REPUBLIC OF GHANA

MINISTRY OF ROADS AND HIGHWAYS

# MANUAL FOR LOW VOLUME ROADS

# PART B - MATERIALS, PAVEMENT DESIGN AND CONSTRUCTION

2019

# MESSAGE FROM THE MINISTER

The mandate of the Ministry of Roads and Highways (MRH) is to provide a reliable and affordable road transport system that facilitates the socioeconomic development of Ghana. Effective and efficient road design, construction and maintenance is a *sine qua non* for achieving this. If high volume roads are the arteries and veins of the country, facilitating the free flow of the nation's socio-economic lifeblood, then low volume roads are the capillaries, extending that flow to the village level. Low volume roads facilitate travel that directly impacts public access to health, education and other essential services in rural areas, as well as the transport of goods that stimulates economic development at both the local and the national level.

MRH recognises the need for the practical application of sound research to its operations. For this reason, in 2014 it joined the African Community Access Partnership (AfCAP) under the Research for Community Access Partnership (ReCAP) funded by UK Aid through the Department for International Development (DFID). The Ministry has since then benefited immensely from global research on rural transport access and mobility. This design Manual is one result of, and a testament to, the Ministry's collaboration with AfCAP.

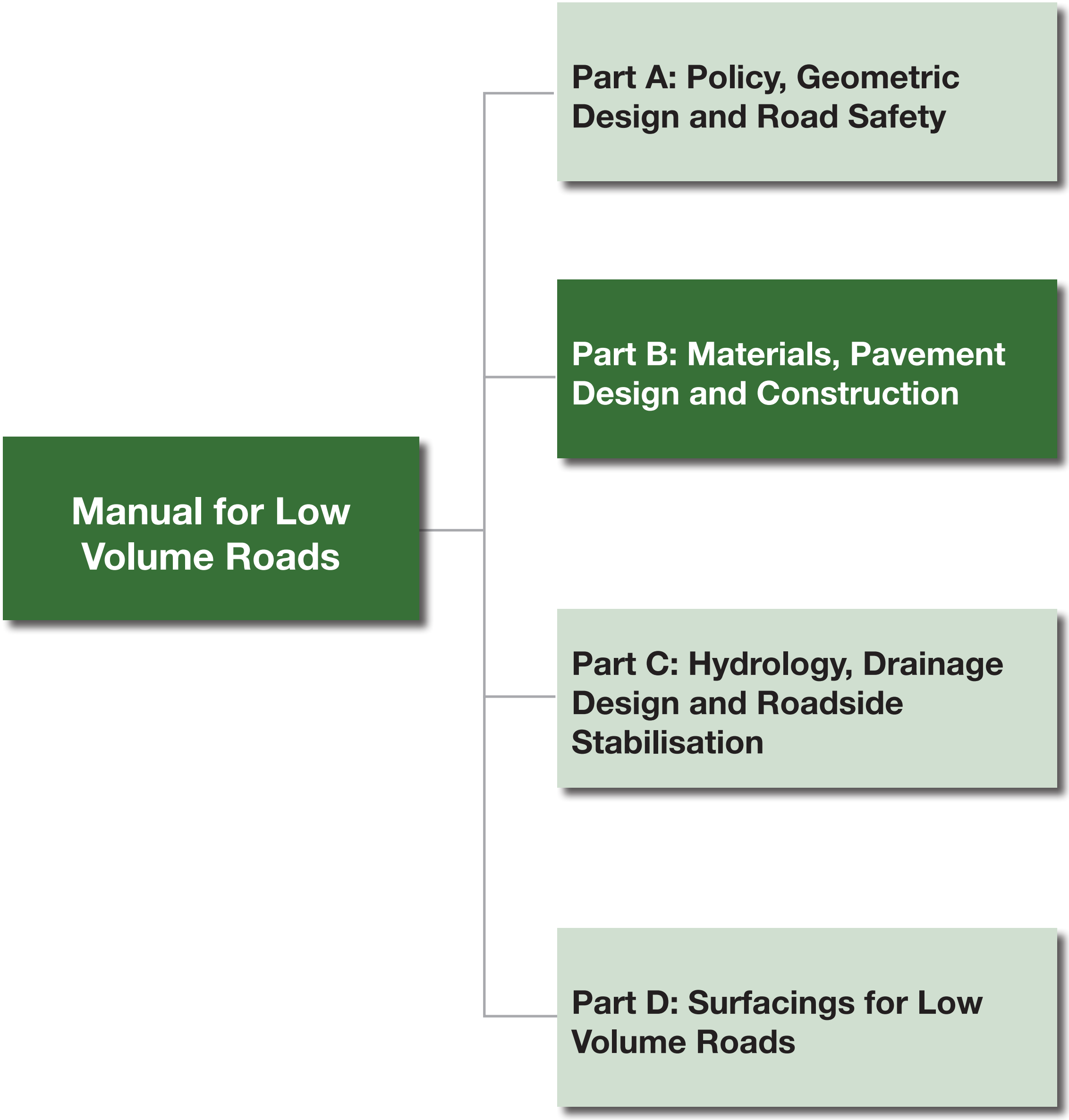
The Manual provides a basis for constructing, rehabilitating, or upgrading low volume roads in a manner that draws on international good practice, yet is relevant to the Ghanaian context. As such it constitutes an essential point of reference for students, or experienced practitioners with a professional interest in achieving value for money in the provision of such roads in Ghana. The Ministry will continue in such pursuits, in ensuring that the future of rural transport infrastructure remains sound through proper designs, construction and maintenance.

On behalf of the Government of Ghana I would like to thank UK Aid through DFID for its support to the Manual's preparation process. I would also like to thank the Project Management Unit of ReCAP and Civil Design Solutions for their role in managing the project.

I recommend this Manual, and am confident that it will provide the essential information and guidance needed for the sustainable provision of appropriate low volume roads that will meet Ghana's growing need for rural travel and transport.

**Hon. Kwasi Amoako-Atta,**
Minister of Roads and Highways

# PART B


Manual for Low Volume Roads
Part A: Policy, Geometric Design and Road Safety
Part B: Materials, Pavement Design and Construction
Part C: Hydrology, Drainage Design and Roadside Stabilisation
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# FOREWORD

The length of low volume roads in Ghana has since the year 2000 been increasing steadily, mainly as a result of changes in settlement patterns, increased agricultural activities and urban sprawl. This calls for a change in the approach to managing the network, with a focus on sustainability in line with the UN Sustainable Development Goals (SDGs).

A sizable proportion of all low volume roads in Ghana are managed by the Department of Feeder Roads (DFR), which accordingly has led the way in formulating standards, manuals and procedures for the design and construction of such roads. There was however no single consolidated design manual for low volume roads as DFR had in some cases relied on Ghana Highway Authority (GHA) standards, which are generally more appropriate for High Volume Roads. The Ministry recognises that the design of low volume roads requires unique attention, hence its support for the proposal by DFR to the Project Management Unit of the Research and Community Access Partnership (ReCAP) for the development of a design manual specifically for low volume roads.

This Manual has been through a robust process to ensure that it is fit for purpose. It draws on the expertise of both international and national specialists, and takes account of the latest relevant research findings, making every effort to ensure its relevance to the needs of our practitioners. A series of stakeholder workshops resulted in a range of perspectives being taken into account, from both the public and the private sector, and the initial complete draft has been subjected to a peer review. Nevertheless, it is recognised that there is no such thing as "perfect" guidance, so associated mechanisms are in place to ensure that sector performance continues to be monitored, and that further updates and improvements to the Manual can be made where necessary.

**Manual Updates**

Significant changes to criteria, procedures or any other relevant issues related to new policies or revised laws of the land or those that are mandated by the Government of Ghana, GHA, DFR or EPA will be incorporated into the Manual from their date of effectiveness. Other minor changes that do not significantly affect the whole nature of the Manual will be accumulated and made periodically. When changes are made and approved, new versions incorporating the revision will be issued.

All suggestions to improve the Manual should be made in writing to the Director of the Department of Feeder Roads.

It is my fervent hope that you find this Manual useful and make every effort to make use of it

**Ing. Edmund Offei-Annor,**
Chief Director of Ministry of Roads and Highways

# PREFACE

DFR has over the past decades led in managing a significant proportion of low volume roads in Ghana. These roads are used by different types of vehicles and non-motorised transport for providing various services ranging from the provision of basic access to transporting agricultural produce. They generally carry fewer than 300 vehicles per day or one million equivalent standard axles over their design life.

DFR has used various standards and codes for the design of the different classes of roads under its jurisdiction. These include the American Association of State Highways and Transportation Officials (AASHTO) design manuals, Transport Research Laboratory (TRL) Overseas Road Notes, and Ghana Highway Authority (GHA) Design Manuals, among others.

The development of the aforementioned manuals drew on technologies and practices emanating from Europe and the USA some 50 years ago. While these “standard” approaches may still be appropriate for much of the main trunk and regional road network, they remain overly conservative, and hence unnecessarily expensive, for application on much of the country’s low volume roads.

The first significant attempt to provide an alternative to the use of the aforementioned standards and manuals was undertaken with the support of DFID around the millennium. This saw the production of design manuals, construction pocket hand books and other useful design tools tailored for low volume roads.

The commencement of African Community Access Partnership (AfCAP) project in Ghana in 2014 led DFR to prioritise the development of a design manual for low volume roads that will consolidate the different existing manuals, while drawing more fully on experience gained.

This Manual will assist in developing optimal designs that use locally occurring natural resources, encourage the use of labour-based construction methods where appropriate and ensure value for money. It is for use as a point of reference for engineering and allied practitioners alike and serves as an excellent guide for the design of low volume roads.

I am grateful to UK Aid working through DFID, to the Project Management Unit of ReCAP, to the consultants from Civil Design Solutions, and to the technical staff of DFR and MRH for the immense support they have provided in ensuring the coming into fruition of this Manual. It is my fervent hope that it will change the face of low volume road design in Ghana.

**Ing. Bernard Badu**

Director of the Department of Feeder Roads

# ABBREVIATIONS, ACRONYMS AND INITIALISMS

| | | |
|---|---|---|
| > | : | Greater than |
| < | : | Less than |
| % | : | Percentage |
| | | |
| AADT | : | Annual Average Daily Traffic |
| AASHTO | : | American Association of State Highway and Transportation Officials |
| AC | : | Asphaltic Concrete |
| ACV | : | Aggregate Crushing Value |
| ADT | : | Average Daily Traffic |
| AfCAP | : | Africa Community Access Partnership |
| AIV | : | Aggregate Impact Value |
| ALD | : | Average Least Dimension |
| ASTER | : | Advanced Spaceborne Thermal Emission and Reflection Radiometer |
| ASTM | : | American Society for Testing and Materials |
| | | |
| BC | : | Binder Course (Base Course) |
| | | |
| CBR | : | California Bearing Ratio |
| CERSGIS | : | Centre for Remote Sensing & Geographic Information Services |
| CIC | : | Cast *in situ* Concrete |
| CS | : | Cobblestone |
| CSIR | : | Council for Scientific and Industrial Research |
| $C_u$ | : | Shear Strength (undrained) |
| | | |
| DBM | : | Dry Bound Macadam |
| DCP | : | Dynamic Cone Penetrometer |
| DES | : | Discrete Element Surfacings |
| DFR | : | Department of Feeder Roads |
| DFID | : | UK Government's Department for International Development |
| DN | : | DCP Number (penetration rate in mm/blow) |
| DSN | : | DCP Structure Number. DSNx is the total number of blows of the DCP required to penetrate the pavement to a total depth of x mm from the surface. |
| DS | : | Dressed Stone |
| | | |
| EF | : | Equivalence Factor |
| e.g. | : | For example, (abbreviation for the Latin phrase exempli gratia) |
| EIA | : | Environmental Impact Assessment |
| EMP | : | Environmental Management Plan |
| EOD | : | Environmentally Optimised Design |
| EPA | : | Environmental Protection Agency |
| ERA | : | Ethiopian Roads Authority |
| esa | : | equivalent standard axles |
| | | |
| FACT | : | Fine Aggregate Crushing Test |
| FMC | : | Field Moisture Content |
| | | |
| GDEM | : | Global Digital Elevation Model |
| GHA | : | Ghana Highway Authority |

| | | |
|---|---|---|
| GM | : | Grading Modulus |
| GPS | : | Global Positioning System |
| GWC | : | Gravel Wearing Course |
| HPS | : | Hand Packed Stone |
| HVR | : | High Volume Road |
| | | |
| i.e. | : | That is (abbreviation for the Latin phrase *id est*) |
| ILO | : | International Labour Organisation |
| InSAR | : | Interferometric Synthetic Aperture Radar |
| IRI | : | International Roughness Index |
| IRR | : | Internal Rate of Return |
| | | |
| kip | : | kilopound. 1 kip = 4.448 kN |
| km | : | Kilometre |
| km/h | : | Kilometres per hour |
| | | |
| lat | : | Latitude |
| LBM | : | Labour-Based Methods |
| LCC | : | Life Cycle Cost |
| LEF | : | Load Equivalency Factor |
| long | : | Longitude |
| LVR | : | Low Volume Road |
| | | |
| m | : | Metre |
| $m^3$ | : | Cubic metres |
| M | : | Million |
| MAP | : | Mean Annual Precipitation |
| MCB | : | Mortared Clay Brick (fired) |
| MCS | : | Mortared Cobblestones |
| MDD | : | Maximum Dry Density |
| MDS | : | Mortared Dress Stone |
| mod | : | Modified |
| MPBS | : | Maintenance Performance and Budgeting System |
| mm | : | Millimetre |
| MMS | : | Maintenance Management System |
| MPa | : | Megapascal (a unit of pressure equal to 1000 kilopascals (kPa) |
| MS | : | Mortared Stone |
| MSSP | : | Mortared Stone Setts or Pavé |
| | | |
| NCT | : | National Competitive Tendering |
| NGL | : | Natural Ground Level |
| NGO | : | Non-Government Organisation |
| NPV | : | Net Present Value |
| | | |
| OD | : | Origin-Destination |
| OMC | : | Optimum Moisture Content |
| ORN | : | Overseas Road Note |
| | | |
| PI | : | Plasticity Index |
| PM | : | Plasticity Modulus |

| | | |
|---|---|---|
| PP | : | Plasticity Product |
| PSD | : | Particle Size Distribution |
| PV | : | Present Value |
| QA | : | Quality Assurance |
| | | |
| RC | : | Reinforced concrete |
| RED | : | Low Volume Road Economic Decision model |
| Ref | : | Reference |
| RI | : | Roughton International |
| | | |
| SADC | : | Southern African Development Community |
| SAR | : | Synthetic Aperture Radar |
| SBL | : | Sand Bedding Layer |
| SDMS | : | Surfacing Decision Management System |
| SME | : | Small and Medium scale Enterprise |
| SN | : | Structural Number of the pave |
| SNC | : | Modified Structural Number of the pavement |
| SID | : | Spot Improvement Design |
| SSP | : | Stone Setts or Pavé |
| | | |
| TLC | : | Traffic Load Class |
| ToR | : | Terms of Reference |
| TRL | : | Transport Research Laboratory |
| TQMS | : | Total Quality Management System |
| | | |
| UK | : | United Kingdom |
| USA | : | United States of America |
| USCS | : | Unified Soil Classification System |
| USGS | : | United States Geological Survey |
| USD | : | United States Dollar |
| UTRCP | : | Ultra-thin Reinforced Concrete Pavement |
| | | |
| vpd | : | Vehicles per day |
| VEF | : | Vehicle Equivalence Factor |
| VOCs | : | Vehicle Operating Costs |
| | | |
| WBM | : | Water-Bound Macadam |

# GLOSSARY OF TECHNICAL TERMS

### Aggregate (for construction)

A broad category of coarse particulate material including sand, gravel, crushed stone, slag and recycled material that forms a component of composite materials such as concrete and asphalt.

