Rural Road Note 01: A Guide on the Application of Pavement Design Methods for Low Volume Rural Roads.

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FOREWORD

This Rural Road Note (RRN) is aimed at engineers, road managers and other practitioners involved with the planning and design of low volume rural roads in developing and emerging countries in tropical and sub-tropical climates. It is intended to provide guidance on key considerations for the use of various pavement design methods for low volume roads. The RRN provides guidance on pavement design methods used in the design of low volume rural roads, ranging from earth roads through gravel surfacing to the various unbound, natural stone, bituminous, cement-based and clay brick surfacing and pavement layers. The RRN compiles the lessons learnt from the design, construction, supervision and monitoring of a range of pavement and surfacing types investigated under the South-East Asia Community Access Programme (SEACAP) in Cambodia, Laos and Vietnam; the Africa Community Access Programme (AFCAP 1) and the Research for Community Access Partnership (ReCAP), together with the knowledge compiled from various related studies in Southern Africa and South-East Asia.

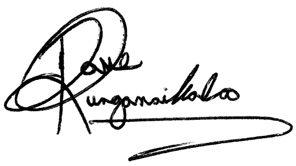
One of the fundamental principles behind the recent pavement research output for low volume roads has been the requirement for locally orientated solutions in Africa and Asia based on available local resources and the local road environment. Over the past two decades significant progress has been made in research on suitable materials for low volume roads. It has been found that materials considered unsuitable for use on high volume roads, or known as “marginal materials”, have been found fit-for-purpose and suitable for use in low volume roads and specifications for their use have been developed. The materials are now referred to as standard materials for low volume roads. This approach is crucial in the development of affordable and sustainable rural road infrastructure. Currently there are on-going ReCAP-supported research trials across Africa and Asia on long-term pavement performance in order to refine country specifications. Data from these trials are important for verification and local uptake of the various empirically developed pavement design methods.

It is often difficult to find funding for upgrading gravel roads to paved road standard. The decision on whether to seal a road or maintain it as a gravel or earth road needs to strike a balance between low initial cost and affordable long-term maintenance. The road authority must be able to demonstrate economic justification for the option selected, taking into consideration local availability of materials and construction technology, and the relative social benefits of different standards of access.

Undertaking research on pavement and surfacing options and developing appropriate solutions are not in themselves enough if any practical outcomes are to be achieved. There has to be a dissemination framework within which solutions can be mainstreamed. This includes guidance for road authorities on how to select a pavement design method that is most appropriate to their needs. A vital part of this framework is the production of practical and locally relevant Road Notes. Many of the pavement design methods in use for the design of low volume roads were empirically developed. It is therefore important that countries use results of monitoring of trial sections and performance of existing roads to continuously refine and indigenise the methods to their climate and materials.

This Rural Road Note is based primarily around recent research and experience on the performance of low volume roads in Sub-Saharan Africa and South East Asia and is therefore principally of use in tropical and sub-tropical regions in Africa, Asia and other parts of world with similar characteristics. It contains no reference to the special conditions governing the performance of rural roads in cold or other climates.

This comprehensive document has undergone substantial peer review and consultation in its development, and we aspire that it will be a key tool and reference point for rural road design, implementation and maintenance in LICs for many years to come.



Dave Runganaikaloo
ReCAP Programme Director

ACRONYMS, UNITS AND CURRENCIES

£	Pound Sterling (£ 1.00 ≈ provide conversion to local currencies)
ΔSN	Difference between existing and required structural number
AADT	Annual Average Daily Traffic
ADT	Average Daily Traffic
AASHO	American Association State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACV	Aggregate Crushing Value
ADT	Average Daily Traffic
AFCAP	Africa Community Access Programme
ALD	Average Least Dimension
ASTM	American Standard Test Methods
AfCAP	Africa Community Access Partnership
AFCAP	Africa Community Access Programme
AsCAP	Asia Community Access Partnership
BS	British Standards
BSI	British Standards Institution
CBA	Cost Benefit Analysis
CBR	California Bearing Ratio
COLTO	Committee of Land Transport Officials (South Africa)
CS	Crushed Stone
DBM	Dry-bound Macadam
DC	Design Class
DCP	Dynamic Cone Penetrometer
DCP-DN	Dynamic Cone Penetrometer – DCP Number
DF	Drainage Factor
DFID	UK Government's Department for International Development
DN	Number of DCP blows per mm of penetration of the DCP.
DN_X	The average penetration rate in mm per blow of the DCP to reach a depth of x mm
DN_{150}	Number of DCP blows required to achieve a penetration of 150 mm
DN_{450}	Number of DCP blows required to achieve a penetration of 450 mm
DSD	Double Surface Dressing
DSN_{800}	Number of blows of the DCP to reach a depth of 800 mm
EIRR	Economic Internal Rate of Return
EIA	Environment Impact Assessment
ENS	Engineered Natural Surface (Earth Road)

EOD	Environmentally-Optimised Design
ESA	Equivalent Standard Axle
ETB	Emulsion Treated Base
FACT	Fine Aggregate Crushing Test
GM	Grading Modulus (defined fully in Chapter 6)
GC	Grading Coefficient
GDP	Gross Domestic Product
gTKP	Global Transport Knowledge Partnership
GVW	Gross Vehicle Weight
HDM-4	Highway Design and Maintenance standards Model - 4
HMA	Hot Mix Asphalt
HPS	Hand Packed Stone
HVR	High Volume Road (in traffic terms)
IDA	International Development Agency
ILO	International Labour Organisation
IMT	Intermediate Means of Transport
Ip or PI	Plasticity Index
IRI	International Roughness Index
IRR	Internal Rate of Return
kg	Kilogramme
km/h	Kilometres per hour
l/m^2	Litres per square metre
LB	Labour-based
LL	Liquid Limit
LS	Linear Shrinkage
LVR	Low Volume Road
LVRR	Low Volume Rural Road
m	Metre
mph	Miles per hour
mm	millimetre
mm/yr	millimetres per year
m/s	metres per second
MESA	Million Equivalent Standard Axles
MC	Medium Curing
MCA	Multi Criteria Analysis
MPa	Mega Pascal
NGO	Non-Government Organisation

NPV	Net Present Value
NRC	Non Reinforced Concrete
OMC	Optimum Moisture Content
ORN	Overseas Road Note
Pen	Penetration
PCU	Passenger Car Unit
PI or Ip	Plasticity Index
PIARC	World Road Association
PL	Plastic Limit
PM	Plasticity Modulus
PMU	Project Management Unit
PP	Plasticity Product
PSD	Particle Size Distribution
QA	Quality Assurance
ReCAP	Research for Community Access Partnership
RED	Road Economic Decision
RC	Reinforced Concrete
RRCs	Road Research Centres
RRN	Rural Road Note
RSA	Republic of South Africa
SADC	Southern Africa Development Community
SANS	South African National Standards
SATCC	Southern Africa Traffic and Communications Commission
SBL	Sand Bedding Layer
SE	Super-elevation
SEACAP	South East Asia Community Access Programme
SN	Structural Number
SNC	Modified Structural Number (SN with subgrade contribution)
SWG	Stakeholder Working Group
TLC	Traffic Load Class
ToC	Table of Contents
ToR	Terms of Reference
TRL	Transport Research Laboratory
UNOPS	United Nations Office for Project Services
UCS	Unconfined Compressive Strength
UK	United Kingdom (of Great Britain and Northern Ireland)
UKAid	United Kingdom Aid (Department for International Development, UK)

USAID	United States Agency for International Development
URC	Unreinforced Concrete
USCS	Unified Soil Classification System
USD	United States Dollar
VOC	Vehicle Operating Costs
WBM	Water Bound Macadam
WLC	Whole Life Costs
WC	Wearing Course

GLOSSARY OF TERMS

Aggregate (for construction)
A broad category of coarse particulate material including sand, gravel, crushed stone, slag and recycled material that forms a component of composite materials such as concrete and pre-mix asphalt.

Asphalt
A mixture of inert mineral matter, such as aggregate, mineral filler (if required) and bituminous binder in predetermined proportions (sometimes referred to as Asphaltic Concrete or Asphalt Concrete). Usually pre-mixed in a plant before transport to site to be laid and compacted. Expensive and usually only used on main roads. Also used as an alternative term for Bitumen in some regions, and may be a petroleum processing product or naturally occurring in deposits.

Binder, Bituminous
Material used in road construction to bind together or to seal aggregate or soil particles, can be bituminous, cement or polymer based.

Bitumen
A non-crystalline solid or viscous mixture of complex hydrocarbons that possesses characteristic agglomerating properties, softens gradually when heated, is substantially soluble in trichlorethylene and is usually obtained from crude petroleum by refining processes. Referred to as Asphalt in some regions.

Bitumen, Cutback
A liquid bitumen product obtained by blending penetration grade bitumen with a volatile solvent to produce rapid curing (RC) or medium curing (MC) cutbacks, depending on the volatility of the solvent used. After evaporation of the solvent, the properties of the original penetration grade bitumen become operative.

Bitumen, Penetration Grade
That fraction of the crude petroleum remaining after the refining processes which is solid or near solid at normal air temperature and which has been blended or further processed to products of varying hardness or viscosity.

Bitumen Emulsion
A mixture of bitumen and water with the addition of an emulsifier or emulsifying agent to ensure stability. Conventional bitumen emulsion most commonly used in road works has the bitumen dispersed in the water. An invert bitumen emulsion has the water dispersed in the bitumen. In the former, the bitumen is the dispersed phase and the water is the continuous phase. In the latter, the water is the dispersed phase and the bitumen is the continuous phase. The bitumen is sometimes fluxed to lower its viscosity by the addition of a suitable solvent.

Bitumen Emulsion, Anionic
An emulsion where the emulsifier is an alkaline organic salt. The bitumen globules carry a negative electrostatic charge.

Bitumen Emulsion, Cationic
An emulsion where the emulsifier is an acidic organic salt. The bitumen globules carry a positive electrostatic charge.

Bitumen Emulsion Grades
Premix grade: An emulsion formulated to be more stable than spray grade emulsion and suitable for mixing with medium or coarse graded aggregate with the amount smaller than 0.075mm not exceeding 2%.
Quick setting grade: An emulsion specially formulated for use with fine slurry seal type aggregates, where quick setting of the mixture is desired.

Spray grade: An emulsion formulated for application by mechanical spray equipment in chip seal construction where no mixing with aggregate is required.
Stable mix grade: An emulsion formulated for mixing with very fine aggregates, sand and crusher dust. Mainly used for slow-setting slurry seals and tack coats.

Blinding
a) A layer of lean concrete, usually 50-100 mm thick, placed on soil to seal it and provide a clean and level working surface to build the foundations of a wall, or any other structure.
b) An application of fine material e.g. sand, to fill voids in the surface of a pavement or earthworks layer.

Borrow Pit
An area where material is excavated for use within another location.

Brick (fired clay)
A hard, durable block of material formed from burning (firing) clay at high temperature.

California Bearing Ratio (CBR)
The value given to an ad-hoc penetration test where the value 100% applies to a standard sample of good quality crushed material

Camber
The road surface is normally shaped to fall away from the centre line to either side. The camber is necessary to shed rainwater and reduce the risk of passing vehicles colliding. The slope of the camber is called the Crossfall. On sharp bends the road surface should fall directly from the outside of the bend to the inside (superelevation).

Cape Seal
A multiple bituminous surface treatment that consists of a single application of binder and stone followed by one or two applications of slurry.

Carriageway
The road pavement or bridge deck surface on which vehicles travel.

Cement (for construction)
A dry powder which on the addition of water (and sometimes other additives), hardens and sets independently to bind aggregates together to produce concrete. Cement can also be used to stabilise certain types of soil. Cement is also sometimes used as a fine filler in bituminous mixes.

Chippings
Clean, strong, durable pieces of stone made by crushing or napping rock. The chippings are usually screened to obtain material in a small size range.

Chip Seal, Single
An application of bituminous binder followed by a layer of stone or clean sand.

Chip Seal, Double
An application of bituminous binder and stone followed by a second application of binder and stone or sand. The second seal usually uses a smaller aggregate size to help key the layers together. A fog spray is sometimes applied on the second layer of aggregate.

Cobble Stone (Dressed stone)
Cubic pieces of stone larger than setts, usually shaped by hand and built into a road surface layer or surface protection.

Compaction
The process whereby soil particles are densified, by rolling or other means, to pack them more closely together, thus increasing the dry density of the soil.

Concrete
A construction material composed of cement (most commonly Portland cement, but occasionally using other available cementitious materials such as fly ash and slag cement), aggregate (generally a coarse aggregate such as gravel or crushed stone plus a fine aggregate such as sand), water, (and sometimes chemical admixtures to improve performance or for special applications).

Crossfall
See Camber

Crushed Stone
A form of construction aggregate, typically produced by mining a suitable rock deposit and breaking the removed rock down to the desired size using mechanical crushers, or manually using hammers.

Curing
The process of keeping freshly laid/placed concrete or stabilised soil moist to prevent excessive evaporation with attendant risk of loss of strength or cracking. Similarly, with cement or lime stabilised layers.

Design speed
The assessed maximum safe speed that can be maintained over a specified section of road when conditions are so favourable that the design features of the road govern the speed.

Distributor
A vehicle or towed apparatus comprising an insulated tank, usually with heating and circulating facilities, and a spray bar capable of applying a thin, uniform and predetermined layer of binder. The equipment may also be fitted with a hand lance for manual spraying.

Ditch (Drain)
A long narrow excavation designed or intended to collect and drain off surface water.

Drainage
Interception and removal of ground water and surface water by artificial or natural means.

Dressed Stone
See Cobble Stone

Dry-bound Macadam
A pavement layer constructed where the voids in a large single-sized stone skeleton are filled with a fine aggregate, vibrated in with suitable compaction equipment.

Earth Road
See ENS.

Embankment
Constructed earthworks below the pavement raising the road above the surrounding natural ground level.

ENS (Engineered Natural Surface)
An earth road built from the soil in place at the road location, and provided with a camber and drainage system.

ESA (Equivalent Standard Axle)

A design concept to enable the damaging effect of a range and number of different axle loads, to be considered in the structural design of a pavement. The equivalent standard axle imposes a load of 8,200 kg.

Expansive soil
Typically, a clayey soil that undergoes large volume changes in direct response to moisture changes.

Filler
Mineral matter composed of particles smaller than 0.075mm.

Formation
The shaped surface of the earthworks, or subgrade, before constructing the pavement layers.

French Drain (Underdrain)
A below-ground form of sub-surface drainage system, typically constructed in a trench and filled with graded rock or a permeable gravel, and a perforated pipe to remove the water. The drainage material (typically gravel) is often wrapped in a geotextile to prevent plugging of the gravel yet let groundwater drain into the system.

Geocells
Typical cellular confinement systems are made with ultrasonically-welded high-density polyethylene (HDPE) or Novel Polymeric Alloy strips that are expanded on-site to form a honeycomb-like structure which may be filled with sand, soil, rock or concrete.

Geosynthetic Material
A planar product manufactured from polymeric material used with soil, rock, and other geotechnical engineering related material as an integral part of a manufactured project, structure, or system. Functions are to improve strength, provide separation, filtration, structural stabilization, or confinement and reduce construction costs and time.

Gravel (Construction Material)
A naturally occurring, weathered or naturally transported rock within a specific coarse particle size range. Gravel is typically used as a pavement layer in its natural or modified condition, or as a road surface wearing course. Suitable gravel may also be used in a graded gravel seal in appropriate circumstances.

Hand Packed Stone
A layer of large, angular broken stones laid by hand with smaller stones or gravel rammed into the spaces between stones to form a road surface layer.

In Situ
Taken in position (i.e. test undertaken on the material within its natural state, rather than a sample taken for a laboratory test).

Intermediate Equipment
Simple or intermediate equipment, designed for low initial and operating costs, durability and ease of maintenance and repair in the conditions typical of a limited-resource environment, rather than for high theoretical efficiency.

Laterite
Residual deposits formed under tropical climatic conditions. Laterite consist of iron aluminium oxides.

Lime
Lime is a material derived from the burning of limestone or chalk. It is normally obtainable in its ‘hydrated’ form (slaked) as Calcium Hydroxide. It can be used for the drying, improvement and stabilisation of suitable

soils, as an anti-stripping agent in the production of bituminous mixes and as a binder in masonry or brick work mortars.

Liquid Limit
The moisture content at which a soil passes from the liquid
state to the plastic state, as determined by a specific test procedure

Local Resources
These can be human resources, local government, private, NGO, and community institutions, local entrepreneurs such as contractors, consultants, industrialists and artisans, local skills, locally made or fabricated intermediate equipment, local materials such as locally produced aggregates, bricks, timber and standard materials for LVRs, locally raised finance or provision of materials or services in kind.

Low Volume Roads
For pavement design purposes, these are roads designed to carry up to 1 million equivalent standard axles in a lane in one direction. For geometric design purposes, these are roads that carry about 300 to 400 vehicles per day.

Low Volume Sealed Roads
These are low volume roads that possess a thin bituminous seal, such as double surface dressing, as a surfacing material. They may also include any surfacing that provides water-proofing to the pavement, for example cement concrete.

Macadam
A mixture of broken or crushed stone of various sizes (usually less than 60 mm) laid to form a road surface layer. Bitumen macadam uses a bituminous binder to hold the material together. Tarmacadam uses tar for the same purposes. Bound macadams are usually expensive for use on LVR.

Otta Seal
A carpet of graded (natural gravel or crushed rock) aggregate spread over a freshly sprayed hot bituminous ‘soft’ (low viscosity) binder and rolled in with a heavy roller.

Pavé
See Sett.

Paved Road
A paved road is a road with a Stone, Bituminous, Brick or Concrete surfacing.

Pavement
The constructed layers of the road on which the vehicles travel.

Peri-urban
Immediately adjoining an urban area or village area.

Penetration Macadam
A pavement layer made from one or more applications of coarse, open-graded aggregate (crushed stone, slag, or gravel) followed by the spray application of bituminous binder. Usually comprising two or three applications of stone each of decreasing particle size, each grouted into the previous application before compaction of the completed layer.

Plasticity Index (PI):
Liquid Limit minus the Plastic Limit, an indication of the clay content of soils; the larger the PI, the larger the clay content.

Plastic Limit
The moisture content at which a soil passes from the plastic
state to the semi-solid state, as determined by a specific test procedure

Plasticity Modulus
The product of Plasticity Index (PI) and percentage fraction passing a 425 micron sieve.

Reinforced Concrete
A mixture of coarse and fine stone aggregate bound with cement and water and reinforced with steel rods or mesh for added strength.

Reseal
A surface treatment applied to an existing bituminous surface.

Roadbase and Sub-base
Pavement layers between surfacing and subgrade.

Road Maintenance
Suitable regular and occasional activities to keep pavement, shoulders, slopes, drainage facilities and all other structures and property within the road margins as near as possible to their as constructed or renewed condition. Maintenance includes minor repairs and improvements to eliminate the cause of defects and avoid excessive repetition of maintenance efforts.

Roadway
The portion within the road margins, including shoulders, for vehicular use.

Seal
A term frequently used instead of "reseal" or "surface treatment". Also used in the context of "double seal", and "sand seal" where sand is used instead of stone.

Sett (Pavé)
A small piece of hard stone trimmed by hand to a size of about a 100 mm cube used as a paving unit.

Shoulder
Paved or unpaved part of the roadway next to the outer edge of the pavement. The shoulder provides side support for the pavement and allows vehicles to stop or pass in an emergency.

Slope
A natural or artificially constructed surface at an angle to the horizontal.

Slurry
A mix of suitably graded fine aggregate, cement or hydrated lime, bitumen emulsion and water, used for filling the voids in the final layer of stone of a new surface treatment or as a maintenance treatment (also referred to as a slurry seal).

Sub-base
See Roadbase.

Subgrade
The natural material or earthworks formation underneath a constructed road pavement.

Surface Dressing
A sprayed or hand applied film of bitumen followed by the application of a layer of stone chippings, which is then lightly rolled.

Surface Treatment
A general term incorporating chip seals, slurry seals, micro surfacing, or fog sprays.

Surfacing
The part of road with which traffic makes direct contact.

Sustainability
A term relating to the capacity of a structure to endure.

Telford Type Construction
It involves placing by hand a layer of broken stone pieces of approximately 75/100 – 175/200mm in depth on a prepared and shaped level soil formation. The larger stones are placed at the centre of the road and the smaller at the edge to create the required crossfall (Minimum 1 in 45). Smaller stones are then packed between them, similar to the Hand Packed Stone technique. The initial layer is compacted and a second (100mm) and third (50mm) layer is placed on top with a combined thickness of 150mm of graded crushed stones. A blinding layer of gravel 40mm thick is then placed as the finished surface

Template
A thin board or timber pattern used to check the shape of an excavation.

Unpaved/Unsealed Road
A road with an earth or gravel surface.

Waterbound Macadam
A pavement layer where the voids in a large single-sized stone skeleton are filled with a fine sand, washed in by the application of water.

Wearing Course
The upper layer of a road pavement on which the traffic runs and that is expected to wear under the action of traffic. This applies to gravel and bituminous surfaces.

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1 INTRODUCTION

1.1 Background

The cost of providing low volume road infrastructure based on traditional high standards and design methods developed for high volume roads can be prohibitive. This is because such roads are aimed at providing mobility whereas the main requirement of local rural transport is access. As a result, road designs based on traditional standards are generally too costly for application in a large rural network. In response to this challenge the UK Department for International Development (DFID) and other development partners and agencies have supported research and knowledge transfer on various aspects of rural infrastructure specifically with the aim of reducing costs and increasing the sustainable effectiveness of the provision of road access for rural and peri-urban communities. In general, the research has concentrated on low volume rural roads, although some elements are also applicable to urban conditions. Much of this targeted research has been highly successful, resulting in innovative approaches that can provide highly beneficial and cost-effective solutions for low volume rural access through, for example, the use of appropriate road building materials and labour-intensive construction methods. Many of the pavement design methods in use for the design of low volume roads were empirically developed. It is therefore important that countries use results of monitoring of trial sections and performance of existing roads to continuously refine and indigenise the methods to their climate and materials.

1.2 Purpose

This Rural Road Note is for the structural design of Low Volume Rural Roads (LVRRs) and provides guidance on:

- the background of the various design methods that are currently used for LVRRs and outline any key strengths and limitations pertaining to their use;
- initial decisions required in selecting an appropriate pavement design and surfacing;
- salient issues pertaining to the selection of an appropriate materials for low volume rural roads;
- salient issues pertaining to the selection of an appropriate pavement design method;
- considerations pertaining to climate resilience and environmental impact;
- economic analysis required for selecting the most economical pavement design options or justifying the selected option.

The Rural Road Note is aimed at government officials who are responsible for formulating policy and planning on the structural design of LVRRs and engineers who are responsible for preparing road designs. It is also aimed at consultants who are responsible for the preparation and design of road projects and academics involved in training engineers in road and highway engineering.

1.3 Definition of Low Volume Rural Roads (LVRRs)

For pavement design purposes, a low-volume road is defined as one designed to carry a cumulative traffic loading of up to about 1 million equivalent standard axles (MESA) per lane over its design life. LVRRs are constructed using locally available natural materials which may be modified to meet standards given in the LVR design catalogues and may be unpaved or sealed with thin bituminous seals or discrete element surfacings.

Recent research from the ReCAP project 'Development of guidelines and specifications for low volume sealed roads through back analysis' shows that the natural materials from which low volume sealed roads are made can often carry in excess of 3 MESA especially when good drainage is provided, and timely surfacing maintenance is carried out.

A number of countries also define LVRRs in terms of the maximum number of commercial vehicles that use the road per day (cvd) at the time of design. The upper limit is typically 300 to 400 four-wheeled vehicles per day of which up to 25% are commercial vehicles. But structural design requires axle load information, hence cvd data alone is insufficient.

1.4 LVRR Hierarchy

In many countries, roads are classified according to their functions. Little emphasis is made on classification by traffic volume carried at any one time because that would result in some roads changing class year on year. The main functional classifications are:

- Trunk roads: these link cities to centres of international importance and to international boundaries;
- Primary roads: these connect centres of national and international importance such as principal towns and urban centres;
- Secondary roads: these connect centres of provincial importance or connect to primary roads;
- Tertiary/Collector roads: these connect locally important centres to each other or to a more important centre or to a higher-class road; and
- Feeder/Access roads: these connect minor centres such as a village market to other parts of the network, and villages to farms.

In many countries, the roads referred to as LVRRs are included in the tertiary and feeder road classes. In some cases, secondary roads are designed to LVRR standards because the traffic loading is low. Some very LVRRs fall into the category of "motorized trails" with only a width of 2-3 meters. These can be sealed to provide all-weather access.

In Africa it is common for LVRRs to be found in the three classes secondary, tertiary, and feeder; whereas in Asia, LVRRs tend to be of the tertiary and feeder classification only.

1.5 Scope

This Rural Road Note is primarily concerned with structural design of pavements for low volume rural roads. It provides guidance on the structural design of new roads and the upgrading of existing roads. Based on the principle that all layers of a pavement are part of the structure including the surfacing, this Rural Road Note also includes advice on selection of surface type.

The six common pavement design methods described in this RRN are:

A CBR -based methods

1) The AASHTO design method
2) The TRL Overseas Road Note 31 method
3) The TRL-SADC method
4) The Foundation Class method

B DCP-based methods

5) The TRL Overseas Road Note 18 DCP-CBR method
6) The DCP -DN method.

The methods are classified as either CBR-based or DCP-based by the initial method used to determine the subgrade strength.

The origins and range of application of these design methods are described in Chapter 4 but the details of their use are dealt with in Chapters 5. Other methods briefly described include: Pure Empirical methods such as Discreet Element pavements, Macadam and Telford pavements; and rigid pavements.

The standard of roads provided normally depends upon their functional classification. This ranges from feeder roads whose primary function is to provide access to trunk roads that connect main towns and cities and border crossings. Roads built primarily for access normally carry low daily volumes of traffic and relatively few heavy vehicles. However, it is an established principle that roads should be designed for the task that they have to perform, and this is defined by their function and the level and characteristics of the traffic that they need to carry. Thus, although function and traffic level are usually closely correlated, the roads classified as low volume are primarily roads whose main function is access but also includes some roads whose main function is mobility, depending on the definitions in each country.

The Rural Road Note is applicable to a wide range of countries therefore it would be impractical and inappropriate to provide recipe solutions for specific situations. Instead, emphasis has been placed on guiding the practitioner towards evaluating alternative options and considering their applications and limitations as a basis for decision making and application to region-specific situations and on a project-by-project basis. 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 pavement structural design. Additionally, relevant references are cited for more in-depth information.

The RRN caters for a range of rural road types, from basic earth tracks to bituminous sealed roads and includes roads surfaced with discrete elements such as cobble stones and block pavings, (semi-rigid surfacings) and also gives some guidance on rigid pavements. The RRN does not deal specifically with the design of urban LVRs but some aspects, for example, the structural design of road sections with discrete element surfacings are equally applicable in urban areas.

Design traffic estimation is not covered in this RRN since the topic is well-discussed in existing country manuals.

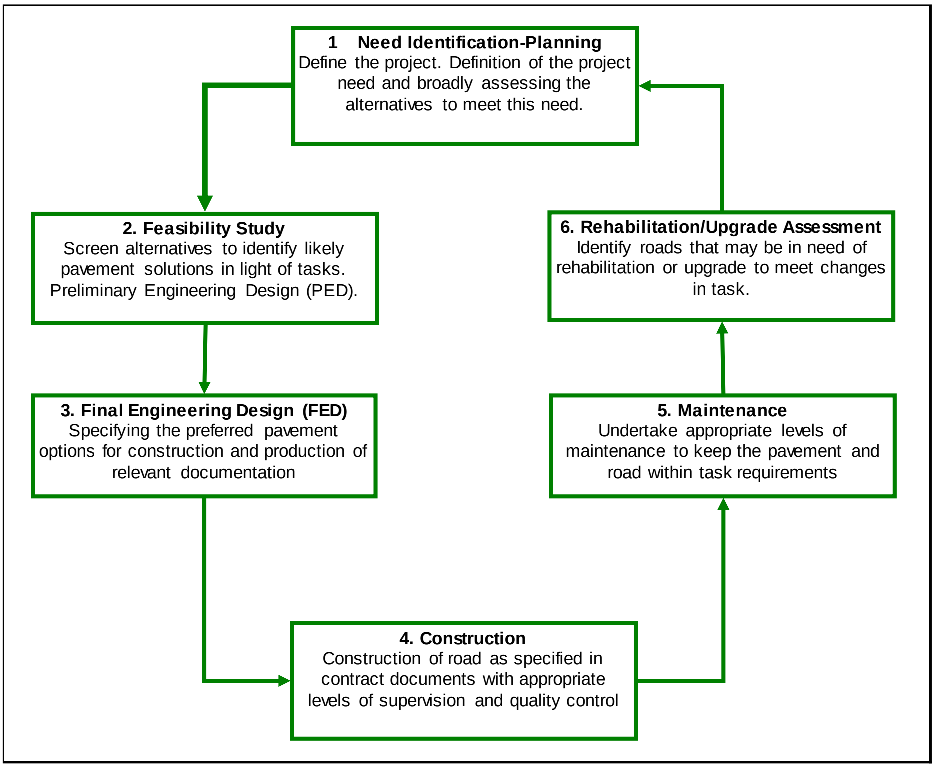
1.6 Principles of the Rural Road Note

The objective is to provide road practitioners, including non-engineering decision-makers, with a practical source document on selecting the most appropriate pavement design method for LVRRs for each situation, which is authoritative and contemporary. As such it lays out a key framework, defines key issues and clearly signposts procedures that may usefully be followed in order to respond to regional and user specific challenges. This Rural Road Note is set within the principle that the roads should be designed to be compatible with the local governing factors; as outlined below and illustrated in the project cycle (Figure 1-1). Roads are designed for a long life but they deteriorate with time and traffic and then require repairing or rebuilding. Thus, the life of a road can be described as a cycle as shown in Figure 1-1.. The design of low volume roads in general is based on the following principles:

1. **Task based:** Roads must suit their identified function and the nature of the traffic (the people as well as the vehicles) which will pass along them, by applying appropriate standards.
2. **Environmentally compatible**: The design must be adapted to the local road environment.
3. **Local resource based**: Road design guidance must be compatible with the capacities of the engineers and technicians who will design the roads and the contractors and labourers who will construct them, utilise construction materials that are readily available, and result in roads that are within the means of communities or local organisations to maintain them.

1.7 The Pavement Cycle

A series of important decisions must be made in the early stages of the development of a rural road project. These decisions have to be taken in a particular sequence. The decision process can be developed and extended into what may be termed the "Pavement Life Cycle", Figure 1-1. The Pavement Life Cycle is a framework for presenting guidance and recommendations. The Pavement Life Cycle acts as a route map to the relevant knowledge and the appropriate levels of information and procedures required for making knowledge-based decisions. The use of the Pavement Life Cycle also allows practitioners flexibility in terms of entry point. This Rural Road Note is concerned with pavement structural design and therefore with Box 3 and Box 6 in Figure 1-1.



Source: Cook et al 2013 – see reference in Chapter 6.

Figure 1-1: The pavement life cycle

1.8 The Road Environment

In order to achieve a sustainable road design, an early assessment of the road environment should be used to ascertain the suitability of unsealed surfacing options and the desirability of alternative options. Research has indicated that rainfall, gradient, material, construction practice and maintenance regime have significant impacts on the sustainability of unsealed gravel roads. The environmental assessment shown in Table 1-1 is necessary to obtain an understanding of the engineering environment of the road in question.

Table 1-1: Road engineering environment impact factors

Impact Factor	Description
Climate/rainfall	The climate influences the supply and movement of water and impacts upon the road in terms of direct erosion through run-off, dust production, and the influences of the groundwater regime. Climatic indices have a significant influence on the selection of pavement options and their design for "wet" or "dry" conditions. Unpaved surface performance is particularly influenced by quantity and intensity of rainfall, and the runoff arrangements.
Surface and sub-surface hydrology	The movement of water within and adjacent to the road structure has a major impact on the performance of the road pavement and surfacing. Seasonal moisture variations influence pavement behaviour adjacent to unsealed shoulders. Changes in near-surface moisture condition are the trigger for significant subgrade and earthwork volume changes in pavements underlain by "expansive" clay materials.
Terrain	The terrain reflects the geological and geomorphological history. Apart from its obvious influence on the geometry (grade and alignment) of the road and earthwork requirements, the characteristics of the terrain also reflect and influence the availability of materials and resources.
Materials properties	The nature, engineering properties and location of construction materials are key aspects of the road environment. For LVRs, where the use of local materials is a priority, the key issue should be: 'what design options are compatible with the available materials?' rather than seeking to find materials that meet standard specifications, as is the case with higher level roads. Specifications need to be appropriate to the local environment and locally available materials.
Subgrade	The subgrade is the foundation layer for the pavement. The assessment of its in-service condition is critical to the pavement design.
Traffic	Although research indicates that the relative influence of traffic on LVRs is often less than that of other road environment factors, consideration must be given to the influence of traffic and, in particular, the risk of vehicle overloading. Traffic is a major influence on the performance of both unpaved and paved or sealed road surfaces.
Construction Regime	The construction regime governs whether or not the road design is applied in an appropriate manner. Key elements include: • Appropriate contractual framework; • Experience of contractors or construction groups; • Skills and training of the labour force and supervisors; • Availability, use, condition and suitability of construction plant; • Selection, handling and placement of materials; and • Quality assurance-, sampling and testing, and compliance with specifications.
Maintenance Regime	All roads, however designed and constructed, require regular maintenance to ensure that their basic function is provided throughout their design life. Achieving this depends on the maintenance strategies adopted, the timeliness of the interventions, and the local capacity and available funding to identify and carry out the necessary works. When selecting a road design option, it is essential to assess the maintenance regime that will be in place during the design life of the road so that designs may be adjusted accordingly, or the maintenance regime enhanced. A ROAD SHOULD NOT BE BUILT WITHOUT A LONG-TERM MAINTENANCE STRATEGY!

Source: Cook et al. 2013 – see reference in Chapter 6.

Other factors that influence the choice of technology to be applied also play an important role in road provision. These are summarised in Table 1-2. An informed choice of approach should only be undertaken after a full understanding of both the engineering and enabling factors.

Table 1-2: Road enabling environment factors

Impact Factor	Description
Policies	National or local policies include priorities and guidelines for the decision-making processes. There are also legal requirements which must be observed.
Classification	Road classifications based on task or function provide road planners and designers with a practical guidance framework to initially select an appropriate road options and to estimate its cost. Having a clear rural road classification linked to relevant standards facilitates design and construction within acceptable performance criteria.
Standards (Geometry and Safety)	Geometric standards influence the comfort and safety of road users, the impact of water management on and across the road, and the requirement for earthworks which may affect the local environment. LVRs accommodate a wide range of users from pedestrians through to trucks. The traffic mix should be taken into account in the basic road geometry, for example the use of wider shoulders where the flow of pedestrian, bicycle and other non-motorised users is high. Through urban areas, pedestrian movement should be accommodated along roadways.
Technical specifications	Technical specifications define the minimum requirements for construction of the pavement and surfacing. . Specifications must be appropriate to the road environment and the prevailing road classification framework.
Funding arrangements	Funding has an over-arching influence on the scale and nature of road provision including the on-going requirements for road management and maintenance.
Contracting regime	The nature of the general contracting regime can influence a road project through the following issues: • Local legislation and contract documentation; • Governance and level of bureaucracy; • State-owned or private contractors; • National or international contractors; • Arrangements for facilitating local SMEs; and • Local resources and low-capital approaches.
The "Green" Environment	Road construction and on-going road use and maintenance have an impact on the natural environment, including flora, fauna, hydrology, slope stability, erosion control, health and safety. These impacts have to be assessed and adverse effects mitigated as far as possible through appropriate design and construction procedures. Key issues include water quality protection (and sometimes collection), wildlife movement and connectivity, possibly fish passage, minimizing movement of invasive species, and social needs.

Source: Cook et al. 2013 – see reference in Chapter 6.

1.9 Environmentally Optimised Design (EOD)

Most rural roads are designed using standard national designs along their entire length. However, this can be expensive and sometimes does not meet the needs of all road users. The EOD approach allows differing pavement designs to be selected in response to changing road environment factors along an alignment. Each road or road section is designed to meet its specific environment conditions, this allows the available budget resources to be concentrated on areas that may, for example;

- be at high engineering risk for example through frequent wetting;
- have significant safety issues;
- have high maintenance liabilities; or
- have a high socio-economic priority.

In selecting the optimised solutions, the factors in the Table 1-1 and Table 1-2 must be considered as a whole. EOD covers a spectrum of solutions for improving or creating low volume rural access for differing lengths of road, including:

1) Spot Improvements - dealing with individual critical areas on a road link that are not connected to each other. Spot Improvement is engineering-based and involves pavement options and other solutions compatible with the design life of the road; it should not be confused with maintenance or emergency repairs.

2) Providing an appropriate design for individual segments of road alignment.

3) Whole Length Improvement.

The EOD approach ensures that specifications and designs support the required function of each different road section. It provides a pragmatic way forward in the use of constrained resources to provide an acceptable level of service.

Under the spot improvement option, the road improvements can be prioritised according to certain criteria, typically the importance of safe and reliable access, or a dust-free road through a village. A section of unformed road that provides access and is not critical may, for example, receive only minimal maintenance while other sites are improved. A road may include sections of unformed track, gravelled sections and a sealed, concrete or block pavement section up a hill, for example, as a result of 'spot improvements'. Using this approach, it is possible to balance low cost surfacing solutions such as gravel and engineered natural surfaces for low risk areas with higher cost solutions for the high risk areas. This approach can be used to ensure that basic access is provided when investment resources are very limited.

The environmentally optimised approach to the design of rural roads is a key feature of the Rural Road Note. It 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.

In addition, it should be noted that for natural materials used in many low volume roads, environmental (mainly climatic) factors play a key role in their performance. The relative influence of environmental factors and traffic varies schematically as shown in Figure 1-2. The actual cross-over between the factors varies with climate, materials used, and pavement configuration. This means extra emphasis must be placed on ensuring that drainage is well designed and maintained if these materials are to perform well.

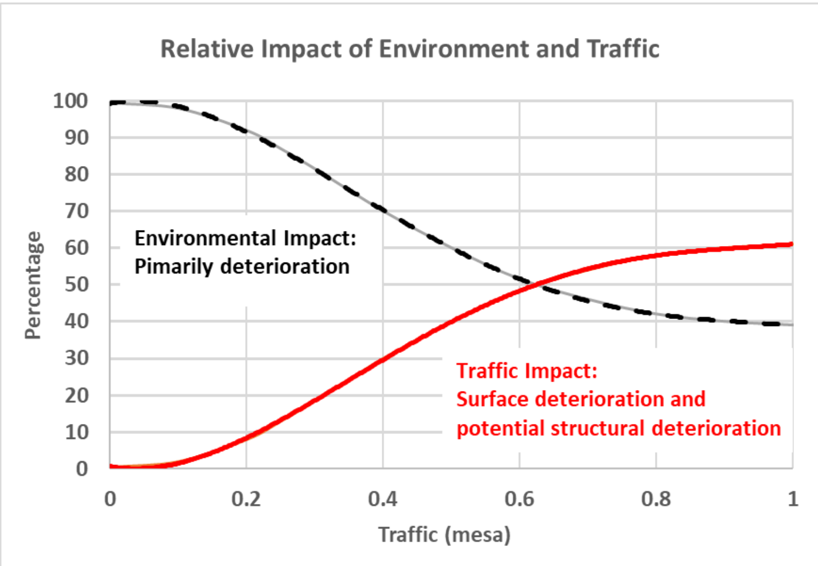


Figure 1-2: Schematic of the relative impacts of environmental factors and traffic

1.10 Structure of the Rural Road Note

The Rural Road Note is organised in such a way that key considerations that are common to all pavement design methods and have a strong bearing on the performance of any design, such as drainage, climate resilience, and selection of materials are discussed first. This is followed by discussion of the pavement design methods and selection of appropriate surfacing types. Finally, economic analysis and ancillary considerations are addressed.

Chapter 2 discusses key considerations for design of drainage and the challenges created by the changing climate.

Chapter 3 is concerned with the characteristics and specifications for the materials for constructing the road and how they differ between the pavement design methods.

Chapter 4 presents the background to the pavement design methods being considered.

The methods of pavement design for paved LVRs are described in detail in Chapters 5 and the design for both minor and major gravel-surfaced roads are discussed in Chapter 6.

There are numerous different surfacings suitable for low volume rural roads and the selection of these form the content of Chapter 7.

Chapter 8 discusses economic analysis for the comparison of different pavement design options.

Factors that affect the design and performance of pavements, but which are not dealt with in most manuals for LVR design, are discussed in a general context in Chapter 9. This includes ground improvements, slope stability for cuttings and embankments, borrow-pit management and compaction. In addition, an outline is provided on how to integrate results of some of the recent ReCAP research findings and ongoing research results into existing manuals.

Annex A includes tables that assist in selecting materials for use in low volume roads.

2 DRAINAGE DESIGN AND CLIMATE RESILIENCE

2.1 Background

Most LVRs are 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. 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 optimum moisture content (OMC). If the drainage is poor, the in-situ moisture will increase above OMC with corresponding loss of strength. In addition, the problems associated with climate change are particularly relevant for drainage design.

The design of climate resilient drainage infrastructure is based on five key principles.

1) Knowing how much water the drainage system needs to deal with. This is essentially based on knowledge of climate and hydrology.
2) Keeping water out of road structures by waterproofing and by diverting flowing water away from the road.
3) Minimising the loss of strength when submerged – the pavements and drainage structures should not lose significant strength when soaked.
4) Providing adequate stability against movement. Road infrastructure should be stable and robust enough to resist the forces of flowing water trying to move it.
5) Removing water quickly from the roadway and surface with frequent ditch relief cross-drains, lateral drains, leadoff ditches, and road surface slope (crown or cross-slope).

This chapter is concerned with the engineering issues that the road designer needs to consider to provide a satisfactory road drainage structure. For convenience, drainage design is generally sub divided into two components namely external and internal drainage.

2.2 Purpose and Scope

Moisture is the single most important factor affecting pavement performance and long-term maintenance. Thus, one of the significant challenges faced by the designer is to provide a pavement structure in which the detrimental effects of moisture are mitigated to acceptable limits in relation to the traffic loading, nature of the materials being used, construction and maintenance provisions and degree of acceptable risk. Hence although drainage is not part of the structural design it is vitally important for the success of the structural design and performance of the road and, as such, the two topics are treated in parallel.

The purpose of this chapter is to provide an overview of the key aspects of drainage and climate resilience that must be considered in order to provide a robust pavement that will be able to perform its function sustainably. The chapter does not cover detailed hydrology and drainage design (these can be found in respective country manuals). Details of the climate vulnerability assessments and engineering measures are contained in handbooks included in the reference list. Nevertheless, for low volume roads, drainage and climate resilience are often intertwined.

2.3 External Drainage

The role of external drainage is to keep water away from the road. It therefore includes drainage ditches, culverts, surfacings , shoulders, road cross section and to some extent alignment. Table 2-1 shows ways in which water can enter or drain from a road pavement. This section is followed by methods of preventing ingress of water into the road and then by methods of draining any water that manages to enter the pavement.

Table 2-1: Mechanisms of ingress and egress of water in pavements

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

2.3.1 *Ingress from the Side Drains and Road Margins*

To achieve adequate external drainage, the road must be raised above the level of existing ground such that the crown height of the road (i.e. the vertical distance from the bottom of the side drain to the finished road level at the centre line) is maintained at a minimum height, h_{min}. This height must be sufficient to prevent moisture ingress into the potentially vulnerable outer wheel track of the carriageway (Figure 2-1), as well as into the entire structural section.

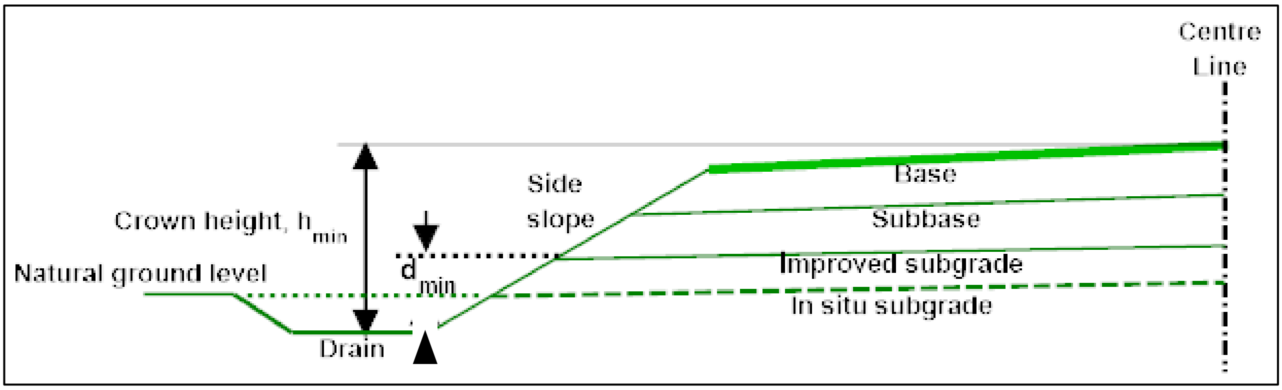


Figure 2-1: Crown Height for paved roads in relation to depth of drainage ditch

The recommended minimum crown height of 0.75 m applies to roads with unlined drains in relatively flat ground (longitudinal gradient less than 1%). The recommended values for sloping ground (gradient > 1%) or where lined drains are used, for example, in urban or peri-urban areas, are shown in Table 2-2.

