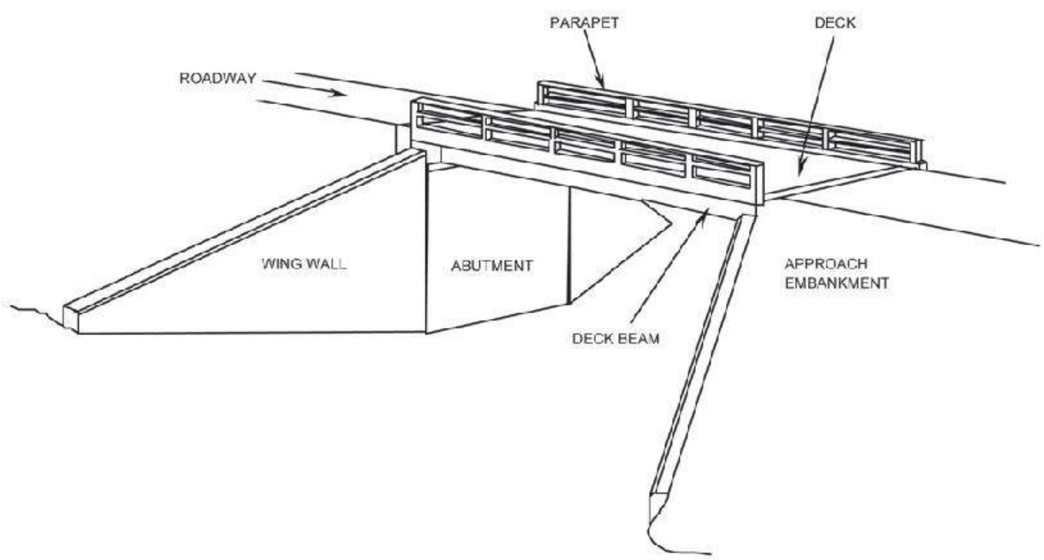


Figure 8-28: Key features of a simply supported bridge deck

Key Features of small bridges (< 10 m length)

Key features are:

- 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.
- 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 groundwater 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 overtopping 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 that are not covered in this Guideline
- Water from the roadside drains should be channeled 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, and 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.

Advantages and disadvantages of small bridges (Lengths <10 m).

The advantages and disadvantages of small bridges are summarized in Table 8-16.

Table 8-16: 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 labor 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 overtopped. • A small amount of scouring and erosion can often result in major damage to structures.

8.7 Scour

8.7.1 General

Scour is the erosion of material from the sides and the bed of the river due to water flow. Damage due to scour is the most likely cause of structural failure of low water crossings, the vast majority of which occurs during flood periods. Minimizing or eliminating the effects of scour should, therefore, receive the most attention when designing any structure. However, erosion and scour are complex subjects that often require specialist advice.

This section provides only general guidance on the types of scour that occur with water crossing structures, mitigation measures to be considered during design for resisting scour and the manner of calculating the depth of scour. More detailed information on the topic of scour and erosion may be obtained from specialists' texts on this subject, as listed in the bibliography.

8.7.2 Types of scour

There are three major types of scour to be considered:

1. **River morphology**: these are long-term changes in the river due to bends and contractions in the channel affecting the shape and course of the channel.
2. **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.
3. **Local scour:** occurs around abutments and piers due to the increased velocity of the water and vortices around these obstructions.

The latter two scour types are the most important to consider when designing a structure.

8.7.3 Designing to Resist Scour

The amount of scour at a structure will be affected by the following factors:

- **Slope, alignment and bed material of the stream**: The amount of scour is dependent on the speed of the water flow and the erodibility of the bed material. Higher water velocities result in more scour.
- **Vegetation in the stream**: Any vegetation growing in the stream can improve the strength of the river bed, reducing scour. The vegetation can also reduce the speed of the water.
- **Depth, velocity and alignment of the flow through the bridge**: The faster the flow, the more scour will occur. If the flow is not parallel to the constriction, more scour will occur on one side of the constriction.
- **Alignment, size, shape and orientation of piers, abutments and other obstructions**: Water is accelerated around these obstructions, creating vortices with high velocities at abrupt edges on the obstruction, increasing the scour depth.
- **Trapped debris**: Debris can restrict the flow of water and cause an increase in water velocity. It is important that structures are designed to minimize the chances of debris being trapped and to ensure that inspections and maintenance are carried out after flood periods to remove any lodged debris.
- **Amount of bed material in the water**: 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 accurately predict the level of scour that may be experienced for a particular design. There are many formulae for predicting the amount of scour around a structure, but these formulae, in general, require detailed knowledge of the river and bed characteristics. They are also based on empirical data and will often give different design scour depths.

Various mitigation measures to be considered during the design include the following:

1. **Choosing locations where the local materials are not scour-susceptible, such as areas of coarse rock or bedrock.**
2. **Armoring the entire channel with materials (grouted gabions, riprap, concrete, etc).**
3. **Provide minimum foundation or cut off wall depths**

Regardless of the required depth for foundations determined by the ground conditions and predicted scour, the minimum foundation depths shown in Table 8-17 should be provided. The depth is measured from the lowest point in the bed of the watercourse at the crossing point. These depths can only be reduced where firm rock is encountered at a shallower depth and the foundations are firmly keyed into the rock.

Table 8-17: Foundation and cut off wall depths

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

Source: Larcher et al, 2010

4. **Create a minimal constriction to the water flow**

The amount of scour experienced at a structure is proportional to the restriction in the normal water flow. If the flow is considered unconstrained, then scour will not exist. If the site conditions permit, the following opening widths should be provided to eliminate the effects of scour.

Table 8-18: Unconstricted width to prevent scour

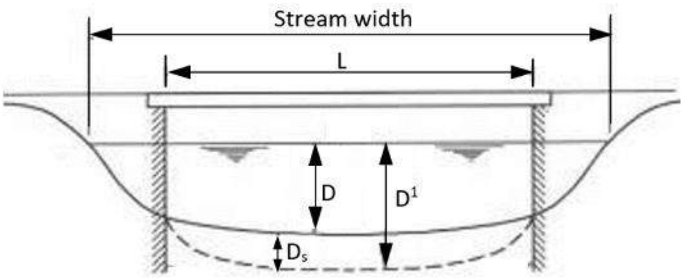
Peak flow rate m^3/s	0.5	1	2	4	6	8	10	15	20	25	30
Minimum width m	3.5	5	7	10	12	14	15	19	21	24	26

Source: Larcher et al, 2010

In some cases, particularly for bridges and larger flows, it will not be possible to provide the opening widths shown in the table above. The design, particularly the level of foundations, should allow for a lowering of the river bed level due to scour. The amount of scour will depend on the following three factors:

- Constricted flow width.
- Maximum flow rate.
- The type of material forming the sides and bottom of the watercourse.

The parameters required for input into the formula for the depth of scour are illustrated in Figure 8-30. This formula is as follows:



W = The original width of the stream

L = The designed waterway when the bridge is assumed to cause constriction

D = The original depth of the stream

D^1 = The expected scour depth under the bridge due to the constriction

D_s = Additional depth of scour due to the constriction

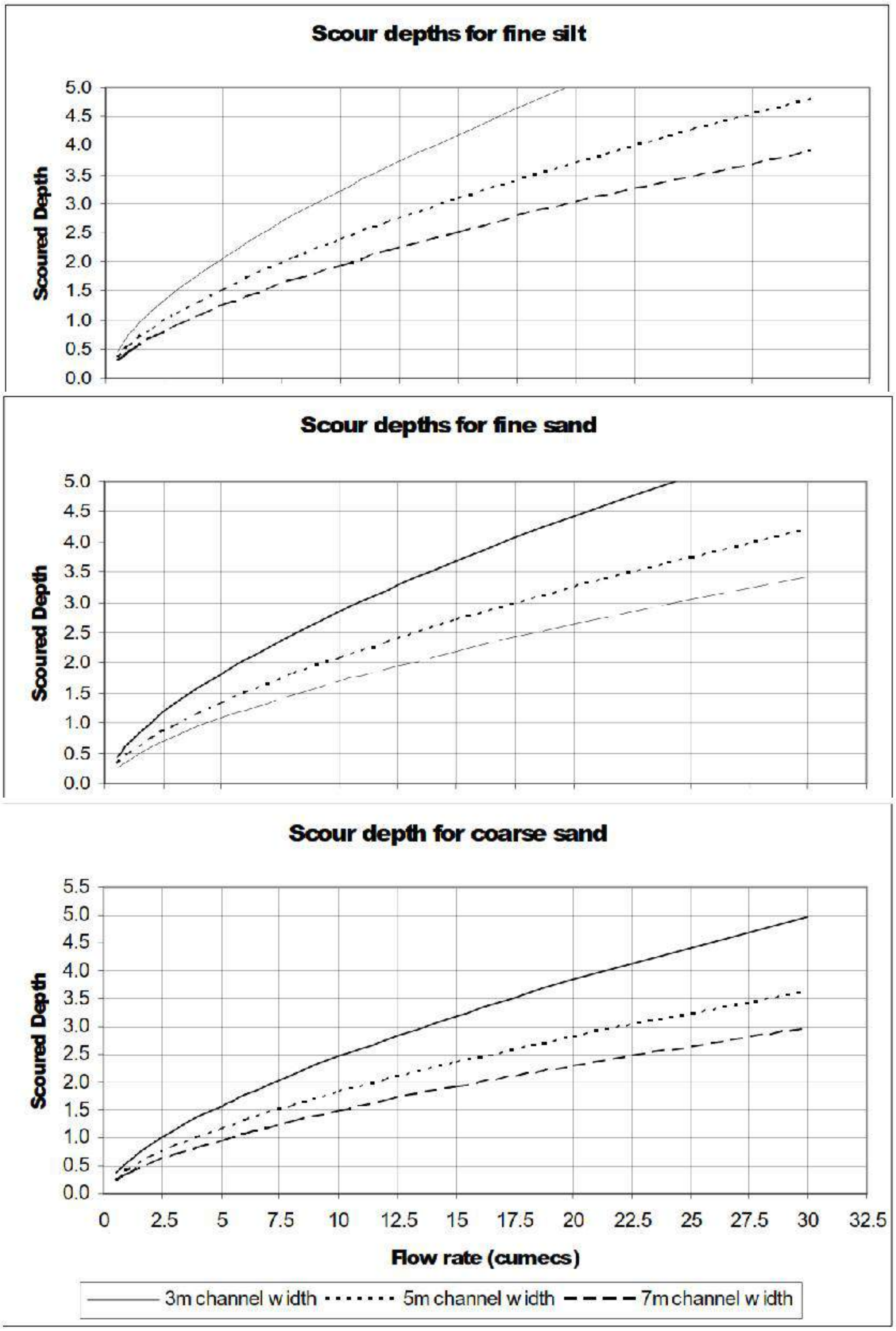
Source: Larcher et al, 2010

Figure 8-29: Depth of flow at constricted water course

Depth of scour = flood water depth at structure - original unconstricted watercourse depth

$D_s = D^1 - D$……………………………………………………………………………………………Equation 8-18

The graphs in Figure 8-31 allow the prediction of the water depth in the channel, which will allow the depth of scour to be calculated.



Source: Larcher et al, 2010

Figure 8-30: Scour depth for various material types

The depth of scour indicates the general level of erosion that will occur in the river bed. Additional local scour will occur near bridge abutments and wing walls and also at the edges of aprons. Table 8-19 shows the factor that the general scour should be multiplied by to calculate the depth of scour that may be encountered near structural elements.

As a general rule, all foundations should be constructed below the predicted depth of scour, which can be predicted by the following formula:

Predicted maximum depth of scour = depth of general scour x local scour multiplier

Table 8-19: Local scour multipliers for structural elements

Local scour at structural element	Local scour multiplier
Long abutments parallel to water flow in straight channels	1.5
Abutments in curving channels and/or part of structures with multiple openings	2.0
Abutments and wingwalls where flow reaches structure at an angle greater than 20 degrees	2.25
End of protective aprons or drift slabs	2.5

Source: Larcher et al, 2010

5. Avoid the use of piers

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

8.8 Downstream Protection

8.8.1 General

Whenever watercourses are channeled 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.

8.8.2 Remedial Measures

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 the cut-off walls for the various structures is shown in Table 8-17.

Table 8-20: 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 scouring. 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. Table 8-17 provides guidance on foundation and cut-off wall depths of various structures. For larger structures, advice should be sought from an experienced engineer.

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 the construction of the culvert. It may be constructed from gabion baskets, cemented masonry or concrete.

Vented drift aprons: The apron for vented drifts 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 drift aprons are the same as culvert aprons.

Figure 8-31: Vented drift with apron

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 8-21.

Table 8-21: 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		

8.8.3 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 that 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 that needs to be imported to or exported from the site. Constructing structures that will not be overtopped may often require large approach ramps/ embankments, as illustrated in Figure 8-30.

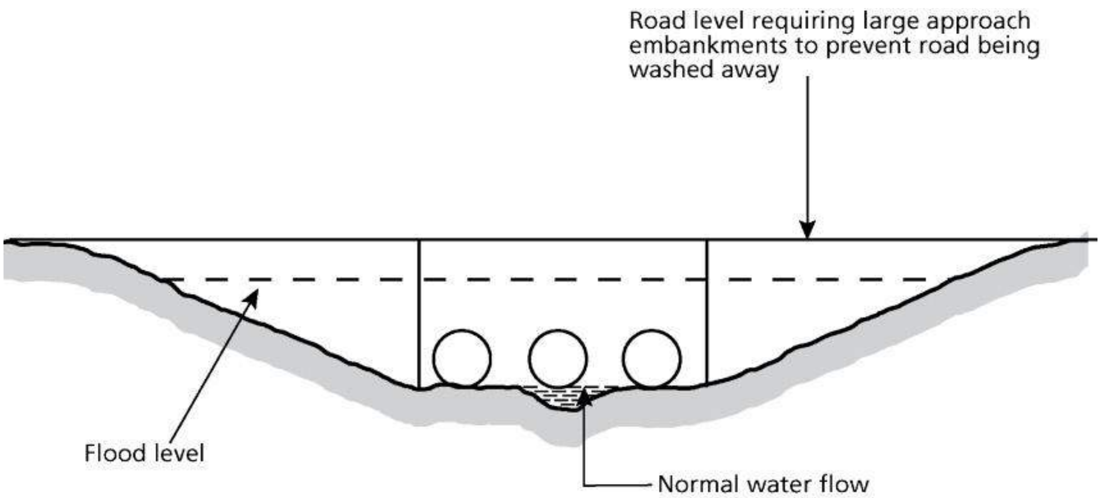


Figure 8-32: Large embankments required to prevent road flooding

Chapter 8 – Hydrology and Drainage Structures

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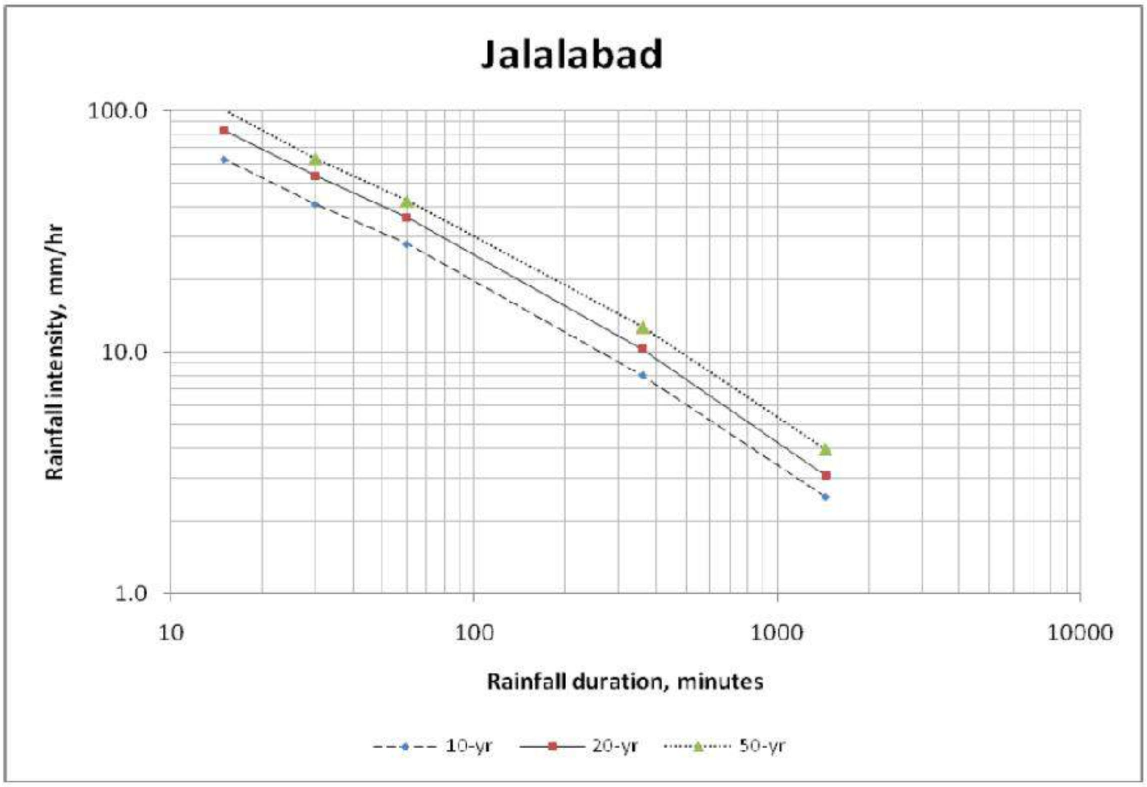
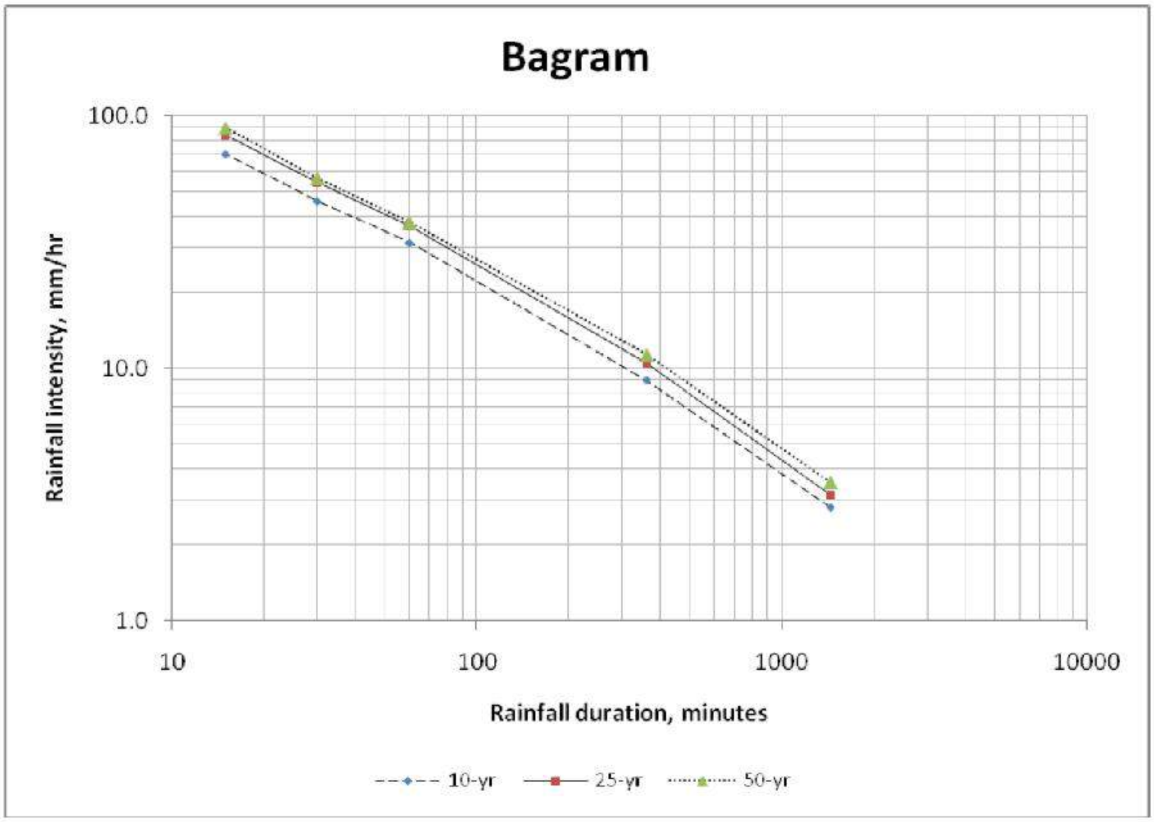
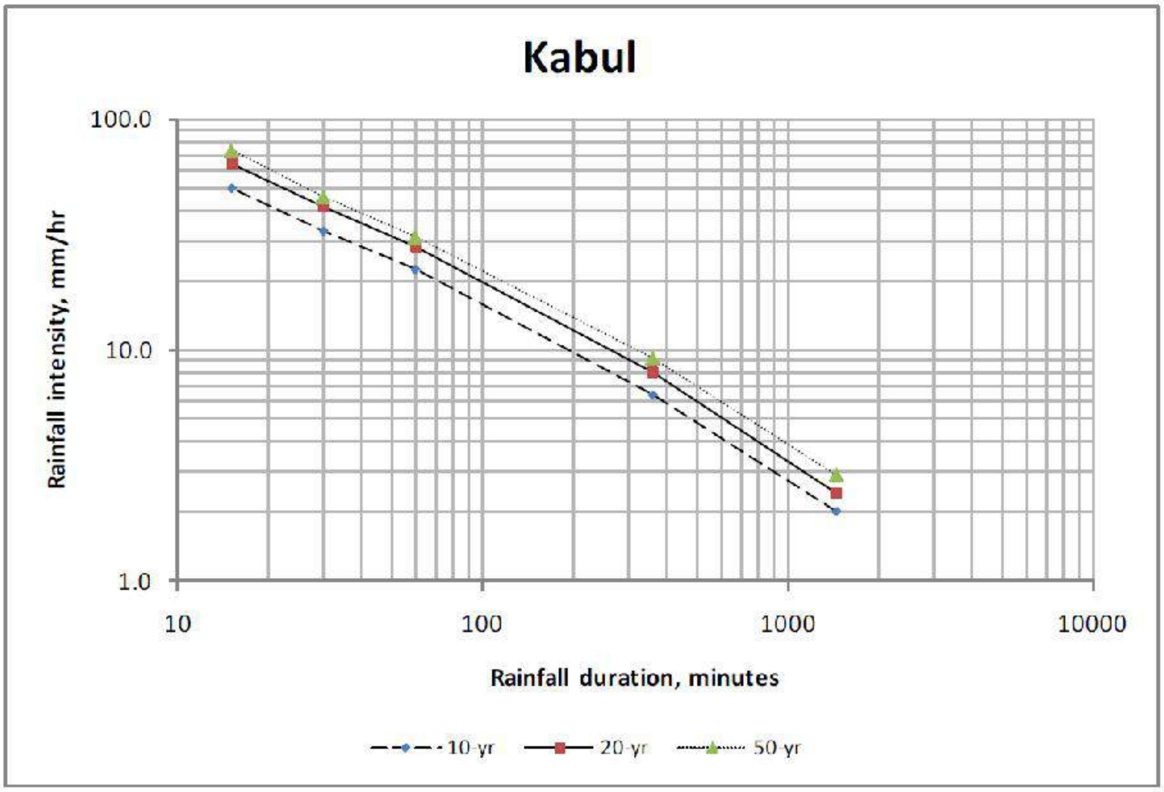
Appendix: Intensity – Duration – Frequency Curves for some locations of Afghanistan

(Source: USACE-AED, 2009)



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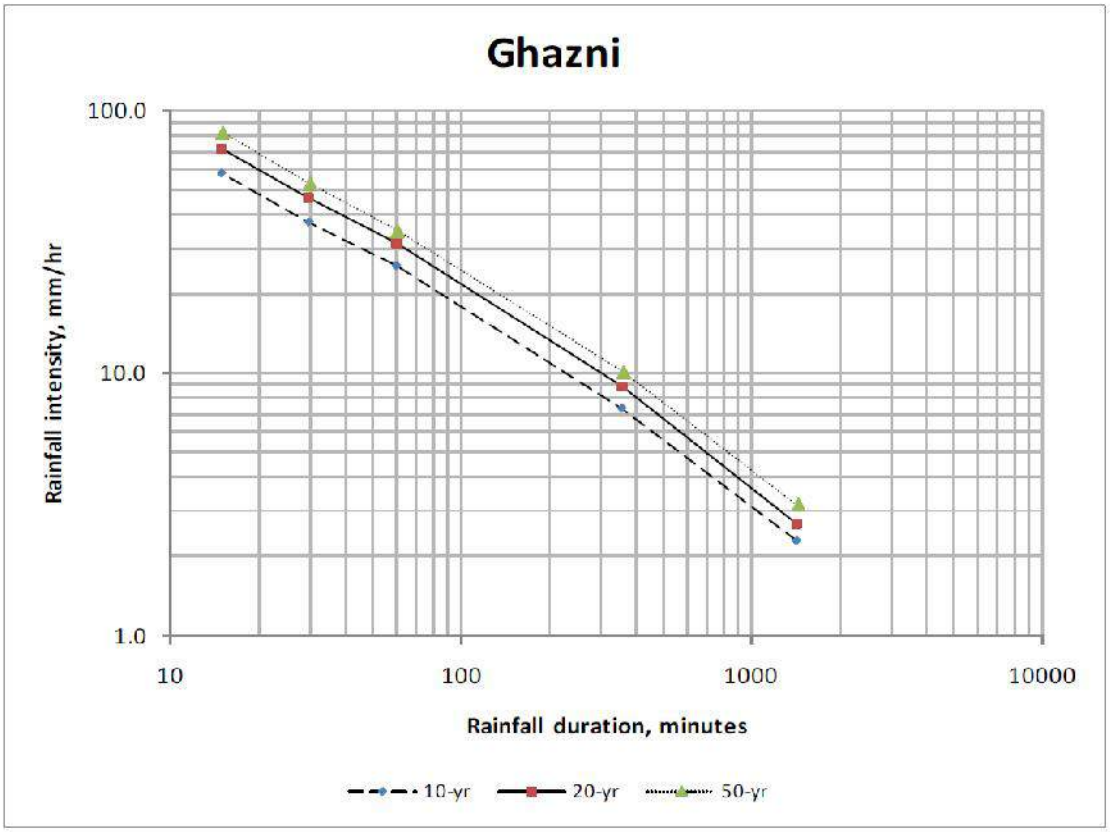
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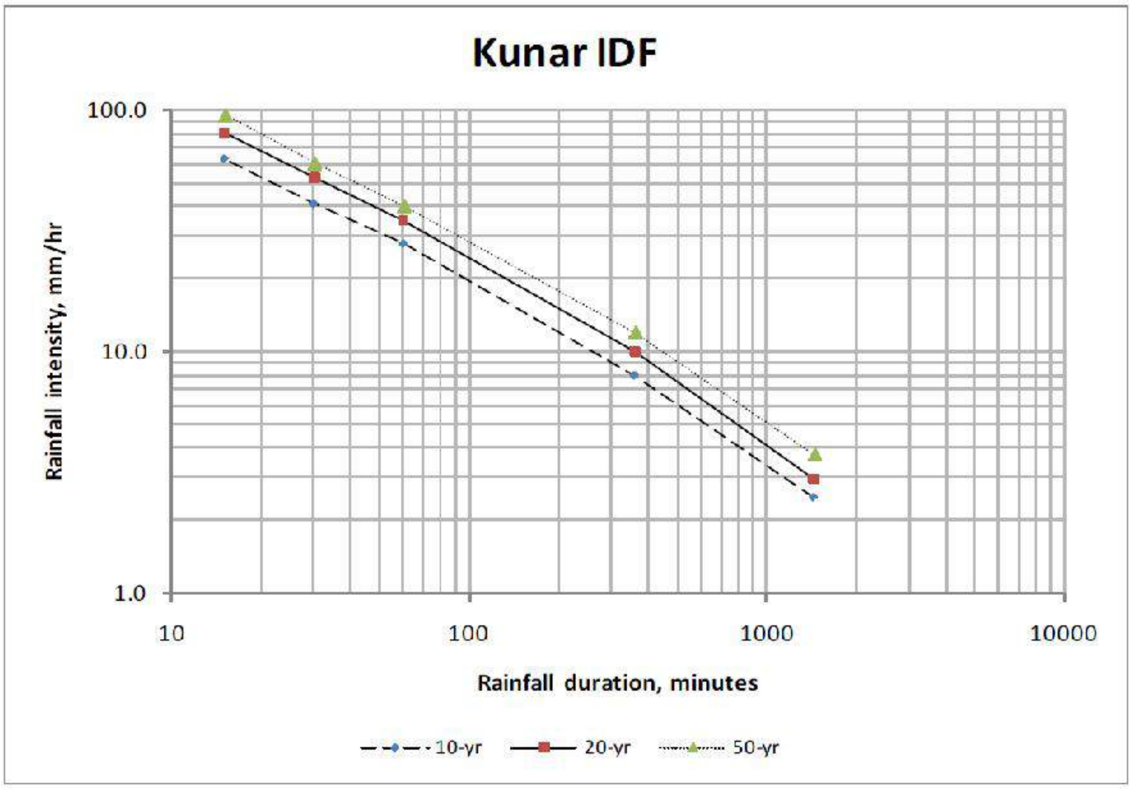
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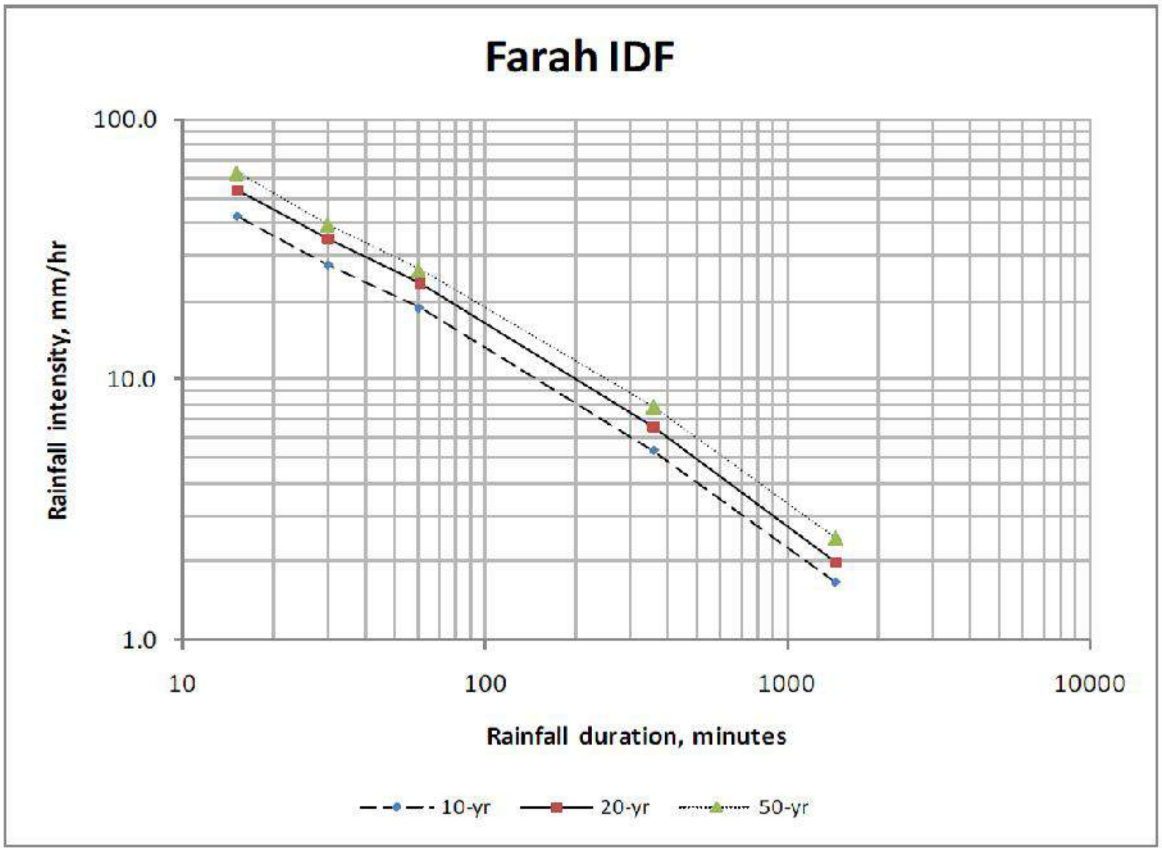
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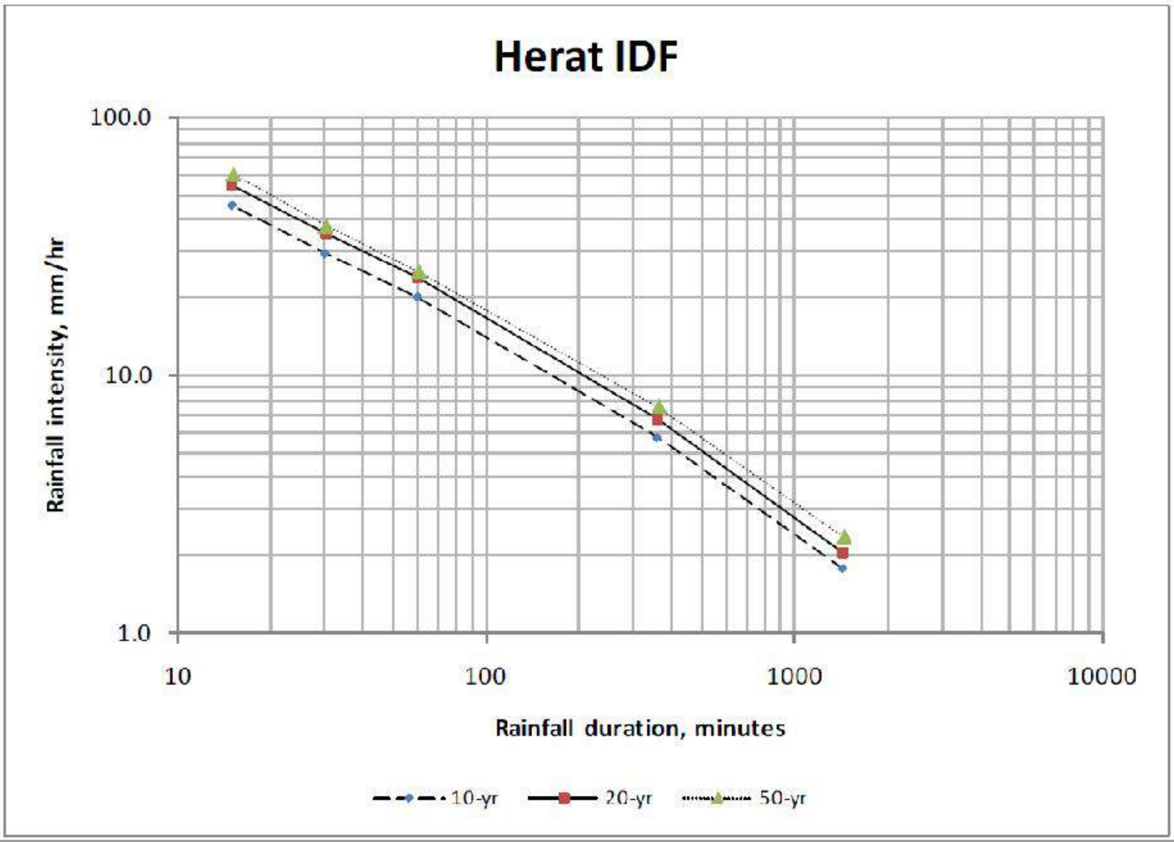
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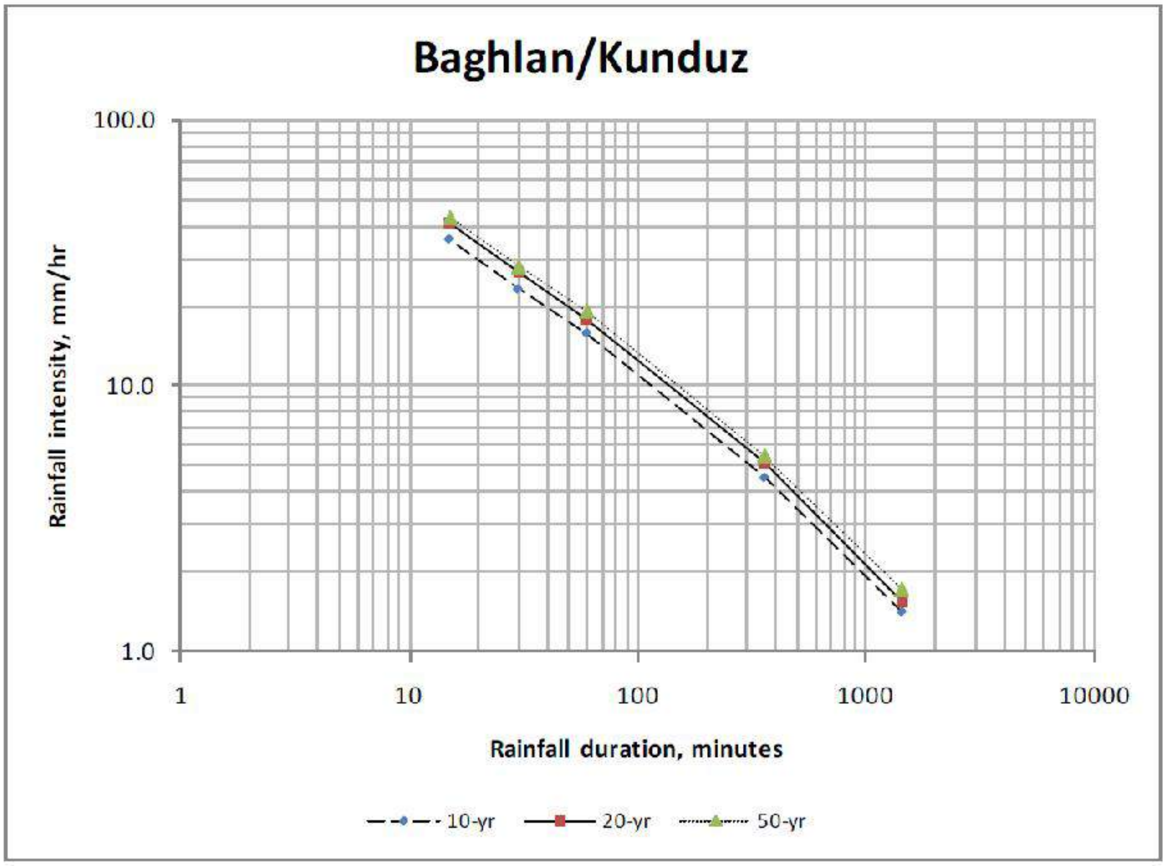
Baghlan/Kunduz
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Rainfall intensity, mm/hr
1
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Rainfall duration, minutes
10-yr
20-yr
50-yr

Low Volume Rural Roads Guideline and Standards
Volume 1 – Pavement Design

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- **Section 2: Investigations**
  - 4. Site Investigations
  - 5. Geoetchnical Investiagtions and Design
  - 6. Materials
  - 7. Traffic
- **Section 3: Design**
  - 8. Hydrology and Drainage Structures
  - 9. Drainage and Erosion Control
  - 10. Structural Design: Paved Roads
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9.1 Introduction

9.1.1 Background

Moisture is the single most important environmental factor affecting pavement performance and long-term maintenance costs of LVRRs. 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 LVRRs 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.