### Asphalt

A mixture of inert mineral matter, such as aggregate, mineral filler (if required) and bituminous binder in predetermined proportions.

### Atterberg limits

Basic measures of the nature of fine-grained soils which identify the boundaries between the solid, semi-solid, plastic and liquid states.

### Base course

The upper layer of the road pavement, located between the sub-base and the surfacing.

### Binder, bituminous

Any *bitumen-based* material used in road construction to bind together or to seal aggregate or soil particles.

### Binder, modified

Bitumen based material modified by the addition of compounds to enhance performance. Examples of modifiers are polymers, such as PVC, and natural or synthetic rubbers.

### Bitumen

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

### Bitumen, cutback

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

### Bitumen, penetration grade

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

### Bitumen emulsion

An emulsion of bitumen and water with the addition of an emulsifier or emulsifying agent to ensure stability. 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 0.075 mm not exceeding 2%.
- Quick setting grade: An emulsion specially formulated for use with fine slurry seal type aggregates, where quick setting of the mixture is desired.
- Spray grade: An emulsion formulated for application by mechanical spray equipment in chip seal construction where no mixing with aggregate is required.
- Stable mix grade: An emulsion formulated for mixing with very fine aggregates, sand and crusher dust. Mainly used for slow-setting slurry seals and tack coats.

**Cape seal**

A single application of binder and stone followed by one or two applications of slurry.

**Cement (for construction)**

A dry powder which on the addition of water and other additives, hardens and sets independently to bind aggregates together to produce concrete.

**Chip seal, single**

An application of bituminous binder followed by a layer of stone or clean sand. The stone is sometimes covered with a fog spray.

**Chip seal, double**

An application of bituminous binder and stone followed by a second application of binder and stone or sand. A fog spray is sometimes applied on the second layer of aggregate.

**Collapsible soil**

Soil that undergoes a significant, sudden and irreversible decrease in volume upon wetting.

**Complementary Interventions**

Actions that are implemented through a roads project which are targeted toward the communities that lie within the influence corridor of the road and are intended to optimise the benefits brought by the road and to extend the positive, and mitigate the negative, impacts of the project.

**Concrete**

A construction material composed of cement (commonly Portland cement) as well as other cementitious materials such as fly ash and slag cement, aggregate (generally a coarse aggregate such as gravel or crushed stone plus a fine aggregate such as sand), water, and chemical admixtures.

**Concrete block paving**

A course of interlocking or rectangular concrete blocks placed on a suitable base course and bedded and jointed with sand.

**Crown height**

The vertical distance between invert of the side drain and the crown of the road.

**Crushed stone**

A type of construction aggregate, typically produced by mining a suitable rock deposit and breaking the removed rock down to the desired size using crushers.

**Design speed**

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

**Dispersive soil**

Soil in which the clay particles detach from each other and from the soil structure in the presence of water and go into suspension.

**Distributor**

A vehicle comprising an insulated tank with heating and circulating facilities and a spray bar capable of applying a thin, uniform and predetermined layer of binder.

**Expansive soil**

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

**Filler**

Mineral matter composed of particles smaller than 0.075 mm.

**Fog spray**

A light application of diluted bitumen emulsion to the final layer of stone of a reseal or chip seal or to an existing bituminous surfacing as a maintenance treatment.

**Gravel**

A naturally-occurring, weathered rock within a specific particle size range. In geology, gravel is any loose rock that is larger than 2 mm in its largest dimension and not more than 63 mm.

**Labour-based construction**

Substitution of equipment with well-managed labour as the principal means of production where technically and economically feasible to produce the standard of construction as demanded by the specification and allowed by the available funding.

**Low Volume Road**

Road carrying up to about 300 vehicles per day in the base year and less than about 1 million equivalent standard axles over their design life.

**Otta seal**

Sprayed bituminous surfacing using graded natural gravel rather than single-sized crushed rock

**Paved road**

A road that has a bituminous, concrete or alternative (e.g. segmental blocks) riding surface

**Prime coat**

A coat of bituminous binder applied to a non-bituminous granular pavement layer as a preliminary treatment before the application of a bituminous base or surfacing. While adhesion between this layer and the bituminous base or surfacing may be promoted, the primary function of the prime coat is to assist in sealing the surface voids and bind the aggregate near the surface of the layer.

**Ravelling**

The dislodging and loss of coarse aggregate from the road surface as a result of the action of traffic.

**Reseal**

A surface treatment applied to an existing bituminous surface.

**Rejuvenator**

A material (which may range from a soft bitumen to petroleum) which, when applied to reclaimed asphalt or to existing bituminous surfacing, has the ability to soften aged, hard, brittle binders.

**Seal**

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

**Selected layers**

Pavement layers of selected gravel materials used to bring the subgrade support up to the required structural standard for placing the sub-base or base course.

**Site Investigation**

Collection of essential information on the soil and rock characteristics, topography, land use, natural environment, and socio-political environment necessary for the location, design and construction of a road.

**Slurry (Slurry seal)**

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

**Slurry-bound Macadam**

A surfacing layer constructed where the voids in single-sized stone skeleton are filled using bituminous slurry.

**Standard**

A set of norms in the form of design guidelines or specifications.

**Subgrade**

The native material underneath a constructed road pavement.

**Sub-base**

The layer in the road pavement below the base course.

**Surface treatment**

A general term incorporating chip seals, micro surfacing, fog sprays or tack coats.

**Surfacing**

The layer with which traffic makes direct contact.

**Tack coat**

A coat of bituminous binder applied to a primed layer or to an existing bituminous surface as a preliminary treatment to promote adhesion between the existing surface and a subsequently applied bituminous layer.

**Ultra-thin Reinforced Concrete Pavement (UTRCP)**

A layer of concrete, 50 mm thick, continuously reinforced with welded wire mesh.

**Unpaved road**

Any road that is not surfaced with a “waterproof” surfacing, whether this be bituminous, concrete, interlocking blocks, cobbles or similar surfacings. Unpaved roads include earth and gravel roads.

**Wearing course**

The upper layer of a road pavement on which the traffic runs and is expected to wear under the action of traffic.

**Waterbound Macadam**

A pavement layer constructed where the voids in a large single-sized stone skeleton are filled with a fine sand.

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## TABLES IN APPENDICES

# 1. INTRODUCTION

## 1.1 Context and scope of the Manual

### 1.1.1 Purpose of the Manual

The Manual for Low Volume Roads (LVRs) promotes the rational, appropriate and affordable provision of LVRs in Ghana. In doing so it aims to make cost effective and sustainable use of local resources, reflecting local experience and advances in LVR technology gained in Ghana and elsewhere.

The Manual is fully adaptable for different clients and users. It is intended for use by roads practitioners responsible for the design and construction of low traffic earth, gravel or paved roads. It is appropriate for roads required to carry an average of up to about 300 vehicles per day in the base year, and less than about 1.0 million equivalent standard axles (Mesa) per traffic lane over their design life. The Manual complements the GHA design manuals for higher traffic roads and the Standard Specification for Road and Bridge Works (2007).

### 1.1.2 Contents of the Manual

The Manual is divided into four parts:

- Part A: Policy, Geometric Design and Road Safety
- Part B: Materials, Pavement Design and Construction
- Part C: Hydrology, Drainage Design and Roadside Stabilisation
- Part D: Surfacings for Low Volume Roads

Part B provides guidance on how to:

- carry out site investigations to collect data for pavement design;
- estimate the traffic loading on the road;
- characterize the subgrade and implement appropriate measures where the subgrade is weak or unstable;
- design the pavement layers and select appropriate pavement materials;
- carry out life cycle costing to compare alternative designs;
- construct the road including the management of borrow pits; and
- implement quality assurance systems and specific quality control measures on site.

### 1.1.3 Application of appropriate standards

In keeping with the Government of Ghana policies, the application of appropriate design standards for LVRs aims to optimise construction and maintenance costs and meet the requirement to:

- improve the economic and social well-being of rural communities and their access to social and other services;
- lower road user costs and promote socio-economic development, poverty reduction, trade growth and wealth creation in rural areas;
- facilitate rural accessibility in a manner that is available and relevant to the needs of disadvantaged and different ethnic groups in society; while
- protecting and managing non-renewable natural resources and reducing import dependency.

### 1.1.4 Clients for Low Volume Roads

The Client for the LVR works could be GHA, DFR, a local authority, a Non-Governmental Organisation (NGO), a community-based organisation, or a private company. Road works require a design, whether they are to be undertaken by a contractor, through an in-house capability or through a community contract. This design should meet national standards set for a particular type of road. The degree of sophistication of the design generally increases as the standard of the road increases. However, this does not mean that earth or gravel roads are any easier to design than a low volume paved road. The opposite may well be the case.

## 1.2 Project implementation

### 1.2.1 The Procurement process and project cycle

Unless carried out in-house, the provision and maintenance of LVRs entails various instances of procurement, the process of creating and fulfilling contracts. Such contracts relate to the provision of the different services and goods needed to plan, appraise, design, supervise and execute the works while ensuring that related environmental and other safeguards are adhered to.

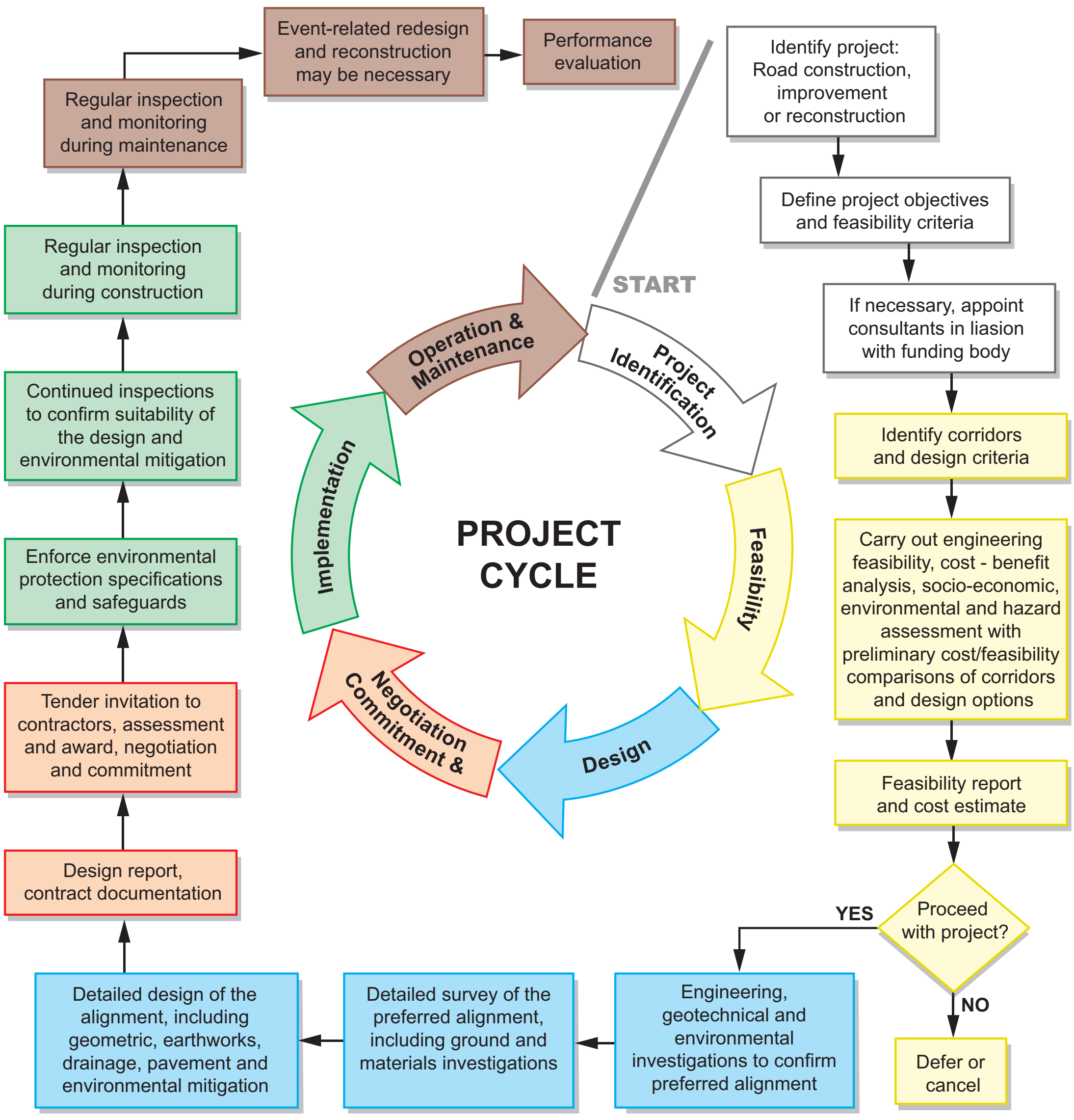



**Figure B.1.1 Step-by-step breakdown of stages of the Project Cycle**

In the case of a specific project, these functions are broken down into step-by-step procedures that contribute to the broader process of ensuring that established good practice is followed at all stages of the Project Cycle. Figure B.1.1 illustrates how each of these procedures, to which reference is made in this Manual, relate to each other and to that Project Cycle.