The capacity of the drain should meet the requirements for the design storm return period.

Table 2-2: Recommended Crown Height, h_{min} (m), above drainage ditch invert

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

Side-drains are designed to accommodate run-off from the road surface and surrounding ground. At design flood flow a maximum depth of flow is allowable and this, combined with the characteristics of the channel, enables the capacity of the drain to be calculated. The depth of flow in turn creates the hydraulic gradient which causes the water to percolate from the side drains into the pavement. The amount of water that seeps through the side slopes of drains and its reach into the interior of the pavement depends on the permeability of the pavement materials and the level of compaction. The extent to which the ingress occurs determines its influence on the performance of the pavement.

Minimising the ingress of water into the pavement from the side of the road involves limiting the height of the water level in the drain to below the bottom of the sub-base for higher trafficked roads and below the base for lower trafficked roads. The former is recommended for secondary roads and the latter for tertiary and vicinal roads. This is based on the level of risk and size of project.

It is highly recommended that engineers should carry out drainage design for every element of the road regardless of climatic conditions. The hydrology and drainage design should be carried out in detail to determine runoff and design flood discharges to ensure that the conditions of maximum depth of water mentioned above are satisfied. This will ensure provision of adequate crown height.

In addition to observing the crown height requirements, it is equally important to ensure that that 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 2-1 and referred to as the 'invert depth') to minimise the likelihood of wetting up of this pavement layer from moisture infiltration from the drain.

2.3.2 Ingress Through the Surfacing

No road is permanently impermeable because of oxidative hardening and subsequent cracking that will inevitably develop in the surfacing and upper pavement layers at some point in its service life unless reseals are applied at the time cracking initiates. While it is widely believed that bituminous seals are impermeable in their early life, open graded cold mixes tend to be permeable, and most pavements become somewhat permeable over time (Cedergren 1977). Many poorly constructed thin bituminous surfacings, especially those involving slurry seals may also suffer this problem.

When designing sealed LVRs it is important to consider ingress of water through the surface in evaluating the risk of using materials in the pavement which are sensitive to moisture in high rainfall areas. Once the surfacing is breached and water enters, softening of the base may lead to accelerated deterioration through progressive cracking and punching leading to the formation of potholes. It is recommended to use low-permeability surfacing such as cape seal, Double Otta Seal, Double Surface Dressing, or dense cold mixes in wet areas; unless the base has a low plasticity product (PP). An appropriate maintenance regime supported by a good network management system is necessary for ensuring that surfacings are maintained, rejuvenated or resealed in a timely manner. A geotextile and bitumen inter-layer can provide good waterproofing. It is also recommended that maintenance regimes should be tailor-made for individual types of surfacings because their performance and resilience may differ significantly.

2.3.3 Ingress Through the Road Shoulders

Unsealed shoulders are often built using the same materials as the base course. Such materials tend to be non-plastic or slightly plastic. In cases where highly plastic material (plasticity modulus greater than 300) is used in the base courses, if the shoulders are not sealed water will easily percolate through the shoulders into the base course and cause weakening.

It is recommended that shoulders should be sealed. Exceptions can be made in situations where funds are limited, or the road is in a low-rainfall area, i.e. semi-arid and arid.

Sealed shoulders offer significant benefits in terms of improved pavement structural performance. The seal prevents precipitation water ingress into pavement layers just below the outer wheel path thus enabling a drier environment. The zone of moisture fluctuation is moved further away from the wheel path. If the design engineer can accurately estimate the moisture condition as a result of this, weaker materials (moisture-sensitive) than would be required when shoulders are unsealed, can be used safely in the upper pavement layers. The design strength (using any pavement design method) of these materials, under these circumstances, can be the strength assessed at that moisture content. In any case, if the designer does not use weaker materials, sealed shoulders will nevertheless result in better performance of the pavement.

Additionally, shoulders allow for non-motorised traffic to use the road safely away from fast-running motorised traffic.

Gourley and Greening (1999) showed that the benefits (through savings of using less-superior materials) of sealing shoulders outweigh the option of unsealed shoulders. They recommend a sealed shoulder width such that the outer wheel-track is more than 1.5 metres from the edge of the sealed area. The simplest way to achieve this is by extending the carriageway seal through to the shoulders, preferably up to the side drain edge.

An unsealed gap between the seal drain (Figure 2-2) should be avoided as much as possible. The drain lining should connect to the edge of the surfacing as shown in Figure 2-3.

Figure 2-2: Bad – Gap between surfacing and drain lining

Figure 2-3: Good – Drain lining connected to edge of surfacing

Other important aspects that must be addressed is the inclusion of scour checks, mitre drains, catch-water drains and chutes. All of these are discussed in the LVRR manuals that are currently in use (see Bibliography).

2.3.4 Preventing Ingress of Water into the Pavement

The following measures should be considered to prevent ingress of water into the pavement:

1) Construct a seal that prevents ingress of water. Such seals should be well-maintained to minimise or delay cracking, using for example fog sprays and reseals.
2) Use impermeable seals such as Otta Seal, Cape Seal, and a dense surfacing for example dense-graded cold mix or asphalt layer with a bitumen impregnated geotextile interlayer.
3) Use a dense base of well-graded materials with high levels of compaction.
4) Prevent ingress of moisture from the pavement edges by sealing the shoulders, raising the embankment, lining the side drain, etc.

2.3.5 Drainage of Water from Surrounding Areas

Water from surrounding areas is drained through side-drains, catchwater drains, mitres and sumps. The design procedure involves hydrological analysis and hydraulics design to determine storm flow characteristics.

Key elements of the design are discharge capacity and prevention of scour. Capacity of the drainage system is determined using the storm return period to calculate flow and using rainfall intensity. The maximum flood levels are sometimes predetermined based on criteria such as maximum level allowable (e.g. to a level no higher than the bottom of the sub-base layer). Scour potential should be calculated with the aim of minimising storm damage. Where scour potential is high, countermeasures should be put in place such as frequent cross-drainage or mitre drains, drain-lining, bioengineering or energy dissipation measures or changes of channel and flow characteristics to prevent excessive scour.

2.3.6 Drainage of Water from the Surface of the Road

The following measures need to be put into place:

1) A positive camber needs to be designed to allow rapid drainage of storm water off the carriageway. Paved roads should have a minimum 2% camber. Unsealed roads should have a 4-6% camber/crown/crossfall.
2) Storm drainage needs to be designed to prevent water ponding on the shoulders or kerbing and flowing onto the trafficked surface. Storm drainage should be designed to the required discharge capacity based on the characteristics of the run-off. Elements of storm drainage include kerbs and channels, chutes, cascades, edge and median drainage, drainage pits, pipe networks and grated inlets.

2.4 Internal and Subsurface Drainage

Internal drainage is concerned with the movement of water that enters the road pavement despite an efficient and functioning external drainage system. This is usually through infiltration, capillary action, and through surfacing.

2.4.1 Shoulder Crossfall

When permeable roadbase materials are used, particular attention must be given to the drainage of this layer. Ideally, the roadbase and sub-base should extend right across the shoulders to the drainage ditches. In addition, crossfall is needed to assist the shedding of water into the side drains. A slope of about 4-6% is recommended for the shoulders. However, it is usually not possible to provide a greater crossfall on the shoulders than on the carriageway and every effort should be made during construction to ensure that the crossfall of the road running surface is correct, preferably at the upper limit (3%) of the specified range. This ensures that even with minor rutting, surface water still continues to be shed from the carriageway. Unsealed roads carriageway crossfall should be 4-6%.

Lateral drainage can 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 Section 2.5.2) occurs. This may include by constructing the top of the fill or lower subgrade with a crossfall of 4%-5%). Although this may cause difficulties in setting out for construction, it is worth considering, particularly as full under-pavement drainage is rarely economically justified for LVRs. In addition, it provides some increase in pavement strength due to the slightly greater thickness of pavement material at the outer wheel path where the structure is more vulnerable to damage. However, this ideal drainage arrangement is more difficult to achieve in practice than constructing all layers with the same crossfall.

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

If it is too costly to extend the base and sub-base 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 sub-base level. These channels should be back-filled with material of base quality but which is more permeable than the base itself and should be given a fall of 1 in 10. Alternatively, a continuous layer of pervious material of 75 mm – 100 mm thickness can be laid under the shoulder such that the bottom of the drainage layer is at the level of the top of the sub-base, with regular discharge points into the side drain.

2.4.2 Avoiding Permeability Inversion

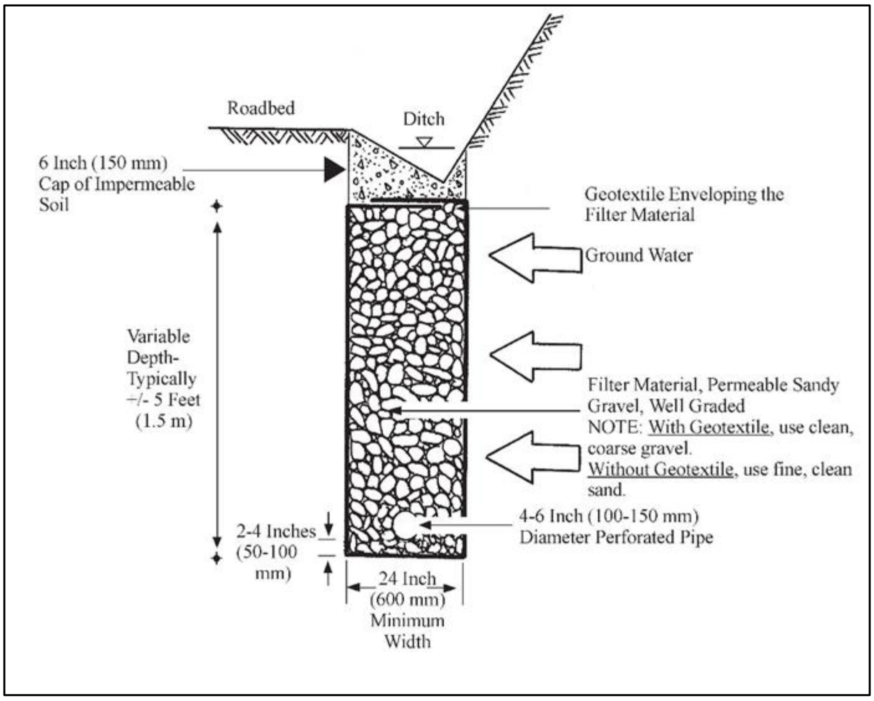
A permeability inversion occurs when the permeability of the pavement and subgrade layers decreases with depth. Under infiltration of rainwater, there is potential for moisture accumulation at the interface of the layers. The creation of such a perched water table often leads to rapid lateral wetting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

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

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, for LVRs there is seldom a wide choice of materials and if permeability inversion is unavoidable, then 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.

2.4.3 Draining Water from the Pavement through a Subsurface Drainage Systems

Subsurface drainage systems such as French Drains (Figure 2-4), underdrains, edge drains, wells, interceptor drains, filter layers or drainage blankets, should be placed in or under the pavement to drain away the water where the ground water level is high. See Figure 2-5 and Figure 2-6.



Source: Keller and Sherar, 2003

Figure 2-4: Illustration of a French Drain

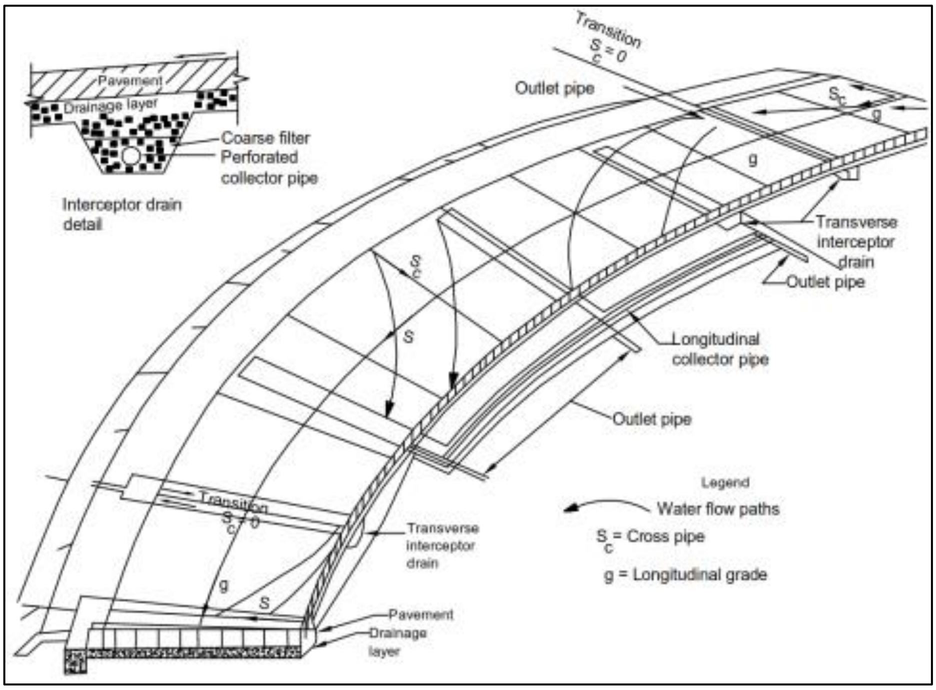


Figure 2-5: Subsurface drainage system *(for short sections of road only)*

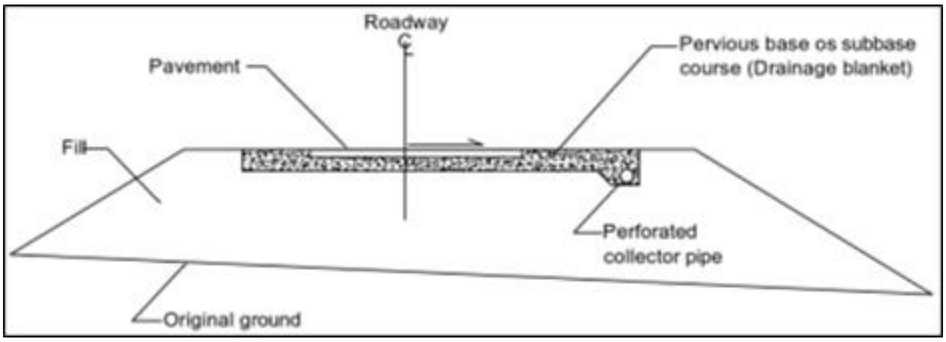


Figure 2-6: Illustration of a drainage blanket *(for short sections of road only)*

2.5 Climate Change and Pavement Design

Road engineers are accustomed to designing infrastructure for a specific type of climate, but the climate they are designing for is no longer fixed. The climate is changing, which means basing designs on historic patterns and averages is no longer sufficient. Information on the future climate needs to be taken into account. The exact type and magnitude of future changes will vary by geographical region and with the amount of future global greenhouse gas emissions. However, the types of changes that are expected, and are already starting to be seen, include:

- Precipitation more likely to fall as more intense rainfall events
- Changes in the timing of wet/dry seasons
- Less precipitation in some areas and more in others
- Sea level rise
- Greater periods of drought and cycles of drought and floods
- Higher average annual temperatures and higher maximum temperatures
- More frequent and intense heatwaves

These changes will impact road infrastructure in a number of ways, including:

- Flood damage / washout to road pavements and structures (e.g. bridges & culverts);
- Scour and washout of bridges, culverts and causeways;
- Inundation of coastal roads;
- Landslides triggered by heavy rainfall;
- Heat damage to paved asphalt and concrete roads;
- Erosion of earth roads and roadside drains;
- Soil moisture changes destabilising earthworks;
- Storm and wind damage to structures, and blowing debris/trees into the road
- Difficulties scheduling contracts and maintenance work
- More dust and need for soil stabilization or dust palliatives.

In addition to the more catastrophic effects of extreme weather events, there is likely to be increased rates of deterioration of roads and structures as a result of the gradual changes. For example, increased rates of bitumen oxidisation in asphalt surfacing as a result of higher average temperatures or greater erosion of unpaved roads due to more extreme drier/wetter periods.

Adapting pavement design to the projected climate changes and considering how design can be used to enhance future resilience will help to reduce the impacts of climate change on the road network; improving safety, maintaining access and reducing future repair and maintenance costs. Key or critical infrastructure design should consider lower risk and more conservative designs.

2.6 Taking a Risk-based Approach

Risk is the probability or likelihood of a hazard occurring multiplied by the magnitude of the impact if it does occur. Key to dealing with the impacts of climate change on roads in an appropriate and cost-effective manner is taking a risk-based approach. A better understanding of the relevant hazards, their associated risk and how these change over time enables road authorities to make informed decisions and prioritise resources. The elements that make up risk and associated terms such as vulnerability, resilience, susceptibility and criticality are often defined differently by different people, however the main points for consideration are the same. For the purposes of this RRN, risk has been divided into (1) exposure, (2) hazard and (3) vulnerability as this reflects the ReCAP handbook on climate change adaptation (Head et al, 2019).

2.6.1 Exposure

One determinant of risk is the amount and type of assets such as asphalt road, unpaved road, drainage, structures, earthworks etc. present in a location where they could be affected by a hazard. Information such as the location of different types of assets (e.g. from road asset management systems) and other spatial information such as topographical, hydrological and geological maps are useful for determining this aspect of risk. GIS can be an invaluable tool for understanding exposure or the extent of the assets at risk from different hazards.

2.6.2 Hazard

Hazard is an event with a potentially adverse impact. In this context, it refers to climate related hazards; either direct such as high temperature or indirect such as fluvial flooding or landslides which are triggered by weather conditions. The frequency of occurrence and magnitude of the climate event will depend on the location but will also change over time as a result of climate change. The design life of the asset being assessed will determine the period of interest.

Information on current hazards can be found from the location and frequency of historic events, trends in recent weather data, hydrological data, geological maps and local knowledge. Information on future weather hazards can be obtained from climate projections. Climate scientists model the global climate and through a technique called downscaling provide information about the type of climate a region may expect in the future. Climate projections vary in spatial and temporal resolution, but even lower resolution data provides insights into future trends and the magnitude of changes that can be expected. Road engineers should engage with their national meteorological institutes to obtain the information available for their country. Even if there is limited information produced nationally, this should provide advice and direct engineers to an appropriate international source of regional information such as that produced by the Intergovernmental Panel on Climate Change (IPCC, 2014), World Bank and UNDP. Climate scientists are continually working to improve climate information and so the most up-to-date information available should be used.

2.6.3 Vulnerability

Vulnerability is the propensity of the exposed assets to be adversely affected. It depends on the susceptibility or sensitivity of the infrastructure and degree of impact on the community. The characteristics and condition of a road will affect its susceptibility to different types of climate change impacts. In general, a paved road is less susceptible than an earth road, and a road in good condition less susceptible than a road in poor condition. Engineering knowledge is required to assess the susceptibly of a specific asset to a specific type of failure mechanism. The ReCAP Climate Change Adaptation Handbook provides information on how to carry out a visual assessment (Paige-Green et al, 2019b).

However, the most important and cost-effective countermeasure to meet the challenges of a changing climate and to reduce vulnerability, is to provide the required funding for and to carry out regular routine and periodic maintenance in line with international best practice. This will ensure that large parts of the infrastructure network will be functional and provide the intended service for most of the time. Agency maintenance crews, local contractors, and local community maintenance organizations can all help ensure timely needed maintenance.

The magnitude of the consequences if the road is closed or restrictions put in place as a result of a climate event relates to asset characteristics such as traffic flow, the criticality or importance of the route and availability of alternative routes. For example, if a road provides access to a hospital or major city, is the only route to a village or is part of an international trade corridor the risk level will be higher.

Another aspect to consider is the capacity of the road authority and community to deal with events if they occur. If the road authority does not have the resources (equipment, trained staff, budget) to repair and recover from events and if the community is highly vulnerable, e.g. with a large degree of poverty and heavily reliant on the road for their livelihood, this element of risk will be higher. When considering low volume rural roads, it is particularly important that the social and economic impacts are fully considered rather than solely using traffic flow to assess impact.

2.7 Risk Assessment Methods

Risk needs to be assessed in a consistent and structured way in order to make robust comparisons between locations, and over time. Risk assessment methodologies can be qualitative e.g. relying on expert judgement, quantitative e.g. mathematical modelling or semi-quantitative. The most commonly used approach when considering the climate change risk to roads is a semi-quantitative indicator method. Indicators are defined for the factors affecting the different elements of risk (exposure, hazard, vulnerability), and these are scored depending on various thresholds. The elements at risk are then combined to give an overall score. The risk assessment should be informed by data, so that the results are as robust as possible. Data requirements include information on:

1) asset location,
2) asset characteristics and condition,
3) hydrology,
4) geology
5) typography,

6) traffic flow,
7) land use,
8) local communities
9) climate.

Where there are data gaps, the judgement of relevant experts may be used to score the indicators. Other future changes such as traffic increases or new developments should also be taken into account.

A climate risk assessment can be carried out at different levels:

- Organisational – to identify the risk to the various activities that a road authority carries out.
- Network – to identify the network sections/assets with the highest risk in order to prioritise adaptation actions.
- Individual assets or schemes – to help identify modifications to assets or designs to minimise risk.

A risk assessment gives a consistent, objective assessment enabling comparison and providing the evidence for adaptation action. It can be used to plan and prioritise upgrades and maintenance to the highest risk assets, modify policy or standards if there is a network-wide vulnerability and help develop the business case for additional funding. The hazards to be assessed should include both historic hazards and potential future hazards, e.g. if a road is not currently affected by coastal flooding, but future sea level rise means it could be in the future, this hazard should be included in the assessment. Ideally risk assessment of infrastructure is done in an interdisciplinary setting including knowledgeable road design and maintenance personnel, resource specialists, relevant organizations, and interested citizens. The results of the risk assessment need to be set within the context of the road authority's appetite for risk, which will be different for each road authority, based on, for example, their own organisational policies, available budget and expectations of the road network users.

More information on assessing risk can be found in the Threats and Vulnerability Road Notes of the AfCAP climate change adaptation handbook (Le Roux et. al. 2019). This covers both new and existing roads.

2.8 Principles of Integrating Climate Change into Road Design

When considering climate resilience in pavement design, the aim is to identify how the future risk to the pavement from climate change impacts can be minimised as far as possible. This should consider all elements of risk; exposure, hazard and vulnerability. However, the main area where pavement design can influence the level of risk is vulnerability, particularly the susceptibility of the infrastructure to the hazards identified as relevant to its location and expected lifespan. Minimising the impact is mainly the role of disaster/incident management although designers should consider how to enable better traffic management and a quick recovery when extreme weather occurs. In fact, one approach is to design infrastructure for rapid repair rather than to withstand extreme conditions. This may be a more cost-effective solution especially for less critical infrastructure. It should also be understood that maintenance has a large role to play in reducing susceptibility, however this Rural Road Note is focused on design rather than asset management, so this is not discussed here.

Specific adaptation measures (both engineering and non-engineering actions) to reduce risk can be found in ReCAP engineering notes section of the Climate Adaptation Handbook or other publications such as the ROADAPT database (CEDR, 2015). The design of drainage facilities is a very important part of improving the resilience of road infrastructure to climate change.

2.8.1 Early Assessment of Climate Change

The earlier in the design process that climate change is considered, the greater the opportunity to influence the resilience of the design. It is recommended that a project/scheme level risk assessment is carried out in the early design stage. The results of the risk assessment should inform the design. Different hazards will be more pertinent depending on the level of risk, and the amount of budget the authority allocates to mitigating them should relate to the risk level. Evidence needs to be gathered to support the risk assessment, and specialists such as climate scientists, engineers, geologists, hydrologists etc. and stakeholders need to be engaged in this process. The longer the asset's expected lifespan, the further into the future the climate projection period considered should be. Risk assessments can be carried out for upgrades to existing roads or plans for future roads. It is normally technically more straightforward and less costly to include adaptation during construction rather than retrofitting in the future.

2.8.2 Design for Future Flexibility

One approach that can be employed is to design with flexibility in mind to make it easier to upgrade the infrastructure in the future. Potential options to increase resilience are identified, but not implemented. Instead the design is adjusted so that these options are left open for the future, for example leaving space for additional drainage to be added if required at a later date or purchasing additional right-of-way if an alignment needs to be shifted in the future. This approach is particularly useful where there are high levels of uncertainty surrounding future conditions, and the adaptation options are costly. Related to this is an approach called flexible adaptation pathways (climateXchange, 2012) whereby the timelines for different options are mapped out, and decision-points set that would enable the option to be implemented in time to maintain the agreed level of risk. Conditions are monitored and when certain thresholds are exceeded, or additional information is available, the options are reviewed. For example, options to protect a coastal road could include raising the level of the road or building sea defences, both costly options with long lead times. If sea level rise projections are uncertain, the decision to adapt could be postponed until a certain sea level or rate of increase is observed. This enables the authority to avoid committing to unnecessary adaptation but to implement the necessary changes before coastal flooding or erosion prevents access.

2.8.3 Improve Rehabilitation Designs

When infrastructure fails, its design should be reviewed in order to identify measures to increase resilience rather than replacing like-for-like. For instance, a failed culvert should likely be increased in size/capacity or add trash racks to prevent plugging. This is especially important where climate projections indicate that the hazard which caused the failure will increase in frequency and intensity in the future.

2.8.4 Include Adaptation Measures with Other Work

Incorporating adaptation actions into planned upgrades and routine maintenance rather than implementing them as individual activities can reduce cost. Climate change adaptation should be integrated into normal operations whenever practical.

2.8.5 Select More Robust Materials

Where possible, use materials which are more robust and better able to withstand the wetter, or drier, or hotter and more rapidly changing conditions expected in the future. For example, use of quality crushed rock for road base in vulnerable sections.

2.8.6 Review Standards

Climate change is an evolving subject and therefore available information will change quite quickly. Thus, design standards should be regularly reviewed to reflect current understanding of climate change and its potential impacts. Potential modifications include:

- the addition of extra capacity (e.g. 15 to 30%) when designing the required drainage, to account for increased volume of precipitation,
- increasing the pavement crossfall so that water drains effectively and does not pond leading to softening of unpaved roads or hydroplaning on paved roads, or allow overtopping of specific sections with the necessary erosion protection measures put in place instead of building an embankment that can be breached and create more severe erosion, but importantly can take time to be repaired thus prolonging inaccessibility.
- Modifying asphalt mix designs to account for hotter temperatures and prevent rutting and bleeding.

2.8.7 Incorporate into Procurement

The design may be developed by contractors, in which case requirements relating to climate resilient design need to be included in procurement, so that they are passed down the supply chain. Actions could include assessing a tenderer's knowledge and competency of climate change adaptation at tender evaluation stage, including requirements for a climate change risk assessment in the specification and putting in place assurance mechanisms to make sure climate change is taken into account in the design.

2.8.8 Additional measures for ensuring pavement durability

The following additional measures are recommended for ensuring pavement durability in relation to drainage and climate change.

1) If suitable material is available, the pavement layers should be made with free draining material. This helps to minimize the time that it takes to regain full strength after flooding.
2) Armouring enhances the performance of weak base layers and prevents loads from punching through into the base layer. Armouring involves hammering a layer of very coarse aggregate (40-60 mm) into a partially compacted base using heavy vibratory steel rollers operating at high amplitude. The aggregate is hammered to refusal.
3) Submergence of the road can often lead to delamination or physical separation (i.e. peeling off) of the surfacings at the interface of the first and second layers of seals and/or the interface between the surfacing and the base. Amalgamated surfacings are seals in which the surface of the base is slightly porous or not too tight such that part of the surfacing layer intrudes into the base to create an amalgamation. There is therefore no clear plane of the interface between the base and surfacing and hence delamination is less likely to occur.

2.8.9 Pavement Resilience against Loss of Strength when Submerged

Unbound pavement materials are weaker when wet and can therefore be at risk of failure if they are submerged. If possible where such a risk is high, materials with a low moisture sensitivity should be selected. This is measured by the change in strength (CBR or DN) from OMC to the soaked condition.

1) The plasticity product of the materials for sub-bases and bases should not be more than 90 to allow for quick internal drainage.
2) Pavement structural design should be based on the subgrade strength expected in the soaked condition.
3) Sub-bases and bases should also be designed to be sufficiently strong in the soaked state.
4) Preference should be given to coarse gravels – the strength from interlocking of large particles can be sustained when the road structure is submerged. The grading modulus (GM) should be ≥ 2.5.

2.8.10 Resilience of Pavements against Damage and Washaways

The following measures are recommended where sections are at risk of washaways:

1) The route should be one that is least likely to be adversely affected by climate. Thus, areas of high flood risk should be avoided whenever possible. Avoid low areas next to streams and rivers. Keep the roadway well above possible high water.
2) Where there is high risk of sheet flooding, high embankments should be avoided. In this case crown height should not exceed 300 mm BUT the embankment should be encapsulated in cladding such as mortared stone pitching. This will allow water to pass over the pavement without causing too much damage.
3) High embankments. Where inundation is likely without the possibility of high-speed flowing water currents, the embankment should be raised to 0.5m above high flood level. Where the high flood levels cannot be determined assumptions should be made of the possibility that the pavement may become submerged and materials design criteria for the retention of minimum strength should be applied.

2.8.11 *Resilience of Drainage Structures against Damage and Washaways*

To protect drainage structures, the following measures are recommended:

1) The width of drain inverts should not be less than 1000 mm. Ideally, the width should be 1500 mm.

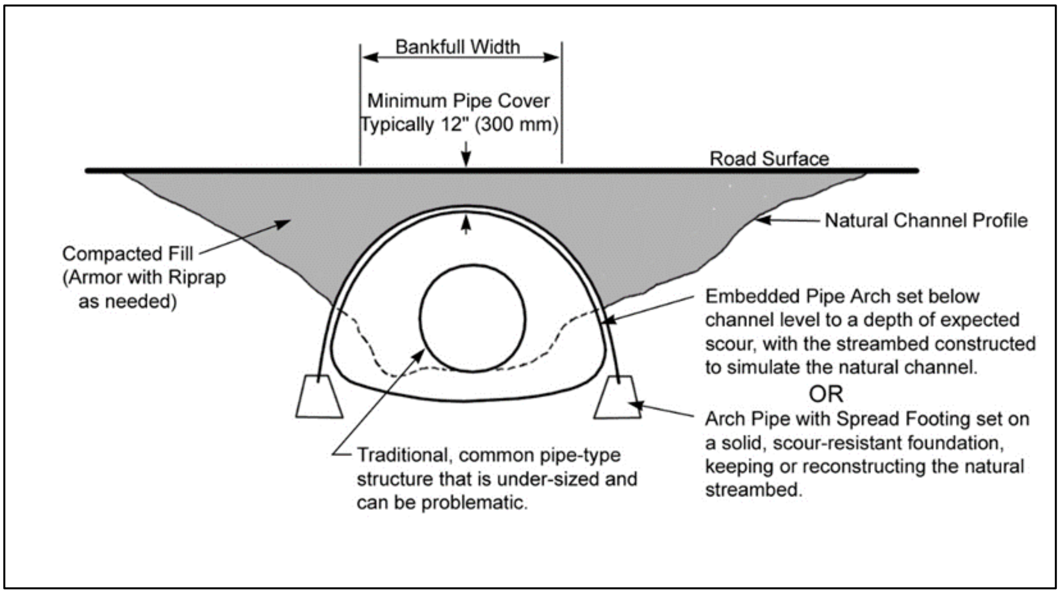
2) The spacing of mitre turn-out drains to divert water away from the road should not be more than 50 m.

3) Appropriate storm or flood return periods for high flood risk areas should be used. For areas prone to tropical storms and cyclones small structures should be designed for a 25-year storm/flood return period with possibility for overtopping. Medium sized structures (e.g. box culverts and small bridges) should be designed for a 50-year storm/flood return period and checked on a 100-year return period with possibility for overtopping. Large structures including large bridges should be designed for a 200-year return period and checked for a 500-year return flood with a possibility of overtopping.

4) Culvert diameter should roughly match the natural channel width to both maximize capacity and minimize risk of plugging and aggradation in front of the culvert. The design concept of "Stream Simulation" is useful to reduce risk of failure on important structures (Figure 2-7).

5) Relatively large culverts are preferred compared to multiple small culverts or openings to minimize risk of plugging. Small pipes and openings plug easily.

6) Monolithic/composite structures consists of a single unit rather than discrete structural units, Figure 2-8. Monolithic structures include box culverts, causeways, etc. Non-monolithic structures include bridges and culverts with discrete elements. A bridge normally consists of discrete structural elements, i.e. abutments, piers and decks, and these are not connected; they provide support discretely. But it is relatively easy for each element to be washed away or displaced, e.g. the bridge deck can be lifted, or a pier can get washed away. These failures do not require much force to occur, but monolithic structures are much stronger and more secure, and it requires much greater force to damage them. Bridges should be made climate resilient by use of anchor bolts connecting the superstructure to the substructure, which then make the bridge monolithic and resilient to separation.



Source: Keller and Ketcheson, 2014

Figure 2-7: Illustration of Stream Simulation

Figure 2-8: **Monolithic structure – box culvert**

Hydrodynamics is the science that allows the profile of the pavement cross-section to be designed to allow water to pass over the pavement exerting only minimal force. This is controlled by the shape of the profile of the cross-section and protection works. All structures should be designed to be hydrodynamically efficient. The key engineering consideration for the structures is that they will get submerged and may experience high lateral forces from strong water currents. Such structures should offer minimal resistance to the passage of water. Structural components should be oval with the narrower side against the current. This helps to minimize lateral forces.

2.9 Selecting the Most Appropriate Adaptation Measure

Engineering adaptation may include measures such as:

1) Pavement sealing: Pavement sealing particularly recommended for steep gradients (> 8-10% depending on other factors). Chapter 7 Selection of Surfacings.
2) Additional or enlarged culverts: Additional or enlarged or improved existing cross culverts considered essential to improve overall road drainage.
3) Side drainage: Additional side drains and associated turn-outs. Scour checks where necessary. Lined drains required with gradients >6%.
4) Raised embankments: Raising of earth embankments where the alignment is low and is being impacted by flooding and/or the weakening of the pavement by saturation. Chapter 11 Practical Considerations
5) Culvert or bridge abutment protection: Gabion, concrete, masonry or bio-engineering protection where erosion of abutments is identified as a significant risk.
6) 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. Chapter 4 Drainage.
7) 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. Chapter 9 Ancillary factors and Considerations.
8) Re-alignment: Re-alignments where an identified climate impact hazard and consequent engineering risk may be most cost-effectively overcome by avoidance.
9) River/stream crossing: Existing fords or low-level bridges might be replaced by climate resilient structures such as vented fords, or submergible multiple culverts. Chapter 4 Pavement Drainage.

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

Once potential design options have been identified, the most appropriate option needs to be selected. Many factors will affect this decision, but for any road authority cost will be an important part of this and the most commonly used appraisal method is cost-benefit analysis (CBA) Chapter 8 *Economic evaluation of pavement design outputs*. When evaluating adaptation options, it is important that the long-term or life-cycle costs, not just the initial capital cost, are considered. Adaptation is designed to reduce future damage so minimising repair and delay costs as a result of extreme weather events, and reduce maintenance costs from increased deterioration rates. Therefore, the longer-term costs and benefits need to be properly taken into account. This means an appropriate appraisal period and discount rate should be used. Also, as mentioned in Chapter 1, when evaluating low volume rural roads, it is important to consider the wider social and economic impacts that lack of access causes not just the direct repair and delay costs. Traditional CBA often does not include this. It can be challenging to calculate the wider economic impacts of road closures on local communities and businesses and some impacts e.g. environmental, are difficult to monetarise. Effort should be made to quantify the costs of environmental impacts. Other qualitative appraisal methods such as multi criteria analysis (MCA) may need to be used in addition to CBA.

2.10 Enhancing Resilience of Existing Pavements and Drainage Structures

2.10.1 Flood Risk Assessment of Existing Infrastructure

It is important to carry out a risk assessment of the existing infrastructures to determine vulnerability to climate change. Maps providing information of the high-risk areas should also be used for this exercise. Priority areas should be determined for detailed assessment. The vulnerability of the infrastructure is dependent on the following factors.

1) The condition of the infrastructure: Infrastructure that is in a state of disrepair e.g. dysfunctional drainage system, cracked or unstable structural elements, etc. Such structures if not repaired will most likely fail during flooding. Periodic bridge inspections or bridge condition rating systems are useful to identify potentially high risk structures.
2) Road environment: Rivers may have become silted causing their capacity to be reduced and increasing the possibility that they might burst their banks causing severe inundation of the surrounding flood plains or terrain.
3) Land use: Changes in land use may have led to deforestation or urbanization, leading to increased runoff or increase in the quantity of debris in flood waters which may lead to local flooding or the clogging of drainage structures.

2.10.2 Monitoring of Existing Infrastructure and Climatic Conditions

The condition of existing infrastructure and its maintenance, data on storm intensities and the severity of any flooding should be collected routinely. Information should include:

a) The mode of failure of the infrastructure. This will help to determine the most appropriate interventions required to improve the resilience of other road infrastructure.
b) Forecasting climate change and its impacts using models and observations.

2.11 Summary

Road designs need to be adapted to be suitable for the future climate.

The type and magnitude of change is likely to include higher temperatures (average and extreme), more intense rainfall events, and different precipitation patterns.

It is recommended that a risk-based approach is taken, with designers considering the future climate for the specific location they are designing for over the duration of the design life of the asset.

A structured methodology should be applied to assess risk and information on future and past climate, asset characteristics and condition, geology, hydrology, traffic flow and socio-economics used to inform the assessment. The risk assessment should evaluate:

- the exposure of the asset to hazards,
- the type of hazards and
- how climate change is expected to affect these,
- the susceptibility of the infrastructure and

- the impact if the infrastructure fails.

The road network should be viewed as a system, rather than individual assets, when considering the consequences of failure. Where there is a high risk from a particular hazard, steps should be taken to:

- reduce the susceptibility of the infrastructure,
- better manage the impacts of failure if it does occur, and
- expediate recovery.

The earlier in the design process climate change is considered, the more effective and less costly adaptation actions will be. When selecting a design option, the longer-term costs and benefits need to be considered. If using Cost Benefit Analysis (CBA), the risk from future events and potential of the action to mitigate this risk needs to be incorporated into the analysis. The appraisal process also needs to consider the wider economic and social costs of failure, which are likely to be larger than the direct repair and delay costs. If these cannot be monetarised other qualitative assessment techniques can be used. Design is only one part of increasing resilience, improving the resilience of roads and adapting to climate change needs to be an overarching aim and integrated in policy, planning, asset management, processes, procurement, monitoring and assurance. Some of these will influence design. For example, if a climate change risk assessment is carried out as part of the environmental impact assessment (i.e. planning procedures) the result can be used to inform design, and if the route alignment is adjusted to reduce exposure to a hazard at the planning stage this would affect the design.

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3 MATERIALS SELECTION

3.1 Purpose and Scope

All design methods for LVRRs attempt to make use of locally available materials so that haulage and processing costs are low. Thus, the use of materials that do not conform to standards for high volume roads but are satisfactory for use in low volume roads are encouraged. In the past two decades, extensive research has been carried on the performance of these materials and many have been found suitable for use in low volume roads. Thus, they are now referred to as 'standard materials for low volume roads'. An understanding of the effects of the variations of the properties of the material will assist in making a decision as to whether to use the material or not, remembering that the proposed LVRR is likely to be subjected to occasional or seasonal excessively loaded trucks, for example during agricultural harvest seasons.

The purpose of this chapter is to give guidance on selecting suitable materials and on how they may be used. The research that has led to these conclusions is described in detail in the references to this chapter. The chapter also describes the specifications for road building materials that have been incorporated in the six main pavement design methods that form Chapter 5.

3.2 Approach to Using Materials

For LVRs, it is usually not critical if the standard specifications applied for high volume roads for grading, Atterberg limits, and aggregate hardness are not fully complied with. For use of LVR standard materials, key concerns are quantity and quality. Is there enough material for the project and does that material have acceptable quality for the job. Use of these materials sometimes implies lower initial construction costs but a shorter design life or increased maintenance costs.

Materials should be selected with the following considerations:

- Sustainability; that is, if presented with a choice of materials, use the material that just meets the specifications first before using material that more than meets the specification;
- Knowledge gained by the designer on the performance of particular materials in specific areas and local experience. Local knowledge could mean overriding imported specifications;
- With consideration for internal pavement drainage;
- If available materials require modification, consider mechanical modification with another natural material before chemical modification. It is sometimes cheaper to chemically modify a material close to the intended site than to haul better quality material from a longer distance. Nevertheless, the options should be evaluated using the economic analysis mechanisms presented in Chapter 8;
- Remember that light chemical modification improves both the strength and plasticity characteristics;
- Gravel-sized and cobble-sized materials offer an option to use Macadam or Telford type constructions. The bulk materials cannot be tested by conventional test methods, therefore if the option is selected, the contractors' skills should be considered in the decision-making process.

There are many ways marginal materials can be improved to make them suitable, to improve performance, or meet specifications. This is a good place to list those ways. This includes screening, blending, crushing, treating with additives, cement/lime modification, use of Central Tyre Inflation in some haul roads, including geogrids or cellular confinement, etc. A discussion of many of these methods is found in "Stabilization and Rehabilitation Measures for Low-Volume Forest Roads" (Keller et.al. 2011). Details of treating each specific material category are included in Annex 1. The material testing requirements, procedures and specifications for the different design methods are described in detail below.

3.3 Test Methods

Different countries use different test methods, for example, American Association of State Highway and Transportation Officials (AASHTO), American Society for Testing and Materials (ASTM), British Standards Institution (BSI), Technical Methods for Highways (TMH), and other country-specific test methods. These methods, even for the same test, will have minor variations in procedure and sometimes yield different results for the same material. It is important that the test methods adopted in each country should not be arbitrarily mixed with other test methods. The procedures for the adopted test methods should be followed meticulously.

Additionally, materials characteristics sometimes change during extraction, stockpiling, batching, and compaction. For example, the particle size distribution, Atterberg limits, and the strength of scoria and highly weathered materials. Therefore, verification tests should be conducted at different stages and necessary adjustments made to maintain the quality used for design.

3.4 Guidelines for Material Selection and Use

The main groups or types of geological material that are appropriate for use in bitumen-surfaced low volume road construction have been identified in Table 3-1.

Table 3-1: Material groups

Group		Sub-Group	Important Examples
I	**Strong Rocks**		• Foliated Metamorphic Rocks • Crystalline Basic Igneous & Metamorphic
II	**Weak Rocks**	Inherently Weak or Poorly Consolidated Rocks	• Coralline Deposits • Marls and Weak Limestones • Weak Conglomerates • Weak Sandstones • Weak Volcanic Tuffs, Pyroclastics • Shale, Siltstone and Mudstone Deposits
		Partially Weathered and/or Highly Fractured Rocks	• Fractured /Weathered (Rippable) Limestones • Weak Volcanic Agglomerates and Breccias • Other partially Weathered Rocks
III	**Natural Granular Deposits**	Transported Soils and Gravels	• Alluvial Sand & Gravel Deposits (including riverbed, river terrace, shore and fan deposits) • Alluvial and Aeolian Sand Deposits • Clayey Sand Deposits • Colluvial Deposits • Volcanic Scoria (Cinder) Gravels • Volcanic Pumice/Ash Deposits
		Residual Soils and Gravels	• Quartz Gravels • Weathered Granite / Gneiss • Other Residual Gravelly Soils or Clayey Sand Deposits
IV	**Duricrust (Pedogenic) Gravels**		• Laterite Deposits • Calcrete Deposits (including Calc Tufa and Caliche)
V	**Manufactured Materials**		• Bricks • Recycled asphalt • Demolition Waste (Concrete and Brick) • Industrial By-products & Waste Material • Products (i.e. blast furnace slag, mine tailings,

The material groups have been defined on the basis of both their geological and engineering characteristics. These two attributes are interrelated so that classification by this method helps engineers to make use of geological maps and geological references during studies to identify and assess potential road building material sources.

This Chapter reviews materials within groups that are defined according to the main manifested property that is identified during laboratory investigations. The “main manifested property groups” considered are:

- High Plasticity Materials
- Poorly Graded Materials
- Poorly Shaped Materials
- Low Particle Strength Materials
- Low Durability Materials

Tables A-1.to A-5 in Annex 1 describe each of these “main manifested property groups” and present a summary of geological material types associated with the each of the properties. Each summary table provides guidance on:

- Problems associated with the defective property,
- Test methods and analysis techniques that may be employed to quantify and limit the problem,
- Particular characteristics (identifiable during laboratory investigations) that strongly influence the material behaviour,
- Factors that may be used during pavement design to accommodate the main manifested property
- Options for improving material quality and performance.

Discussion on the material main manifested property groupings is sometimes complicated because appropriate materials for LVRs are frequently associated with more than one challenging property. Nevertheless, the performance indicators and existing guideline specifications for LVRs show that their performance in service is satisfactory.

3.5 High Plasticity Materials

High plasticity is primarily associated with the presence of clay in the fine fraction of the aggregate. The adverse effects associated with the presence of clay minerals in a pavement material relate primarily to the property of clay minerals to attract moisture, which may soften the fine fraction and cause swelling. Some clay minerals are more sensitive than others in terms of their ability to attract water and to swell.

The presence of clayey fines and exposure to water has long been associated with the poor performance of pavement aggregates. However, the presence of fine materials to fill the voids within a well-graded material is also recognised as beneficial since this enables strong dense pavements to be constructed. However, for improved density and performance, the percentage of clayey fines should be limited to 15 to 20%.