9.1.2 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 8 – Hydrology and Drainage Structures*.

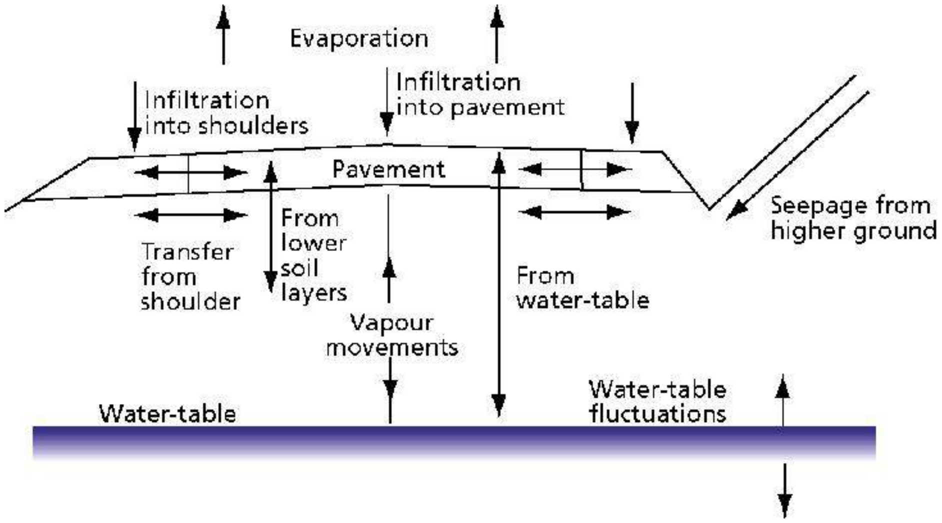
9.2 Sources of Moisture in a Pavement

9.2.1 General

The various causes of water movement into and out of a pavement are listed in Table 9-1 and illustrated in Figure 9-1. The Table highlights those aspects that should be addressed when designing an effective drainage system.

Table 9-1: 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 9-1: Moisture movements in road pavements

9.2.2 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 as effectively as possible. 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 that can be adopted to avoid trapping water within the pavement.

9.3 External Drainage

9.3.1 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 channeling it safely away from the road by means of drainage channels.
- Allowing water to cross the road effectively from one side to the other.

9.3.2 Road Surfacing

The 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 appropriate plasticity for binding the material together (refer to *Chapter 11 – 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 be used for this purpose, some being more impermeable than others. These are addressed in *Chapter 12 – Surfacing.*

9.3.3 Camber/Crossfall

Effective surface drainage is facilitated by ensuring that the road is designed with an appropriate crossfall to drain water from the road surface. Three types of crossfall may be used for this purpose, as illustrated in Figure 9-2.

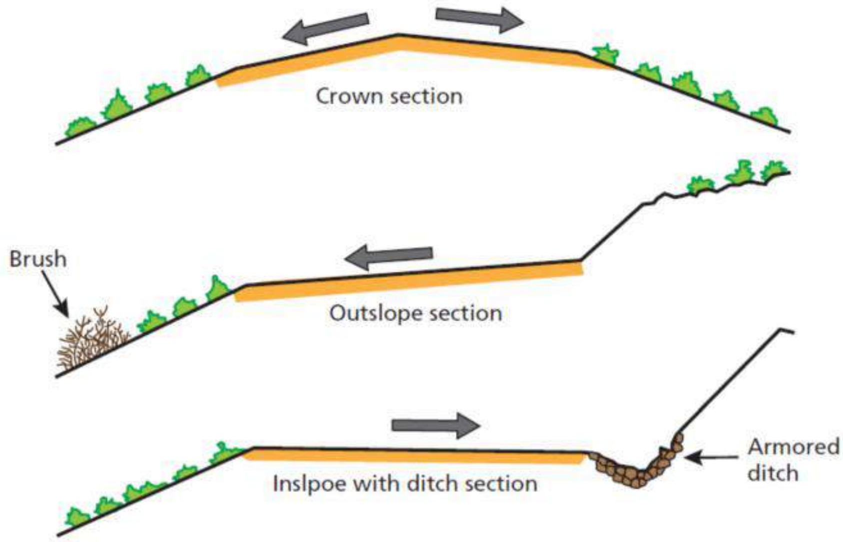


Figure 9-2: Three types of crossfall

Carriageway camber/crossfall slope: The design of the crossfall is often a compromise between the need for a reasonably steep crossfall for drainage and a relatively flat crossfall for driver comfort and safety. The ideal crossfall depends on the pavement surfacing. The carriageway crossfall on straight road sections for various types of pavement surfaces is shown in Table 9-2.

Table 9-2: 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 4%–5% camber would be more suitable to avoid washout 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 labor-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 9-2.

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 9.4). 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 LVRRs. 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 9-3, may be more difficult to achieve in practice than constructing all layers with the same crossfall.

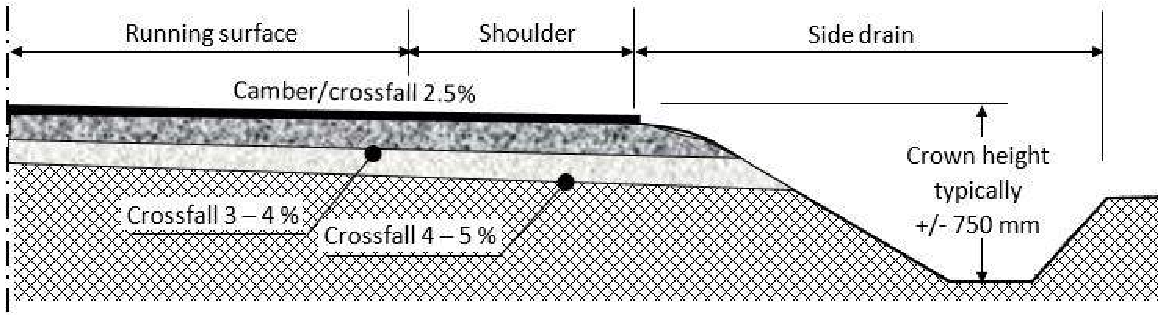


Figure 9-3: Illustrative drainage arrangements

Figure 9-4: 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 9-4, 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 the subbase level. These channels should be back-filled with a material of base quality but which is more permeable than the base itself, and should be given a fall of 1 in 10. Alternatively, a preferable option would be to provide a continuous layer of pervious material of 75 mm – 100 mm thickness laid under the shoulder such that the bottom of the drainage layer is at the level of the top of the subbase, with regular discharge points into the side drain.

9.3.4 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 centerline) 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 9-5). The recommended minimum crown height of 0.75 m applies to unlined drains located in relatively flat ground (longitudinal gradient, g, less than 1%). The recommended values for sloping ground (g > 1%) or where lined drains are used, for example, in urban or peri-urban areas, are shown in Table 9-3. The capacity of the drain should meet the requirements for the design storm return period (*Chapter 8 – Hydrology and Drainage Structures*).

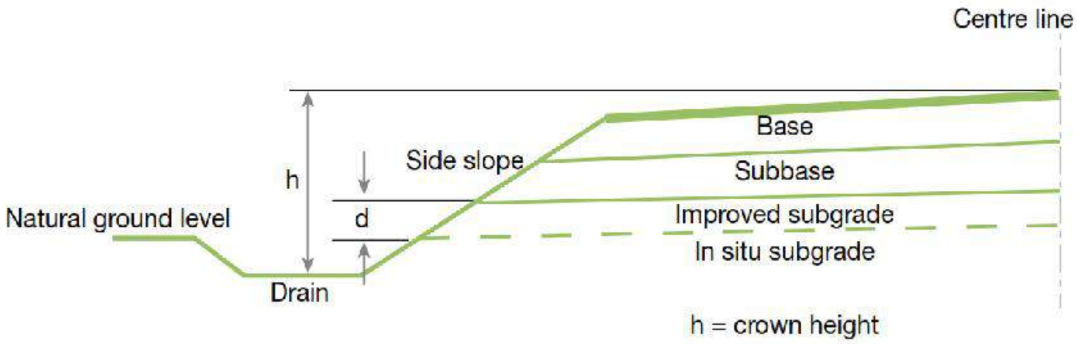


Figure 9-5: Crown height for paved road in relation to depth of drainage ditch

Table 9-3: 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 9-5) to minimize the likelihood of wetting up of this pavement layer from moisture infiltration from the drain.

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

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 the climatic zone, as the soils in engineered roads are generally more moisture sensitive than those in a well-compacted gravel road (see Section 11.2.3).

Gravel roads: The minimum crown height is dependent on the climate and road design class, as shown in Table 9-4. Unless the existing road is well below the 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 the existing ground level. Where necessary, additional fill will have to be imported, or obtained from shallow cuttings, to achieve the required h_{min}.

Table 9-4: 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)
LVRR 5	0.55	0.45
LVRR 4	0.50	0.40
LVRR 3	0.45	0.35
LVRR 2	0.40	0.30
LVRR 1	0.35	0.25

Because of the critical importance of observing the minimum crown height and the 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 9-6.

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 9-6.

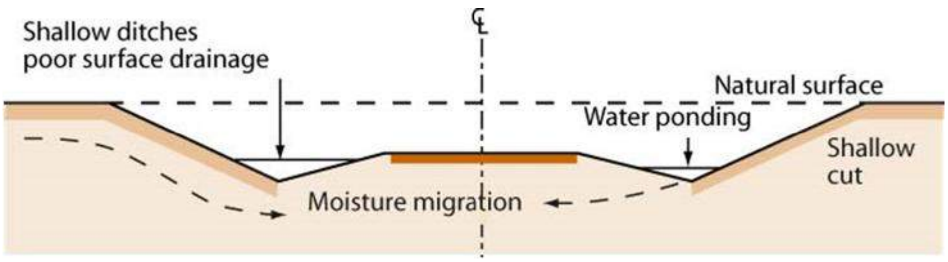


Figure 9-6: Potential drainage problems associated with sunken road profiles

The options for dealing with a sunken profile are presented in Table 9-5.

Table 9-5: Options for dealing with a sunken profile

Option	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 miter drains.	Primary option.	May not be required if water can be discharged satisfactorily through miter 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 options presented in Table 9-5 are discussed below.

Lined ditches: 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.

Parallel drains: In some locations, the volume of water in the side drains may be relieved by constructing additional drains (catch water drains) parallel to the road, and several meters outside of the side drains. These should be 1m wide and excavated to a level just below the side drains. Water should be channeled from the side drain to the parallel drain by constructing miter 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, miter 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.

9.3.5 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 9-7). The choice depends on the type of technology (use of graders, labor-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.

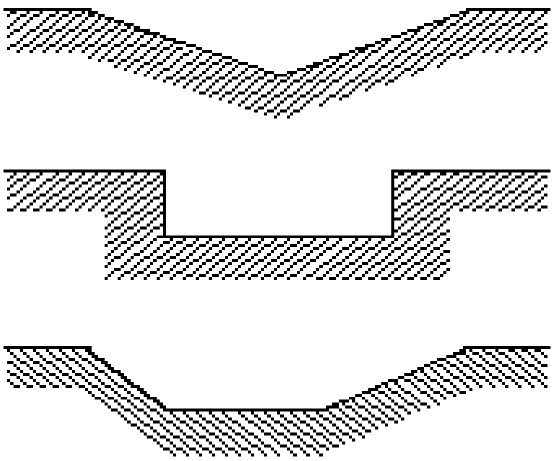


Figure 9-7: Typical types of side drains

The minimum recommended width of the side drain is 500 mm, and the minimum recommended longitudinal gradient is 0.5%. A slackening of the side drain gradient in the lower reaches over significant lengths of the drain should be avoided in order to prevent siltation.

Side drains are normally located beyond the shoulder breakpoint and parallel to the centerline 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 of 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. Table 9-6 shows the recommended values for normal soils. The spacing should be reduced for highly erodible soils.

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.

Table 9-6: 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

Source: Robinson and Thagesen (1996).

Miter drains

These drains are constructed at an angle to the centerline 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 miter drains), and hence the depth of flow, but also by positioning the miter drain so that its toe is virtually parallel to the natural contours. The downstream face of a miter 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 miter drain.

In order to ensure that water flows out of the side drain into the miter drain, a 'block-off' is required, as shown in Figure 9-8.

It is essential that the miter drain is able to discharge all the water from the side drain. If the slope of the miter drain is insufficient, the miter drain needs to be made wide enough to ensure this.

The angle between the miter 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 9-8.

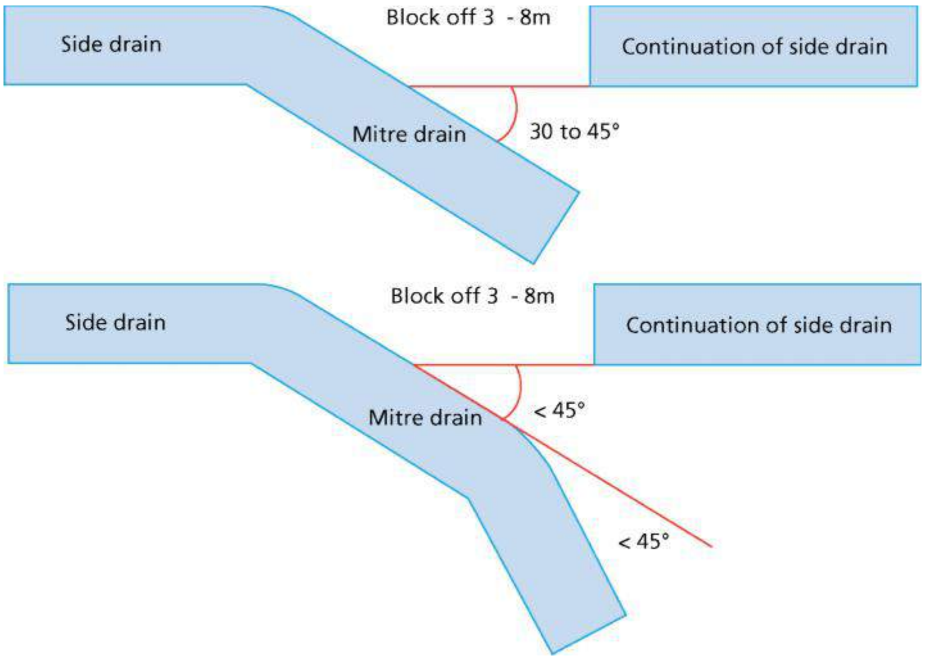


Figure 9-8: Schematic layout of miter drains

The desirable slope of miter 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, miter drains are required at frequent intervals to minimize 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 9-7, the maximum spacing of miter drains is dependent on the road gradient. However, depending on engineering judgment, miter 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 9-7: Maximum spacing of miter drains

Road Gradient (%)	Maximum miter 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 miter 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 relatively wide and shallow, as illustrated typically in Figure 9-9 with sides back-sloped at about 2:1 (vertical:horizontal) or less.

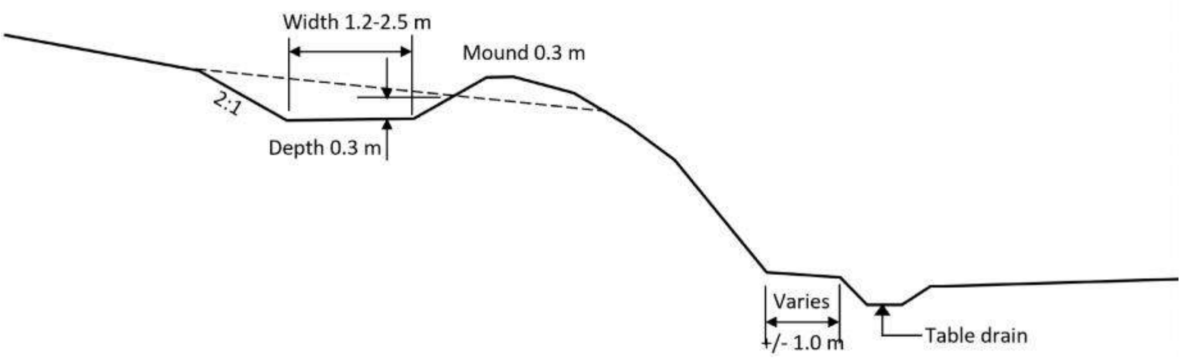


Figure 9-9: Catch-water drain

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 9-10. 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 9-10: 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 energized 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.

9.4 Internal Drainage

9.4.1 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.

9.4.2 Avoiding Permeability Inversion

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

9.4.3 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 9-11 and Figure 9-12. 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. Localized 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 9-13. 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.

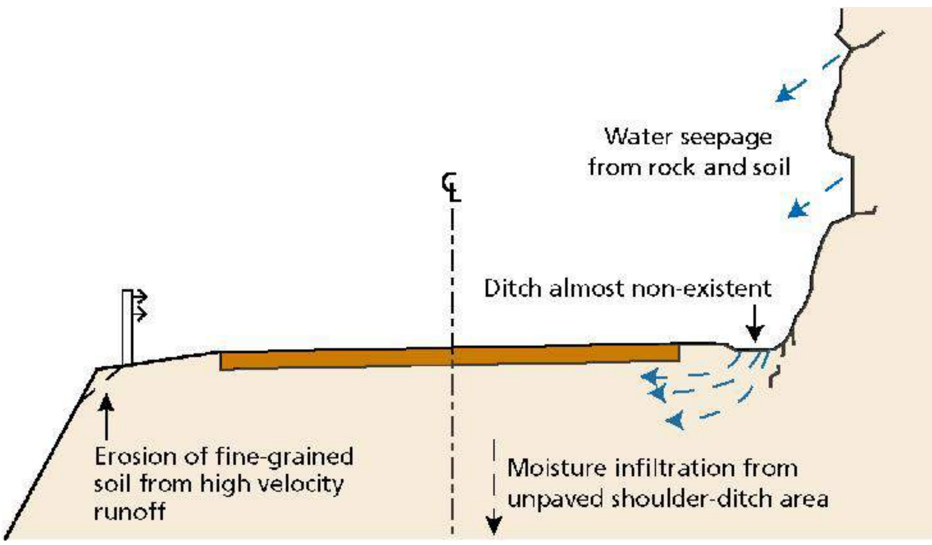


Figure 9-11: Inadequate side drains

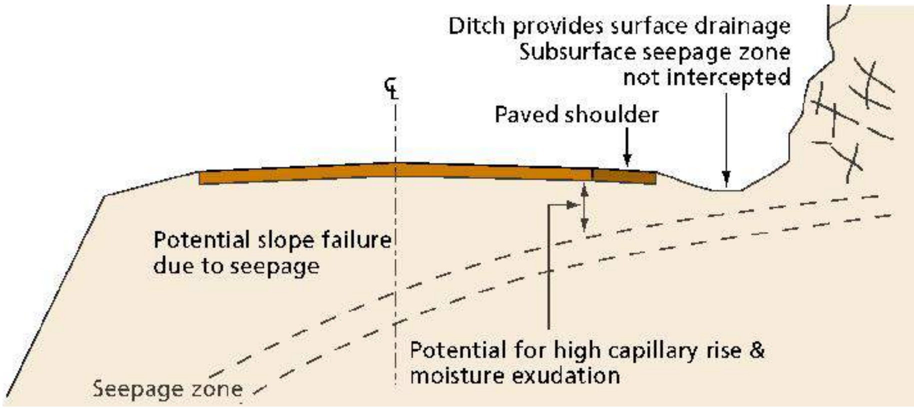


Figure 9-12: Inadequate side drains and subsurface drainage

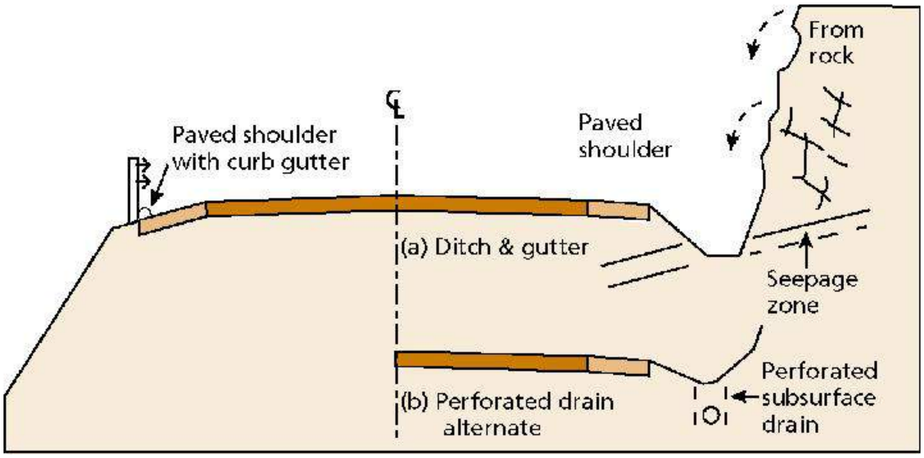


Figure 9-13: Proper interception of surface runoff and subsurface seepage

9.5 Types of Erosion

9.5.1 General

Parts of the drainage system and surrounding terrain are subject to erosion if the quantity and velocity of the flowing water are 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 traveling on the surface of the soil at a velocity that produces stresses in the soil that exceed the cohesion of the soil 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.

9.5.2 Erosion Problems

Erosion problems may be categorized based on the mitigation measures required in the three distinct areas of the road (Figure 9-14) 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 9-14: Erosion areas–upper catchment, road reserve and lower catchment

Upper catchment area

Washouts 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, footpaths 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 the 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 runoff 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 miter drain outfalls.
- Flooding and silt deposition causing damage to crops and property.

9.6 Erosion Control Measures

9.6.1 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.

9.6.2 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 9-15.

Source: Erikson and Kidanu (2010)

Figure 9-15: 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 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 in 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 9-8.

Table 9-8: 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 sidewall 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 stabilizing erosion damages.
Head of gulley	Dam construction downstream of the fall. Sloping and protecting the channel. Construction of drop structures (steps that dissipate the energy of the water flow).

9.6.3 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 the 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 8 – Hydrology and Drainage Structures, Section 8.4*). 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 the unlined ditch in this material can then be determined.

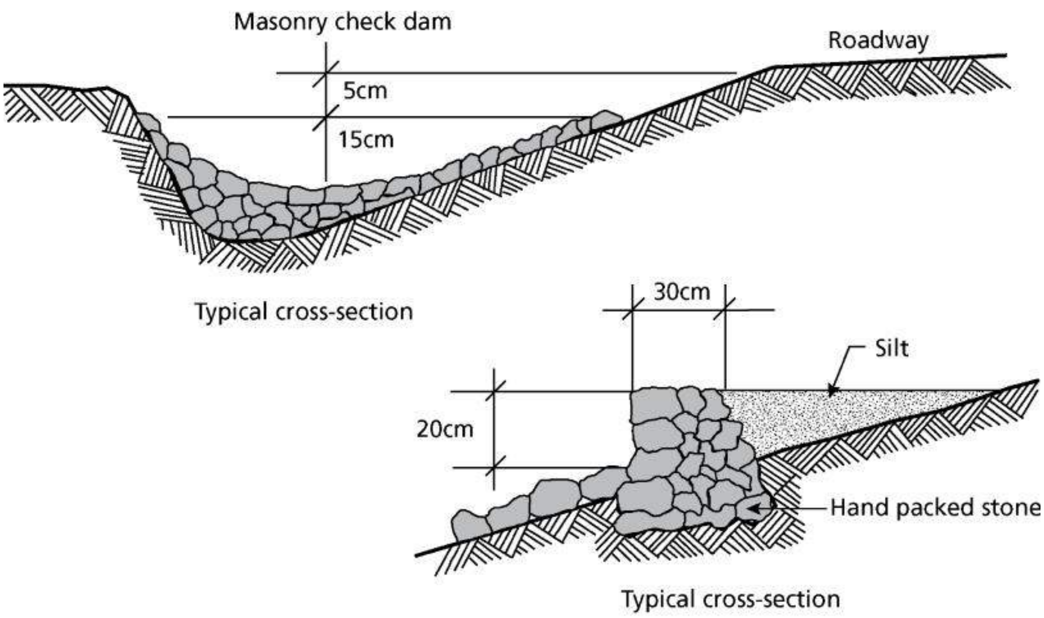
Where flow velocities exceed those shown in Table 9-14 and are likely to cause scouring, the velocity in the drains must be reduced. Methods include:

- 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.

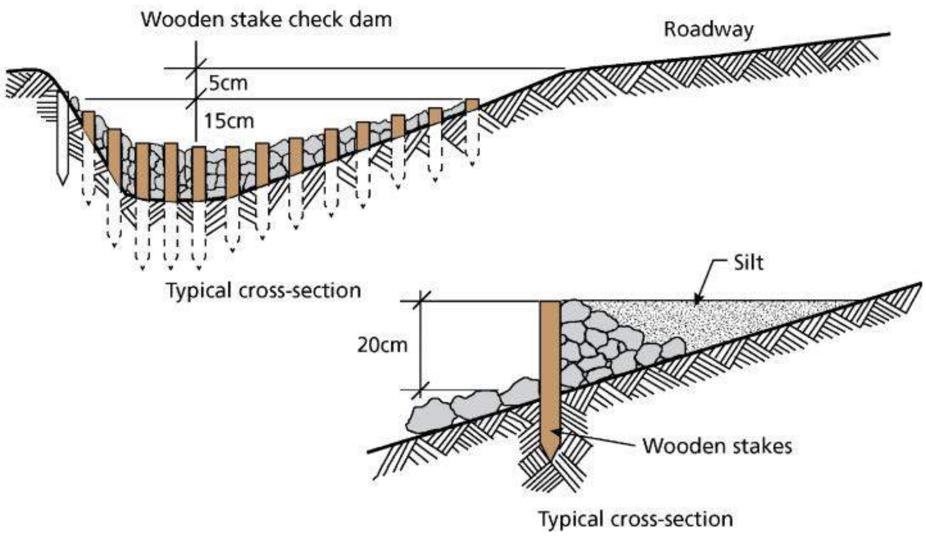
The level of the scour check must be a minimum of 200 mm below the edge of the carriageway in order to avoid the water being diverted out of the side drains.

Typical designs for scour checks are shown in Figures 9-16 and 9-17.



Source: Benhakker et al (1987)

Figure 9-16: Scour checks made from hand-packed stone



Source: Benhakker et al (1987)

Figure 9-17: Scour check made from wooden stakes

The distance between scour checks depends on the road gradient and the erosion potential of the soils. Table 9-5 in Section 9.3.5 shows the 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 the lined side drains is trapezoidal.

Lined Drains

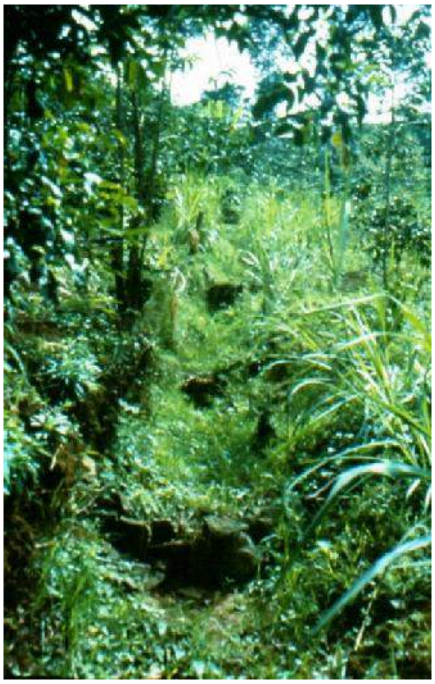
Particularly in steep gradients or where the distance between miter drains is too long, it may be necessary to consider lining the side drains to avoid severe erosion. The drain lining should connect to the edge of the surfacing, as shown in Figure 8-19. Drain lining can be made from mass concrete, concrete 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 9-18: Poor – Gap between surfacing and drain lining

Figure 9-19: Good – Drain lining connected to edge of surfacing

9.6.4 Protection of Outfalls

In principle, unless culverts and miter drains discharge directly into a natural watercourse, onto a non-erodible area, or into water harvesting structures, there is a 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 stabilized 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 9-20. On relatively steep slopes, it may be necessary to stabilize the waterway with check dams, which are similar in principle to scour checks, as shown in Figure 9-20.



Source: Erikson and Kidanu (2010)

Figure 9-20: Grassed waterways with and without checkdams

9.6.5 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 stabilization as well as slope protection in which the principal advantages are:

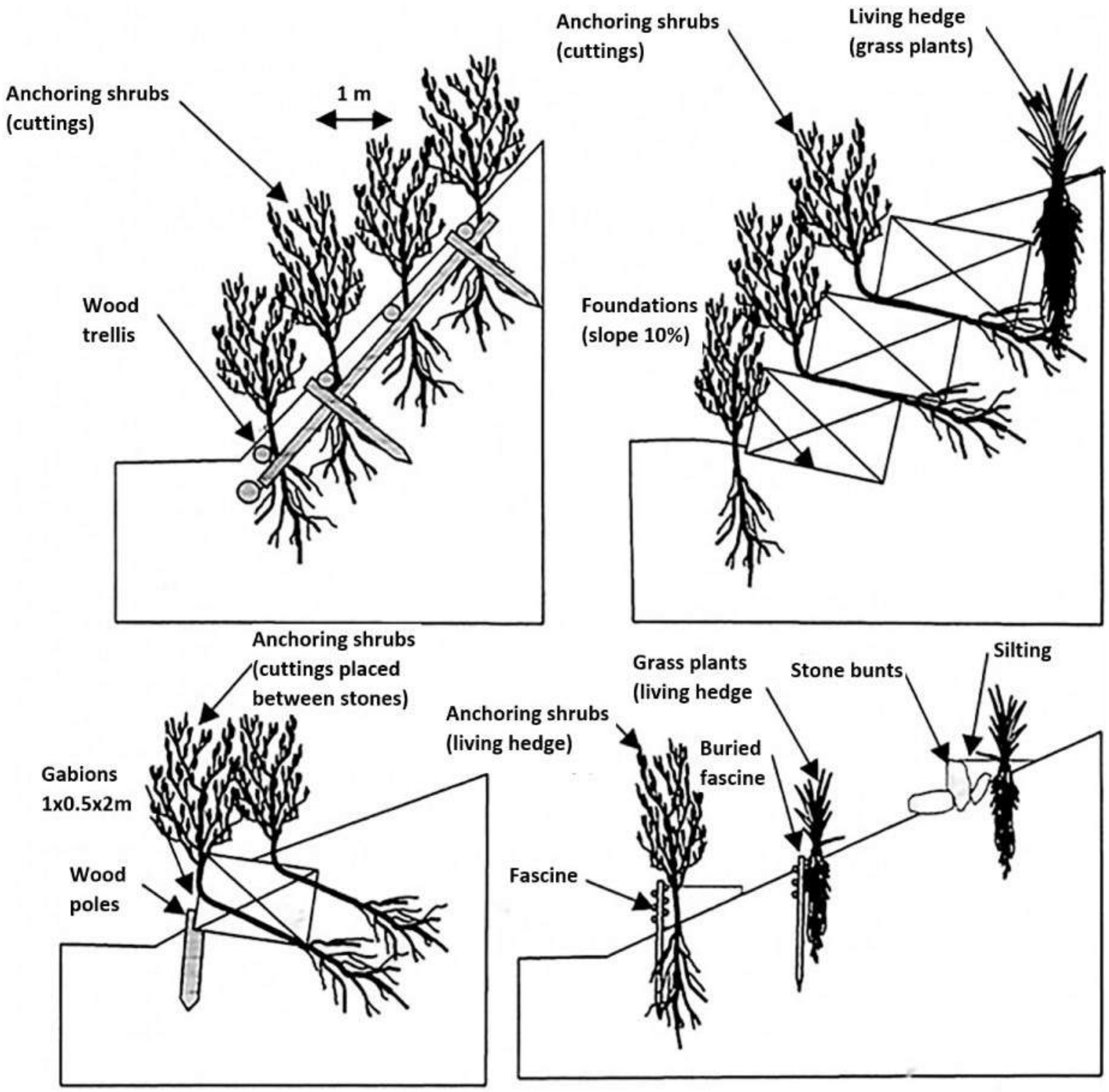
- Vegetation cover protects the soil against rain splash and erosion and prevents the movement of soil particles downslope under the action of gravity.
- Vegetation increases the soil infiltration capacity, helping to reduce the volume of runoff.
- Plant roots bind the soil and can increase resistance 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.
- Taproots 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 stabilizing terraces and gullies.
- The use of trees, shrubs and other grasses to stabilize slopes, protect embankments, and provide live check structures in drains.

Chapter 9 – Drainage and Erosion Control

Figure 9-21 illustrates bio-engineering measures aimed at controlling erosion on moderate slopes.



Source: Lebo and Schelling (2001)

Figure 9-21: 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:
    - o A grass that forms large clumps.
    - o A shrub or small tree that can be grown from woody cuttings.
    - o A shrub or small tree that can grow from seed in rocky sites.
    - o 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 destabilize the whole slope above.

All small protrusions and large unstable 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 8-8 indicates the different types of bio-engineering techniques recommended for various kinds of slopes and soil materials for both cut and fill situations.

Table 9-9: 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 the 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.	
A 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.	Tree planting using seedlings of local species from a nursery.

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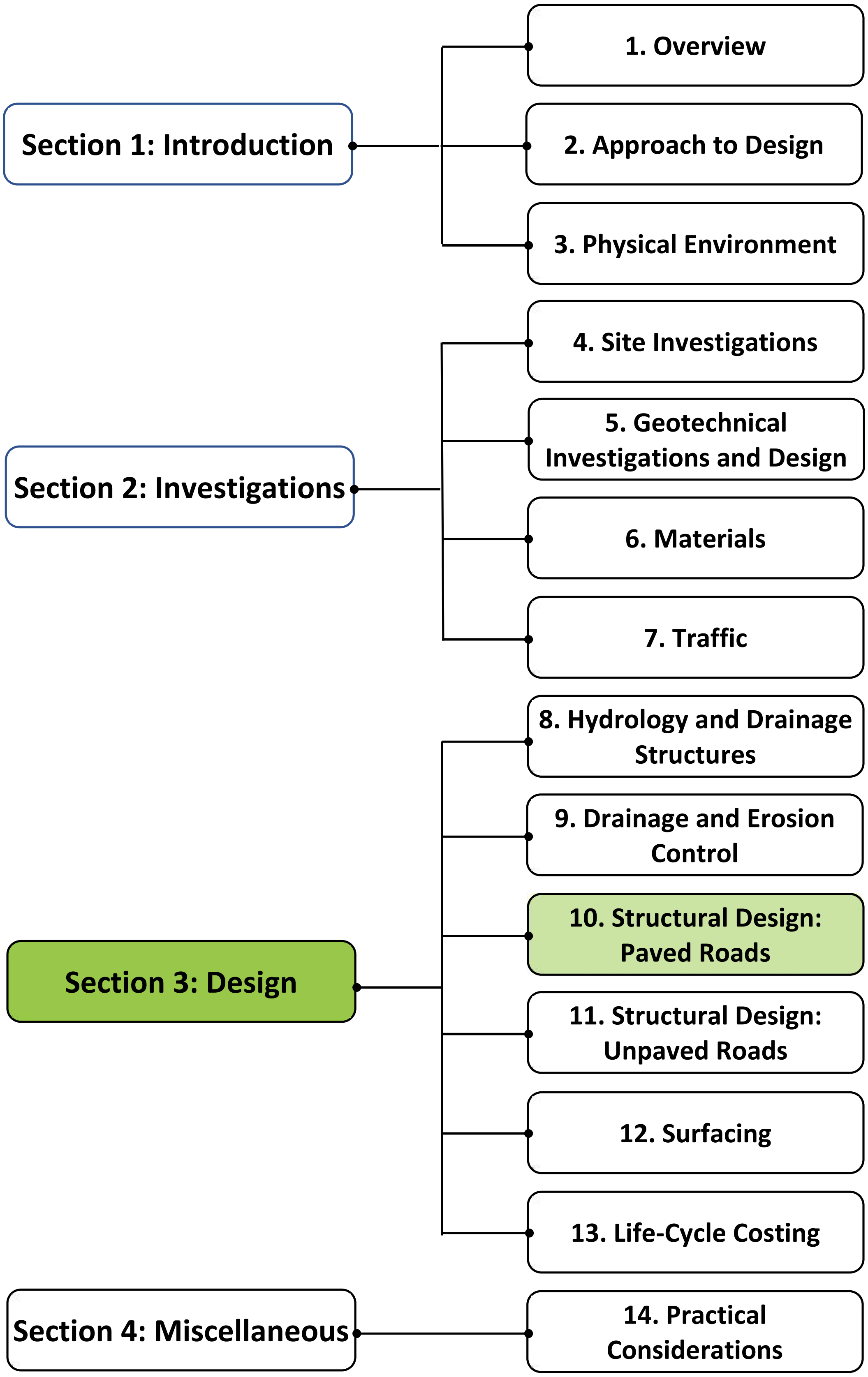

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10.1 Introduction

10.1.1 Background

The objective of pavement design is to produce an economical, well-balanced pavement structure, in terms of material types and layer 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 the performance of any pavement configuration with a reasonable degree of accuracy. 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 LVRRs 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 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 guidelines. 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 LVRRs.