Though not explicitly shown, there is implied associated lesson-learning at every stage, as Monitoring and Evaluation (M&E) functions identify scope for improvement.

There are four stages of project preparation leading up to and including final engineering design. These are:

- Identification and general planning;
- Pre-feasibility study;
- Feasibility study or preliminary engineering design; and
- Final engineering design and tender documentation.

The description of these stages that follows focuses on activities normally carried out for new road construction. However, the same principles apply to the upgrading and improvement of existing roads, where the assessment of environmental, engineering and cost factors equally apply and where stakeholder consultation should form an integral component of decision-making and design.

### 1.2.2 Identification and general planning

This is the stage at which the need for the project is identified and projects that do not meet selection criteria are rejected. For LVRs this will usually be done at a relatively local level and will be the output of a planning process. Only a desk study supplemented by reconnaissance survey are carried out. Consultation with relevant stakeholders forms an important element of this work.

### 1.2.3 Pre-feasibility study

At this stage a broad economic and engineering assessment is made, taking into account the main engineering considerations and any other issues affecting the project (for example, environmental and cultural issues). As part of the pre-feasibility study it is important to identify and investigate the major technical, environmental, economic and social constraints through preliminary surveys in order to obtain a broad appreciation of the viability of competing alignment or upgrading design options that might apply. For LVRs, one of the most important aspects of the pre-feasibility study is communication with the people who will be affected by the new or improved road. Their views are vital for the completion of a successful project and interacting with them is essential from the outset.

Field surveys provide the opportunity for checking the actual conditions on the ground and for noting any omissions and errors in the desk study. During these surveys, it is necessary to record details of topography that might constrain alignment options and geometric design, soil and rock conditions, slope stability conditions (in hilly and mountainous terrain), slope and stream channel erosion, potential soil engineering problems, availability of construction materials, water crossings and potential drainage problems.

For the lower classes of predominantly earth or gravel roads, the information from the pre-feasibility study may be the only available data to assist in the design of the road upgrade due to financial constraints. Accordingly, it is important to bear this in mind when deciding upon the strategy of the survey.

For route comparison and decision-making over project implementation, cost estimates should where possible be made to within 30% of the final out-turn cost.

### 1.2.4 Feasibility study or preliminary engineering design

At this stage, sufficient data are required to allow the final choice of alignment and the structural design of the road to be undertaken, and any requirements for environmental and geotechnical mitigation to be defined. Topographical, engineering and environmental surveys undertaken as part of the feasibility study should be of sufficient detail to develop a preliminary engineering design and cost estimate accurate to approximately 15% of the final out-turn cost. Construction costs for similar roads in the region can be used as a means of comparison. In critical areas it may be necessary to carry out ground investigations, for example in order to provide a preliminary assessment of foundation conditions for bridge abutments or to investigate problematic soils in order to confirm construction feasibility and allow sufficient provision in the cost estimate.

In the case of upgrades and improvement projects to existing roads an important element of the fieldwork at this stage will be an assessment of the road's past performance so that necessary design changes and mitigation measures can be built-in to the new design. Some of the key issues to look out for are summarized below.

**Visual assessment of road condition**

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

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

It is important to distinguish between those defects caused by inadequate structural capacity of the existing pavement, if any, and those caused by poor drainage, particularly in the shoulders or outer wheel path. The former will probably require the structural capacity of the pavement to be increased, for example by importing one or more new pavement layers. The latter defects could be rectified by improving the drainage without importing new layers. A spot improvement approach where isolated problem areas are rectified individually rather than taking them as representative of the section as a whole is often adopted based on the severity and extent of the problem areas. The Dynamic Cone Penetrometer (DCP) is an effective tool for the assessment of the existing pavement.

**Drainage and erosion**

It is important to identify any defects and inadequacies in the drainage system. The replacement of major drainage structure, such as bridges and large culverts, can be an expensive exercise and existing infrastructure should be used as much as possible. However, design and construction modification will be required wherever drainage structures are failing to perform as intended. This might include under-sized culverts, poorly-performing side drains and inadequate scour protection below outlets and turnouts.

**Geometric design and road safety**

Geometric characteristics of the road, in terms of its horizontal and vertical alignment, will normally be retained for the upgraded road with minor improvements. Nonetheless, any locations of traffic hazard or obvious geometric shortcoming, such as sharp bends combined with poor sight distance, should be noted for improvement, including appropriate measures for producing a safer road environment.

### 1.2.5 The design process

The road Design Engineer is normally supported by a team of individuals, with varying areas of specialist expertise, who are collectively capable of dealing with all aspects of the road design. The job of the design team is to provide a robust technical design (geometric, drainage and pavement), and to reflect this design in the Specifications, Drawings and the Bills of Quantities. The design team should include environmentalists and social development specialists, or other professionals who possess the necessary knowledge, skills and experience in these fields.

The general approach to the design is guided by the Client and builds on information and data collected during the project pre-feasibility and feasibility stages. The Client normally has a budget provided for the works. The general location of the route is known, as well as the preferred approach to the works, for example whether that should be labour- intensive or equipment-based. The Client may have requirements on apportioning works and contract size, technical issues, social, environmental requirements and time constraints. The job of the road Design Engineer is to develop the project within and around these constraints, whilst at the same time alerting the Client to issues and problems that may require an adjustment of expectations.

The approach to the design of LVRs follows the general principles of any good road design practice. There are, however, subtle but important differences from the traditional approach. This Manual provides the Design Engineer with the requisite tools to achieve an optimised design based on the financial, technical and other constraints that define the project.

Optimising a road design requires a multi-dimensional understanding of all of the project elements. In this respect, the design elements become context-specific. The design team may need to work outside their normal areas of expertise and to understand the implications of their recommendations or decisions on all other elements of the design.

The successful design of LVRs relies on:

- a Client who is open and responsive to innovation;
- a full understanding by the Design Engineer of the road environment;
- an ability to work within the demands of the road environment and to turn these to a design advantage;
- innovative and flexible thinking through the application of appropriate engineering solutions rather than simply following traditional thinking related to road design;
- recognition and management of risk; and
- guaranteed routine and periodic maintenance after construction.

The Design Engineer is required to provide a road that meets the level of service expected by road users and local communities. Design engineers are traditionally risk-averse and build in factors of safety that cater for their perceptions of risk. Such an approach does not necessarily encourage innovation, uses scarce or inappropriate resources, and may result in unduly high financial and economic costs for the Client and the country. There is often a temptation to upgrade roads to a level of service not justified by economics or by road user requirements. This can unnecessarily absorb limited available resources and thereby prevent extension of access to other areas. The Design Engineer has an overriding professional obligation to act in the interests of the public good.

The level of attention and engineering judgement required for optimal provision of LVRs is no less than that required for the provision of other roads. In many cases the required level of judgement is higher. The Design Engineer needs to draw on all of their engineering skills, judgement and experience to develop appropriate designs without incurring unacceptable levels of risk.

## 1.3 Road network classification

The total current road network in Ghana comprises approximately 71,000 km of road of which LVRs make up approximately 42,000 km.

Ghana's classified roads are assigned to one of three main classes: Trunk roads, Urban roads and Feeder roads. These are further subdivided or categorised as follows:

- **Trunk roads** are the main veins and arterials of the national road network. These are the responsibility of the Ghana Highway Authority (GHA).

  They are subdivided into the following categories:
  - **National roads**: Roads linking Ghana to centres of international importance and to international boundaries
  - **Regional roads**: Roads connecting centres of national importance such as regional capitals and principal towns
  - **Inter-Regional roads**: Roads connecting centres of regional importance
- **Urban roads (Metropolitan / Municipal roads):** Roads connecting local centres of importance to each other or connecting important centres to higher class roads. These are the responsibility of the Department of Urban Roads (DUR).

  Urban roads are further sub categorised into Arterials, Collectors and Local or residential streets.

**Feeder roads**: Roads connecting minor centres such as towns, villages, rural settlements and markets to other parts of the network. These are the responsibility of the Department of Feeder Roads (DFR).

Feeder roads generally carry relatively low volumes of traffic and make up a significant proportion of the road network in Ghana. They are further divided into three sub-categories:

- **Inter-District Feeder roads**: Connect two or more districts
- **Connector Feeder roads**: Connect to the road network at both ends, providing access to rural communities along the road itself
- **Access / Spur Feeder roads**: Connect to the road network at one end and terminate at a village or rural settlement at the other. These are generally short roads of limited economic importance.

LVRs are divided into four categories, or types, namely Type 1 to Type 4. The hierarchy of the various types of LVRs is based on traffic volume. Informal tracks are considered to be LVRs, but they are generally existing and don't conform to any basic engineering standards. This Manual should be consulted when there is a need to upgrade a track to provide more reliable access by incorporating basic engineering standards.

Table B.1.1 shows the classification of roads in Ghana based on geometric standards, appropriate Level of Service (LOS), and Average Annual Daily Traffic (AADT) volumes they generally carry. LVRs in Ghana are generally of Type 4 standard or below and meeting C or D level of service criteria.

For LVRs, the LOS is not normally linked to congestion because traffic volumes are low. The LOS for a higher volume roads is primarily linked to the average speed of travel that the road users expect to achieve, while for very low volume roads the primary focus is on achieving basic access. The desirable LOS for roads is related to the road classification and is defined as follows:

**Level of Service A**: This is the highest level. Traffic is free flowing, with the volumes and types of traffic readily accommodated. Safety is a high priority. Design speed is very important and takes precedence over topographic constraints.

**Level of Service B**: Traffic may not flow smoothly in all situations. Safety is a high priority, but in some instances safety risks may need to be mitigated through the enforcement of safety controls such as local speed limits and associated signage. Design speed is important, but topography may dictate some design changes and controls.

**Level of Service C**: The efficiency of traffic movement and flow is not a limiting factor. Traffic will be accommodated, but some design controls may need to be applied, such as for speed, sight distance, access control and road carriageway configuration. Safety provisions are adapted to lower and variable speed scenarios. The topography will dictate alignment and the design speed.

**Level of Service D**: This level is geared to provision of basic access rather than efficiency. Design standards for water crossings may allow temporary service interruption and some entire roads may even be closed at times (such as during and immediately following heavy rain) to protect these assets. Other design standards for geometrics, surfacing and safety will reflect lower speed environments and basic access requirements.

**Table B.1.1 Road classes in Ghana**

| Road Functional Classification | | | | | | Geometric Standard | Desirable Level of Service | AADT |
|---|---|---|---|---|---|---|---|---|
| | | | | NATIONAL | HIGH VOLUME | Refer to GHA | A | > 10,000 |
| | | | REGIONAL | NATIONAL | HIGH VOLUME | Refer to GHA | A | 3,000 - 10,000 |
| | | | REGIONAL | NATIONAL | HIGH VOLUME | Refer to GHA | B | 1,000 - 3,000 |
| | | INTER-REGIONAL | REGIONAL | NATIONAL | HIGH VOLUME | Refer to GHA | B | 300 - 1,000 |
| | METROPOLITAN MUNICIPAL | INTER-REGIONAL | REGIONAL | NATIONAL | LOW VOLUME | Type 4 | C | 150 – 300 |
| FEEDER | METROPOLITAN MUNICIPAL | INTER-REGIONAL | | | LOW VOLUME | Type 3 | C | 75 –150 |
| FEEDER | METROPOLITAN MUNICIPAL | | | | LOW VOLUME | Type 2 | C | ≤ 75 |
| FEEDER | | | | | LOW VOLUME | Type 1/Track | D | ≤ 15 |

## 1.4 Definition of a Low Volume Road

For the purposes of this Manual a road is classified as “low volume” if:

- the average daily traffic in the base year is less than 300 motorised vehicles with four or more wheels; and
- the cumulative number of equivalent standard axles is less than 1.0 million per traffic lane over the design life.

The most important aspect of such roads is that the performance of the road is more dependent on environmental influences than it is on traffic, as indicated in Figure B.1.2. This has important implications for many aspects of their design.

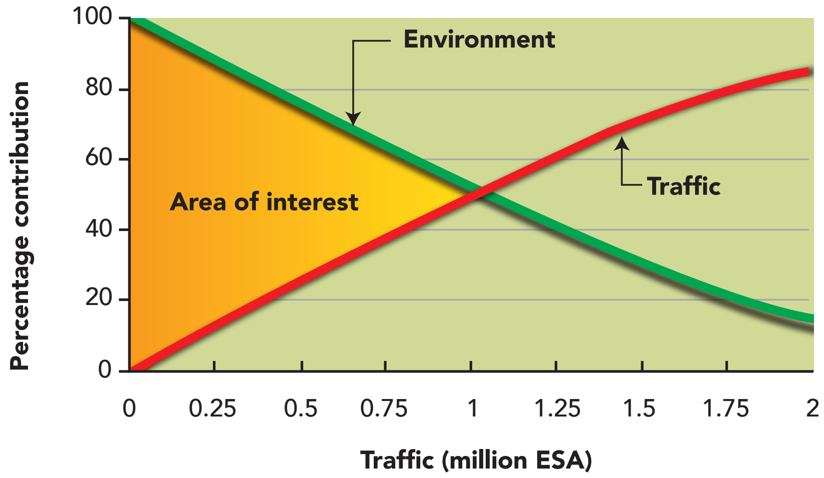



**Figure B.1.2 The relative impact of environment and traffic on road performance**

## 1.5 Road environment

### 1.5.1 Road environment impact factors

Ghana's LVRs are typically unpaved with a gravel or earth wearing surface. Deterioration of such unpaved roads is governed by the type of material used on the surface, the strength of the underlying soil, the type and action of traffic and most importantly, the influence of the environment within which the road is located. Typical road environment impact factors are presented in Figure B.1.3.