Although clay particles may attract available moisture, densely packed clay fines exhibit low permeability that inhibits movement or passage of water. These two characteristics undoubtedly contribute significantly to the difficulty in predicting the engineering behaviour of well-compacted aggregates containing cohesive fines. Essentially the presence of plastic fines will be detrimental to the compacted pavement if it can be weakened through moisture ingress. Some types of clay-rich materials can be compacted to form very dense materials that have great resistance to the ingress of moisture. Examples of materials in this category include some laterite deposits, calcrete gravels and coral gravels. These types of material have low permeability as demonstrated by their characteristic of providing high strength when tested in a soaked condition.

3.5.1 Assessment of Materials of High Plasticity

The basic method of assessing the characteristics of clayey fines are the determination of Atterberg limits and linear shrinkage. These tests provide indices for the portion of material in the road aggregate that is finer than 0.425 mm in size. Standard road building materials have historically been specified on the basis of a maximum Plasticity Index (PI, sometimes denoted as Ip) that is applied in conjunction with a specified grading envelope.

In the case of materials that have Plasticity Index values that exceed recommended maximum values it is also recognised that the relationship between the proportion of fine material and its plasticity characteristics is highly relevant and should be investigated. Increasing plasticity characteristics (i.e. defined by the value of Ip or Linear shrinkage) may be tolerated when accompanied by a decreasing proportion of the fine material. Several parameters have been defined to evaluate the relationship between plasticity and fines content, these are:

- Plasticity Modulus= Plasticity Index x % passing 0.425 mm sieve
- Plasticity Product= Plasticity Index x % passing 0.075 mm sieve
- Shrinkage Product= Linear Shrinkage x % passing 0.425 mm sieve

These parameters are useful performance indicators when considered in association with the results of other test procedures such as those listed below:

- Soaked and unsoaked CBR value testing including measurement of swell,
- Sensitivity to compaction moisture content as determined by performing combined compaction/CBR testing,
- Hydrometer grading analysis to determine the proportion of clay size particles (< 0.002 mm),
- Analysis of clay mineralogy,
- Clay activity.

The engineering properties of some materials with high plasticity characteristics may be influenced by changes that occur after compaction that enhance the strength of the pavement through self-cementation. Self-cementation of lateritic materials is discussed in detail in CIRIA (1988). This property may be investigated by carrying out CBR testing or triaxial testing on similar materials over a prolonged curing period.

Conversely, there are materials that despite showing an initially high strength in testing can over time degrade in service due to attrition and generation of deleterious fines, for example basalts that contain montmorillonite.

3.5.2 Using High Plasticity Materials

Table A-1 in Annex 1 summarises key aspects of high plasticity materials relevant to their use as sub-base or roadbase aggregate. Engineering risk connected with the use of unmodified high plasticity materials will be largely associated with the degree of plasticity, the amount of fines and the extent to which water will have access to the sub-base or roadbase layers. These factors may be accommodated within an appropriate design. If this is not possible, then the options for improvement of the engineering properties of the material include the neutralisation of the plasticity effects by mechanical or chemical stabilisation/modification, or the removal of at least part of the fines content by materials processing. This latter option may add significantly to the cost.

Recent studies have shown that the boundaries of use of these materials is actually quite wide provided good drainage is guaranteed.

3.6 Poorly-graded Materials

Poor grading has been associated with a number of performance defects as illustrated in Figure 3-1 and summarised below:

(A) **Too coarse** (e.g. some partially weathered rocks, quartzitic materials):

- Reduced stability and increased risk of shear
- Low in situ density and increased risk of settlement or permanent deformation under traffic
- Difficult to compact
- Increased transmissivity
- Pumping
- Potholing associated with the use of very coarse particles
- High binder demand

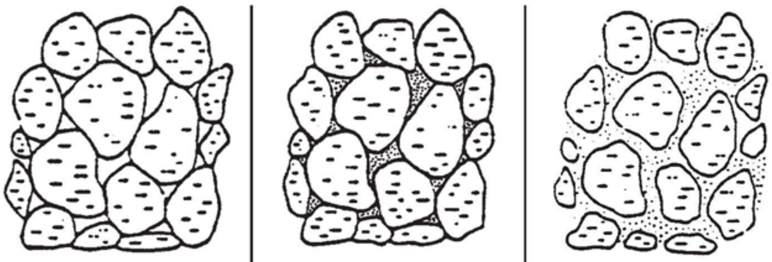
- Poor surface finish
- Ravelling

(B) **Too many fines** (e.g. sands, fine grained lateritic and calcareous materials, pyroclastic ash)

- Low compacted strength and increased risk of permanent deformation through shear failure
- High potential for capillary rise
- Moisture susceptibility
- Production of dust

(C) **Gap-graded** (e.g. pedogenic deposits)

- Difficult to compact
- Increased risk of settlement or permanent deformation under traffic
- Increased moisture susceptibility
- Pumping of fines thereby decreasing stability of the pavement under load
- Lower maximum density with decreased strength.



Aggregate with no Fines (0 % Fines)	Aggregate with Sufficient Fines for Maximum Density (Ideally 8-15 % Fines)	Aggregate with High Amount of Fines (> 25 % Fines)
• Grain-to-grain contact • Variable density • High permeability • Non-frost susceptible • High stability when confined, low when unconfined • Not affected by adverse water conditions • Difficult to compact • Ravels easily	• Grain-to-grain contact with increased resistance against deformation • Increased to maximum density • Low permeability • Frost susceptible • Relatively high stability in confined or unconfined conditions • Not greatly affected by adverse water conditions • Moderately easy to compact • Good road performance	• Grain-to-grain contact destroyed, aggregate is "floating" in soil • Decreased density • Low permeability • Frost susceptible • Low stability and low strength • Greatly affected by adverse water condition • Easy to compact • Dusts easily

Source: Yoder and Witczak, 1975

Figure 3-1: Illustration of problems presented by poor grading

3.6.1 *Particle Size Distribution Assessment*

Particle Size Distribution (PSD) (also called Gradation Analysis or Sieve Analysis), is normally assessed by testing suitably representative samples. Sieving is used for gravel and sand particles whilst for the smaller silt and clay size particles a sedimentation procedure is used (either the pipette method or the hydrometer methods). Although both wet and dry sieving are catered for in the standard procedures, dry sieving is only recommended for materials known to be free of fine particle sizes. Aggregate containing silt or clay requires the use of the wet method, in which the material must first be washed to remove the fine particles (passing 63 µm or 75 sieve µm). Grading acceptability is based on the following general criteria:

- The overall distribution of grain sizes
- The amount of fine material ("fines")
- The amount of very coarse material ("oversize")

Results are normally presented as a particle size distribution curve although for specification comparison purposes tabulated outputs may be used. It is, in addition, possible to use a range of grading indices to define materials. Specifications can use a variety of these to set limits to the grading acceptability of materials. Commonly used indices are defined below:

- Fines Ratio (FR) = P 0.075/P 0.425
- Grading Coefficient (GC) = (P 26.5 - P2.00) x P4.75/100
- Grading Modulus (GM) = [300 - (P2.00 +P 0.425 + P0.075)]/100
- Coarseness Index (IC) = (100 - P2.36)
- Fineness Index (IF) = P 0.075
- P 26.5 = percentage of material passing the 26.5 mm sieve
- P 4.75 = percentage of material passing the 4.75 mm sieve
- P 2.36 = percentage of material passing the 2.36 mm sieve
- P 2.00 = percentage of material passing the 2.00 mm sieve
- P 0.425 = percentage of material passing the 0.425 mm sieve
- P 0.075 = percentage of material passing the 0.075 mm sieve
- Reject Index (IR) or Oversize Index (IO) = per cent retained on 37.5 mm sieve.

3.6.2 *Using Poorly-graded Materials*

Table A-2 in Annex 1 summarises key aspects of poorly graded materials relevant to their use as sub-base or roadbase aggregate. Engineering risk associated with the use of an unmodified poorly graded material may be accommodated within an appropriate design, or the assumption may be made that construction and in--service use will improve the grading characteristics. If these scenarios are not valid, then the options for improvement of the engineering properties of the material include:

- Material processing (crushing and/or screening),
- Mechanical stabilisation,
- Lime/cement modification to deal with the effects of excess fines (c.f. high plasticity materials),
- On-road removal or crushing of oversize material.

3.7 Poorly-shaped Materials

Sub-base or roadbase aggregate are inappropriate for use in terms of shape if it is not only too flaky or elongated but also if its particles are over-rounded with no angular faces. Although not strictly a shape criterion, surface texture is also considered within this group, particularly a smooth polished texture if it occurs in association with over-rounded particles.

Poor particle shape in terms of high flakiness or elongation, in either natural or processed aggregate, is largely a function of inherent characteristics of the parent material, although in some cases it may be made worse by poor crushing procedures. Typical materials that have high flakiness or elongation problems are strongly anisotropic metamorphic rocks such as shales, slates, phyllites or schists. Some igneous lava materials also have shape problems as a result of inherited flow banding anisotropy.

Over-rounding in contrast is largely the result of secondary sedimentary processes, although the character of the parent material does have an influence. Typical of these materials would be unprocessed alluvial sands and gravels and wind-blown sands. Rounded river gravels typically need crushing for good performance on the road.

Poor shape can be associated with a number of performance defects such as those as summarised below:

(A) **Rounded** (in association with smooth surface texture)

- Poor inter-particle friction and loss of stability
- Compaction difficulty
- Low density and high air voids content

(B) **High flakiness or elongation**

- Particle breakdown or crushing
- Compaction problems and high air voids
- Rebound with mica rich materials.
- Perform poorly in seal coats.

3.7.1 Shape Assessment

Particle shape is generally defined by British Standards (BSI, 1990a) in terms of a flakiness index (If) and an elongation index (le) with reference to standard shape gauges. Where:

If = Mass of particles whose least dimension is <0.6 mean dimension/ Total mass

le = Mass of particles whose long dimension is >1.8 mean dimension/ Total mass

Additional shape factors that may be used include angularity and sphericity. The former can be arrived at by indirect methods in terms of the angularity number (AN) which is reported as a comparison with a standard well-rounded river gravel of 33% air voids (BSI, 1975). The Average Least Dimension (ALD) is *a* shape parameter used in assessing surface dressing aggregate and is defined as the least perpendicular distance between two parallel plates through which a particle will pass.

3.7.2 Using Poorly-shaped Material

Table A-3 in Annex 1 summarises key aspects of poorly shaped materials relevant to their use as sub-base or roadbase aggregate. If shape characteristics cannot be accommodated within an appropriate road design, then options exist for improvement of the engineering properties as follows:

- Over rounding - material crushing,
- Flakiness or elongation - modified crushing procedures, e.g. choke feeding or high impact crushing plant,
- Mechanical modification with other materials.

3.8 Low Particle Strength Materials

Sub-base and roadbase aggregates are inappropriate for use in terms of particle strength if the individual particles, either through inherent defects or degradation, fail to meet crushing strength criteria, usually defined in terms of the laboratory performance of a mass sub-sample. Low particle strength is normally due to:

- Inherent fabric defects (e.g. shales, sandstones, mudstones, some metamorphic rocks)
- Incomplete or weak induration (e.g. pedogenic gravels, laterites, sedimentary rocks, manufactured materials and wastes,
- Weathering (e.g. potentially all weathered materials)

Low particle strength is also a function of aggregate shape and size. The greater weakness of flaky or elongated particles is associated with the greater bending moments applied to their cross-sectional areas, as well as the point crushability of angular fragments. Size effects can be related to the statistical distribution of flaws, whereby there may be a greater likelihood of flaws in larger rather than a smaller aggregate particle (Smith and Collis, 1993).

Weak particle strength can be associated with a number of performance defects, such as:

- Breakdown under compaction or traffic,
- Generation of excess fines,
- Reduced compacted strength,
- Increased risk of settlement or permanent deformation under traffic,
- Pumping of broken-down fines.

Low particle strength may, however, also have advantages for some materials, for example:

- Easier excavatability,
- Easier densification (compactability),
- Breakdown of particles may improve grading,

3.8.1 Assessment of Low Particle Strength Materials

Commonly used laboratory aggregate strength tests are the Aggregate Impact Value (AIV), the Aggregate Crushing Value (ACV) and its related 10% Fine Aggregate Crushing Test (10% FACT), as well as the Los Angeles Abrasion (LAA) (or LA Rattler) test. The main disadvantage of these tests, in their unmodified form, is that they use only a single size (9.5 mm – 14 mm) fraction of the aggregate. In the context of low-strength materials there may be a requirement to modify standard procedures for these tests.

The AIV test measures the ability of aggregate particles to resist 15 blows of standard weight dropped through a standard height by measuring the amount of fines (passing 2.36 mm sieve) produced. Apart from procedural non-compliance the test result can be influenced by factors such as particle shape, moisture condition, base plate seating and the cushioning effects of the produced fines. Modified AIV test procedures have been proposed by Hosking & Tubey (1969), which can be employed to:

- Measure the intermediate breakdown between 10 mm and 2.36 mm,
- Limit the number of blows and then extrapolate to the full 15 blows,
- Test both in an unsoaked and soaked condition.

The ACV evaluates the resistance of aggregate particles to a continuous load of 400 kN over a period of 10 minutes by measuring fines produced as in the AIV test. In order to reduce cushioning effects, the 10%FACT is more generally used in which the load to achieve 10% breakdown is measured. As with the AIV, both soaked and dry samples can be tested. In some aggregate specifications the ratio of soaked to dry values is stipulated.

The use of ethylene glycol-soaked aggregate can be used in the above tests as a rapid means of evaluating suspect materials such as basic igneous rocks. Ethylene glycol AIV tests that are five percentage units above an AIV (wet) test result are considered indicative of a problem material (Sampson, 1990).

3.8.2 Using Materials of Low Particle Strength

Table A-4 in Annex 1 summarises key aspects of low particle strength materials relevant to their use as sub-base or roadbase aggregate. Engineering risk connected with the use of low particle strength materials may be accommodated within an appropriate design or the assumption may be made that in service use will lead to “improved” characteristics such as grading. If these scenarios are not valid, then the options for improvement of the engineering properties of the material include:

- Material screening,
- Mechanical stabilisation,
- Lime/ cement modification.

3.9 Materials of Low Durability

Durability can be defined as the ability of a construction material to maintain its mechanical and physio-chemical integrity within the road design life. The low durability group may contain materials that meet all other specification criteria but can deteriorate whilst in service. Durability problems can range from short-term degradation during construction to long term in-service deterioration. In the context of standard materials for LVRs, the latter issue is of key importance when dealing with the apparent failure of specification-acceptable materials e.g. weathered basalts, or shales.

This deterioration can take a number of forms including:

- Inherent fabric defects (e.g. shales, some sandstones and metamorphic rocks),
- Incomplete or weak induration (e.g. pedogenic gravels, some sedimentary rocks, manufactured materials),
- Weathering (e.g. potentially all weathered materials).

3.9.1 Assessing Durability

Durability assessment should ideally have elements of behaviour modelling and prediction and should not rely on one-off strength evaluations. In practice, durability procedures involve assessing the performance of aggregate when subjected to some form of artificially imposed degradation or weathering. Some test procedures, such as Los Angeles Abrasion (LAA), encompass elements of both strength and durability testing.

The LAA test is sometimes linked in specifications to AIV and ACV tests. However, the test mechanism is more one of mechanical degradation rather than particle strength and it should be considered more reasonably as a durability indicator. The Durability Mill test, or Texas Ball Mill test, has the advantage over other strength-degradation tests in that a larger range of grading is tested, less bulk samples are required, and the degraded products of the test are retained for identification and assessment. The National Institute for Transport and Road Research (NITRR) procedure includes testing the plasticity of the degraded fraction and recording the results as part of the durability index (Sampson and Roux, 1987).

Sodium and magnesium soundness tests measure resistance to mechanical degradation through cycles of crystallisation and rehydration. They have, however, come under criticism for lack of reproducibility in comparisons between different laboratories. The test is unfortunately susceptible to poor laboratory management practices, in particular with respect to temperature control and the type of sodium sulphate used.

The slake durability index test (ISRM, 1981), in addition to being a useful performance indicator can perform a significant role in indexing materials in the rock to hard soil range. The combination of slake index with plasticity has been suggested as a useful means of presenting results for argillaceous materials.

Combined mineralogical, textural and fabric examination procedures, such as those as described by Cole and Sandy (1980), are strongly recommended as part of any comprehensive aggregate assessment for materials with actual or suspected durability problems. Table A-6 in Annex 1 summarises common petrographic procedures that may be employed in durability assessment.

3.9.2 Using Materials of Low Durability

Table A-5 in Annex 1 summarises key aspects of low durability materials relevant to their use as sub-base or roadbase aggregate. Engineering risk associated with the use of low durability materials may be accommodated within an appropriate design or the assumption may be made that in-service use will lead to "improved" characteristics such as grading or plasticity. If these scenarios are not valid, then the options for improvement of the engineering properties of the material include:

- Material screening,
- Mechanical stabilisation,
- Lime/ cement modification.

3.10 A Risk-based Approach to Material Selection for Sealed Roads

A recent publication (Austroads, 2020) uses a combination of factors to select appropriate materials for base and sub-base layers of sealed roads. A matrix of traffic measured in MESA and probability of mean annual rainfall exceeding 500 mm/y is used. Three classes of traffic and three risk levels of rainfall exceeding 500 mm/yr are used to form nine risk levels (a 3 x 3 matrix). Based on the traffic level and the expected mean annual rainfall for a given site, for example traffic less than 0.25 MESA and low probability of rainfall exceeding 500 mm/yr, a risk category representing that combination is obtained. Using the risk category obtained from the matrix, the risk of using different types of materials are shown. The materials characteristics are measured in terms of the Fineness Ratio (%age of particles passing the 75 µm sieve to that passing the 425 µm sieve), Grading Modulus (GM), and Shrinkage Product (SP). The decision of the risk level to be selected for each project has to be made by the design engineer based on the choice of materials available. Details of how to use this approach are found in Austroads (2020).

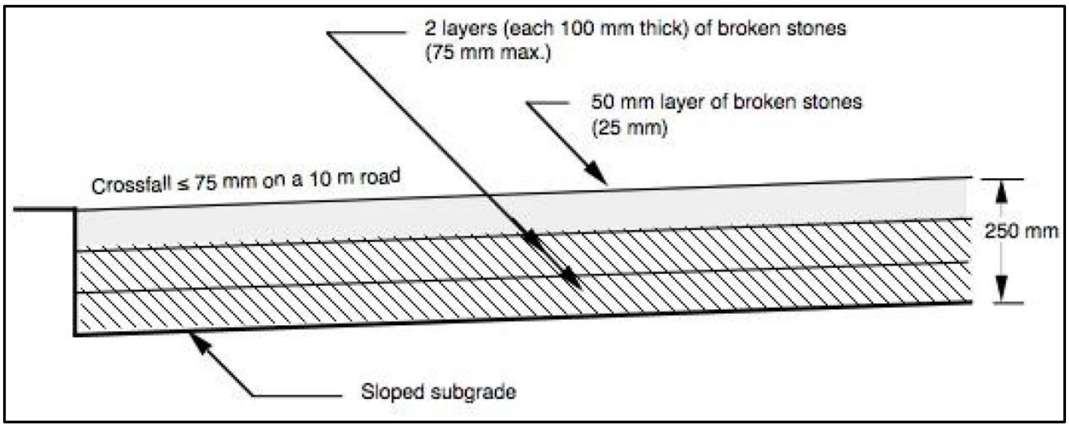
3.11 Macadam and Telford Materials

In areas where rocks, coarse aggregates, or crusher run (large crusher reject), natural sand or crusher sand are available, it is prudent to consider designing the pavement as a Macadam or Telford structure (Figure 3-2 and Figure 3-3). Usually a layer of 150-200 mm of Macadam or Telford laid on a sub-base of at least soaked CBR 30% is sufficient for low volume roads. The particle size distribution of Macadam is shown Table 3-2.

Table 3-2: Particle size distribution of Macadam roadbase

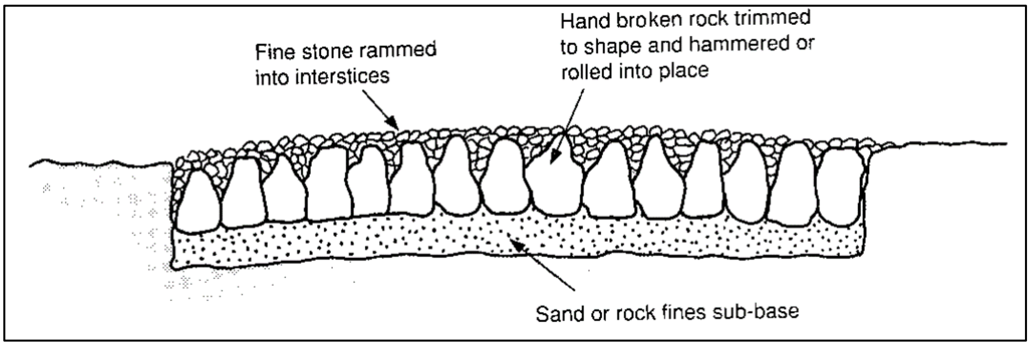
Sieve Size (mm)	Mechanically Crushed Aggregate (%age passing)	Hand-broken Stone (%age passing)
75	100	100
50	85-100	85-100
37.5	0-30	35-70
28	0-5	0-15

Non-plastic rock fines (crusher sand) or natural sand – both materials containing particles no more than 5 mm in size can be used for grouting the Macadam.



Source http://www.pavementinteractive.org/article/pavement-history/

Figure 3-2: Macadam pavement cross-section



Source: Millard, 1993

Figure 3-3: Telford pavement

The Telford construction type is more suited to areas where easily-shaped stone and labour are largely available; and there is a drive to provide employment. It involves wedging boulders (about 200 mm in size) into the substrate (at least CBR 15%) and then ramming finer stone into the interstices so that the large stones are completely covered as in Figure 3-3. This differs from 'Hand-Packed Stone (HPS)' in that for HPS, the large stones are laid with flat faces up and finer stone only fills the gap between large stones. Bituminous prime and thin surfacings can then be applied on top of the Macadam or Telford base.

3.12 Laboratory Assessment of Pavement Materials

Laboratory procedures for investigating the properties of different materials are required in almost all design methods. The main difference is that for the CBR based methods the CBR test is the primary test of strength whereas in the DCP-DN method the CBR is replaced with a DCP test. The Laboratory DN test is used both for evaluating the subgrade as well as imported materials for new pavement layers. It is recommended that in addition to the standard compaction procedure in the laboratory, each specimen shall be subjected to a CBR test (CBR-based methods) or penetrated with the DCP (DCP-DN method) to obtain a measure of the CBR or DN value at the different moisture contents and densities. This gives an initial indication of the suitability of the material for a particular Traffic Loading Class (TLC). The normal procedure is to test at three moisture contents and 3 levels of compaction as shown in the illustration in Table 3-3. The three levels of moisture content are sufficient to estimate the minimum likely strength of the material at the required moisture content. Generally, for CBR-based methods, materials are often classified on the basis of the soaked strength. However, if the engineer is satisfied that the material will be subjected to a drier environment (such as where shoulders are sealed or a high crown height), the strength can be classified at lower moisture content decided by the engineer.

Table 3-3: An example test matrix for compaction and strength assessment of materials

Compactive effort	Moisture regime		
	Soaked	OMC	0.75 OMC
Light	3 samples	3 samples	3 samples
Intermediate	3 samples	3 samples	3 samples
Heavy	3 samples	3 samples	3 samples

3.13 Material Specifications for CBR-based Pavement Design Methods

Natural gravel materials for these design methods are classified as shown in the example presented in Table 3-4. Note that differences will vary from country to country and country specifications that override this example. It should also be noted that variability in quality of natural materials (even within the same borrow-pit) means that results of laboratory tests that vary within a range of ±20% of the listed minima are acceptable. Thus CBR 30 and CBR 25 may not differ much in performance.

It is important that engineers are aware that more recent and ongoing research has, and is likely to show that the values shown in these examples could be revised based on evidence. It is recommended that researchers and engineers develop indigenous specifications through in-country research to calibrate the evidence given here.

The most common test methods are the American Association of State Highway and Transportation Officials (AASHTO), American Society for Testing and Materials (ASTM), British Standards Institution (BSI), and Test Methods for Highways (TMH) methods – now superseded by South Africa National Standards (SANS). There are often differences in results obtained by using 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. However, values of the differences are material dependent and, in some cases, may even lie within experimental error.

Table 3-4: An example of pavement material specifications

Code	Material	Abbreviated Specifications
NG80	Natural Gravel	1) Min. CBR: 80% @ 98/100% BS heavy compaction and 4 days soaking. 2) Max. Swell: 0.2% 3) Max. Size and grading: Max size 37.5 mm, grading as specified. 4) PI: Dependent on material type, traffic and subgrade strength or as specified.
NG65	Natural Gravel	1) Min. CBR: 65% @ 98/100% BS heavy compaction and 4 days soaking. 2) Max. Swell: 0.2% 3) Max. Size and grading: Max size 37.5 mm, grading as specified. 4) PI: Dependent on material type, traffic and subgrade strength or as specified.
NG55	Natural Gravel	1) Min. CBR: 55% @ 98/100% BS heavy compaction and 4 days soaking. 2) Max. Swell: 0.2% 3) Max. Size and grading: Max size 37.5 mm, grading as specified. 4) PI: Dependent on material type, traffic and subgrade strength or as specified.
NG45	Natural Gravel	1) Min. CBR: 45% @ 98/100% BS heavy compaction and 4 days soaking. 2) Max. Swell: 0.2 % 3) Max. Size and grading: Max size 37.5 mm, grading as specified. 4) PI: Dependent on material type, traffic and subgrade strength or as specified.
NG30	Natural Gravel	1) Min. CBR: 30% @ 95/97% BS heavy compaction & highest anticipated moisture content 2) Max. Swell: 1.0% @ 100% BS heavy compaction. 3) Max. Size and grading: Max size 63 mm or 2/3 layer thickness. 4) PI: Dependent on material type, traffic and subgrade strength or as specified.
NG15	Gravel/soil	1) Min. CBR: 15% @ 93/95% BS heavy compaction & highest anticipated moisture content. 2) Max. Swell: 1.5% @ 100% BS heavy compaction. 3) Max. Size: 2/3 of layer thickness 4) PI: Dependent on material type, traffic and subgrade strength or as specified.
NG7	Gravel/soil	1) Min. CBR: 7% @ 93/95% BS heavy compaction & highest anticipated moisture content. 2) Max. Swell: 1.5% @ 100% BS heavy compaction. 3) Max. Size: 2/3 layer thickness. 4) PI: Dependent on material type, traffic and subgrade strength or as specified.
NG3	Gravel/soil	1) Min. CBR: 3% @ 93/95% BS heavy compaction and highest anticipated moisture content. 2) Max. Swell: N/A. 3) Max. Size: 2/3 layer thickness.

Source: Modified from Gourley and Greening, (1999).

In Table 3-4 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.

Table 3-5: **An example of particle size specification for natural gravel roadbases**

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)

Unlike the DCP-DN method, materials for these design methods must simultaneously satisfy the specifications for grading (grading envelope and GM) and Atterberg limits (PI and PM) as given inTable 3-5, Table 3-6, and Table 3-7, which are dependent on the Subgrade Class defined in Table 3-7. The rationale is that since the methods are empirically developed, certain key materials characteristics must also be replicated if the design is to perform successfully.

The strength and plasticity specifications vary depending on the traffic level and subgrade class as outlined in Table 3-6. The soaked CBR test is used to specify the minimum base material strength.

Table 3-6: Basic plasticity specifications for natural gravel roadbases

Subgrade Class (CBR)	Property	Upper limit of design traffic class (MESA)					
		TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0	TLC 3.0
S2 (3-4%)	I_P PM Grading	<12 <400 B	<12 <300 B	<12 <240 A	<9 <180 A	<9 <180 A	<6 <90 A
S3 (5-7%)	I_P PM Grading	<15 <550 C(1)	<15 <320 B	<15 <320 B	<12 <240 A	<9 <180 A	<6 <90 A
S4 (8-14%)	I_P PM Grading	Note (2) <800 D(3)	<15 <320 B	<15 <320 B	<12 <240 B	<9 <180 A	<6 <90 A
S5 (15-29%)	I_P PM Grading	Note (2) n/s D(3)	<15 <400 B	<15 <320 B	<12 <240 B	<9 <180 A	<6 <90 A
S6 (>30%)	I_P PM Grading	Note (2) n/s D(3)	<15 <550 C(1)	<15 <500 B	<12 <240 B	<12 <240 A	<6 <90 A

Notes:

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

(2) Maximum I_P = 8 x GM

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

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

I_P = Plasticity Index

PM = Plasticity Modulus; PM = Plasticity modulus = PI x $P_{0.425}$

n/s = Not specified

Source: Gourley and Greening, 1999.

A maximum PI of 6% is specified for higher traffic classes and also on weaker subgrades. When the climate is dry and the drainage arrangements (i.e. including crown height and sealed shoulders) are satisfactory as outlined in section 5.2.3 a higher PI is often specified ranging up to 9-12% can be used . Subgrades are often classified on the basis of the soaked CBR strength at a given design density (usually 90% to 95% maximum dry density) as shown in Table 3-7. Note that in some countries only 3 or 4 subgrade classes are defined.

Table 3-7: An example of subgrade class definitions

Subgrade Class	Design CBR (%)	Notes
S2	3 - 4	May be used in fills not exceeding 2 m in height.
S3	5 - 8	May be used in all fills.
S4	9 - 14	May be used in all fills.
S5	15 -29	May be used in all fills and as selected fill layer: the selected fill is usually compacted to 95% heavy compaction.
S6	≥ 30	May be used in all fills and as sub-base if the upper 150 mm or the sub-base layer is fully compacted to 95% heavy compaction.

Source: Gourley and Greening, 1999.

3.13.1 *Example Specifications for Lateritic Gravel Pavement Materials from East Africa*

An example of incorporating the understanding of the behaviour of materials with certain geological characteristics with engineering characteristics are specifications specific to materials such as lateritic gravels. The requirements for the selection and use of lateritic gravels for bases, as shown in Table 3-8, are slightly different to those given for other natural gravels, as shown above. A maximum PI of 9% has been specified for some of the higher traffic levels (0.3 – 0.5 MESA) and weak subgrades (S2).

Table 3-8: Specification for typical East African lateritic gravel base materials

Subgrade Class (CBR)	Property	Limit of design traffic class (MESA)				
		<TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
S2 (3-4%)	PI PM GE	<15 <400 B	<12 <150 B	<9 <150 A	<9 <120 A	<6 <90 A
S3 (5-7%)	PI PM GE	<18 <550 B	<15 <250 B	<12 <180 B	<9 <120 A	<6 <90 A
S4 (8-14%)	PI PM GE	<20 (1) (800 GM 1.6-2.6	<15 <320 B	<15 <300 B	<9 <200 B	<9 <90 A
S5 (15-29%)	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 (>30%)	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). More details on laterites are found in CIRIA (1988).

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 reduced to a liquid limit of less than 35%. Where sealed shoulders over one-metre wide are specified in the design, the maximum plasticity modulus may be increased by 40 %. A minimum field compacted dry density of 2,000 kg/m^3 is required for these materials.

Other treatment includes the modification of gravels using low content of lime (about 3%) or cement (about 2%). It should be noted that in this case the material still behaves as a granular material unlike a stabilised material with lime or cement content in the region of 4%. However, gravels modified in this way should be stronger and will have a reduced PI compared with their unbound versions and will perform better.

It is important to assess all possible options in order to utilise locally available materials. The options should then be compared using life-cycle cost analysis discussed later in this document. It should not come as a surprise that some options that are initially expensive turn out to have lower life-cycle costs.

3.14 Material Specification for the DCP-DN Design Method

Selection of pavement materials for use with DCP-DN method is based on the following procedure:

(a) The evaluation of earthworks, subgrade and pavement materials on the basis of their characterisation as defined by relevant materials testing in terms of grading, plasticity, deleterious inclusions (e.g. organics) or other specific properties such as swell, erodibility or collapse potential.

(b) The selection of materials in terms of acceptability for specific use is then based on judgment related to a combination of specified criteria allied to engineering judgment, bearing in mind the preference for local material use on LVRRs.

(c) Once acceptability is agreed, the use of DCP-DN procedures to select and control the use of materials that have been previously defined as acceptable.

Testing to ascertain the durability properties of the material is undertaken separately from the DCP-DN test based on appropriate durability testing.

The primary specification of strength assessment for the DCP-DN method is the DN value (resistance to penetration measured in mm/blow of the DCP) of a material intended for use in the pavement. Limits on the material plasticity index are not specified for the DCP-DN method. Nevertheless, Atterberg limit tests must be carried out for all material samples to enable the design engineer to make a judgement on their influence on the expected long-term performance of the materials. Particle size distribution test is carried out and specified in terms of Grading Modulus (GM); the acceptable range being 1 to 2.25.

In addition, the method also emphasises two critical factors that affect the long-term performance of the road:

a) The need to specify as high a level of density as practicable (so-called compaction to refusal) by employing the heaviest rollers available and compaction at optimum moisture content (OMC). 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 breakdown of individual particles of the material.

b) 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.

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4 THE PRINCIPAL PAVEMENT STRUCTURAL DESIGN METHODS FOR LVRS

4.1 Background and Scope

Pavement design methods have been developed and published by many authors. For a specific set of conditions, the various methods do not give the same solutions. In particular, and most importantly, there are some major differences that are responsible for the different solutions and have led to some element of confusion and uncertainty amongst road designers concerning which method to choose and how to use it. Although these are discussed in detail in this Road Note, it is worth summarising those major differences and similarities here as a brief introduction before looking at the consequences.

The purpose of this chapter is to provide the basic background information on six pavement design methods (The AASHTO method, the ORN 31 method, the TRL-SADC Method, the Foundation Class method, the ORN 18 DCP-CBR method, and the DCP-DN method) currently incorporated in a number of LVR manuals. Before doing so the process of developing a pavement design method in general is described followed by a description of the origins of each one so that any limitations on their use can be appreciated.

It should be noted that the methods discussed here were empirically developed and therefore should be used within the boundaries of materials, traffic, and climate (more especially rainfall) that were experienced during their development. It is recommended that researchers and engineers refine these methods, and extend their boundaries through in-country evidence-based research.

4.2 Developing Design Charts

4.2.1 The Task

The purpose of the road pavement is simply to provide a good quality running surface for vehicles to run on because the normal subgrade is too weak to support the traffic. Additionally, the pavement should provide a cost-effective surfacing solution and last for many years (ideally 15-25 years). Hence the pavement must be thick enough and strong enough to last and to protect the subgrade from failing. At the same time the pavement layers themselves should not fail. Hence the design of a pavement requires sufficient strength in the pavement layers and sufficient thickness to protect the subgrade. The structural design of a road depends on and requires four principle parameters namely:

1) Traffic level (volume and axle loads). This is a measure of the stresses and strains that the road must support.
2) Strength of the subgrade.
3) Strength of the pavement layers.
4) Thickness of the pavement layers.

These of course depend on other factors but if the traffic level and subgrade strength are known, it is straightforward to present the structural designs for each pavement type as a catalogue of structures comprising a matrix with traffic level as one component and subgrade strength as the other component. This can be replicated for pavement materials of differing strengths to cover a range of available materials. This is how the majority of experimentally derived design methods are presented.

In general, as traffic levels increase, then the thickness and sometimes the strength of the pavement layers increase. At least this used to be the case, but it has now been established that once the stresses and strains in the pavement layers are reduced to below critical levels then the layers do not need to increase in strength or thickness. This is the principle of the 'long-life' pavement described by Nunn et al. (1997) but it occurs at considerably greater thicknesses and strengths than are required for LVRs.

The majority of design methods and associated design charts are based on the results of empirical studies in which the performance of a number of roads has been monitored for a period of time and their performance analysed to determine reliable designs for the range of conditions included in the studies. Many design methods in use today are based to a greater or lesser extent on the AASHO Road Test which took place in the USA in 1960. One of the reasons for this is that a number of concepts that were developed in this study are still relevant today. The Road Test took place in one place in the USA on just one subgrade and in one climate so therefore there are some limitations to its use and differences as to how it was adapted and extended to different conditions and situations. Nevertheless, there are some fundamental principles that are relevant.

When investigating the performance of roads to improve designs it is vital that the reasons for failure or poor performance are investigated because any improvements that may be required depend critically on what is not satisfactory. Unfortunately, such a diagnostic process is not as straightforward as it first appears. Road pavements materials are notoriously variable, and many other factors come into play to affect their performance. A consequence of this is that poor performance has often been attributed to a deficiency in protecting the subgrade and, as a result, thickness designs have tended to drift upwards.

A key requirement for developing design charts is a method of comparing pavements of different layer thicknesses and layer strengths with each other to determine their traffic carrying capacities. Such a method is also essential because in the empirical studies of road performance on which design charts are based, it is impossible to include every subgrade strength and every potential structural design hence it is vital to be able to interpolate between the designs of structures whose performance has been shown to be acceptable for their specific set of conditions. The structural number concept initially developed during the AASHO Road Test is often the basis of this.

4.2.2 Validity of Design Charts

An important principle of the experimental scientific method that underlies the development of any design method and the resulting design charts is that the results are only strictly applicable within the range of the experimental data on which each one is based. Extrapolation to conditions that were not included in the design of the experimental study is extremely dangerous (and represents poor science). Nevertheless, a successful design method can be developed that is configured to allow calibration to a wider range of conditions, but such a process essentially requires further empirical studies.

Although the validity of any method is confined to the range of conditions in which it was developed, the edges of this range are never precise and it is inevitable that a designer may be required to provide a design for conditions that are not all fully within the range of those of the original study. Some of those conditions are much more important than others in terms of controlling the performance of the road but there is an area of uncertainty where the less important factors play only a marginal role and where good engineering judgement and common sense may be required to make a design decision. The danger here is that if more than one marginal factor is present at the same time, the combination could be unfortunate. Such uncertainties represent risks that the designer must consider. It is for these reasons that to define the boundaries of each of the methods, the range of the variables in the studies is shown in sections 4.4 to 4.8, where each of the design methods are presented. The most important limitations are also highlighted.

With most engineering design there are also associated risk factors caused by uncertainty in being able to predict future events. Within the design methods there are different ways whereby risks can be reduced. These are also discussed and summarised in Chapter 5.

4.2.3 Thickness Design

The first and most important point is that each method must be used in the way that the developers intended. Unfortunately, this is not always made clear with the consequence that methods can easily be used incorrectly. An example of this is if the worst-case moisture condition does not necessarily mean soaked, but many designers base subgrade strength on soaked conditions whereas the worst case may be drier than soaked and sometimes quite a lot drier. One way that this problem can be avoided is if the analysis of the source data that the authors have used to develop their design method is known and understood by the design engineer. However, it is unreasonable to expect the road designer to investigate the research on which the design method is based and so the onus should be on the authors to provide sufficient information to satisfy the road designer. The methods used and the designs produced should be prudent and cost-effective, but some conservatism is warranted given the variability in climates today and the uncertainties that climate change variability produces.

This Rural Road Note will guide the reader through the methods to ensure that each method is used as the authors intended.

4.2.4 Materials

All methods designed for LVRRs attempt to make use of locally available materials whose haulage and processing costs will be low. The selection and use of materials suitable for low volume roads standards are discussed in Chapter 3 (*Material Selection for LVRs)* and Chapter 6 *(Design of Unpaved Roads)* for unpaved roads.

4.2.5 Methods of Measurement and Data Analysis

The input data required for any pavement design method are well known in general terms. They are primarily:

1) The design period and how it is defined. Though many ways exist for evaluating this, the same approach can be applied in any pavement design method.
2) Information about the expected traffic; numbers and types of vehicle, their loading, etc. Likewise, although many ways exist for evaluating this, the same approach can be applied to any pavement design method.
3) Details concerning the properties of the materials available for construction, especially the subgrade and roadbase.
4) Selection of the materials including the surfacing layer.

Despite the apparent simplicity of these elements there are different methods of obtaining the required information and each can influence the values obtained for the pavement design and subsequently the final design.

Most importantly the choices made by the designer are critically influenced by the risks that are perceived and the risks that are considered reasonable and therefore acceptable.

4.3 Key Decisions

There are numerous decisions to be made when designing a LVR but there are actually very few to be made when carrying out the structural design, as shown in the following Table 4-1.

Table 4-1: Key decisions required for structural design

Decision		Comments
1	What type of road to build or rebuild	Most of the time this will be a choice between an unsurfaced or gravel road or a sealed road with a thin bituminous surfacing. Such a decision is usually made at the planning stage (Figure 1-1) but economic considerations, as described in Chapter 8, are perhaps the most significant factors. More guidance on selection between paved and unpaved options is given in Cook et. al. (2013).
2	The following steps and associated decisions are required for a sealed road. These differ between the various design methods and are essentially the only real differences between the methods.	
2.1	Determine or select the subgrade strength for design.	This is a major difference between methods and is where the designer will need to exercise his or her judgement of risk.
2.2	Determine the strength (specifications) of the pavement layers and decide whether lower cost materials will be adequate. A series of material tests must be implemented as described in the methods.	Check that such material is available.
2.3	Check whether any further reduction of specifications is viable	(e.g. for better than average drainage conditions)
2.4	Determine the thickness of each layer	At this stage it will involve only a look up catalogue which will obviously differ for some of the methods and those differences need to be explained

4.3.1 Risk

Risk is a necessary but unwelcome aspect of civil engineering that is addressed in different ways in the various pavement structural design methods and which is often the cause of uncertainty for designers. The method of dealing with risk for drainage structures such as culverts and bridges is to base designs on the probability of a specified storm event. For relatively large structures such as bridges, failure is very expensive and so very safe designs are selected to withstand the very rarest severe storms (e.g. the 1 in 100-year storm, i.e. a storm with only a 1 per cent probability of occurring in any year). For small structures that are relatively inexpensive to repair or replace (e.g. small culverts) the 1 in 20 or 25-year storm (4% or 5% probability of occurring in any year) is used for design.

For pavement structures few design methods actually define the risk levels that they are catering for, even though they are ideally designed for a 15 to 25-year life. Road designers are well aware that over the design life of a typical road there are likely to be certain severities of storm and variable levels of risk because, for example, ideal maintenance to keep culverts and drainage ditches clear and to rectify erosion and surface damage cannot be guaranteed. Thus, with no easy quantification of risk available to designers they are often tempted to ere on the side of safety. To do so however, the designers must understand how their design charts were devised. In the sections that follow these are described to assist the designer in understanding and catering for the risk element.

4.3.2 Anomalies and Inconsistencies

There are also anomalies and inconsistencies in some of the methods that often arise because of inadequate original data and sometimes through technical disagreements between experts.

It is the purpose of this Road Note to guide the user through all of these issues by:

- Reminding users of the key features of the principal methods of designing LVRRs,
- Ensuring that the methods are used as the authors intended,

- Highlighting discrepancies and technical uncertainties that contribute to risk that require the engineering judgement of the designer.

The chapter further presents the contexts of use, risks, and limitations the designer needs to be aware of while using the various methods.

4.4 The AASHTO Method

4.4.1 Background

The AASHO Road Test on which the AASHO pavement design method was originally based, consisted of six two-lane loops. Each lane was subjected to repeated loading by a specific vehicle type and weight. The pavement structure within each loop was varied so that the interaction of vehicle loads and pavement structure could be investigated. The results from the Road Test were used to develop a pavement design guide, first issued in 1961 with major updates issued in 1972 and 1993 (AASHTO 1993). A summary of the test characteristics is shown in Table 4-2. The 1993 version is still in widespread use in the United States and in many other countries including some in Asia and South East Asia. A new guide, originally planned for release in 2002 is the first AASHTO pavement design guide not primarily based on the results of the AASHO Road Test.

"Satellite studies" were planned in other parts of the USA so that climate and subgrade effects could be investigated, but unfortunately these were never carried out. Despite this limitation the Road Test has led to several important concepts that are used in other design methods but there are also several limitations.

Table 4-2: Summary of the characteristics of the AASHO Road Test

Variable	Ranges	Remark
Total Traffic Loading (axle load repetitions)	1,114,000	Note that these are not ESAs. Axle load ranges were 2-48 Kips
Precipitation (mm/year)	860	65 mm occurs as snow
Number of Sections (Flexible Pavement)	468	
Number of Sections (Rigid Pavement)	368	
Period of Study (years)	2	This can therefore be considered as accelerated loading
Age of sections (years)	2-4	
Materials	Asphalt Concrete, Crushed Dolomitic Limestone, Sand-gravel, Cement-treated Sand-gravel, Cement Concrete	
Subgrade CBR (%)	2-4	Material is A-6 classification. Capping layers and sub-bases treated as other subgrade classes during analysis

Compiled from: Highway Research Board (1962).

4.4.2 *Scope and Concepts*

The AASHTO Method is essentially a method that is based on the California Bearing Ratio (CBR) test, as are many methods in use today. The CBR test is used to quantify the strength of the subgrade soil and also the strength of any unbound pavement layers in the proposed design. Based on this, a Structural Number (SN) of the pavement required to protect the pavement from the estimated traffic loading is computed. The structural number is then converted to materials characteristics and layer thicknesses.

The Road Test consisted of 468 trial sections of flexible pavements and 368 sections of rigid pavements that were trafficked for about two years. The final combined measure of a road's condition was termed the Present Serviceability Index (PSI) which ranged from a maximum of 5 for a new road down to typically 2.0 or 2.5 when the road had reached a poor surface condition. This pavement deterioration was correlated with the Roughness of the road surface, the amount of Surface Cracking and the depth of Ruts in the wheel path.

The amount of damage to the road created by different axle loads was essentially a small decrease in the PSI value and was found to depend on the following exponential law:

Damage = $(L/8.2)^4$ which only identifies the ratio of damage caused by an axle of load L tonnes with a standard axle load of 8.2 tonnes.