10.1.2 Approach to design

The general approach to the design of LVRRs 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 LVRRs 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 LVRR 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 LVRRs is to provide a pavement that is appropriate to the road environment in which it operates and fulfills its function at minimum life cycle cost at an optimal level of service. However, positive action in the form of timely and appropriate maintenance, as well as adequate control of vehicle overloading will be necessary to ensure that the assumptions of the design phase hold true over the design life of the road.

10.1.3 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 primarily through inter-particle friction and shear strength (as measured with the DCP), and which depends 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 10-1 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 the use of pavement materials of appropriate quality in sufficiently-thick pavement layers.

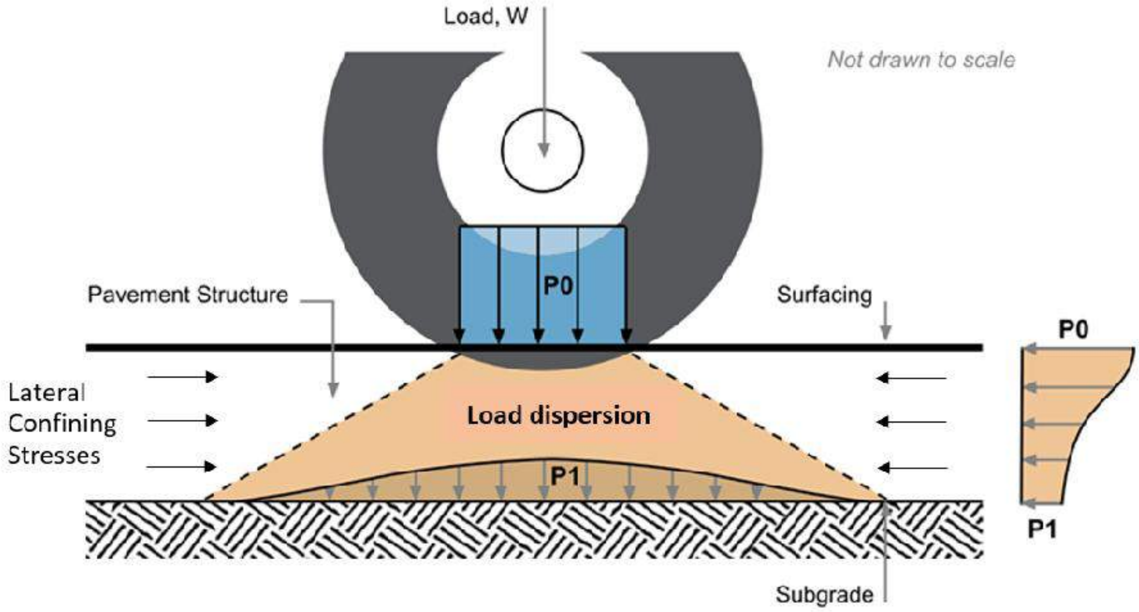


Figure 10-1: Dispersion of surface load through a granular pavement structure

10.1.4 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 two methods of design, both of which are essentially 'catalog' methods, which are the most common methods of design for LVSRs. For each 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 12 - Surfacing*. Such seals do not add any significant structural strength to the pavement and thus do not affect the pavement design.

Section 10.2 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.

10.2 Design of Low Volume Paved Roads

10.2.1 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 the multiple social, economic and environmental requirements of Afghanistan 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 two design methods presented in this chapter.

10.2.2 Design Methods

The two methods of LVSR pavement design addressed in this chapter are as follows:

- The DCP-DN method
- The DCP-CBR method

As illustrated in Figure 10-2, 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).

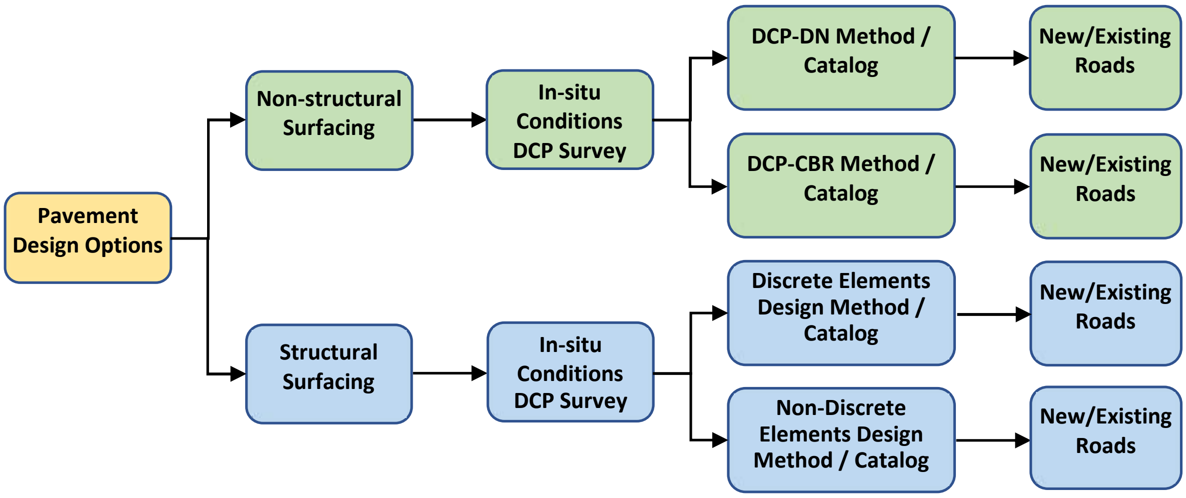


Figure 10-2: Pavement design options available

The design procedure and requirements for each of the two design methods mentioned above are summarized below.

10.2.3 Design Procedure

As with all empirical methods of pavement design, the four main requirements of the design procedure are generally as follows:

- Assessment of design traffic loading.
- Assessment of subgrade strength.
- Selection of pavement materials.
- Determination of pavement layer requirements (thickness and strength).

Apart from the determination of traffic loading, which is generally quite straight forward, as presented in *Chapter 7 – Traffic*, the other aspects of the design procedure vary between the two 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 0 to 10.3.5 for the DCP-DN and DCP-CBR method respectively.

10.3 Design of Roads with Non-structural Surfacings

10.3.1 General

A primary objective of the pavement design of LVSR is to make optimum use of natural-occurring 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 behavior 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.

10.3.2 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 catalog and the associated specifications. A comparison of the existing situation with the required pavement structure then provides the engineer with the information required to design the additions and/or modifications to the structure that is necessary.

In both pavement design methods described in these Guidelines, 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.

10.3.3 Choice of design method

The designer can choose which design method to use or decide to use either of the two methods described in these Guidelines in order to compare the resulting designs and associated costs before the final decision is made on the preferred option.

The primary differences between the three methods are summarized in Table 10-1, and the various issues are discussed in detail later in the chapter.

Table 10-1: Comparison of DCP-DN and DCP-CBR methods

Property	DCP-DN method	DCP-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.
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 catalogs for both methods assume adequate drainage in terms of a minimum crown height above the drain invert (h_{min}) as discussed in *Chapter 9 – Drainage and Erosion Control*.

[1] CUSUM (Cumulative Sum) analysis is a technique used to detect changes in a data set. See page 10-9.

10.3.4 The DCP-DN 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 optimized pavement design (EOD) solutions. The DCP can also be used on site during construction to verify that the design requirements have been achieved.

Design philosophy

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

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

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

Design concepts

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

Layer-strength diagram: Each DCP test provides a profile through the depth of the pavement, which gives an indication of the in-situ properties of the materials in all the pavement layers down to the depth of penetration of 800 mm, as illustrated in Figure 10-3.

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 LVRRs, 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.

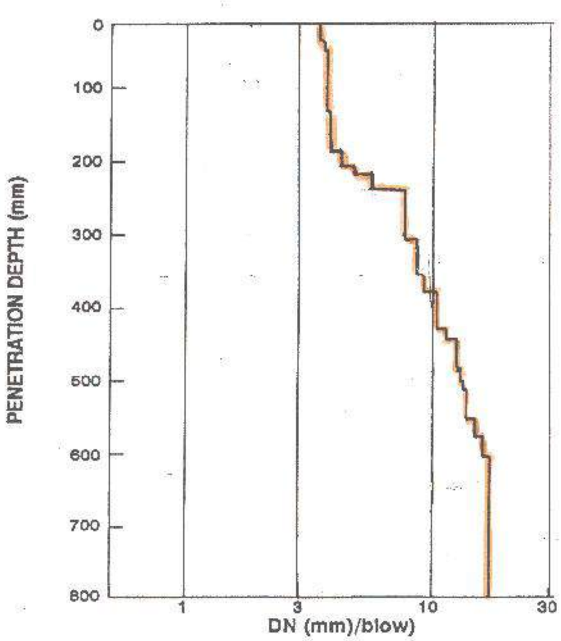


Figure 10-3: Typical profile of DN values with depth (schematic)

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 10-4 shows a typical output of the materials assessment process.

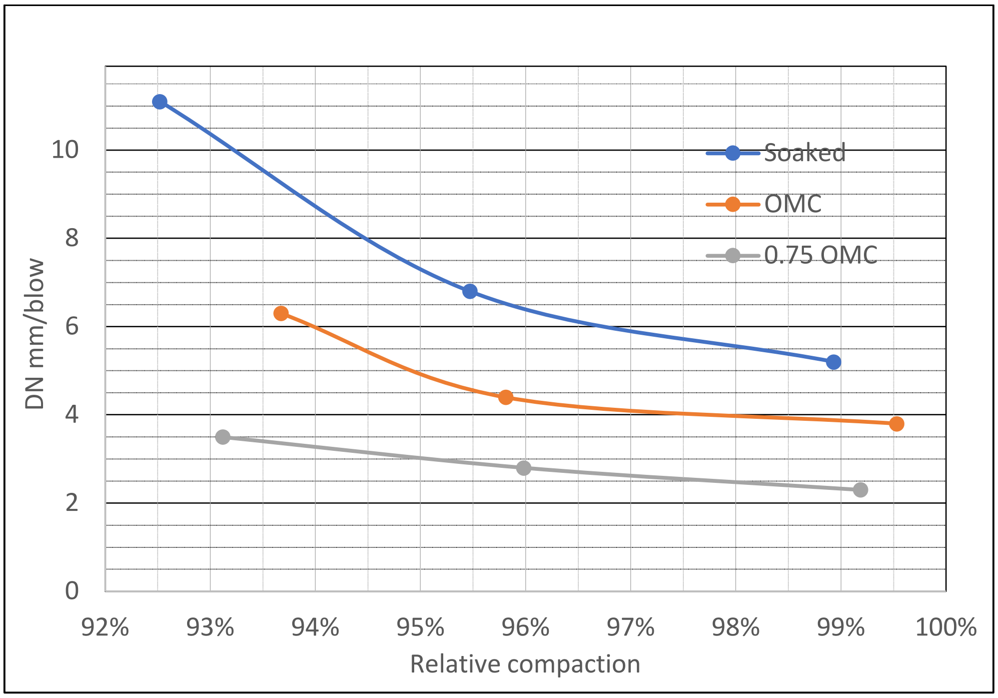


Figure 10-4: DN/density/moisture relationship

Figure 10-4 illustrates how a material's strength, as measured by the laboratory DN value, varies with changes in moisture and density. The gradient of the curves and the separation between them indicate the following about the particular material:

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

It follows, therefore, that:

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

From the output of the materials assessment process illustrated in Figure 10-4, 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.
- Adopting appropriate measures to keep the subgrade and pavement layer materials as dry as possible in service. This can be achieved by the provision of adequate drainage, both external and internal, as discussed in *Chapter 9 – Drainage and Erosion Control.*

The information presented in Figure 10-4 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 10-5 below and explained below.

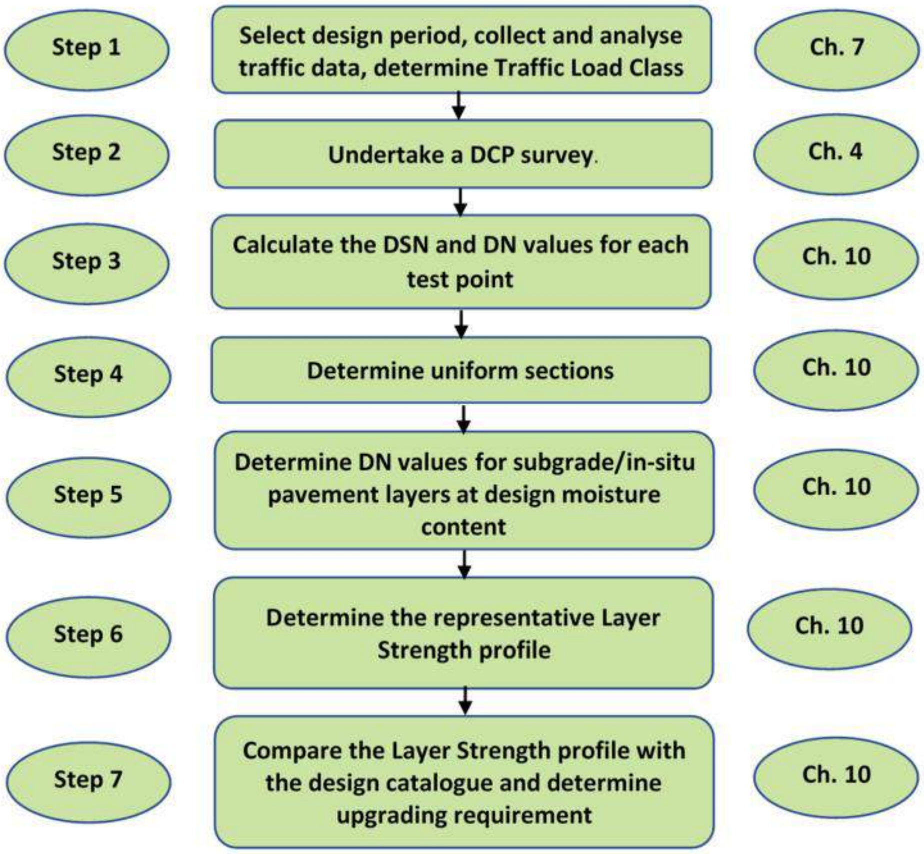


Figure 10-5: DCP-DN design procedure

Step 1: Determine the Traffic Load Class as described in *Chapter 7 – 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[2], the calculation of the following parameters is done automatically:

- The weighted average DN of each 150 mm layer down to a depth of 800 mm. This is the standard configuration of the AfCAP LVRR DCP program, 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.

[2] AfCAP (2016). AfCAP DCP analysis software. http://www.research4cap.org/SitePages/Research.aspx

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 10-2.
- 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 10-6.
- 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 10-2: 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	

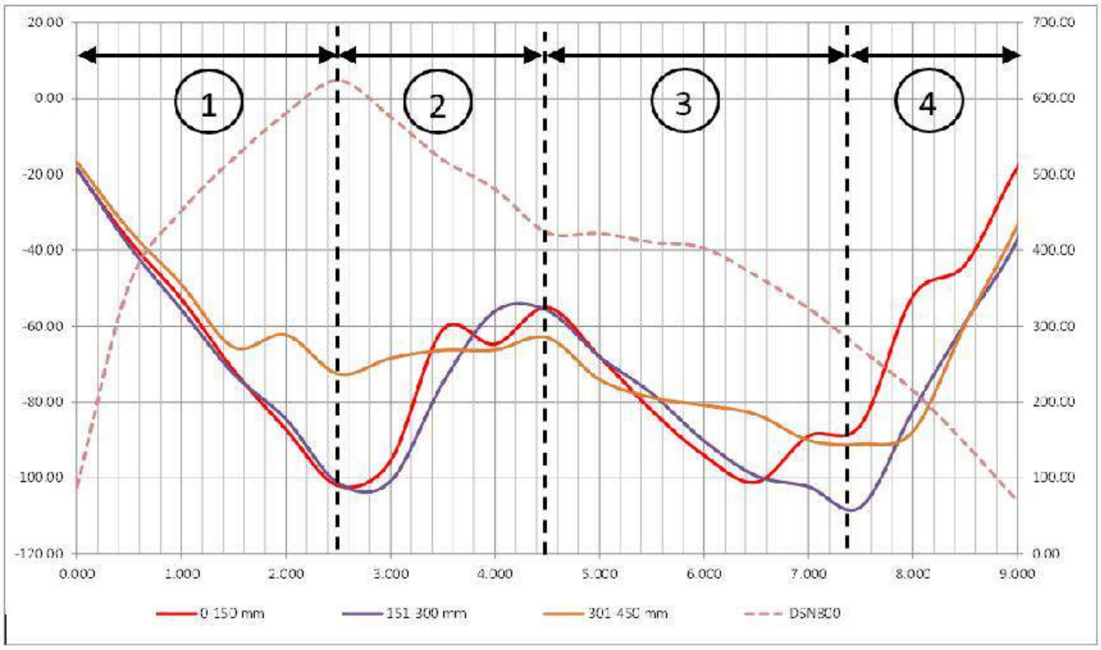


Figure 10-6: Plot of the CUSUM analysis for determination of uniform sections

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 analyzed 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 10-7 and Figure 10-8 illustrates the variability of the DCP results, and the extreme and average values, respectively.

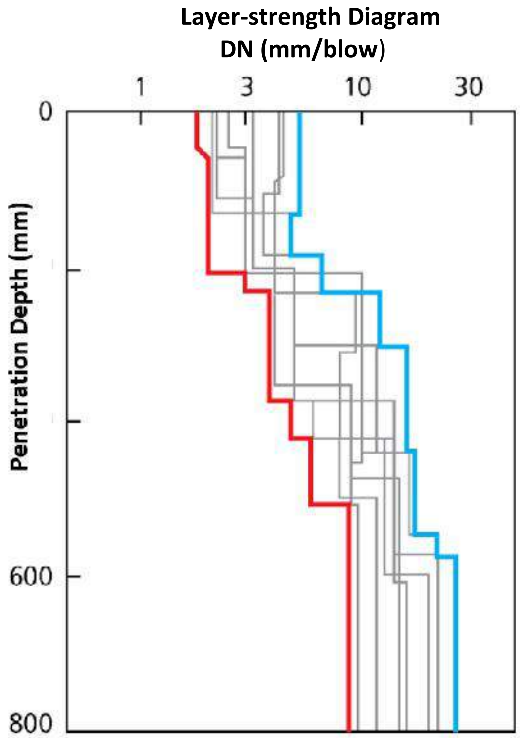


Figure 10-7: Collective DCP strength profile for a uniform section

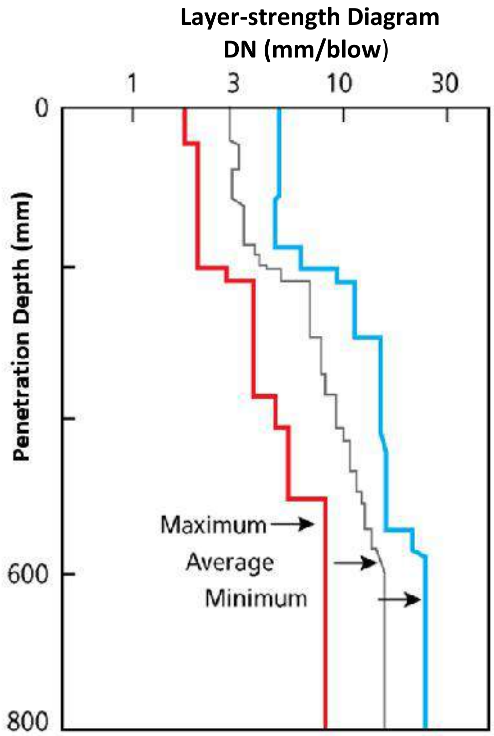


Figure 10-8: Average & extreme DCP strength profiles for a uniform section

The determination of uniform sections, as explained and illustrated above, can also be accomplished from within the AfCAP LVR DCP program.

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 favorable 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 14 – Practical Considerations: Section 14.6 - Compaction)* without degrading the material by breaking down the coarse aggregates.
- The minimum densities will be achieved, as per the DCP-DN Design Catalog.

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 10-3:

Table 10-3: 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 10-4, 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 10-9 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 catalog 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.

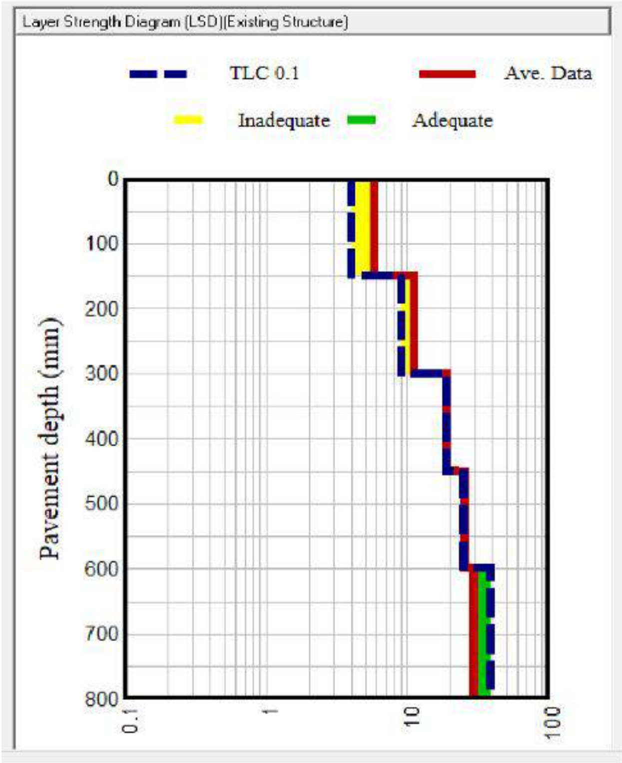


Figure 10-9: Typical output from the analysis of a uniform section

The required strength profile indicated in Figure 10-9 is derived from the DCP-DN structural design catalog shown in Table 10-4 below. This catalog 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 10-4: DCP-DN Design Catalog 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 of 1.2 OMC, whereas the third layer had a moisture content approximately at OMC as shown in Table 10-5. The pink color indicates the layers with insufficient strength, and the green color indicates layers with sufficient strength compared to the requirement of the TLC.

Table 10-5: 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, hence the question marks in Tables 10-5 to 10-8. 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 10-6, are established from the laboratory DN test.

Table 10-6: 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 10-6 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 10-7. 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 10-7: 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 satisfying the design requirements:

1. Rip and recompact the two top layers.
2. Rip and recompact Layer 2 and mechanically stabilize the Layer 1.
3. Import a new base layer with a Laboratory DN value ≤ 4 mm/blow, as shown in Table 10-8.

Table 10-8: 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 catalog shown in Table 10-4, and determine the upgrading requirements.

The design catalog 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 catalog, the upgrading requirements are as shown in Table 10-9.

Table 10-9: 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 Catalog 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 catalog 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 the 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 catalog.

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 catalog 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 stabilization. However, the design and construction requirements of stabilized layers are outside the scope of this Guideline, which focuses on the use of natural, untreated materials.

Assessment of borrow pit materials

As indicated in *Chapter 6 – 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 10-3.

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 judgment related to the outcome of the above tests, bearing in mind the preference for local material use on LVRRs.

(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:

$$GM = \lfloor 300 - (P2 + P425 + P075) \rfloor / 100$$

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 catalog. The pavement layers must be compacted to the highest practicable density, i.e., “compaction to refusal”.

10.3.5 The DCP-CBR method

General

In the DCP-CBR method, the structural designs are obtained directly from a catalog of structures, which is based on the in-situ CBRs derived from the DN values, as explained below. Similar to the DCP-DN method, the DCP-CBR method aims to make optimum use of the in-situ pavement layers, if any, to achieve cost-effective, environmentally optimized designs.

The DCP survey provides the thicknesses and in situ strengths of the subgrade and pavement layers of the existing road along the entire alignment. By converting the DN values to CBR values, a diagram of in-situ CBR versus depth is obtained. The conversion is based on the TRL correlation of DN to CBR, which is:

$$\text{Log}_{10}\text{ CBR} = 2.48 - 1.057\text{ Log}_{10}\text{ DN, or}$$

$$\text{DN} = 10^{(2.48\text{-LogCBR})}/1.057$$

For example, for a CBR of 145, the calculation is as follows:

$$\begin{aligned} \text{DN} &= 10^{(2.48\text{-Log }145)}/1.057 \\ &= 10^{(2.48-2.161)}/1.057 \\ &= 10^{0.319}/1.057 \\ &= 2.084/1.057 = 2.0 \end{aligned}$$

The analysis of the DCP data, using the UK DCP program, provides the overall strength of the pavement at each test point expressed as the Structural Number (SN), which is similar to the DCP Structural Number, DSN_{800}, used in the DCP-DN method.

In practice, two different scenarios can be encountered:

- For a new road, the design is quite simple since there are no existing structural pavement layers, and only the design subgrade strength is required. The required pavement structure can then be selected from the catalog for the respective subgrade and traffic load class.
- For an existing road, e.g., a gravel road with an intact gravel wearing course, the design strength of both the subgrade and the existing pavement layer(s) is required. For the respective subgrade and traffic load class, the existing pavement layer strength and thickness must be assessed against the requirement of the catalog and augmented, as required.

In both cases, the DN values from the DCP survey are converted to in-situ DCP-CBR values using the UK DCP program. The in-situ DCP-CBR values are then converted to soaked CBR values using an empirical relationship between in-situ CBR values at various in-situ moisture contents for various material types. The soaked CBR values, and structural layer thicknesses, if any, are then compared to the requirements of the catalog.

The subgrade is classified using the standard soaked CBR value to provide a subgrade class. The design catalogs for roads with non-structural bituminous surfacings show different pavement structures expressed in terms of the required layer thicknesses and soaked CBR values based on two climatic zones for the same subgrade class, as shown in Tables 10-12 and Table 10-13.

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 for either new or existing roads is shown in Figure 10-10 and explained below.

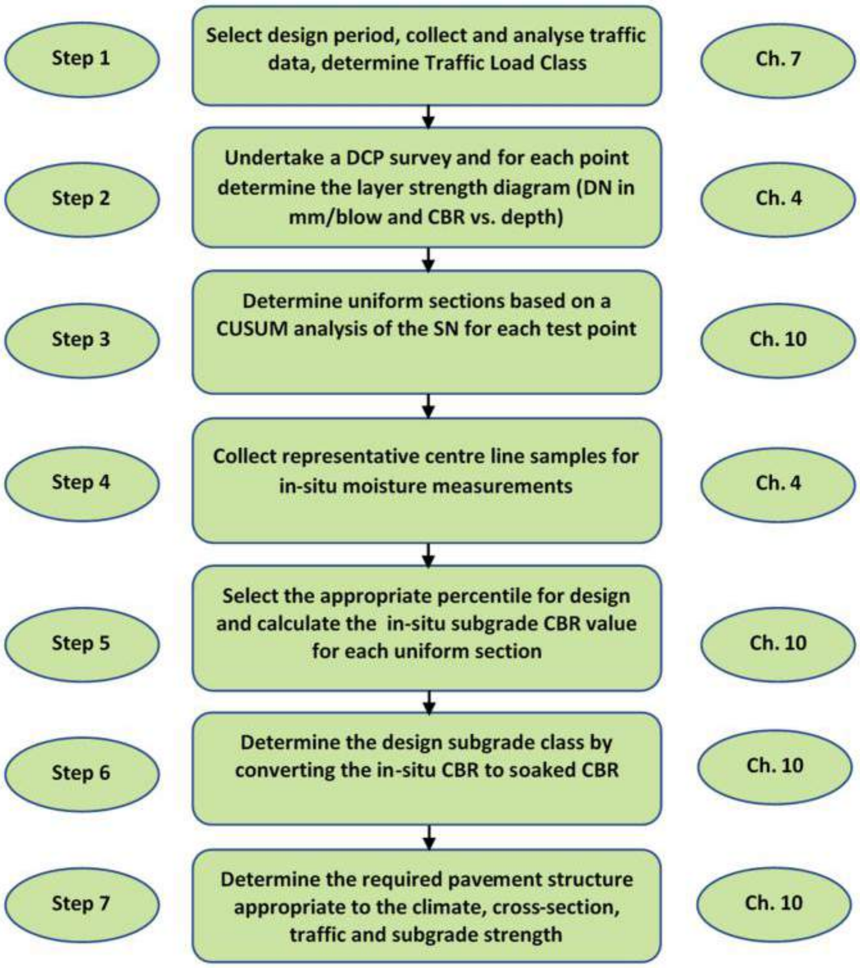


Figure 10-10: The DCP-CBR design procedure

Step 1: Determine the Traffic Load Class as described in *Chapter 7 – Traffic.*

Step 2: Carry out a DCP survey as described in *Chapter 4 – Site Investigations*. For each point, determine the layer strength diagram (DN in mm/blow and CBR vs. depth) and the layer boundaries.

Step 3: Determine uniform sections based on a CUSUM analysis of the Structural Number (SN) for each test point, as illustrated in Table 10-2 and Figure 10-6.

Step 4: Collect representative center line samples from each uniform section for the determination of the in-situ moisture content.

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 10-10.

Table 10-10: 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

Source: Gourley and Greening, 1999.

Alternatively, collect at least ten bulk samples from each uniform section and determine the subgrade soaked CBR value for each sample as well as the layer thickness and soaked CBR of any existing pavement layer(s).

Apply the correct percentile from Table 10-10 to determine the design subgrade class.

In this case, Step 6 is not necessary since soaked CBR values have been obtained directly from the standard CBR test. Continue to Step 7 to determine upgrading requirements.

Step 6: Determine the design subgrade class by converting the in-situ CBR to soaked CBR.

To convert from the in-situ values to the soaked values requires 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 10-11. The in-situ moisture condition is obtained from the samples collected from uniform sections for laboratory analysis. 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 analyzed, 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 to avoid a significant change in ground moisture conditions.

The relationship between soaked and in situ strength (CBR) depends on the characteristics of the materials, as presented in Table 10-11, which is based on extensive research.

Table 10-11: 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 7: Determine upgrading requirements.

For both new and existing roads:

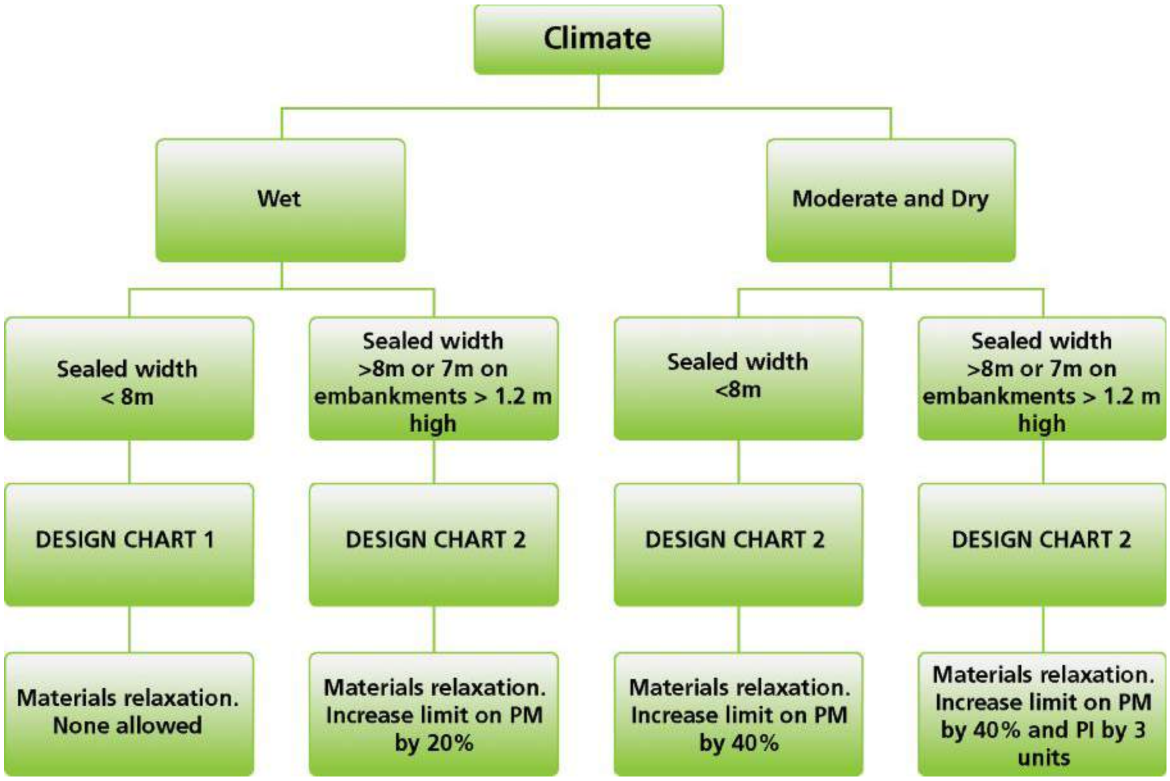
a) Select the required pavement structure appropriate to the climate, cross-section, traffic and subgrade strength (Figure 10-11).

For existing roads:

b) Convert the in-situ CBR of any existing pavement layer(s) to a soaked CBR using Table 10-11 and assess the strength and thickness against the requirements of the catalog.

c) Augment the layer(s), as required, using the most economical of the following options:
   - Add material of similar quality to make up a layer of the required thickness; or
   - Mechanically stabilize the existing layer to satisfy the strength requirement.

Note that if the existing structural layer(s) will serve as subbase or base in the upgraded structure, the material must also satisfy DCP-CBR material specifications.



Source: Gourley and Greening, 1999

Figure 10-11: 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 10-12) 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 10-13) should be used. The limit on the plasticity modulus of the base may be increased by 20% (refer to Figure 10-11 and Table 10-12).

b) Where the total sealed surface is less than 8 m, but the pavement is on an embankment in excess of 1.2 m in height, Pavement Design Chart 2 (Table 10-13) should be used. The limit on the plasticity modulus of the base course may be increased by 20% (refer to Figure 10-11 and Table 10-12).

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 10-13) 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%. (refer to Figure 10-11 and Table 10-12).

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. (refer to Figure 10-11 and Table 10-12).

Table 10-12: Pavement design Chart 1 (wet areas)

Sub-grade CBR	Pavement layer	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%)	Base	150 NG45	150 NG65	150 NG80	175 NG80	200 NG80
	Subbase	150 NG15	125 NG30	150 NG30	150 NG30	175 NG30
	Capping		150 NG15	175 NG15	175 NG15	175 NG15
S3 (5-7%)	Base	125 NG45	150 NG65	150 NG65	175 NG65	200 NG80
	Subbase	150 NG15	100 NG30	150 NG30	150 NG30	150 NG30
	Capping		100 NG15	125 NG15	125 NG15	150 NG15
S4 (8-14%)	Base	200 NG45	150 NG65	150 NG65	175 NG65	200 NG80
	Subbase		125 NG30	200 NG30	200 NG30	200 NG30
S5 (15-29%)	Base	175 NG45	125 NG65	150 NG65	150 NG65	175 NG80
	Subbase		100 NG30	125 NG30	150 NG30	150 NG30
S6 (>30%)	Base	150 NG45	150 NG65	175 NG65	200 NG65	200 NG80

Source: Gourley and Greening, 1999.

Table 10-13: Pavement design Chart 2 (moderate and dry areas)

Sub-grade CBR	Pavement layer	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%)	Base	150 NG45	150 NG65	150 NG65	175 NG80	175 NG80
	Subbase	150 NG15	125 NG30	150 NG30	150 NG30	175 NG30
	Capping		100 NG15	150 NG15	150 NG15	175 NG15
S3 (5-7%)	Base	125 NG45	150 NG55	175 NG65	175 NG80	175 NG80
	Subbase	125 NG15	175 NG30	175 NG30	200 NG30	250 NG30
S4 (8-14%)	Base	175 NG45	150 NG55	150 NG55	175 NG65	175 NG80
	Subbase		100 NG30	150 NG30	150 NG30	175 NG30
S5 (15-29%)	Base	150 NG45	200 NG55	125 NG55	125 NG65	150 NG80
	Subbase			125 NG30	125 NG30	125 NG30
S6 (>30%)	Base	150 NG45	175 NG45	175 NG55	175 NG65	175 NG80

Source: Gourley and Greening, 1999.

Note: 150 NG45 means 150 mm layer thickness of natural gravel (NG) with a soaked CBR of 45%

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-CBR Design Method

General
Natural gravel materials for this design method are classified, as shown in Table 10-14.

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 10-14: 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 10-15 and Table 10-16, which again is dependent on the Subgrade Class defined in Table 10-17.

Table 10-15: 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 10-16. The soaked CBR test is used to specify the minimum road base material strength.

Table 10-16: 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

Source: Gourley and Greening, 1999.