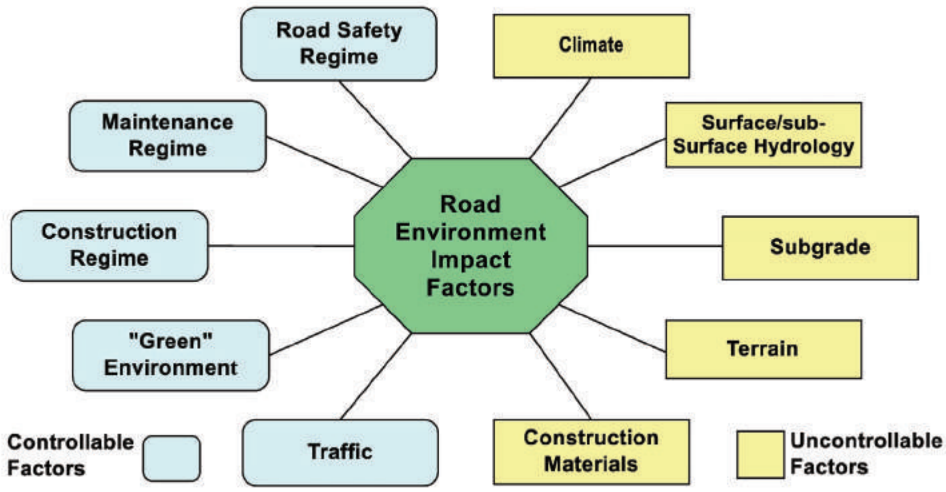


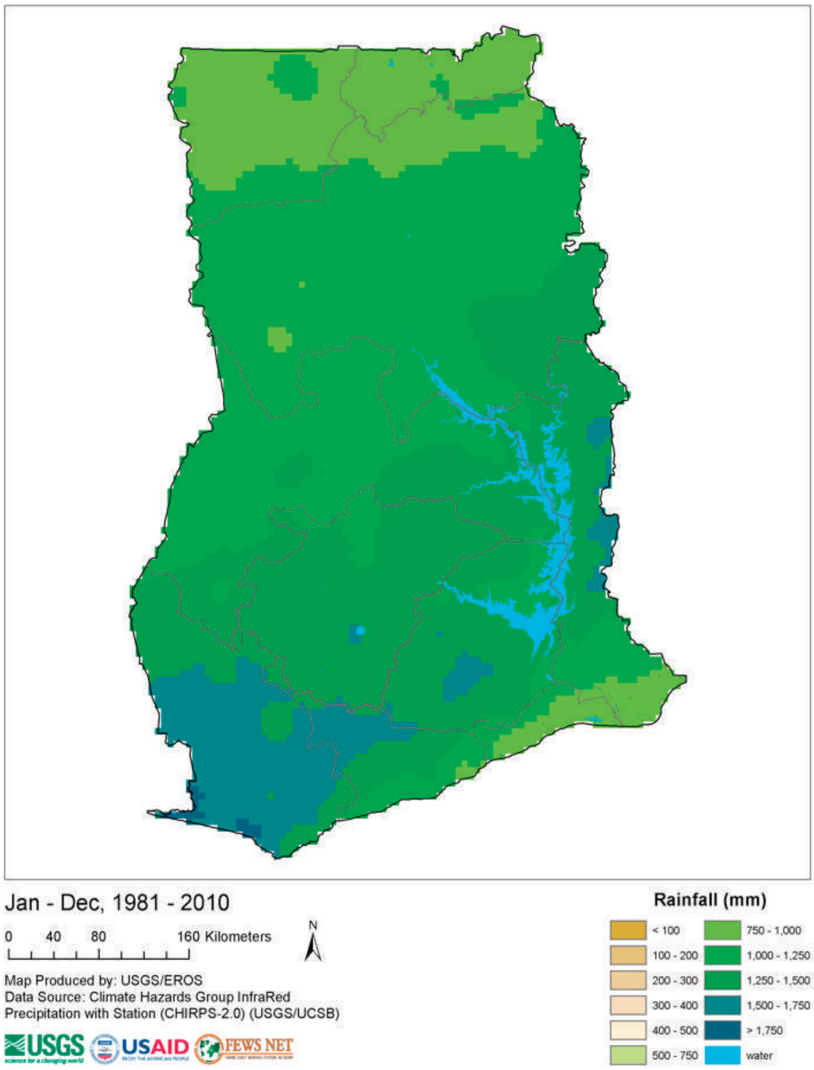

**Figure B.1.3 Road environment impact factors**

The "road environment" covers both the natural and the human environment and includes the interaction between the different environmental factors and the road structure. Some of these factors are uncontrollable. These include those attributable to the natural environment, the interacting influence of weather (wind, and rainfall), local hydrology and drainage, and the general terrain and gradient. The design approach needs to recognise such individual and collective influences by providing options that minimise any negative effects. Other factors that are largely controllable and can more readily be taken into consideration through the design approach include the construction and maintenance regime, safety and environmental policies, and the extent and type of traffic.

### 1.5.2 Topography

Ghana exhibits a generally flat topography, with hilly and mountainous areas. It should be noted that the topographical properties of each individual project road need to be determined using a combination of available maps and site reconnaissance. The topography will influence the design parameters.

### 1.5.3 Rainfall




*SOURCE: United States Geological Survey (USGS)*

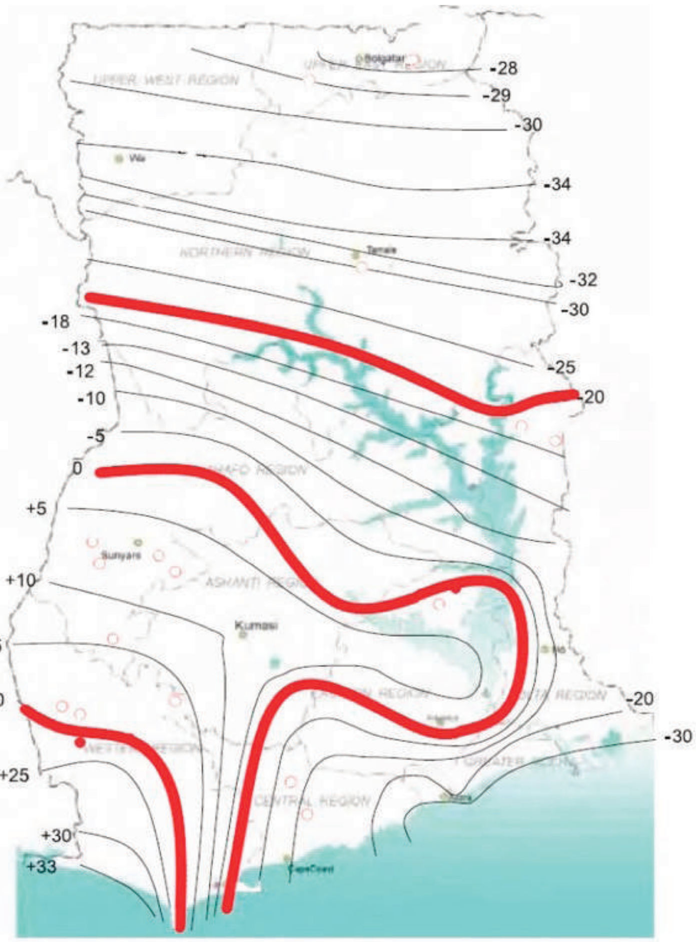
**Figure B.1.4 Mean Annual Precipitation (MAP) of Ghana**

Figure B.1.4 provides a map of the average annual rainfall in Ghana. It can be observed that Ghana is drier in the northern regions and becomes progressively wetter in the south and east, except for the region along the coastline which is relatively dry. The monthly rainfall patterns will influence the pavement materials and design as well as the method of construction.

### 1.5.4 Macro-climatic regions

The meteorological environment is divided into macro-climatic regions with different moisture and temperature conditions.

Figure B.1.5 shows the macro-climatic regions for moisture conditions based on Thornthwaite's Moisture Index. Thornthwaite is a function of evapo-transpiration, i.e., dependent on vegetation and rainfall.




*SOURCE: ILO, 2006*

**Figure B.1.5 Thornthwaite's Moisture Index for Ghana**

Table B.1.1 provides an interpretation of the Index. Ghana can be divided into four major climatic zones:

- Semi-arid;
- Dry sub-humid;
- Moist sub-humid; and
- Humid

Of the above, the semi-arid zone is regarded a dry region, the dry and moist sub-humid zones as a moderate region and the humid zone as a wet region.

**Table B.1.2 Interpretation of Thornthwaite's Moisture Index**

| Index Range | Climatic Region | |
|---|---|---|
| | Description | Design Climate |
| < -40 | Arid | Dry |
| -40 to -20 | Semi-arid | |
| -20 to 0 | Dry sub-humid | Moderate |
| 0 to 20 | Moist sub-humid | |
| > 20 | Humid | Wet |

## 1.6 Principles of LVR pavement design

### 1.6.1 Approach to LVR pavement design

The general approach to the pavement design of LVRs differs in a number of respects from that for High Volume Roads (HVRs). For example, conventional pavement designs are typically directed at relatively high levels of service and require multiple layers of selected materials. in the case of LVRs, significant reductions in the cost of the pavement can be achieved by reducing the number of pavement layers and/or layer thicknesses, by using local materials more extensively as well as at lower cost, and through the adoption of more appropriate surfacing options and construction techniques.

In addition, research has indicated that the road deterioration mechanisms of LVRs are significantly different to those of HVRs. One of the implications of this is that, as illustrated in Figure B.1.2, appropriate pavement design options need to be fully responsive to a range of factors that may collectively be referred to as the road environment.

The adoption of appropriate designs for LVRs does not necessarily mean an increased risk of failure. Rather, it requires a greater degree of pavement engineering knowledge, experience and judgement and the careful application of fundamental principles of pavement and material behaviour derived from local or regional research.

The challenge of good pavement design for LVRs is to provide a pavement that is appropriate to the road environment in which it operates, and which fulfils its function at minimum life cycle cost at an optimal level of service. However, positive action in the form of timely and appropriate maintenance will always be necessary in order to ensure that the assumptions of the design phase hold true over the design life.

### 1.6.2 Pavement structure and function

Road pavements have three primary components: the wearing surface; the pavement structure; and the subgrade, with each serving a specific function. For paved roads the function of the surfacing is to keep the pavement dry and waterproof. The function of the pavement structure for all road types is to support the wheel load on the surface and to transfer and spread that load to the natural underlying subgrade without exceeding either the strength of the subgrade or the internal strength of the pavement itself. This implies that the pavement materials themselves should not deteriorate to such an extent as to affect the riding quality and functionality of the pavement. These goals must be achieved throughout the specified design period.

The function of the surfacing differs slightly for gravel and for earth roads, where the wearing surface is often permeable and wears away under the action of traffic and rainfall. As the stresses on the subgrade must still be limited safe levels, the gravel needs to be replaced regularly. Considerably more maintenance is thus necessary if gravel and earth roads are to perform satisfactorily.

Figure B.1.6 shows a wheel load, W, being transmitted to the pavement surface through the tyre at an approximately uniform vertical pressure, $P_0$. The pavement then spreads the wheel load to the subgrade so that the maximum pressure on the subgrade is only $P_1$. By proper selection of pavement materials and with adequate pavement thickness, $P_1$ will be small enough to be easily supported by the subgrade.

Because of the different functions of the surfacing and pavement structure, these basic components of a road are often independent of one another and a large number of combinations are possible. However, in terms of the design of the overall road, some surfacings (e.g. bituminous surface treatment such as surface dressing) do not contribute to the overall structural strength of the road, while others (e.g. penetration macadam) do. In the case of earth or gravel roads, the natural soil or gravel is the main structural component. Table B.1.3 provides an illustration of the relative reduction, from $P_0$ to $P_1$, in pressure for a typical pavement structure.

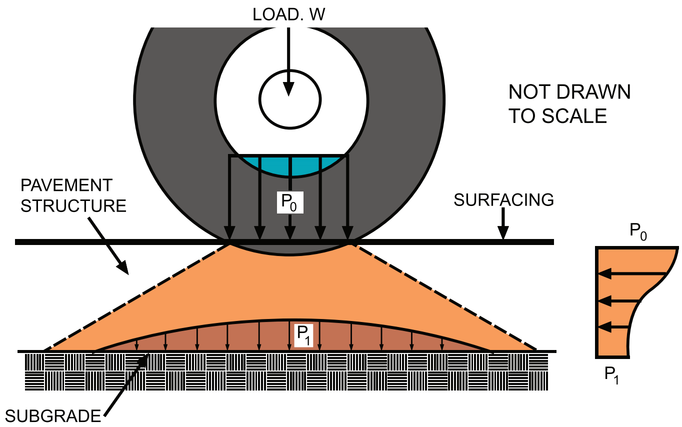



**Figure B.1.6 Wheel load transfer through pavement structure**

**Table B.1.3 Comparison of applied pressure $P_0$ to subgrade pressure $P_1$**

| Vehicle type | Axle load (kN) | Tyres per axle | Subgrade cover (mm) | $P_0$ (kPa) | $P_1$ (kPa) |
|---|---|---|---|---|---|
| Car | 7 | 2 | 450 | 200 | 4 |
| Truck | 80 | 4 | 450 | 750 | 31 |

### 1.6.3 Pavement and surfacing options

There is a wide range of pavement and surfacing options, both bituminous and non-bituminous, that can be used in various combinations and are well suited for incorporation in LVR pavements. These options allow maximum use to be made of locally available materials and least use to be made of more expensive pavement materials, especially where they have to be processed or hauled long distances.

Many of the pavement and surfacing options also allow the use of a high level of local labour, both skilled and unskilled, and a low requirement for imported equipment. This provides the flexibility to use small and medium scale enterprises (SMEs) with the accompanying benefits of higher local employment, skills development, and multiplier effects on the local economy.

### 1.6.4 Reliability and terminal condition

There are many reasons why it is impossible to predict exactly how a road will deteriorate and at what rate. Roads built to apparently identical specifications and quality show a wide range of performances as illustrated in the definitive AASHO Road Test carried out in the USA in 1960. In this test, individual roads of similar design and quality reached their terminal level of deterioration after anything between 30% and 200% of the average life for those sections. Thus, a section of road could last for only one third of the average life or could last twice as long. This variability is a consequence of the practicalities of road building in which relatively unprocessed and inherently variable materials are used and is an entirely natural effect. Roads constructed with less associated quality management will exhibit even greater variability in performance.

In any pavement design strategy, it is therefore necessary to specify the level of reliability required or, in other words, the safety factor to be applied to the design. For roads carrying high levels of traffic, the safety factor is normally set high because the poor performance of such roads has a large effect on travel times, road roughness, vehicle operating costs and thereby on the economic efficiency of the country. Such roads are therefore designed so that there is only a very small probability of not performing to the desired standard for the desired length of time. This is defined as a high level of reliability, typically 95% or 98%. This is equivalent to setting the planned terminal level of serviceability to a high value (i.e. the amount of deterioration that is considered acceptable before the road needs improving is relatively small).