The exponent of 4 differs slightly for different pavements and axle configurations but 4.0 or 4.5 are the most commonly used and traffic load is almost universally quantified in terms of cumulative equivalent standard axles over the life of the road (ESA).

Finally, the Road Test was not designed for LVRs although sections of roads surfaced with a thin bituminous seal were included. The limitations of the method are that the Road Test took place in only one location and therefore one climate and subgrade and the fact that the 'satellite' studies designed to permit variations based on climate and subgrade strength to be studied were never carried out. Various techniques were used to remedy these problems, but they amounted to the calibration of the basic design equation.

More details of the method are described in Chapter 5.

A problem with this method is that on its own it does not identify the cause of the deterioration or whether additional pavement thickness is the correct solution for an improved design with longer serviceable life or where in the pavement the strength might need to be enhanced as long as the SN is achieved.

4.5 The TRL Overseas Road Note 31 Method

The current TRL ORN 31 method has evolved from a first early Overseas Road Note published in 1962, a second edition published in 1966, based on research in East Africa on the design of LVRs and on the moisture content under surfaced/paved areas of roads and airfields. It proved to be a much needed manual at the time and a third edition was published in 1977, based on further research in East Africa that provided the data and analysis of the study that produced the Road Transport Investment Model (RTIM) which was an early version of the World Bank's Highway Design and Maintenance Standards Model, HDM. The principles involved were then extended to the main HDM study in Brazil in which the ranges of the variable are as shown in Table 4-3.

The current edition of the Overseas Road Note has extended the designs to different structures and up to 30 MESA in traffic, but its roots lie in the earlier versions based on LVRs.

The most critical parameters are highlighted in italics and the method should only be used with caution if the designer needs to use the method outside the ranges of the variables shown.

It is a CBR based method similar in many ways to the AASHTO method in that CBR specifications are used for unbound pavement layers and for the assessment of subgrade strength for structural design.

Table 4-3: Summary of the characteristics of the roads studied (ORN 31)

Variable	Kenya	Brazil
Total MESA	0.004-3.3	0.003-18
Rainfall (mm/year)	400-2000	1200-2000
Number of Sections	49	116
Period of Study (years)	4	3-5
Age of sections (years)	0-14	0-24
Vehicles/day	323-1618	75-5600
Modified Structural Number	2.5-5.1	1.5 -7.0
Roughness (IRI)	2.9-6.0	1.8-10.2

Compiled from: Paterson (1987).

4.6 The Foundation Class Method

In addition, research in Kenya by Courteille and Serfass has also led to a CBR-based design method that is very similar in principle to ORN 31 but with a slightly different procedure based on designing foundations that are common to all structures on the same subgrade. The method is referred to here as the Foundation Class method. The Foundation Class method provides a logical method of designing the supporting layers i.e. selected subgrade (capping) and sub-base that produces only four (actually 5 but one is rarely needed) foundations that are the same for all traffic levels instead of the rather diverse range of lower layers found in most design charts. This provides an excellent opportunity for better quality control and therefore potentially better overall performance. The data are based on studies in Kenya including the TRL studies that also provided data for ORN 31 but also Kenyan experience leading up to the new designs (MoTIHUD, 2017) published as an output from the ReCAP programme. The method is similar to that used in pavement design in the United Kingdom (Highways Agency, 2009) and France.

4.7 The TRL-SADC Method

This is commonly known as the Gourley – Greening method. This study describes a design method that is very similar to the two described above in principle but the manner of selecting the subgrade strength for design and the way that the performance data were analysed was different and as a result the method of use is different. The study involved the long-term performance monitoring of 59 test sections in Botswana, Zimbabwe and Malawi. The ranges of the variables are shown in Table 4-4. The results indicated that performance was better than predicted using ORN 31 and design charts were produced in which a number of aspects of specifications were modified under appropriate conditions.

Table 4-4: Summary of the characteristics of the roads studied (TRL-SADC)

Variable	Zimbabwe	Malawi	Botswana
Period of Study (years)	4	4	4
Number of Sections	33	16	10
Subgrade Strength (CBR)	10- >30 (S3-S6)	4->30 (S2-S6)	6-24 (S3-S6)
Roadbase Strength Min/max (CBR)	15->100	25->100	30-95
Sub-base and Base Materials	Quartz gravel, lateritic gravel, mixed gravels, weathered rock, calcrete, Kalahari sand, ferruginous gravel.	Quartz gravel, lateritic gravel, weathered rock, ferruginous gravel, crushed stone.	Calcrete, Kalahari sand, weathered basalt, calcareous sand, crushed stone.
Total MESA	0.003-0.04	0.032-1.2	0.2-0.3
Rainfall (mm/year)	400-1400 Weinert Number (2-5)	1000-1700 Weinert Number (<1-2)	300-500 Weinert Number (4-5)
Age of Pavement (years)	3-35	13-27	8-14
Age of Surfacing before Resealing (years)	2-29	8-28	2-15

Compiled from: Gourley and Greening (1999).

4.8 The TRL Overseas Road Note 18 DCP-CBR Method

This method is essentially the same as the TRL Overseas Road Note 31 method except that, in order to develop a statistically correct method of upgrading an existing track or road, a method needed to be devised that allowed the structural deficiency of the existing road or track to be quantified so that a cost effective rehabilitation or upgrading strategy could be developed. The method is based on one of the methods described in the AASHTO Pavement Design Manual and is included in the TRL DCP analysis software and described in Chapter 5.

4.9 The DCP-DN Method

The DCP-DN method represents a different approach in that the CBR test, which is a key feature of the first four methods is not used at all. Whereas in the DCP-CBR method, DCP measurements are converted to CBR values using correlation equations, the DCP-DN does not convert the resistance to penetration (expressed as millimetres of penetration per blow) and instead uses the resistance directly to undertake the pavement design. The DCP-DN design method is empirical in nature and the findings are currently based on measurements and observations on a range of soil types and environmental conditions prevailing in the Republic of South Africa. In most parts of the Republic of South Africa, rainfall does not exceed 1000 mm/year. To adapt the method to regions of higher rainfall and other country-specific variables, a number of trial sections have been constructed in various countries in Africa. Some of the trial sections are undergoing monitoring to provide additional results for adopting the method. The parameters of the Republic of South Africa study used in the development of the method are shown in Table 4-5.

Table 4-5: Summary of the characteristics of the Transvaal study (DCP-DN Method)

Variable	**Range**
Length off road studied	750 km
Number of test pits and DCP measurements	1100
Rainfall (mm/yr)	300 – 1200
Age of roads	6 – 45 years
Traffic range (ESA)	0.04 – $20x10^6$
Vehicles/day	Typically < 500

Compiled from: Paige-Green and van Zyl (2019).

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5 DESIGN METHODS FOR PAVED ROADS

5.1 Purpose and Scope

The purpose of this chapter is to provide guidance on the key steps followed in design of LVRR pavements using the CBR and DCP based methods contained in many design manuals. All together these form six methods namely: The AASHTO method, the ORN 31 method, the TRL-SADC Method, The Foundation Class Method, The Overseas Road Note 18 DCP method, and the DCP-DN method. Other methods discussed in this chapter include: Pure Empirical methods such as Discreet Element pavements, Macadam and Telford pavements; and rigid pavements. A number of countries will have design methods that may not be included here, but it is likely that the methods will be similar in approach to at least one of these presented here. The chapter further presents the contexts of use, risks, and limitations the designer needs to be aware of while using the various methods.

For most design methods it is usually simpler to design an entirely new road than it is to upgrade an existing one that requires rehabilitation and improvement. This is because there are no existing structural pavement layers which need to be utilised in the new structure. In the simplest case only the subgrade properties are required because the structural designs are then obtained directly from a 'look-up' catalogue of structures.

The methods are arranged and discussed in the following order:

A CBR -based methods

1) The AASHTO design method
2) The TRL Overseas Road Note 31 method
3) The TRL-SADC method
4) The Foundation Class method

B DCP-based methods

5) The TRL Overseas Road Note 18 DCP-CBR method
6) The DCP-DN method.

Other methods briefly described in this chapter include: Pure Empirical methods such as Discreet Element pavements, Macadam and Telford pavements; and concrete pavements.

5.2 CBR-based Methods

The CBR-based methods rely on the determination of the laboratory CBR values of the subgrade and/or that of the other pavement layers. The CBR can be used directly or it is converted to a resilient modulus as is the case of the AASHTO method, or converted into a material coefficient, or converted into a foundation number, for determination of required pavement thickness and materials.

5.2.1 The AASHTO Design Method

5.2.1.1 Structural Number

The concept of structural number was first introduced as a result of the AASHO Road Test as a measure of overall pavement strength. It is essentially a measure of the total thickness of the road pavement with each layer weighted by a factor (usually referred to as the 'a' coefficient) for each material (Table 5-1) depending on its strength as follows:

$$SN = 0.0394 \sum a_i.h_i.m_i \qquad \text{Equation 5-1}$$

where:

SN = structural number of the pavement,

a_i = strength coefficient of the i^{th} layer,

h_i = thickness of the i^{th} layer, in millimetres,

m_i = drainage coefficient for layer 'i' for calibrating for different environmental conditions.

and the summation is over the number of pavement layers, n.

The individual layer strength coefficients (Table 5-1) are determined from the normal tests that are used to define the strength of the material in question e.g.

1) CBR for unbound granular materials,
2) Unconfined Compression Strength (UCS) for lightly cemented materials, and
3) Stability for bitumen bound materials.

The coefficients are modified using the 'm' coefficients to take into account the deterioration or weakening of the materials caused by environmental effects, for example, high moisture contents in unbound materials caused by poor drainage and high temperature conditions affecting bituminous materials. These modification or drainage factors (denoted by 'm') are essentially calibration factors for more or less severe conditions of climate. They were introduced to replace the regional overall calibration factor R which was used in earlier versions of the design method.

The AASHTO equation is:

$$\log_{10}(W_{18}) = Z_R \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

Where:

SN = Structural number required inches.

W_{18} = Accumulated 18-kip ESAL over the life of the project.

Z_R = Standard normal deviate.

M_R = Resilient modulus psi

S_O = Standard deviation.

ΔPSI = Change in serviceability.

Table 5-1: Pavement Layer Strength Coefficients

<table>
<tr><th>Layer</th><th>Layer Type</th><th>Condition</th><th>Coefficient</th></tr>
<tr><td rowspan="5">Surfacing</td><td>Surface dressing</td><td></td><td>$a_i = 0.1$</td></tr>
<tr><td rowspan="4">New asphalt concrete [1,2] wearing</td><td>MR_{30} = 1500 MPa</td><td>$a_i = 0.30$</td></tr>
<tr><td>MR_{30} = 2000 MPa</td><td>$a_i = 0.35$</td></tr>
<tr><td>MR_{30} = 2500 MPa</td><td>$a_i = 0.40$</td></tr>
<tr><td>$MR_{30} \geq$ 3000 MPa</td><td>$a_i = 0.45$</td></tr>
<tr><td rowspan="15">Roadbase</td><td>Asphalt concrete</td><td>As above</td><td>As above</td></tr>
<tr><td rowspan="8">Granular unbound</td><td>Default</td><td>$a_i = (29.14\ CBR - 0.1977\ CBR^2 + 0.00045\ CBR^3)\ 10^{-4}$</td></tr>
<tr><td>GB 1 (CBR > 100%)</td><td>0.145</td></tr>
<tr><td>GB 2 (CBR = 100%)</td><td>0.14</td></tr>
<tr><td>GB 3 (CBR = 80%)
With a stabilised layer underneath
With an unbound granular layer underneath</td><td>
0.135
0.13</td></tr>
<tr><td>GB 4 (CBR = 65%)</td><td>0.12</td></tr>
<tr><td>GB 5 (CBR = 55%)</td><td>0.107</td></tr>
<tr><td>GB 6 (CBR = 45%)</td><td>0.1</td></tr>
<tr><td rowspan="3">Bitumen treated gravels and sands</td><td>Marshall stability = 2.5 MN</td><td>a = 0.135</td></tr>
<tr><td>Marshall stability = 5.0 MN</td><td>a = 0.185</td></tr>
<tr><td>Marshall stability = 7.5 MN</td><td>a = 0.23</td></tr>
<tr><td rowspan="3">Cemented[3]</td><td>Equation</td><td>$a_i = 0.0238.UCS + 0.0895$</td></tr>
<tr><td>CB 1 (UCS = 3.0 – 6.0 MPa)</td><td>a = 0.18</td></tr>
<tr><td>CB 2 (UCS = 1.5 – 3.0 MPa)</td><td>a = 0.13</td></tr>
<tr><td rowspan="4">Sub-base</td><td rowspan="3">Granular unbound</td><td>Equation</td><td>$a_i = 0.0174+0.0272*\log_{10}CBR$</td></tr>
<tr><td>GS (CBR = 30%)</td><td>a = 0.10</td></tr>
<tr><td>GC (CBR = 15%)</td><td>a = 0.08</td></tr>
<tr><td>Cemented</td><td>CB 3 (UCS = 0.7 – 1.5 MPa)</td><td>a = 0.1</td></tr>
</table>

Notes:

1. Unconfined Compressive Strength (UCS) is quoted in MPa at 14 days.
2. MR_{30} is the resilient modulus by the indirect tensile test at 30 °C.

The design equation relates the Total Cumulative Equivalent Number of Standard Axles in the heavily trafficked direction to the following input variables:

1) **Subgrade Modulus.** The modulus of the subgrade (usually estimated from the CBR) was expressed as the value at the weakest condition but to cope with large variations throughout the year a weighted value was used to combine the weakest condition in each month of the year in a statistically correct manner. Recent advances in the use of Falling Weight Deflectometers (FWDs) and Light Falling Weight Deflectometers (LWDs), and their increasing availability have allowed the modulus of the subgrade layer to be estimated fairly accurately.

2) **Overall Standard Deviation (S_0) of the performance data from the Road Test and a Reliability Factor (R).** The design equation includes a term that accounts for the variability in performance of similar pavements in the original Road Test. This then allows designs to be produced for different levels of probability (e.g. typically 90%, 95% or 98%) of reaching the design life.

3) **Allowable decrease in surface condition (ΔPSI).** The terminal condition at which a road is deemed to require major structural repair or rehabilitation can be selected. Thus, the same structural design can have more than one design life depending on how much deterioration can be tolerated.

4) **Structural Number SN and the associated strength and drainage coefficients for each pavement layer.** The design can be modified by changing the strength of the pavement layers using the 'drainage' factors m_i (Equation 5.1) for each layer to adjust for good or poor drainage or climate conditions.

Thus, it can be seen that the method can be calibrated for different conditions of climate, terminal condition and reliability using the results of empirical studies in the locations required. If a design engineer wishes to use a version of the AASHTO method, it is important to determine what assumptions have been made to calibrate the method. The reliability of the method therefore depends on the skill of the authors of particular versions to carry out good research and to calibrate it in an accurate and scientific manner.

The design equation is very non-linear and contains five variables, hence to determine the required SN for a particular traffic level the equation has to be used iteratively.

Although the range of pavement structures and the traffic loadings used were not aimed at low volume roads, sections of roads surfaced with a thin bituminous seal were included and many road authorities in South East Asia use the AASHTO design method (for example Table 5-2). The various methods will be different because the calibration for local use will differ from the original and from each other, but they are all classified as CBR-based methods.

Table 5-2: Design factors (Vietnam example using the AASHTO Method)

AASHTO Factor	Value
Design Period (Years)	15
Traffic (Cumulative Mesa)	7.2
Overall Reliability (R%)	85
Overall Standard Deviation (S_0)	0.45
Change in Terminal Serviceability (ΔPSI)	2.2
Subgrade Modulus (MPa)	50
Drainage Quality (Roadbase) (m_2)	1.0
Drainage Quality (Sub-base(m_3)	1.0
Layer Strength Coefficients Asphalt Concrete (a_1)	0.37
Roadbase (a_2)	0.135
Sub-base (a_3)	0.11

Roadbase materials are generally quite strong hence strength measurements in a dry state are not very sensitive. To increase sensitivity to distinguish different strength materials it is traditional to test the materials using the soaked CBR test. It was not assumed that the materials would be soaked when used in a road.

It is also important to note that the AASHTO design equation indicates that the traffic carrying capacity is proportional to a high power of SN (about 9.4). Thus, a small increase in SN provides a large increase in traffic carrying capacity. In subsequent studies on LVRs a similar sensitive result has not been obtained (a power between 5.5 and 7.5 is much more common (e.g. see HDM 4 relationships). Such an issue would not be resolved by calibration. A 'calibrated' AASHTO equation will overstate the carrying capacity of a road if it is used for roads with high SNs.

5.2.1.2 Calibration

The AASHTO design method is unique in that it provides several methods of catering for risk which, in some ways, makes it more complicated. First of all, the assessment of subgrade strength was critical at the Road Test because the area of the USA where the experiments were carried out underwent a freeze/thaw cycle every year and most of the deterioration of the road took place during the thawing conditions when the subgrade was extremely weak. This so dominated performance that it had to be factored into the method and this was accomplished by defining the subgrade strength for design as a correctly weighted soaked value applied for each month of the year. This method is only required under similar freeze thaw conditions and is not necessary in most areas where this Rural Road Note is applicable. Nevertheless, the design method requires a soaked subgrade strength assessment.

In addition, the method also requires calibrating for climate using the drainage factors (m). These are important because the original design method produced pavement structures whose performance could range over a factor of more than 5 times in terms of traffic carrying capacity as climate changed from wet to dry. This was in addition to the variability observed with nominally identical pavements in the same location. This latter variability was of similar range so the difference between an average design and one with a 95% probability of achieving its design life was also large. Unfortunately, both of these variabilities (random and climate induced) represent the reality of the pavement design problem but are not quite as severe where freeze thaw conditions do not occur. Nonetheless, the problem of calibrating the AASHTO method requires local road performance data and competent data analysts. Using the method without calibration is not recommended.

5.2.1.3 Modified Structural Number

The AASHO Road Test was constructed on a single subgrade, therefore the effect of different subgrades could not be estimated and the structural number could not include a subgrade contribution. To overcome this problem and to extend the concept to all subgrades, a subgrade contribution was derived as described by Hodges et al. (1975) and a modified structural number defined as follows:

$$SNC = SN + SNG = SN+3.51\,(\log_{10} CBR_s) - 0.85\,(\log_{10} CBR_s)^2 - 1.43$$

where:

SNC = Modified structural number of the pavement

SNG = Subgrade contribution

CBR = in-situ CBR of the subgrade

The modified structural number (SNC) has been used extensively and forms the basis for defining pavement strength in many pavement performance models.

5.2.1.4 Adjusted Structural Number (SNP)

When evaluating a pavement in order to design rehabilitation measures, it is found that many pavements cannot be divided easily into distinct roadbase and sub-base layers with a well-defined and uniform subgrade. Hence, when calculating the structural number according to the equation above, the engineer must judge which layers to define as roadbase, which as sub-base, and where to define the top of the subgrade. For many roads this has proven quite difficult. There are often several layers that could be considered either as sub-bases or part of the subgrade, especially where capping layers or selected fill have been used. The simple summation over all the apparent layers allows the engineer to obtain almost any value of structural number since the value will depend on where the engineer assumes that the sub-base(s) end and the subgrade begins. In the past this problem has been addressed by simply limiting the total depth of all the layers that are considered to be road pavement. However, this is somewhat arbitrary, has not been used universally, and has led to large errors in some circumstances.

The problem arises because the contributions of each layer to the structural number are independent of depth. This cannot be correct since logic dictates that a layer that lies very deep within the subgrade can have little or no influence on the performance of the road. To eliminate the problem, a method of calculating the modified structural number has been devised in which the contributions of each layer to the overall structural number decrease with depth (Rolt and Parkman, 2000).

To distinguish the structural number derived from the original Modified Structural Number (SNC), the new structural number is called the Adjusted Structural Number (SNP). The equation for calculating it takes into account the strength, thickness and depth of all layers below the sub-base. It is calculated as follows:

SNP = SNA + SNS + SNG

Where:

SNP is the Adjusted Structural Number;

SNA is the Structural Number contribution of the Base and Surfacing

SNS is the Structural Number contribution of the Sub-base

SNG is the Structural Number contribution of the Subgrade

The equations are presented in the reference and contained within the TRL program for computing DCP results from a DCP test.

5.2.2 The Overseas Road Note 31 Method

The original design charts In Overseas Road Note No 31 were based on an assessment of subgrade strength representing the anticipated weakest conditions and three classifications were provided to assist with this.

Category 1. Subgrades where the water table is sufficiently close to the ground surface to control the subgrade moisture content. The type of subgrade soil governs the depth below the road surface at which a water table becomes the dominant influence on the subgrade moisture content. For example, in non-plastic soils the water table will dominate the subgrade moisture content when it rises to within 1m of the road surface, in sandy clays (PI<20 per cent) the water table will dominate when it rises to within 3m of the road surface, and in heavy clays (PI>40 per cent) the water table will dominate when it rises to within 7m of the road surface.

Category 2: Subgrades where the water table is deep, but rainfall can influence the subgrade moisture content under the road. These conditions occur when rainfall exceeds evaporation and transpiration for at least two months of the year

Category 3: Subgrades where the water table is deep and the climate is arid. These conditions occur where the climate is dry throughout most of the year with annual rainfall of 250 mm or less.

These categories provided a relatively simple method of assessing the subgrade strength for design. Only in category 1 is the design CBR moisture content likely to approach a soaked condition. The value depended on the minimum depth of the water table and the properties of the subgrade soil. So, although based on the likely weakest value of subgrade strength, in many of the drier areas the weakest was unlikely to be equivalent to a soaked condition

The CBR design procedure for new roads is straightforward because it is based on assessing the subgrade strength for design, the new traffic loading and the design life and the general climate and moisture conditions and then simply looking up the appropriate design based on these four parameters. However, it transpired that many designers, presumably because of their perception of risk over the life of the road, generally used the soaked CBR at the compacted density thereby designing in a conservative manner and not as recommended in the ORN.

The lowest traffic class in ORN 31 is < 0.3 MESA whereas other methods provide two or three traffic classes below 0.3 MESA. Therefore, the method could be conservative for lower traffic roads in some situations where materials of lower classes that are adequate for example 0.1 MESA, are readily available.

5.2.3 *The TRL-SADC Method*

5.2.3.1 Description

This method is commonly referred to as the Gourley – Greening method. After extensive studies conducted in Zimbabwe, Malawi and Botswana in 1999 by TRL (C. S. Gourley and P. A. K. Greening) new design charts for sealed LVRs were developed. These were based on identifying the type of subgrade for each road by using a soaked subgrade CBR strength criterion at the anticipated density. The principle adopted is that if two roads under the same climate and drainage conditions and the same structure have the same soaked subgrade CBR then they will usually perform in a very similar way. Thus, the structure of the roads that performed well in the experimental studies form the basis of the design charts and it is only necessary to copy the structure of such a road with the same soaked subgrade CBR from the design charts provided that the climate and drainage conditions are the same. Thus, it is not necessary to estimate the likely strength of the subgrade (e.g. either the worst case subgrade strength or the long term equilibrium strength). However, it is very important to note that this does not assume that the subgrades are likely to be soaked in practice. The soaked test simply classifies the subgrades as likely to be similar in performance to those that had performed well in the empirical studies under the same conditions. It is recognised that this is not guaranteed but the study itself obtained a great deal of in-situ data concerning the moisture conditions in the roads over an extended time period and the principle has been verified.

For roads with non-structural bituminous surfacings, two climatic zones are defined and two design catalogues (charts), Table 5-3 and Table 5-4, are used as shown below. The second design chart is provided for conditions in which the climate and/or drainage conditions are inferior and under these conditions the recommended structures are stronger. The subgrade strength of uniform road sections for design are determined as follows:

- For design traffic less than 0.3 MESA, the 50th percentile subgrade strength;
- For design traffic between 0.3 and 0.5 MESA, the 25th percentile subgrade strength;
- For design traffic great than 0.5 MESA, the 10th percentile subgrade strength.

The standard soaked CBR test is also used to evaluate the strength of the imported pavement materials but the design thicknesses developed from the study clearly showed that specifications for more heavily trafficked roads could be reduced for low volume roads whilst still attaining the same standard of service. The design charts and specifications reflect this.

These charts introduced more traffic classes below 1 MESA than were available in ORN 31. Figure 5-1 shows the decision flow chart for selecting the appropriate design chart. The design catalogues therefore show different thickness designs based on the climate and drainage conditions for the same indexed subgrade class.

5.2.3.2 Selecting the Design Chart

The required pavement structure appropriate to the climate, cross-section, traffic and subgrade strength is identified from Figure 5-1 and Table 5-3 and Table 5-4. Climatic zones are classified based on the Weinert Number (N). Wet climatic zones are defined by N<4 and dry zones are defined as N>4.

$N = 12*E/P_a$ where E is evaporation in mm in the warmest month of the year and P_a is the annual precipitation in mm.

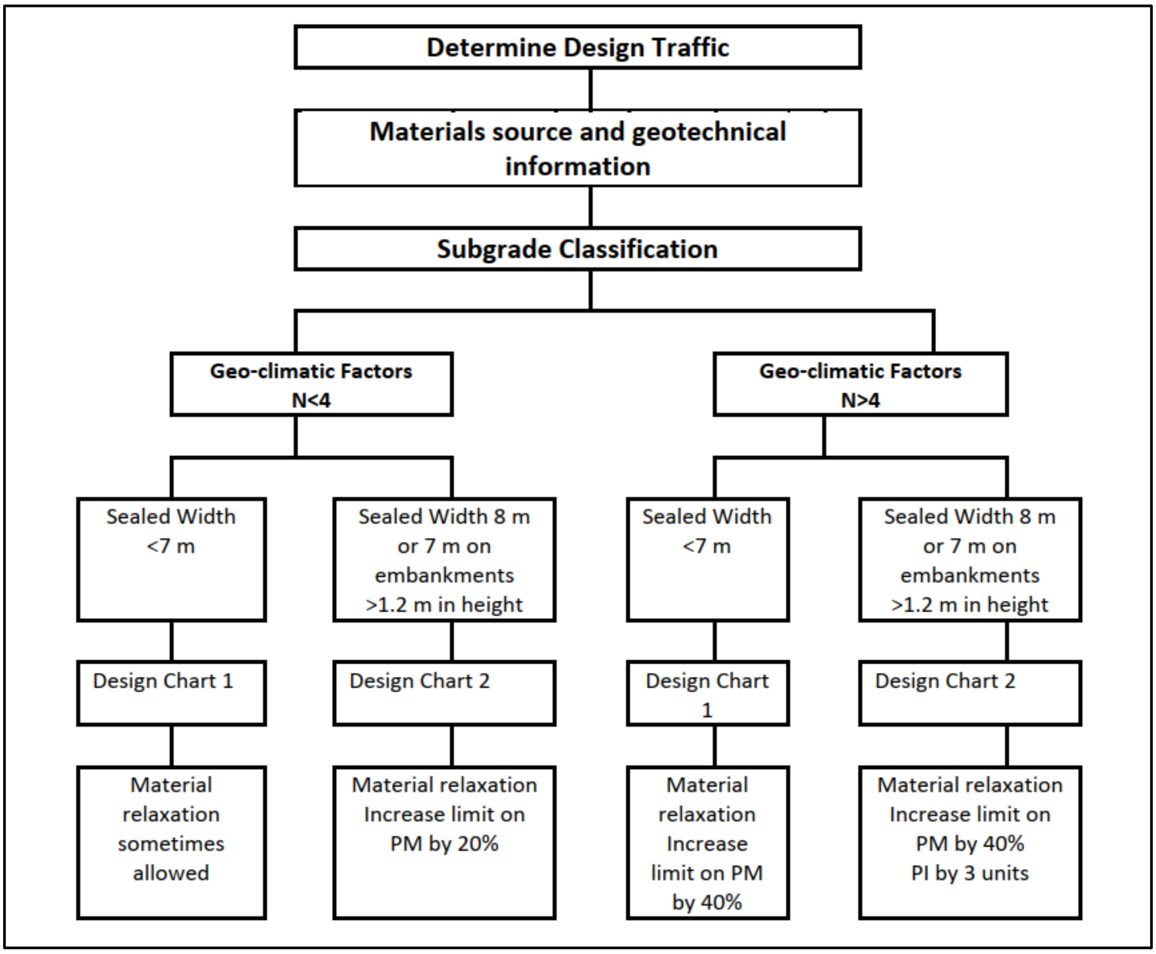


Figure 5-1: Pavement design flow chart

(A) Wet climatic zone:

In the wet climatic zone, the following situations and solutions apply:

a) Where the total sealed surface width is less than 7 m, Pavement Design Chart 1 (Table 5-4) should be used. No adjustments to the base material requirements are required.

b) Where the total sealed surface width is 8 m or more: Pavement Design Chart 2 (Table 5-4) should be used. The limit on the plasticity modulus of the base may be increased by 20%.

c) Where the total sealed surface is less than 8 m but the pavement is on an embankment in excess of 1.2 metres in height, Pavement Design Chart 2 (Table 5-4) should be used. The limit on the plasticity modulus of the base course may be increased by 20%.

d) If the design engineer deems that other risk factors are high (e.g. poor maintenance and/or moderate construction quality) then Pavement Design Chart 1 should be used (Table 5-3).

***(B)* Moderate and dry climatic zone:**

In a moderate or dry climatic zone, the following solutions apply:

a) Pavement Design Chart 2 (an example is shown in Table 5-4) should be used, however:

b) Where the total sealed surface width is less than 8 m, the limit on the plasticity modulus of the roadbase may be increased by 40%.

c) 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 roadbase may be increased by up to 40% and the plasticity index increased by 3 units.

Once the quality of the available materials and haul distances are known, the design charts can be used to review the most economical designs.

The choice of design chart separates the designs into essentially two risk categories depending on climate and drainage provision but a third option namely that if there was any risk perceived that the subgrade of a road was likely to become saturated, the option is simply to design on the next lower strength subgrade (preferred) or the next higher traffic category. The relationship between structural number and traffic carrying capacity indicates that this is a perfectly acceptable approach and is indeed why the subgrade strength ranges for design were selected initially.

Unfortunately, it has also proved difficult to prevent heavy vehicles, often overloaded, from using LVRRs. Although the cumulative damage caused by heavy axles is taken into account in the traffic classes defined in the design charts and based on the equivalent standard axle concept, on LVRRs overloaded vehicles with high tyre pressures pose a risk especially if the strength of the roadbase layers is in the lower part of the acceptable range.

The use of the charts is described below. 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. If the road is expected to carry any unusually heavy loads, for example from industries such as sawmills, mines, and the like, it may be prudent to adjust the design class upwards, to reduce risks.

The design charts do not cater for very weak subgrades (CBR < 3%) and other problem soils, which need specialist input and design. Treatment of problem soils are discussed in most country manuals and is outlined briefly in Chapter 9 (Ancillary Factors and Considerations) of this guide, typically requiring imported better-quality selected subgrade materials.

Table 5-3: An Example of Chart- Bituminous pavement design Chart 1 (Wet Climate Zones)

Subgrade Class (CBR)	**Design Traffic Classes in MESA**				
	LVR 1	**LVR 2**	**LVR 3**	**LVR 4**	**LVR 5**
	<0.01	**0.01 – 0.1**	**0.1 – 0.3**	**0.3 – 0.5**	**0.5 – 1.0**
S2 (3-4%)	125 G45 125 G30 100 G15	175 G45 150 G30 175 G15	175 G60 150 G30 200 G15	175 G60 175 G30 200 G15	200 G60 175 G30 200 G15
S3 (5 – 7%)	125 G45 100 G30 100 G15	150 G45 150 G30 125 G15	175 G60 150 G30 150 G15	175 G60 150 G30 175 G15	200 G60 150 G30 150 G15
S4 (8-14%)	125 G45 125 G30	175 G45 175 G30	150 G60 200 G30	175 G60 200 G30	200 G60 200 G30
S5 (15 – 29%)	200 G45	150 G45 125 G30	150 G60 125 G30	150 G60 150 G30	175 G60 150 G30
S6 (>30%)	175 G45	200 G45	200 G60	200 G60	200 G60

In a seasonal tropical and wet climate zone it may be more economical to use a wider cross-section and then use Design Chart 2 rather than to design a narrow cross-section and a pavement using Pavement Design Chart 1.

Table 5-4: An Example of Chart - Bituminous pavement design Chart 2 (Moderate Climate Zones)

Subgrade Class (CBR)	Design Traffic Classes in MESA				
	LVR 1	LVR 2	LVR 3	LVR 4	LVR 5
	<0.01	0.01 – 0.1	0.1 – 0.3	0.3 – 0.5	0.5 – 1.0
S2 (3-4%)	150 G45 175 G15	150 G45 125 G30 150 G15	150 G60 150 G30 150 G15	175 G60 150 G30 150 G15	175 G60 175 G30 175 G15
S3 (5 – 7%)	150 G45 150 G15	150 G45 150 G30	175 G60 175 G30	175 G65 200 G30	175 G60 125 G30 150 G15
S4 (8-14%)	125 G45 125 G30	150 G45 125 G30	150 G60 150 G30	175 G60 150 G30	175 G60 175 G30
S5 (15 – 29%)	100 G45 125 G30	150 G45 100 G30	150 G60 100 G30	150 G60 125 G30	175 G60 125 G30
S6 (>30%)	175 G45	200 G45	175 G60	200 G60	200 G60

Chapter 3 and 6 describe the material types in detail for the various pavement layers. The roadbases can be either unbound granular material or a stabilised material. Generally, sub-bases are unbound and comprise G30 or G25 materials. Both are suitable but G30 is preferred.

It is these design charts that comprise the basic CBR-Structural Number (CBR-SN) design method described in recent LVR manuals. They are now considered to be a little conservative because material specifications were not refined quite as much as current research is beginning to show may be possible. The specifications could probably be reduced more without sacrificing performance as shown in the ongoing ReCAP project '*Development of guidelines and specifications for low volume sealed roads through back analysis*' also known as the Back Analysis project.

5.2.4 *The Foundation Class Method*

An important modification to the basic CBR method is the method based on defining Foundation Classes. The results are usually presented as a catalogue of structures in the normal way, but the original pavement design is carried out in two stages. First a foundation is designed to carry construction traffic and act as a construction platform. This is similar in principle to the capping and sub-base layers used in the basic CBR-based methods. There are typically four or five foundation designs to cover the range of subgrade strengths, but the same foundation layer designs are used for all pavement structures. The foundation classes are sometimes expressed as moduli that can be verified by the use of Light Falling Weight Deflectometers (LWDs), Falling Weight Deflectometers (FWDs), or through bearing plate tests. With the increasing availability of LWDs and FWDs, the verification of the foundation class achieved is easily done before addition or adjustment of the upper pavement layers.

The upper layers of each pavement are then designed based on the structural type of the pavement selected, the traffic level and the appropriate foundation design based on the subgrade strength. Thus, the resulting catalogue of structures for the different structures based on the different materials look very similar to those using the basic CBR-based methods described in earlier sections of this document, except that the foundation layers are identical. An example of the chart is shown in Figure 5-2.

An example of this method is the Kenyan Pavement Design Guideline for Low Volume Sealed Roads (2017) which contains catalogue designs for 17 different pavement structures on four different foundations and five traffic classes up to 1 million ESA.

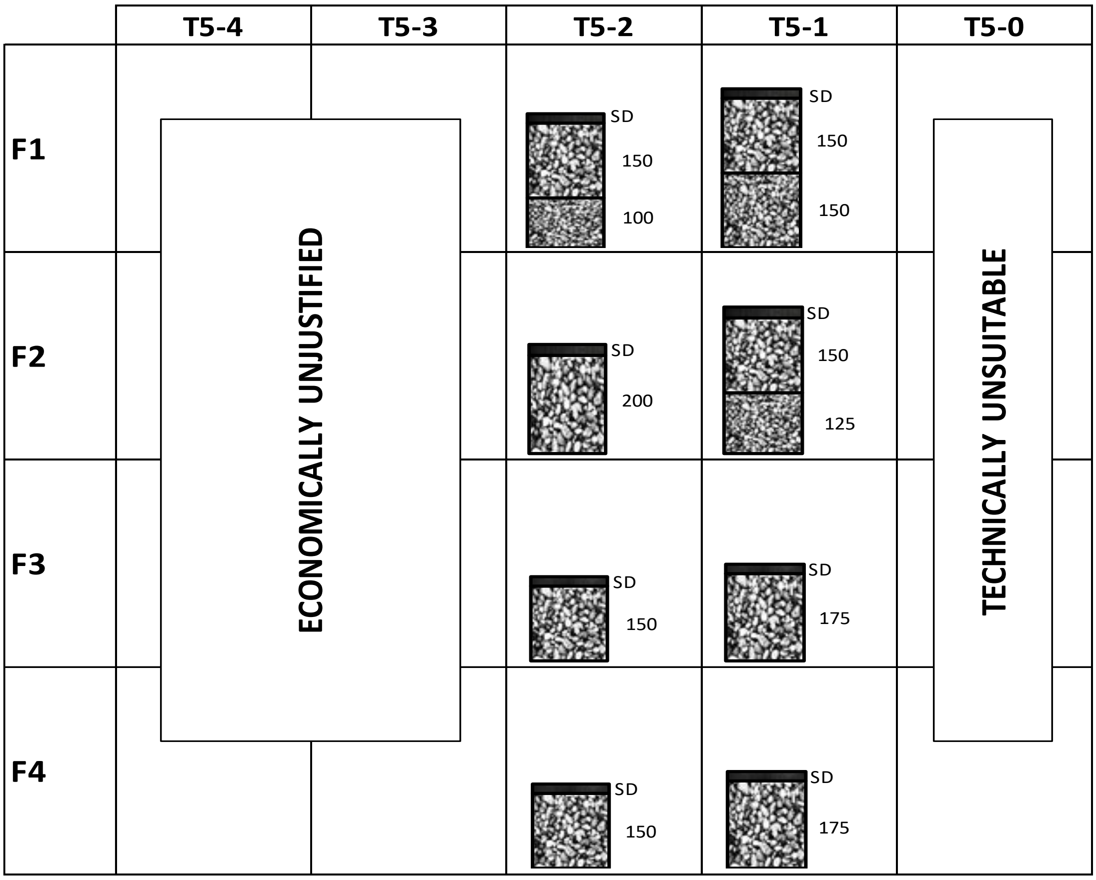
The variability of strength in a road pavement is largely caused by the variability in the strength of the subgrade. Thus, one of the advantages of this method of design is that after constructing the foundation it can be quickly and easily tested comprehensively for uniformity, for example by carrying out deflection tests and, if weak areas are identified, corrections or remedies can be applied. Furthermore, the foundations can be more easily designed with a high level of reliability to provide a basis for a road structure with a long life. Finally, the fact that the foundation designs are common to all structures has important practical implications. The method quickly becomes familiar to both contractors and consultant and this has spin-offs in better quality control.

MINISTRY OF TRANSPORT, INFRASTRUCTURE, HOUSING AND URBAN DEVELOPMENT

STANDARD PAVEMENT STRUCTURES FOR LOW VOLUME ROADS

STANDARD PAVEMENT STRUCTURE TYPE **LV2**

BASE : Gravel Material of Minimum CBR of 50% (G50)
SUBBASE : Gravel Material of Minimum CBR of 25% (G25)



TRAFFIC CLASSIFICATION

Traffic Class	Cumulative Equivalent Standard Axles
T5-4	<25,000
T5-3	25,000 - 100,000
T5-2	100,000 – 250,000
T5-1	250,000 – 500,000
T5-0	500,000 – 1 million

SUBGRADE CLASSIFICATION

Subgrade Class	CBR Range (%)
S1	2 – 5
S2	5 – 10
S3	7 – 13
S4	10 – 18
S5	15 – 30
S6	>30

FOUNDATION CLASSIFICATION

Foundation Design Class	Effective Modulus (MPa)	Equivalent Subgrade Class
F1	65	S3
F2	90	S4
F3	125	S5
F4	250	S6
F5	400	-

Foundations and Improved Subgrade Design

Native Subgrade Class		S1					S2				S3			S4		S5
Capping	Upper Layer	G8	G10	G10	G15	G15	G10	G15	G25	G30	G15	G25	G30	G25	G30	G30
	Thickness (mm)	300	275	350	150	150	100	175	200	225	100	150	175	100	125	100
	Lower Layer				G8	G10										
	Thickness (mm)				200	175										
Improved Subgrade Class		S2	S2	S3	S3	S3	S3	S4	S5	S6	S4	S5	S6	S5	S6	S6

Source: MoTIHUD Kenya (2017).

Figure 5-2: Example of Foundation Class Method chart from Kenya Pavement Design Guideline

5.3 DCP-based Methods

DCP-based methods require the use of the DCP in-situ as the primary means of assessing the existing in-situ strength of an existing substrate. The DCP rate of penetration is either converted to an equivalent CBR or used directly to determine the thickness and strength of the other required pavement layers. Two major advantages of the DCP instrument are that it is very useful where laboratory facilities are limited, and several in-situ measurements can be made rapidly.

Three limitations that apply to both DCP-CBR and DCP-DN stem from the use of the DCP instrument. These limitations are:

1. When used to evaluate the strength of pavement layers containing very coarse particles (retained on the 37.5 mm sieve), large variability in the DN or equivalent CBR values occur.
2. The DCP instrument cannot be used for initial strength assessment where the geometric design of the proposed new road will result in significant changes in vertical alignment resulting in deep cuts and fills (exceeding 1.5 m), such as in undulating terrain. This is because the length of a DCP rod (including extension) cannot penetrate beyond 1.5 m. Thus, before embarking on expensive surveys, the designer should establish whether the road will have significant changes in the vertical alignment. If deep cuts and fills are anticipated, some subsurface investigation (drilling) may be needed.
3. DCP readings are significantly affected by slight variations in moisture content, particle size distribution, or density the material being assessed. Thus in-situ strength measurements can be easily misinterpreted, for example, improving drainage may be all that is required instead of addition of a new layer.

5.3.1 The TRL-ORN 18 DCP-CBR Method

5.3.1.1 Design Steps

A final modification of the TRL-ORN 18 method (sometimes referred to as DCP-CBR method) is to make more use of the DCP. This method is based on the CBR test, but, because of its many advantages, the designer can normally make extensive use of a DCP to obtain much of the required design information, particularly a longitudinal profile of in situ strengths of the pavement layers of the existing road in terms of DN values (penetration per blow in mm/blow). The method requires the conversion of DN to in-situ CBR. There are several published in the literature but the TRL relationship is recommended because it is based on in-situ CBR and DCP test values rather than on tests carried out in moulds in the laboratory.

$$\mathrm{Log}_{10}(\mathrm{CBR}) = 2.48 - 1.057 * \mathrm{Log}_{10}(\mathrm{DN})$$

The in-situ CBR is then converted into soaked values. Finally, the soaked CBR values are converted into layer strength coefficients in the normal way. The overall procedure is essentially the same as for the CBR-SN method except for this conversion step from DN to CBR.

The use of the DCP allows many measurements of the in-situ material strength to be determined and thereby provides a very important set of material strength data for a statistically reliable design to be produced. Allowance must be made for the likely long-term material strength under the completed road, but this adjustment needs to be made for all design methods. There is no substitute for the number of measurements available from the use of the DCP. Some CBR testing is desirable for correlation to the DCP test values.

Like other methods, this design approach and method of selection of pavement materials is also evaluated by classification tests such as Particle Size Distribution, Atterberg limits and determination of MDD/OMC. Again, as is the case with the CBR method, determination of material strength is based on the standard soaked CBR test to specify the minimum strength of pavement materials and evaluating the strength of the imported pavement materials. The plasticity requirement is the same as the CBR method and the relationship between soaked and in situ strength (CBR) depends on the characteristics of the materials.

The CBR-SN-DCP method is mostly applicable for upgrading existing roads having one or more structural layers. To make optimum use of the existing layers, the method makes use of the SN concept which is based on the in-situ CBRs derived from the DN values. The difference between SN of the existing road and that required for the upgraded road, which is obtained from the catalogue of structures, defines the additional requirements for upgrading, rehabilitation or reconstruction. For a new road where the alignment and depth of subgrade may change during construction care is required to obtain the correct subgrade strength values for the design but the method is somewhat simpler because there are no existing structural pavement layers. The structural designs are obtained directly from the catalogue of structures.

Step 1: Select design period. Determine the TLC. This is based on standard vehicle counting and axle load surveys and is identical in all three design methods.

Step 2: Undertake a DCP survey and collect bulk samples for moisture and strength measurements

Step 3: For each test point determine the layer strength profile (LSP). The LSP shows the DN in mm/blow and the CBR versus depth, and the layer boundaries. A typical output from the UK DCP analysis software is shown in Figure 5-3. The software can be obtained freely by emailing enquiries@trl.co.uk.