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 10-17: Subgrade class definitions

Subgrade Class	Design CBR (%)	Notes
S1	< 3	Special treatment required. Refer to Section 4.4.3.
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 10-18, are slightly different from 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-meter 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 10-18: 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, then 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 that meet the recommendations of Table 10-19 and Table 10-20 will usually be found to have an adequate bearing capacity.

Table 10-19: 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

Source: Gourley and Greening, 1999.

Table 10-20: 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
Weinert N value = (12 x Evaporation in warmest month)/annual precipitation (Weinert, 1980)					

Source: Gourley and Greening, 1999.

10.4 Design of Pavements with Non-discrete Surfacings

10.4.1 General

Structural surfaces may have a place for use on LVRRs. The initial cost is usually a constraining factor, but the whole life costs may sometimes make these options favorable. 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 steep or otherwise difficult from an engineering point of view.

10.4.2 Un-reinforced Concrete (URC)

General

The un-reinforced cement concrete option for LVRRs 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 10-21. 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 10-21: Thickness design for un-reinforced concrete (URC) pavement (mm)

Subgrade Class (CBR)	Pavement layer	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%)	Base	160 URC	170 URC	175 URC	180 URC	190 URC
	Subbase	150 NG30	150 NG30	150 NG30	150 NG30	150 NG30
S3 (5-7%)	Base	150 URC	160 URC	165 URC	170 URC	180 URC
	Subbase	125 NG30	125 NG30	125 NG30	125 NG30	125 NG30
S4 (8-14%)	Base	150 URC	150 URC	160 URC	170 URC	180 URC
	Subbase	100 NG30	100 NG30	100 NG30	100 NG30	100 NG30
S5 (15-29%)	Base	150 URC	150 URC	160 URC	170 URC	180 URC
	Subbase	100 NG30	100 NG30	100 NG30	100 NG30	100 NG30
S6 (>30%)	Base	150 URC	150 URC	160 URC	170 URC	180 URC

Notes: 1. Cube strength = minimum 30 Mpa at 28 days. 2. For 20-25 Mpa concrete strength, add steel mesh re-enforcement or increase slab thickness of about 30%. 3. On subgrades > 30%, the material should be scarified and re-compacted to ensure the depth of material with an in situ CBR >30% is in agreement with the recommendations.

Concrete Strips

Concrete strips are currently not commonly used in Afghanistan, 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 10-12. Should the traffic volume exceed, 30 vpd consideration can be given to using twin strips. The pavement thickness under discrete elements given in Table 10-23 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 10-12: Concrete strips to provide access up a steep, sandy section

The strips must be constructed of minimum C20 class concrete (20 Mpa). If heavy trucks are expected, mesh wire reinforcement shall be used and placed at 1/3 depth from the surface. The concrete strips are constructed as panels 0.9 m wide, 1.5 m to 3.0 m (max) in length, and 0.2 m thick. The distance from center to center shall be 1.55 m (see *Chapter 12 – Surfacing*, Table 12-25.

The end of the panels on a downward slope should be thickened to act as an anchor block on each panel at grade > 10%, every 2nd panel at grade 6-10%, every 3rd panel at grade 3-6% (see Perrie (2003) for anchor block details).

The concrete strips are connected by transverse, v-shaped concrete ribs, pointing up the grade, to help stop excessive erosion between the strips.

10.5 Design of Pavements with Discrete Element Surfacings

10.5.1 General

Discrete element surfacings (DES) can provide a viable alternative to non-discrete surfacings, as described above, for use on LVRRs. However, there are several aspects of their design that require careful consideration, as discussed below.

10.5.2 Design considerations

Climate: DES surfacings are more susceptible to water infiltration into the pavement than bituminous surfacings, particularly in high rainfall areas. To minimize this problem, adequate internal drainage of the pavement through the use of a sand bedding layer and the construction weep holes through the edge restraints are essential.

The manner of dealing with freeze-thaw conditions, as described in Section 10.6 below, should be observed.

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 some discrete element surfacings. This applies to Hand-packed Stone, Stone Setts (or Pavé), Cobblestone and Fired Clay Brick pavements. The construction procedure is similar to that adopted for un-mortared options except that cement mortar is used instead of sand for bedding and joint filling.

The behavior 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.

The design of roads with DES surfacings is addressed below, whilst the advantages and disadvantages of DES for use on LVRRs are discussed in *Chapter 12 – Surfacings.*

10.5.3 Pavement Design

Hand-packed Stone (HPS): HPS paving consists of a layer of large broken stone pieces (typically 150 mm to 300 mm 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 10-22. 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, e.g., a 50 mm emulsion treated gravel layer, can typically be used to provide a smooth(er) riding surface, but this will, of course, also add to the cost of the pavement.

Table 10-22: Pavement designs for Hand Packed Stone (HPS) pavement (mm)

Subgrade Class (CBR)	Pavement layer	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%)	Base	150 HPS	200 HPS	200 HPS	250 HPS	NA
	Sand blanket	50 SBL	50 SBL	50 SBL	50 SBL	
	Subbase	175 NG30	125 NG30	150 NG30	150 NG30	
	Capping		150 NG15	200 NG15	200 NG15	
S3 (5-7%)	Base	150 HPS	200 HPS	200 HPS	250 HPS	NA
	Sand blanket	50 SBL	50 SBL	50 SBL	50 SBL	
	Subbase	125 NG30	200 NG30	150 NG30	150 NG30	
	Capping			150 NG15	150 NG15	
S4 (8-14%)	Base	150 HPS	200 HPS	200 HPS	250 HPS	NA
	Sand blanket	50 SBL	50 SBL	50 SBL	50 SBL	
	Subbase	100 NG30	150 NG30	200 NG30	200 NG30	
S5 (15-29%)	Base	150 HPS	200 HPS	200 HPS	250 HPS	NA
	Sand blanket	50 SBL	50 SBL	50 SBL	50 SBL	
	Note 3					
S6 (>30%)	Base	150 HPS	200 HPS	200 HPS	250 HPS	NA
	Sand blanket	50 SBL	50 SBL	50 SBL	50 SBL	
	Note 3					

Notes: 1. The capping layer of NG15 material and the sub-base layer of NG30 material can be reduced in thickness if stronger material is available.

2. The capping layer can be NG10 provided it is laid 7% thicker.

3. On subgrades > 15%, the material should be scarified and re-compacted to ensure a uniform layer thickness

Pavé or Stone Setts: A Pavé or Stone sett surfacing consists of a layer of roughly cubic (100 mm) stone setts laid on a bed of sand or fine aggregate within a 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 10-23.

Cobblestone 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 a 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 are brushed into the spaces between the stones and the layer then compacted with a roller. Cobblestones 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 labor-based means.

The thickness designs for Pavé/Stone Setts and Cobblestone/Dressed Stone pavements are given in Table 10-23.

Table 10-23: Pavement designs for Pavé/Stone Sett and Cobblestone surfacings (DES) (mm)

Subgrade Class (CBR)	Pavement layer	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%)	Discrete elements	DES	DES	DES	DES	DES
	Sand blanket	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	100 NG65	125 NG65	150 NG80	150 NG80	150 NG80
	Subbase[1]	100 NG30	150 NG30	150 NG30	175 NG30	200 NG30
	Capping[1, 2,]	100 NG15	150 NG15	175 NG15	200 NG15	200 NG15
S3 (5-7%)	Discrete elements	DES	DES	DES	DES	DES
	Sand blanket	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	125 NG65	150 NG65	125 NG80	150 NG80	150 NG80
	Subbase[1]	100 NG30	175 NG30	125 NG30	150 NG30	175 NG30
	Capping[1, 2]			150 NG15	150 NG15	175 NG15
S4 (8-14%)	Discrete elements	DES	DES	DES	DES	DES
	Sand blanket	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	150 NG65	150 NG65	150 NG80	150 NG80	175 NG80
	Subbase[1]		100 NG30	150 NG30	200 NG30	225 NG30
S5 (15-29%)	Discrete elements	DES	DES	DES	DES	DES
	Sand blanket	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	125 NG65	100 NG65	125 NG80	150 NG80	150 NG80
	Subbase[1]		125 NG30	125 NG30	125 NG30	150 NG30
S6 (>30%)	Discrete elements	DES	DES	DES	DES	DES
	Sand blanket	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	125 NG65	150 NG65	150 NG80	150 NG80	150 NG80
	Note 3					

Notes: 1. The capping layer of NG15 material and the sub-base layer of NG30 material can be reduced in thickness if stronger material is available.
2. The capping layer can be NG10 provided it is laid 7% thicker.
3. On subgrades > 15%, the material should be scarified and re-compacted to ensure a uniform layer thickness

Precast Concrete Blocks: Precast concrete blocks can be constructed on a natural gravel or a cement-stabilized subbase. A 20 mm (minimum) sand blanket should be placed on top of the subbase to provide a cushion and a drainage layer. The blocks shall be made of concrete of 28-day cube strength of a minimum 25 Mpa with a minimum thickness of 60 mm, depending on the expected traffic as per the manufacturer's specifications. For increased bearing capacity, interlocking blocks are recommended.

The pavement design for precast concrete blocks surfacings is shown in Tables 10-24 and 10-25 for moderate/dry and wet regions, respectively.

Table 10-24: Pavement design for precast concrete blocks (Moderate and dry regions)

	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 mm PB 20 mm SND 125- 150 mm NG45 60 mm PB 20 mm SND 125 mm C4	
Residential access collectors, car parks and lightly trafficked bus routes				60 mm PB 20 mm SND 100 - 125 mm NG45		60 mm PB 20 mm SND 125 - 150 mm NG45 60 mm PB 20 mm SND 125 mm C4	
Local access roads, loops, -ways, -courts, -strips and culs de sac			60 mm PB 20 mm SND				150 mm NG7 NG3

PB = 60 mm (minimum) interlocking paving block as per manufacturers specifications

SND: Bedding sand

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

Source: Adapted from Draft Urban Transport Guidelines 2 (UTG 2), 1988

Table 10-25: Pavement design for precast concrete blocks (Wet regions)

	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 mm PB 20 mm SND 150 mm NG45 60 mm PB 20 mm SND 150 mm C4	
Residential access collectors, car parks and lightly trafficked bus routes				60 mm PB 20 mm SND 100 - 125 mm NG45 60 mm PB 20 mm SND 100 - 125 mm C4		60 mm PB 20 mm SND 125 - 150 mm NG45 60 mm PB 20 mm SND 125 - 150 mm C4	
Local access roads, loops, -ways, -courts, -strips and culs de sac			60 mm PB 20 mm SND				150 mm NG7 NG3

PB = 60 mm (minimum) interlocking paving block as per manufacturers specifications

SND: Bedding sand

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

Source: Adapted from Draft Urban Transport Guidelines 2 (UTG 2), 1988

10.6 Pavement Design under Freeze-thaw Conditions

10.6.1 General

Large parts of Afghanistan experience winter temperatures well below zero °C, in some areas down to -20°C and below, and heavy snowfall. The pavement design must, therefore, take into consideration the effects of the freezing of the pavement during the winter months and subsequent thawing during the spring.

10.6.2 Sources of Water

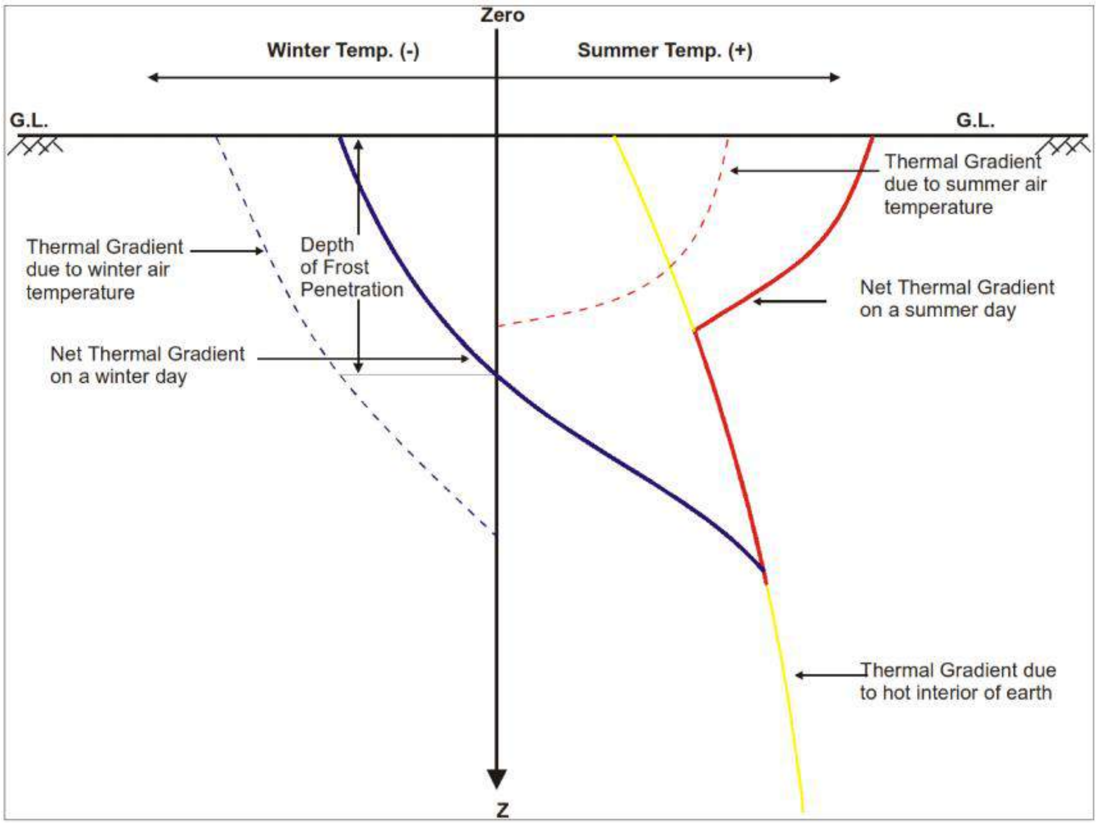
The two basic forms of frost action (frost heave and thawing as discussed below) both require water. Water sources can be separated into two broad categories:

1. Surface water. Enters the pavement primarily by infiltration through surface cracks and joints, and through adjacent unpaved surfaces, during periods of rain and melting snow and ice. Many crack-free pavements are not entirely impermeable to moisture.
2. Subsurface water. Can come from three primary sources:
   - Groundwater table (or perched water table).
   - Moisture held in soil voids or drawn upward from a water table by capillary forces.
   - Moisture that moves laterally beneath a pavement from an external source (e.g., a pervious water-bearing strata, etc.).

10.6.3 Freezing or Thawing Depths in Pavements

It is possible to estimate the frost depth in pavement using fairly complicated formulae. However, for pavement design, the frost depth that is used for the installation of water reticulation can be used with reasonable factors of safety. Information on this can usually be obtained from local authorities.

Figure 10-13 illustrates the seasonally changing thermal regime of the ground in cold regions.



Source: Kachroo et al (2002)

Figure 10-13: Illustration of seasonal thermal regime of the ground in cold regions.

10.6.4 Frost action in pavements

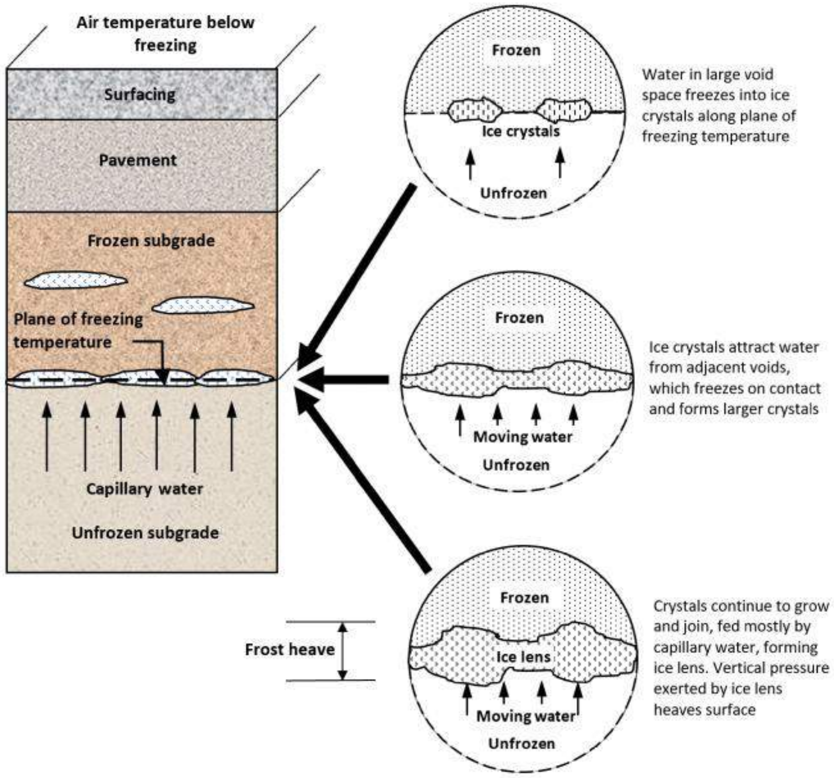
There are two potentially damaging effects of frost in the pavements:

- **Frost heave.** Results from the accumulation of moisture in the soil during the freezing period. These accumulations (ice lenses) expand perpendicular to the direction of heat ow and push the pavement up, often causing severe cracking.
- **Thaw weakening**. Once a subgrade is frozen, it can be severely weakened when it thaws (usually in the springtime). Therefore, loading that would not normally damage a given pavement may be quite detrimental during thaw periods.

10.6.5 Frost heave

Frost heaving of soil is caused by crystallization of ice within the larger soil voids and usually a subsequent extension to form continuous ice lenses, layers, veins, or other ice masses. An ice lens grows through capillary rise and thickens in the direction of heat transfer until the water supply is depleted or until freezing conditions at the freezing interface no longer support further crystallization. As the ice lens grows, the overlying soil and pavement will "heave" up, potentially resulting in a cracked, rough pavement (see Figure 10-17). This problem occurs primarily in soils containing fine particles (often termed "frost susceptible" soils), while clean sands and gravels (small amounts of fine particles) are non-frost susceptible (NFS). Thus, the degree of frost susceptibility is mainly a function of the percentage of fine particles within the soil. Many agencies classify materials as being frost susceptible if 10 % or more passed a 0.075 mm sieve or 3 % or more passed a 0.02 mm sieve. Figure 10-16 illustrates the formation of ice lenses in a frost susceptible soil.

Figure 10-14: Pavement affected by frost heave



Source: Mahoney et al, 1986

Figure 10-15: Formation of ice lenses in a pavement structure

The three elements necessary for ice lenses and thus frost heave are:

- Frost susceptible soil (a significant amount of fines).
- Subfreezing temperatures (freezing temperatures must penetrate the soil and, in general, the thickness of an ice lens will be thicker with slower rates of freezing).
- Water (must be available from the groundwater table, infiltration, an aquifer, or held within the voids of fine-grained soil).

Remove any of the three conditions above, and frost effects will be eliminated or at least minimized. If the three conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur, resulting in pavement cracking and roughness. Differential heave is more likely to occur at locations such as:

- Where subgrades change from clean, not frost susceptible (NFS) sands to silty frost susceptible materials.
- Abrupt transitions from cut to fill with groundwater close to the surface.
- Where excavation exposes water-bearing strata.
- Drains, culverts, etc., frequently result in abrupt differential heaving due to different backfill material or compaction and the fact that open buried pipes change the thermal conditions (i.e.,, remove heat resulting in more frozen soil).

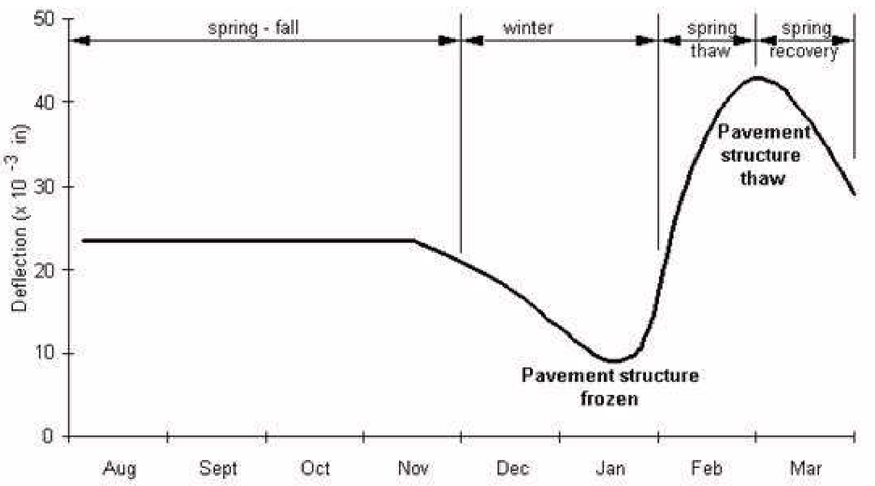
Additional factors which will affect the degree of frost susceptibility (or the ability of a soil to heave):

- Rate of heat removal.
- Temperature gradient
- Mobility of water (e.g., the permeability of soil)
- Depth of water table
- Soil type and condition (e.g., density, texture, structure, etc.)

10.6.6 Thaw Weakening

Thawing is essentially the melting of ice contained within the subgrade. As the ice melts and turns to liquid, it cannot drain out of the soil fast enough, and thus, the subgrade becomes substantially weaker (less stiff) and tends to lose bearing capacity. Therefore, loading that would not normally damage a given pavement may be quite detrimental during thaw periods (e.g., spring thaw).

Figure 10-16 is an example of typical pavement deflection changes throughout the year caused by winter freezing and spring thawing. Figure 4 shows pavement damage as a result of thaw weakening.



Source: Mahoney et al, 1986

Figure 10-16: Typical pavement deflections illustrating seasonal pavement strength changes

Thawing can proceed from the top downward, or from the bottom upward, or both. How this occurs depends mainly on the pavement surface temperature. During a sudden spring thaw, melting will proceed almost entirely from the surface downward. This type of thawing leads to extremely poor drainage conditions. The frozen soil beneath the thawed layer can trap the water released by the melting ice lenses so that lateral and surface drainage are the only paths the water can take.

The effects of refreezing after a thaw are also accentuated by the fact that the first freeze leaves the soil in a more or less loosened or expanded condition (Taber, 1930).

This shows that (1) the reduced density of base or subgrade materials helps to explain the long recovery period for material stiffness or strength following thawing, and (2) that refreezing following an initial thaw can create the potential for greater weakening when the “final” thaw does occur.

Figure 10-17: Freeze-thaw damage

10.6.7 Mitigating Frost Action

Mitigation of frost action and its detrimental effects generally involves structural design considerations as well as other techniques applied to the base and subgrade. The basic methods used can be broadly categorized into the following techniques:

- Limiting the depth of frost into the subgrade soils. This is typically accomplished by specifying the depth of pavement to be some minimum percentage of the frost depth. By extending the pavement section well into the frost depth, the depth of the frost-susceptible subgrade under the pavement (between the bottom of the pavement structure and frost depth) is reduced. The assumption is that a reduced depth of soil under frost action will cause correspondingly less damage.
- Removing and replacing frost-susceptible subgrade. Ideally, the subgrade will be removed at least down to the typical frost depth. Removing frost-susceptible soils removes frost action.
- Designing the pavement structure based on reduced subgrade support. This method simply increases the pavement thickness to account for the damage and loss of support caused by frost action.
- Providing a capillary break. By breaking the capillary flow path, frost action will be less severe because frost heaving requires substantially more water than is naturally available in the soil pores (Tabor, 1930).

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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 2 - 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 mold 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 mold 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 mold 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 mold instead of the CBR plunger.

Each of the specimens should be subjected to DCP testing in the CBR mold as summarized below.

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

(b) Measure the height of the compacted specimen inside the mold. 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 mold upside down or another device (e.g., bricks or cement blocks) next to the full mold, 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 mold.

(d) Position the tip of the DCP cone in the middle of the CBR mold, 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 mold).

(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 mold. 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.

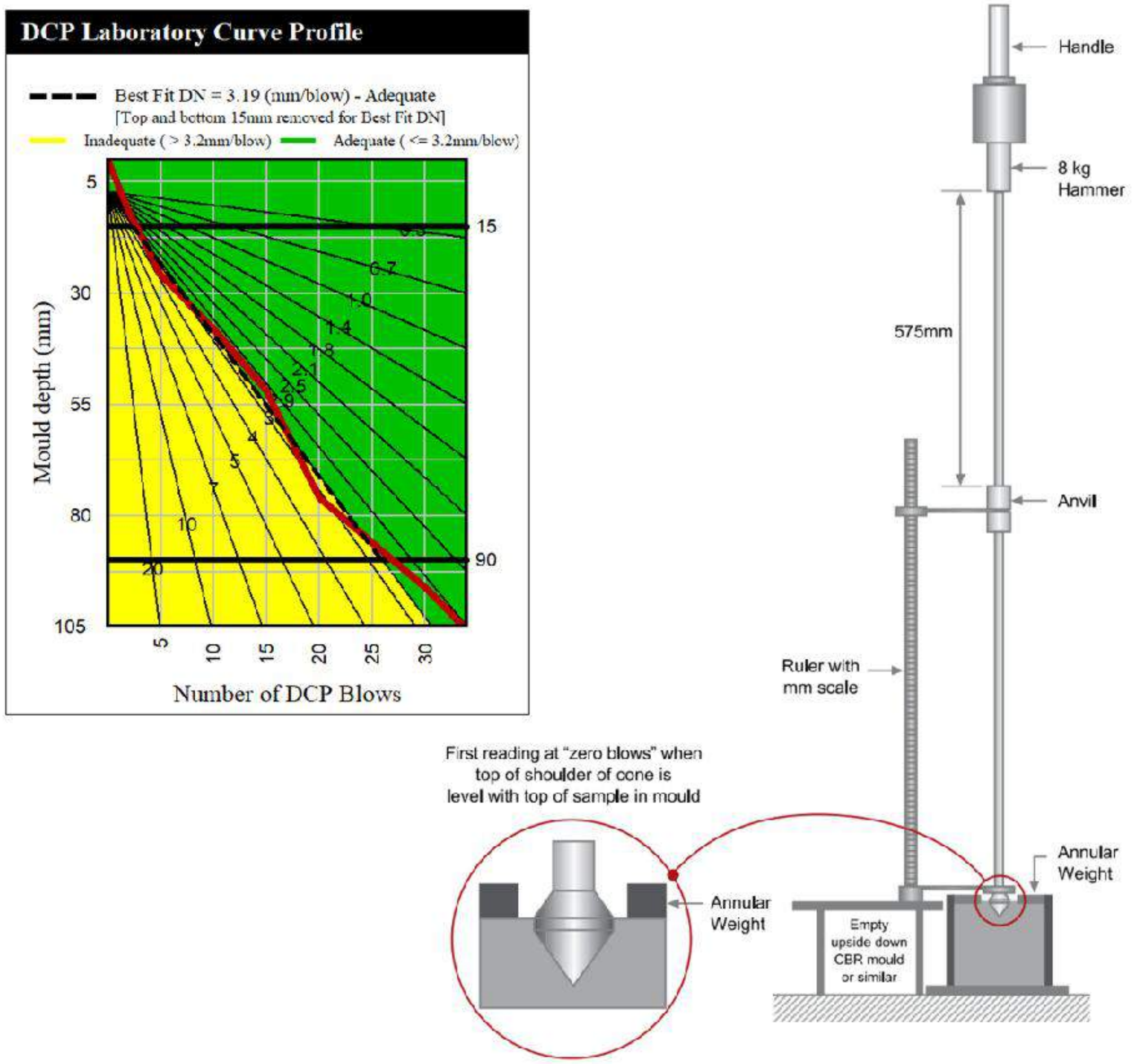


Figure A1-1: Set-up and typical output from the laboratory DN test

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 mold 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 catalog for each pavement layer.

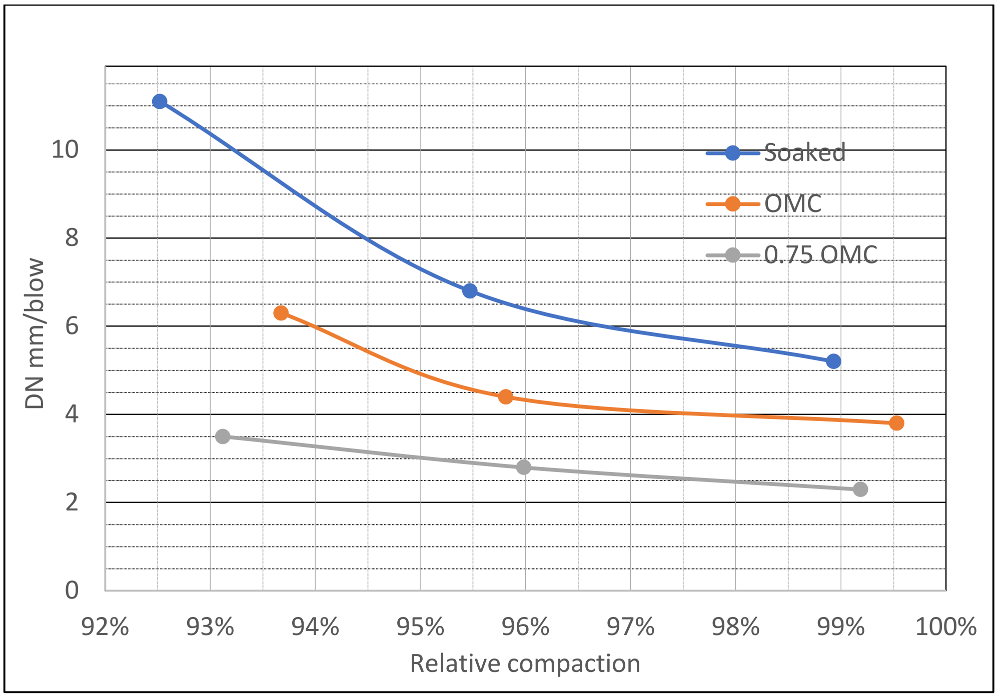


Figure A1-2: DN/density/moisture relationship

Figure A1-2 illustrates two critical factors that crucially affect the long-term performance of the road:

- 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.
- 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 9 – Drainage and Erosion Control.*

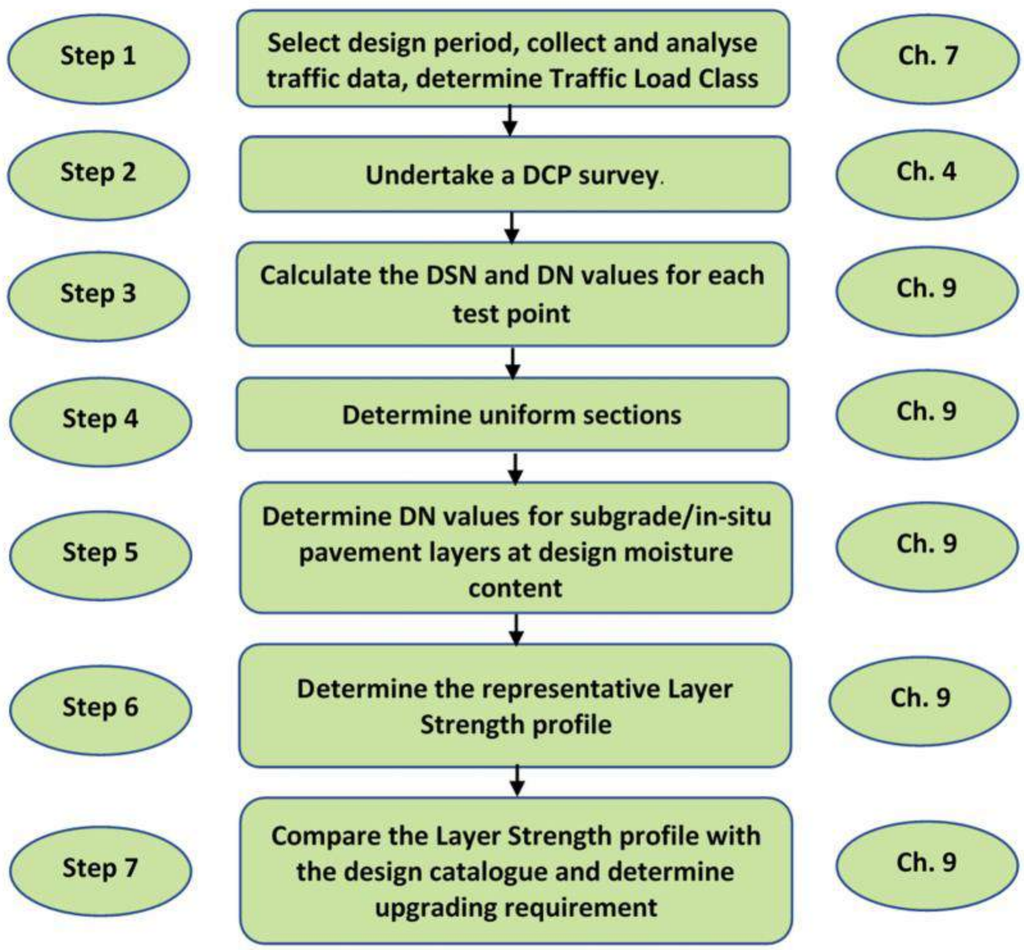
Appendix 2: Design Examples

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

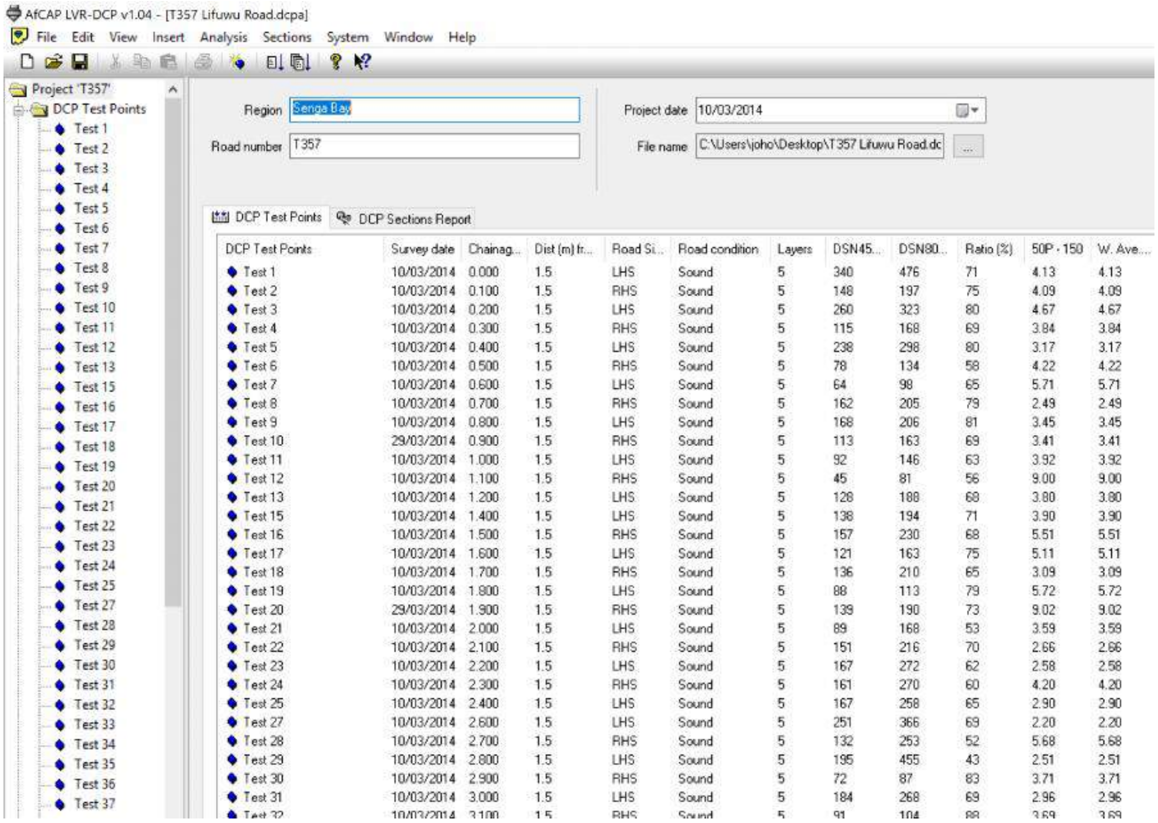
- Design period: 15 years
- Traffic class: TLC 0.3 (see *Chapter 7 - Traffic*)

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.



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:
  - Section 1: km 0+000 to 2+000
  - Section 2: km 2+000 to 7+000
  - 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.