On the other hand, roads carrying low levels of traffic can be allowed to deteriorate a little more before they are deemed to need repair. They are therefore designed with a lower safety factor. The level of reliability is set lower, typically 90% to 70% for the lowest traffic levels. This is equivalent to designing them with a lower level of planned terminal serviceability as shown in Figure B.1.7, in which TLC refers to the Traffic Load Class as defined in Section B.2.3.

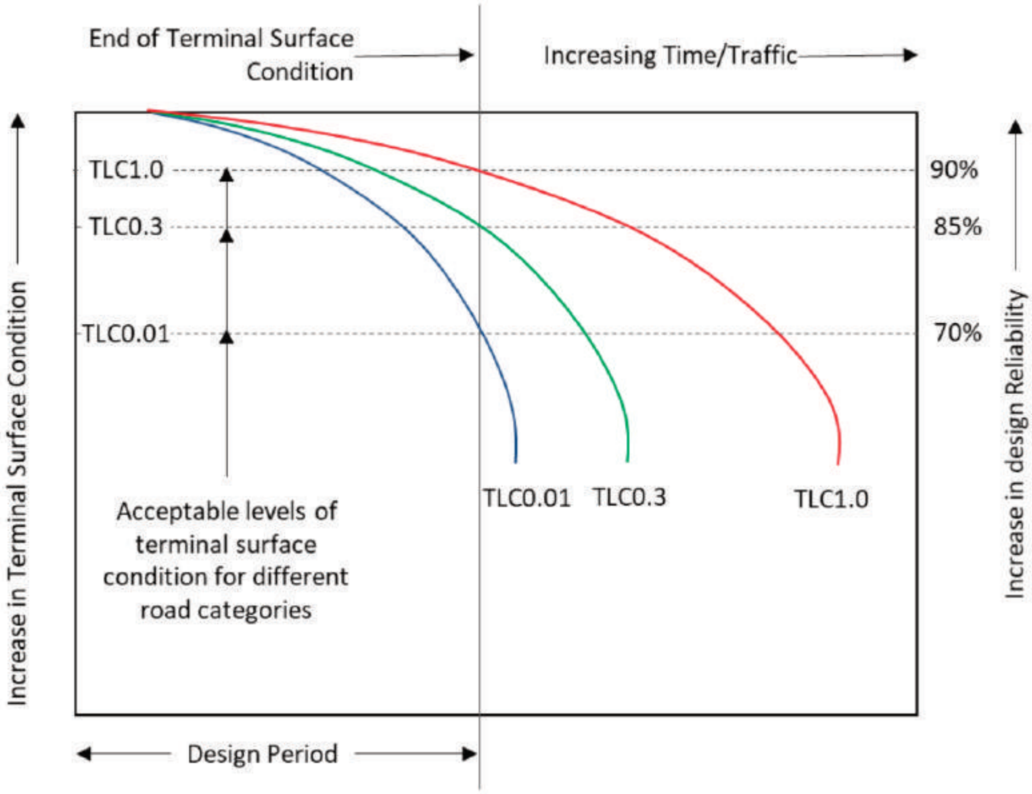



**Figure B.1.7 Design reliability in relation to road category and terminal surface condition**

The level of reliability is a statistical measure of deterioration representing the probable behaviour of the worst performing examples of road. The majority of roads will perform better and will show a level of serviceability higher than the designed value at the end of their design period.

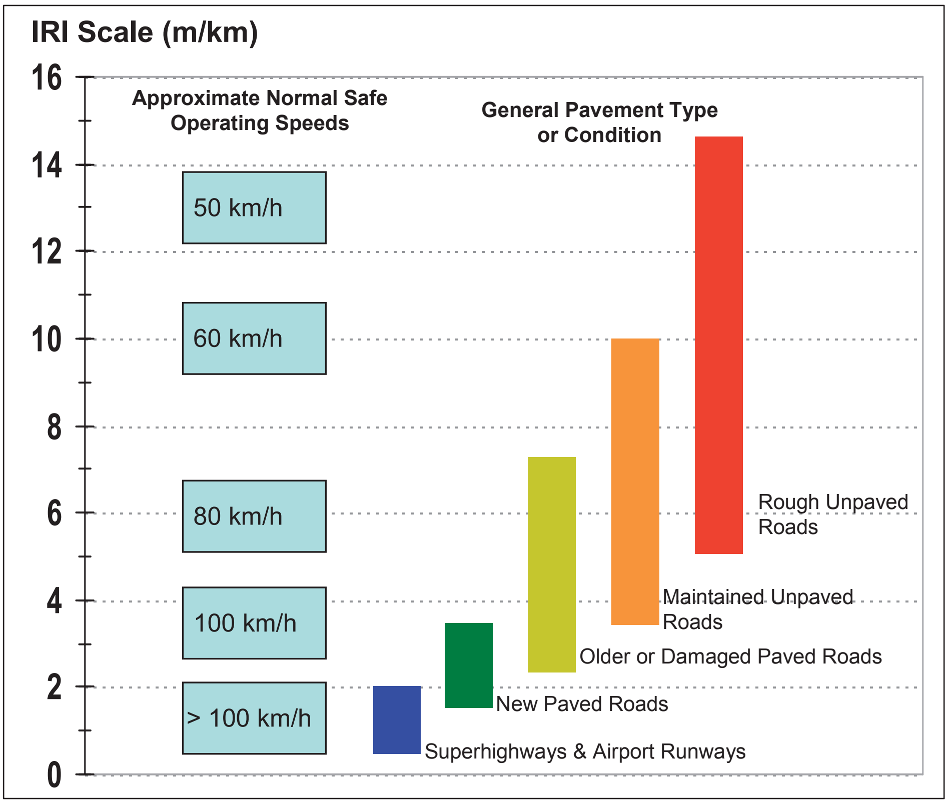
Although a pavement may have reached its terminal surface condition at the end of the design period, it will still be able to carry traffic. However, if, at this time, rehabilitation is not carried out, the level of service provided will decrease quite quickly and will soon became unacceptable.

### 1.6.5 Design period and Levels of Service

The road pavement is designed to carry the expected traffic loading over the selected design life of the road. The design period is therefore defined as the time span in years before the road pavement reaches a terminal condition of serviceability after which major rehabilitation or reconstruction would be required. It is conventional practice to specify this terminal condition in terms of an International Roughness Index (IRI) value. The IRI provides an indication of the riding quality of the road.

Figure B.1.8 provides a graphical interpretation of various IRI values expected for different classes of road. If, for example, the minimum level of servcie required for an unpaved road includes an average speed of travel of 60 km/h over (say) 80% of the length of the road, the terminal condition would be reached when the IRI exceeds about 10 m/km over 80% of the length of the road. The design of the pavement thickness and the selection of pavement materials must ensure that the design life is achieved. The design must consider the maintenance regime that is likely to be applied to the road.

Other criteria that are used to determine the terminal condition of a road pavement are rutting and pavement deflection. These criteria are normally applied only to paved roads.




*SOURCE: ASTM E1926-08 (Standard Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements)*

**Figure B.1.8 The IRI interpretation scale**

### 1.6.6 Upgrading strategy

The decision as to when a road should be upgraded to a higher, more expensive structural standard is often not a simple choice between a paved and an unpaved road. Over a period of time, a road will undergo a number of improvements and will eventually consist of a mixture of structural designs. Figure B.1.9 illustrates the various types of pavement structure that are likely to be found on such a road. When the traffic level is high enough a decision may be made to upgrade the entire road, or at least substantial sections of it, to a specific structural standard. Such a decision depends on a consideration of whole life (or life-cycle) costs.

As traffic levels increase it becomes more economically viable to seal gravel roads. In some countries the practice is to construct a flexible pavement to sub-base level and use the sub-base material as a gravel wearing course for a period. This continues until traffic levels justify upgrading the road to paved standard, or when funding becomes available. A new base material and surfacing are then constructed on top of the sub-base. This practice is not recommended, as the material design considerations for a sub-base and wearing course are vastly different. There can be a gap of many years between the construction of the sub-base and upgrade to surfaced standard. Over these years, the sub-base material will experience gravel loss and decomposition or weathering and will have to be maintained accordingly. Regravelling of the sub-base may also lead to blending of material over time. When a decision is reached to upgrade a gravel road to surfaced standard, the in situ material, especially the future sub-base, requires careful testing. The existing materials must meet the requirements of the flexible pavement structure. The DCP-DN design method described in Section 6.3 is particularly suited to the upgrading of gravel roads to paved standard.

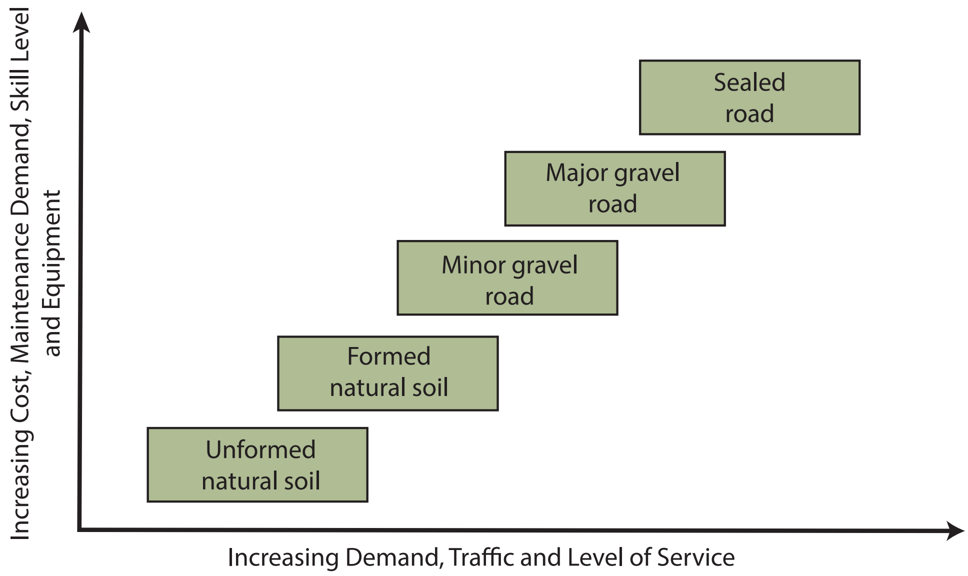



**Figure B.1.9 Stages of upgrading in relation to demand on resources**

## 1.7 Environmentally Optimised Design

Environmentally Optimised Design (EOD) is an approach to road design that considers the variation of different road environments along the length of the road. Such features may include steep gradients, wet and marshy areas as well as passage over easy terrain. The EOD approach then requires consideration of a range of options for improving or creating LVR access – from dealing with individual critical areas on a road link from Spot Improvement Design (SID), to providing a total whole link design, which, in the latter case, could comprise different design options along its length. EOD is addressed in various form in this Manual, with specific focus on the appropriate selection of pavement materials and thickness for each uniform section along the road.

The SID principle can be applied within the context of an EOD strategy with the overall aim of ensuring that each section of a road is provided with the most suitable pavement type for the specific circumstances to provide sustainable access along the road. This requires analysis of a broad spectrum of solutions to improve different road sections, depending on their individual requirements, ranging from engineered natural surfaces to bituminous pavements. The chosen solution must be achievable with the materials, plant and contractors available locally.

The EOD/SID approach ensures that specifications and designs support the functions of different road sections. In so doing, it assesses whether the standard design is sufficient for problematic areas and whether it is necessary for the good areas. An under-design of poor sections can lead to premature failure and an over-design will often be a waste of resources which would be better applied on the problematic sections. The EOD/SID principle is illustrated in Figure B.1.10.

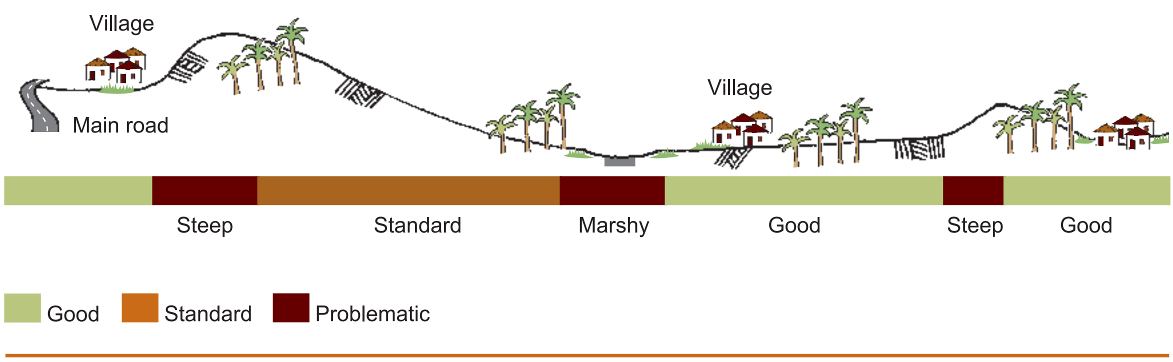



**Figure B.1.10 Environmentally optimised and spot improvement design**

The recommendations for pavement design included in this Manual support the EOD approach. This includes:

- Accurate assessment of the subgrade or the existing road pavement through a DCP survey and materials investigation, which allows an optimal pavement design to be developed for each uniform section of the road (see Chapters B.6 and B.7); and
- Paving of roads on steep slopes to control erosion of the road surface, even if traffic levels are low.

Part C of the Manual provides guidance on drainage design, including the identification of appropriate drainage structures depending on the class of road and required level of service.

# 2. TRAFFIC LOADING

## 2.1 Introduction

Reliable data on traffic volumes and characteristics is an essential requirement for both geometric and pavement structural design. It also assists in the planning of road safety measures as summarized below:

- **Geometric design**: The volume and composition of traffic, both motorized and non-motorized, influence the cross-section design (carriageway and shoulders).
- **Pavement design**: The deterioration of the pavement is influenced by both the magnitude and the frequency of individual axle loads.
- **Road safety**: The volume, type and characteristics of the traffic using the road will each influence the type of road safety measures required to ensure a safe road environment.

In view of the above, a reliable estimate of existing (baseline) and future traffic loading is required so as to prepare an appropriate and cost-effective design of the road.

The purpose of this Chapter is to outline the procedures to be followed in determining the traffic loading over the design life of the road as a basis for designing the road pavement. The Chapter considers types of surveys that provide the inputs for determining the design traffic loading. Such data must be sufficiently accurate to select the correct Traffic Load Class for structural pavement design from the three classes appropriate to LVRs.