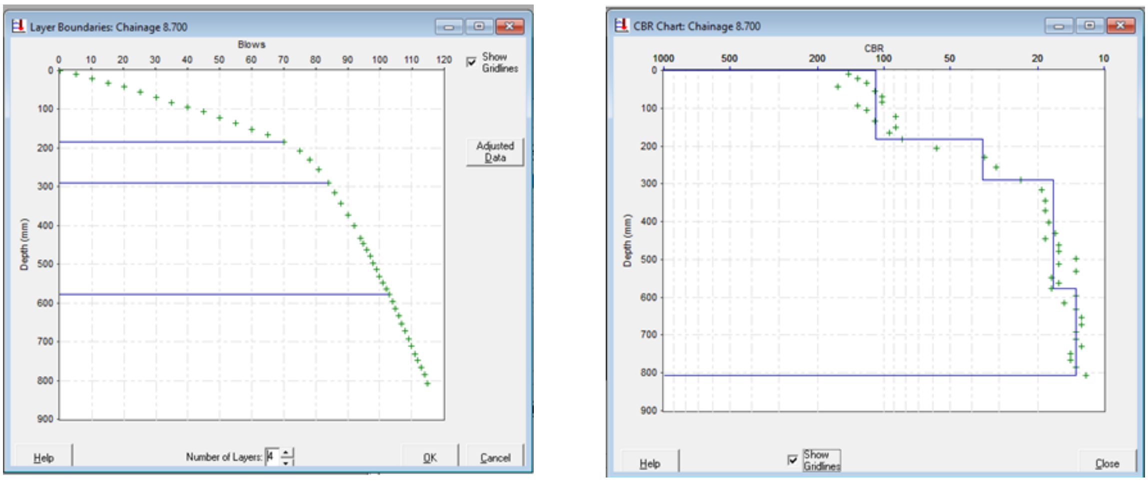
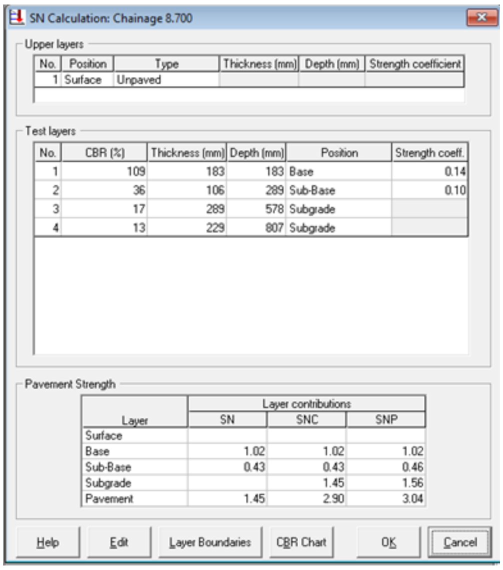


Figure 5-3: Typical Layer-Strength diagrams computed in the UK DCP Program

Step 4: Determine the in-situ SN values for each layer and the total SN (and SNC) for the pavement for each test point.

To determine the total in-situ SN and SNC, the layers must first be defined as base, sub-base or subgrade as shown in Figure 5-4.



SN Calculation: Chainage 8.700

Upper layers

No.	Position	Type	Thickness (mm)	Depth (mm)	Strength coefficient
1	Surface	Unpaved			

Test layers

No.	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength coeff.
1	109	183	183	Base	0.14
2	36	106	289	Sub-Base	0.10
3	17	289	578	Subgrade	
4	13	229	807	Subgrade	

Pavement Strength

Layer	Layer contributions		
	SN	SNC	SNP
Surface			
Base	1.02	1.02	1.02
Sub-Base	0.43	0.43	0.46
Subgrade		1.45	1.56
Pavement	1.45	2.90	3.04

Figure 5-4: Calculation of SN/SNC in the UK DCP Program

Step 5: Uniform sections are determined based on a CuSums analysis of the SN values for each test point.

Step 6: The design subgrade class requires the soaked subgrade CBR values (as explained in Section 5.5) rather than the in-situ values. To convert from the in-situ values to the soaked values requires a measurement of the in-situ moisture condition, expressed as the ratio of in-situ moisture content divided by the optimum moisture content, and the use of Figure 5-5. The relationship between soaked and in situ strength (CBR) depends on the characteristics of the materials. However, for the level of accuracy required, Figure 5-5, is adequate. The in-situ moisture condition is obtained from the samples collected for laboratory analysis after determination of uniform sections. A minimum of three samples per uniform section is recommended. It is often more useful to obtain the samples once the DCP survey has been analysed and the most appropriate sampling points can be identified to ensure that maximum benefit is obtained from the sampling and testing. However, the delay between the in-situ testing and sampling must be less than 14 days.

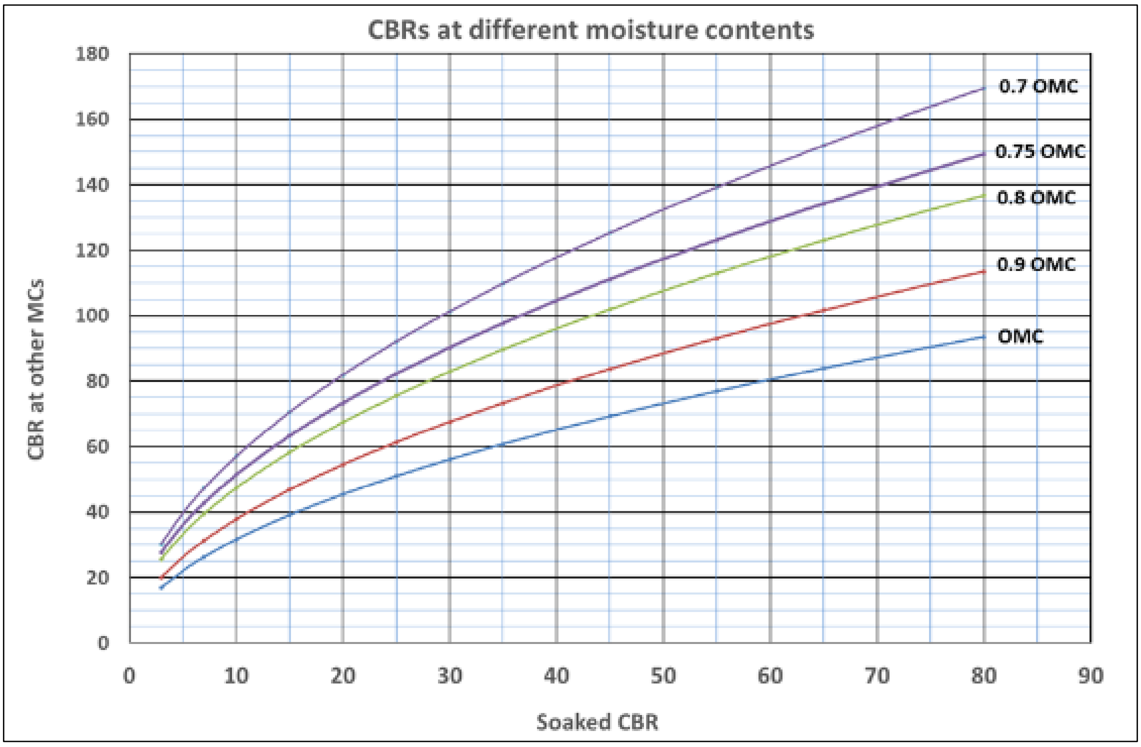


Figure 5-5: Relationship between in-situ DCP-CBR and soaked CBR

Table 5-5: Structural Numbers for pavement design Chart 1 -Wet Areas

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	**0.01 – 0.1**	**0.1 – 0.3**	**0.3 – 0.5**	**0.5 – 1.0**
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	1.71	1.86	2.04	2.13	2.28
S3 (5-7%)	1.51	1.70	1.77	1.83	1.95
S4 (8-14%)	1.35	1.43	1.53	1.59	1.68
S5 (15-29%)	0.86	1.04	1.19	1.29	1.45
S6 (>30%)	0.76	0.8	0.85	0.93	0.96
Note: These values exclude a contribution from the surfacing.					

Table 5-6: Structural Numbers for pavement design Chart 2 - Mod/Dry Areas

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	**0.01-0.1**	**0.1-0.3**	**0.3-0.5**	**0.5-1.0**
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	1.24	1.59	1.78	1.87	2.08
S3 (5-7%)	1.19	1.40	1.50	1.59	1.78
S4 (8-14%)	1.15	1.21	1.28	1.40	1.50
S5 (15-29%)	0.86	1.04	1.09	1.20	1.31
S6 (>30%)	0.76	0.87	0.87	0.96	0.96
Note: these values exclude a contribution from the surfacing.					

Step 7: Compare the SN/SNC with the required SN/SNC from the relevant catalogue to determine the structural deficiency at each test point (Table 5-5 and Table 5-6).

Step 8: Identify areas (a) where the structural deficiency is large and (b) areas where layers are very weak and unlikely to meet the specifications for the layer that they will become in the upgraded design. These points must be investigated separately to identify the likely cause and possible solutions to the problem.

5.3.1.2 Upgrading Requirements

Step 9: Identify revised uniform sections omitting sections to be treated separately (i.e. as identified in Step 8) based on the structural deficiencies (ΔSN or ΔSNC) at each test point using the CuSum method

Subgrade failures are rare and prevented from occurring by making sure that the thickness and/or structural number of the pavement is adequate for the traffic level. This depends on traffic level and subgrade strength.

Step 10: Table 5-5 to Table 5-6 show the target values of SN and SNC for different subgrade conditions and for different traffic levels calculated from the design charts for roads with a thin bituminous surfacing. The difference between the required SN and the existing SN (ΔSN) is the deficiency that needs to be corrected.

It is very important at this stage that the actual distribution of structural deficiencies in terms of ΔSN or ΔSNC (as calculated in Step 7 at each test point) is considered. This is because the relationship between traffic carrying capacity and SN is very non-linear (it is proportional to SN^n where the exponent, n, is typically 6 or 7). Thus, using an average value of ΔSN for the design of the strengthening is a serious mistake because there will be many points (50%) where the ΔSN is more than the average and the traffic carrying capacity of these points will be very low indeed compared with the requirements. This is a vital step but unfortunately is not included in all design methods.

For each uniform section the percentiles in Table 5-7 of the ΔSN or ΔSNC for determination of the strengthening requirements should be used.

Table 5-7: Design Percentile for Traffic Classes

Traffic Load Class	Percentile of ΔSN or SN
TLC 0.01 and TLC 0.1	Median
TLC 0.3	Upper 75th percentile
TLC 0.5 and TLC 1.0	Upper 90th percentile

However, when the strengthening requirements are large it may be more cost effective to carry out some reconstruction and, conversely if they are small, maintenance may be all that is required. Table 5-8 is a guide to the treatments.

Table 5-8: Structural deficiency criteria

Structural deficiency based on appropriate percentiles	Action	Notes
0.2 or negative	Maintain with a surface treatment (e.g. a surface dressing).	After patching as required, granular overlay can be used to correct other road defects.
0.2 – 1.2	New granular layer. The existing layers must be checked for quality (sub-base or base course). The minimum thickness of new base course should be 75 mm.	Some localised remedial works can be expected. A surface treatment is required.
1.2 – 1.8	The existing base course is likely to be only of sub-base quality and should be checked. Additional sub-base and a new base course are required.	Some localised remedial works is needed. A surface treatment is required.
> 1.8	The existing layers are likely to be less than sub-base quality, hence a new sub-base and base course are required. Chemically stabilising existing material should be considered.	Localised remedial treatment and a surface treatment are required.

The materials specifications are as discussed in Chapter 3.

5.3.2 The DCP-DN method

The result of each DCP test is a diagram of the strength of the existing pavement measured as DN values as a function of depth. Under the DCP-DN method the quality of the base material is expressed in terms of its DCP resistance to penetration, i.e. its DN value, at the specified compaction density and expected in service moisture condition.

The DCP-DN method is basically similar in principle to the DCP-CBR method in that the DCP is used to assess in-situ conditions and allows many measurements of the in-situ material strength to be determined, resulting in high reliability of the pavement designs. However, the major difference is that the DCP-DN method is based entirely on using the DCP and does not require the engineer to convert the results to equivalent CBR values. Instead, the DN values rather than the equivalent CBR values are used throughout, and the resulting catalogue is also presented in terms of DN values. The in-situ DN values obtained from a survey of the proposed road are plotted on a chart versus the depth and are compared directly with DN values from the design catalogue. The steps of the method are outlined in the following sections and details of the method can be found in Generic DCP-DN Manual 2020.

5.3.2.1 Steps in the Design Method

The steps in the design method are shown and described in Table 5-9.

Table 5-9: Steps in the process

Step	Action
1	Determine the TLC. This is identical in all methods
2	Carry out a DCP survey and calculate the DSN and DN values for all test points. Essentially the same procedure in any method that uses the DCP
3	These values are required for the determination of uniform sections. After entering all the DCP data in the AfCAP LVR DCP programme, the calculation of the following useful parameters is done automatically. The software is available for free download at http://www.research4cap.org/SitePages/LVRDCPSoftware.aspx a) The weighted average DN of each 150 mm layer down to a depth of 800 mm. This is the standard configuration of the AfCAP LVR DCP software, but the layer thicknesses can be varied if required. b) 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. c) 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 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.
5	This procedure does not identify where the subgrade actually begins but although this might be useful, the DN value down to 800 mm is adequate for LVRs.
6	Uniform sections must then be identified. All methods identify uniform sections using a Cusum technique. Identifying uniform sections, can also be done from within the AfCAP LVR-DCP software.
7	The in-situ moisture contents of all the pavement layers must be measured. Classification and Laboratory DN tests on three bulk samples from each uniform section should be carried out to determine the DN values at the anticipated field density and anticipated long-term moisture content.
8	The method differs from most design methods because the strength of the pavement layers is essentially a design variable (Table 5-12) that is adjusted for traffic, subgrade strength and assumed moisture content (expected long term moisture content of the pavement layers). In the DCP-DN method the specification for the roadbase strengths vary depending on subgrade strength and overall traffic level. In other design methods including the CBR and the CBR-SN methods the required strengths of the roadbase and sub-base and capping layers (if any) are specified as minimum values and these are treated as pass/fail tests rather than as design variables. This is done because failure of unbound roadbases is one of the most common forms of failure of LVRs. This is partly because heavy vehicles with high tyre pressures cannot be prevented from travelling on such roads. Also, the methods of evaluating the strength of the subgrade for design are also rather different.
9	Comparison of the strength-depth diagram with standard designs is carried out and, if weaknesses are identified, a new additional layer is usually specified that essentially moves all other layers down one layer and restores an adequate strength balance. This is no different in principle to adding a layer to meet a new revised overall design thickness or SN value as in both of the TRL ORN 18 DCP methods discussed above.
10	To use the existing gravel/earth road strength that has been developed over the years, the materials in the pavement structure need to be tested for their actual in situ strength, using a DCP. The designer must also determine the subgrade design strength properties by use of the Laboratory DN test. Laboratory strength testing is carried out with a DCP on specimens compacted into CBR moulds in the laboratory. Samples are compacted to a range of densities and tested for DN value at OMC, 0.75xOMC and in the soaked condition as for normal CBR testing except that the DN is measured instead of CBR. The DCP-DN design catalogue is based on the anticipated, long term, in-service moisture condition which is assumed to be OMC or 0.75xOMC but if there is a risk of prolonged moisture ingress into the road pavement, then the pavement design should be based on the soaked or a selected wetter condition. The DN value for any selected in-situ moisture condition can be estimated from a

Step	Action
	relationship between standard soaked DN values and in situ DN at various moisture contents for different material strengths but this should be used only as a guide and testing of the actual materials involved should preferably be carried out. The values from the above relationship can be highly material dependent, especially for moisture sensitive materials and certain other materials such as laterites and calcretes.
11	A typical output from the Laboratory DN testing is shown in Figure 5-6. This provides the designer with information on the moisture and density sensitivity of the materials. A large difference between soaked and OMC DN values means that the material is very moisture sensitive. The gradient of the curves indicates the sensitivity of the DN to density variations, i.e. a ‘flat’ curve means that the material is not very sensitive to density variations.
12	The recommended schedule of tests to be carried out is shown in Table 5-10. The number of tests will depend on the climatic zone and micro-climate for the uniform section. Normally testing at 0.75xOMC may not required since the design in most cases will be based on the OMC strength or on the soaked strength if there is risk of flooding or with unfavourable drainage conditions. Only in dry climate with favourable drainage conditions should basing the design on the 0.75xOMC strength be considered.
13	For each uniform section, the AfCAP LVR DCP programme determines the representative Layer Strength profile (that is the DN value with depth and the required values for the subgrade strength and traffic level of the design at the anticipated long-term moisture content and field density. Figure 5-7 shows a typical output from the AfCAP LVR DCP program for the analysis of a uniform section at the time of the DCP survey: a) Number of blows with depth. b) The pavement balance against the Standard Pavement Balance Curves (SPBC). c) The in-situ layer strength profile against the catalogue strength profile.
14	Comparison of the representative LSP for each uniform section with the required LSP as shown in the DCP-DN design catalogue (Table 5-11) indicates the upgrading requirements by identifying which layers are of inadequate strength and/or thickness.

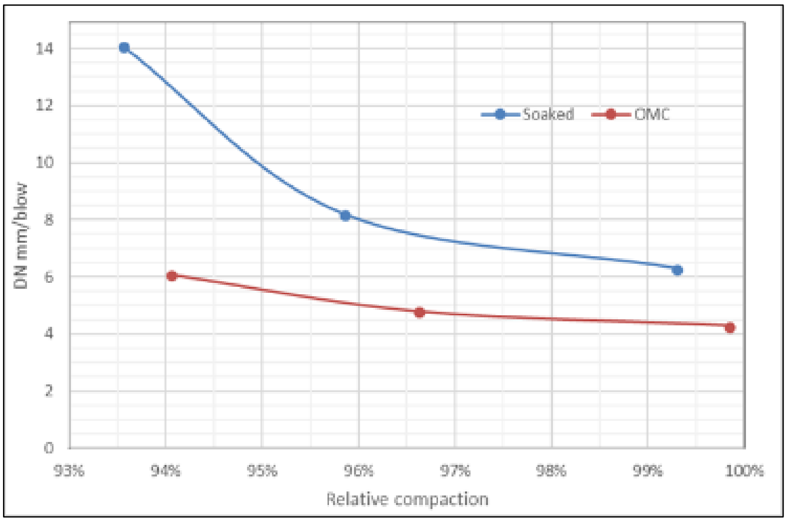


Figure 5-6: Typical output from laboratory DCP tests

Table 5-10: Schedule of laboratory DN tests for determination of subgrade design strength

Climatic zone	Micro-climate	Laboratory DN tests (to be done in triplicate)	
		No @ compactive effort	Sample moisture at testing
All	Risk of flooding, in marshy areas and/or poor drainage condition	2 @ Heavy 2 @ Intermediate 2 @ Light	For each compactive effort: 1 @ 4-day soaked 1 @ OMC
Wet and Moderate Dry	No risk of flooding, reasonable / good drainage conditions	3 @ Heavy 3 @ Intermediate 3 @ Light	For each compactive effort: 1 @ 4-day soaked 1 @ OMC 1 @ 0.75 OMC

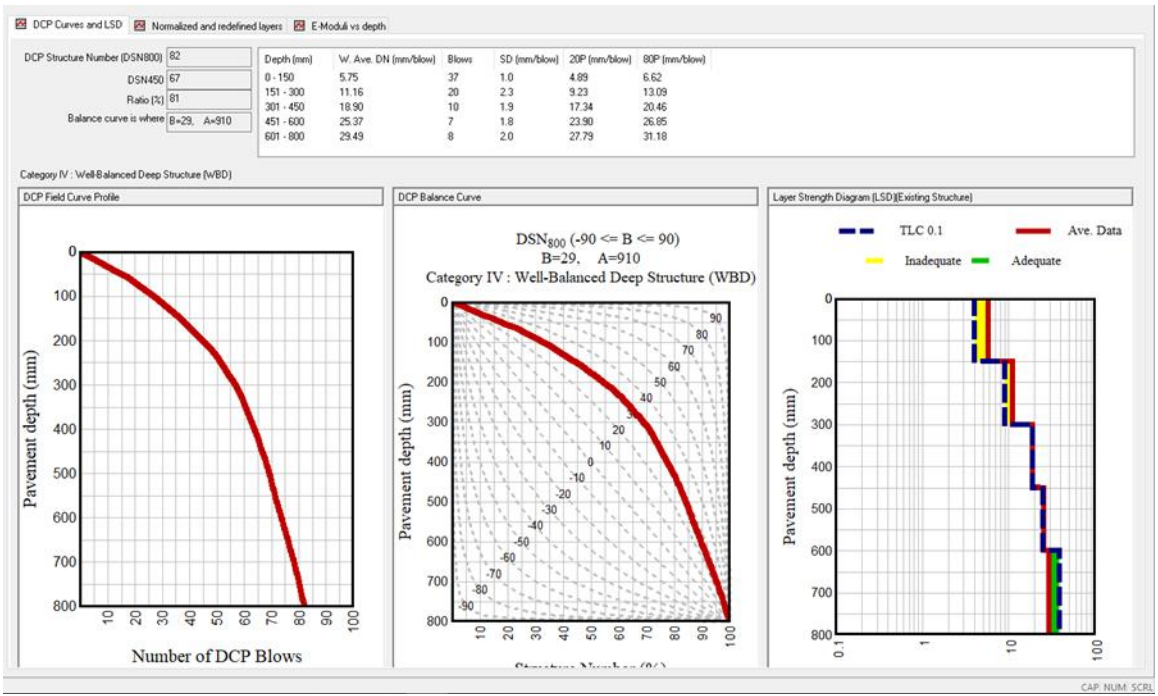


Figure 5-7: Typical output from the analysis of a uniform section

(a) blows versus depth (b)layer balance and (c) catalogue strength profile

5.3.2.2 Determination of Pavement Layer Requirements

The example layer strength profile shown in Figure 5-7 is compared in Table 5-11 with the strength profile for the traffic level required from the design catalogue (Table 5-12). The pink colour indicates the layers with insufficient strength and the green colour indicates layers with sufficient strength compared to the requirement of the design catalogue.

Table 5-11: Layer Strength Profile in a tabular format

Pavement Layer (mm)	Required DN value for TLC 0.1	Section
		4
		4.400 to 8.380 km
0-150	<= 4	5.8
150-300	<= 9	11
300-450	<= 19	19
450-600	<= 25	25
600-800	<= 39	29

The method does not identify the actual layers of an existing road by the change in strength between them but uses a fixed thickness of 150 mm for each assumed layer. This can be changed in the AfCAP DCP computer analysis software but the problem is that the design catalogue (Table 5-12) are fixed in 150 mm layers. If there are several layers of different strength within the 150 mm, then the software characterises the 150 mm by means of a weighted average DN. However, the relationship between DN and potential traffic carrying capacity is highly non-linear. This means that the actual strength of the 150 mm layer will be overstated. The designer must therefore be vigilant to identify situations where a relatively weak layer exists and to design a suitable remedial treatment.

Table 5-12: DCP-DN design catalogue for different Traffic Load Classes (TLCs)

Traffic Class MESA	**TLC 0.01 0.003-0.01**	**TLC 0.03 0.01-0.03**	**TLC 0.1 0.03-0.10**	**TLC 0.3 0.1-0.3**	**TLC 0.7 0.3-0.7**	**TLC 1.0 0.7-1.0**
0- 150mm Base ≥ 98% Mod. AASHTO	DN ≤ 8	DN ≤ 5.9	DN ≤ 4	DN ≤ 3.2	DN ≤ 2.6	DN ≤ 2.5
150-300 mm Sub-base ≥ 95% Mod. AASHTO	DN ≤ 19	DN ≤ 14	DN ≤ 9	DN ≤ 6	DN ≤ 4.6	DN ≤ 4.0
300-450 mm Subgrade ≥ 95% Mod. AASHTO	DN ≤ 33	DN ≤ 25	DN ≤ 19	DN ≤ 12	DN ≤ 8	DN ≤ 6
450-600 mm In situ material	DN ≤ 40	DN ≤ 33	DN ≤ 25	DN ≤ 19	DN ≤ 14	DN ≤ 13
600-800 mm In situ material	DN ≤ 50	DN ≤ 40	DN ≤ 39	DN ≤ 25	DN ≤ 24	DN ≤ 23
DSN_{800}	≥ 39	≥ 52	≥ 73	≥ 100	≥ 128	≥ 143

Source: Ministry of Transport and Public Works, Malawi, (2019).

By comparing the representative LSP with the requirement as per the design catalogue, the upgrading requirements are as shown in Table 5-11. In this example no additional layers are required because all layers are strong enough.

5.3.2.3 Upgrading Requirements

The upgrading requirements are very similar to those described in Section 5.3.1.1.

Option 1: If the in-situ strength profile of the existing road complies with the required strength profile indicated by the DCP-DN catalogue for the particular traffic class, the road would only need to be re-shaped, compacted and surfaced assuming that the existing road is adequately above natural ground level to permit the necessary drainage requirements and the existing carriageway is wide enough. In the case of a sunken profile, the subgrade must be raised to achieve adequate drainage by filling in layers complying with the DN requirements of the DCP-DN design catalogue.

Option 2: If the in-situ strength profile of the existing road does not comply with the required strength profile for the particular traffic class, then the upper pavement layer(s) need to be:

1) **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.
2) **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).
3) **Mechanically stabilised:** As above, but new, better quality material is blended with the existing material to improve the overall quality of the layer.
4) **Augmented:** If the material quality (DN value) is adequate but the layer thickness is inadequate, then material of appropriate quality will need to be imported to make up the required thickness prior to compaction.

If none of the above options produces the required quality of material, recourse may be made to more expensive options, such as soil stabilisation. However, the design and construction requirements of stabilised layers is outside the scope of this RRN which focuses on the use of natural or lightly chemically-treated materials.

5.3.2.4 Laboratory Tests

The sample preparation for materials in CBR moulds and for other laboratory classification tests for DCP-DN method follows the Republic of South Africa SANS test protocols. Only if these are not available should the ASTM or AASHTO methods be followed. There are often differences in results obtained by the different test methods. It should be noted that there are no SANS or any other standard test methods for conducting a DCP test in moulds. Although the limits on the material grading and plasticity index are not specified for the DCP-DN method, the standard tests to determine the Particle Size Distribution, Atterberg limits and MDD/OMC must be carried out for all material samples to enable the design engineer to interpret the influence of these properties on the material strength under the anticipated long-term conditions in the pavement.

The determination of material strength at the anticipated field density and long-term moisture conditions is based on the Laboratory DN test, hence making it the most important test in the DCP-DN design method. It is always recommended that DCP testing is conducted at the height of the wet season. If this cannot be achieved, the DN values should be adjusted for wet season conditions.

The subdivision into uniform sections limits the variability within the sections and thus the risk associated with design decisions based on assessment of average strength of the in-situ pavement within each section. By excluding outlier values from the CuSum analysis, the risk is further reduced. However, any such weak points excluded, must then be assessed separately.

5.4 Pavement Performance Risk in General

The risk of premature pavement failure comprises two main components namely:

1) the risk of failure because the subgrade is not protected adequately, and
2) the risk of failure in one of the main load bearing layers (roadbase or sub-base) because of weakness in the layer or very heavy vehicle.

Both could be triggered through the ingress of water and the risk of either is increased if adequate drainage maintenance is not carried out regularly and if severe storms occur.

The principal methods of dealing with different levels of risk are summarised in Table 5-13. If there is any risk perceived that the subgrade of a road is likely to become saturated, one option is simply to design on the next lower strength subgrade (preferred) or the next higher traffic category. In general, there is always an option to design for a weaker subgrade if risks of water ingress and weakening from any mechanism is perceived although this is not always pointed out in the manuals or guides.

Unfortunately, it has proved difficult to prevent heavy vehicles, often very overloaded, from using LVRs. The risks of failure of upper pavement layers from vehicle overloading are often assumed to be correlated with cumulative traffic, which is generally true. As traffic increases then the number of cumulative equivalent standard axles also increase, and the design methods all call for a stronger structure to cope as traffic increases. However, there is always a risk that a road may carry a few grossly overloaded vehicles which may impose stresses greater than the stresses that the materials can withstand (for example vehicles carrying dense building materials). The CBR-based methods define a minimum roadbase and sub-base strength for each traffic class which should normally be adequate. However, if the spectrum of axle loads exceeds the normally expected range, thereby possibly putting the design at risk, then the designer should choose a higher traffic level than strictly necessary based on ESAs. In several of the design methods the standard soaked CBR test is used to evaluate the strength of the imported pavement materials and although the designs developed in some recent studies clearly show that specifications can be reduced in many circumstances, retaining the strength specifications defined in the various methods provides a safety net for higher risk situations.

In the DCP-DN method the strength specifications of the roadbase and sub-base as well as the subgrade are also based on the anticipated moisture content, compacted density and cumulative equivalent standard axles and can potentially cover a wider range than in the CBR-based methods. The method however does not provide for choosing a higher traffic class if risk of overloading is perceived.

Table 5-13: Reducing risks

Design Method	Subgrade Failure	Roadbase Failure	Other
AASHTO Method	Design for soaked subgrade	There are adequate calibration options to strengthen roadbase and/or sub-base using: 1) Drainage factors 2) The reliability factor 3) A lower Terminal Serviceability level	General requirements for maintenance especially of drainage and alignment/cross section
Overseas Road Note 31	1) Use the lowest anticipated subgrade strength value 2) Option to use soaked subgrade CBR in severe situations	The minimum strength of pavement layers (these are slightly conservative for LVRRs	General requirements for maintenance especially of drainage and alignment/cross section
TRL-SADC	3 levels of protection 1) Design charts for two different climates. 2) Option to design for a weaker subgrade for higher risk scenarios	Pass/fail minimum strength for both roadbase and sub-base to avoid risk of failure caused by extra heavy vehicles.	General requirements for maintenance especially of drainage and alignment/cross section
DCP-DN	Design thickness to protect subgrade when at its long-term equilibrium moisture content	The DN of the road pavement layers increase as traffic decreases. There is no specific risk reduction for possible heavier trucks except for the cumulative traffic (mesa) category. But there is the option of using a higher moisture content for design (i.e. design for weaker subgrade).	Extra emphasis on maintenance especially of drainage and alignment/cross section

5.5 Alternative Paved Structures

The methods described above are based on roads using unbound granular materials as the pavement layers because these are the most common and popular for LVRRs. However, several other structures are also used and can have advantages. These are described below.

5.5.1 Design of Roads with Discrete Element Surfacings

These pavement types are not as common as those constructed with unbound granular materials and therefore the literature concerning their design and performance is not as prolific. The exception is block pavers such as concrete blocks and bricks which have provided adequate designs. Using SN principles these designs can be extended to other discrete element surfacings. However, their success is highly dependent on the skill of the contractor and stonemason.

5.5.1.1 Hand-Packed Stone (HPS)

HPS paving consists of a layer of large broken stone pieces (typically 150 to 300mm thick) tightly packed together and wedged in place with smaller stone chips rammed by hand into the joints using hammers and steel rods. The remaining voids are filled with sand or gravel. A degree of interlock is achieved and has been assumed in the designs shown in Table 5-14. The structures also require a capping layer when the subgrade is weak and a conventional sub-base of G30 material or stronger.

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.

Table 5-14: Thicknesses designs for Hand-packed Stone (HPS) pavement (mm)

Subgrade Class (CBR)	TLC1	TLC2	TLC3	TLC4	TLC5
	< 01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4%)	150 HPS 50 SBL 175 G30	200 HPS 50 SBL 125 G30 150 G15[1,2]	200 HPS 50 SBL 150 G30 200 G15[1,2]	250 HPS 50 SBL 150 G30 200 G15[1,2]	NA
S3(5-7%)	150 HPS 50 SBL 125 G30	200 HPS 50 SBL 200 G30	200 HPS 50 SBL 150 G30 150 G15[1,2]	250 HPS 50 SBL 150 G30 150 G15[1,2]	NA
S4(8-14%)	150 HPS 50 SBL 100 G30	200 HPS 50 SBL 150 G30	200 HPS 50 SBL 200 G30	250 HPS 50 SBL 200 G30	NA
S5(15-29%)	150 HPS 50 SBL *Note 3*	200 HPS 50 SBL *Note 3*	200 HPS 50 SBL *Note 3*	250 HPS 50 SBL *Note 3*	NA
S6(>30%)	150 HPS 50 SBL *Note 3*	200 HPS 50 SBL *Note 3*	200 HPS 50 SBL *Note 3*	250 HPS 50 SBL *Note 3*	NA

Notes:

1. The capping layer of G15 (soaked CBR 15%) material and the sub-base layer of G30 (soaked CBR 30%) material can be reduced in thickness if stronger material is available
2. The capping layer can be G10 (soaked CBR 10%) provided it is laid 7% thicker
3. On subgrades of CBR > 15%, the material should be scarified and re-compacted to ensure the depth of material of in situ CBR >15% is in agreement with the recommendations in Chapter 6.
4. HPS = Hand Packed Stone, SBL = Sand Bedding Layer

5.5.1.2 Stone Setts

Stone sett surfacing 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. 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. Very little interlock between the stone setts is assumed. Suitable structural designs are shown in Table 5-15.

5.5.1.3 Clay Bricks

Fired Clay Bricks are the product of firing moulded blocks of silty clay. The surfacing consists of a layer of edge-on engineering quality bricks within mortar bedded and jointed edge restraints, or kerbs, on each side of the pavement. The thickness designs are as shown in Table 5-15 for TLC1 and TLC2. Fired clay brick surfacings are not suitable for traffic classes above TLC2.

5.5.1.4 Cobble stones or Dressed Stone Pavements or Concrete Block Pavers

Cobble or Dressed Stone surfacing 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. The individual stones should have at least one face that is fairly smooth, to be the upper or surface face when placed. Each stone is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. Sand or fine aggregates is brushed into the spaces between the stones and the layer then compacted with a roller. Cobble stones are generally 150 mm thick and dressed stones generally 150-200 mm thick. These options are suited to homogeneous rock types that have inherent orthogonal stress patterns (such as granite) that allow for easy break of the fresh rock into the required shapes by labour-based means. The same technique is used for concrete block pavers. The only difference being that the pavers are manufactured using concrete in moulds. Figure 5-8 shows photographs of dressed stone and concrete block pavers.The thickness designs are shown in Table 5-15 except that the thickness of the cobblestone is generally 150 mm instead of the 100 mm shown in Table 5-15.

Table 5-15: Thicknesses designs for various Discrete Element Surfacings (mm)

Subgrade Class (CBR)	TLC1 < 01	TLC2 0.01-0.1	TLC3 0.1-0.3	TLC4 0.3-0.5	TLC5 0.5-1.0
S2 (3-4%)	100 DEL 25 SBL 100 G80 100 G30 100 G15	100 DEL 25 SBL 125 G65 150 G30 150 G15	100 DEL 25 SBL 150 G80 150 G30 175 G15	100 DEL 25 SBL 150 G80 175 G30 200 G15	100 DEL 25 SBL 150 G80 200 G30 200 G15
S3(5-7%)	100 DEL 25 SBL 125 G65 100 G30	100 DEL 25 SBL 150 G65 175 G30	100 DEL 25 SBL 125 G80 125 G30 150 G15	100 DEL 25 SBL 150 G80 150 G30 150 G15	100 DEL 25 SBL 150 G80 175 G30 175 G15
S4(8-14%)	100 DEL 25 SBL 150 G65	100 DEL 25 SBL 150 G65 100 G30	100 DEL 25 SBL 150 G80 150 G30	100 DEL 25 SBL 150 G80 200 G30	100 DEL 25 SBL 175 G80 225 G30
S5(15-29%)	100 DEL 25 SBL 125 G65	100 DEL 25 SBL 100 G65 125 G30	100 DEL 25 SBL 125 G80 125 G30	100 DEL 25 SBL 150 G80 125 G30	100 DEL 25 SBL 150 G80 150 G30
S6(>30%)	100 HPS 25 SBL 125 G65 *Note*	100 HPS 25 SBL 150 G65 *Note*	100 HPS 25 SBL 150 G80 *Note*	100 HPS 25 SBL 150 G80 *Note*	100 HPS 25 SBL 175 G80 *Note*

Notes:

1. SBL is a sand bedding layer
2. The capping layer of G15 (soaked CBR 15%) material and the sub-base layer of G30 (soaked CBR 30%) material can be reduced in thickness if stronger material is available.
3. The capping layer can be G10 (soaked CBR 10%) provided it is laid 7% thicker
4. DEL = Discrete Element Layer, SBL = Sand Bedding Layer

Dressed Stone

Concrete Block Pavers

Source: Cook et al. 2013

Figure 5-8: Concrete pavers and hand-packed stone

5.5.2 Rigid Pavement Methods

This type of pavement has a high initial cost compared to other types, but lasts a very long time if constructed well. They are well-suited for use in road sections of gradient equal to or greater than 10%, or at road junctions with a significant number of heavy vehicle turning movements. They are also very well-suited to labour-based construction or as a low maintenance option for LVRR to avoid constant maintenance. However, the initial cost is often high. Lastly, concrete pavements are recommended for use on road sections designed to be climate resilient. Details for use of concrete pavements as a climate resilient option can be found in Johnson et al. (2019), including design and construction information on jointed concrete slabs, concrete block pavers, geocell concrete, and roller compacted concrete.

For low volume roads, a 75 - 200 mm thick concrete layer placed on a sub-base of CBR 30%, covers most the requirements of traffic on subgrades of CBR 3% and higher. Thus, the design process is not complex. For slabs 100 mm or lower, it is recommended to construct these as continuously reinforced concrete pavements. Mesh steel reinforcement should be used in that case. Slabs thicker than 100 mm may be constructed as plain (unreinforced) concrete – jointed and undowelled. The concrete should be laid on at least 100 mm of G30 material compacted to 98% MDD Mod AASHTO. On subgrades of G30, the material should be scarified and re-compacted to ensure the depth of the in-situ material is in agreement with the recommendations.

If a plain concrete unjointed pavement is used then the recommended slab joints should be at 4 to 5 m intervals. If the carriageway width is 4.5 m or more, then the slab widths should be half the carriageway width. Otherwise for roads of 4.5 m width or less, the slab width may be equal to the carriageway width. The joints between slabs should be filled with bitumen slurry.

The concrete cube (150 mm) compressive strength should be 25 MPa or higher at 28 days of curing. A mix design with materials to be used on site should be used to achieve the strength.

During construction, good quality formwork should be used at the edges to ensure good vibratory compaction. Grooves should be made on the slab surface after initial set of the concrete to improve skid resistance during use. A wire brush or broom may be used to achieve this.

Although concrete road options are typically very durable, good drainage is still critical around a concrete pavement. Failures have occurred where the concrete slab has been undermined or soil along the edges eroded by running water. Standard designs for concrete roads for LVRs are normally available in the manuals listed in the bibliography.

5.6 Summary of Key Decisions

5.6.1 Materials

The choice of materials is a major decision but meeting the specifications is the key to success. There are simply many options altogether but perhaps not many at each road site. Chapter 3 discusses the many ways that materials can fail to meet the specifications and, in many cases, how to overcome the deficiencies. A common situation occurs when materials do not quite meet the specifications but unless the design engineer has experience of the materials in question it is not advisable to deviate from the specifications. If in doubt it may be advisable to consult an acknowledged expert on the materials in question.

5.6.2 Subgrade Strength

There are essentially three distinct methods but the method used must be the method that the authors of the overall design method used in their analysis of data from which they built their design method. Table 5-16 summarises the options for the methods considered.

Weakest anticipated condition. This appears to be a logical choice but it is very difficult to determine accurately. It is often suggested that the subgrade beneath a nearby road on the same subgrade is measured in the rainy season but not only is this likely to be inconvenient for the contractor or consultant but there is considerable variability from year to year therefore the result may not be very representative. Secondly the road is generally designed for at least 15 years in which time many changes can occur and the weakest period may occur for reasons other than the direct result of climate. For example, poor or deficient maintenance, agricultural practices.

Long-term empirical data. In relatively recent years long term data from road performance studies in moderate climates have confirmed that under most circumstances the moisture content of the pavement layers rarely exceeds the Optimum Moisture Content (OMC) for compaction at the design compaction level. In less moderate conditions a moisture content of about 0.75xOMC is often more realistic and only in exceptional circumstances is a soaked condition found. In some design methods the choice is simply left to the experience of the design engineer based on an assessment of the drainage features of the road, primarily the height of the crown of the road above the bottom of the drain, whether the height of the sub-base is above the likely water level in the side drain and whether the shoulders of the road are sealed or not. Even with these characteristics there are still additional risks that can arise over a 15 year design period.

Conservative low risk strategy. Finally, a low risk strategy of catering for the worst likely condition is to assume that at some stage in the design life the road the subgrade will be saturated and that designing on a soaked value of subgrade strength will be the safest option. However, this method will only be satisfactory if the design charts have been developed with this option in mind. However, when the subgrade is very weak (< 3% CBR), special treatment is usually required and the basic design charts may not cater for such a value.

Table 5-16: Selection of the subgrade strength for design

Design Method	Summary	Details	Advantages
AASHTO	Based on a comprehensive Road test in one location and one subgrade. Subgrade assessed as a weighted value based on anticipated worst conditions every month but designer not restricted to this.	The design equation provides 3 methods of calibrating the design equation based on local road performance data. The concepts of equivalent standard axles, structural number, reliability (variability), service level are concepts developed during the Road Test and are in worldwide use.	Can be used anywhere provided that road performance data are available to calibrate the method and develop a design catalogue. This is essential. Designer can choose any moisture content for the subgrade design class depending on assessment of risk. The strength of pavement layers is variable within the SN concept and the range used in the Road Test
ORN 31	Based on anticipated weakest value.	Three climate categories are defined based on the depth of water table, and the engineering characteristics of the subgrade to produce the anticipated subgrade CBR weakest value. The strength of pavement layers is the traditional minimum strength pass/fail soaked CBR test	Only the general climate category, depth of water table and basic properties of the subgrade soil are required but designer can choose any moisture content depending on risk assessment.
TRL-SADC	Based on the in situ CBR of the pavements whose performance was monitored to develop the design charts. The designer does not need to estimate the subgrade strength The design chart shows the soaked subgrade CBR of these pavements not the in situ value.	The subgrade is measured in the soaked condition but this merely identifies which subgrade strength to use in the design chart to make sure that the correct chart is used. Two design charts are used depending on whether conditions are moderate/dry with good drainage and one for conditions that are expected to be wetter. For severe conditions a higher class or a lower subgrade CBR can be used. The strength of pavement layers is the traditional minimum strength pass/fail soaked CBR test but the values are reduced for the lower traffic levels.	Three levels of risk are catered for. Material specifications are comprehensive therefore risk of failure is low.

Design Method	Summary	Details	Advantages
		Provided 5 traffic classes below 1.0 mesa. This allows for less overdesign.	
FOUNDATION Class	For each subgrade strength the method involves unifying the design of the lower pavement layers to form the same foundation which is used for all traffic levels. Four such foundations are defined	Based on a soaked CBR test. The strength of pavement layers is the traditional minimum strength pass/fail soaked CBR test but the values are reduced for the lower traffic levels.	As with most methods the designer can choose a lower foundation subgrade or higher traffic class depending on assessment of risk. The foundations can be assigned a different level of reliability from that of the pavement layers Testing the foundations becomes very routine and increases the eventual quality considerably
ORN 18 -DCP (DCP-CBR)	This is a method of designing for rehabilitation. It is the ORN 18 method with the DCP to provide sufficient data and the TRL DCP program to assist with the analysis	Subgrade strength is that of the existing road and not assessed here	The use of the DCP in this method and also in other methods during in-situ assessment enables areas of weakness to be detected easily for further investigation and subsequent treatment. Weak areas can be a result of poor drainage, de-densification, or material degradation.
DCP-DN	The DCP test completely replaces the CBR test both in the field and in the laboratory.	Based on in situ subgrade and pavement layer strengths	The use of the DCP in this method and also in other methods during in-situ assessment enables areas of weakness to be detected easily for further investigation and subsequent treatment. Weak areas can be a result of poor drainage, de-densification, or material degradation.
Pure Empirical Methods	Macadam, Telford Bases, HPS. Requires estimate of the worst case subgrade strength but the estimate is not critical		Makes use of coarse material that might be rejected. Climate resilient because material strength little affected by soaking or saturation.
Rigid pavements	Standard method from design manual		

5.7 Summary of Strengths and Limitations of the Design Methods

This summary focusses on some key strengths and limitations (Table 5-17) but the absence of a comment concerning any aspect should not be seen as negative or positive. The comments here are highlighting notable features and should not be seen as exhaustive.