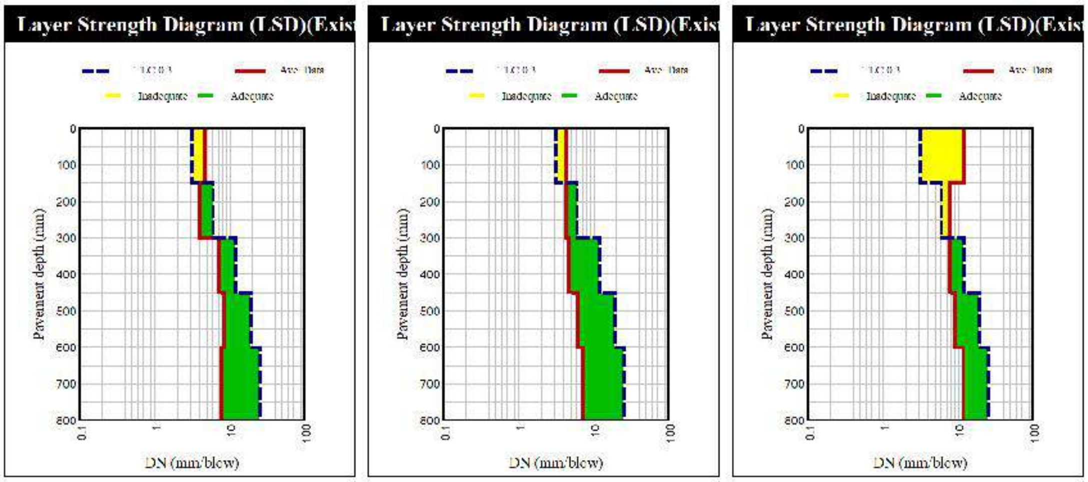


Figure A2-3: Representative LSD for the uniform section 1, 2 and 3 respectively

- The yellow color indicates layers that are too weak compared to the requirements of the design catalog 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 catalog 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 catalog for TLC 0.3

Table A2-2: LSD for uniform sections in tabular format after upgrading

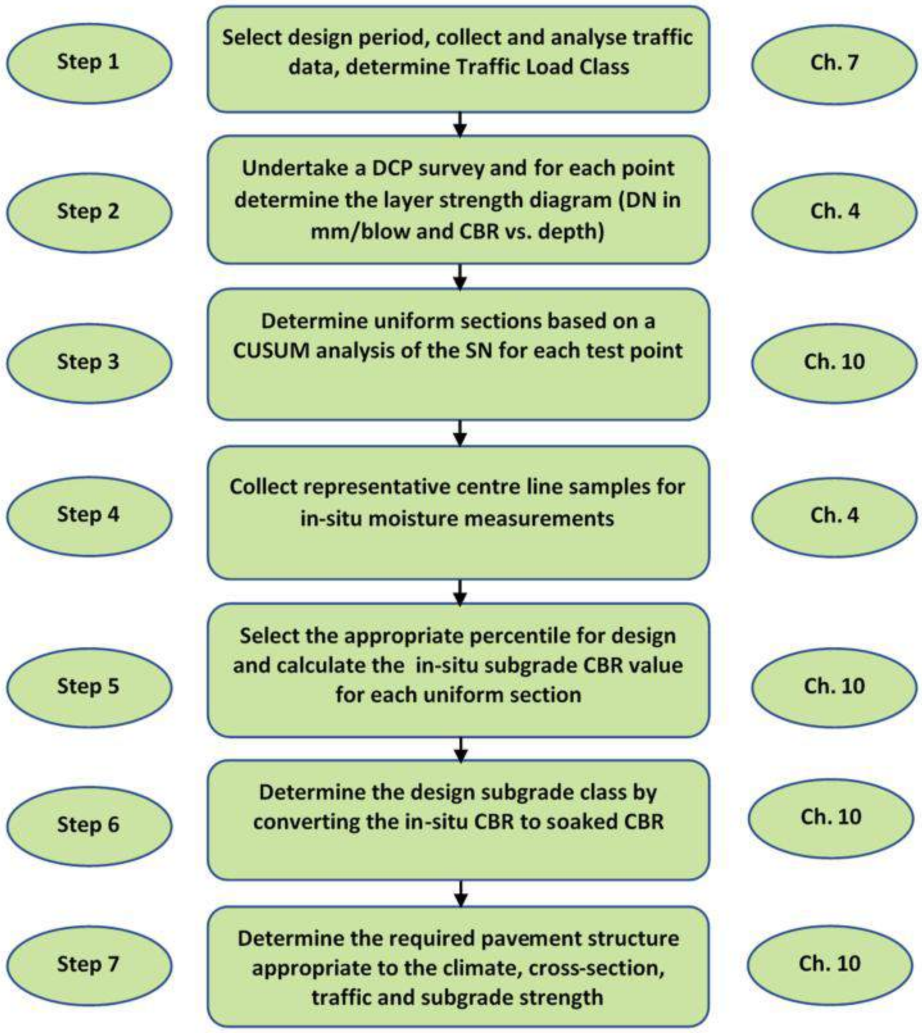
Pavement Layer (mm)	Required DN value for TLC 0.3	Section no.		
		1	2	3
		0.000 to 2.000 km	2.000 to 7.000 km	7.000 to 8.600 km
0-150	<= 3.2	≤3.2	≤3.2	≤3.2
150-300	<= 6.6	4.6	4.3	≤6.6
300-450	<= 11	4.0	4.3	11
450-600	<= 17	7.0	4.7	7.7
600-800	<= 25	8.5	6.1	7.7

Design example DCP-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: 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

- A DCP survey comprising 55 test points was carried out at 100 m intervals.
- The DCP data were analyzed with the UK DCP program to determine the Layer Strength profile and layer boundaries, as illustrated in Figure .

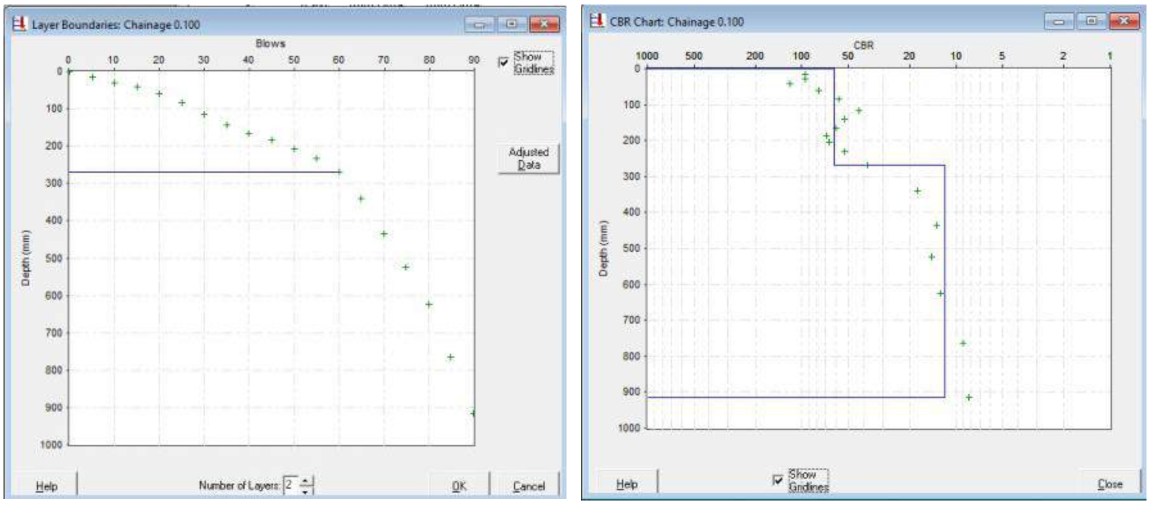


Figure A2-4: Typical Layer Strength Diagram and CBR Chart

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.

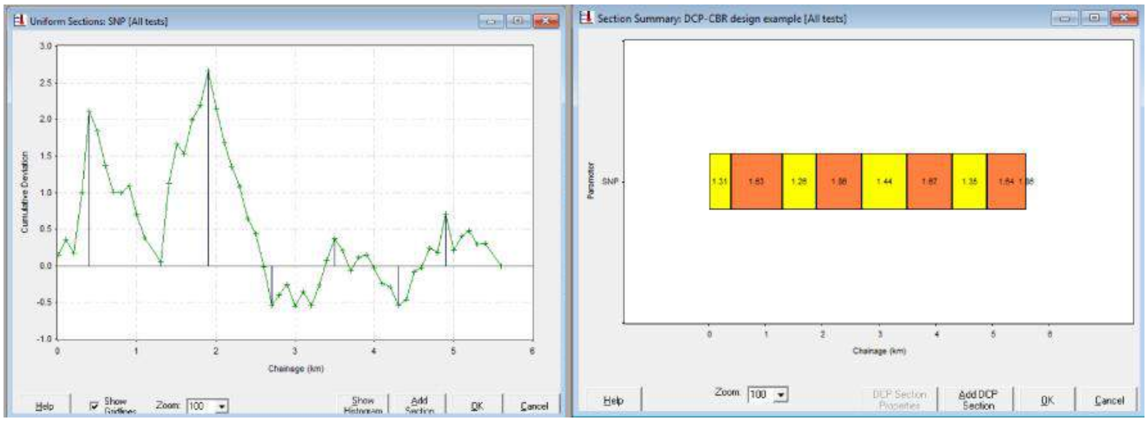


Figure A2-5: Uniform sections determined on the basis of SNP

Step 5: Select appropriate percentile and determine in-situ subgrade CBR

- The program 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 .

Table A2-3: Pavement design of uniform sections

Pavement Design - Moderate area - Road width 6.50 m - Design Chart 2 (Table 10-13)								
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.

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  - 2. Approach to Design
  - 3. Physical Environment
- Section 2: Investigations
  - 4. Site Investigations
  - 5. Geotechnical Investigations and Design
  - 6. Materials
  - 7. Traffic
- Section 3: Design
  - 8. Hydrology and Drainage Structures
  - 9. Drainage and Erosion Control
  - 10. Structural Design: Paved Roads
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11.1 Introduction

11.1.1 Background

More than 90 % of the road network in Afghanistan 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 low volume sealed road (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 Guideline as any road that is not surfaced with a non-structural "waterproof" bituminous surfacing or structural surfacings such as concrete, interlocking blocks, cobblestones 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. These tracks or earth roads may, in some cases, be ripped, shaped and compacted (engineered), but generally, the only compaction is that applied by vehicles moving over them (un-engineered). If these tracks or earth roads are being used where access is difficult due to soil conditions, stones or gravel may be imported locally to improve strength and traction.

In service, these "roads" reach a stage 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 layer of 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 itself.

Unpaved roads will typically carry a maximum of about 150 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.

Figure 11-1 shows a schematic hierarchy of low volume roads.

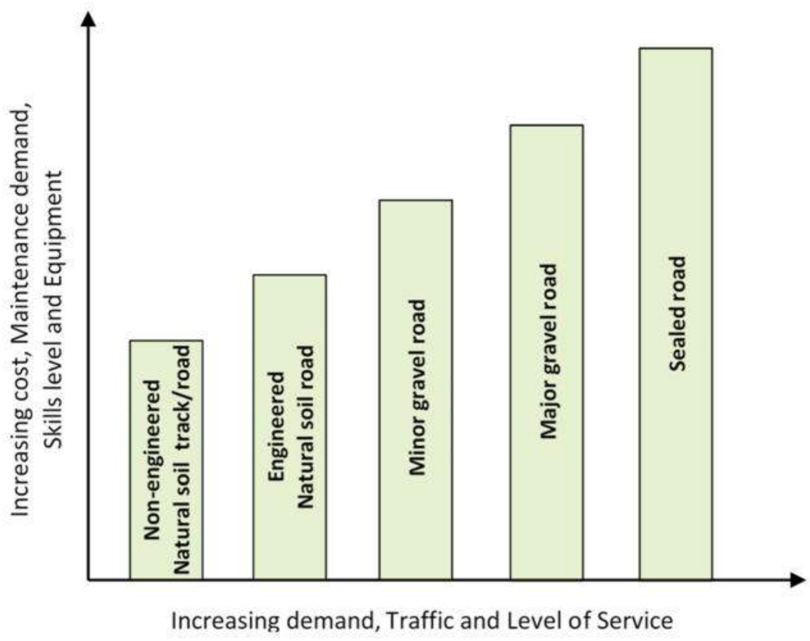


Figure 11-1: Schematic hierarchy of low volume roads

11.1.2 Purpose and Scope

The purpose of this chapter is to provide a framework for the design of unpaved roads economically and sustainably such that the appropriate levels of quality are produced.

The chapter covers the design of all levels of unpaved roads from tracks and earth roads, making use of the in-situ soil to engineered and treated gravel roads. Material selection and thickness design are treated in detail.

11.2 Tracks and Footpaths

The majority of Afghans, some 72%, live in rural areas in about 40,000 villages. Many rural communities, however, are still hampered by a lack of all-weather road infrastructure because of the ruggedness of the terrain, and in many cases, linkage to the road system will be a challenge for the foreseeable future. Access routes in the form of footpaths and mule tracks are currently the only means to connect them to markets and other public amenities such as educational centers, health facilities and shops. Furthermore, villagers are often cut-off from basic services because of flooding rivers and locally constructed trail bridges being washed away and have to wade through at high risk of drowning.

In villages that are not yet connected to the road network by an all-season road, motorcycles offer a vital means of transport for passengers, freight, and emergency access. It is, therefore, important that tracks and footpaths can provide safe passage for all users such as motorcyclists, pedestrians and pack animals. The construction and maintenance of tracks and footpaths, as well as trail bridges for the crossing of rivers, thus constitute a vital first step in providing reliable all-weather access to these communities.

Figure 11-2: Typical challenges experienced in remote rural areas

For the detailed design and construction of tracks and trail bridges, practitioners should refer to the following manuals:

- *Footpaths and Tracks – A Field Manual to their Construction and Improvement* (ILO, March 2002)
- *Footbridges*. *A Manual for Construction at Community and District Level* (DFID, June 2004).

Source: ILO Project Aceh and Nias

Figure 11-3: Examples of trail bridges

11.3 Earth Roads

11.3.1 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.

11.3.2 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. 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 binding of the material that is also contributed by the binding 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. However, 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.

11.3.3 Engineered earth roads

Engineered or improved earth roads differ from the un-engineered earth roads 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 material, but the material that is excavated from the side of the road to form side drains (at least 150 mm below natural ground level) is added to the road to increase its height and provide a better-drained road structure (see Figure 11-4). 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 minimum crown height of the earth road above the drain invert depends on the road class. See Table 11.1.
- Where the topography allows, wide, shallow longitudinal drains are preferred. They minimize erosion and will not block as easily as narrow V-drains, which should always 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 the 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 the road surface should have a minimum camber of 4% - 6% to ensure water runs off the surface and into the side drains.
- In areas where there are specific problems (usually due to water or to the poor condition of the subgrade), the in situ material may be treated in isolation by localized replacement of the subgrade, graveling, installation of culverts, raising the roadway or by installing other drainage measures. These measures are the basis of a “spot improvement” approach.

- Water should be drained away from the carriageway side drains by excavating miter drains to divert the flow into open space. The spacing of the miter drains should be as indicated in *Chapter 9 – Drainage and Erosion Control: Table 9-6.* 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.

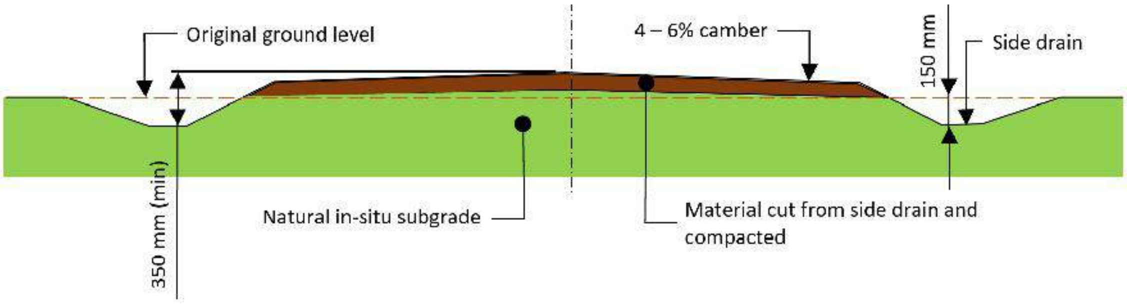
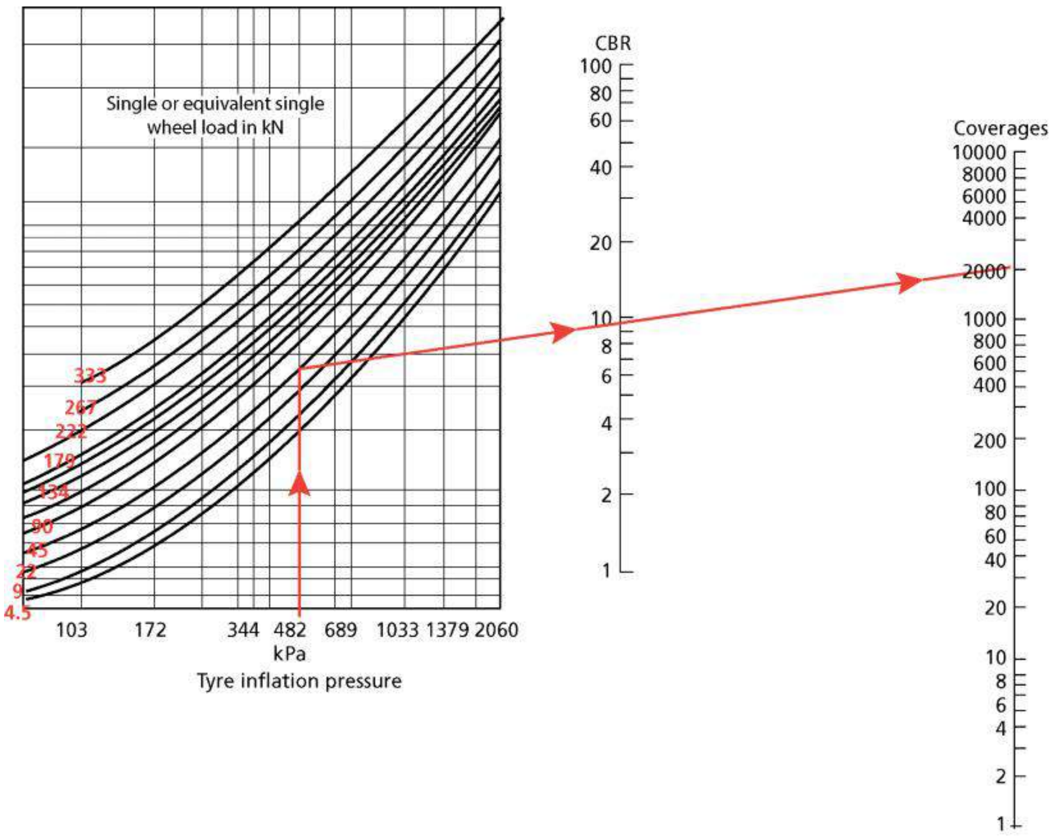


Figure 11-4: Cross section of typical improved earth road

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 apply to earth roads, but if the local materials comply with the requirements for gravel roads, a 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. This requires knowledge of the bearing capacity (CBR) of the soil, the equivalent single wheel load of the vehicles and the tire inflation pressures, as shown in Figure 11-5. 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.



Source: Ahlvin R G and G M Hamitt (1975).

Figure 11-5: Carrying capacity of engineered earth roads

As illustrated in Figure 11-5, 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 tire inflation pressure of 482 kPa before serious deformation (a 75 mm rut) is likely to occur. Since the wheel loads will not be concentrated on 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 ton wheels), this translates into a need to maintain, re-grade or reshape the surface about every four to six months. For soils with a higher CBR, this will be longer. Both designers and road managers need 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 perpendicular 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, to ensure discharge is at ground level. However, culvert pipes smaller than 750 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, consideration should be given to constructing a traditional gravel road using materials from a selected borrow pit for the wearing course.

11.4 Gravel Roads

11.4.1 General

Roads described as gravel roads imply that several factors have been taken into account in their design and construction. These include:

- Using the material of a selected quality to provide an all-weather wearing course.
- Protecting the subgrade from excessive strains under traffic loads by the use of road structure materials of adequate strength.
- Designing the camber of the road to allow drainage of water (mainly precipitation) from the road surface and from alongside the road.
- Installing the necessary cross and side drainage.
- Constructing the road to appropriate standards, including shape, compaction and finish.

Even with the provision of an all-weather wearing course, the road may not necessarily be passable at all times of the year as a result of periodic flooding of low-level water crossings. Such flooding, however, is not a function of the gravel road design but, rather, the adequacy of the low-level water crossings as discussed in *Chapter 8 - Hydrology and Drainage Structures*.

11.4.2 Pavement Structure

A gravel road consists of a wearing course and a structural layer (base) which covers the in-situ material (Figure 11-6). 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. This wearing course should under no circumstances be allowed to become thinner than about 50 mm.

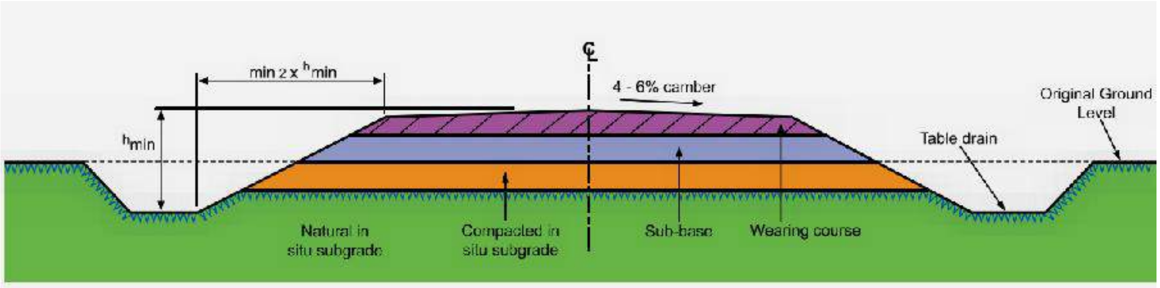


Figure 11-6: Typical gravel road cross section in flat terrain

To achieve adequate external drainage, the road must be raised above the level of existing ground such that the road crown is maintained at a minimum height (h_{min}) above the drain inverts (Figure 11-6). The minimum height is dependent on the climate (annual rainfall) and road design class, as shown in Table 11-1.

Table 11-1: Minimum height h_{min} of road crown above drain invert

Road Class	Annual rainfall (mm)	
	>1000	<1000
	h_{min} (mm)	
LVRR5	550	450
LVRR4	500	400
LVRR3	450	350
LVRR2	400	300
LVRR1	350	250

11.4.3 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 (washboard), become slippery when wet, lose gravel rapidly and generate excessive dust. Table 11-2 summarizes the required properties of good wearing course gravels.

Table 11-2: 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 at 95% AASHTO T180 compaction (soaked)
Treton Impact value (%)[1]	20 – 65

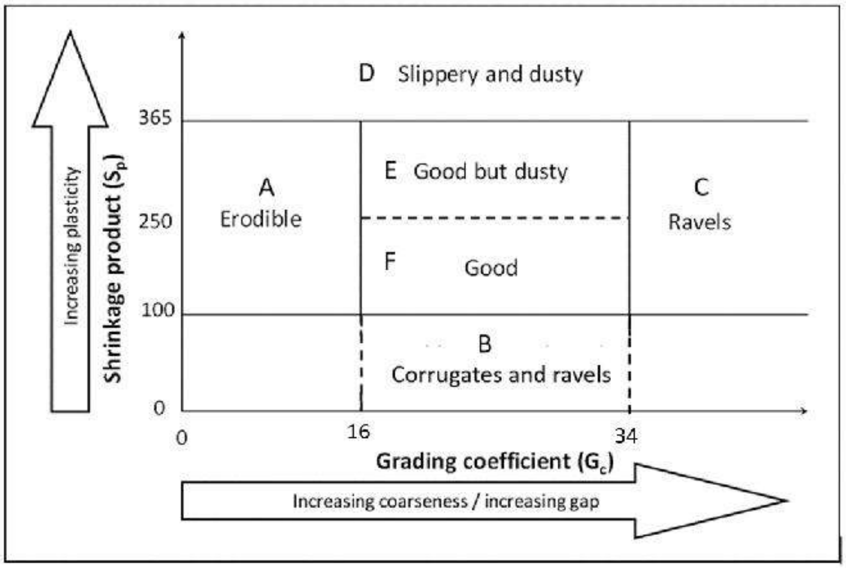
Source: Paige-Green P (1989)

Note: The Bar Linear Shrinkage and Treton Impact Value tests are not standard AASHTO tests and are described in Appendix 11-2 and Appendix 11-3, respectively.

The recommended grading and cohesion (shrinkage) specifications for gravel wearing course materials can also be shown diagrammatically in relation to their predicted performance defined by the values of the Shrinkage Product and Grading Coefficient, as shown in Figure 11-7, where:

Shrinkage Product (SP) = Bar Linear Shrinkage$_{0.425}$ x $P_{0.425}$

Grading Coefficient (GC) = ($P_{26.5}$ - $P_{2.0}$) x $P_{4.75}$/100



Source: Department of Transport, South Africa (1990)

Figure 11-7: Chart showing performance of unpaved road materials

In the chart presented in Figure 11-7, the five zones indicated (A to E) 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 (washboarding) and raveling/loosening.
- Zone C: Poorly graded materials that are prone to raveling.
- Zone D: Fine plastic material that is prone to slipperiness when wet and excessive dust.
- Zone E: Good performance (more dusty if SP > 240).
- Zone F: Optimum materials for best performance.

In the DCP-DN method, requirements for both material and aggregate strength are provided. The material strength is specified as the DCP-DN value (13.5 mm/blow), which initially appears very low. However, investigations of many roads in various countries have 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 tire load), even when soaked. Materials of significantly higher quality than this should be preserved for later use in paved roads. The Treton Impact Value (TIV) differentiates between aggregate particles that will perform well (TIV 20 to 65), aggregates that are too soft and will disintegrate under traffic (TIV > 65) and aggregates that are too hard to be broken down by conventional or grid rolling during construction and, if large particles are not removed, will result in stony roads.

As indicated in Figure 11-7, potential performance problems that could affect the road should the materials not fall into Zone E can be identified. Should this be the case, engineering judgment must be 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, experience has 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 11-2. 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 11-8 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 11-8: Examples of gravel wearing course performance

Pavement Design

General: The mechanism of deterioration of unpaved roads differs from that of paved roads and is directly related to the number of vehicles using the road rather than the number of equivalent standard axles. The traffic volume is, therefore, used in the design of unpaved roads, as opposed to 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 11-1) 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 10 – Structural Design: Paved Roads*, the subgrade should be divided into uniform sections based on a DCP survey. Slightly different methods will be used for the design of a new road as compared to the improvement of an existing earth road: 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 10 – Structural Design Paved Roads, Section 10.2.6*, should be applied for the determination of the characteristic subgrade strength of the two upper layers, DN_{150}, $DN_{150-300}$, for each uniform section at the anticipated field density and long-term moisture content. For gravel roads, the soaked Laboratory DN values should normally be applied.

In mountainous areas, the excavation of cuttings will usually expose fresh to partly weathered rock as a subgrade. This material is sufficiently strong for any unpaved road subgrade and typically only requires a regulating layer to smoothen off the surface before application of the wearing course. In many cases, the addition of a slightly thicker wearing course can be considered, depending on the roughness of the exposed subgrade. If care is taken during the excavation of this layer (ripping or blasting) the additional thickness need only be 50 or 75 mm. However, it is also useful to roll the excavated subgrade with heavy vibratory rollers and provide a coarse gravel layer (< 75 mm maximum size) beneath the imported wearing course, which can act as an internal drainage layer as well as the “bedding” layer for the wearing course.

Pavement layer design

This makes use of the following procedure:

- Compare the relevant subgrade strength profiles with the necessary design given in Table 11-3 or the graphical representation as layer strength diagrams shown in Figure 11-9, 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.

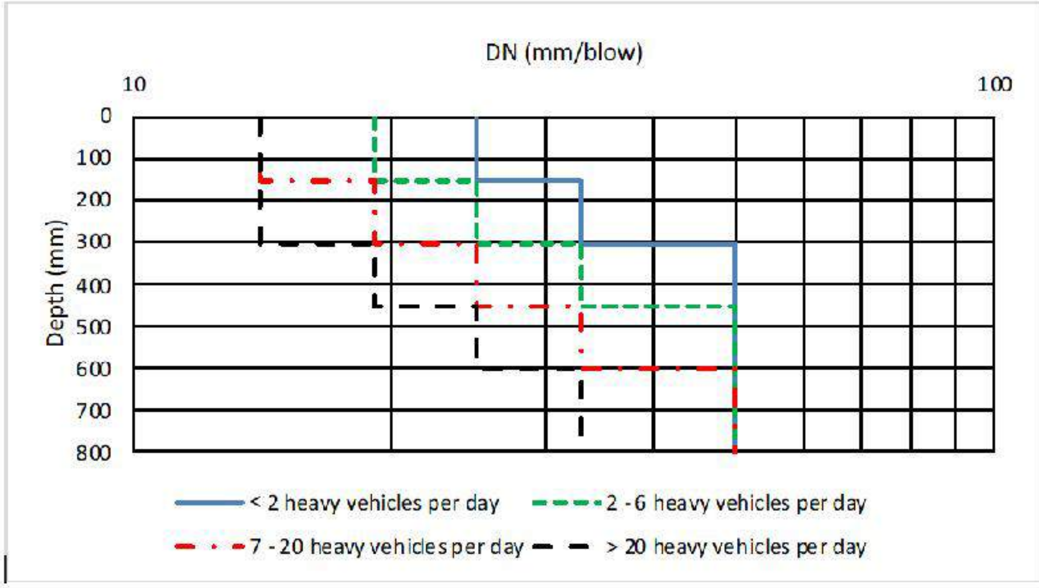
It is not possible to determine the DSN_{800} based on the Laboratory DN_{150} and $DN_{150-300}$. However, as long as these satisfy the criteria in Table 11-3, the DSN_{800} requirement will also be above the lower limit, unless the layers below 300 mm depth are particularly weak.

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 from Table 11-3 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, this imported material having an in situ DN value of between 13.5 mm and 25 mm/blow depending on the traffic.

Table 11-3: Gravel road pavement design for different traffic categories (DN)

Layer, depth and DCP Structural Number	Traffic (Heavy vehicles/day)			
	< 2	2 – 6	7 – 20	21-60
	DN (mm/blow)			
150 mm Wearing course (Table 11-2)	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 ≥ 95% 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)				

Source: Adapted from TRH20, 2009



Source: Adapted from TRH20, 2009

Figure 11-9: 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 directly on the in situ material. This wearing course would normally consist of 150 mm of specified material, as shown in Table 11-2 and Figure 11-7, but if the potential for delayed maintenance (i.e., re-graveling) exists, an additional 50 mm should be added as a buffer layer.

As indicated in Table 11-3, the minimum strength of the support layer beneath the wearing course need not be very high and, for relatively high levels of traffic (> 7 heavy vehicles per day), the minimum strength requirement of 13.5 mm/blow is the same as the strength requirement for the wearing course (see table 11-2). However, this material may not have the required 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 11-7, 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 on the cumulative axle loads as traditionally used for paved roads. This is because the mode of distress is related to the shear failure of the layers under loading as opposed to cumulative deformation with time, which is removed during routine maintenance and re-graveling. The reliability of the design is thus accepted as being slightly lower than for paved roads 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 life-cycle 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 are ways 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 11-4.

Table 11-4: Typical estimates of gravel loss

Material Quality Zone	Material Quality	Typical gravel loss (mm/year/100 vpd)
Zone A	Satisfactory	20
Zone B	Poor	40
Zone C	Poor	40
Zone D	Marginal	20
Zone E	Good	15

The gravel losses shown in Table 11-4 apply only for the first phase of the deterioration cycle lasting possibly two or three years. Beyond that period, as the thickness of the wearing course is reduced, 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 a simple aid to the planning for re-graveling 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 in these sections.

Regravelling should take place before the underlying layer is exposed. The re-graveling frequency, R, is typically in the range of 5 years to 8 years.

The optimum wearing course thickness is expressed as:

R x GL

where:

R = re-graveling frequency in years
GL = expected annual gravel loss (mm/yr/100vpd).

Where suitable sand is available adjacent to the road, the application of a sand cushion (25 mm to 40 mm) on top of the wearing course allows low-cost regular maintenance of the road. The sand cushion preserves the wearing course and reduces material loss as long as the sand covers the road.

11.4.4 DCP-CBR Method

Materials

Material specifications for the DCP-CBR method are similar to those for the DCP-DN method.

The material 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 to provide an acceptably smooth ride (low roughness).

The material used for the wearing course should not be:

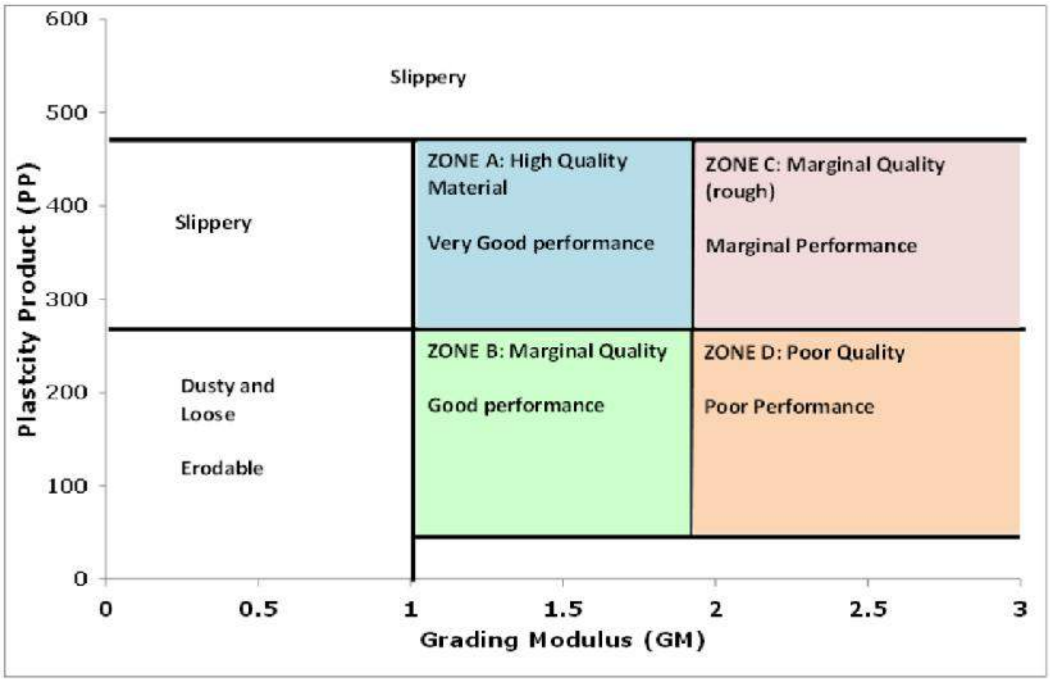
- lost too soon (low gravel loss);
- slippery; and
- greater than 40 mm maximum particle size.

The wearing course thickness depends on the annual gravel loss and the number of years between re-graveling operations. Commonly, 150 mm is used at the construction stage, and the layer is re-graveled to 150 mm thickness during each operation. Table 11-5 and Figure 11-10 show the specifications and criteria used for the selection of materials in the wearing course based on durability, rate of roughness progression and, ultimately, on whole-life costs (i.e., performance-based specifications).

Table 11-5: 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%

Source: Mukura K, 2005



Source: Mukura K, 2005

Figure 11-10: 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

c) Grading Modulus GM = [300 – (P2 + P425 + P075)]/100

Where: P_2 = percentage passing the 2.36 mm sieve

P_{425} = percentage passing the 0.425 mm sieve

P_{075} = percentage passing the 0.075 mm sieve

The particle size distribution test for the material must be carried out 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 (LVRR1/LVRR2) and major gravel roads (LVRR3/4/5), for which slightly different design criteria are applied.

Determination of subgrade strength: For both minor and major gravel roads, catalog designs have been developed based on subgrade class (as for paved roads) and ADT (minor) or CESA (major).

A DCP survey must be carried out, as described for the DCP-DN method, to determine uniform sections based on the Structural Number (SN) for each test point. The subgrade strength is then determined as described in *Chapter 10 – Structural Design Paved Road, Section 10.2.5*.

Minor Gravel Roads: Minor gravel roads (LVRR 1 and LVRR 2) are designed with two layers, a sub-base and a wearing course. If there is only one material source available, and this material satisfies the strength requirements shown in Table 11-6, then these layers can be constructed with 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 11-1.
- The recommended sub-base thicknesses and wearing course material strengths for different subgrade and traffic conditions are shown in Table 11-6.

Table 11-6: Design chart for minor gravel roads

Subgrade Class (CBR %)	LVRR1/LVRR2(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, thickness can be reduced to 150 mm

Major Gravel Roads: For major gravel roads, the design 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 greater than about 6%.
- In areas of high and intense rainfall.
- When the road shape is poor 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 7 - Traffic*).
- Determine the strength of the subgrade at the appropriate moisture condition.
- Establish the quality of the gravel that is to be used (Table 11-5). 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 gravel base that is necessary to avoid excessive compressive stresses in the subgrade from Table 11-7 or Table 11-8.
- Calculate the thickness of the wearing course based on the expected rate of gravel loss and a realistic choice of the frequency of re-graveling.

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 11-7: Gravel base thickness for major gravel roads - (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

Source: Adapted from FHWA, 2015

Table 11-8: Gravel base thickness for major gravel roads – (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

Source: Adapted from FHWA, 2015

11.5 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. However, 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 application of the products can be achieved through 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 provide some kind of financial, social or environmental benefit in terms of 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 stabilizer? This can be checked in a laboratory using traditional CBR testing. However, the application rate is critical, and some materials react better with chemicals than others. 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 the road before full-scale use. It is very difficult to test their effectiveness in the laboratory as such tests do not replicate 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 as part of the normal gravel loss process. The gravel loss may be reduced, but the road is still an unpaved road and will still be subjected to traffic and environmental erosion and material loss.

11.6 Effect of Freeze-thaw on Gravel Roads

The severe winters in the higher areas of Afghanistan can result in severe frost conditions. This will result in the freezing of water in the pavement layers and a consequent “heave” or loss of density in the materials. This has significant consequences when the frost thaws in spring, and the material is both loose and saturated, losing much of its strength. The depth of freezing depends on the temperatures reached as well as the duration of the frost conditions, but will usually affect all layers within a gravel road in Afghanistan due to the thin nature of a typical gravel road construction. Although the main problem in sealed or paved roads is the effect of freezing of subgrades and subbases, in gravel roads the biggest problem is the wearing course itself, which loses strength on thawing.