## 2.2 Traffic surveys

### 2.2.1 General

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

- Classified traffic counts;
- Origin-Destination (OD) surveys
- Axle load surveys

### 2.2.2 Traffic counts

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

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

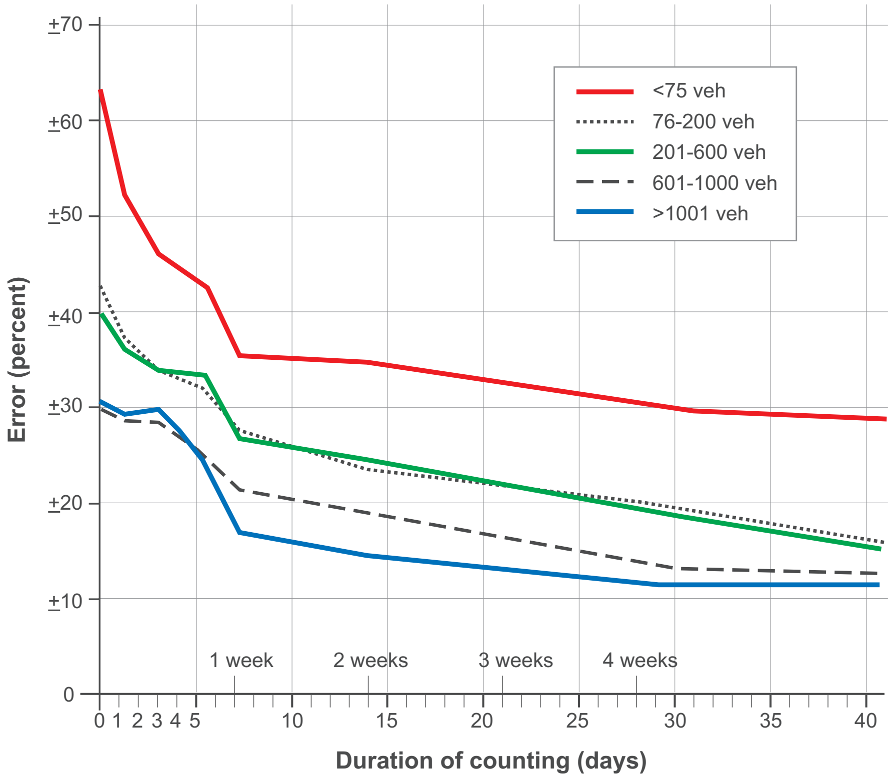
- Manual traffic count;
- Automatic traffic count; and
- Moving observer methods.

Axle load surveys are also required to determine vehicle loading.  OD surveys are sometimes carried out for planning purposes.

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

Motorised traffic volumes tend to fall in the wet season to, typically, 80% of their dry season level. On poor quality roads this difference can be even more marked and wet season traffic can fall to 35% of dry season levels.

The timing, frequency and duration of traffic surveys should therefore be given careful consideration in terms of striking a balance between cost and accuracy. As indicated in Figure B.2.1, short duration traffic counts in low traffic situations can lead to large errors in traffic estimation.




*SOURCE: Howe (1972)*

**Figure B.2.1 Possible errors in ADT estimates**

**Reducing errors in estimating traffic for LVRs**

Errors in estimating traffic can be reduced by:

- Counting for seven consecutive days.
- On some days counting for a full 24 hours, preferably with one 24-hour count on a weekday and one during a weekend. On other days, 16-hour counts (typically 06.00 – 22.00 hours) should be made and expanded to 24-hour counts using a previously established 16:24 hour expansion ratio.
- For single day traffic counts, repeating the count to capture data both on a market and a non-market day. Especially in rural areas, this may demonstrate a higher demand on specific days of the week.
- Avoiding counting at times when road travel activity increases abnormally. For example, just after the payment of wages and salaries, or at harvest time, public holidays or any other occasion when traffic is abnormally high or low. However, if there is a harvest during the wet season, it is important to obtain an estimate of the additional traffic typically carried by the road on account of this during these periods.
- Repeating the seven-day counts several times throughout the year.

Care should be exercised in selecting appropriate locations for conducting the traffic counts to ensure a true reflection of the traffic using the road and to avoid under- or over-counting. Local knowledge should be used to help with this.

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

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

**Adjustments for season**

An appropriate, weighted average adjustment should be made according to the season in which the traffic count was undertaken and the length of the wet and dry season. Figure B.2.2 shows a typical average monthly distribution of rainfall for a location (Yendi) in Ghana, though actual rainfall patterns and quantities can vary considerably depending on location.

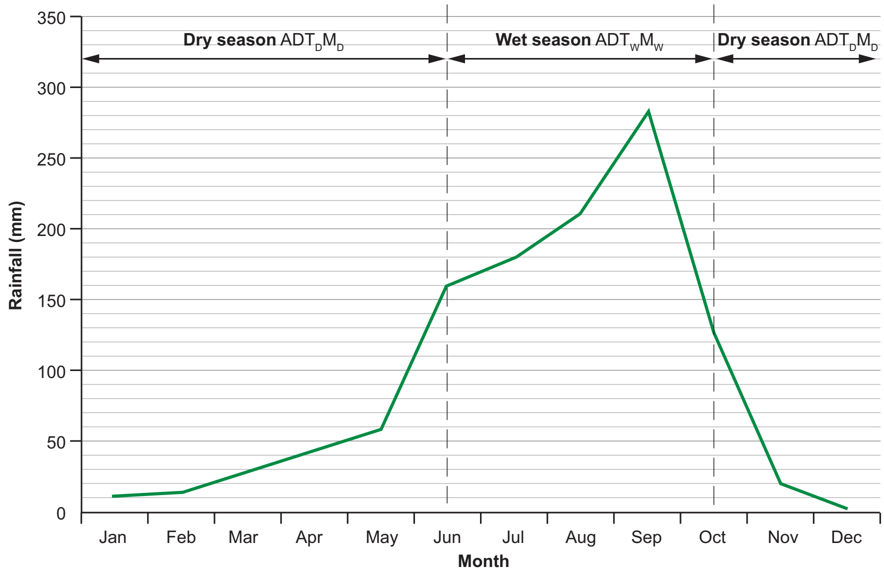



**Figure B.2.2 Basis for traffic count adjustment in relation to seasonal characteristics**

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

$$\text{Weighted Average Daily Traffic (ADT)} = \frac{(ADT_W x M_W) + (ADT_D x M_D)}{12}$$

Where: $ADT_W$ = Average daily traffic count in wet season

$ADT_D$ = Average daily traffic count in dry season

$M_W$ = Number of months comprising the wet season

$M_D$ = Number of months comprising the dry season

Note that $M_w + M_d = 12$.

**Vehicle Classification**

Table B.2.1 shows the vehicle classification system used for compiling the results of the traffic survey described above.

**Table B.2.1 Vehicle classification system categories**

| Category | Vehicle | Description |
|---|---|---|
| 1 | Cars/Taxis | This category includes taxi, private or hired and saloon or estate cars. |
| 2 | Vans, Pick-Ups | This category includes pick-ups, Land Rovers, vans, Jeeps |
| 3 | Small Buses | Small buses include 19-seater buses. |
| 4 | Mammy Wagons/ Medium Buses | Mammy Wagons are special trucks having wooden bodies for conveying both passengers and goods; medium buses include 33-seater buses |
| 5 | Large Buses | Large buses include 54-seater buses and upwards. |
| 6 | Light Trucks | This category comprises 2-axle trucks with single rear wheels or 2-axle truck with twin rear wheels, under 10 tons. |
| 7 | Medium Truck | This category comprises 2-axle trucks with twin rear wheels. |
| 8 | Heavy Truck | This category comprises 3-axle rigid trucks, including tankers. |
| 9 | Semi-Trailers (Light) | These are semi-trailers with any configuration of 3 axles. |
| 10 | Semi-Trailers (Heavy) | These are semi-trailers with any configuration of 4 axles. |
| 11 | Truck Trailers | These are large trucks with any configuration of 5 axles. |
| 12 | Large Truck and Others | These are extra-large trucks with any configuration of 6 axles. This category also includes bulldozers, graders, or other heavy agricultural or construction machinery. |

### 2.2.3 Origin-Destination surveys

OD surveys can be undertaken using a variety of survey techniques. They are carried out to establish the nature of travel patterns in and around the area of enquiry and would normally form part of a regional planning exercise. These surveys serve a number of useful purposes including providing a quantitative assessment of the amount of traffic likely to be affected by the proposal and the resulting impacts on various elements in the road system. Conventional OD survey techniques can be quite labour-intensive, though scope increasingly exists to potentially mitigate this to some extent through the use of bulk travel pattern data generated by mobile phones.

### 2.2.4 Axle load surveys

Axle load surveys provide critical and essential information that is required both for cost-effective pavement design and for the preservation of existing roads. The importance of this parameter is highlighted by the "fourth power law" which exponentially relates increases in axle load to pavement damage; an increase in axle load of 20% would for example result in a more than doubling of the associated pavement deterioration. Information about the loading of vehicles is essential for pavement design. Simplified methods of acquiring vehicle load data are described below.

**Full axle load surveys**

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

- static or dynamic weighing equipment;
- manual or automatic recording of loads; and
- portable or fixed installation.

The quality of the data obtained will depend on the type of equipment used, the duration of the survey and the degree of quality management exercised. In general, the higher the quality of the data, the greater will be the resources required to collect it. Details of how to carry out axle load surveys can be found in the GHA Guidelines for Operation and Management of Weighbridge Stations. Overseas Road Note 40 (A Guide to Axle Load Surveys and Traffic Counts for Determining Traffic Loading on Pavements) provides further guidance on carrying out axle load surveys.

There is a trade-off between available resources and the accuracy obtainable from a sample survey. The art of good survey design is to know when the optimal value for money from the survey is achieved. Further constraints exist for the data analysis stage. Some analysis techniques require expertise, computer hardware and software which may not always be available. The choice of analysis procedures may therefore also involve trade-offs.

An appropriate choice of equipment should be made in relation to such factors as:

- access to back-up support (technical and maintenance);
- ease of installation and use;
- accuracy of measurement required; and
- acquisition and operational cost of equipment.

Axle load information can also be obtained from weighbridge data, though it is important to note that if vehicle operators believe that the purpose of weighing is to enforce the legal limits, they can respond by running reduced loads on their vehicles whilst the monitoring exercise is going on but increasing them again afterwards. As a result, the axle load information obtained from such an exercise may not be accurate.

The minimum information typically derived from axle load surveys is:

- axle load of every axle of all heavy vehicles whether empty or loaded;
- vehicle category; and
- loading in each direction of the road.

Each axle in a multi-axle combination must be measured separately. The survey point should also be equipped with sufficient capacity to weigh all heavy vehicles, both empty and loaded, that are passing in one direction during a defined period of time.

**Simplified method of acquiring axle load data**

Axle load surveys can be expensive and are unlikely to be undertaken for an individual LVR project. Only the heavy vehicles (Categories 5 to 12) need to be evaluated and even they only contribute significantly to pavement deterioration if they are well loaded rather than nearly empty. The type of load is also important because some materials are of high volume but low density and therefore contribute little to the pavement loading. Roads that are likely to carry lorries that are transporting timber, quarry products, building materials and other heavy and dense goods will often be overloaded but a road serving a single village is unlikely to carry such vehicles. Thus, the axle loading of vehicles depends on the function of the road and estimating axle loading without the benefit of a representative axle load survey is not straightforward.

If an axle load survey is not being carried out, information about the vehicle loading can be obtained by observation during the traffic counting survey. The enumerator makes a record, for every heavy vehicle in the heavy vehicle classes, of the state of loading (full, partial or empty) and the type of load (heavy, medium, or light).

## 2.3 Procedure for determining design Traffic Load Class

### 2.3.1 General

The various stages/activities typically followed to determine the traffic loading for pavement design purposes is summarized in Figure B.2.3.

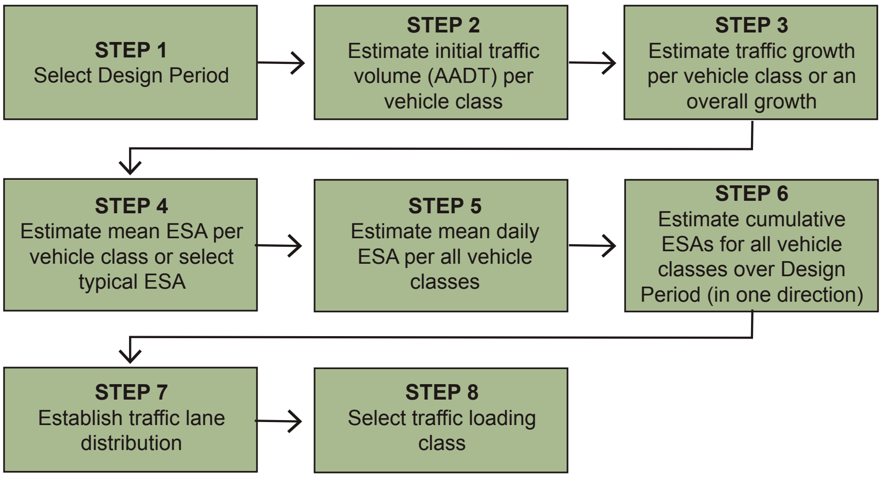



**Figure B.2.3 Procedure for establishing design Traffic Load Class**

### 2.3.2 Step 1 - Select design period

Various factors influence the choice of design period for a road. These include:

- its functional classification;
- its strategic importance;
- funding considerations;
- associated maintenance strategies (highly trafficked facilities will demand long periods of low maintenance activity);
- anticipated time before future upgrading of the road may be necessary; and
- the likelihood that factors other than traffic, e.g. a highly reactive subgrade, will cause distress necessitating major rehabilitation in advance of any load-related distress.

Based on the above factors, Table B.2.2 provides guidance on the selection of the structural design life. It should be noted that the implication of selecting a shorter design life is that another rehabilitation intervention would be required within a relatively short period after the initial construction intervention. This may not always be feasible for budgetary reasons and also not be optimal when considering total life cycle costing. Choosing a longer design life requires greater care in estimating the design traffic loading in order to avoid over/under-design of the pavement, and related cost implications.

**Table B.2.2 Variation of structural design life with importance/level of service of road**

| Type of road | Design Life | |
|---|---|---|
| | Low Importance/ Level of Service | High Importance/ Level of Service |
| Surfaced | 5 – 10 years | 10 – 15 years |
| Unsurfaced | 4 – 7 years | 5 – 10 years |

### 2.3.3 Step 2 - Estimate initial traffic volume per vehicle class

Based on the traffic surveys, the initial traffic volume for each vehicle class can be determined. For structural design purposes, only the commercial vehicles in Categories 3 to 12 inclusive (refer to Table B.2.1) will make any significant contribution to the total number of equivalent standard axles.