Table 5-17: Summary of strengths and Limitations of the pavement design methods

Method	Strengths	Limitations
AASHTO	Can easily be adapted or calibrated to suit a wide range of conditions. The properties of materials including very coarse particles can accurately be incorporated. It is the only method that includes a true reliability factor that recognises the inherent variability in road performance. Widely used in the USA (and elsewhere) with considerable, literature, debate and feedback amongst practitioners.	The basic performance data was dominated by the northern USA climate of freeze thaw conditions and a very weak subgrade hence careful calibration is essential for different climates Not adequately tested on strong subgrades Primarily for high volume roads hence the manual is a substantial document, but this should not deter users. It has been used successfully in the USA for 60 years and in many other countries.
ORN 31	Subgrade strength for design can be based on any moisture content. Offers designs for 8 road types including stabilised roadbases Has been in successful use for over 40 years in a wide range of climates.	Not specifically designed for the lowest LVR classes and provides only 3 traffic classes below 1.0mesa. For LVRs material specifications can be conservative therefore method can be more expensive if alternative suitable materials are available.
TRL-SADC	Offers two design charts based on climate and basic drainage quality and for a range of various road types suitable for LVRs Provides 5 traffic classes up to 1.0 mesa Deals with subgrade strength for design using a very pragmatic method that discourages overdesign. Material specifications are comprehensive but have been tailored to pragmatic level suitable to the task of a LVR and relaxed when appropriate to do so. Essentially an up to date version of ORN 31 for LVRs but with modified material specifications to capture the results of recent research.	Several material specifications need to be met hence the initial cost may be a little higher than other methods. Based on research in relatively moderate climates.
Foundation Class	The construction of reliable foundations that can be readily and routinely checked by measurement of bearing capacity by means of plate-tests (FWD, LWD, and plate bearing test) and improved where necessary provides a very valuable method of facilitating the construction of a uniform and high quality road with a foundation that should serve well for several design lives. The large plate test area also allows	Requires the availability of equipment for testing (FWD, LWD, Plate bearing) during construction for the full benefits to be realised and it requires the local road construction industry to become familiar with the method and for it to become routine. Based on research in East Africa and therefore not an extreme climate

Method	Strengths	Limitations
	for bulk stress assessment of materials containing large particles. The method is flexible because the foundation can be constructed with a different level of reliability to that of the pavement layers. In principle the method can be adapted for use in other design methods	
ORN 18 DCP-CBR	Layer detection is automatic; not predetermined. This allows pockets of thin weak layers to be identified and removed or otherwise dealt with. Uniform sections can be assigned using several different parameters (subgrade strength, layer strength, thickness, total structural numbers, etc) and hence dynamic layer sectioning that reflects the variability of each of the parameters can enhance the selection of uniform layers. Errors or inaccuracies of layer strength averaging caused by the non-linearity of traffic carrying capacity versus thickness design are very significantly reduced.	Only suitable for upgrading or rehabilitating an existing road The use of DCP means that the designer cannot be sure which of the factors (moisture content, particle size distribution or density) is influencing the in-situ strength. Thus, for example, improving drainage may be all that is required instead of a new layer, however other laboratory tests can also be carried out to resolve this. This affects all DCP-based The method is not applicable on roads requiring deep cuts and fills more than 1.5 m, since the length of a DCP rod (including extension) cannot penetrate beyond this depth.
DCP-DN	The use of a DCP to identify uniform sections saves on the cost of test pits. The method is less stringent on materials specifications, hence the flexibility to use different locally available materials. The design process can be completed with minimal laboratory equipment, as is the case with a number of low income countries.	Not yet established in more severe climates, but being trialled in several countries. Less stringent materials specifications mean the selection of materials could lead to increased risk for the less experienced engineer. The method assumes that the subgrade is always 450 – 800 mm below the finished pavement surface. This is not always the case. The method is only applicable to natural untreated materials. The DCP instrument alone cannot be used for in-situ identification of uniform sections on roads requiring deep cuts and fills more than 1.5 m, since the length of a DCP rod (including extension) cannot penetrate beyond this depth.
Purely Empirical Methods	Makes use of very coarse materials that are otherwise rejected by several methods. These materials can be extracted or obtained from crusher plants. Often labour-based and hence creates employment. Often highly suited as a climate resilient options since material strengths are largely unaffected by soaking or saturation. Quality control during construction is not a major challenge	Not many areas have an abundance of suitable rock Often slow to construct

Method	Strengths	Limitations
Rigid Pavement Method	Highly suited as climate resilient options since material strengths hardly affected by soaking or saturation. Well-suited for very steep gradients. Quality control during construction is not a major challenge. Low maintenance costs	High initial cost of provision of the option Requires sufficient curing if the design is to be achieved – this is not always possible in areas of water scarcity or where supervision is poor.

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6 DESIGN OF UNPAVED ROADS

6.1 Background

In their simplest forms, unpaved roads consist of tracks or earth roads over which goods or persons are moved directly on the in-situ material surface. This may in some cases be ripped, shaped and compacted (engineered) but generally the only compaction is that applied by vehicles moving over it (un-engineered). There comes a point with these roads when passage is excessively affected by the weather and vehicles can no longer traverse the road during inclement conditions.

This problem is solved by applying a selected material with specific properties over the in-situ material to ensure all-weather passability and the roads then become gravel roads. Roads with a gravel wearing course should only be seriously considered as an option where**:**

1) Maintenance is guaranteed;
2) Gravel quality is adequate;
3) Gravel quantities are available;
4) Haul distances are short;
5) Rainfall is low to moderate;
6) There are no steep gradients;
7) Dust does not constitute a constraint;
8) The traffic levels are low (less than 300 vpd).
9) Soil conditions during wet weather do not allow passage without some structural reinforcement.

Unpaved roads will usually carry a maximum of about 200 to 300 vehicles per day (with less than 10 % being heavy) but in areas where materials are poor, upgrading to paved standard can often be economically justified at traffic volumes much lower than this.

6.2 Purpose and Scope

This chapter deals with the provision of roads that are completely unpaved. These roads are either engineered natural surfaced roads (ENS) or are surfaced with an unbound gravel layer. The purpose of this chapter is to provide best practice guidance on the design of such roads in an economic and sustainable manner such that the appropriate levels of quality are achieved and maintained.

Unpaved roads require more care than sealed roads. When well-constructed and maintained, their performance can be very satisfactory and much of their layers can eventually be fully utilised in a surfaced road if or when it is decided to upgrade them.

A comprehensive literature review was carried out as part of this project and the international literature shows that there are two quite distinct schools of thought concerning the design of unsurfaced roads. The first is that full structural design is required and the depth and strength of the structure beneath the gravel surface should be the same or very similar to that of a road that is surfaced. This approach is very costly and unnecessary in most situations for several reasons. Firstly, any deep structurally induced deformation is corrected before it becomes serious by regular grading and, secondly, on a low volume unpaved road, the channelling of vehicle wheels is less concentrated than on a paved road and structural failure therefore much less likely to occur. The exception to this is for unpaved roads that serve a different function and are required to carry much more and heavier traffic. However, these would not be classified as LVRs and are therefore outside the scope of this guide. The second school of thought is that gravel roads do not require any structural design and focus should be made on only providing a suitable wearing course.

The review indicated a definite downward trend in thickness design in recent years and the latest version of the Federal Highways Administration (FHWA) guide, 'Gravel Roads Construction and Maintenance Guide; FHWA Technology Partnership Programs, 2015' rewrites the recommendations of the previous edition published in 2000. The recommendations made here are therefore considered to be current best practice.

The approach adopted in this chapter therefore is that an ENS should be provided where appropriate. When traffic levels exceed 25 vehicles per day or for basic access over very clay-rich soil under wet conditions, then gravel roads should be considered. For gravel roads, it is recommended that focus should be on finding suitable wearing course material. Structural requirements should be considered secondary because any structural failure is easily corrected by routine maintenance grading. This will differ from the approach given in many country manuals.

6.3 Principles of Good Design of Unpaved Roads

The following principles apply for good design:

1) The crown height of the earth road should be at least 350 mm above the bed of the drain.
2) Where the topography allows, wide, shallow longitudinal drains are preferred. They minimise erosion and will not block as easily as narrow ditches. The ditches grass over in time, binding the soil surface and further slowing down the speed of water, both of which act to prevent or reduce erosion. The side drains should be 1m at invert level with side slopes of 1:3.
3) 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 5-7% to ensure that water runs off the surface and into the side drains.
4) Areas where there are specific problems (usually due to water or to the poor condition of the subgrade) may be treated in isolation by localised replacement of subgrade, gravelling, installation of culverts, raising the roadway or by installing other drainage measures. This is the basis of a 'spot improvement' approach.
5) Water should be drained away from the carriageway side drains by excavating mitre drains to divert the flow into open space. The spacing of the mitre drains should be no greater than 100 m. 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.
6) Whatever the case, gravel roads that have been well-compacted tend to have significantly lower gravel loss and better ride quality. Also, a dense, well-graded gravel will have better performance than poorly graded materials.

Pavement design for both paved and unpaved roads requires knowledge of the expected traffic and of the strength of the subgrade soil. For the structural pavement design of LVRs, three traffic classes have been defined as shown in Table 6-1.

Table 6-1: Traffic Classes for design of gravel roads

Traffic range (mesas)	LVR1	LVR2	LVR3
	< 0.01	0.01 – 0.1	0.1 – 0.3

To achieve adequate external drainage, the road must also be raised above the level of existing ground such that the crown of the road is maintained at a minimum height (h_{min}) above the drain inverts. The minimum height is dependent on the climate and road design class as shown in Table 6-2.

Table 6-2: Minimum Height (h_{min}) between Road Crown and Invert Level of the Drain.

Road **Class**	**Climate factor N(1)**	
	Wet (N < 4)	**Dry (N > 4)**
	h_{min} (mm)	**h_{min} (mm)**
LVR1	350	250
LVR2	400	300
LVR3	450	350

(Weinert, H, 1980.)

Various road authorities have published design methods for gravel roads but the principal methods are based essentially on the following:

1) A surfacing layer is required comprising material that will be gradually lost or worn away during use. The loss of gravel is the primary form of long-term deterioration. (Loss of surface shape, for example, is simply corrected by maintenance grading). Gravel loss depends on traffic, climate, road geometry and gradient, and material properties. Many studies have been carried out to quantify gravel loss for maintenance and network management purposes, e.g. Hodges et al. (1975); Paterson, W.D.O. (1987); Paige-Green (1989); Cook, J. and Petts, R. (2005); Mukura, K. (2008). The initial thickness of such a layer is typically 150 mm but values up to 300 mm have sometimes been recommended. Thicker layers obviously last longer and need replacing less frequently.
2) At least one additional layer is required to maintain structural strength when the surfacing layer is becoming thin. The layers of pavement in unsurfaced roads are exposed to climatic conditions and are frequently trafficked when their moisture content is high and their strength is low.
3) Research by the US Army Engineering Research Centre on the traffic carrying capacity of soils and gravels has shown that materials of CBR > 15% are capable of carrying several thousand coverages of heavy army vehicles. If wheel path channelling does not occur (as in a normal gravel road of appropriate width) each coverage is equivalent to 2-3 passes of the vehicles hence the traffic capacity of a relatively wet gravel road can be substantial.
4) Naturally different road authorities have developed their own design methods for unsurfaced roads but the method illustrated below is fairly representative.

The choice between an earth road, a gravel road and a paved road will depend on many factors, but economic consideration as described in Chapter 8 is perhaps the most significant factor. More guidance on selection between paved and unpaved options is given in Cook et. al. (2013). It should be noted that the scarcity of good quality gravels for unpaved road construction is becoming more common, with the consequential impact on durability and significantly more frequent re-gravelling, thus strengthening the case for paving at lower levels of traffic.

6.4 Engineered Natural Surfaces (ENS)

Such roads are made only from the natural subgrade but their shape and the drainage design determine their performance. When well-designed their performance can be greatly improved, hence they have an important role to play in provision of access for roads or tracks carrying up to 25 vpd.

The details of the cross-section for an ENS are shown schematically in Figure 6-1.

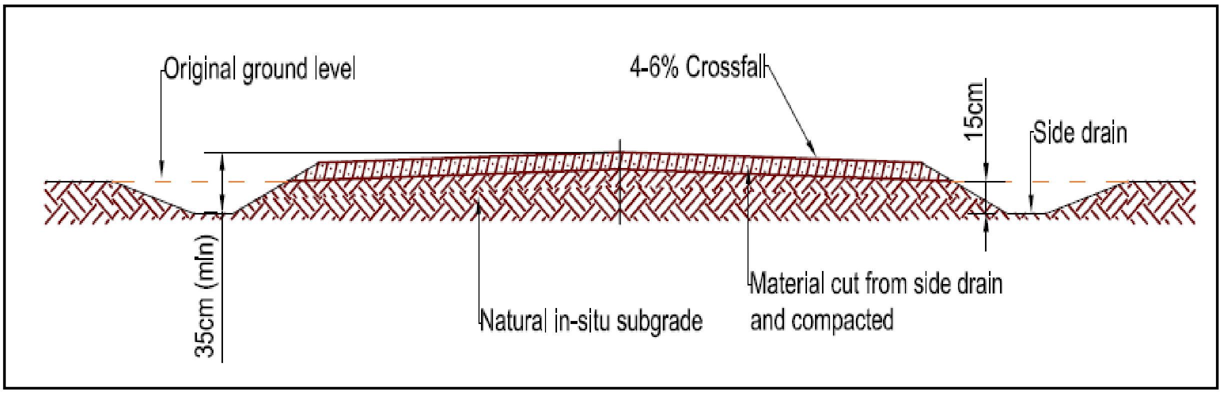


Figure 6-1: Cross-Section Details of an ENS

6.5 Material Design Requirements for Gravel Wearing Course

6.5.1 General

The materials requirements for the gravel wearing course applies to both minor and major gravel roads but NOT to earth roads. The wearing course should constitute part of the structures shown in Table 6-6 and Table 6-7. The main purpose of the gravel wearing course is to protect the structural layer and to provide a good riding quality. The wearing course should be designed in such a way as to enable an acceptably smooth ride (low roughness). The material used for the wearing course should:

1) Have low gravel loss,
2) Should not be slippery,
3) The maximum particle size should not exceed 40 mm. A smoother ride is achieved with a maximum size of 25 mm.

Where natural materials are scarce, blended materials can work well. Successful blends can often be obtained through mixing non-plastic sand or river gravel with a material with high PI. Laboratory tests should be performed in order to determine the optimum proportions of the materials to be blended. The materials should be blended in proportions of 70/30, 60/40, 50/50, 40/60 and the best blends determined by comparison with the specifications for a gravel wearing course.

6.5.2 The Grading Modulus and Plasticity Product Method

Figure 6-2 illustrates the relationship between material properties and performance as a wearing course. The wearing course thickness depends on the annual gravel loss (Table 6-5) and the planned number of years between re-gravelling operations. Commonly, 150mm is used at construction stage and the layer is re-gravelled to 150mm thickness during each operation.

Table 6-3: Recommended Specifications for the Gravel Surfacing based on GM

Property	Specification
Maximum size (mm)	37.5
Oversize Index (% retained on 37.5mm sieve)	< 5%
Plasticity Product (PP)	50[1] - 480
Grading Modulus (GM)	1.0 – 1.9
Soaked CBR at 95% Mod AASHTO	>15%
Note 1. A minimum PP of 280 is preferred to enhance adhesion of larger particles	

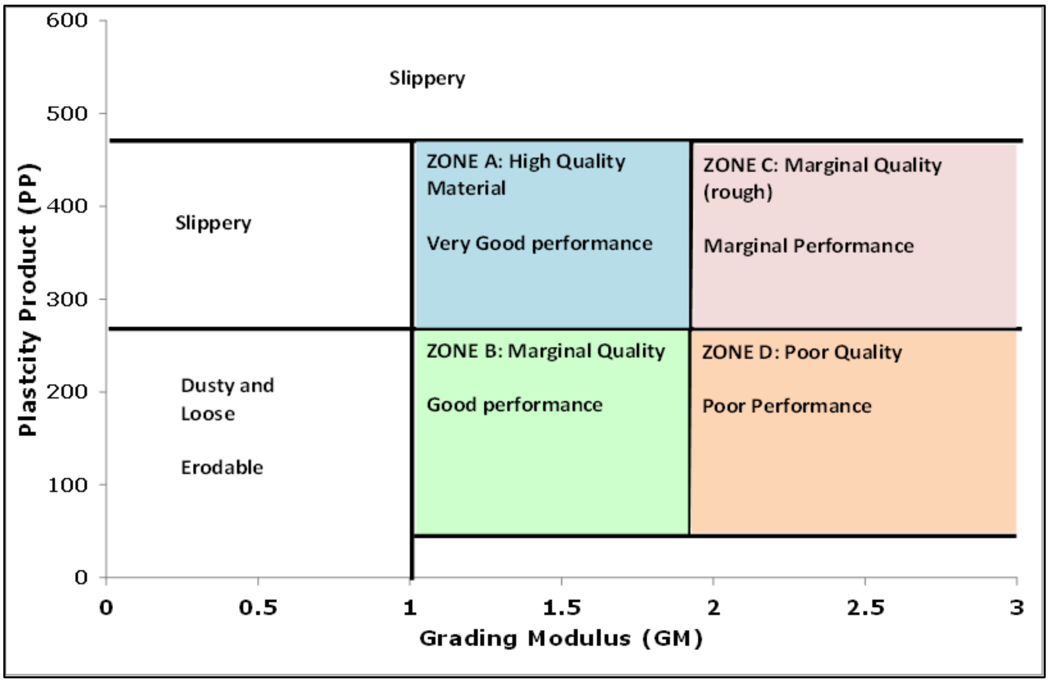


Figure 6-2: Selection Chart for Gravel Wearing Courses using PP and GM

Where

a) PI = the Plasticity Index of the material passing the 0.425 mm sieve

b) Plasticity Product (PP) = PI x $P_{0.075}$ (The preferred range is 280-480)

c) Grading Modulus (GM) =

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

Where: $P_{2.36}$ = percentage passing the 2.36 mm sieve

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

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

The preferred range is 1.0-1.9

The particle size distribution test for the material must be carried out using the wet sieving method.

6.5.3 The Grading Coefficient and Shrinkage Product method

A similar chart based on Grading Coefficient and Shrinkage Product is shown in Figure 6-3. The two charts show similar quality areas but Figure 6-2 is more closely linked to factors that reflect costs namely gravel loss and road roughness.

Table 6-4: Recommended Specifications for the Gravel Surfacing based on GC

Property	Specification
Maximum size (mm)	37.5
Oversize Index (% retained on 37.5mm sieve)	< 5%
Shrinkage Product (Linear Shrinkage x $P_{0.425}$) (SP)	100 – 365[(1)]
Grading Coefficient (GC)	16-34
Soaked CBR at 95% Mod AASHTO	>15%
Note 1. A maximum SP of 240 preferred to reduce dust generation in service	

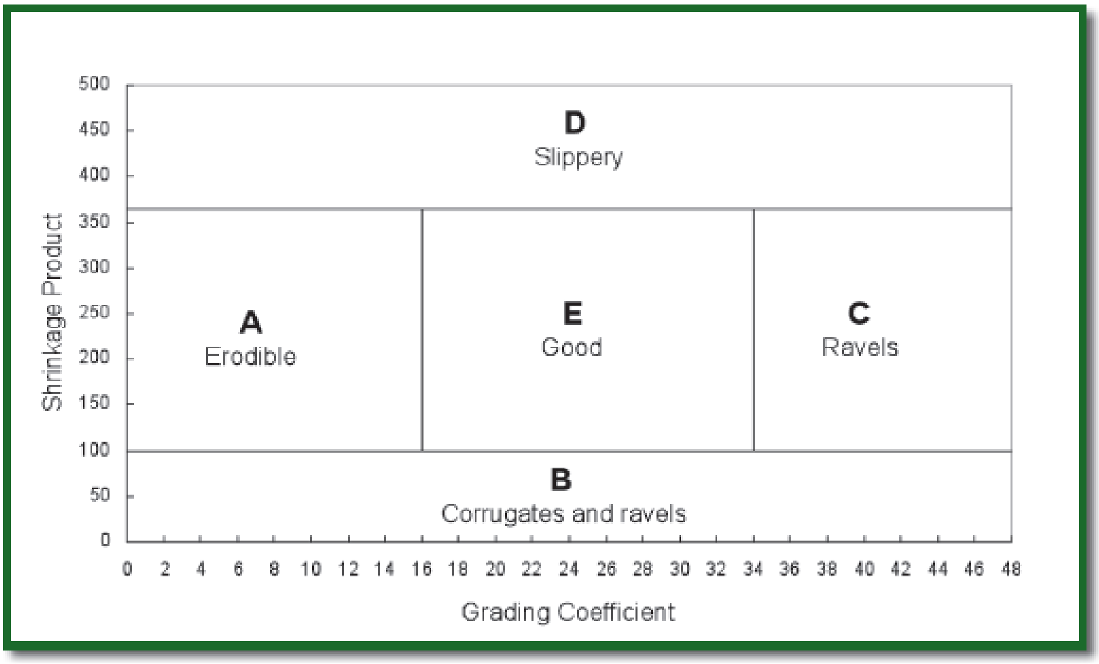


Figure 6-3: **Selection Chart for Gravel Wearing Courses using SP and GC** (*Source: Paige-Green 1989)*

On the chart in Figure 6-3, the 5 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 and ravelling/loosening.
- Zone C: Poorly graded materials that are prone to ravelling.
- Zone D: Fine plastic material prone to slipperiness and excessive dust.
- Zone E: Optimum materials for best performance.

Where:

Grading Coefficient (GC) = (% passing 26.5 - % passing 2.00) x (% passing 4.75) / 100.

Shrinkage Product (SP) = Linear Shrinkage$_{0.425}$ x P0.425

It should be noted that the chart Figure 6-3 assumes good compaction of the wearing course (95% maximum dry density) at construction and maintenance. Additionally, it requires modification (calibration for local conditions and materials) for:

- use in areas of high rainfall (>1000 mm/yr),
- roads that carry more than 25% heavy vehicles,
- materials whose particles breakdown easily e.g. weathered rocks and scoria (cinder) gravels.

Finally, it should be noted that two different materials, one fine-graded and another coarse-graded can possess exactly the same Gc values. For example, fine clay and coarse crushed stone both possess Gc close to zero. The materials would obviously perform differently, but the identical Gc values would erroneously imply that they are the same and would perform the same way.

Consider adding a figure that shows materials performance based on gradation curves, also commonly used in many parts of the world.

6.5.4 Gravel Loss

Gravel loss is the single most important reason why gravel roads are expensive in whole life cost terms and often environmentally unsustainable, especially when traffic levels increase. Reducing gravel loss by selecting better quality gravels or modifying the properties of poorer quality materials is one way of reducing long term costs. Gravel losses (gravel loss in mm/year/100vpd) are determined in relation to the quality of the gravel wearing course (Table 6-5).

Table 6-5: Typical Gravel Loss

Material Quality Zone	Material Quality	Typical gravel loss (mm/year/100vpd)
Zone A	Satisfactory	20
Zone B	Poor	45
Zone C	Poor	45
Zone D	Marginal	25
Zone E	Usually Good	15

Note: Zones refer to the zones shown on Figure 6-3.

The gravel losses shown in Table 6-5 hold only for the first phase of the deterioration cycle lasting possibly two or three years. Beyond that period, as the wearing course is reduced in thickness, other developments, such as the formation of ruts, will also affect the loss of gravel material. The rates of gravel loss increase significantly:

1) On gradients greater than about 6%,
2) In areas of high and intense rainfall,
3) Where grader operators cut too deeply,
4) Where compaction is poor. Spot improvements should be considered on these sections.

Re-gravelling should take place before the lower layer is exposed. The re-gravelling frequency, R, is typically in the range 5 - 8 years.

The wearing course thickness = R x GL

R = re-gravelling frequency in years

GL = annual gravel loss.

6.5.5 A Risk-based Approach to Wearing Course Selection

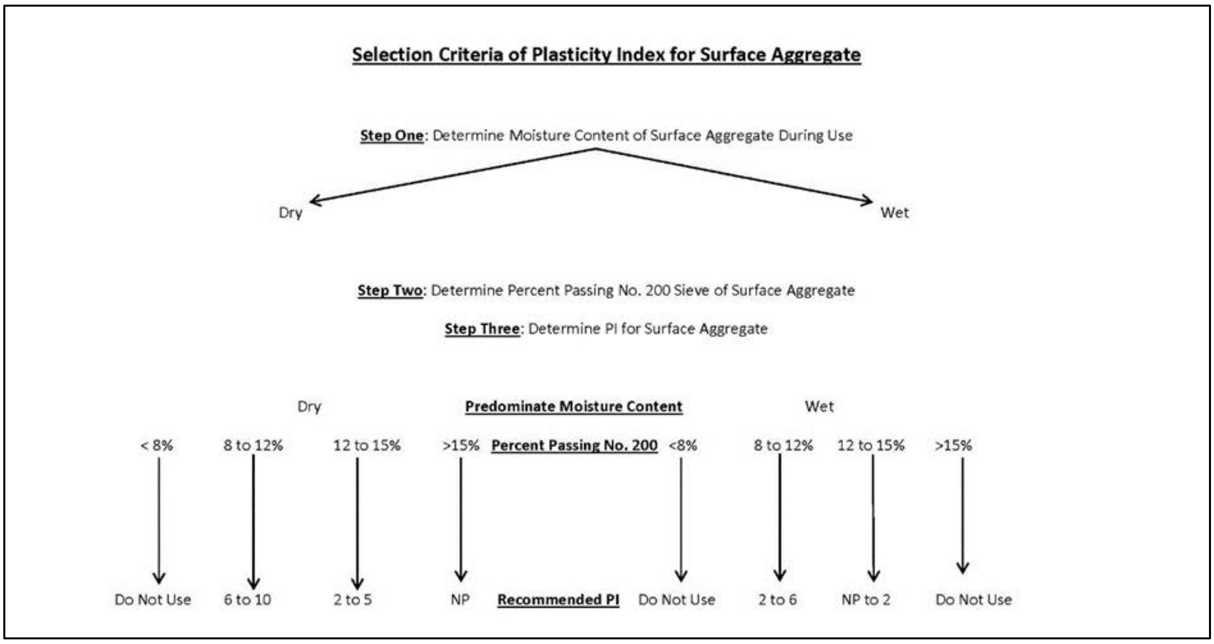
A recent publication (Austroads, 2020) uses a combination of factors to select appropriate gravel wearing course materials. A matrix of traffic measured in MESA and probability of mean annual rainfall exceeding 500 mm/y is used. Three classes of traffic and three risk levels of rainfall exceeding 500 mm/yr are used to form nine risk levels (a 3 x 3 matrix). Based on the traffic level and the expected mean annual rainfall for a given site, for example traffic less than 0.25 MESA and low probability of rainfall exceeding 500 mm/yr, a risk category representing that combination is obtained. Using the risk category obtained from the matrix, the risk of using different types of materials are given. The materials characteristics are measured in terms of Grading Modulus (GM) and Shrinkage Product (SP). The decision of the risk level to be selected for each project has to be made by the design engineer based on the choice of materials available. Most materials of GM between 0.8 and 2, and SP between 50 and 360 are classified as having low to medium risk level of performance expectation for most levels of low volume traffic and moderate levels of rainfall. Details of how to use this approach are found in Austroads (2020).

6.5.6 Other Wearing Course Selection Methods

Another approach to the selection of surfacing is based primarily on percent of material passing the 200 µm sieve, plasticity index, and the climatic condition as shown in Figure 6-4.

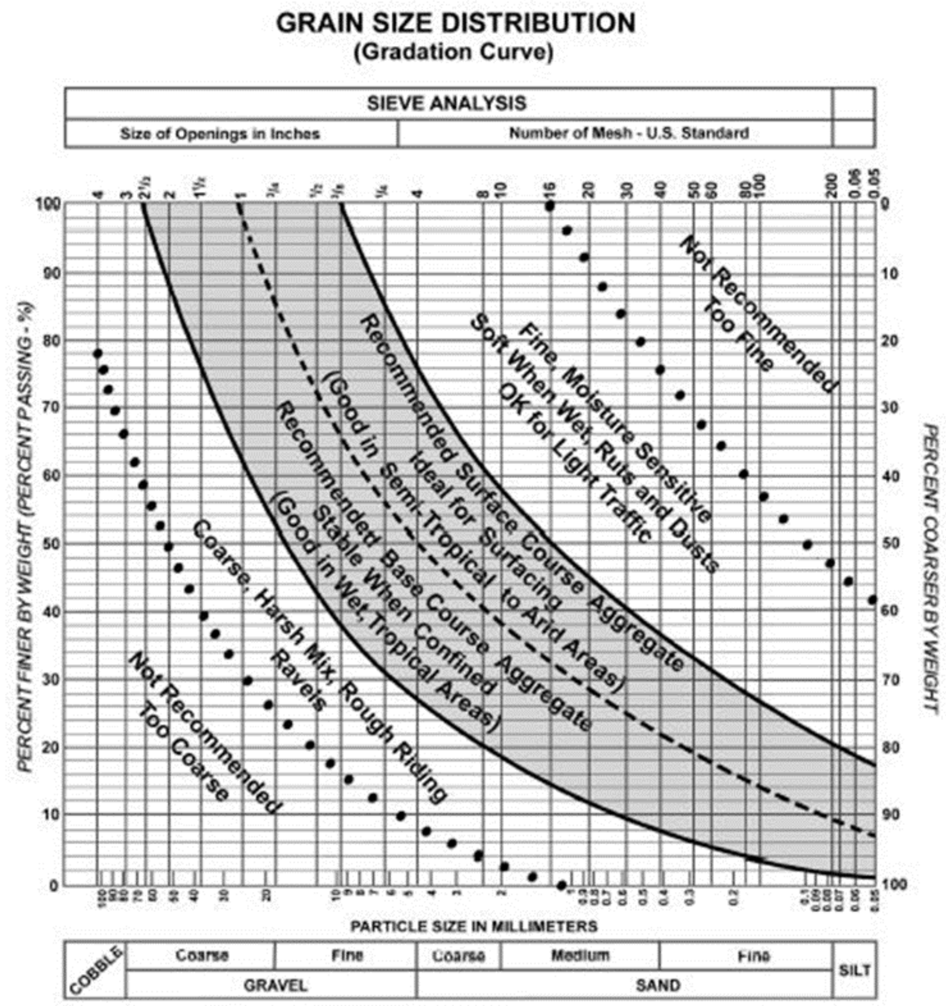
Generally, the method is a proxy for Plasticity Product (PP) as described in section 6.5.2.

Another approach is based on gradation as shown in Figure 6-5. Generally, the wearing course has different requirements from a base course, with typically higher plasticity and more fines to help bind the material together, even though it does compromise somewhat on strength.



Source: Bolander, P. 2020 - under publication

Figure 6-4: Wearing course selection by plasticity product and climatic condition



NOTE: Gradation Ranges Shown Are Approximate.

The best roadbed surfacing materials have some plasticity and are well graded. They have gradations parallel to the curves shown above, and are closest to the "Ideal" dashed curve in the middle of the gradation ranges shown.

Source: Keller et al., 2011

Figure 6-5: Gradation method of selecting wearing course material by climatic condition

6.6 Thickness Design of Gravel Roads

In some cases, especially when the traffic is between 100 to 300 vpd, gravel roads will require a careful selection of the gravel layer. The approach to design should be as follows:

1) The subgrade should be prepared in the same way as for sealed LVRs.
2) It is assumed that the wearing course will be replaced at intervals related to the expected annual gravel loss and any structural deformation corrected .
3) 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. For example;

1) When the quality of the gravel is poor.
2) Where no sources of gravel are available within a reasonable haul distance.
3) On road gradients greater than about 6%.
4) In areas of high and intense rainfall.

5) 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:

1) Determine the traffic volume and traffic loading. The design period is usually for 7 to 10 years but varies for each road authority.
2) Determine the strength of the subgrade at the appropriate moisture condition.
3) Establish the quality of the gravel that is to be used. If only very poor gravel is available, blending with another gravel or soil to improve its properties may be an option.
4) Determine the thickness of gravel base from Table 6-6 and Table 6-7.
5) These charts will differ from those that are currently available in many design manuals. They have been developed using results from FHWA (2015) during preparation of this RRN. They eliminate the need for CBR 30% and 45% specified for wearing course materials in some manuals. The common practice is to apply 100 to 150 mm thickness of gravel for medium to strong subgrades and light traffic, and for heavier traffic and weak subgrades, the thicknesses of 200 to 300 mm.
6) Where the required gravel layer is greater than 200 mm, a two-layer design can be adopted reserving the better quality material, if there is any, for the wearing course.

Table 6-6: Base Thickness for Major Gravel Roads – Medium Gravel (Soaked CBR 20%)

Subgrade Class CBR (%)	**Traffic Classes (mesas)**		
	TLC1 (<0.01)	**TLC2 (0.01-0.1)**	**TLC3 (0.1-0.3)**
S1 <3	165	180	270
S2 (3-4)	165	165	245
S3 (5-7)	150[1]	150[1]	210
S4 (8-14)	150[1]	150[1]	185
S5 (15-29)[2]	150[1]	150[1]	150

Notes:

1. Actual required thickness is less than 150 mm. This has been marked up to reduce regravelling frequency.

2. If subgrade CBR is equal or greater than 20%, then reshape to 4%-6% camber, scarify to recommended thickness, and compact.

Table 6-7: Base Thickness for Major Gravel Roads – Weak Gravel (Soaked CBR 15%)

Subgrade Class CBR (%)	Traffic Classes (mesas)		
	TLC1 (<0.01)	**TLC2 (0.01-0.1)**	**TLC3 (0.1-0.3)**
S1 <3	180	200	300
S2 (3-4)	180	200	270
S3 (5-7)	160	200	250
S4 (8-14)	150	150	200
S5 (15-29)[2]	150[1]	150[1]	160

Notes:

1. Actual required thickness is less than 150 mm. This has been marked up to reduce regravelling frequency.
2. If subgrade CBR is equal or greater than 15%, then reshape to 4%-6% camber, scarify to recommended thickness, and compact.
3. If the native subgrade is in the S6 class (CBR>30%), then the thicknesses in Table 6-6 should be used.

The following should be noted:

1) It is unlikely that the maintenance of gravel roads designed to carry more than 0.3 MESA will be economical. Beyond that traffic level paved options are likely to be more economical.
2) The thicknesses required increase if the gravel is weak hence stronger gravels should generally be used if they are available at reasonable cost.
3) Where the available gravel is not homogeneous, it will be necessary to substitute a particular class of gravel with one or more different classes of gravel of appropriate thickness. The following conversion factors may be used for this purpose.

$$G20 = 1.12 \times G15$$

Thus, a 200 mm layer of G20 material could be substituted with a 225 mm layer of G15 material.

The wearing course constitutes part of the structures shown in Table 6-6 and Table 6-7, and should conform to the wearing course requirements specified in Table 6-3 or Table 6-4.

For effective compaction of the gravel layer, it is necessary to restrict the loose thickness of gravel to a maximum lift of about 200 mm. Thus, any of the gravel layers that require a compacted thickness of more than 150 mm must be compacted in more than one 200 mm lift.

Austroads (2009) provides alternative guidance on structural design of unpaved roads up to 0.5 MESA. The approach taken is to provide designs similar to that for sealed roads with the exception that the bituminous seal is replaced with a gravel wearing course of at least 150 mm. This approach should only be adopted if the designer is certain that the road is likely to be sealed by the next regravelling period or if the road has a high number of heavy vehicles (>15%). However, it should be noted that a primary function of many gravel roads in Australia is for the haulage of bulk goods and therefore the roads have to withstand heavy traffic loads travelling at relatively higher speeds than passenger transporters. For these reasons, the risk of subgrade failure needs to be reduced.

As a practical matter for many minor low-volume roads that do not receive heavy traffic, gravel thicknesses of 100 to 150 mm are often specified, without design, based upon empirical performance history. If subgrade soils are reasonably strong and climate is not tropical, a 100 mm layer of compacted gravel is used. This thickness is also used for erosion control over very erosive soils such as decomposed granites or volcanic ash. Over poor soils and under wet conditions, a 150 mm layer of aggregate is place on the road. If rutting occurs, additional gravel is spread in the rutted areas. This approach is somewhat validated by approximate thickness design information seen in Figure 6-4, based upon US Army Corps of Engineers test track data (Keller et.al. 2011). Aggregate thickness is determined as a function of subgrade CBR and traffic (ESALs). This approach does not consider the costs of resurfacing the road or life-cycle costs.

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7 SELECTION OF SURFACINGS

7.1 Purpose and Scope

The design of thin surfacings is similar for all pavement design methods. It is not within the scope of the guide to discuss their design in detail, but the choice of surfacing is important for road performance. Summaries and recommendations will be made as to the pros and cons of each surfacing type. This chapter provides guidance on:

- factors to be considered in the selection of appropriate surfacings for any given pavement;
- the potential life of different surfacings and the factors that affect these, for example the skill of the contractor;
- surfacing construction and maintenance strategy to minimise lifecycle costs.

7.2 Description of Common Surfacings

The most common surfacings for LVRRs are thin bituminous surfacings as described in Table 7-1. In addition to thin bituminous surfacings, structural surfaces including 'discrete element' surfacings (e.g. concrete blocks, cobble stones, and hand-packed stone) all have a place for use on LVRs. Initial cost and availability of local materials are usually the constraining factors, but because of the structural value of such surfacing, overall pavement thickness can be reduced and the whole life costs may sometimes make these options favourable. The most common use is for semi-urban areas where marketing and trading takes place and where vehicle movements are unpredictable, and on sections that are very steep or otherwise difficult from an engineering point of view.

It must be pointed out here that the use of single application of bituminous surfacing (for example, sand seal, slurry seals and Single Surface Dressing also known as Single Chip Seal) will often turn out economically disastrous due to poor performance. A single chip seal is typically applied to a paved surface but a double chip seal is usually needed on an unsealed aggregate surface. The reasons accepted for their poor performance is due, at least in part, that any slight construction defect rapidly develops into a pothole whereas in double seals, apart from the increased durability due to increased thickness, the chances of defects occurring on the two applications at precisely the same point is relatively small. The application of a primer seal (as done in Australia, and Ghana) may minimize the problems.

Table 7-1: General characteristics of bituminous surfacings

Surfacing	Description and Characteristics	Expected life (years)
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 6-12 months, depending on traffic. Should then last for another 6 -7 years before another seal would be needed. • Especially useful if good aggregate is hard to find. • Very suitable for labour-based construction, especially where emulsions are used, and requires simple construction plant. • Need to be broomed back into the "worn" wheel tracks. There is an extended curing period (typically 8 – 12 weeks) between the first and second seal applications to ensure complete loss of volatiles and thus prevent bleeding. During this period, the sand may need to be broomed back into the "worn" wheel tracks.	Single seal 2-3 Double seal 3-6
Slurry Seal	• Rational design with both simplified and detailed approaches. • Consists of a mixture of fine aggregates, Portland cement, emulsion	Single

Surfacing	Description and Characteristics	Expected life (years)
	binder and additional water to produce a thick creamy consistency which is spread to a thickness of 5-15 mm. • Can be used on LVRs carrying only light traffic. More typically used for re-texturing surface dressings prior to resealing or for constructing Cape seals. • Very suitable for labour-based construction using relatively simple construction plant (concrete mixer) to mix the slurry. Thin slurry (5 mm) is not very durable; performance can be improved with the application of a thicker (15 mm) slurry or the application of a second seal after 6-12 months, depending on traffic. Should then last for another 5 -10 years. .	3-5 Double seal 5-8
Single Surface Dressing + Sand Seal	• Partly rational (surface dressing) and partly empirical design. • Consists of a single 13 mm or 9.5 mm surface dressing followed by a single layer of Sand Seal (river sand or crusher dust). • The primary purpose of the sand seal is to fill the voids between the chips to produce a tightly bound, close-textured surfacing. • Fairly suitable for labour-based construction and, when emulsion is used, requires relatively simple construction plant. • More durable than a Single Surface Dressing.	8-11
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 one or two layers of slurry. The primary purpose of the slurry is to fill the voids between the chips to produce a tightly bound, dense surfacing. • Fairly suitable for labour-based construction and, when emulsion is used with the surface dressing; can be constructed with relatively simple plant. • Produces a very durable surfacing, particularly with the 19 mm aggregate plus two slurry applications (life of 12 years – 15 years).	(13 mm + Single Slurry) 10-14
Double Surface Dressing	• Partly rational (surface dressing) and partly empirical design. • Usually consists of a single 19 mm or 14 mm surface dressing followed by a single layer of aggregate of 9 or 7mm. 14mm and 7 mm preferred. However, recent research has shown that the second application aggregate size should be about 1/3 of the aggregate used in the first application therefore 19 mm and 7 mm sizes are preferred. • The primary purpose of the second layer is to fill the voids between the chips to produce a tightly bound, close-textured surfacing. • Fairly suitable for labour-based construction and, when emulsion is used, requires relatively simple construction plant. • More durable than a Single Surface Dressing.	10-14
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 both labour-based construction and equipment-based	Single: 8-10 Single + sand seal: 10-12 Double Otta Seal 12-15

Surfacing	Description and Characteristics	Expected life (years)
	construction, Requires extended aftercare (replacement of aggregate and rolling).	
Cold Mix	• 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 roadbase and rolled. Thickness about 20 mm. • Very suitable for labour-based construction; requires very simple construction plant; reduces the potential hazard of working with hot bitumen; does not require the use of a relatively expensive bitumen distributor.	8-10
Sand Asphalt	• Empirical design. • Consists of 30 mm – 50 mm thick admixture of sand and bitumen, mixed at high temperature (130°C – 140°C) which is spread and rolled when the temperature has reduced to 80 degrees Celsius. • Performance not yet proven, so not often considered for use.	8-10
Thin Asphalt < 30 mm	• Rational design. • Consist of nominal 10 mm crushed aggregate mixed in asphalt hot mix plant and placed by a paver.	10-12

Before application of the first bituminous seal for new construction, a bituminous prime (known as 'penetration treatment in the US) must be applied to the substrate and allowed 3-5 days to penetrate and cure before application of the first seal.Selection of Appropriate Surfacings for LVRs

The choice of the appropriate surfacing type will depend on the relevance or otherwise of many factors:

- Traffic (volume and type),
- Pavement (type – strength and flexural properties),
- Materials (type, quality and availability),
- Environment (climate – temperature, rainfall),
- Operational characteristics (geometry – gradient, curvature, etc.),
- Safety (skid resistance - surface texture),
- Construction (techniques and contractor experience),
- Maintenance (capacity and reliability),
- Economic and financial factors (available funding, life cycle costs, etc.),
- Other external factors.

The suitability of various types of surfacings for use on LVRs, in terms of their efficiency and effectiveness in relation to the operational factors outlined above, is summarized in Table 7-2 Details of how to carry out the design of specific surfacings can be found in most country manuals and in guidelines such as Overby (1999), TRL(2000), SANRA (2007), and Austroads (2013).

Whilst not exhaustive, the factors listed in the table provide a basic format which 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.

Table 7-2: Suitability of various surfacings for LVRs

Attributes	Thin seal/phased strategy				Double/combination seal strategy							
	SSS	DSS	SIS	SSD	SSD+ SS	DSD	SOS	SOS+SS	DOS	*CS 13mm*	*CS 19mm*	CMA
Ease of design	***	***	***	*	*	*	*	*	*	*	*	**
Ease of construction	***	***	***	*	*	*	**	**	**	*	*	***
Service life					**	***	***	***		***	***	**
Suitability for LBM				**	**	**	*	*	*	**	**	***
Avoidance of Risk of poor maintenance					*	*	*	***	***	***	***	**
High skid resistance			***	***	*	***	*	*	*	**	***	*
Early road marking	*	*	***	***	*	***				***	***	***
Suitability for turning actions					*	*	**	**	**	**	***	***
Insensitivity to material quality	*	*	*				***	***	***			**
Sensitivity to gradients >8%					*	*				*	*	**

***	Very good	**	Good	*	Reasonable		Poor, not suited

Notes:

SSS-Single Sand Seal; DSS-Double Sand Seal; SLS-Slurry Seal; SSD-Single Surface Dressing; SSD+SS - Single Surface Dressing and Sand Seal capping; SOS-Single Otta Seal; SOS+SS - Single Otta Seal and Sand Seal capping; DSD-Double Surface Dressing; DOS-Double Otta Seal; CS-Cape Seal 13/19mm+ Single/Double SLS; CM-Cold Mix.

The type of bitumen or emulsion used will be largely influenced by the availability of different types in the area where the surface treatment is carried out, together with the associated costs. The choice of bitumen for spray treatments is also affected by the following requirements.

The bitumen must:

1) Be capable of being sprayed and wetting the road surface in a continuous film;
2) Not run off the road surface on the camber or form pools in local depressions;
3) Wet the chippings/aggregate and adhere to them at ambient temperature, the adhesion being strong enough to resist traffic forces at the highest ambient temperature;
4) Remain flexible at the lowest ambient temperature, neither cracking nor becoming brittle enough to allow traffic to remove chippings.

It is not normally possible to satisfy all these requirements fully; therefore, the choice of binder should give the best possible compromise.

Important factors (in no particular order) that affect the selection of thin bituminous surfacing include:

1) Traffic: The anticipated traffic volume and types of vehicles carried by the road; the higher the volume of heavy traffic the shorter the surfacing life;
2) Climate: Very high temperatures cause rapid binder hardening and extreme brittleness through accelerated loss of volatiles, while at low temperatures binders are also brittle leading to cracking or aggregate loss resulting in reduced surfacing life.
3) Pavement strength: Lack of underlying pavement stiffness will lead to excessive flexure, fatigue cracking and reduced surfacing life.

4) Base materials: Unsatisfactory roadbase performance including inadequate stiffness and shear strength and absorption of binder into certain base materials (e.g. pedogenic materials) will lead to reduced surfacing life.

5) Binder durability: The lower the durability of the binder, the higher the rate of its hardening, and the shorter the surfacing life.

6) Design and construction of surfacing: Improper design and poor construction techniques (e.g. inadequate prime, uneven rate of binder application or 'dirty' aggregates) will lead to reduced surfacing life.

7) Stone polishing: The faster the polishing of the stone, the earlier the requirement for resurfacing

8) Characteristics of the project: Whether it is new construction or resealing;

9) Environmental conditions of the site; Both shade and moisture on the surface can affect how emulsions "break".

10) Road geometry: Sharpness of bends and steepness of gradients;

11) Safety requirements;

12) Experience and skill of the contractor and consultant;

13) Reliability and capacity of future maintenance;

14) Quality and condition of the equipment. Chip spreader and oil distributor truck must be capable of applying uniform rates of materials. Oil distributer nozzles must be clean and apply the oil uniformly.

15) Available Funds: for the **initial construction and future maintenance**.

Table 7-3 discusses many chip seal issues and how they can be resolved.