Silty materials are the most prone to frost heave and should be avoided as far as possible. This proves a problem for gravel roads where cohesion (generally provided by the silt and clay fraction) is necessary for unpaved wearing course gravels, However, provided sufficient grave (particles > 2 mm) is included and the silt fraction (0.002 – 0.075 mm) is minimized, the materials will be less sensitive to frost action. However, an adequate clay component (< 0.002 mm) is essential to provide the necessary cohesion. This requires an assessment of the full grading analysis as well as the Grading Coefficient. If materials fulfilling these requirements cannot be located, it is better to ensure a good coarse gravel is selected, although this will usually require more maintenance and could lead to loss of traction when dry and high gravel losses during precipitation events, on steep slopes.

Conventionally, in many countries subjected to prolonged frost conditions, roads are closed for varying periods during the Spring thaw. This is probably not practicable on low volume access roads in Afghanistan. However, the period of potential damage for unpaved roads is much less than for paved roads (quicker evaporation and drying out, especially under arid mountain conditions) and it is usually possible to restore the wearing course surface with light maintenance after any damage has occurred (grader blading, preferably with some additional compaction).

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Appendix 11-1: Design Examples

Design example DCP-DN method

The use of the DCP-DN 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 analyzed using the AfCAP LVR DCP program as described in *Chapter 9 – Section 9.2.6*. The results were then tabulated in a spreadsheet, as illustrated in Table 11-10, and the CUSUMs calculated for all of the DSN_{800}, $DN_{150\text{-}300}$, $DN_{301\text{-}450}$ and $DN_{451\text{-}800}$ values. This data was used to identify the uniform sections, as illustrated in Figure 11-12. 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 A11-1: Spreadsheet showing 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	

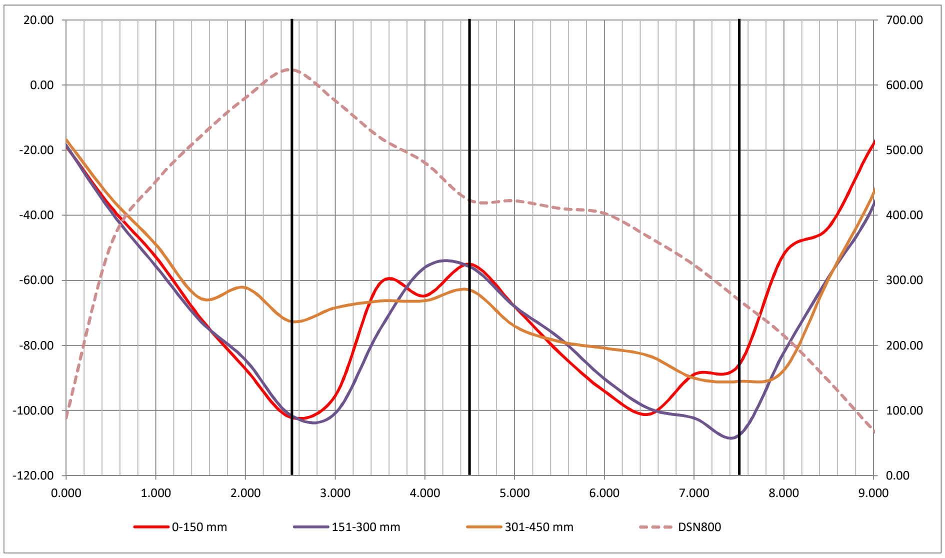

Figure A11-1: Plot of CUSUMs for different parameters

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 11-3, as shown in Table A11-2.

Table A11-2: Comparison of Layer Strength (soaked) profile against design catalog

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 requirement results were obtained for each of the uniform sections as shown in Table A11-3:

Table A11-3: 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 recompacted. 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 11-2 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 11-2 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 stabilization. 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 10 – 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 A11-4:

Table A11-4: 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 11-7 and 11-8), as shown in Table A11-5.

Table A11-5: 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
NG20	R&R	R&R	150	175
NG15	R&R	R&R	170	190

*R&R = Rip, shape and recompact existing gravel base

Appendix 11-2: Method of test for Bar Linear Shrinkage

Step 1

The interior of a clean, dry shrinkage mold is sprayed evenly with the silicone lubricant. The shrinkage molds have internal dimensions of 150 ± 0.25 mm long x 10 ± 0.25 x 10 ± 0.25 mm and made of 10 mm thick stainless-steel bar, open on two sides.

Step 2

The fines (fraction passing 0.425 mm) saved during a grading analysis are used for this test. Add water from a squeeze bottle to the fines and mix thoroughly until the consistency is at the liquid limit.

Step 3

The lubricated mold should be placed on the plate provided, and one half should be filled with the moist soil by taking small pieces of soil on the spatula and pressing the soil down against the one end of the mold and working along the mold until the whole side is filled and the soil forms a diagonal surface from the top of one side to the bottom of the opposite side (see Figure a).

The mold is now turned around and the other portion is filled in the same manner (see Figure b). The hollow along the top of the soil in the mold is then filled so that the soil is raised slightly above the sides of the mold (see Figure c). The excess material is removed by drawing the blade of the spatula once only from the one end of the mold to the other. The index finger is pressed down on the blade so that the blade moves along the sides of the mold (see Figure d). During this process, the wet soil may pull away from the end of the mold, in which case it should be pushed back gently with the spatula. On no account should the surface of the soil be smoothed or finished off with a wet spatula.

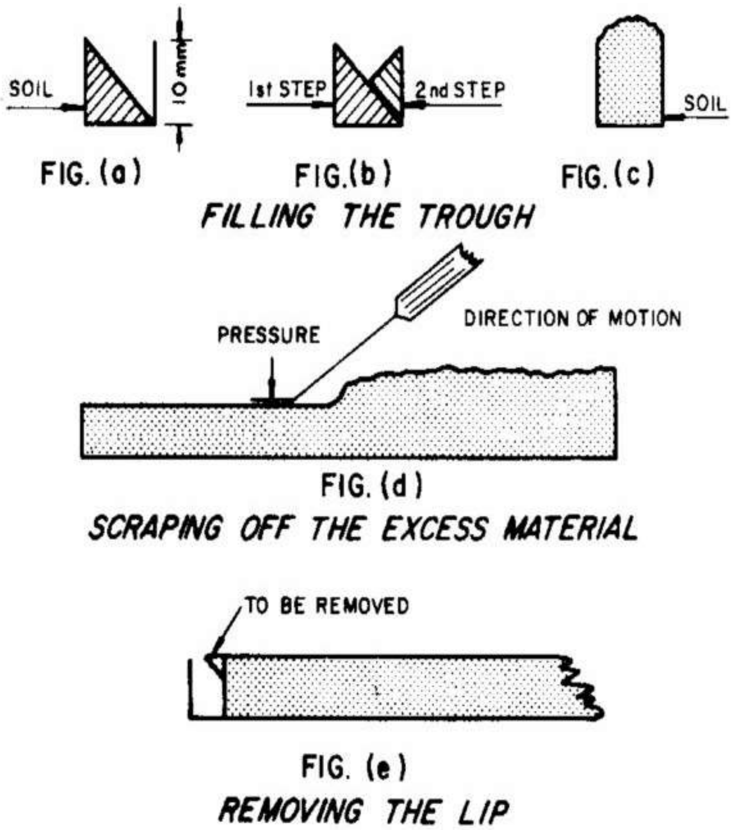

Figure A11-2: Preparation of material for the linear shrinkage test

Step 4
The filled mold is placed in the drying oven and dried at a temperature of between 105 and 110 ⁰C until no further shrinkage can be detected. As a rule, the material is dried out. The mold with the material is taken out of the oven and allowed to cool.

Step 5
It may be found that the ends of the dry soil bar have a slight lip or projecting piece at the top. These lips should be removed by abrading with a sharp, narrow spatula so that the end of the soil bar is parallel to the end of the mold (see Figure e). If the soil bar is curved, it should be pressed back into the mold with the finger-tips so as to make the top surface as level as possible.

The loose dust and sand, removed from the ends, as well as any loose material between cracks should be emptied out of the mold by carefully inverting the mold whilst the material is held in position with the fingers. The soil bar is then pressed tightly against one end of the mold. It may be noticed that the soil bar fits better at the one end than at the other end. The bar should be pressed tightly against the end at which there is a better fit. The gap between the soil bar and the end of the mold is measured by means of a good pair of dividers, measuring on a millimeter scale, to the nearest 0.5 mm and recorded.

Step 6
The bar linear shrinkage (BLS) is calculated from the measured shrinkage LS (in mm) as follows:

$$BLS = LS \times 0.67\ (\%)$$

Note: After the test, the soil bar should be examined to ensure that the corners of the mold were filled properly and that no air pockets were contained in the soil bar. If air pockets were contained, the material should be retested.

Appendix 11-3: Method of test for Treton Impact Value

Step 1
From the field sample, screen out a sufficient quantity (at least 200 g) of the -19.0 + 16.0 mm fraction. If the aggregate is noticeably variable as regards type or hardness, each type should be tested and reported separately. In this case, an estimate should be made of the percentage of each type.

Step 2
Select 15 to 20 of the most cubical pieces. Weigh the aggregate pieces to an accuracy of 1 g, and place them as evenly spaced as possible on the anvil shown in the Figure in such a manner that their tops are approximately in the same horizontal plane.

Step 3
Place the cylinder over the anvil and tighten the clamp screws. Place the hammer in the cylinder so that the top of the hammer is level with the top of the cylinder and let it drop ten times from this position.

Step 4
Remove the cylinder and sieve all the aggregate on the anvil and base plate thoroughly through a 200 mm sieve. Weigh the aggregate retained on the sieve to the nearest 0.1 g and record the mass. The test should be carried out in triplicate. (If any individual result differs from the others by more than five units, further tests should be carried out.)

Step 5
Calculate the Treton value to the first decimal place as follows:

Treton value = (A - B)/A x 100

where

A = the mass of the stone particles before tamping (g)

B = the mass of the stone particles retained on the 2.0 mm sieve after tamping (g).

Report the value to the nearest whole number.

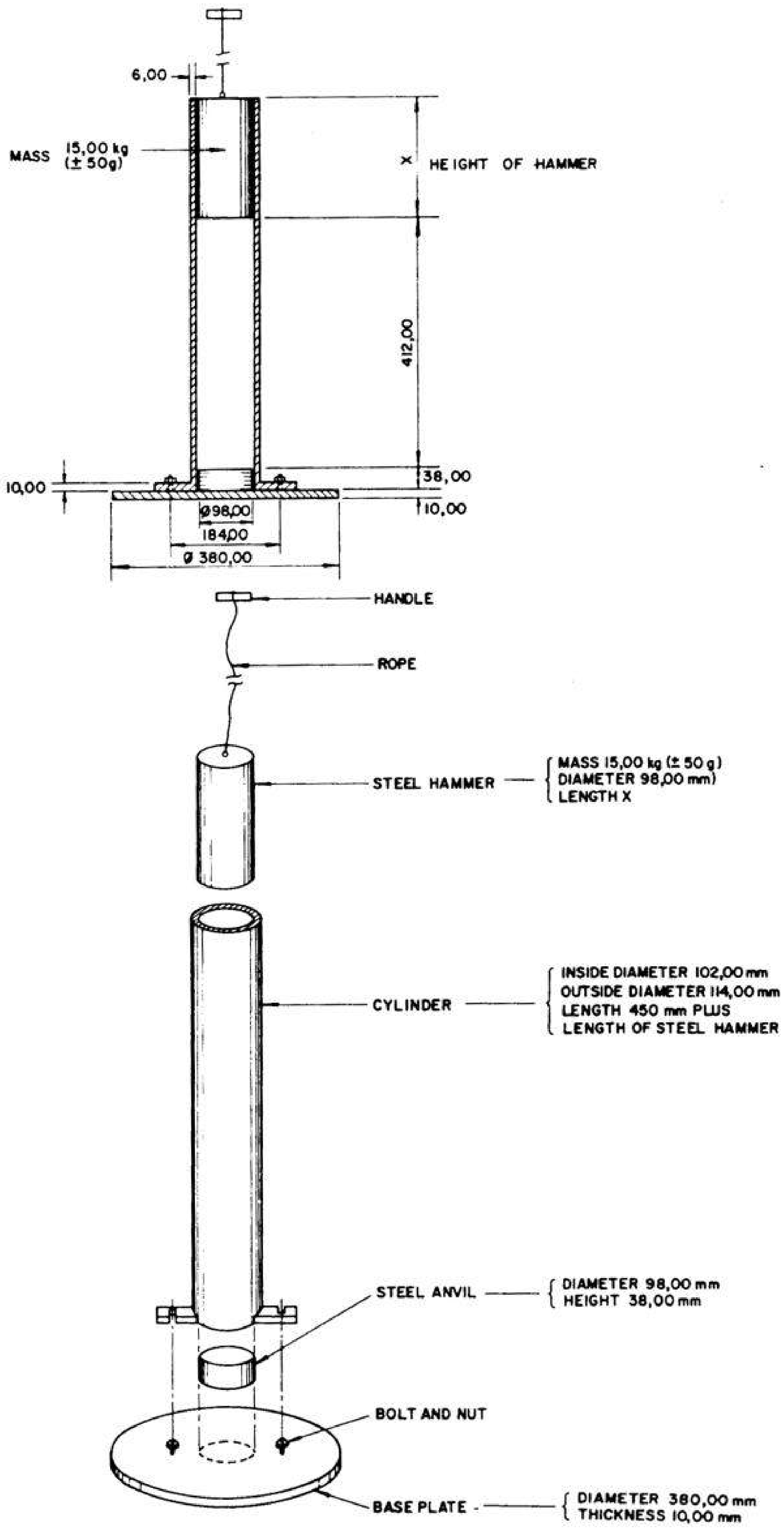

Figure A11-3: Treton Impact test apparatus

Low Volume Rural Roads Guideline and Standards
Volume 1 – Pavement Design

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  - 2. Approach to Design
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- **Section 2: Investigations**
  - 4. Site Investigations
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  - 6. Materials
  - 7. Traffic
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  - 8. Hydrology and Drainage Structures
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12.1 Introduction

12.1.1 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 LVRRs, where moisture sensitive materials are often used.

There are several surfacing options, both bituminous and non-bituminous, that are available for use on LVRRs. 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.

12.1.2 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 LVRRs.
- 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 LVRRs and are not considered in this chapter.

12.2 Bituminous Surfacings

12.2.1 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 produced in a variety of forms depending on the particular functional 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.

12.2.2 Main Types

Terminology for different surfacing types varies for different countries within the region. For purposes of this Guideline, Figure 12-1 illustrates the main types of bituminous surfacings that are potentially suitable for use in Afghanistan.

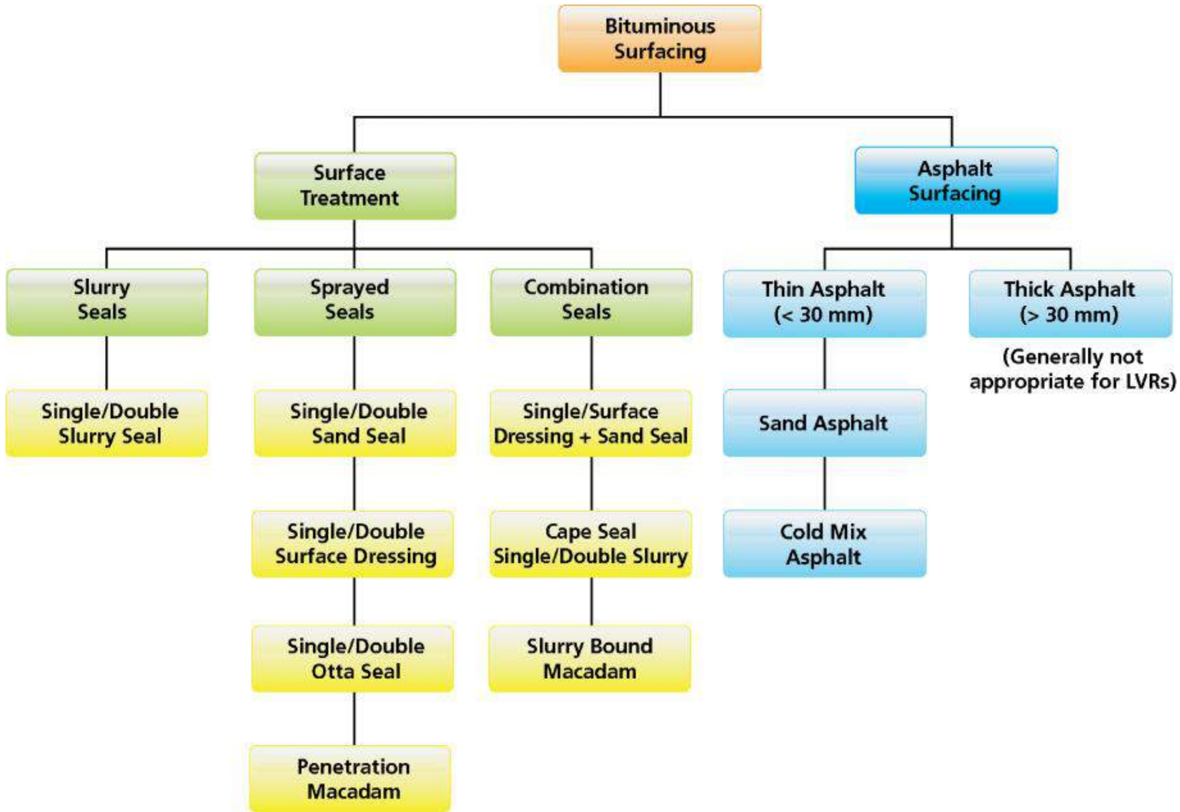


Figure 12-1: Terminology and categorization of bituminous surfacings

Some of the typical types of bituminous surfacings used on LVRRs are shown in Figure 12-2.

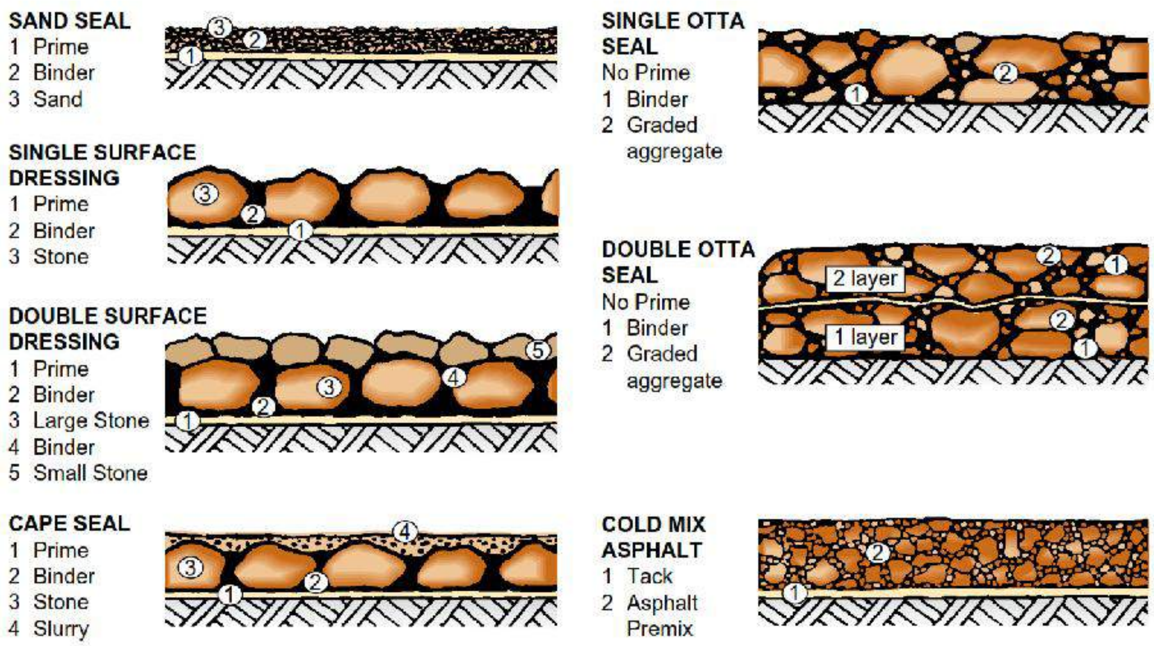


Figure 12-2: Common types of bituminous surfacings

12.2.3 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 12-3.

Figure 12-3: 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 mobilized. Under trafficking, the aggregate is in direct contact with the tire 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 12.1 indicates the relative difference in the required properties between the various surfacing types.

Table 12-1: 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. Labor-based approaches are relatively easy to adopt if desired.	Sensitive to standards of workmanship. Labor-based approaches are less easy to adopt if desired.
Durability of seal	Enhanced durability due to the use of relatively soft binders and, in the case of the Otta Seal, a dense seal matrix.	Reduced durability due to use of relatively hard binders and open seal matrix.

12.2.4 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 12-2: 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

12.2.5 General Characteristics

The general characteristics of the different types of bituminous surfacings are summarized in Table 12-3.

Table 12-3: 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 labor-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 12-3 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 LVRRs carrying only light traffic. More typically used for re-texturing surface dressings prior to resealing or for constructing Cape seals. • Very suitable for labor-based construction using a 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, it 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 labor-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 labor-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 LVRRs due to the very high rate of application of bitumen (7/9 kg/m²). Not considered appropriate for use on LVRRs in Afghanistan.
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 labor-based construction and, when an emulsion is used, requires relatively simple construction plant. • More durable than a Single Surface Dressing.

Table 12-3 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 labor-based construction and, when an 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 a light pedestrian roller (vibrating at low amplitude and high frequency). A fine slurry is normally applied after the curing of the penetration slurry. • Acts simultaneously as a base and surfacing layer. • Very suitable for labor-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 LVRRs due to the high rate of application of bitumen and may not be appropriate for use on LVRRs in Afghanistan.
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, therefore, not considered for use on LVRRs in Afghanistan.
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 labor-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.

12.2.6 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 authorities have developed various methods 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 the 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 12-4.

Table 12-4: 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

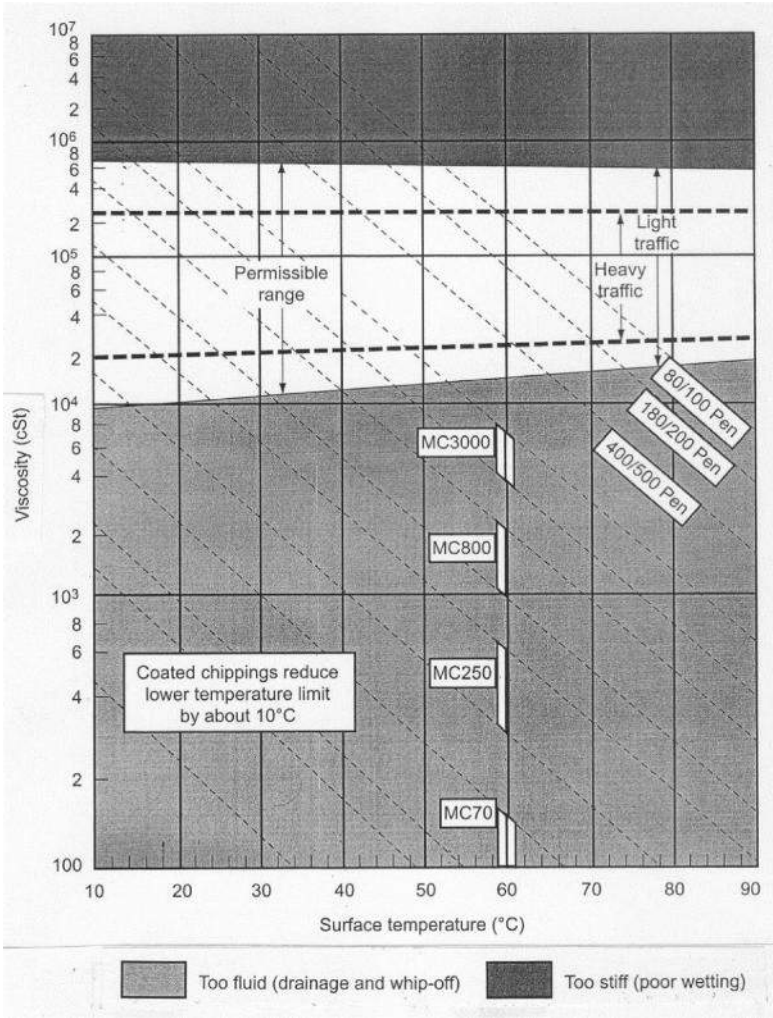
Source: TRL, 2000

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, intersections, etc.) and the environment. A correct choice of binder is, therefore, crucial for achieving a good performance of the surfacing.

The binder used in a surface treatment must fulfill 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 12-4 shows the permissible range of binder viscosity that applies to the construction of surface treatments. In Afghanistan, daytime temperatures can reach 50°C in lower-lying areas in the south-west and drop to as low as -20°C in winter in the mountainous areas in the north-east. In general, penetration grades of bitumen are most appropriate for the higher road temperatures and cutback grades for the lower road temperatures.



Source: ORN 3, 2000

Figure 12-4: Surface temperature/choice of binder for surface treatments

The following are the types of binders that are generally appropriate for use in Afghanistan.

- **Penetration grade bitumen: R**efined and blended to meet specific requirements. They are relatively stiff and are graded by their penetration. Commonly used grades in Afghanistan are shown in Table 12-5.

Table 12-5: Commonly used penetration grade bitumen

Asphalt concrete	Surface treatment
40/50 50/70	70/100 150/200

- **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 bitumen varies in behavior according to the type of cutter or flux used as the diluent. Commonly used grades in Afghanistan are:
  - MC 3000
  - MC 800
  - MC 70 and MC 30

Possible environmental impacts should be considered when using cutback 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,** which is produced 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 aging, 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 12-6 must be met. However, in the case of a relatively high proportion of motorbikes, coarser sand or grit may be considered (max. 7 mm) in order to improve the skid resistance when wet.

Table 12-6: 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)		

Source: Adapted from TRH3, 2007

For planning or tender purposes, typical binder and aggregate application rates for Sand Seals are shown in Table 12-7.

Table 12-7: Binder and aggregate application rates for Sand Seals

Application	Net bitumen application rate (l/m^2)	Aggregate application rate (m^3/m^2)
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

Source: Adapted from TRH3, 2007

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 12-8.

Table 12-8: 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)		

Source: Adapted from TRH3, 2007

For planning or tender purposes, the typical composition of the slurry may be based on the mass proportions indicated in Table 12-9.

Table 12-9: 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

Source: Adapted from TRH3, 2007

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 Tables 12-10.

Table 12-10: 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}			

Source: Adapted from ORN 3, 2000

In view of the fact that the 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 Tables 12-11.

Table 12-11: 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)				
1^{st} layer	0.015	0.011	0.012	0.010
2^{nd} 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

Source: Adapted from ORN 3, 2000

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 12-10.
2. Calculate application rates.
   a) Determine the Average Least Dimension (ALD) from the chart (Figure 12-5) 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 measured from their most stable face. Measure this for each stone with calipers and take the average.
   b) Determine the weighting factor, the sum of the individual factors given in Table 12-12.
   c) Determine the binder application rate using the following formula:

$$R = 0.625 + (0.023xF) + [0.0375+ (0.0011 \times F)] \times ALD$$

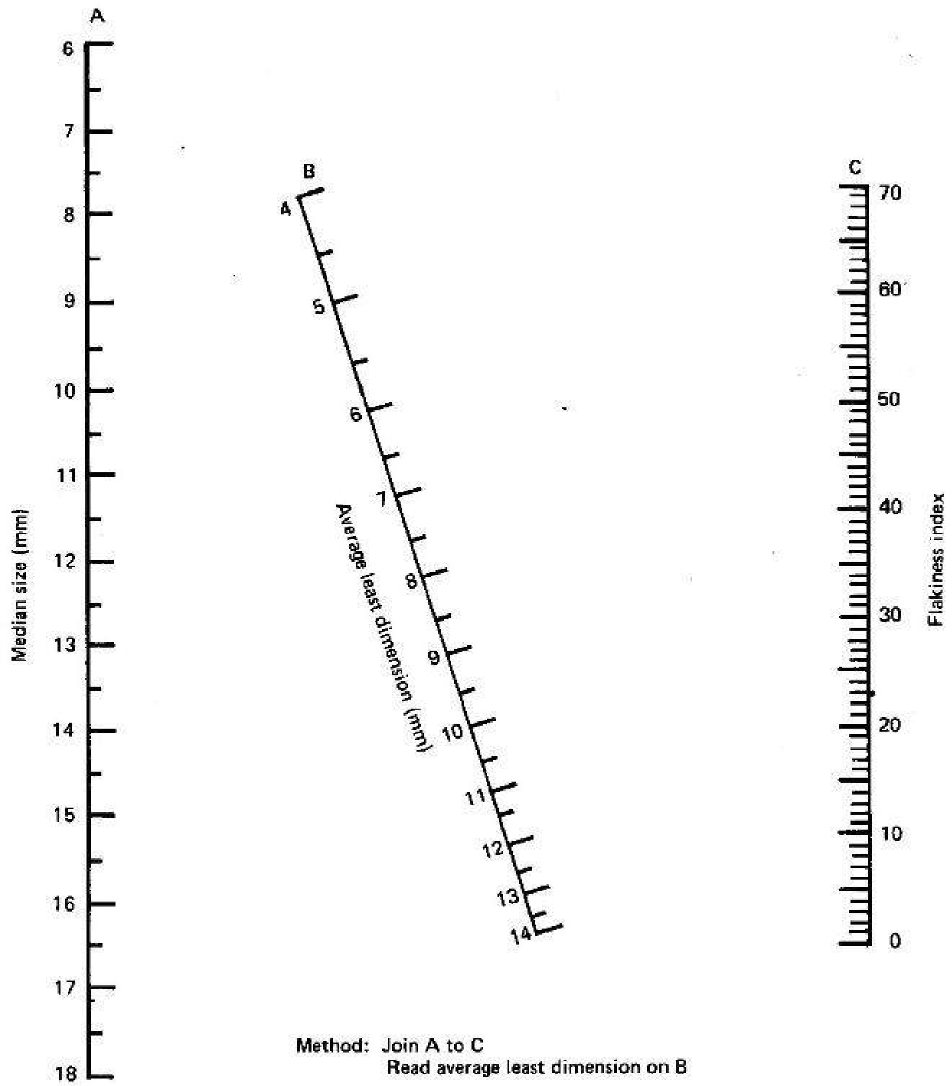
Where: F = Overall weighting factor (see Table 11-12 below)
ALD = Average least dimension
R = Rate of binder application in kg/m2

This formula is correct for MC3000 grade bitumen. Adjustment factors for different cutback 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/liter:

$$\text{Chipping application rate} = 1.364 \times ALD$$



Source: Adapted from ORN 3, 2000

Figure 12-5: Determination of Average Least Dimension (ALD)

The weighting factor, F, is obtained from Table 12-12.

Table 12-12: 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		

Source: Adapted from ORN 3, 2000

Conversions from hot spray rates in volume (liters) 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 carriageway 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 Guideline 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 12-13, where the shaded cells indicate the preferred grading in relation to traffic.

Table 12-13: 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 12-14 gives the recommended hot spray rates for primed base courses.

Table 12-14: 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 12-15.

Table 12-15: 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 12-16.

Table 12-16: 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 12-17.

Table 12-17: 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 12-18 summarizes the minimum rolling requirements.

Table 12-18: 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 minimize 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 a minimum 3 weeks - 4 weeks when any excess aggregate should be swept off.

Figure 12-6: 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 12-19.

Table 12-19: Nominal binder and aggregate application rates for a Cape Seal

Nominal size of aggregate (mm)	Nominal rates of application for tendering purposes	
	Binder (liters of net cold bitumen per m^2)	Aggregate (m^3/m^2)
14 20	0.8 1.1	0.019 0.013

Source: Adapted from TRH 3, 2007

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 cutback bitumen.

Binder: Different types of emulsion may be used for different circumstances. Table 12-20 describes the binder options.

Table 12-20: 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 labor-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 an emulsion is the loss of water from the emulsion. Determining whether an emulsion has broken is very easy as the color 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 12-21 and Table 12-22.

Table 12-21: 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%	

Source: Adapted from ILO, 2013

Table 12-22: 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

Source: Adapted from ILO, 2013

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 12-23. The exact percentage to use shall be determined after the Engineer's approval of the mix design and the result of field trials.

Table 12-23: Residual bitumen content in CMA

Aggregate grading	0/14
Residual bitumen content	5.5 – 7.0%

Source: MTRD, 2017

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 liters.
- K3 65 Cationic Emulsion: 6.5 liters.
- Water: 1 liter (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 laborers 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 leveled 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 12-7: Levelling and compaction of CMA

Compaction should be undertaken with a double drum steel roller, as shown in Figure 12-7. 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 12-7.

Care must be taken to plan the CMA works to prevent washout by rain before the emulsion has set properly.

12.2.7 Suitability for Surface treatment on LVRRs

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 LVRRs, in terms of their efficiency and effectiveness in relation to the operational factors outlined above, is summarized in Table 12-24.

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 12-24: Suitability of various surfacings for use on LVRRs

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

12.3 Non-bituminous Surfacings

12.3.1 General

There are many situations in which bituminous surfacings are unsuitable for use on LVRRs, 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 LVRRs, 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 judgment 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.

12.3.2 Main Types

The main types of non-bituminous surfacings are summarized in Figure 12-8.

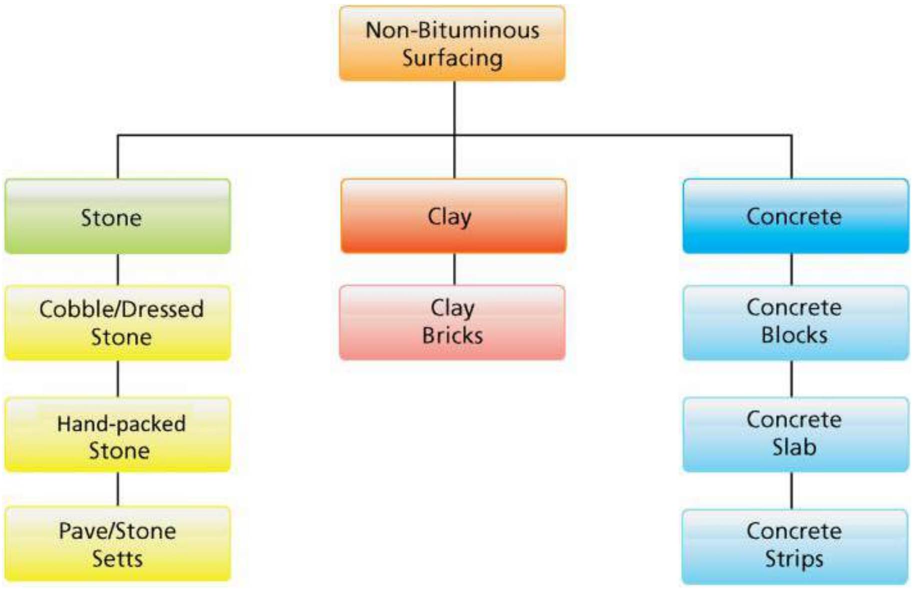


Figure 12-8: Terminology and categorisation of non-bituminous surfacing types

12.3.3 Discrete Element Surfacings

Discrete Element Surfacings (DES) are typically used on LVRRs and urban streets and parking lots, rather than on higher-level national roads. They offer several advantages over bituminous surfacings, such as long life, negligible maintenance, suitability for labor-based construction and environmental friendliness. In addition, they have the advantage that individual elements can be uplifted and replaced if damage to individual elements occurs or if there is a need to repair the underlying layers because of soil movement and deformation.

Some of the typical types of non-bituminous surfacings are shown in Figure 12-9.

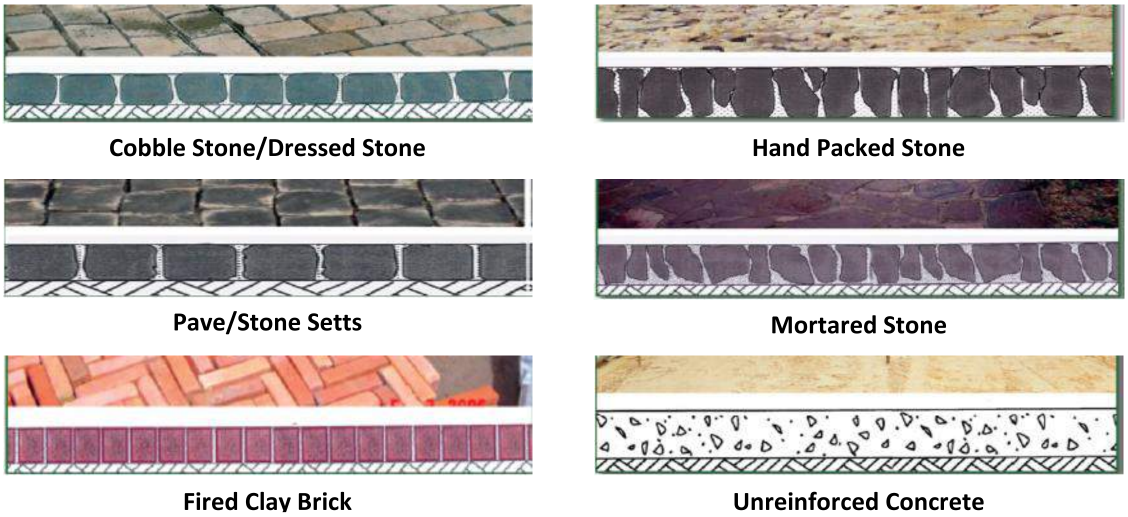


Figure 12-9: Common types of non-bituminous surfacings

12.3.4 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. Such options include 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 behavior of mortared pavements is different from that of sand-bedded pavements and is more analogous to a rigid pavement than a flexible one. There is, however, little formal guidance on the mortared option, although empirical evidence indicates that inter-block cracking may occur. Reference is made to *Chapter 10 – Structural Design: Paved Roads,* Section 10-4, 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.