### 2.3.4 Step 3 - Traffic growth per vehicle class

Following the establishment of the baseline traffic, further analysis is required to establish the total design traffic based on forecast of traffic growth in each vehicle class. To forecast such growth, it is first necessary to sort traffic in terms of the following categories, as illustrated in Figure B.2.4:

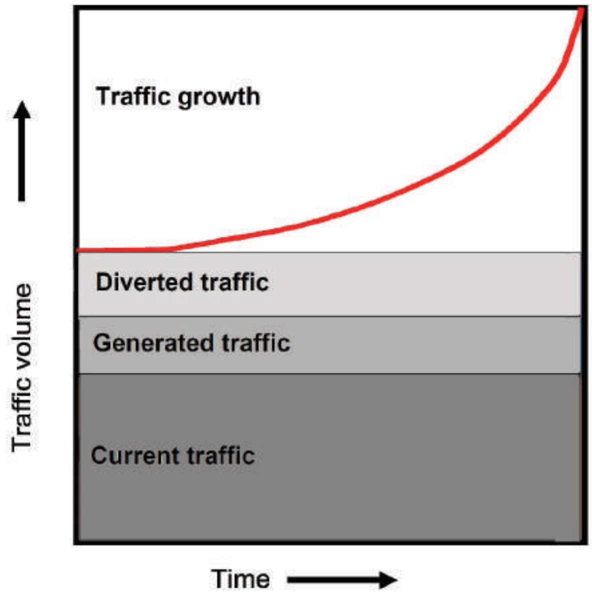



**Figure B.2.4 Traffic development on an improved road**

- **Current or baseline traffic** - Traffic that would pass along the existing road in the absence of any upgrading to a higher standard.
- **Generated traffic** – Additional traffic that occurs in response to the new or improved road. This is 'suppressed' traffic that does not currently exist because of the poor state of the existing road. Local historic precedent can sometimes assist in estimating this. Examining development plans for factories, mines and other commercial developments along the road can be useful in fine-tuning the anticipated generated traffic. A rule of thumb is that generated traffic is typically about 20% of the existing traffic though it can be considerably higher.
- **Diverted traffic** – Traffic that changes from another route to the project road, but still travels between the same origin and destination points. Unless OD surveys have been carried out this can only be estimated based on judgement of the traffic on nearby roads that could benefit from a shorter or more comfortable route.

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

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

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

- **Local historic precedent –** Evidence of traffic growth on roads recently upgraded in the area can be a good guide as to what to expect.
- **Government predictions of economic growth –** Economic growth is closely related to the general growth of traffic. Economic growth rates can be obtained from government plans and government estimated growth figures. The growth rate of traffic should preferably be based on regional growth estimates because there can be significant regional differences.

It should be borne in mind that both geometric design classes and structural design classes are quite wide in terms of traffic range, typically accommodating a doubling or more of traffic levels. As a result, the precision required of traffic estimation is not high.

The AADT in both directions in the first year of analysis consists of the current traffic plus an estimate of the generated and diverted traffic. Thus, if the total traffic is denoted by $AADT_0$ and the general growth rate is "r" per cent per annum, then the traffic in any subsequent year, x, is given by the following equation:

$$\mathrm{AADT_x} = \mathrm{AADT_0}\left(1 + \frac{r}{100}\right)^x$$

This is illustrated in Figure B.2.5, which shows the multiplier for the AADT in the year zero to obtain the AADT in any other year. This Figure also presents an example where an eight-year design life and 4% growth rate has been assumed. The y-axis can be used to read off the resultant multiplier as approximately 1.35. In this case, the year zero AADT can be multiplied by 1.35 to determine the AADT at year eight. A general annual growth rate can be applied to the overall AADT to estimate the increasing traffic.

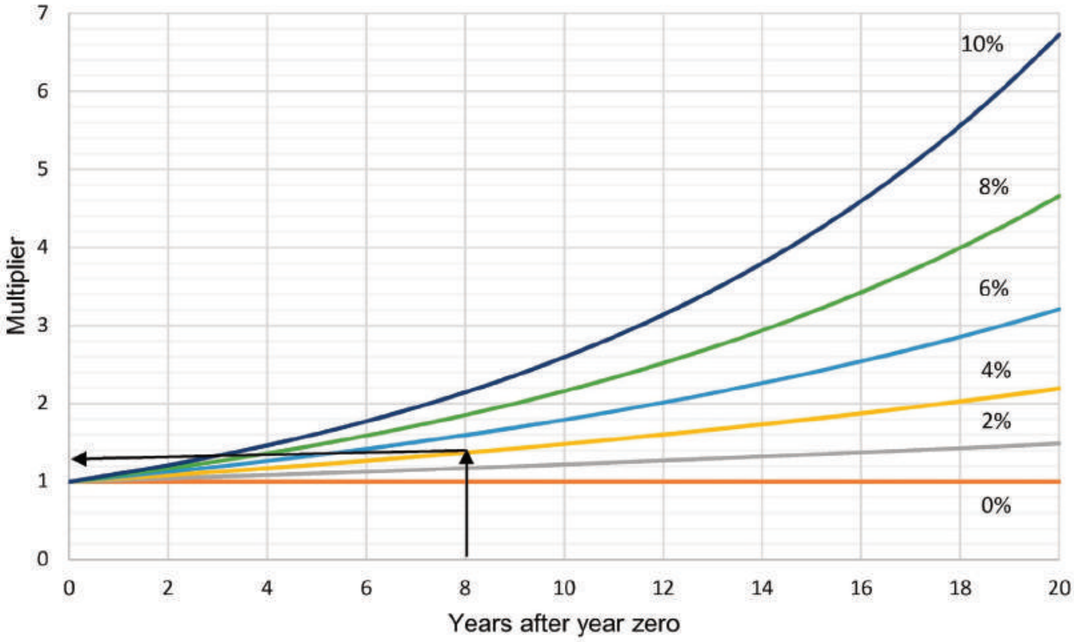



**Figure B.2.5 Multiplier to obtain AADT in any year for different growth rates**

## 2.3.5 Step 4 - Mean ESA per vehicle class

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

The VEF is determined by converting the surveyed individual axle loads to axle load EF, adding up the EFs for each vehicle, and then deriving a representative weighted average value for each vehicle class. In some cases, there will be distinct differences in each direction and separate EFs should be derived for each direction. An example of this would be a LVR which leads to a port, commercial hub, or quarry, where loaded vehicles travel in one direction and travel unloaded in the opposite direction.

The EF is derived as follows:

$$\mathrm{EF} = \left(\frac{P}{80}\right)^n$$

where: P = axle load (in kN) ;

n = power exponent (a value of 4 is typical for LVRs).

The Equivalent Standard Axle (ESA) load is taken as 80 kN.

**Table B.2.3 Average equivalency factors for different vehicle types**

| Category | Vehicle | Axles | Average ESA per vehicle - all vehicles loaded | Average ESA per vehicle – half of vehicles loaded |
|---|---|---|---|---|
| 1 | Cars/Taxis | 2 | The contribution of these vehicle categories to the total cumulative traffic loading over the design period is generally negligible | |
| 2 | Vans, Pick-Ups | 2 | | |
| 3 | Small Buses | 2 | | |
| 4 | Mammy Wagons/ Medium Buses | 2 | | |
| 5 | Large Buses | 2 | 8.9 | 4.45 |
| 6 | Light Trucks | 2 | 8.9 | 4.45 |
| 7 | Medium Truck | 2 | 8.9 | 4.45 |
| 8 | Heavy Truck | 3 | 9.3 | 4.65 |
| 9 | Semi-Trailers (Light) | 3 | 17.0 | 8.5 |
| 10 | Semi-Trailers (Heavy) | 4 | 17.4 | 8.7 |
| 11 | Truck Trailers | 5 | 17.9 | 8.95 |
| 12 | Large Truck and Others | 6 | 16.6 | 8.3 |

Guidance on the average VEF for different vehicle classes derived from historical data in Ghana is given in Table B.2.3. However, data from any recent axle load survey on the road in question or a similar road in the vicinity is preferred over using countrywide averages. If an axle load survey is not being carried out, information about the vehicle loading can be obtained by observation during the traffic counting survey. The enumerator merely records, for every heavy vehicle in the heavy vehicle classes, the state of loading (full, partial or empty) and the type of load (heavy, medium, or light). This should be modified based on the nature of traffic in the project area. Only full vehicles will make a significant contribution except for vehicles carrying dense loads which may be overloaded even when partially full.

It should be noted that the overloading limits set by government policy and legislation is not the same as an ESA.

### 2.3.6 Step 5 - Mean daily ESA for all vehicle classes

The estimated mean daily ESAs for each vehicle class (Daily $ESA_i$) is obtained from the traffic data derived in Step 2 and the VEFs derived in Step 4 as follows:

$$\text{Daily ESA}_i = \text{AADT}_i \text{ x } \text{VEF}_i$$

The sum of the daily ESA for each vehicle category will provide the total Daily ESA as demonstrated in Figure B.2.6.

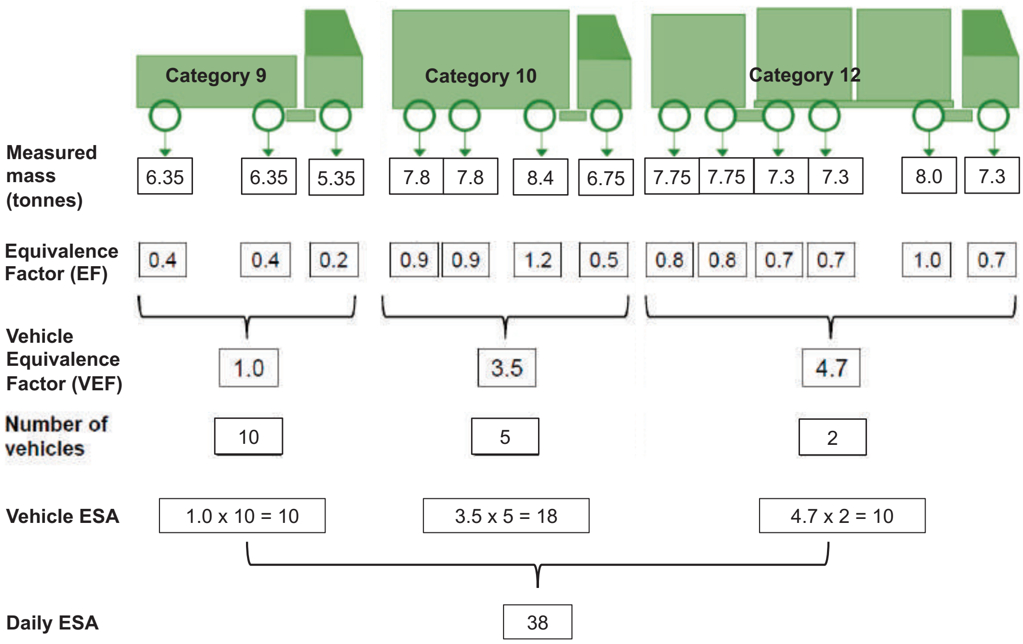



**Figure B 2.6 Example of derivation of mean daily ESA for mix of vehicle types**

The VEFs given in Table B.2.3 are based on the legal permissible axle loads in the ECOWAS region. The VEFs presented in Figure B.2.6 are an example of how a designer would calculate the daily ESA based on measured axle loads. If axle load data are not available, the designers may use Table B.2.3 as a starting point.

### 2.3.7 Step 6 - Cumulative ESA for all vehicle classes over the design period

For pavement design the cumulative ESAs in each direction for each traffic category expected over the design life may be obtained from the following formula:

$$\text{Cumulative ESA} = 365 \times \text{Daily ESA} \times \left[\frac{[(1+r)^n - 1]}{r}\right]$$

Where: Daily ESA = mean daily ESAs for all vehicle classes in the year zero (each direction) as taken from Step 5)

r = assumed annual growth rate expressed as a decimal fraction

N = design period in years (from Step 1)

Figure B.2.7 shows the multiplier for the Cumulative ESA in year zero to calculate the Cumulative ESAs after any other number of years up to year 20. An example is also presented in this figure, where a 10-year design life and 6% growth rate has been assumed. The y-axis can be used to read off the resultant multiplier as approximately 13. In this case, the year zero ESA can be multiplied by 13 to determine the cumulative ESA at year 10.

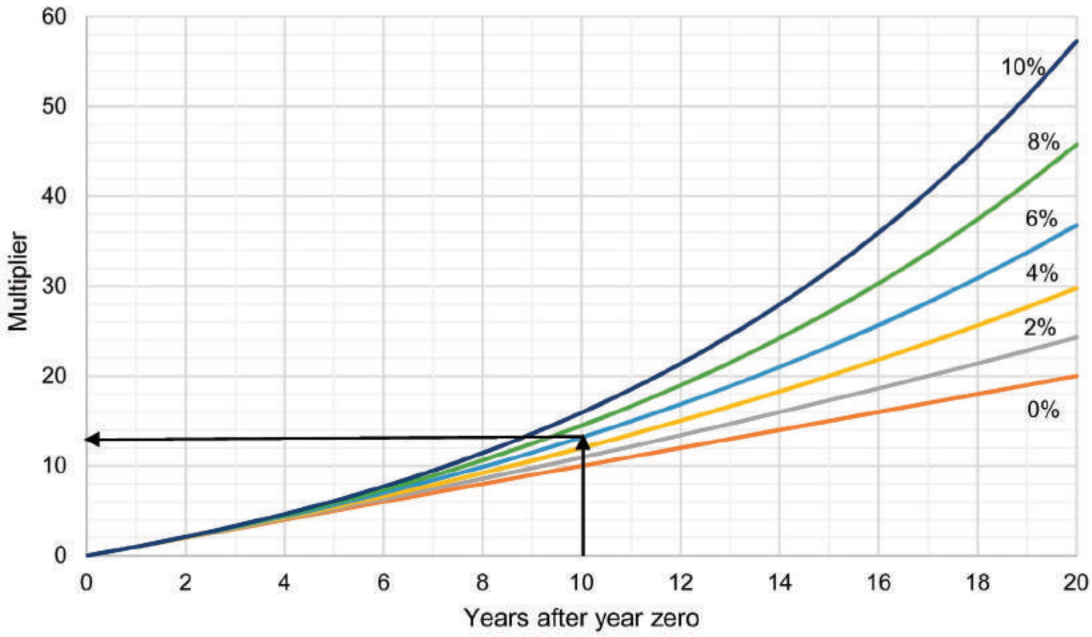



**Figure B.2.7 Effect of traffic growth rates on cumulative ESA over time**

### 2.3.8 Step 7 - Traffic lane distribution

The actual design traffic loading (ESAs) needs to be corrected for the distribution of heavy vehicles between the lanes in accordance with Table B.2.4 for paved roads and Table B.2.5 for unpaved roads.