Table 7-3: Resolving chip seal issues

Situation	Possible Resolution
Steep Grades (> 6-percent)	Use polymer modified or polymer modified high float emulsions or use hot asphalt cement
Tight Horizontal Alignment	Use polymer modified or polymer modified high float emulsions, or increase asphalt rate up by 0.05 gal/SY (0.23 l/m^2), or place a fog seal on the completed chip seal (CSS-lh mixed 50/50 with water at 0.10 to 0.15 gal/SY(0.45-0.70 l/m^2)), or choke, or fog seal w/choke the final surface, or more tightly control the traffic (speeds <15 mph) (24 km/h), or increase the percentage of fractured faces in the aggregate, or use hot asphalt cement.
Cross Slope > 6-percent	Use polymer modified or polymer modified high float emulsions or use hot asphalt cement or consider non-bituminous surfacing.
Heavy Tree Canopy or other situations where low temperatures and high humidity would deter evaporation of the standard emulsions	Use cut-back bitumens, or polymer modified or polymer modified high float emulsions, or increase asphalt rate up by 0.05 gal/SY (0.23 l/m^2);, or place a fog seal on the completed chip seal (CSS-lh mixed 50/50 with water at 0.10 to 0.15 gal/SY (0.45-0.70 l/m^2)),or choke, or fog seal w/choke the final surface, or more tightly control the traffic (speeds < 15 mph) (24 km/h), or increase the percentage of fractured faces in the aggregate, or use hot asphalt cement, or use Rapid Set Low Temperature emulsion (1)
Dusty Aggregate Chips	Use cutback bitumens or wash the aggregate, or use high float emulsion or use CMS-2, or pre coat the aggregate chips
Need Early Chip Retention	Use racked-in surfacing (see TRL, 2000), polymer modified or polymer modified high float emulsions or use hot asphalt cement
Non-uniform Surface	Sand seal the necessary areas a few weeks prior to chip sealing to provide a more uniform textured surface, or adjust the asphalt application rate longitudinally by the use of different size nozzles; note the aggregate rate can also be adjusted longitudinally by adjusting the chip spreader gates; both methods need close coordination between the inspector and the contractor to ensure the proper application rates
Gal = gallon and SY = square yard (1) Rapid Set Low Temperature emulsions are designed to “chemically” break at temperatures as low as 40 °F (4 o C). They still need warm temperatures (60 to 70 o F) (16 o C to 21 o C) to completely cure. 1 US gallon =3.79 litres , 1 SY = 0.84 m^2	

Source: modified from Keller et al., 2011

7.3 Selection of Surfacings Based on Materials Availability

In some circumstances, only one type of material may be locally available, and therefore this influences the choice of surfacings. Table 7-4 shows the most suitable surfacing options based on the major material type. In other circumstances the risk of overloaded vehicles using the road influences the choice of surfacing. This is shown in Table 7-5. Lastly, the risk of erosion of existing unpaved sections of road sometimes influences the type of surfacing, as well as the choice to surface or not add surfacing. This is the case in high rainfall areas or in areas of high road gradients where erosion potential can be very high, particularly in fine-grained but low plasticity soils. These are covered in Table 7-6 and Table 7-7.

Table 7-4: Surfacing selection based on material availability

	Engineered Natural Surface	Gravel Surface	Waterbround/Drybound Macadam	Hand Packed Stone	Stone Setts or Pavé	Mortared Stone	Dressed Stone/Cobble Stone	Fired Clay Brick Pavement: Un-/mortared Joints	Bituminous Sand Seal	Bituminous Slurry Seal	Bituminous Chip Seal	Cape Seal	Otta seal	Non-Reinforced Concrete
Economically available Materials	S01	S02	S03	S04	S05	S06	S07	S08	S09	S10	S11	S12	S13	S14
Crushed stone aggregate			✓	✓						✓	✓	✓		✓
Stone pieces/blocks				✓	✓	✓	✓							
Natural gravel		✓											✓	
Colluvial/alluvial gravel		✓											✓	
Weathered rock		✓												
Fired clay bricks								✓						
Clay soil								✓						
Sand					✓	✓	✓	✓	✓	✓				✓
Cement						✓		●						✓
Lime										✓				
Bitumen									✓		✓	✓	✓	
Bitumen Emulsion									✓	✓	✓	✓		
Notes: ● Cement for mortared joints only.														

Source, Cook et. al., 2013

Table 7-5: Surfacing Selection based on traffic and erosion regimes

PAVING CATEGORY	BASIC		STONE					BR			BITUMEN					CONC
	Engineered Natural Surface	Gravel Surface	Waterbound/Drybound Macadam	Hand Packed Stone	Stone Setts or Pavé	Mortared Stone	Dressed Stone/Cobble Stone	Fired Clay Brick Pavement: Un-/mortared Joints	Bituminous Sand Seal	Bituminous Slurry Seal	Bituminous Chip Seal (single)	Bituminous Chip Seal (double)	Cape Seal	Otta seal (single)	Otta seal (double)	Non-Reinforced Concrete
Traffic Regime: (See Table 7-5)	S01	S02	S03	S04	S05	S06	S07	S08	S09	S10	S11	S11	S12	S13	S13	S14
Light traffic	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Moderate traffic		✓	✓	✓	✓	✓	✓	✓				✓	✓	✓	✓	✓
Heavy traffic (overload risk)					✓		✓					✓			✓	✓
Erosion Regime (See Table 7-6)																
A: low erosion regime	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
B: Moderate erosion regime				✓	✓	✓	✓	✓				✓	✓	✓	✓	✓
C: High erosion regime					✓	✓	✓							✓	✓	
D: Very high erosion regime					✓	✓	✓							✓	✓	

Source, Cook et al., 2013

Table 7-6: Definition of Indicative Traffic Regime

Indicative Category	Traffic Description
Light	Mainly non-motorised, pedestrian and animal modes, motorbikes & less than 25 motor vehicles per day, with few medium/heavy vehicles. No access for overloaded vehicles. Typical of a Rural Road with individual axle loads up to 2.5 tonne.
Moderate	Up to about 100 motor vehicles per day including up to 20 medium (10t) goods vehicles, with no significant overloading. Typical of a Rural Road with individual axle loads up to 6 tonne.
High	Between 100 and 300 motor vehicles per day. Accessible by all vehicle types including heavy and multi-axle (3 axle +) trucks, Construction & timber materials haulage routes. Specific design methodology to be applied.

Table 7-7: Definition of Erosion Potential

Road alignment longitudinal gradient	Annual Rainfall (mm)			
	< 1000	1000 - 2500	2500 - 4000	>4000
Flat (< 1%)	A	A	B	C
Moderate (1-3%)	A	B	B	C
High (3-6%)	B	C	C	D
Very High (>6%)	C	C	D	D
A = Low; B = Moderate; C High; D = Very High				

Note: Areas prone to regular flooding should be classed as “High Risk” irrespective of rainfall.

Source, Cook et al., 2013

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8 ECONOMIC EVALUATION OF PAVEMENT DESIGN OUTPUTS

8.1 Background

Economic analysis is often an important component of the decision process relating to road investment. In general, the larger the road investment, the more likely that some form of economic appraisal will need to be undertaken to provide a justification for the investment. This is particularly true for investments funded by donors or carried out by governments in high income countries. Correspondingly, small interventions, for example relating to road maintenance, or the provision of and upgrading of very low trafficked roads will not usually be subject to an economic analysis. In high income countries a full economic analysis is very widely used to take account of passenger and vehicle time savings in the planning and design of interurban and urban road infrastructure. However, there is often much less clarity as to how economic analysis is used to select pavement design. This Rural Road Note provides guidance on a few economic tools that can be used to choose between available options or to show that a design option is economically viable.

8.2 Purpose and Scope

This Chapter considers alternative economic analysis tools and procedures for planning Low Volume Rural Roads. The process often starts with ranking to select which LVRRs to upgrade, therefore a methodology for ranking is discussed briefly. Life Cycle Cost Analysis (LCCA), and models such as the Road Economic Decision Model (RED), and the Highway Development and Management Model (HDM-4) are discussed. Key input data are identified, together with the use of different decision criteria such as the Net Present Value (NPV) and Internal Rate of Return (IRR). An example of the use of the RED model is given.

8.3 Ranking, and Cost Effectiveness Criteria

A simple approach of deciding whether to upgrade a road or road section to a paved standard is by considering how much is being spent on grading and re-gravelling annually. Traffic volume and environmental factors often influence the required number of grading and re-gravelling cycles. If a road is functionally important, it will be kept in good condition by timely grading and re-gravelling. Therefore, the expenditure on these roads is often a good indicator of which roads should be considered for upgrading and subsequent detailed economic analysis.

Nevertheless, selection of which low volume rural and feeder roads to upgrade is often planned and prioritised using formal “Ranking” or “Screening” procedures. Cost-Effectiveness and Multi-Criteria Analysis (MCA) are examples of these procedures. Although these methods are not primarily used to determine pavement design according to engineering criteria, they will inevitably influence the choice of interventions in different circumstances.

Different Ranking, and Cost-Effectiveness procedures have many formulations, however they are not deliberately designed to fit within a conventional economic framework. The procedures often include indicators or measures of social as well as economic demand, need or benefit. Compared with a conventional economic appraisal less attention is given to the precision of coverage of benefits. Sometimes the procedure will include a method of incorporating consultation in the selection and prioritisation of road investments.

An example of a Cost-Effectiveness criterion was proposed by Lebo and Schelling (2001) for very low volume roads. Here construction costs and population are the critical factors.

$$Cost\ Effectiveness\ Indicator\ of\ link(j) = \frac{Cost\ of\ Upgrading\ link(j)\ to\ basic\ access\ standard}{Population\ served\ by\ link(j)}$$

With this method improvement of links that have the lowest ratio is given the highest priority. In this example, it can be seen that road design solutions that achieve a basic access standard, at minimum cost for the greatest population, (for example through a ‘spot improvement’ approach), will be selected.

Multi-criteria Analysis (MCA) is often used to combine economic, social, environmental and other considerations in the final choice of alternatives for both major and rural road investment. For each characteristic, the different projects are assessed and put into rank order (e.g. 1st, 2nd, 3rd, etc). This process is then repeated for the other characteristics. Weights are then assigned to each characteristic and an overall score is obtained. The process is demonstrated below in Table 8-1. In this table, to achieve the desired result, the ranking is presented in reverse order, i.e. the highest number rank refers to the best. The ranking and weights assigned are typically subjective, so quality of the analysis depends a great deal on knowledgeable individuals and their experience.

Table 8-1: Example of Multi-Criteria Analysis

Criteria	Alternative 1			Alternative 2			Alternative 3		
	Rank	Weight (%)	Score	Rank	Weight %	Score	Rank	Weight %	Score
Economic evaluation	3	50	150	1	50	50	2	50	100
Environmental evaluation	2	30	60	3	30	90	3	30	90
Development	3	10	30	2	10	20	1	10	10
Public transport	3	5	15	2	5	10	2	5	10
Accessibility/ Severance	1	5	5	2	5	10	3	5	15
Overall score	-	-	260	-	-	180	-	-	225

Although both HDM-4 and the RED model are predominantly used as economic models, both have a Multi-Criteria Analysis facility to assist with planning.

8.4 Economic Decision Criteria (NPV, IRR, FYRR, NPV/C)

Economic road appraisal models summarise their final results through economic decision criteria. The different criteria may be used to meet different objectives.

- <u>Net Present Value (NPV)</u>. The NPV is calculated through discounting the costs and benefits of the project using an economic discount rate. (The economic discount rate is discussed later in the chapter.) A positive NPV is the criterion for a project being worthwhile or not. The larger the NPV the better the project. ***The NPV is the best criterion to choose between alternatives,*** provided there are not major uncertainties or budget constraint issues.
- <u>Internal Rate of Return (IRR).</u> The IRR (also referred to as the Economic internal Rate of Return, EIRR) is calculated through estimating the discount rate that equalises the cost and benefit streams. Provided the IRR is above the economic discount rate the project is viable. The higher the IRR the more robust the project is. The IRR is very good for project screening, particularly where there is limited information or major uncertainties. However, the IRR cannot distinguish between a large or small project. ***So, it cannot help to choose between mutually exclusive projects, such as whether to build a gravel or a paved road in the same location.***
- <u>First Year Rate of Return (FYRR).</u> The FYRR is calculated by dividing the first year of full net benefits by the construction costs of a project. The FYRR is used to see whether the project timing is optimal. So, if a project has a FYRR above the economic discount rate, then the timing of the project is optimal and the project can proceed. If, however, the FYRR is below the discount rate, even though the project may be economically viable in the long term, it would be better to postpone the start of the project.
- <u>The Net Present Value/ Construction costs (NPV/C).</u> The NPV/C is used when there are budget constraint issues. A set of projects may all be economically viable, however because of shortage of

funds, not all of them can be built. If projects are ranked in order of the NPV/C ratio, then by selecting the highest ranked project first and the next highest second and so on, it is possible to achieve the overall highest NPV for the limited budget.

It is possible to calculate the NPV and IRR of a net benefit flow very simply using a spreadsheet like Excel. So, the Excel expression: =IRR (E5:E25) will return the IRR of the column of numbers represented from E5 to E25. Similarly, the Excel expression: =NPV (0.06, E5:E25) will return the NPV of the same column discounted by a 6% discount rate. The example below illustrates the use of the NPV, IRR and FYRR.

8.5 Selection of the Appropriate Economic Tool

After ranking and selecting which roads to upgrade, the economic analysis tool to be used in the appraisal of LVRRs should be selected in this hierarchy:

1. A simple comparison of initial construction costs if two or more options that provide similar ride quality (roughness) are to be compared or the design traffic is less than or equal to 300,000 ESA or the total initial cost of the works is less than the equivalent of US $ 3 million. Country-specific thresholds take precedence over these limits;
2. If the design traffic is greater than 300,000 ESA or the total initial cost of the works is greater the equivalent of US $ 3 million, or there are significant differences in ride quality but the design traffic is less than 300,000 ESA, then conduct Life Cycle Cost Analysis (LCCA) analysis using a simple spreadsheet, RED or other tool. Include other indicators such as NPV, IRR, FYRR, NPV/C for comparison of options. Country-specific thresholds take precedence over these limits;
3. Use of the Road Economic Decision Model (RED) if the options to be compared result in differences in ride quality (roughness) if required by the financier at any level of investment;
4. Use of HDM-4 analysis for upgrading a large number of roads as part of an improvement programme or if required by the financier at any level of investment.

It should be noted that common options to be considered are often whether to maintain a gravel road or to upgrade it to a paved standard.

Economic analysis shall not be required for the following cases:

1. For spot improvement works to improve access or safety,
2. Roads that have been selected for improvement on the basis of fulfilling a socially important function such as access to health facilities or schools.

8.6 Life Cycle Cost Analysis (LCCA)

The fundamental approach of Life Cycle Cost Analysis is to minimise the long-term costs of providing the road pavement. Hence in most examples the design choice is based on estimating of construction and maintenance costs of alternative designs over the lifetime of the pavement -which may range from say 15 to 20 years. However, the analysis period can be up to 40 years. This is because the pavement structure will still have a salvage value at the end of its design life (15-20 years) and because other improvements such as culverts and lined drains will last more than 20 years. For most LCCA examples future maintenance costs are discounted using a planning discount rate so that a dollar's worth of maintenance costs is worth less in the future than today. (Discounting is discussed in more detail later in the text.)

The effect of road surfaces on user costs are not, in general, included in LCCA approaches. The US Department of Transportation (1998) and the UK Highways Agency (2006) take into account user costs associated with disruption during construction and maintenance but neither agency take into account the effects on road user costs resulting from differences in road pavements (e.g. road roughness). It should be noted that traffic disruptions do not apply to LVRRs in low income countries, unless the road in question is cut off. Both the US Department of Transportation (1998) and the Austroads Guide (2009) recognize that vehicle operating costs (VOCs) are likely to rise with rough road surfaces, however because of a lack of data for their respective countries they do not explicitly take VOCs into account in their analysis. Austroads confines Life Cycle Cost analysis to construction and maintenance. In their analysis of flexible road pavements, the South African Department of Transport (1996) minimises present worth of construction and maintenance costs into the future and assumes that the user costs in the alternatives will be the same.

It is worth pointing out that a Life Cycle Cost Analysis that omits user costs is very unlikely to favour a paved road solution when it is compared with a gravel road option. This is because of the construction and maintenance costs of gravel roads tend to be much cheaper than paved options. In such an example, the benefits of lower user costs associated with the paved road would be ignored. Hence it is not recommended to use the LCCA approach in comparing gravel and paved roads.

A detailed example of how to calculate and compare life cycle costs is provided by the US Department of Transportation.[1] The results of the analysis are shown in Table 8-2 and Table 8-3. In the example two alternatives are considered that involve different initial and subsequent agency and user costs. Table 8-1 shows the constant dollar costs of the two alternatives, while Table 8-3 shows how the future costs are discounted with a planning discount rate of 4%. A residual value is included at the end of planning period, after 35 years. The present values of the total life cycle costs are calculated and shown in the Table 8-3.

Table 8-2: An Example of Life Cycle Cost Analysis *(Source US DoT)*

	Alternative A Activities		**Alternative B Activities**	
Year	**Agency Costs ($)**	**User Costs ($)**	**Agency Costs ($)**	**User Costs ($)**
0	26,000,000	11, 000,000	20,000,000	8,000,000
12			6,000,000	10,000,000
20	15,000,000	30,000,000	6,000,000	16,000,000
28			6,000,000	28,000,000
35	(3,750,000)	(7,500,000)	(750,000)	(3,500,000)

The Year 0 costs include initial construction costs and the user costs associated with diversions that may be necessary due to construction. The Costs in year 12, 20 and 28 are the rehabilitation costs and associated user costs. Each alternative has a different service life. The (negative) values shown in year 35 are a residual value of the remaining service life of the investment, including estimates of both agency and user costs.

Table 8-3: Life Cycle Costs Analysis showing discounted costs

		Alternative A		**Alternative B**	
Year	**Discount Factor**	**Discounted Agency Costs $**	**Discounted User Costs $**	**Discounted Agency Costs $**	**Discounted User Costs $**
0	1.000	26,000,000	11,000,000	20,000,000	8,000,000
12	0.6246			3,747,582	6,245,970
20	0.4564	6,845,804	13,691,608	2,738,322	7,302,191
28	0.3335			2,000,865	9,337,369
35	0.2534	(950,308)	(1,900,616)	(190,062)	(886,954)
Total Present Value of Costs		31,895,496	22,790,992	28,296,707	29,998,567

In the example the alternative with the highest initial agency costs (Alternative A) is shown in Table 8-3 to give the lowest overall discounted costs and therefore the best option. This example shows the dangers of using only initial cost to evaluate alternatives and the advantage of using life-cycle discounted costs.

Residual (or Salvage) values may be incorporated into the analysis when a substantial economic value remains at the end of the analysis period. As is shown in the Life Cycle Costs Analysis above a negative cost (i.e. a benefit) may be incorporated at the end of the analysis period. However, this will need to be multiplied by the appropriate discount factor. When the discount rate is high, and the planning time horizon is long

[1] https://www.fhwa.dot.gov/asset/lcca/010621.pdf

then residual values will usually make little difference to the overall viability of the project. Nevertheless, the issue may be worth considering. As an example, for a 6% discount rate, the discount factor after 20 years is: 0.3305. If a 20% residual value is estimated for an investment then this would be the equivalent of reducing the investment costs by 6.6%.

8.7 The Road Economic Decision Model (RED)

The Road Economic Decision (RED) model was prepared for the Sub Saharan African Transport Program (SSATP) by the World Bank (World Bank 2006)[2]. RED is a spreadsheet-based model that is derived from HDM-4. RED is relatively simple to use and does not require specialist training. However, RED does require someone with engineering knowledge that is familiar with road roughness and typical interventions. RED is principally used for the evaluation of unpaved roads and can be used for upgrading to a paved road standard.

The main advantage of RED is that it calculates vehicle operating costs from simple input data[3]. RED does not include the road deterioration and works effects relationships (for example gravel loss equations) that are incorporated into HDM-4. Hence, unlike HDM-4, road condition is not forecast to change year-by-year. For the purpose of the model, maintenance costs must be averaged over the lifetime of the project and, road roughness must be assumed to be constant during the appraisal period for each alternative considered. Using engineering judgement, the user needs to select the levels of roughness associated 'with' and 'without' the proposed intervention. Because vehicle speed is associated with roughness, speed may also be used in the model, to check or determine, the appropriate roughness levels.

Like HDM-4, the model has an economic framework and can calculate decision criteria such as the EIRR and NPV. To increase flexibility and explore the cost consequences of different periodic maintenance treatments many consultants use the VOC and time savings of RED and incorporate the results into their own models. An example of how RED may be used is given below.

8.8 The Highway Development and Management Model (HDM-4)

The basic approach of the HDM-4 (World Bank 2000) differs from the LCCA approaches outlined above in that it explicitly takes into account the effect of different road pavements on road user costs. In effect it adopts a 'Total Transport Cost Savings' approach, in which savings in road user costs are compared with incremental investment and maintenance costs. The main factors affecting road user costs are vehicle speed (determined by road width and alignment, in combination with traffic volume) and road roughness. A substantial amount of research went into the development of the model in the 1970s to 1990s that was carried out in Brazil, Kenya, Caribbean and India. HDM-4 contains a wide range of modelled relationships for both paved and unpaved roads, covering road deterioration, maintenance effects, vehicle speeds and vehicle operating costs. As roads deteriorate over time with traffic and weather, so the roughness is forecast to increase and vehicle operating costs are forecast to rise. The model has an economic framework that includes decision criteria such as Economic Internal Rate of Return (EIRR) and the Net Present Value (NPV). Alternative road designs and maintenance treatments can be assessed in detail.

Despite the sophistication of the HDM approach there are particular concerns over the accuracy of the modelling of VOCs (Cundill et. al. 1997), furthermore vehicle fleets have changed substantially over time, without an up-date of the relationships.

HDM-4 is the main appraisal method recommended by the World Bank and other donor agencies and it is used particularly for high traffic roads. The model can also be used to explore the suitability and timing of different road maintenance interventions. So, for unpaved roads, gravel loss can be predicted from data relating to traffic, climate, gradient, and the properties of the gravel. For paved roads the onset of cracking,

[2] https://collaboration.worldbank.org/content/sites/collaboration-for-development/en/groups/world-bank-road-software-tools.html

[3] An updated version of the road user cost model, HDM4RUC Version 5.0.zip is also available at the same World Bank Road Software Tools website: https://collaboration.worldbank.org/content/sites/collaboration-for-development/en/groups/world-bank-road-software-tools.html

ravelling and potholing can be predicted for different road surfaces and layer thicknesses according to climate and traffic volumes.

However, to use the model requires at least a week's formal training together with a substantial familiarisation period.

8.9 Key Data Inputs into a Transport Economic Analysis

8.9.1 Traffic

Traffic patterns vary from location to location, day by day, and by time of year. In general, the higher the flow the less the variation. In some locations there may be a pronounced surge on market days. However, to complicate matters, the periodicity of markets will also vary. Public holidays, weekends, and the end of the month can also have pronounced effects. Many countries have carried out comprehensive day-by-day classified traffic counts throughout the year on the main road network. With these data it is possible to provide factors that show in broad terms how the traffic flow varies by vehicle type and from month by month. From these data traffic count data can be adjusted to estimate annual average flows. In the absence of other data, on rural roads it has been suggested that counts be taken two weeks a year, one week in the dry season, and one week in wet season (Howe, 1972). If the principal counting effort covers just day-time flows adjustments need to be made to take account of night time flows (say count for three nights). Care also needs to be taken to ensure that counting stations are representative of the road under investigation, and steps should be taken to avoid being located within a village or town, and away from important junctions.

Traffic growth rates can be established from longer term traffic counting programmes, held at the same location each year. Annual fuel sales can also provide an indication of traffic growth.

8.9.2 Vehicle Operating Costs (VOCs) and Time Values

To estimate vehicle operating costs in models such as HDM-4 or RED it is necessary to input the economic cost of vehicles, tyres, fuel, oil, crew and mechanic costs. In modelling spare parts consumption new vehicle prices are used, even if vehicles are imported into the country second hand. In the overall modelling of costs, it is assumed that there is a trade-off between depreciation and maintenance costs. All prices are in economic terms so all taxes and subsidies must be removed. Data such as distance driven per year, annual hours of use, the number of passengers, and service life are required together with gross vehicle weight.

Passenger time values are also required. Studies have been carried out to estimate time values in developing countries (Whittington and Cook, 2018). They are often estimated to be around a third to half of the wage rate. In Indonesia for example, passenger values of time per hour were found to be between one hundredth to one two hundredth of the monthly household expenditure, with higher values relative to expenditure (or income) for poorer people. (Hine et al, 2000).

8.9.3 Inflation

Throughout the planning time horizon of the economic analysis, all prices must be expressed in constant price terms. Hence inflation in prices, in the planning framework, is not permitted within the appraisal period.

8.9.4 Economic Prices and Standard Conversion Factors (SCF)

Both vehicle operating costs and construction and maintenance costs must be in economic price terms, excluding all taxes and subsidies, expressed at a given date. In some countries Standard Conversion Factors (SCF) are published and used to convert financial construction and maintenance costs into economic prices. Typically, the SCF might be around 0.85 for converting construction and maintenance costs.

It is not common to have a SCF for vehicle operating costs because the rate of taxation will vary significantly between components such as fuel, tyres and vehicles.

8.9.5 The Economic Discount Rate

The Economic Discount Rate, (sometimes referred to as a Social Discount Rate) is similar to, (but not the same as) a financial interest rate or 'Bank Rate' used by governments to borrow money. It represents a forecast decline in value over time; a pound, or dollar, is worth less in the future than it is today. This both represents a social preference, and the fact that funds can be invested to produce a higher return later. Over the last thirty years it was standard practice to adopt relatively high discount rates (typically around 10 to 12%) for development projects. However, in 2016 the World Bank issued new guidance. This suggests that for countries growing at around 3% per capita per year the rate should be 6%. The note points out that in the period 1990 to 2010 the annual average per capita growth for World Bank client countries was 2.5 %.

A higher rate may be justified for exceptionally high growth rates, and a lower rate where the long-term prospects for growth are limited (World Bank, 2016).

8.9.6 Construction and Maintenance Costs

Construction and maintenance costs vary significantly from country to country, location to location and over time, with variations in availability of materials, the efficiency of the economy, and factors such as fuel prices. However perhaps the most comprehensive source of information on construction and maintenance costs is the World Bank's Road Cost Knowledge System -ROCKS database, which has been built up from more than 3,000 records in 89 countries, over the period from 1995 to 2005 (World Bank, 2006)[4]. Examples of key cost components are shown in Table 8-4 (Note that the costs in the Table are now rather old and new up-to-date costs should be sought if the World Bank does not update them).

Although there have been concerns about the difficulties and costs of finding good quality gravel, nevertheless, gravel roads have been found to be substantially cheaper to build than paved roads. It is often the life-cycle cost analysis that show the benefits of upgrading gravel roads. Table 8-4 indicates that the initial cost of upgrading to a 6-metre wide bituminous two-lane road is approximately four to five times as expensive as upgrading to a 6-metre two-lane gravel road.

In recent years, through the development of low-cost sealed roads using local and less expensive materials, the cost of sealed roads has decreased whereas the cost of regular gravelling of a gravel road has not. Although this can substantially reduce pavement costs, nevertheless paved roads often remain expensive because much higher design standards, reflecting higher design speeds and needed safety standards, are invariably adopted for paved roads.

However, where a major decision on infrastructure needs to be made it is important that accurate up-to-date costing is made to inform the decision, whether an economic analysis is undertaken or not.

[4] https://collaboration.worldbank.org/content/sites/collaboration-for-development/en/groups/world-bank-road-software-tools.html

Table 8-4: Comparative costs from the ROCKS Database (version 2.3)

	(Average Intervention Costs US$/Km for 6 m wide roads)		
Work Type	**Africa**	**East Asia Pacific**	**World**
Routine Maintenance, Gravel road	1,617	604	788
Routine Maintenance, Paved	2,323	543	1,964
Light grading	205	-	110
Heavy grading	491	-	430
Regravelling	9,775	11,362	11,278
Slurry or Cape Seal	12,970	6,256	8,695
Single Surface Treatment	18,353	18,254	18,254
Double surface treatment	18,767	13,146	27,968
Asphalt Overlay 40 to 59 mm	51,332	52,438	68,183
Asphalt Overlay 60 to 79 mm	91,905	81,750	81,270
Asphalt Overlay 80 to 99 mm	127,548	86,316	112,285
Asphalt Overlay > 99 mm	181,992	114,365	157,360
Reconstruction Earth	17,830	-	17,724
Reconstruction Gravel	30,473	56,054	43,924
Reconstruction Bituminous	190,031	134,965	178,945
Upgrading to Earth 2L.	12,592	-	13,432
Upgrading to Gravel 2L.	56,605	27,802	49,636
Upgrading to Bituminous 2L.	254,578	165,754	222,707

Source: World Bank, 2006. Note: Data was drawn from World Bank Projects undertaken from 1995 to 2005. Prices are expressed in US$ prices for year 2000. Routine Gravel Road Maintenance includes grading, spot-regravelling, drainage cleaning, and grass cutting. Routine Paved Road Maintenance includes pothole patching, shoulder repairs, drainage cleaning, and grass cutting.

8.9.7 Road Roughness

Road roughness is a measure of the unevenness of the road surface. It is particularly important for road planning because it is a key determinant of vehicle operating costs and is a key input into the HDM-4 and RED models. The higher the road roughness, the higher the maintenance costs and fuel consumption of the vehicles. Roughness is measured by different instruments (e.g. laser profilometer, vehicle mounted bump-integrator) that are calibrated to the International Roughness Index (IRI). Roughness is measured in terms of metres per kilometre.

A paved road will normally have a roughness ranging from IRI 1 to 8 while the roughness of a gravel road will typically vary from IRI 4 to 16. A road in poor condition may have a roughness above IRI 20. A road with a roughness above IRI 25 is often considered to be 'undrivable'.

On gravel and earth roads roughness is strongly influenced by:

1) Maintenance policy;
2) Traffic volume;
3) Quality of the gravel. Large particles increase road roughness. Earth roads usually deteriorate faster than gravel roads. Thus for the same traffic volume and the same grading frequency they will tend to be rougher than the gravel road equivalent.

The HDM-4 model can be used to predict road roughness with different grading frequencies, traffic levels and gravel characteristics. The effects of grading frequency on roughness is shown in Figure 8-1.

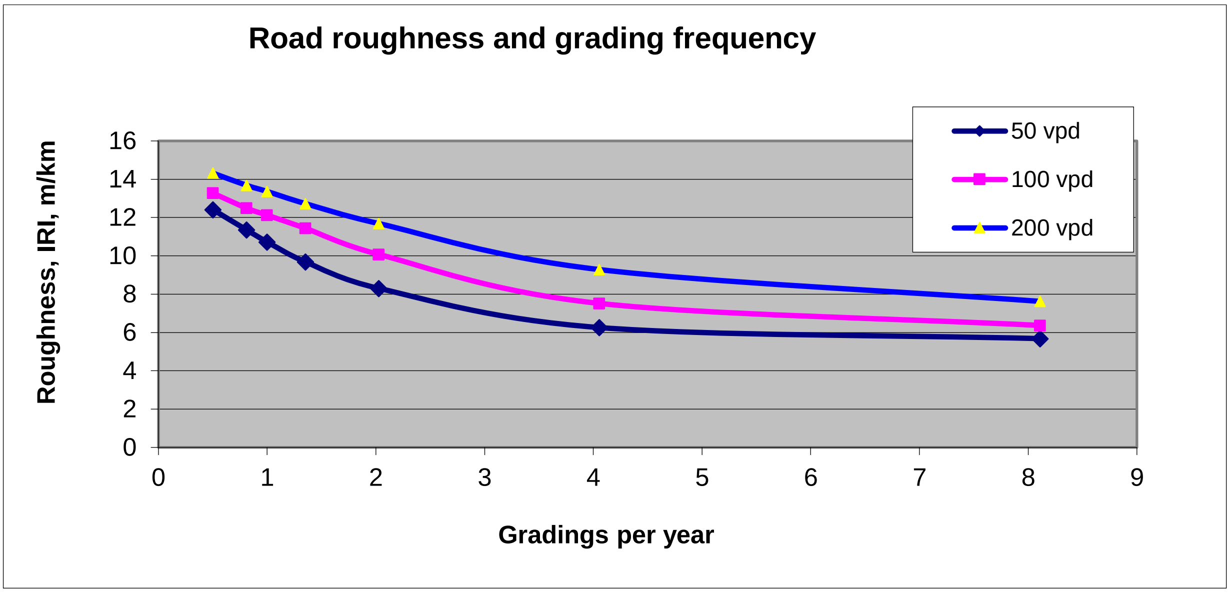

Figure 8-1-: Road Roughness and grading frequency for a gravel road

Vehicle speed can be used to help predict road roughness in terms of a comfortable driving speed on a flat straight road. The HDM-4 and RED models predict vehicle speed from roughness, but speed is also strongly influenced by gradient, road width, sight distance and road curvature. For example, on a road with a roughness of IRI 4 an unconstrained car's speed may be 90 km/h but it could be 40 km/h or less for a roughness of IRI 15.

8.10 An Example of Economic Analysis using the RED Model

The following hypothetical example is included to show how an economic analysis may be carried out by making use of the Road Economic Decision (RED) model. RED is a spreadsheet-based model that is relatively simple to use and does not require specialist training. In contrast training is required to use HDM-4 effectively.

The main advantage of RED is that it calculates vehicle operating costs and time savings from simple input data. RED does not include the road deterioration and works effects relationships that are incorporated into HDM-4. However, RED does require someone with engineering knowledge that is familiar with road roughness and typical interventions. For each year of the planning time horizon, HDM-4 will predict road roughness values from the road geometry, road surface condition, traffic and maintenance interventions. However, for RED, the user must input the expected average roughness value for each alternative considered. For unpaved roads, roughness is dependent on traffic volumes and gradings per year.

The RED model comprises a vehicle operating cost (VOC) module and the Main Module. The user firstly calculates the VOCs in the VOC module and then these data are input into the main module which can calculate the VOC and value of time benefits, for a specific investment, as well as estimating NPV and the IRR.

Many planners extract the results from RED's VOC module and time benefits from RED's main module to undertake additional analyses in their own spreadsheet. For example, it is easier to add periodic re-gravelling, or resealing, through a separate analysis, than within the RED module.

An example of the data input in the VOC module is given in Table 8-5. Here economic cost data and vehicle utilisation is input by the user. Economic prices are used, that exclude taxes and subsidies. The vehicle price refers to a new vehicle.

Once VOCs have been calculated the user inputs traffic data (Table 8-6) and the investment (Table 8-7) and maintenance costs (Table 8-8) and predicted roughness. Examples of the VOCs per km and traffic speeds are shown in Table 8-9.

Table 8-5: Vehicle input data

Item	Car Medium	Four-Wheel Drive	Bus Light	Bus Heavy	Truck Medium	Animal Cart	Motor-cycle	Bicycle
Economic Unit Costs								
New Vehicle Cost ($)	22,000	50,000	30,000	110,000	95,000	200	1,000	110
Fuel Cost ($/litre)	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0
Lubricant Cost ($/litre)	2.5	2.5	2.5	2.5	2.5	2.5	2.5	0
New Tire Cost ($/tire)	100	180	160	370	370	20	40	0
Maint. Labour Cost ($/hr)	3	3	3	3	3	2	3	1
Crew Cost ($/hr)	1.8	1.8	3.9	5.4	3.9	1	1.5	1.5
Cost per passenger/hr.	2	2	0.5	0.5	0.5	0	0.5	0.5
Interest Rate (%)	10	10	10	10	10	10	10	10
Utilization and Loading								
No. of Passengers	3	3	10	40	2	0	1.5	0.5
km driven per Year	25,000	45,000	90,000	90,000	60,000	3,000	1,5000	2,000
Hours Driven per yr.	500	1000	1200	2000	2000	1000	1000	300
Service Life (years)	12	15	9	15	15	10	10	10
GV Weight. (tons)	1.5	2.2	2	10	15	0.4	0.2	0.09

Table 8-6: Traffic data

Design values	
Vehicle Type	**Daily Traffic (veh/day)**
Car Medium	30
Four-Wheel Drive	48
Bus Light	36
Bus Heavy	48
Truck Medium	60
Animal Cart	12
Motorcycle	30
Bicycle	30
Total	294
Traffic Growth rate	3.5%

Table 8-7: Investment data

Road Intervention	Economic Cost ($ km)
Upgraded 2-lane Gravel Road	$ 70,000
Upgraded 2-lane Bitumen Road	$ 250,000

Table 8-8: Maintenance information

Surface Type	IRI	Maintenance Activity	Economic Cost
Base Earth /Gravel Road Poor Condition	15	Routine maintenance, (inc. grading 2x per year, and gravel patching)	$ 3,000 per km /yr
Gravel Road in fair condition	10	Routine maintenance, (inc. grading 6 x year) Re-gravelling every 5 years	$ 4,000 per km/yr $ 20,000 per km
Surface dressed road	4	Routine maintenance (what tasks?) Surface dressing every 7 years	$ 2,500 per km/yr $ 20,000 per /km

Table 8-9: RED Vehicle Operating Costs and vehicle speed output data

Rolling Terrain	Car Medium	4-Wheel Drive	Bus Light	Bus Heavy	Truck Medium	Animal Cart	Motor-cycle	Bicycle
Costs: $ Veh-km								
Paved, Roughness: IRI 04	0.25	0.32	0.30	0.64	0.72	0.33	0.06	0.09
Gravel, Roughness: IRI 10	0.32	0.46	0.37	0.89	0.99	0.57	0.07	0.16
Earth, Roughness: IRI 15	0.40	0.59	0.45	1.14	1.22	0.86	0.09	0.27
Speeds : Km/hr								
Paved, Roughness: IRI 04	91.15	89.78	81.92	78.59	71.25	3.31	86.85	18.02
Gravel, Roughness: IRI10	60.77	59.91	57.98	54.91	53.66	1.90	60.38	9.90
Earth, Roughness: IRI 15	41.45	40.84	40.49	36.88	39.57	1.26	41.42	5.85

For the economic analysis a degraded earth/gravel road (IRI 15) is upgraded to a fair gravel road (IRI 10) or upgraded to a bitumen road (IRI 4). The results of the economic analysis are shown in Table 8-10 and Table 8-11.The Tables show that although the fair gravel road gives a much higher EIRR, the paved road gives the highest NPV. <u>On economic grounds the paved road should be chosen because it gives the highest overall discounted benefits, as measured by the NPV.</u> Both the gravel road and the paved road have FYRRs (a measure of optimal timing) that are higher than the discount rate of 6%, hence there would be no advantage in postponing either option.

Table 8-10: Economic analysis for upgrading 10 km earth/gravel goad in Poor Condition (IRI 15) to gravel road in Fair Condition (IRI 10)

Yr.	Investment	Maintenance with Project	Maintenance without Project	VOC Benefits	Time Benefits		Net Benefits
1	700,000	0	0	0	0		-700,000
2		40,000	30,000	207,977	51,628		249,604
3		40,000	30,000	215,256	53,435		258,690
4		40,000	30,000	222,790	55,305		268,095
5		40,000	30,000	230,587	57,241		277,828
6		200,000	30,000	238,658	59,244		127,902
7		40,000	30,000	247,011	61,318		298,328
8		40,000	30,000	255,656	63,464		309,120
9		40,000	30,000	264,604	65,685		320,289
10		40,000	30,000	273,865	67,984		331,849
11		200,000	30,000	283,451	70,363		183,814
12		40,000	30,000	293,371	72,826		356,198
13		40,000	30,000	303,639	75,375		369,014
14		40,000	30,000	314,267	78,013		382,280
15		40,000	30,000	325,266	80,744		396,010
16		200,000	30,000	336,650	83,570		250,220
17		40,000	30,000	348,433	86,494		424,928
18		40,000	30,000	360,628	89,522		440,150
19		40,000	30,000	373,250	92,655		455,905
20		40,000	30,000	386,314	95,898		472,212
						EIRR	37.1%
			NPV at 6% disc rate				2,545,943
						FYRR	35.7%

Table 8-11: Economic analysis for upgrading 10 km earth/gravel road in Poor Condition (IRI 15) to paved road (IRI 4)

Year.	Investment	Maintenance with Project	Maintenance without Project	VOC benefits	Time Benefits		Net Benefits
1	2,500,000	0	0	0	0		-2,500,000
2	0	25,000	30,000	334,469	78,987		418,455
3	0	25,000	30,000	346,175	81,751		432,926
4	0	25,000	30,000	358,291	84,613		447,904
5	0	25,000	30,000	370,831	87,574		463,405
6	0	25,000	30,000	383,810	90,639		479,450
7	0	25,000	30,000	397,244	93,811		496,055
8	0	200,000	30,000	411,147	97,095		338,242
9	0	25,000	30,000	425,538	100,493		531,031
10	0	25,000	30,000	440,431	104,010		549,442
11	0	25,000	30,000	455,846	107,651		568,497
12	0	25,000	30,000	471,801	111,419		588,220
13	0	25,000	30,000	488,314	115,318		608,632
14	0	25,000	30,000	505,405	119,354		629,759
15	0	200,000	30,000	523,094	123,532		476,626
16	0	25,000	30,000	541,403	127,855		674,258
17	0	25,000	30,000	560,352	132,330		697,682
18	0	25,000	30,000	579,964	136,962		721,926
19	0	25,000	30,000	600,263	141,756		747,018
20	0	25,000	30,000	621,272	146,717		772,989
						EIRR	18.5%
				NPV at 6% disc rate			3,199,663
						FYRR	16.7%

8.11 Social Benefits, Road Safety and Environmental Costs and Benefits

8.11.1 *Social Benefits*

A transport cost-benefit economic framework is most appropriate when the alternatives considered provide year-round vehicle access. For most situations, where there is good vehicle access, and incomes are sufficient, then people will be willing to make a trip (to hospital, market or to visit friends) irrespective of normal fare levels. In this case the conventional transport cost savings approach represents a reasonable approximation of the benefits of the road investment and there is little pressure to include additional "social" benefits in the analysis. However, problems arise where income levels are low and where communities are inaccessible or roads are cut off for much of the year. In the extreme case where a new road is being introduced to a remote poor community, where, for example, in the "without project " case people have to walk twenty kilometres and carry sick relatives to hospital, but in the "with project" case they can travel by motor vehicle, trip purpose begins to matter. It can be a matter of "life or death" and the conventional valuation of transport user cost savings, is completely unrealistic. Similarly, without adequate vehicle access it is unlikely that social facilities such as schools, clinics, markets and other government services will be accessible to the local population.

Transport economic approaches are inevitably based on the volume of traffic, and do not explicitly adjust for income inequalities. Where social benefits are considered, in most instances the size of the population affected, rather than traffic volume, is a critical factor. Ranking approaches are often the only way of incorporating social benefits into road planning. Overseas Road Note 22 (TRL, 2004) discusses the issue in more detail.

Often the most cost-effective way of meeting social objectives and securing basic vehicle access for the rural population is through a "Spot Improvement" approach. Here identified trouble spots such as a missing or broken culvert, a slippery slope, a low-lying road subject to flooding will be treated rather than the whole road. This approach is discussed in Chapter 6, *Design of Unpaved Roads,* and by Lebo and Schelling (2001).

8.11.2 Road Safety

In general, there are two approaches to accommodating road safety in road design and planning. It may either be directly incorporated into economic analysis of road options or included in the final decision via a form of a MCA.

For example, in the HDM-4 Road User Cost Model programme the total costs of fatal, injury or damage only accidents may be calculated from models developed under the International Road Assessment Programme Methodology (iRAP)[5]. The user needs to define the vehicle fleet fatality rate and the serious injury rate per 100 million vehicle-km. Estimates of the value of life, or serious injury, need to be included, typically based on a ratio of GDP per head. The value of life to be used in economic decision-making is, of course, controversial. Nevertheless, stated preference techniques may be used to assess the Value of Statistical Life, i.e. to estimate the value that the public would spend to avoid the loss of one life.

8.11.3 Environmental Costs and Benefits

There are a wide range of environmental issues and some may be incorporated directly into an economic analysis; for example, alternative design options may be ranked through a MCA. It is sometimes possible to directly estimate the costs of mitigation to deal with the adverse consequences of road investment. For example, the costs of sound barriers, or the costs of relocation. These may be directly included in the costs of construction.

Both the HDM-4 model and the RED model calculate vehicle emissions of different gases (e.g. hydrocarbons, sulphur dioxide, particulates, lead etc). However, these have wide consequences in terms of premature deaths and illness from respiratory and other diseases, and also contribute to global warming. The effects on respiratory diseases predominantly occur in urban areas and will be affected by terrain and weather factors such as wind speed and rainfall, and hence are not easily incorporated directly in transport models. Nevertheless, both models can help provide some relative guidance on different solutions within a MCA framework.

Economic analysis should also include the costs and benefits of environmental mitigations such as dust control, addition of wildlife crossing structures, fencing, or relatively large fish passage culverts, noise barriers, landscaping and erosion control measures, hard surfacing such as pavers through a village, etc. The costs of these measures can typically be directly measured and incorporated into analysis. The benefits of these measures are often qualitative and more difficult to determine. However, every effort should be made to quantify these benefits and incorporate some value into the economic analysis.

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[5] http://irap.net/

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9 ANCILLARY FACTORS AND CONSIDERATIONS

9.1 Introduction

9.1.1 Background

The concepts and technology for cost-effective provision of LVRs have undergone significant advances over the last 20 to 30 years towards what now constitutes appropriate technical solutions for pavement and geometric design as presented in this RRN. Nonetheless, there are several aspects of LVR provision, which are intricately linked to successful LVR project implementation, that are not covered in-depth in this document. However, for most of these aspects of project implementation, through planning and procurement to construction and maintenance, there are relevant reference documents, Road Notes and manuals available which should be used – both by clients, designers and contractors – as required.