12.3.5 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 LVRRs 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 LVRR pavement.

12.3.6 General Characteristics

The general characteristics of a range of non-bituminous surfacings that may be considered for use in Afghanistan are summarized in Table 12-25.

Table 12-25: 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 250 mm 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 is normally compacted by a vibratory tamper or vibratory plate compactor. • An edge restraint or kerb constructed, for example, of large or mortared stones improves durability and lateral stability.
Pave/Stone Setts	• Consists of a layer of roughly cubic (100 mm) stone setts laid on a bed of sand or fine aggregate within mortared stone or concrete edge restraints. • Individual stones should have at least one face that is fairly smooth to be the upper or surface face when placed. • Each stone sett is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. • Sand or fine aggregate is brushed into the spaces between the stones and the layer is then compacted, normally by a vibratory tamper or vibratory plate compactor.
Fired Clay Brick	• Consists of a layer of high-quality bricks, typically each 100 x 200 mm and 70-100 mm thick, laid by hand on a sand bed with joints also filled with 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 molds typically 100x200 mm and 60 mm thick. • Laid by hand, side-by-side on a 20 mm (min) 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 labor-based construction with a modest requirement for a 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.

Surfacing	Characteristics (Cont'd)
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. Increases slab strength and prevents 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 strips with panels 0.9 m wide, 3.0 m (max) in length and 200 mm in thickness unreinforced concrete strips spaced at a distance from center-to-center at 1.55 m so that both sets of vehicle wheels would run on the strips. The end of the panels on a downward slope should be thickened to act as an anchor block on each panel at grade > 10%, every 2nd panel at grade 6-10%, every 3rd panel at grade 3-6%. • Strips contain transverse concrete strips between the wheel tracks to help stop excessive erosion down the center of the strips.

12.3.7 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 10 – Structural Design: Paved Roads*.

Figure 12-10: Concrete strip road with dangerous edge drop

The non–bituminous surfacings and their typical thicknesses and strength requirements are given in Table 12-26.

Table 12-26: 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[1]
Hand Packed Stone	150 - 300	20[1]
Pave/Stone Setts	100	20[1]
Fired Clay Brick[2]	70 - 100	20
Precast Concrete Blocks	60 - 80	25
Un-Reinforced Concrete (URC)	150 -190	30
Lightly Reinforced Concrete	150 - 190	25
Concrete Strips	200	20
Mortared Stone	70	20

Notes: 1 – Uniaxial compressive strength
2 - Water absorption < 16% of their weight of water after a 1-hour soaking

12.3.8 Suitability for Use on LVRRs

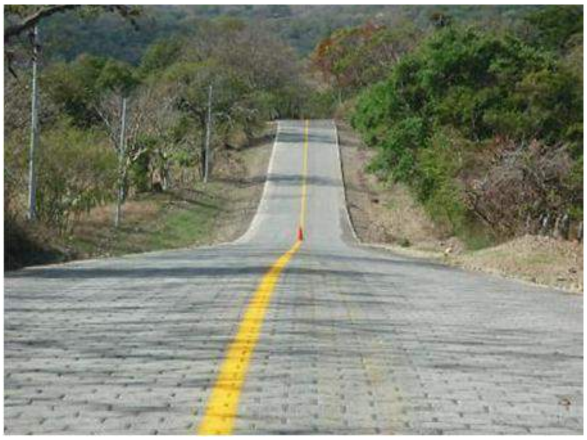
A variety of non-bituminous surfacings can be successfully applied to LVRRs and are particularly useful under certain circumstances. However, most of the discrete element surfacings provide a relatively rough running surface and their use is therefore typically limited to the situations described in Table 12-27.

An advantage of discrete element surfacings is that they are relatively easy to maintain and repair and that they are eminently suitable for labor-based construction.

As most of the non-bituminous surfacings have a long service life, when properly constructed, they can be cost-effective. However, their initial construction cost may be higher than that of bituminous surfacing options. A careful life-cycle costing of the various surfacing alternatives should, therefore, be carried out, as discussed in *Chapter 13 – Life-cycle Costing*. However, additional, less tangible benefits, such as the creation of work opportunities for communities and small-scale contractors, would have to be weighed against the purely financial considerations. Policy directives must also be taken into account.

Table 12-27: Suitability of non-bituminous surfacings on LVRRs

Surfacing type	Suitable situations	Application
Precast concrete blocks	All	1. On relatively steep gradients where high tire traction is required. 2. In high rainfall areas where slipperiness may be a problem on steep grades. 3. On severely stressed sections, such as near marketplaces and at traffic checkpoints. 4. In locations where oil spillage is likely to occur. 5. At junctions with heavy turning vehicles. 6. At parking bays with prolonged static loading. 7. When very low maintenance capability is likely. 8. When a very long service life is required. 9. Where a natural stone is in plentiful supply. 10. As a surfacing on an entire link. 11. Where traffic is very low (< 50 vpd). 12. In low-speed environments 13. On very weak subgrades. 14. Where high roughness levels are unacceptable to local road users.
Cobblestones	All except 10, 14	
Dressed stones	All except 10, 14	
Fired clay bricks	All except 1, 2, 8, 10, 13	
Hand packed stones	1, 13, 14	
Concrete strips	11, 12	
Un-reinforced concrete	All	
Lightly reinforced concrete	All	
Mortared stone	11, 12, 13, 14	



Precast concrete block surfacing

Cobblestone surfacing

Fired clay brick surfacing Dressed stone surfacing

Figure 12-11: Various discrete element surfacings

12.3.9 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 a maximum of 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 from developing and to keep them clear of vegetation and loose and oversized material.
- The gravel area between the two strips should be maintained to prevent edge-drops from 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 from 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.
- In a transition between a concrete surface and an earth or gravel surface, the end of the concrete surface should be beveled 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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13.1 Introduction

13.1.1 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 13-1, for a given analysis period, one alternative might entail the use of a relatively thick, more expensive pavement with fewer interventions (Alternative A). In contrast, the other alternative may entail the use of a relatively thin, inexpensive pavement, which requires multiple strengthening interventions (Alternative B).

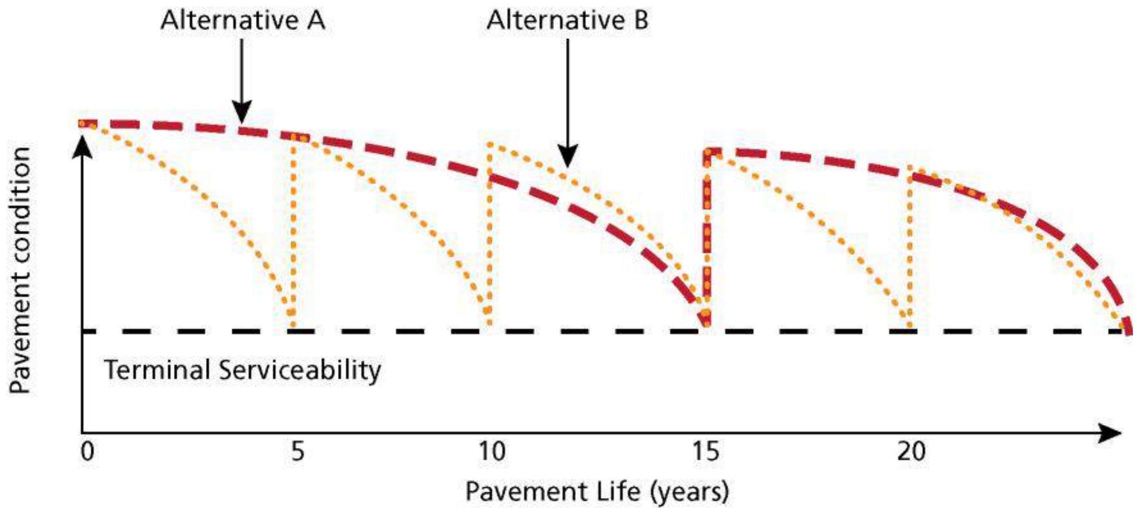


Figure 13-1: Alternative pavement options

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.

13.1.2 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 method of carrying out an LCC analysis, including the necessary inputs to the analysis.

The chapter focuses on the LCC analysis of alternative road surfacing options as well as the upgrading of unpaved roads to a paved standard. However, the principles of LCC analysis can also be applied to comparing road projects involving alternative alignments, or alternative maintenance strategies, etc., which are outside the scope of this Guideline.

13.2 Life-cycle Cost Analysis

13.2.1 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 analyze mutually exclusive projects, as this method can lead to conflicts in the ranking of projects.

13.2.2 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:

$$NPV = C + \sum M_i (1 + r)^{-X_i} - S(1 + r)^{-Z}$$

Where NPV = present worth of costs
C = present cost of initial construction
M_i = cost of the i[th] maintenance and/or rehabilitation measure
r = real discount rate
X_i = number of years from the present to the i[th] maintenance and/or rehabilitation measure within the analysis period
Z = analysis period
S = salvage value of the pavement at the end of the analysis period expressed in terms of present values

The NPV is computed by assigning monetary values to benefits and costs, discounting future benefits (b_i) and costs (c_i) using an appropriate discount rate, and subtracting the sum of total discounted costs from the sum of total discounted benefits. A positive NPV 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.

13.2.3 Components of a LCC Analysis

The principal components of an LCC analysis are illustrated in Figure 13-2. They include the initial investment or construction cost and the future costs of maintaining or rehabilitating the road, as well as the benefits due to savings in user costs over the analysis period selected. An assessment of the residual value of the road is also included to incorporate the possible different consequences of construction and maintenance strategies for the pavement/surface options being investigated.

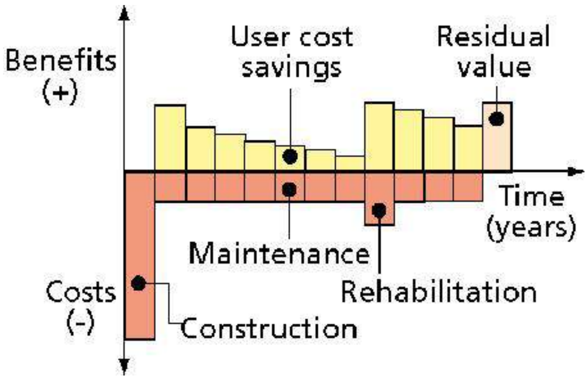


Figure 13-2: Distribution of costs and benefits during the life cycle of a road

Where a new pavement is to be constructed in which the difference in vehicle operating costs (VOC) benefits is considered negligible, then only the initial construction costs and the future maintenance costs of each type need to be considered.

The components of an LCC analysis associated with a particular design alternative are listed below and illustrated in Figure 13-3.

- Analysis period
- Structural design period
- construction/rehabilitation costs
- Maintenance costs
- Road user costs
- Salvage value
- Discount rate

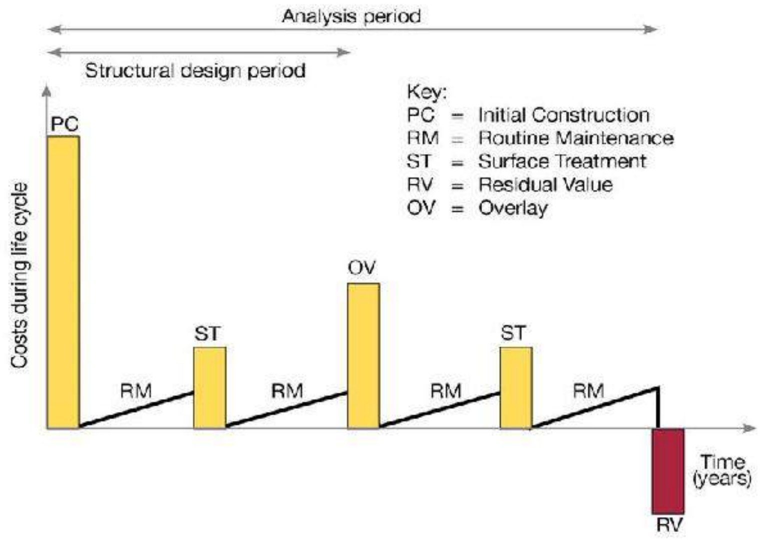


Figure 13-3: Components of a typical life cycle cost analysis

13.2.4 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 13.3)
6) Compute NPV of both options
7) Analyse results, including sensitivity analysis, if warranted
8) Decide on the preferred option, i.e., the option with the highest NPV.

In view of the uncertainty of future costs, e.g., hauling distances for gravel, 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 which time it would be expected to have reached its terminal serviceability level and to require an appropriate intervention such as an overlay.

Construction costs

Unit costs for alternative pavement designs will vary widely depending on such factors as locality, availability of suitable materials, the scale of project and road standard. Other factors that would typically warrant consideration include:

- Land acquisition costs.
- Supervision and overhead cost.
- Establishment costs.
- Accommodation of traffic.
- Relocation of services.

Maintenance costs

The nature and extent of future maintenance will be dependent on pavement composition, traffic loading and environmental influences. An assessment needs to be made of future routine and periodic maintenance as well as rehabilitation requirements.

Road user costs

These are the costs that each driver will incur in using the road system. They typically comprise vehicle operating costs (fixed costs, fuel, tires, repair and maintenance and depreciation costs), the costs of accidents and congestion, and travel time costs. VOCs are related to the roughness of the road in terms of its International Roughness Index (IRI) and will change over the life of the road due to changes in surface conditions and traffic. Relationships can be developed for main vehicle types that relate VOCs to variations in road surface conditions (IRI) under local conditions.

Road user costs are normally excluded from the LCC analysis that is confined to comparing alternative pavement/surfacing options, as the pavement options are considered to provide "equivalent service" during the analysis period. However, when evaluating the viability of upgrading a gravel road to a paved standard, the savings for the road user (primarily vehicle operating costs) on the latter versus the former option can be significant and are treated as benefits which should be incorporated as one of the components in the LCC analysis (ref. Figure 13-2).

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 cost in current value terms for construction in future to subbase level discounted to the evaluation year.

Discount rate

This rate is used as the means for comparing future expenditure in terms of present values when evaluating alternative options. It is based on a combination of policy and economic considerations.

13.3 Selection of Design Standard

13.3.1 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 minimize total life cycle costs, as illustrated in Figure 13-4. 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 LVRRs.

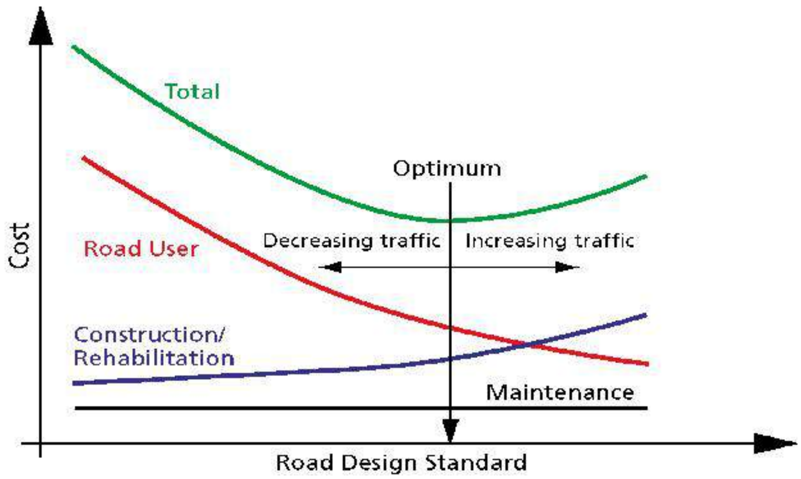
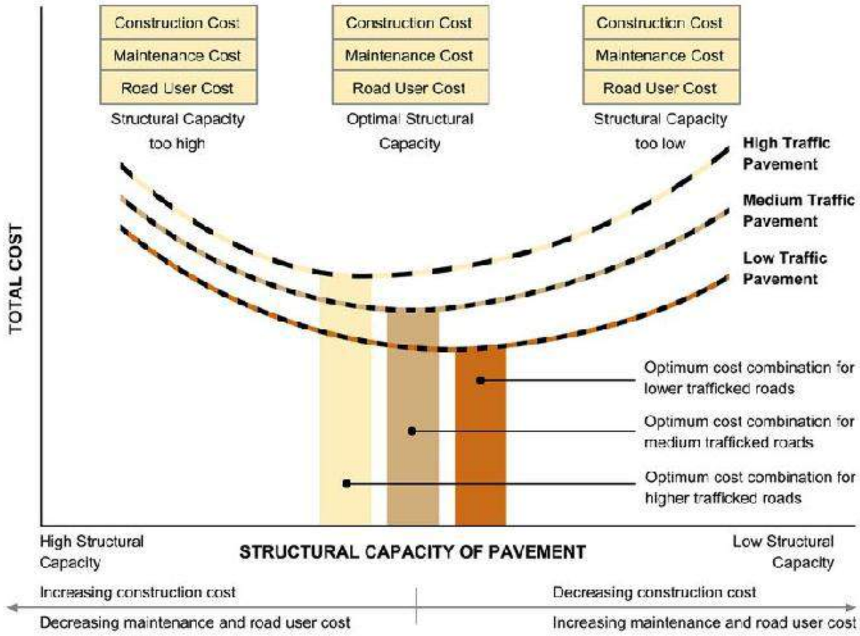


Figure 13-4: Economic analysis of optimum road design standard

As indicated in Figure 13-4, 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 13-5, 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.



Source: Van der Merwe, 1999

Figure 13-5: Combined cost for various pavement structure capacities

13.3.2 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 13-6 and Figure 13-7, 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 13-6: Gravel road option

Lower construction costs, higher maintenance and road user costs.

Figure 13-7: Paved road option

Higher construction costs, lower maintenance and road user costs.

The typical components of the LCC analysis are illustrated in Figure 13-8 and could be undertaken using an appropriate appraisal model such as the World Bank's Roads Economic Decision (RED) model which performs an economic evaluation of road investment options using the consumer surplus approach and which is customized to the characteristics and needs of low-volume roads. Using RED, the VOC relationships may need to be calibrated for local conditions. The option with the higher NPV would be the preferred one.

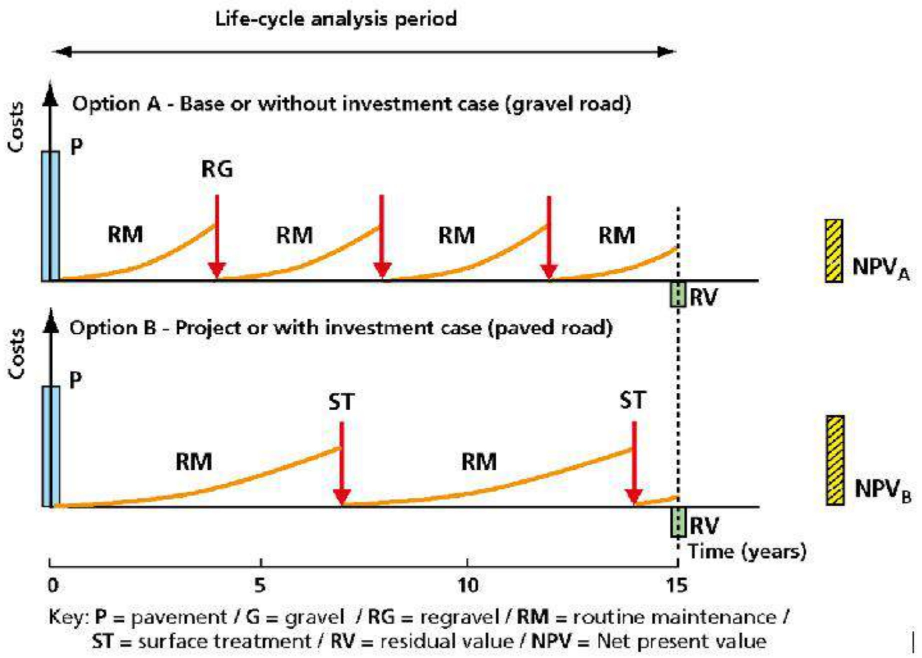


Figure 13-8: Typical components of a LCC: Gravel versus Paved road

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, rule-of-thumb 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 13-1.

Table 13-1: 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-motorized transport	Increased benefits
Quantified adverse impacts of traffic on gravel roads	Increased benefits
Reduced environmental damage	Increased benefits
Quantified assessments of social benefits	Increased benefits

The impact of these factors is illustrated conceptually in Figure 13-9, which reflects the outcome of recent research and which indicates that, in principle, in some circumstances, bitumen sealing of gravel roads may be economically justified at traffic levels of less than 100 vpd. This is in contrast to the previously accepted figures, which indicated a first-generation bitumen surface at a traffic level of over 200 vpd.

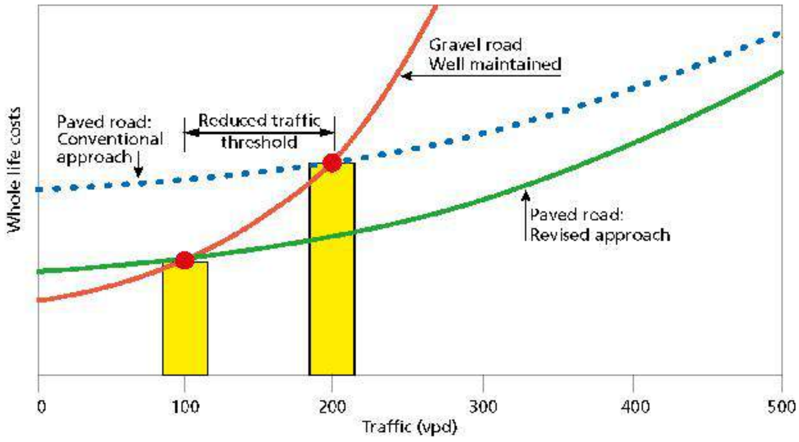


Figure 13-9: Break-even traffic levels for paving a gravel road: Traditional versus revised approaches.

13.3.3 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 13-10, Table 13-2 and Table 13-3 illustrate the manner of undertaking an LCC analysis for two typical types of bituminous surfacings by comparing the PV of all costs and maintenance interventions that occur during a given analysis period using a 12% real discount rate.

As indicated in Table 13-2 and Table 13-3, the Single Otta Seal + Sand Seal Option has the lower PV of costs and is the preferable option on economic grounds. This example is a hypothetical one used for illustrative purposes only and does not necessarily reflect a real-life situation.

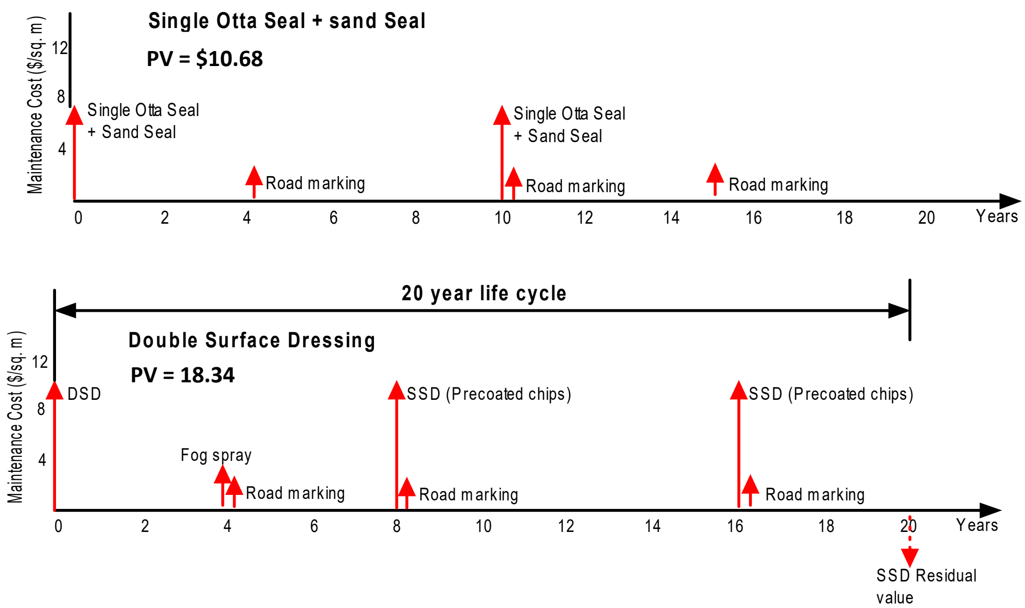


Figure 13-10: LCC comparison between a single Otta Seal + Sand Seal and a DSD

Table 13-2: Life-cycle cost analysis for Double Surfacing

Activity	Years after construction	Base Cost/m² ($)	12% Discount Factor	PV of Costs/m² ($)
1. Construct Double Chip Seal	-	10.00	1.0000	10.00
2. Fog spray	4	02.00	0.636	1.27
3. Road marking	4	0.96	0.636	0.61
4. Single Chip Seal (pre-coated)	8	10.00	0.404	4.04
5. Road marking	8	0.96	0.404	0.39
6. Fog spray	12	2.00	0.257	0.51
7. Road marking	12	0.96	0.257	0.25
8. Single Chip Seal (pre-coated)	16	10.00	0.163	1.63
9. Road marking	16	0.96	0.163	0.16
10. Residual value of surfacing	20	(5.00)	0.104	(0.52)
				Total 18.34/m²

Table 13-3: Life-cycle cost analysis for Single Otta Seal + Sand Seal

Activity	Years after construction	Base Cost ($)	12% Discount Factor	PV of Costs ($)
1. Construct single Otta Seal + sand Seal	-	7.25	1.00	7.25
2. Road marking	5	0.96	0.567	0.54
3. Single Otta reseal	10	7.25	0.332	2.41
4. Road marking	10	0.96	0.332	0.32
5. Road marking	15	0.96	0.183	0.16
Assume life span of 20 years. Thus, no residual value.				0.00
				Total 10.68/m²

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Section 4
Miscellaneous

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Volume 1 – Pavement Design

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  - 2. Approach to Design
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Chapter 14 – Practical Considerations

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14.1 Introduction

14.1.1 Background

The concepts and technology for cost-effective provision of LVRRs 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 Guideline. Nonetheless, there are several aspects of LVRR provision, which are intricately linked to successful LVRR project implementation, that is not covered in-depth in this Guideline. 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.

14.1.2 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 LVRRs.

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.

14.2 Engineering Adaptations to Climate Change

14.2.1 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, which is more vulnerable than the higher-order network. These roads are often constructed to lower standards using local materials and labor and are thus more susceptible to climate damage than higher trafficked roads.

To reduce this impact, new roads must be designed to incorporate 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 prioritize 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 to assist with prioritization, it is necessary to carry out visual assessments of existing roads (in addition to the conventional routine assessments for pavement management purposes). Particular attention should also be 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. Only those that are deemed to be particularly vulnerable should be identified and improved wherever possible. Thus, for existing LVRR infrastructure, "retrofitting" the most vulnerable facilities to a climate-resilient condition is often required, but this is a costly option.

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

To take account of future changes in climate, either in the design of new roads and structures, or in improving the resilience of existing infrastructure, various steps need to be followed.

1. The expected changes in climate along the road should be identified from the best available maps, i.e., those with the most detailed predictions.

2. These changes will generally include increased temperatures, decreased rainfall, more extreme rainfall events and increased numbers of consecutive very hot days. More windy conditions should also be taken into account where appropriate.
3. The expected effects of these on the infrastructure need to be assessed.
4. The road, road environment (earthworks and pavement drainage structures) and any larger drainage structures need to be assessed. This should follow a standard assessment protocol as outlined in the ReCAP or other relevant guidelines and concentrate on those issues not normally assessed during standard visual assessments for Road or Bridge Management Systems.

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

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

Engineering adaptation may include measures such as:

- **Pavement sealing:** Particularly for steep gradients (> 8-10%).
- **Additional or enlarged culverts:** Additional or enlarged or improved existing cross culverts considered essential to improve overall road drainage.
- **Side drainage:** Additional side drains and associated turnouts. Scour checks where necessary, lined drains required with gradients >6%.
- **Raised embankments**: Raising of earth embankments where the alignments are low and impacted by flooding and/or the weakening of the pavement by saturation.
- **Culvert or bridge abutment protection:** Gabion, concrete, masonry or bio-engineering protection where erosion of abutments is identified as a significant risk.
- **River/stream erosion protection:** Gabion, concrete, masonry or bio-engineering protection where erosion of the alignment by rivers or streams is identified as a significant risk.
- **Cut and fill slope protection:** Gabion, concrete, masonry or bio-engineering protection where erosion or deterioration of existing earthwork slopes is identified as a significant risk.
- **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 drifts and low-level structures might need to be replaced by climate-resilient structures, such as vented fords or submersible bridges.

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

14.3 Environmental Issues

14.3.1 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 minimized. Accordingly, it is necessary to analyze the construction, maintenance and routine operations associated with a road project to identify the potential environmental impacts that might arise. To this end, information must be provided to help design mitigation and/or preventative measures to ensure the protection of the natural and social environment and cultural heritage of Afghanistan.

Environmental issues in Afghanistan are governed by the Environmental Law of 25th January 2007, in which the environment is defined as follows:

“Environment”2means2natural2resources,2interactions2between2the2components2of2natural2resources2 and between those components and humans or animals, and physical, aesthetic and cultural qualities that may affect the health and well-being of humans.

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 14-1, 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 14-1: Cornerstones of the Road Environment

Road Environment	**Ecological**	• impact on flora and fauna • deforestation • disturbance of natural eco-systems and 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 • sterilization 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 • health and safety (e.g., hazardous cut slopes and borrow pits) • potential transmission of communicable diseases
	Physical	• aesthetic (e.g., loss of natural beauty and scars on landscape) • destabilization of soils and slopes • noise, air, water pollution • dust impact • interruption and modification of natural drainage systems

Although most LVRR projects by design minimize 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.

14.3.2 Typical Causes of Environmental Problems

The following are typical causes of environmental problems related to the provision of LVRRs.

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 characterized 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.

14.3.3 Regulatory Framework

As stated above, all environmental issues in Afghanistan are governed by the Environmental Law of January 2007.

Any person or legal entity proposing to undertake a project or activity shall submit to the National Environmental Protection Agency (NEPA) accurate

The power to determine the potential negative or positive environmental impacts of a proposed policy, plan, project or activity, as well as the severity of such impacts, is vested in the National Environmental Protection Agency (NEPA).

Acting on the advice of the Environmental Impact Assessment (EIA) Board of Experts, NEPA may authorize the policy, plan, project or activity, with or without conditions, if the adverse effects are unlikely to be significant. Conversely, if NEPA has determined that the adverse effects are likely to be significant, it may require the proponent to submit an Environmental Impact Assessment (EIA) and a comprehensive mitigation plan in accordance with the law.

14.3.4 The ESIA Process

Environmental and Social Impact Assessment (EIA) 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 optimize 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 interaction with them. The framework for the EIA process is presented in Table 14-2.

Table 14-2: A framework for the ESIA process

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 undertaken. For projects which are large in physical extent and/or pose particularly sensitive biophysical or socio-economic concerns, a full Environmental and Social Impact Assessment (EIA) would be required, out of which a full ESIA report would be produced. For small projects, or those with limited potential impacts, a less extensive assessment is required, and an abbreviated ESIA report will be sufficient.

The project proponent, for example, the MRRD or any other road agency, would be the responsible entity to ensure that an ESIA is carried out in accordance with the law and a report submitted to NEPA. MRRD 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 EIA, to ensure that appropriate mitigation measures can be included in the Environmental Management Plan to be included in the submission to NEPA.

Relevant information from a design team that may be necessary for the EIA process would include:

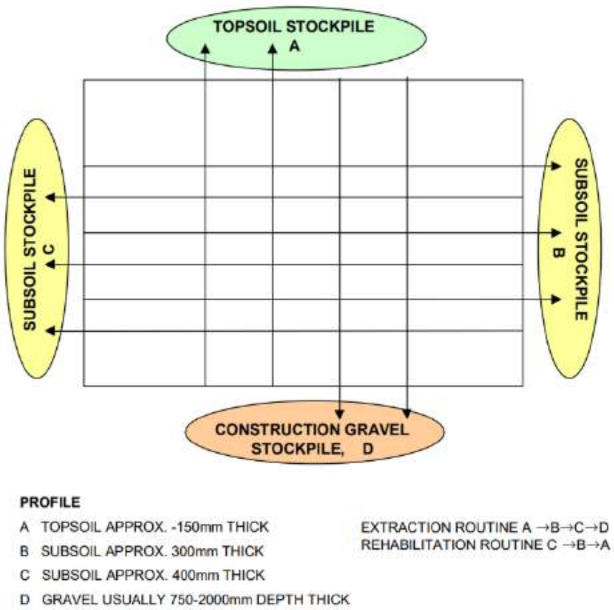
- Road alignment and reserve width;
- Classification of materials along the alignment, with an indication of their erodibility
- Gradients along the alignment;
- Locations of potential borrow pits and quarries;
- Likely routes for borrow materials overhaul;
- Locations of potential water sources;
- Anticipated 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.

14.4 Borrow Pit Management

14.4.1 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 LVRR is essential for achieving cost-effective construction and ongoing maintenance operations.



Source: Gourley and Greening, 1999

Figure 14-1: Recommended procedure for removal of overburden and stockpiling

Up to 70% of the construction cost of a typical LVRR may relate to pavement materials production and supply. Also, aggregate replacement costs are often as high as 60% of the maintenance costs of 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 basecourse are of little use if they are used wastefully used in earthworks layers. Too often, borrow pit excavation is carried out with only the plant operator present without vigilant supervision. In many cases, this results in good quality gravel getting contaminated and having to be spoiled. Good management of materials (including skillful 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.

14.4.2 Main Factors to Consider

There are many, and varied, factors to consider when locating and exploiting borrow pits, such as:

Figure 14-2: Children exposed to risk of drowning and water-borne disease

- Minimizing environmental impact by reducing the dust and noise pollution for residents in the area.
- Preparing of a borrow pit plan to ensure that topsoil and overburden are stockpiled for later use in the restoration of the borrow pit and separation of different material types/qualities in different stockpiles to ensure that materials are not contaminated.
- Draining of the borrow pit to:
  - ensure accessibility and that the materials are not getting soaked;
  - prevent accidental drowning of children;
- Ensuring adequate security for people and livestock by fencing, if required, and avoidance of steep cut slopes.
- Preventing 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.
- Minimizing watercourse pollution.

14.5 Labor vs. Equipment

14.5.1 General

Given the emergence of labor-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 labor-based projects.

Many items need to be investigated in terms of their suitability for labor-based methods (LBM) of construction or maintenance. Contractual aspects need to be established, and appropriate designs prepared. Such planning must extend beyond engineering technology and the practicality of construction and consider such factors as the financing and management of labor-based projects.

In the final instance, the use of labor-based methods in the provision of LVRRs is a policy issue that should be adhered to in the undertaking of LVRR projects.

14.5.2 Project Design

Many LVRR projects lend themselves readily to the application of LBM in that the road alignments normally follow the existing ground level quite closely. The avoidance of large quantities of earthworks makes such projects conducive to the use of the LBM approach

In the design of small structures, different options are available, some of which would permit the use of labor and others would not. Thus, from the outset, the designer is faced with the choice of whether or not to design for the use of LBM. In many cases, a combination of labor- and equipment-based technology would be the optimal solution.

14.5.3 Construction Strategy

General

LVRRs 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 labor- 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.

Labor-based technology

Labor-based technology can be defined as the construction technology which, while maintaining cost competitiveness and engineering quality standards, maximizes opportunities for the employment of labor (skilled and unskilled) together with the support of light equipment and with the utilization of locally available materials and other resources. This implies that LBM do not exclude the use of machines since 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 labor.

The objective of this approach is to maximize the number of job opportunities per unit of expenditure. However, despite the substantial potential benefits offered by labor-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 labor-based approach is to be successfully deployed on LVRR projects.

Common myths:

a) *Standards should be lowered to allow for labor-based methods.*

It commonly believed that standards <u>must be</u> 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 laborers is incorporated in the project design. Examples abound with successfully executed LBM projects for construction of low volume sealed roads, as illustrated in Figure 14-3.

Figure 14-3: Shutter system for construction of ETB base by LBM

b) *Labor-based construction is out-of-date and incompatible with the modern world.*

Where employment opportunities are scarce, laborers 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 labor-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) *Labor-based construction is only for welfare relief schemes.*

Poorly managed relief programs have contributed to the bad reputation of LBM for infrastructure provision and should not be promoted.

e) *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 labor can be used to keep costs down.*

Voluntary labor on LVRR 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 labor-based construction where appropriate.

Suitability of construction activities for labor-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 14-4: Tree removal and de-stumping

Figure 14-5: Screening of aggregates

To facilitate the increased use of LBM, emphasis should be placed on developing designs to increase labor 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 labor if standards are not to be compromised. For example:

- Compaction plant is required for pavement layers. However, light plant that is more compatible with LBM can be used successfully if layer thicknesses are reduced.
- Sheepsfoot and padfoot 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 usually requires heavy impact or heavy vibratory rollers.

Labor-based projects usually employ a relatively large number of laborers. 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 amount of materials to be moved within a fixed construction period, are the governing factors when determining whether labor-based or plant-intensive methods are to be used. However, even the largest plant-intensive projects can accommodate many labor-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.

14.6 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 LVRR 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 14-6. 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.