9.1.2 Purpose

The purpose of this chapter is to highlight some of the most important aspects that should be borne in mind during design and construction of low volume roads. Additionally, the chapter presents some of the most recent research findings for custodians of design manuals and specifications to consider during the next revision of their documents. It further highlights how research data from trial sections currently undergoing monitoring in some countries can be used to improve the pavement design methods presented in this guideline.

9.1.3 Scope

The scope of the chapter is limited to a discussion of the key aspects and the user of the guideline should seek further guidance from the documents included in the bibliography list.

9.2 Selected Geotechnical Engineering Considerations for LVRs

This section outlines guidance on aspects of ground improvement and slopes, embankments and cuttings for LVRs.

For all the pavement methods described in the preceding sections of this document, the aspects of ground improvement as well as slope and embankment considerations would essentially be the same, where required.

Geotechnical engineering considerations are specifically important in pavement design for LVRs for a number of reasons including but not limited to:

- Ground improvement offers a number of options for modifying soil along the road alignment. This offers potential reduction in cost by eliminating haulage and other costs associated with importing more superior construction materials.
- Undertaking proper ground improvement could potentially reduce the thickness of pavement layers. By enhancing the strength of subgrade layers, considerable reduction in thickness and class of material for the pavement layer(s) overlaying the subgrade could be achieved.
- Appropriate design of slopes, cuttings and embankments mitigates pavement failure associated with the failure plane of a landslide making its way into the pavement, which could compromise the integrity of the pavement structure and subsequently cause pavement failure or remove the entire road. Moreover, sufficient slope protection measures protect the road from slope failure-induced damage culminating from debris physically damaging the road, debris deposit in the side drains leading to drainage-induced failure and debris deposit on the road that could make the road impassable in addition to loss of lives and property.
- Incorporating adequate surface and subsurface drainage into a road design to prevent slope failures, prevent erosion and scour of structures, and minimize damage or loss of surfacing materials.

Additionally, properly sized and protected drainage crossing structure are need to prevent washouts and structure failures which often close a road.

However, it should be noted that from the economic perspective, LVRs offer relatively small user benefits compared to higher trafficked roads. Therefore, a lifecycle analysis is recommended to establish the feasibility of the use of extensive and costly geotechnical interventions to determine whether their benefits justify their cost. Appropriate interventions also depend on the need and importance of the road, whether a high standard or low-volume road, and factors such as whether there are reasonable alternative routes.

9.2.1 Ground improvement

Given that the design of LVRs optimises the use of locally available soil material as much as possible, ground improvement may be required in some cases to enhance the physical properties which affect pavement performance. In some cases, ground improvement may be undertaken to reduce the thickness of pavement layers and improve the soil strength to make it more suitable for supporting structures.

Additionally, ground improvement may be undertaken to achieve any of the following:

1) Improve drainage;
2) Improve permeability;
3) Improve compaction and soil strength;
4) Accelerate consolidation and shear strength gain;
5) Provide lateral support;
6) Improve workability of soil.

9.2.1.1 Identification of soils requiring ground improvement

Several country-specific design specifications recommend different materials requirements for embankment fill and pavement layers for different traffic and local ground conditions. These specifications provide guidance to the Engineer on the minimum allowable properties of materials requirements as obtained from standard tests of grading, Atterberg limits, strength and swell, among others (refer to respective Site Investigation Manual; Laboratory Testing Manual; and Specifications for Road and Bridge Works; or their equivalent). Where the properties of the materials fall below the specification requirements, the designer may modify the existing soil as opposed to importing new material that meets the requirements.

Some of the common situations where ground improvement may be considered as a practical and cost-effective alternative for LVRs include:

1) Expansive clays that undergo shrinkage and swell under varying moisture conditions;
2) Soils with high organic content;
3) Low strength soils (CBR < 3%) including swampy conditions;
4) Collapsible soils. These are predominantly loose silts and/or sands with traces of clay which undergo sudden and irreversible decrease in volume when loaded after wetting;
5) Saline subgrade soils with soluble salts and other material containing soluble salts whose aqueous form evaporates making the salts crystallise in the bituminous surfacing layer. This subsequently leads to pavement damage;
6) Micaceous soils with large quantities of coarse mica (muscovite);
7) Dispersive and erodible soils.

9.2.1.2 Ground improvement techniques

This section outlines the typical ground improvement techniques that are commonly used when dealing with ‘problem’ soils along the alignment of LVRs. For the different ‘problem’ soils encountered, the appropriate measure for implementation would depend on the extent of the problem soils, anticipated traffic loading, economics involved, available equipment, experience in implementing the measure, road environment, among others. In some cases, the ground conditions may necessitate localised or general replacement of the existing material (subgrade) prior to ground improvement.

Table 9-1 summarises the common ground improvement techniques that are applicable to different conditions.

Table 9-1: Summary of ground improvement techniques

Conditions	Technique	Application
Expansive clays[2]	Excavate expansive clays and backfill with inert material[1,2,3]	Reducing settlement
	Pre-wetting the soil for 2 – 3 months	Inducing equilibrium moisture content before construction
	Surcharge (increase height of the fill)	Suppressing heave
	Lateral and vertical moisture barriers	Minimising moisture variation within the pavement
	Lime stabilisation	Enhancing bearing capacity and other soil properties such as plasticity
	Geosynthetics	Enhancing strength reducing shrinkage and expansive stresses
Organics	Excavate and backfill with suitable material[1,3]	Minimising settlement (for shallow occurrence of organic material)
	Surcharge[5]	Reducing settlement
	Geotextiles	Strength enhancement
	Sand columns	Reducing settlement
Low strength soils	Removal and replacement with suitable material[1,3]	Minimising settlement
	Lime stabilisation	Enhancing bearing capacity and other soil properties
	Blending with gravel	Enhancing bearing capacity and improving other soil properties
	Geosynthetics	Enhancing strength and improving drainage
	"Do-nou" reinforcement bags (a technique developed in Japan for protecting soils of low-bearing capacity by encapsulating them in bags).	Reducing settlement; improving trafficability during rainy seasons
	Grout injection techniques	Enhancing bearing capacity
	Sand columns	Reducing settlement; improving drainage
	Wick drains	Dissipating excess pore water pressure that can develop under load applied to the soil
Collapsible soils	Blending with gravel	Improving the properties of the pavement layer material
	Lime admixture	Stabilising silty base and sub-base
	Cement admixture	Stabilising sandy base and sub-base
	High energy impact compaction	Densifying soil
	Stone columns	Increasing support capacity
Saline subgrade soils[4]	Addition of lime	Increasing the pH to suppress the solubility of the more soluble salts
	Use of impermeable surfacing	Minimising accumulation of salts in the pavement layers
	Removing micaceous soil layer	Avoiding compaction problems associated with muscovite materials

Micaceous soils	Lime or cement stabilisation	Minimising subsequent rutting
Dispersive and erodible soils[6]	Removing and replacing with non-erodible material[1]	Avoiding erosion
	Providing adequate drainage	Managing and redirecting water flows
	Compaction at 2 – 3% above Optimum Moisture Content to the highest possible density	Densifying soil
	Lime/gypsum treatment of soil	Reducing erosion through cationic exchange

Note:

1) Depth of excavation of material for replacement typically varies with extent of the unsuitable soil and the anticipated traffic loading over the pavement.
2) For traffic loading ≥ 0.3 MESA on highly expansive soil, excavate expansive soil up to 600 mm and place a 100 – 200 mm thick layer of sand, gravel or rock fill before the pavement layers. Excavation does not normally exceed 200 mm for traffic loading < 0.3 MESA.
3) Fill material above ground level should be constructed with side slopes of 1:2
4) Details can be found in guidelines for prevention and repair of salt damage to roads and runways that were developed by Botswana Roads Department in 2003 (based on research in southern Africa)
5) Surcharge heights would vary depending on the anticipated traffic loading on the pavement.
6) Refer to Elges (1985)

9.2.1.3 Input data for ground improvement analysis

In general, the geotechnical investigation conducted for design of the pavement layers, foundation of structures, retaining walls, cut, fill, etc. that will be supported by the improved ground are adequate for the design of most proposed soil improvement techniques. However, specific soil information may be emphasised depending on the ground improvement technique selected. For instance, in choosing the type of chemical stabiliser to be used, the particle size distribution of the non-modified soil as well as its Atterberg limits are normally used as the determining factors. Specific requirements for the site may also need to be evaluated.

9.2.2 Slopes, Embankments and Cuttings

This section covers geotechnical approaches, stability analysis and design considerations for slopes, cuttings, embankments as well as slope protection. They do not differ significantly from those for higher trafficked roads except that for LVRs, the available budget is often insufficient for costly geotechnical interventions. Nevertheless, in mountainous areas experiencing high rainfall and areas with problem soils, detailed geotechnical design for slopes, embankments and cuttings is often needed..

Slope and embankment failure may take a number of forms including but not limited to:

- Deep failure in original ground.
- Failure along the surface of the slope/embankment arising from erosion. This may lead to formation of gullies and subsequent slope failure.
- Failure within the slope/embankment, such as from oversteepening or lack of compaction
- Failure at the toe of the slope/embankment from surcharging a slope or inadequate drainage of a seepage area.

The above failures may occur in isolation or combination.

The performance of slopes, embankments and cuttings relies on adequate understanding of the subsurface and interpretation of the soil and rock characteristics. Table 9-2 provides a summary of the engineering properties and field and laboratory tests that are essential for slope and embankment design and stability analysis.

Table 9-2: Field and laboratory investigations for slope and embankment design and analysis

Slope/Embankment condition under investigation	Required information for analyses	Field testing	Laboratory testing
Settlement magnitude and rate	• Time-rate consolidation parameters • Compressibility parameters • Shrink/swell and degradation of soils	• Cone Penetrometer Test (CPT) • Settlement plates • Standard Penetration Test (SPT)	• Consolidation test • Organic content • Shrink/swell
Strength assessment	• Relative density • In situ strength	• CPT (for weak materials such as clays) • Standard Penetration Test (SPT) – for sand and silt • Plate load test	• Moisture-density relationship • Strength measurement
Slope stability	• Shear strength parameters • Unit weights • Pore water pressure	• Slope inclinometers • Vane shear (for soft to firm clays) • Piezocones	• Triaxial test • Unit weight • Direct shear test
Lateral pressure	• Horizontal earth pressure coefficients	• Pressuremeter	• Soil angle of internal friction from triaxial or direct shear test • Rock backfill angle of internal friction
Embankment fill material source evaluation (quality and quantity)	• General classification of soil/rocks for material assessment • Strength	• CPT • SPT	• Particle size distribution • Atterberg limits • Organic content • Relative density • Moisture-density relationship • CBR
Potential for subsidence (karst, mining, etc.)	• Geologic mapping including orientation and characteristics of rock discontinuities	• Geophysical testing • Rock coring (RQD)	• Slake durability
Liquefaction	• Liquefaction potential of saturated sands	• SPT • CPT (where piezocones are used) • Ground water level measurement	• Particle size distribution • Atterberg limits • Triaxial test

Slope stability is commonly analysed in terms of factor of safety (FS), which is defined as the ratio of resisting force to driving force. Some minimum FS values are recommended for use where a geotechnical designer has enough information to adequately define soil profile, slope geometry, soil shear strength and pore water pressure in a slope stability model. Larger FS should be used if there is significant uncertainty in the analysis input parameters. For general slope stability analysis of permanent cuts, fills, and landslide repairs, a minimum FS of 1.25 is recommended.

For slopes adjacent to but not directly supporting structures, a minimum FS of 1.3 is recommended. A minimum FS of 1.5 is recommended for slopes that support structures such as bridges and retaining walls. This increased FS also applies to slopes that do not directly support a structure, but whose failure could damage the structure. Exceptions to this include minor walls that have a minimal impact on the stability of the existing slope, in which case FS of 1.3 may be applied.

Where seismic analysis is conducted, a minimum FS of 1.1 is recommended for slopes involving or adjacent to walls and structure foundations. For other slopes (cuts, fills, and landslide repairs), a minimum FS of 1.05 is recommended.

Detailed information on slope stability analysis and design can be found in a number of geotechnical engineering documents (Samtani & Nowatzki, 2006; WSDOT, 2013; NYSDOT, 2014; TRB (Turner and Schuster, 1996, 2012).

9.3 Design Considerations for Embankments and Cuttings

9.3.1 General

Generally, embankments less than 5 m high with slopes not greater 1.5 Horizontal (H):1 Vertical (V) in areas of stable ground do not require as much detailed geotechnical investigation and analysis as embankments over 5 m high constructed on soft ground. However, in all cases, the design, basis of calculation and recommendations must be meticulously undertaken. Table 9-3 provides a guide of the recommended slope angles and H:V extent for different conditions. These slopes are based on geotechnical properties of the materials, observations, and experience, but flatter side slopes are often preferred for road safety purposes on low embankments.

Table 9-3: Recommended slope angles for different soil and rock conditions*

Material strength and type		Cut slope height (m)			Fill slope height (m)	
		< 5	5 - 10	10 - 15	< 5	5 - 10
Soil	Coarse-grained ferruginous residual soil	1H:1V (45°)	1.5H:1V (34°)	NA	1.5H:1V (34°)	2H:1V (27°)
	Fine-grained ferruginous residual soil	1.5H:1V (34°)	2H:1V (27°)	NA		
	Dense – medium dense (such as sands)	1H:1V (45°)	1.25H:1V (39°)			
	Medium dense – loose (such as silts and sandy silts)	1.25H:1V (39°)	1.5H:1V (34°)		2H:1V (27°)	3H:1V (°18)
	Loose (such as silts)	1.5H:1V (34°)	1.75H:1V (30°)	2H:1V (27°)	3H:1V (°18)	NA
	Soft to stiff (such as silty clays or clayey silts)	1.5H:1V (34°)	2H:1V (27°)			
	Expansive clays	2H:1V (27°)	NA	NA	NA	NA
Rock	Strong (such as slightly weathered basalt, well-cemented sandstone or limestone)	0.2H:1V (79°)	0.25H:1V (76°)	0.3H:1V (73°)	1.5H:1V (34°)	
	Weak (weathered basalt, poorly cemented sandstone and volcanic ash, weathered limestone and marl, pyroclastic rocks	0.5H:1V (63°)	0.7H:1V (55°)	0.8H:1V (51°)		
	Very weak (slightly weathered tuff, mildly weathered mudstone or marl)	1H:1V (45°)	1.25H:1V (39°)	1.5H:1V (34°)		
	Extremely weak (highly weathered tuff, marl/mudstone/shale)	1H:1V (45°)	1.5H:1V (34°)	1.75H:1V (30°)		
Note: Slope angles may vary depending on height of embankment and depth of cut. Stable slope angles are very particular to local conditions of soil type, rainfall, and site history. Local experience and judgment should be applied. Compound cut slope angles may be desirable, where steep cut slopes are made in underlying rock and soil, with flatter slopes used in overlying softer or more weathered soil.						

*Modified from Keller & Sherar (2003) and Ethiopian Road Authority (2017).

The following design considerations should be made for slopes, cuttings and embankments:

1) Slope failure occurs when the forces acting to cause failure exceed the forces resisting failure. The analysis therefore requires the application of a factor of safety (FS) to prevent the acting forces from causing failure.
2) The soil moisture condition (dry, partially saturated or saturated) affects the stability of slopes and embankments. The higher the soil moisture, the greater the magnitude of forces acting to cause failure. Therefore, the soil parameters used in design should be in accordance with the rainfall patterns (soil moisture) and drainage.
3) If seepage or a high groundwater table are encountered, drainage measures should be incorporated into the slope, including underdrains, surface drains, collection galleries, horizontal drains, etc.
4) Care should be taken to ensure that problem soils (expansive, dispersive, collapsible soils, etc.) are not used as embankment fill material. Where it is inevitable, appropriate soil improvement measures (as discussed in the section 9-2- should be implemented. Slope protection should also be carried out where required.

9.3.2 Slope Protection

The commonly applied slope protection measures for different failure scenarios are summarised in Table 9-4.

Table 9-4: Slope protection measures

<table>
<tr><th>Failure condition</th><th>Slope protection and stability measures</th><th>Drainage intervention</th></tr>
<tr><td>Erosion[1] and mass flow along the slope</td><td>Bioengineering[2,3] for slip surfaces within the depth of influence of grass and plant roots (max. 1-2 meters)
Small check dams for long erosion gullies and steep slopes
Revetment wall to protect the side drain
Masonry or stone pitching to protect slope from weathering
For rock slopes, spray concrete protection in highly fractured zones</td><td>Cut-off drain above the slope; it should be well-maintained to avoid blockage
Weep holes may be required where impermeable slope protection measures are used to prevent development of high water pressure in the slope by draining out ground water</td></tr>
<tr><td>In-slope failures</td><td>Reduce slope grade to appropriate level
Internal slope reinforcement (such as anchors, bioengineering, soil nailing, stone columns, rock bolts, etc.) to increase the resistance of the soil
For fill slopes, consider removing and replacing compact fill, reinforced soil slopes
Revetment wall for failures that are unlikely to recur
Retaining structures (such as gabion walls, crib walls, MSE walls, etc.); retaining structures should be last option for LVRs.</td><td>Subsoil drain where there is evidence of water seepage
Herringbone surface drainage where required
Deep horizontal drains to lower the groundwater table</td></tr>
<tr><td>Failure in ground beneath road and/or slope</td><td>Bioengineering[2,3] and Biotechnical Slope Stabilization
Retaining Structures
Consider re-alignment of road from unstable ground</td><td>Provide road surface and subsurface drainage</td></tr>
<tr><td>Removal of slope support due to erosion at the toe</td><td>Walls (gabion walls, dry stone walls, mortared masonry walls)
Rip-rap</td><td>None</td></tr>
<tr><td colspan="3">Notes:
1) Further erosion control measures may be taken according to slope angle (θ) as follows:
θ≤15°: No particular requirement
15°<θ≤35°: Biodegradable erosion control mat
35°<θ≤45°: Biodegradable erosion control mat at shallower angles or non-biodegradable erosion control mat
45°<θ≤55°: Non-biodegradable erosion control mat with wire mesh if needed
Θ>55°: Non-biodegradable erosion control mat with wire mesh, up to about 60° if ground conditions are suitable and the upslope catchment is small; thereafter, use hard surface cover
2) Bioengineering can be implemented through:
Turfing: Application of grass with developed roots onto the surface of the slope. As the grass grows, its roots extend into the soil thereby strengthening the slope surface. Use grass species with relatively deep roots on steep slopes.
Seeding (manual or hydro): Hydroseeding involves spraying an aqueous mixer of grass seed and fertiliser on the slope surface; for manual seeding, grass seeds are manually spread evenly over</td></tr>
</table>

the slope surface The grass seeds grow and their roots act as organic reinforcement for the soil on the slope

- Cuttings: placing cuttings of brush that will re-sprout in brush layering or as live stakes
- Contour hedges: planting grasses such as Vetiver, or brush, on slopes on contour in lines to strengthen the slope and catch erosion and rockfall.
- Tree planting: Strengthening effect is provided by deep roots of trees.
- Biotechnical solutions: Incorporating vegetation into physical structures, such as "live gabions".

3) Selection of plant and grass species is influenced by the type of roots (deep or spreading) required for the function; site conditions such as moisture, permeability, temperature, stoniness, nutrients in the soil, etc.
   Details of selection of slope protection techniques can be found in Table 12.3 of Overseas Road Note 16 (Transport Research Laboratory, 1997). Other useful resources on bioengineering and biotechnical slope stabilization include: Keller & Sherar (2003), Howell (2008), Hunt et al. (2008), Scott Wilson (2009),Salter et al. (2020), Gray and Leiser (1982), and Gray and Sotir (1996).
   Plant species that are normally used for slope protection include grasses and ferns, climbers, shrubs, small trees (≤ 3 m height), medium trees (taller than 3 m but with a small crown), large trees (taller than 10 m with a large crown). Pictures of typical plants under these categories as well as guidelines for selection of plant species are detailed in Technical Guidelines on Landscape Treatment for Slopes (Geotechnical Engineering Office, 2011).

Plant species used in bioengineering ideally should be local, native species for the region where being used. Pioneer species are often best for initial planting. It is often desirable to work with local nurseries, agronomy personnel, or local farmers to determine the best local species to use. Some of the common plants and their scientific names are shown in Table 9-5.

Table 9-5: Examples of common plant species for bioengineering slope protection

Climbers	 *Bougainvillea spectabilis*	 *Smilax glabra*
Herbs	 *Axonopus compressus*	 *Paspalum notatum*
Shrubs	 *Ardisia crenata*	 *Rhapis excelsa*
Trees	 *Cyclobalanopsis neglecta*	 *Machilus breviflora*

It is important to note that appropriate geotechnical design has to be undertaken for the slope protection measures listed above. Additionally, the following considerations regarding slope stability problems and solutions for LVRs should be made:

- Adequate roadside and sub-surface drainage should be ensured.
- Cause and extent of slope instability and erosion should always be determined.
- Effect of land use on slope stability should always be determined and appropriate measures taken.

- Side slopes as well as road and slope drainage features should be inspected regularly to identify potential problems.
- Adequate bearing capacity of the founding rock or soil should be ensured for retaining structures.

Maintenance as well as appropriate slope protection and reinforcement measures should be applied to avoid damage to the road infrastructure. This minimises accumulation of debris and rock fall within the shoulders and roadway.

9.4 Borrow Pit Management

9.4.1 Introduction

The identification and development of good sources of pavement construction material at regular intervals along the length of an LVR is essential for achieving cost effective construction and ongoing maintenance operations.

Up to 70% of the construction cost of a typical LVR may relate to pavement materials production and supply. Also, aggregate replacement costs are often as high as 60% of the maintenance costs of an unpaved road. There are therefore significant cost-benefits that can be achieved by implementing improved borrow pit management procedures and material supply strategies. Material sources need to have 1) adequate quantity of material and 2) satisfactory quality material, or material that can be modified to meet LVR specifications.

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 roadbase are of little use if this material is wastefully used in earthworks layers. Too often (and in particular for borrow pits located for LVRs), borrow pit excavation is carried out with only the plant operator present and no correct supervision. In many cases this results in good quality gravel getting contaminated and having to be spoiled. Good management of materials as shown in Figure 9-1 (including skilful supervision during all operations in the borrow pit) is therefore a critical operation in LVR construction. Zones of quality material should be identified, isolated, and excavated as useable material. Topsoil should be preserved for use in site reclamation.

An awareness of the potentially damaging effects (negative impacts) that borrow pits and quarries may have on the local environment and on the income of local farmers if the topsoil is not preserved is also required so that mitigating measures may be incorporated in the tender documents for enforcement during the construction operations. A Pit Development Plan and Reclamation (Restoration) Plan should be developed for any quarry or borrow pit to identify zones of excavation, working areas, stockpiles of rock and topsoil, access routes, etc. Also, the final shape of the site and excavated slopes should be shown. This plan offers control over development and reclamation of the site to minimize problems, dictate the most efficient way to develop the site, and show the condition that the site will be left in once exploitation is finished. Figure 9.2 shows an example of a Pit Development Plan used for a basalt quarry.

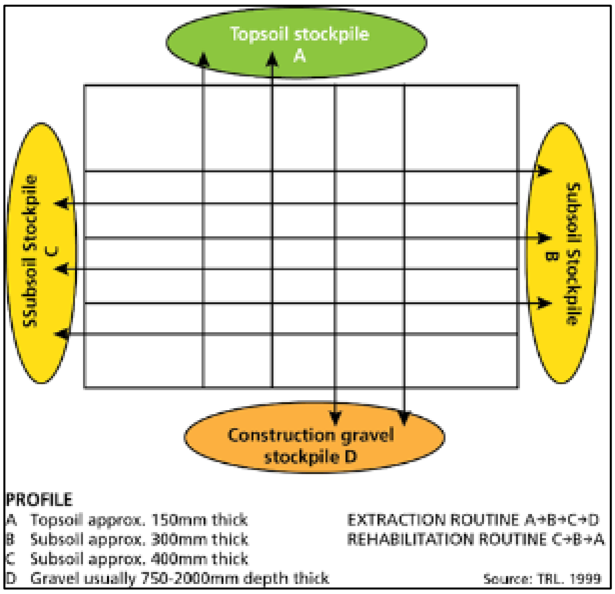
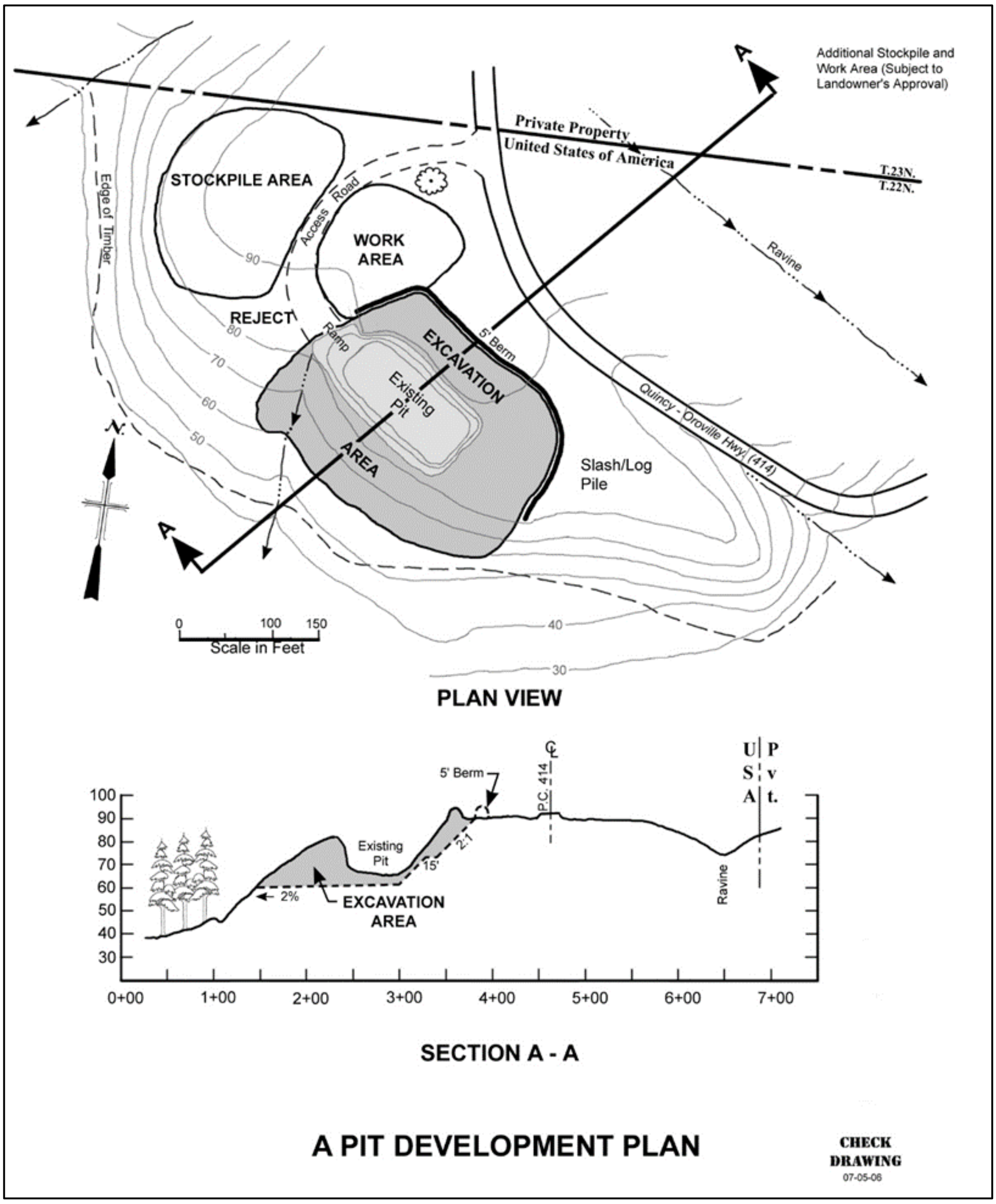


Figure 9-1: Recommended procedure for removal of overburden and stockpiling



Source: Keller, Unpublished "Minimum Impact Low-Volume Roads Manual".

Figure 9-2: An example of a Pit Development Plan

9.4.2 Main Factors

There are many factors to consider when locating and exploiting borrow pits:

1) Minimising environmental impact by reducing the dust and noise pollution for residents in the area. Other environmental impacts that should be considered for both locating a site and restoring a site include visual impacts, drainage, erosion control, proximity to streams, wildlife corridors and impacts, vector development in standing water, a water source, proximity to towns, etc.
2) Preparation of a borrow pit plan to ensure that topsoil and overburden is stockpiled for later use in restoration of the borrow pit and separation of different material types/qualities in different stockpiles to ensure that materials are not contaminated.
3) Drainage of the borrow pit to:
   - ensure accessibility and that the materials are not getting soaked
   - prevent accidental drowning of children

4) Ensure adequate security for people and livestock by fencing, if required, and avoidance of steep cut slopes.
5) Prevention of landslides where borrow pits are located on hill slopes.
6) 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. Use of these should be mandatory.
7) Ensuring adequate quantity and quality of material. Representative samples of the material should be tested. Drilling, excavation, or other subsurface investigation methods may be needed to determine the extent of the deposit, zones of suitable and unsuitable material, and quality of material.

9.5 Compaction

Effective compaction of the existing running surface of the earth and/or gravel road which is to be upgraded is one of the most cost-effective means of improving the structural capacity of the LVR pavement. A well compacted running surface;

1) possesses enhanced strength, stiffness and bearing capacity;
2) is more resistant to moisture penetration and less susceptible to differential settlement;
3) 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 carried out 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 9-3. 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.

Caution should however be exercised when compacting certain materials for sub-base and base layers (e.g. lateritic gravels, scoria [cinder] gravels) since excessive compaction can lead to significant breakdown of individual particles thereby leading to loss of shear strength. At a minimum, even if compaction is not specified, some compaction can be achieved with the wheels of construction equipment or haul trucks running over the soil. Equipment movement needs to spread out across the road surface, and soil or aggregate should be moist and near optimum moisture content.

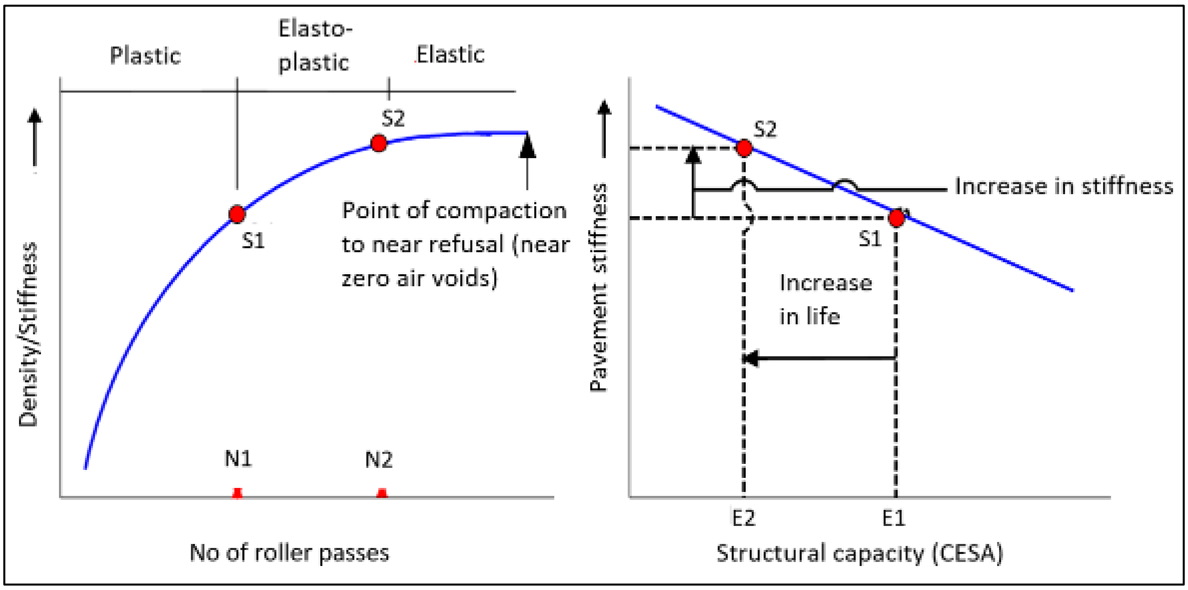


Figure 9-33: Benefits of compaction to refusal

9.6 Improvements in Pavement Design Charts

Under the ReCAP programme, several research projects have been carried out towards improving road provision and maintenance.

Most pertinent to pavement design is the project '*Development of guidelines and specifications for low volume sealed roads through back analysis*' also known as the Back Analysis project. The key finding of the project is that selected natural materials from which low volume sealed roads are made can often carry traffic in excess of 3 MESA. The climatic range over which the design catalogue is applicable has also been extended to a rainfall level of 2200 mm/yr. The project has provided proposed revised pavement design catalogues for the CBR-based pavement design method. Particle size distribution envelopes and Atterberg limits have also been revised following evidence gathered from performance of sections studied. The new limits allow for the use of a wider range of appropriate materials for low volume roads. Roads authorities will be able to apply these new limits in any future revisions of LVR manuals or designs. Lastly, the importance of providing and maintaining good surfacing/seals and drainage have once again been found to influence pavement performance significantly. These can be found in the project Final Report stored on the ReCAP website at http://research4cap.org/Library/Ottoetal-TRL-2020-BackAnalysisLVR-Phase3FinalReport-AfCAP-RAF2069A-200504.pdf.

9.7 Long Term Pavement Performance Sections

It is important to note that the pavement design methods presented in this guideline were all empirically developed. The performance of roads designed by these methods are therefore affected by the climate, the traffic characteristics, and the materials characteristics in relationship to those originally used for development of the methods. Therefore, the boundary conditions under which each method was developed and is applicable need to be clearly understood.

Under ReCAP, a project entitled: 'Capacity Building and Mentorship for the Establishment and Implementation of Monitoring & Evaluation Programmes on Experimental and Long-Term Pavement Performance (LTPP) Sections in Six African Countries and Myanmar', in-country capacity has been developed to carry out performance monitoring of several long-term pavement performance sections located on low volume roads. These roads consist of different materials, climates, and traffic characteristics. Low volume road pavements are generally designed for 15 to 25 years. The performance data from these roads will be vital for refining the empirical design methods included in the country manuals. This is more so once the roads have been in service for at least 10 years. The refinement will be in terms of pavement structures for various traffic and subgrade classes and the accompanying material and climatic characteristics.

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ANNEX 1 MATERIALS SELECTION TABLES

Table A-1: High plasticity materials

Types of Materials	Problems Associated with High Plasticity	Test Methods & Analysis to Quantify and Limit the Problem	Particular Characteristics that Strongly Influence Material Behaviour	Options for Improving Material Quality / Performance
• Coralline Gravels • Calcretes • Laterite Gravels • Quartz Gravels • River Terrace Deposits • Colluvial deposits • Clayey Sands • Weathered Limestones • Other Rippable Weathered Rocks • Deeply weathered soil and rock	**Engineering Properties** • Poor soaked CBR results (i.e. poor load bearing capacity) • Compaction problems • Susceptibility to loss of strength on wetting • **Pavement Defects** • Potholes • Rutting Cracking Expansion and swelling	• **Standard Tests:** • Liquid Limit (LL), Plastic Limit (PL), Linear Shrinkage (LS), Activity • Hydrometer Grading • Compaction and CBR • Grading Modulus (Particles retained on 2.00mm + 0.425mm + 0.075mm/100) • Plasticity Index (LL-PL) • Plasticity Product (PI x % passing 0.075 mm sieve) • Plasticity Modulus (PI x % passing 0.425 mm sieve) • Shrinkage Modulus (LS x % passing 0.425 mm sieve) **Special Tests** • Mineralogy • Chemical analysis, Volume change • Clay Activity • (PI / %finer than 0.02 mm)	• Restrict use according to climatic or road environment factors, (take note of potential flood or drought risks) • Restrict use according to traffic type & loading, e.g. low-volume roads only • Ensure protection from pavement saturation; • Good bituminous surface seal • Sealed Shoulders • Prevent upward migration of moisture (i.e. from underlying layers) Maintenance of waterproof seals	• Mechanical Stabilisation Blend with low plasticity material • Lime Treatment: typically suitable for base when: Passing 0.425mm min 15% Passing 0.075 mm 5-35% PI 10-25% Soaked CBR min 20% • Lime Treatment, typically suitable for sub-base when: Passing 0.425mm min 15% Passing 0.075 mm max 40% PI 10-30% • Cement Treatment: typically suitable for base when: PI max 25%, Passing 0.075 mm 5-35%, Soaked CBR min 20% • Cement treatment: typically suitable for sub-base when: Passing 0.075 mm max 40%, PI max 30% • Bitumen Treatment: typically suitable for clayey sand base when: Passing 0.075 mm 10-30% LL max 40%, PI max 15%

•		• Clay Mineralogy (% active and inactive minerals) • Permeability (time taken to absorb/ transmit water) • Relationship between Compaction, Moisture Content and CBR (i.e. sensitivity to loss of strength with increasing	•	• Chemical Treatment: may be considered as an alternative for cement or lime treatment but more difficult to construct and more expensive.

Source: Cook et al. 2002 – see reference in Chapter 3.

Table A-2: Poorly-graded materials

Typical Materials	Potential Problems	Standard Tests	Pavement Design to Accommodate Poor Grading	Options for Improving Material Quality / Performance
• Any natural granular deposit • Weak or poorly cemented materials (e.g. laterite, weak conglomerate) • Highly fractured competent rocks	**a) Coarse Gap Graded** Compaction Problems **b)** High % of voids, will result in high point loads, break- down of weaker particles and high permeability. Potential for collapse. **c)** Poor load bearing capacity (CBR) associated with poor particle interlock and internal friction. **d) Excess Fines Content** • Compaction Problems • Poor internal friction characteristics with poor interlock between larger particles (i.e. they "float") resulting in low CBR. • If fines are plastic the material will be prone to weakening on saturation **e) Uniformly graded** Poor compaction, low density and high permeability	• Particle Size Distribution, - • Sand Equivalence Testing • Grading Modulus and Uniformity Coefficient • Reject Index (% retained on 37.5mm sieve) • Coarseness Index • Fineness Index **a) Fine Materials** • Void ratio • Permeability • Level of compaction and air voids • Relationship between Compaction, Moisture Content and CBR	• Restrict use according to climatic factors and road environment • Restrict use according to traffic type and loading. • Select aggregate grading specification that allows optimum use of available material. For example, consider: Water bound macadam, Dry bound Macadam, Telford base • Ensure protection from pavement saturation if excess plastic fines.	• Mechanical stabilisation - Blend with materials that will improve grading characteristics • Screen. Removal of oversize usually feasible, but removal of sticky excess fines may be difficult when materials damp. • Crush and screen to create desirable grading, using one or more material sources • Lime or cement treatment: typically suitable for improving materials with excess fines

Table A-3: **Poorly-shaped materials**

Typical Materials	Potential Problems	Standard Tests	Pavement Design to Accommodate Poor Particle Shape	Options for Improving Material Quality / Performance
Foliated Metamorphic rocks (flaky and elongated) Alluvial Gravels and Sands (rounded to subrounded) Desert (Aeolian) Sands (rounded) Conglomerates (rounded to sub-rounded	Compaction Problems High % of voids, will result in high point loads that will cause breakdown of weaker particles and high permeability. May give poor CBR results (i.e. poor load bearing capacity) associated with poor particle interlock and internal friction.	• Flakiness Index, • Elongation • Average Least Dimension • Grading Modulus (P2.00mm + P0.425mm + P0.075mm/100). • Particle Size Distribution, Well graded materials are better able to tolerate poor shaped particles due to reduced point load contacts, % voids and permeability • % Crushed Particles • Visual inspection • Level of compaction	• Restrict use according to traffic type and loading. • Restrict use according to climatic and road environment factors.	• Mechanical Stabilisation (Blend with suitably graded materials that have good (cubical) particle shape). • Crush (Rounded materials will be improved by crushing) • Improve crushing procedures (flaky materials). The type of crushing apparatus (i.e. whether toggle jaw crusher or cone crusher etc) may significantly influence the proportion of flaky particles produced during aggregate processing. Select compaction plant that will limit break-down of carefully processed aggregate during pavement laying.

Source: Cook et al. 2002 – see reference in Chapter 3.

Table A-4: Materials with low particle strength

Typical Materials	Potential Problems	Standard Tests	Pavement Design to Accommodate Low Particle Strength	Options for Improving Material Quality / Performance
• Inherently weak rocks: • Marls and Limestones; Mudstone and Siltstones; • Weak Sandstones • Weak Tuffs; • Partially Weathered Rocks (all types) • Weak Natural Gravels: • Some calcretes; some laterites some silcretes; most volcanic scoria cinders); volcanic ash and pumice. • Weak manufactured materials • Weak Bricks - Weak demolition and industrial waste; d	• Change in grading characteristics during compaction. Including generation of excess fines. • Difficulty in identifying MDD and OMC • Compaction Problems. Difficulty in achieving required field density. • Low density will be linked to low CBR strength.	Aggregate strength properties as determined by selected tests • Aggregate durability decreases in strength within engineering time - c.f. Table 4.5. • Aggregate Crushing Value (ACV) • Los Angeles Abrasion (LAA) Value • Aggregate Impact Value (AIV) • 10 % FACT Aggregate Pliers Test (APT) • Aggregate Fingers Test (AFT). • Water Absorption Test • 10% FACT Wet and Dry Modified AIV procedures • Flakiness Index, • Elongation Index, • Particle Size Distribution, • % Crushed Particles • Average Least Dimension	• Restrict use according to traffic type and loading • Restrict use according to climatic factors - do not use in environments that will induce aggregate deterioration.	• Mechanical Stabilisation Blend with stronger materials that will improve grading characteristics • Crushing and Screening Removal of weaker particles in a mixed strength material. • Lime or cement treatment may significantly improve material performance. • Match construction plant and construction procedures with material characteristics.

		• Visual inspection • Aggregate durability decreases in strength within engineering time • Achievable pavement density		

Source: Cook et al. 2002 – see reference in Chapter 3.

Table A-5: Materials with poor durability

Typical Materials	Potential Problems	Standard Tests	Pavement Design to Accommodate Poor Particle Durability	Options for Improving Material Quality / Performance
• Marl; Limestone Mudstone; Shale; Argillaceous Sandstone • Poorly Cemented Rock Weak Tuffs; Weak Sandstones • Partially Weathered Rocks (all types) • Some basic intermediate igneous rock Basalt; Dolerite; Gabbro; Andesite	• Apparently strong pavement aggregates decompose in-service or during construction /stockpiling procedures. (the climatic influence is important). •	• Standard Tests: • Sodium Sulphate and Magnesium Sulphate Soundness Tests LAA • Texas Ball Mill Slake durability • **Mineralogical Analysis** See Table 4.6 • Degradation of engineering properties within the design-life of the pavement. • Unsatisfactory test results from investigations listed above.	• Restrict use according to climatic or road environment factors - do not use in environments that will induce aggregate deterioration. • Ensure protection from pavement saturation Good bituminous surface seal • Sealed Shoulders Prevent upward migration of moisture (i.e. from underlying layers) Maintenance of waterproof seals	• Mechanical Stabilisation: Blend with materials that will diminish overall degradation • Lime or cement treatment may inhibit durability problems but will require detailed investigation and possibly long-term field trails.

Source: Cook et al. 2002 – see reference in Chapter 3.

Table A-6: Petrographic assessment procedures

Petrographic Procedure	Procedure Description	Procedure Application
Aggregate: Qualitative Visual Examination	Record general character of aggregate sample including grading, texture, shape and rock type	A quick and rapid assessment
Aggregate: Quantitative Visual Examination	Sieve into separate size fractions and examine each fraction in terms of grading, texture, shape, rock type and mineralogy. Utilise additional procedures set out below as appropriate	Detailed petrographic procedure for identification of weak and/or unsuitable materials and recognition of potentially deleterious minerals.
Methylene Blue Value	Based on absorption of methylene blue by clay minerals. Powdered rock or fine soil sample suspended in solution and then titrated with methylene blue.	Rapid method of indicating the presence of deleterious clay minerals Does not give any indication of mineral type. May need additional fabric assessment work for more reliable results.
Binocular Microscopy	The use of plane light binocular microscope requires little sample preparation. Small hand-held microscopes can be used in the field.	A quick and straightforward method for the examination of soil fabric and texture of hand specimens. Photographs can be easily taken to support descriptions
Thin Section Microscopy	The traditional geological method of examination of mineralogy and fabric of thin sub-samples of hand specimens under both plane and polarized light.	May be used for the examination of fabric and as a means of establishing mineral composition by point-count techniques. Difficult to make sections in friable materials. Possible to take photographs.
Scanning Electron Microscope (SEM)	Utilises a focused beam of electrons to scan a specially prepared sample. Some electrons back scattered others produce secondary electrons. Patterns can be captured on film for observation.	Needs careful operation to achieve meaningful results. Most useful in finer grained soils. The use of stereoscopic photographic pairs of photographs increases the effectiveness of interpretation.
X-Ray Diffraction (XRD)	Utilises the identifiable diffraction angle that X-rays make with differing minerals. Powdered samples may be oriented, non-oriented, dried, glycolated or heated to aid identification.	Widely used in the identification of tropical soil mineralogy. By itself this method is only semi-quantitative. Cannot identify non-crystalline clay minerals
Thermal Analysis	Based upon whether the thermal reaction, which occurs as a clay mineral is heated is exothermic or endothermic and an interpretation of the resulting diagram.	These methods may be usefully employed in the study of clays and may provide useful mineralogical information in conjunction the methods listed above.

Source: Cook et al. 2002 – see reference in Chapter 3.