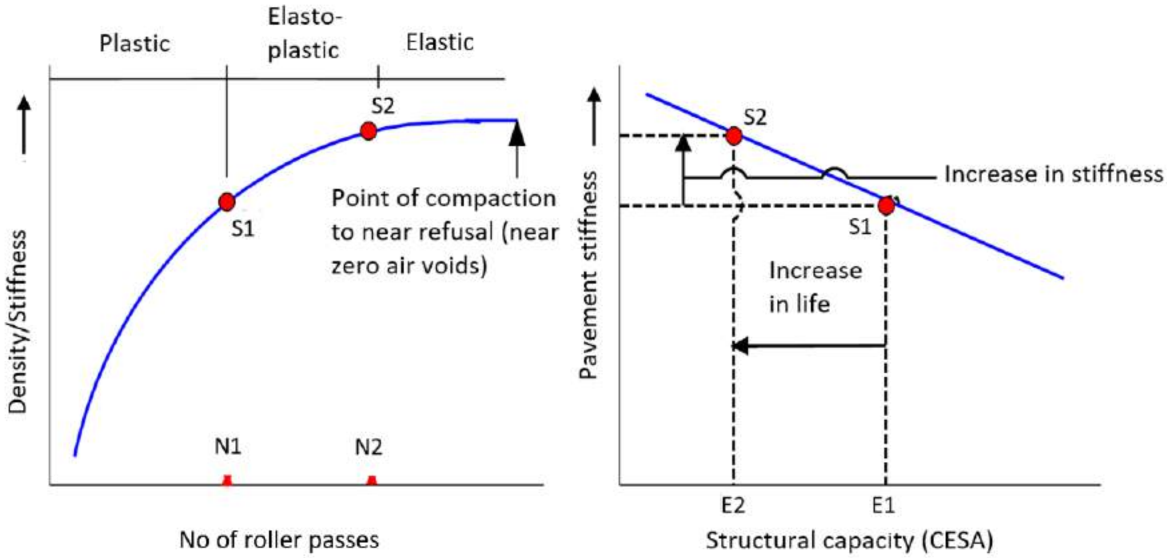


Figure 14-6: Benefits of compaction to refusal

14.7 Quality Assurance and Control

14.7.1 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 LVRRs 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 LVRR is likely.

In developing a QC/QA system, it should be borne in mind that LVRR projects in Afghanistan 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 the skills of 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.

14.7.2 Approach

There are various means of ensuring that an acceptable quality of the final LVRR 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.

14.7.3 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 is 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, 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 LVRR projects. Alternatively, these tests may be replaced for quality control purposes by the easier-to-perform DCP test. This test is undertaken initially in conjunction with in situ density and moisture content testing for correlation purposes to establish a "target" DN value, as illustrated in Figure 14-7 below. The attainment of this target value 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. This is due to the lesser confinement of the material in the field compared to the confinement in the CBR mold, as well as the presence of pore pressure.

Compaction control is typically based on randomly-located 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 an acceptable level of compaction. It is for this reason that a statistical approach to quality control should be adopted, particularly for the larger LVRR projects.

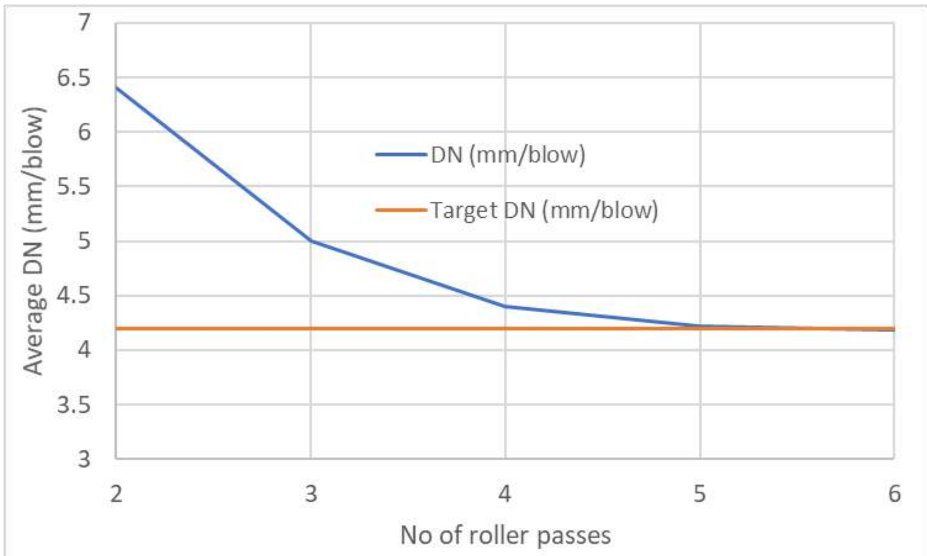


Figure 14-7: Establishment of Target DN for compaction quality control

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 utilized and continuously monitor its activities during operation. The QA would then be prescribed by certification of QC tests and reports based on intermittent field inspections.

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 offers 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 LVRRs are sometimes limited, depending upon the size of the contract. In such a situation, it is important to utilize 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 that a 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.
- Correlation of results obtained from relatively simple methods of testing with those from more complex methods (e.g., correlation of the DCP-DN value with density/CBR).
- Laboratory testing for 'calibration' of method specifications.

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 LVRR:

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 endpoints are in place.
- Ensure that longitudinal and transverse joints are correctly constructed.
- 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 site.
- Ensure that the tested stockpile(s) is the only one(s) used.
- Prepare a method specification for compaction control using, for example, 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. This requires that the materials are processed at or about OMC and rollers of adequate mass are used. Trial sections should be constructed using the materials and plant that will be used on site, and that the increase in density is 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, and must be reflected in the project specifications and payment items.

14.8 Maintenance

14.8.1 General

Road maintenance is an integral component of the LVRR 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 LVRRs 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 14.2 above, LVRRs 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. Regularly and adequate maintenance should, therefore, be of the highest priority for road agencies. This can be ensured by including a term maintenance component in the project whereby the contractor is obliged to carry out maintenance as part of the project package, e.g., through a Performance-Based Contract lasting for, say, five years.

14.8.2 Importance of Maintenance

The case for maintenance is a compelling one. Having spent time, effort and money in planning, designing and constructing an LVRR, 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 “cost” phases from which no benefits are derived. In practice, the major benefits are derived from the maintenance and operations phases that occupy by far the largest proportion of the total project cycle.

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.

14.8.3 Typical Maintenance Activities

General

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 14-3: 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-graveling / reshaping		X

Many of the activities in Table 14-3 can be carried out cost-effectively using LBM. Some periodic maintenance work may still require specialized equipment, e.g., bitumen sealing operations, but LBM 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 minimize organizational/institutional arrangements and costs.

Winter maintenance

Road maintenance during the winter season in the mountainous areas causes particular problems which must be attended to regularly to ensure the safe passage of vehicles:

Snow removal: Snow covering the road, and sometimes obscuring the actual road alignment, can be removed manually or by snowplows. To aid both the maintenance crews and vehicle drivers to see the road alignment, the installation of semi-permanent wooden or bamboo edge markers, as shown in Figure 14-8, is a cost-effective solution.

Removal of slipperiness: Roads covered in snow and ice can be extremely slippery, particularly in temperatures around zero degrees. Slipperiness is best removed by applying coarse sand on the icy surface or road salt, which lowers the freezing point of water and can remove thin films of ice. Road salt has the added effect of acting as a dust palliative on gravel roads in the summer months if any of it remains on the road from the winter months.

Source: Gardsdrift.no Source: Hafjellmaskin.no

Figure 14-8: Wooden edge markers for winter maintenance

14.9 Overload Control

LVRRs are typically designed to carry a given number of traffic load applications over their design life. The damage caused by a particular load is related to the AASHO 4th Power Law which postulates an exponential relationship between axle loads and damaging power. Thus, if a single axle is overloaded by just 20 % over an assumed legal limit of 10 tons, i.e., loaded to 12 tons, then the damaging effect would be just over twice that of the legally loaded vehicle, assuming a power exponent of 4.0. Moreover, if the pavement were to be continually subjected to such overloading in service, its life would be reduced from 20 years to just less than 10 years. This example illustrates the vital importance of controlling vehicle overloading on LVRRs.

As illustrated in Figure 14-9, the impact of overloading on a LVRR is to accelerate its deterioration and to cause the pavement to reach its terminal serviceability well before the end of its design life. As a result, it becomes necessary for rehabilitation to be carried out sooner than necessary.

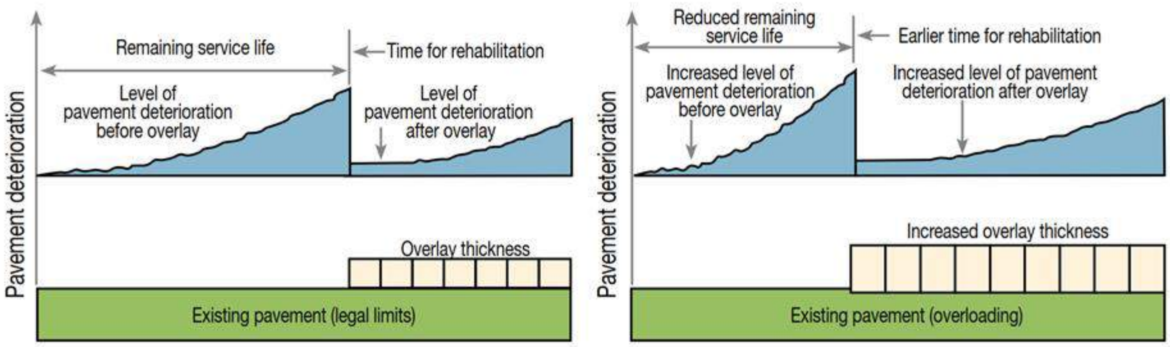


Figure 14-9: Impact of overloading on pavement performance

In view of the above, every effort should be made to limit the amount of overloading (illegal loading). Ideally, overload control on LVRRs 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, as shown in Figure 14-10, could also be considered to prevent access to heavy vehicles, especially on typical access roads. However, such measures may require political acceptance and amendment of existing regulations.

While the design process should take account of the heavy vehicle axle loads in determining the design traffic loading (refer to *Chapter 7 – Traffic*), the specific effects of the very heavy abnormal axle loads on the pavement must also be considered in finalizing the design. The challenge is that the traffic loading characteristics for LVRRs are not well known and are often difficult to predict over the design life of the individual road.

Figure 14-10: Steel gantry to prevent heavy vehicles from entering access roads

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 specialized advice/analysis in order to maintain a well-balanced structure.

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Glossary of Terms

Glossary of Terms

A

AADT Annual Average Daily Traffic is calculated by counting the number of vehicles passing a roadside observation point in a year and dividing this number by 365. The number given is the sum of both directions.

Abutment An end support of a bridge or similar structure. These are often designed

Access 1 The driveway by which vehicles and/or pedestrians enter and/or leave property adjacent to a road.

Access 2 To come to or reach a destination.

Access Control The condition whereby the road agency either partially or fully controls the right of abutting landowners to direct access to and from a public street or road.

Adverse Cross Fall A slope on a curved pavement that generates forces detracting from the ability of a vehicle to maintain a circular path.

Alignment The geometric form of the centreline (or other reference line) of a carriageway in both the horizontal and vertical directions.

Alignment Co-ordination (Co-ordinated Alignment) A road design technique in which various rules are applied to ensure that the combination of horizontal and vertical alignment is both safe and aesthetically pleasing.

Aquaplaning Full dynamic aquaplaning occurs when a tyre is completely separated from the road surface by a film of water.

Arterial Highway designed to move relatively large volumes of traffic at high speeds over long distances. Typically, arterials offer little or no access to abutting properties.

Auxiliary Lane A portion of the carriageway adjoining the through traffic lanes for speed change, or for others purposes supplementary to the through traffic movement.

Average Daily Traffic (ADT) Total volume of traffic during a given time period in whole days, greater than one day but less than one year, divided by the number of days in the period.

Average Recurrence Interval (ARI) The Average Recurrence interval (ARI) is the average interval of interval time during which an event will be equalled or exceeded once. It should be based on a lengthy period of records of the event. Statistically it is the inverse of the Average Exceedance Probability. The term replaces recurrence interval.

Average Running Speed The distance summation for all vehicles divided by the running time summation for all vehicles. Also referred to as space mean speed whereas time mean speed is simply the average of all recorded speeds.

Axis of Rotation The line about which the pavement is rotated to super-elevate the roadway. This line normally maintains the highway profile.

B

Barrier An obstruction placed to prevent vehicle access to a particular area.

Barrier Kerb A kerb with a profile and height sufficient to prevent or discourage vehicles moving off the carriageway.

Barrier Sight Distance The limiting sight distance below which overtaking is legally prohibited.

Bicycle or Motorcycle Taxi The use of a bicycle or motorcycle to transport people for gain.

Black Spot A site on a road where accidents happen at regular intervals.

Braking Distance The distance required for the braking system of a vehicle to bring the vehicle to a stop from the operating speed.

Bridge A structure erected with a deck for carrying traffic over or under an obstruction and with a clear span of six metres or more. Where the clear span is less than six metres, reference is to a culvert.

Broken-back Curve Two curves in the same direction with a tangent shorter than 500 metres long connecting them.

Bus Bay An auxiliary lane of limited length at a bus stop or terminus, usually indented into the shoulder or verge.

Bus Stop An area in which one or more buses load and unload passengers. It consists of one or more loading areas and may be on line or off line.

C

Camber The slope from a high point (typically at the centre line of the highway) across the lanes of a highway. Negative camber refers to a central low point, usually with a view to drainage of a small urban street or alley.

Capacity The maximum number of vehicles that can pass a point on a highway or in a designated lane in one hour without the density being so great as to cause unreasonable delay or restrict the driver's freedom to manoeuvre under prevailing roadway and traffic conditions.

Carriageway The lanes of the cross-section. The carriageway excludes the shoulders.

Catch Drain A surface channel constructed along the high side of a road or embankment, outside the batter to intercept surface water.

Catchment Area The area that will contribute to the discharge of a stream after rainfall at the point under consideration.

Catchwater Drain Located above a cut face to ensure that storm water does not flow down the cut face causing erosion and deposition of silt on the roadway.

Centre Line The basic line that defines the axis or alignment of the centre of a road or other works.

Channel Grading Where side channels are designed to gradients that differ from those of the road centre line, typically on either side of the highest points on crest curves and the lowest points on sag curves where the centre line gradient is less than 0.5 percent.

Channelization The use of pavement markings or islands to direct traffic through an intersection.

Channelized Intersection An intersection provided with channelized islands.

Clear Zone An area adjacent to the traffic lane that should be kept free from features potentially hazardous to errant vehicles.

Clearance The space between a stationary and/or moving object.

Clearance Profile Describes the space that is exclusively reserved for provision of the road or street. It defines the minimum height of the soffit of any structure passing over the road and the closest approach of any lateral obstacle to the cross-section.

Climbing Lane A special case of an overtaking lane located on a rising grade, allowing faster vehicles to pass trucks and other vehicles.

Coefficient of Run-off The ratio of the amount of water that runs off a catchment area to the amount that falls on the catchment.

Collector A road characterised by a roughly even distribution of its access and mobility functions.

Commercial Vehicle A vehicle having at least one axle with dual wheels and/or having more than two axles.

Complete Street A street that accommodate all street users, allow for the safe movement of people and goods, giving priority to the most efficient modes of transport, respond to the neighbourhood character, create a vibrant public realm, contribute to a healthy and sustainable environment and create harmonious streetscapes in a cost-effective way.

Compound Curve A curve consisting of two of more arcs of different radii curving in the same direction and having a common tangent point or being joined by a transition curve.

Context Sensitive Design (CSD) Is defined as a project development process, which include geometric design, and attempts to address safety and efficiency while being responsive to, the street or road's natural and human environment. It addresses the need for a more systematic and all-encompassing approach in project development which recognizes the interdependency of all stages in the process. In short, the term CSD refers as much to an approach or process as it does to an actual outcome.

Criterion A yardstick according to which some or other quality of the road can be measured. Guideline values are specific numerical values of the criterion. For example, delay is a criterion of congestion.

Critical Length of Grade The maximum length of a specific upgrade on which a loaded truck can operate without an unreasonable reduction in speed. Very often, a speed reduction of 15 km/h or more is considered "unreasonable".

Cross-section The transverse elements of the longitudinal elements**.**

Crossfall The slope, measured at right angles to the alignment, of the surface of any part of a carriageway.

Crosswalk A demarcated area or lane designated for the use of pedestrians across a road or street.

Crown The highest point on the cross section of a carriageway with two-way cross fall.

Crown Run-off (Also referred to as tangent runout) The rotation of the outer lane of a two-lane road from zero cross fall to normal camber (NC).

Culvert A structure, usually for conveying water under a roadway or parallel to a street.

Curvilinear Alignment The alignment is a continuous curve with constant, gradual and smooth changes of direction.

Cut Section of street or road below natural ground level. Sometimes referred to in other documents as a cutting or excavation.

Cycle Lane A portion of the roadway which has been designated by road markings, striping and signing as being exclusively for the use of cyclists.

Cycleway A facility provided for cyclist movement and segregated from vehicular traffic by a kerb, or provided for on a separate right-of-way.

D

Deceleration Lane An auxiliary lane provided to allow vehicles to decrease speed.

Decision Sight Distance Sometimes referred to as anticipatory sight distance, allows for circumstances where complex decisions are required or unusual manoeuvres have to be carried out. As such, it is significantly longer than Stopping Sight Distance

Depressed Median A median lower in elevation than the carriageway and so designed to carry portion of the storm water falling on the road.

Design Domain The range of values of a design criterion that are applicable to a given design, e.g. lane widths of more than 3.3 metres.

Design Hour The hour in which the condition being designed for, typically the anticipated flow, is expected to occur. This is often the thirtieth highest hour of flow in the design year, or the peak hour traffic determined by modelling.

Design Hourly Volume (DHV) The hourly traffic volume selected for design purposes.

Design Life The period during which the quality of a structure (e.g. riding quality of a pavement) is expected to remain acceptable.

Design Period A period considered appropriate to the function of the road. It is used to determine the total traffic for which the pavement is designed and does not concern the geometric design.

Design Speed A speed fixed for the design and correlation of those geometric features of a street or road, that influence vehicle operation. Design speed should not be less than the operating speed.

Design Traffic The volume of traffic in the design year in equivalent vehicles, used for determining the required lane configurations of a street or road, normally taken as the design hourly volume.

Design Vehicle A hypothetical road vehicle whose mass, dimensions and operating characteristics are used to determine geometric requirements.

Design Year The last year of the design life of the road or any other facility, often taken as twenty years although, for costly structures such as major bridges, a longer period is usually adopted.

Directional Distribution (split) The percentages of the total flow moving in opposing directions, e.g. 50:50, 70:30, with the direction of interest being quoted first.

Discharge The volumetric rate of water flow.

Divided Road (divided carriageway) A road with a separate carriageway for each direction of travel created by placing some physical obstruction, such as a median or barrier, between the opposing traffic directions.

Drainage Natural or artificial means for the interception and removal of surface or subsurface water.

Driveway A road providing access from a public road to a street or road usually located on an abutting property.

E

Eighty-fifth Percentile Speed The speed below which 85 per cent of the vehicles travel on a given road

F

Footway Separate pedestrian facility.

Frangible Term is used to describe roadside furniture designed to collapse on impact. The severity of potential injuries to the occupants of an impacting vehicle is reduced, compared to those that could occur if the furniture was unyielding.

Freeway Highest level of arterial characterised by full control of access and high design speeds. (Class 1 road)

Frontage Road A road adjacent and parallel to but separated from the highway for service to abutting properties and for control of access. Sometimes also referred to as a service road.

G

Gap The elapsed time between the back of one vehicle passing a point on the road or highway and the nose of the following vehicle passing the same point. A lag is the unexpired portion of a gap, i.e. the elapsed time between the arrival of a vehicle on the minor leg of an intersection and the nose of the next vehicle on the major road crossing the path of the entering vehicle.

Grade The straight portion if the grade line between two successive vertical curves.

Grade Separation The separations of road, rail or other traffic so that crossing movements, which would otherwise conflict, are at different elevations.

Gradient The slope of the grade between two adjacent Vertical Points of Intersection (VPI), typically expressed in percentage form as the vertical rise or fall in metres/100 metres. In the direction of increasing stake value, upgrades are taken as positive and downgrades as negative.

Guardrail A rail erected to restrain vehicles that are out of control. It could also take the form of a set of vertical strung and anchored cables.

Guideline A design value establishing an approximate threshold, which should be met if considered practical. It is a recommended value whereas a standard is a prescriptive value allowing for no exceptions.

H

High Occupancy Vehicle Lane (HOV) A lane designated for the exclusive use of buses and other vehicles carrying more than two passengers. The actual number varies between authorities.

High-speed Typically where speeds of 80 km/h or faster are being considered.

Horizontal Curve A curve in the plan or horizontal alignment of a carriageway.

Horizontal Sight Distance The sight distance determined by lateral obstructions alongside the road and measured at the centre of the inside lane.

Human Factors Design This represents a paradigm shift from a Newtonian physics approach to design to a more complex process of the modelling of driver behaviour. In short, design is now also predicated on what the driver's capabilities are and wishes to do as opposed to only what the vehicle can do.

I

Intensity of Rainfall The rainfall in a unit of time.

Intersection A place at which two or more roads intersect at grade or with grade separation.

Intersection (at-grade) An intersection where carriageways cross at a common level.

Intersection Angle 1 An angle between two successive straights on the centreline of a carriageway.

Intersection Angle 2 The angles between the centrelines of two intersecting carriageways.

Intersection Leg Any one of the carriageways radiating from and forming part of an intersection.

Intersection Sight Distance The sight distance required within the quadrants if an intersection to safely allow turning and crossing movements.

K

Kerb Concrete, often precast, or hewn stone element adjacent to the carriageway and used for drainage control, delineation of the pavement edge or protection of the edge of surfacing. Usually applied only in urban areas.

Kerb Clearances A distance by which the kerb should be set back in order to maintain the maximum capacity of the traffic lane.

Kerb Ramp The treatment at intersections for gradually lowering the elevation of sidewalks to the elevation of the street surface.

K-Value The length required for a 1% change of grade on a parabolic vertical curve.

L

Lane (Traffic) A portion of the paved carriageway marked out by kerbs, painted lines or barriers, which carries a single fine of vehicles in one direction.

Lane Separator A separator provided between lanes carrying traffic in the same direction to discourage or prevent lane changing, or to separate a portion of a speed change lane from through lanes.

Lateral Friction The force which, when generated between the tyre and the road surface assists a vehicle to maintain a circular path.

Lay-by A place at the side of a road where a vehicle can stop for a short time without interrupting other traffic.

Level of Service (LOS) A qualitative concept, from LOS A to LOS F, which characterises acceptable degrees of congestion as perceived by drivers. Capacity is defined as being at LOS E.

Line of Sight The direct line of uninterrupted view between a driver and an object of specified height above the carriageway in the lane of travel.

Longitudinal Friction The friction between vehicle tyres and the road pavement measured in the longitudinal direction.

Low Speed Typically where speeds of 70 km/h or slower are being considered.

M

Median A strip of road, not normally intended for use by traffic, which separates carriageways for traffic in opposite directions.

Median Island A short length of median serving a localised purpose in an otherwise undivided road.

Median Lane The traffic lane nearest the median.

Median Opening An at-grade opening in the median to allow vehicles to cross from a roadway to the adjacent roadway on a divided road.

Minimum Turning Path The path of a designated point on a vehicle making its sharpest turn.

Minimum Turning Radius The radius of the minimum turning path of the outside of the outer front tyre of a vehicle.

Modelling A mathematical process to replicate traffic movements by computation

Modal Transfer Station The public facility at which passengers change from one mode of transport to another, e.g. rail to bus, passenger car to rail.

Movement Networks Movement networks comprise public right of ways, incorporating roads and streets as well as footways and cycleways which provide in a continuous and friendly manner for all human travelling needs. Movement networks recognises the multi-faceted nature of local residential streets, morphing into public transport orientated wider area movement facilities operating at higher speeds for the efficient transport of people and goods.

Mountable kerb A kerb designed so that it can be driven across.

Mountainous Terrain Longitudinal and transverse natural slopes are severe and changes in elevation abrupt. Many trucks operate at crawl speeds over substantial distances.

N

Non-motorized transport All road users that are not using motorized transport,including pedestrians, cyclists and animal drawn carts**.**

Non-Mountable Kerb A kerb so designed to discourage being driven across

Normal Cross Section The cross section of the carriageway where it is not affected by superelevation or widening.

Normal Crown (NC) The typical cross-section on a tangent section of a two-lane road undivided road.

O

O-D Survey Origin-Destination survey. This is a survey carried out to study the patterns and movements of road users so as to guide a road planner/designer on who and what to cater for.

Off-tracking The radial off-set between the path traced by the centre of the front axle and the centre of the effective rear axle on a turning vehicle.

One-way Road A road or street on which all vehicular traffic travels in the same direction.

Operating Speed Refer: Speed.

Outer Separator The portion of road separating a through carriageway from a service road or frontage road.

Overpass A grade separation where the subject road passes over an intersecting road, and/or pedestrian crossing and/or animal crossing.

Overtaking The manoeuvre in which a vehicle moves from a position behind to a position in front of another vehicle travelling in the same direction.

Overtaking Distance The distance required for one vehicle to overtake another vehicle.

Overtaking Lane An auxiliary lane provided to allow for slower vehicles to be overtaken. It is lined- marked so that all traffic is initially directed into the left-hand lane, with the inner lane being used to overtake.

Overtaking Zone A section of road on which at least 70 per cent of drivers will carry out overtaking manoeuvres subject to availability of adequate gaps in the opposing direction.

P

Passenger Car Equivalents(units) (PCE or PCU) A measure of the impedance offered by a vehicle to the passenger cars in the traffic stream. Usually quoted as the number of passenger cars required to offer a similar level of impedance to the other cars in the stream.

Passing The manoeuvre by which a vehicle moves from a position behind to in front of another vehicle, which is stationary or travelling at crawl speeds.

Passing Sight Distance The total length of visibility, measured from an eye height of 1,05 metres to an object height of 1,3 metres, necessary for a passenger car to overtake a slower moving vehicle. It is measured from the point at which the initial acceleration commences to the point where the overtaking vehicle is once again back in its own lane.

PC (Point of Curvature) Beginning of horizontal curve, often referred to as the BC.

PI (Point of Intersection) Point of intersection of two tangents.

PRC (Point of Reverse Curvature) Point where a curve in one direction is immediately followed by a curve in the opposite direction.

Property Line The boundary between a road reserve and the adjacent land.

PT (Point of Tangency) End of horizontal curve, often referred to as BC or EC.

PVC (Point of Vertical Curvature) The point at which a grade ends and the vertical curve begins, often also referred to as BVC.

PVI (Point of Vertical Intersection) The point where the extension of two grades intersect. The initials are sometimes referred to as VPI.

PVT (Point of Vertical Tangency) The point at which the vertical curve ends and the grade begins. Also referred to as EVC.

R

Rainfall Intensity The rate of rainfall (mm/h).

Rate of Rotation The rate of rotation required to achieve a suitable distance to uniformly rotate the cross fall from normal to full superelevation.

Reaction Distance The distance travelled during the reaction time.

Reaction Time The time between the driver's reception of stimulus and taking appropriate action.

Re-alignment An alteration to the control line of a road that may affect only its vertical alignment but, more usually, alters its horizontal alignment. A method of widening a road reservation.

Relative Gradient The slope of the edge of the carriageway relative to the grade line.

Renewable energy lighting sources: The provision of street lighting in villages or along LVRs through the use of solar energy or wind energy driven devices.

Residual Median The remnant area of the median adjacent to right turn lanes.

Reverse Camber (RC) A superelevated section of roadway sloped across the entire carriageway at a rate equal to the normal camber.

Reverse Curve A section of road alignment consisting of two arcs curving in opposite directions and having a common tangent point or being joined by a short transition curve.

Road Accident An incident in which a single vehicle, or two or more vehicles are involved in an accident that could include human injury or death.

Road Furniture A generic term used for various road related assets within the road reserve, including bus laybys and shelters, guardrails, pedestrian barriers, traffic signals, traffic signage, rail crossings and other small structures such as pedestrian bridges across rivers or streams.

Roadway A route trafficable by motor vehicles; in law, the public right-of-way between boundaries of adjoining property. The roadway includes the carriageway and the shoulders.

Road (Street) Furniture A general term covering all signs, streetlights and protective devices for the control, guidance and safety of traffic, and the convenience of road users.

Road Prism The lateral extent of the earthworks.

Road Reserve Also referred to as Right-of-way. The strip of land acquired by the road authority for provision of a road.

Road Safety Audit A structured and multidisciplinary process leading to a report on the crash potential and safety performance of a length of road or highway, which report may or may not include suggested remedial measures.

Road Safety Inspection Road Safety Inspections (RSIs) to be carried out on an ongoing basis once a road is fully operational to ensure the safe performance of the road.

Roadside A general term denoting the area beyond the shoulder breakpoints.

Roadside Safety Barrier A device erected parallel to the road to retain vehicles that are out of control.

Rolling Terrain The natural slopes consistently rise above and fall below the road grade with, occasionally, steep slopes presenting some restrictions on highway alignment. On general, rolling terrain generates steeper gradients, causing truck speeds to be lower than those of passenger cars.

Roundabout An intersection designed on the principle of gap acceptance and where all traffic travels in one direction around a central island.

Run-off That part of the water precipitation onto a catchment which flows as surface discharge from the catchment area past a particular point.

S

Sag Curve A concave vertical curve in the longitudinal profile of a road.

Safe Systems: An approach to build a road transport system that tolerates human error and minimises casualties following road accidents.

Section Operating Speed The 85th percentile speed of cars traversing a section of road alignment.

Semi-Mountable Kerb A kerb designed so that it can be driven across in emergency or on special occasions without damage to the vehicle.

Servitude A servitude is a registered right that a person has over the immovable property of another. It allows the holder of the servitude to do something with the other person's property, which may infringe upon the rights of the owner of that property.

Shared Path A paved area particularly designed (with appropriate dimensions, alignment and signing) for the movement of cyclists and pedestrians.

Shoulder Usable area immediately adjacent to the traffic lanes provided for emergency stopping, recovery of errant vehicles and lateral support of the road pavement structure.

Shoulder Breakpoint The hypothetical point at which the slope of the shoulder intersects the line of the fill slope. Sometimes referred to as the hinge point.

Side friction (f) The resistance to centrifugal force keeping a vehicle in a circular path. The designated maximum side friction (max) represents a threshold of driver discomfort and not the point of an impending skid.

Sidewalk The portion of the street cross-section reserved for the use of pedestrians.

Sideways Friction The ratio of the resistance to sideways motion of the tyre of a vehicle (on

Sight Distance

- **Approach Sight Distance (ASD)** The distance required for a driver to perceive marking or hazards on the road surface and to stop.
- **Car Stopping Sight Distance (SSD)** The distance required for a car driver to perceive an object on the road and to stop before striking it.
- **Entering Sight Distance (ESD)** ESD is the sight distance required for minor road drivers to enter a major road via a left or right turn, such that traffic on the major road is unimpeded.
- **Manoeuvre Sight Distance** The distance required for an alert car driver to perceive an object on the road and to take evasive action.
- **Minimum Gap Sight Distance (MGSD)** The minimum sight distance based on the gap necessary to perform a particular movement.
- **Overtaking Sight Distance** The sight distance required for a driver to initiate and safely complete an overtaking manoeuvre.
- **Railway Crossing Sight** Triangle The clear area required for a truck driver to perceive a train approaching an uncontrolled railway crossing and to stop the truck.
- **Safe Intersection Sight Distance (SISD)** The distance required for a driver in a major road to observe a vehicle entering from a side road, and to stop before colliding with it.
- **Sight Distance Through Underpass** The distance required for a truck driver to see beneath a bridge located across the main road, to perceive any hazard on the road ahead, and to stop.
- **Sight Triangle** The area in the quadrants of an intersection that must be kept clear to ensure adequate sight distance between the opposing legs of the intersection
- **Stopping Sight Distance** The sight distance required by an average driver, travelling at a given speed, to react and stop.
- **Truck Stopping Sight Distance** The distance required for a truck driver to perceive an object on the road and to stop before striking it.

Simple Curve A curve of constant radius without entering or exiting transitions.

Skid Resistance The frictional relationship between a pavement surface and vehicle tyres during braking or cornering manoeuvres. Normally measured on wet surfaces, it varies with the speed and the value of ‘slip’ adopted.

Slope

- The inclination of a surface with respect to the horizontal, expressed as rise or fall in a certain longitudinal distance.
- An inclined surface.

Speed

- **Operating Speed** The speed at which 85 percent of car drivers will travel slower and 15 percent will travel faster.
- **Operating Speed of Trucks** The 85th percentile speed of trucks measured at a time when traffic volumes are low.
- **Section Operating Speed** The value at which vehicle speeds on a series of curves tend to stabilise and is related to the range of radii on the curves.

Speed-change Lane A subdivision of auxiliary lanes, which cover those lanes used primarily for the acceleration or deceleration of vehicles. It is usual to refer to the lane by its actual purpose (such as deceleration lane).

Speed Profile The graphical representation of the 85th percentile speed achieved along the length of the highway segment by the design vehicle.

Standard A design value that may not be transgressed, e.g. an irreducible minimum or an absolute maximum. On the sense of geometric design, not to be construed as an indicator of quality, i.e. an ideal to be strived for.

Standard Axle A single axle with dual wheels loaded to a total mass of 8.16 tonnes.

Superelevation A slope on a curved pavement selected so as to enhance forces assisting a vehicle to maintain a circular path.

Superelevation Run-off (Also referred to as superelevation development) The process of rotating the outside lane from zero crossfall to reverse camber (RC), thereafter rotating both lanes to the full superelevation selected for the curve.

Sustainable Safety A safe road traffic system that aims to prevent deaths, injuries and damage to vehicles and property by systematically reducing the underlying risks of the entire traffic system.

Swept Path The area bounded by lines traced by the extremities of the bodywork of a vehicle while turning.

Swept Width The radial distance between the innermost and outermost turning paths of a vehicle.

T

Table Drain The side drain of a road adjacent to the sidewalk or shoulder, having its invert lower than the pavement base and being part of the formation.

Tangent The straight portion of a highway between two horizontal curves.

Tangent Run-off See crown runoff.

Tangent Run-out The length of roadway required to accomplish the change in crossfall from a normal crown section to a flat crossfall at the same rate as the superelevation runoff.

Terrain Topography of the land.

- **Level Terrain** Is that condition where road sight distance, as governed by both horizontal and vertical restrictions, are generally long or could be made to be so without construction difficulty or major expense**.**
- **Undulating Terrain** Is that condition where road sight distance is occasionally governed by both horizontal and vertical restrictions with some construction difficulty and major expense but with only minor speed reduction.
- **Rolling Terrain** Is that condition where the natural slopes consistently rise above and fall below the road grade and where occasional steep slopes offer some restriction to normal horizontal and vertical roadway alignment. The steeper grades cause trucks to reduce speed below those of passenger cars.
- **Mountainous Terrain** Is that condition where longitudinal and transverse changes in the elevation of the ground with respect to the road are abrupt and where benching and side hill excavation are frequently required to obtain acceptable horizontal and vertical alignment. Mountainous terrain causes some trucks to operate at crawl speeds.

Time of Concentration The shortest time necessary for all points on a catchment area to contribute simultaneously to run-on at a specified point.

Traffic A generic term covering all vehicles, people, and animals using a road.

Traffic calming Speed reducing measures implemented to force drivers to reduce speed at unsafe road conditions.

Traffic Composition The percentage of vehicles other than passenger cars in the traffic stream e.g. 10 per cent trucks, 5 per cent articulated vehicles (semi-trailers) etc.

Traffic Control Signal A device that, by means of changing coloured lights, regulates the movement of traffic.

Traffic Island A defined area, usually at an intersection, from which vehicular traffic is excluded. It is used to control vehicular movements and as a pedestrian refuge.

Traffic Lane A portion of the paved carriageway marked out by kerbs, painted lines or barriers, which carries a single line of vehicles in one constant direction.

Traffic segregation The vertical or horizontal separation of motorized and non-motorized road users.

Traffic Sign A sign to regulate traffic and warn or guide drivers.

Transition Length for increasing or decreasing the number of lanes.

Transition Curve A curve of varying radius used to model the path of a vehicle as it enters or leaves a curve of constant radius used for the purpose of easing the change in direction.

Transition Length for Alignment The distance within which the alignment is changed in approach from straight to a horizontal curve of constant radius.

Transition Length for Crossfall The distance required rotating the pavement crossfall from normal to that appropriate to the curve. Also called superelevation development length.

Transition Length for Widening The distance over which the pavement width is changed from normal to that appropriate to the curve.

Turning Lane An auxiliary lane reserved for turning traffic.

Turning Roadway Channelized turn lane at an at-grade intersection.

Turning Template A graphic representation of a design vehicle's turning path for various angles of turn. If the template includes the paths of the outer front and inner rear points of the vehicle, reference is to the swept path of the vehicle.

Typical Cross Section A cross section of a street showing typical dimensional details, utility services and street furniture locations.

U

Underpass A grade separation where the subject road passes under an intersecting road, pedestrian crossing or railway.

Urban Road or Street Characterised by adjacent property development, traffic volumes in keeping with the nature of the adjacent development, moving at relatively low speeds and pronounced peak or tidal flows. Usually within an urban area but may also be a link traversing an unbuilt up area between two adjacent urban areas, hence displaying urban operational characteristics.

V

Value Engineering A management technique in which intensive study of a project seeks to achieve the best functional balance between cost, reliability and performance.

Verge That portion of the road reserve outside the road prism

Vertical Alignment The longitudinal profile along the design line of a road.

Vertical Curve A curve (generally parabolic) in the longitudinal profile of a carriageway to provide for a change of grade at a specified vertical acceleration.

W

Walkway A facility provided for pedestrian movement and segregated from vehicular traffic by a kerb, or provided for on a separate right-of-way.

Warrant A guideline value indicating whether or not a facility should be provided. For example, a warrant for signalisation of an intersection would include the traffic volumes that should be exceeded before signalisation is considered as a traffic control option.