**Table B.2.4 Lane width adjustment factors for design traffic loading – paved roads**

| Paved Width | Corrected design traffic loading (ESA) | Explanatory notes |
|---|---|---|
| *≤ 4.5 m* | The sum of ESAs in both directions. | Traffic in both directions uses the same lane. |
| > 4.5 m, but < 6 m | 80% of the sum of ESAs in both directions. | To allow for overlap in the centre section of the road, but not all vehicles use the same wheel tracks as for the narrower road. |
| ≥ 6 m | The ESAs can be determined for each direction, with 100% of the higher ESAs of the two directions being adopted as the corrected design loading ESA. | No traffic overlap in the centre section of the road. |

If there is more than one lane in a particular direction the corrected design traffic loading can be assumed to be 90% of the total ESAs in the studied direction.

**Table B.2.5 Lane width adjustment factors for design traffic loading – unpaved roads**

| Carriageway width | Corrected design traffic loading (ESA) | Explanatory notes |
|---|---|---|
| ≤ 4.5 m | The sum of ESAs in both directions. | Traffic in both directions uses the same middle part of the road in a channelised manner. |
| > 4.5 m, but < 6 m | 80% of the sum of the ESAs in both directions | Traffic in both directions uses the same middle part of the road, but with more lateral wander than for a narrower road |
| ≥ 6 m | 60% of the sum of the ESAs in both directions. | To allow for some overlap in the middle part of the road |

## 2.4 Traffic Load Classes

All survey data are subject to errors. Traffic data, in particular, can be very inaccurate and predictions about traffic growth are particularly prone to large errors. Accurate calculations of cumulative traffic are therefore very difficult to make. To minimize these errors there is no substitute for carrying out specific traffic surveys for each project. Methods of carrying out classified traffic counts for this purpose are described in Section 2.2.

The design method recommended in this Manual provides standard pavement structures for ranges of traffic as shown in Table B.2.6. Therefore, as long as the estimate of cumulative equivalent standard axles is not near one of the traffic class boundaries, traffic errors are unlikely to affect the choice of pavement design. However, if estimates of cumulative traffic are close to the boundaries of a traffic class, then the basic traffic data and forecasts should be re-evaluated, and sensitivity analyses carried out to ensure that the choice of traffic class is appropriate. If there is any doubt about the accuracy of the traffic estimates, it is prudent to select the next higher traffic class for design.

**Table B.2.6 Traffic Load Classes for Pavement Design**

| Traffic Load Class (TLC) | | | |
|---|---|---|---|
| *Traffic range (ESA x $10^6$)* | *TLC 0.01* | *TLC 0.3* | *TLC 1.0* |
| | < 0.01 | 0.01 – 0.3 | 0.3 – 1.0 |
| Typical No. heavy vpd | ≤ 2 | 3 - 20 | > 20 |

NOTES:

1. Heavy vehicles are taken as the vehicles within categories 5 to 12 as described in Table B.2.1
2. The number of heavy vehicles per day are included as an indication of typical daily volumes that can be expected in each Traffic Load Class, but the ranges of heavy vehicles per day as shown in the table should not be used to determine the Traffic Load Class

# 3. SITE INVESTIGATION

## 3.1 Introduction

Site investigation is an integral component of the location, design, improvement and maintenance of a LVR and provides essential information on the soil characteristics, construction materials availability, topography, land use, drainage, environmental issues (including climate) and social considerations that are essential for decision-making and design. While site investigation is most critical for the route selection and the design of new roads, it is also indispensable to the improvement, maintenance and operation of existing roads. The site investigation should not be confined or limited to the Right of Way but should be broadened to ensure that the relationships between the road engineering and the surrounding geological, hydrological, social and environmental factors are fully considered.

There already exist in Ghana a number of documents that contain guidance on techniques for site investigation. These include the Guidance Notes for the Design of Rural Feeder Roads (DFR, 2004) and the Guidance Notes for the Design of Drainage Structures (DFR, 2005). These documents emphasise the importance of visual observation and inspection during site investigation and outline the approaches that should be adopted for each activity.

## 3.2 Stages of site investigation

Site investigations are usually undertaken as an iterative process, combining desk studies, field investigations analysis and reporting. They vary in areal extent and detail according to the stage of the project, from broad and area-specific at pre-feasibility study and route selection stage through to detailed and site-specific at detailed design stage. The scope of a site investigation also varies according to project type, with comprehensive environmental, social, engineering, economic and traffic studies usually being required for new roads and more localised, site-specific studies required for road improvement schemes. Site investigations themselves are also usually phased, with an initial fact-finding and scoping phase being followed by a more detailed phase for design purposes.

Methods and techniques of desk study and field investigation are described below. These are focused on the breadth of investigations normally required for new road construction or road realignment and major widening associated with road improvement schemes. The scope of any site investigation needs to be ‘fit for purpose’ and cost-effective, covering all issues that need to be addressed, avoiding unnecessary investigations, but ensuring that all the key considerations are fully addressed and catered for in the design.

### 3.2.1 Desk studies

Desk studies are usually most critical at the early planning stages of a project when a wide range of factors need to be considered. As described in Section 3.2.2, desk studies also allow effective planning of field investigations, and it is vital to study all the relevant information available. Studying existing documents, including site investigations from nearby projects or earlier project phases, and examining maps, aerial photographs (if available) and satellite imagery often provides useful contextual information and can help to rationalize and consolidate the main site investigation, potentially saving a considerable amount of time and money. Topographic maps provide an overview of the relief, including the main rivers, streams, ridges, cliffs and areas of steep ground, and well as the location of towns and other settlements and the existing road network. However, recent changes to the human landscape, such as new roads and houses, may not be recorded, and more up to date information can usually be obtained from satellite imagery available from Google Earth.  Table B.3.1 lists some of the sources of satellite data that are capable of providing terrain data for use in the location, design and management of LVRs. It should however be noted that most satellite sensors are unable to penetrate a dense tree canopy.

**Table B.3.1 Potential application of satellite imagery to LVRs**

| Application type | How satellite imagery can be used | Suitable scales | Most suitable satellite data* |
|---|---|---|---|
| Mapping of route corridors or mapping for route alignment selection | ▪ Mapping of geology, soils and land use<br>▪ Mapping of topography (if not available from elsewhere)<br>▪ Mapping of geohazards such as landslides, areas of erosion and flood prone areas<br>▪ Broad terrain classification<br>▪ Mapping of existing infrastructure | 1:50,000 to 1:25,000 | Combination of multi-spectral and optical data — i.e., Landsat, SPOT V, ASTER or any of the high-resolution sensors |
| Landslide susceptibility and hazard mapping for existing or proposed road corridors | ▪ Landslide identification<br>▪ Mapping landslide conditioning and triggering factors (Chapter A.4)<br>▪ Mapping of land use and infrastructure | 1:25,000 to 1:5,000 | Combination of multi-spectral and optical data — i.e., Landsat, SPOT V, ASTER or any of the high-resolution sensors |
| Mapping catchment areas and drainage patterns | ▪ Derivation of topography and drainage network from DEM data | 1:25,000 to 1:5,000 | Combination of multi-spectral and DEM data — i.e., SPOT V or ASTER GDEM data |
| Management of existing road corridors | ▪ Mapping changes in land use<br>▪ Mapping changes in drainage conditions<br>▪ Monitoring of geohazards such as landslides, areas of erosion and flood prone areas | 1:25,000 to 1:5,000 | Any of the high-resolution sensors repeated at regular intervals |
| Identification of potential sources for suitable construction materials | ▪ Mapping of geology and soils in suitable detail | 1:25,000 to 1:5,000 | Combination of multi-spectral and optical data — i.e., ASTER, SPOT V or any of the high-resolution sensors |
| Management of existing roads | ▪ Mapping of existing assets<br>▪ Monitoring of problematic areas (such as unstable slopes, landslides or areas of erosion) | 1:10,000 to 1:5,000 | Any of the high-resolution sensors repeated at regular intervals |
| Road design and construction | ▪ Detailed topographical mapping (if not available elsewhere)<br>▪ Mapping of geology, soils and land use in detail<br>▪ Mapping of geohazards such as landslides, areas of erosion and flood prone areas<br>▪ Detailed terrain classification<br>▪ Mapping of any existing infrastructure | 1:5,000 | Combination of multi-spectral and optical data — i.e., SPOT V, ASTER, or any of the high-resolution sensors plus ground control points for topographical mapping |
| Landslide monitoring | ▪ Detection of landslide movements | 1:5,000 | Combination of optical and SAR data — i.e., any of the high-resolution sensors, as well as InSAR technology repeated at regular intervals |

**ASTER, Advanced Spaceborne Thermal Emission and Reflection Radiometer; GDEM, Global Digital Elevation Model; SAR, Synthetic Aperture Radar; InSAR, Interferometric Synthetic Aperture Radar*

*SOURCE: Hearn (2011).*

Stereo aerial photographs, where available, provide a valuable means of assessing topography, geology and drainage features. Unfortunately, as far as can be ascertained, stereo aerial photographs are only available in Ghana for the main cities.

Topographical maps are available in digital format at 1:50,000 scale in Ghana and are available from the Centre for Remote Sensing and Geographic Information Systems (CERSGIS). Although these maps provide only limited detail, they do allow broad assessment of the terrain for route selection (see Chapter A.4) and drainage network analysis. Topographic maps can be supplemented through the interpretation of Google Earth satellite imagery and the free to download digital elevation data available from ASTER and SRTM satellites as summarised in Table B.3.2.

**Table B.3.2 Digital mapping data from the common satellite sensors**

| Sensor | Horizontal resolution | Horizontal accuracy | Vertical accuracy | Scene size | Data collected |
|---|---|---|---|---|---|
| ASTER | 30 m | 50 m | 15-30 m | 60 km x 60 km | Since 1999 |
| GDEM | | | | | |
| SRTM | 3 Arc seconds (90 m) | 50 m | 15 m | 1° lat x 1° long | Acquired 2000 |
| SPOT | 20 m | 15 m | 10-15 m | 60 km x 60 km | Since 2002 |
| IKONOS | 1-2 m | 1-2 m* | 1-2 m* | 11 km x 11 km | Since 2000 |
| WorldView-2 | 0.5 m | 1-2 m* | 1-2 m* | 16.4 km x 16.4 km | 2009 |
| GeoEye-1 | 0.5 m | 1-2 m* | 1-2 m* | 15 km x 15 km | 2008 |
| Airborne LiDAR | 0.5 m | 0.5 m | 0.10-0.25 m | Dependent on area of interest | On demand |

** Dependent on the supply of Differential GPS points.*

*SOURCE: Hearn (2011)*

Geological mapping of Ghana is available at 1:250,000 and 1:1 million scales. While of interest, mapping at such small scales offers little detailed interpretation for site investigation and will need to be supplemented by field observations. It is recommended that the Ghana Geological Survey Authority of the Ministry of Lands and Natural Resources is consulted at the commencement of the desk study to ensure that all available relevant data is collected and reviewed. For environmental and biodiversity conservation data, consultation is recommended with the Environmental Protection Agency and the Forestry Services Department and the Wildlife Division of the Ghana Forestry Commission. Rainfall and river gauging data can be obtained from the Meteorological Services Department. The Ghana Statistical Service should be able to provide data and advice regarding the collection of social and economic data, where appropriate. Consideration should also be given to the effects of expected climate change. In Ghana, average annual temperatures are expected to increase over the coming decades and annual rainfall is expected to reduce, possibly by up to 20% in the worst-case scenario by 2080 compared to the 1960-2000 average (Twerefou et al, 2015). Despite this, rainfall is likely to become more erratic, and there may be an increased tendency for higher intensity rainfall.

## 3.2.2 Field investigations

Field investigations are undertaken to complement the findings of the desk study, and to gather data that will allow the preferred route to be selected (in the case of a new road construction project) and enable the design to be progressed taking into account the following factors and considerations:

**Environmental and social**

- Areas of environmental conservation and habitat value;
- Areas of historical, cultural and religious significance;
- High value agricultural land and land use that has significance for livelihood sustainability;
- Important sources of community water supply;
- Thoroughfares for community access that must be preserved in the design; and
- Other social aspects that may need to be accommodated or mitigated in the design.

**Traffic and engineering**

- Traffic composition and OD data for feasibility studies and design traffic loadings;
- Topographic data for geometric design and design of earthworks and structures;
- Location of watercourses and other drainage features, including field assessments of flow regimes and scour potential;

- Inspection of existing structures, including bridges, culverts and retaining walls to assess their suitability for inclusion in the works;
- Inspection of stream and river crossings for the design of new hydraulic structures; and
- Assessment of pavement performance in the case of upgrades of existing roads as well as the performance of pavements and pavement materials on neighbouring roads.

**Ground conditions and materials**

- Subgrade suitability, location and thickness of any problematic soils and bearing capacity for foundations (see Section B.3.4);
- Inspections of slopes (natural and man-made) for evidence of distress caused by ground movement and erosion (see Sections B.3.5 and B.3.6); and
- Sources of potential construction materials, including fill for subgrade replacement, embankments and sub-base and aggregate for base course, road surfacing and structural concrete (see Section B.3.7).

This information is usually presented in a series of documents for consideration by the client and other interested parties and stakeholders for disclosure, consultation and decision-making, allowing the project to be progressed through the project cycle. These documents normally include separate volumes dealing with the following aspects:

- Route Selection, Alignment Survey and Geometric Design;
- Traffic and Traffic Loading;
- Materials and Subgrade Design;
- Pavement Design;
- Hydrology, Drainage and Water Crossings;
- Ground Stability and Geotechnical Design;
- Environmental Impact Assessment (EIA) and Environmental Management Plan (EMP); and
- Social and Complementary Activities.

## 3.3 Ground investigation techniques

### 3.3.1 Applicable standards

Ground investigation is a term used to describe the investigation of subsoil conditions, usually by trial pitting or trenching, probing and drilling, and sometimes with the assistance of surface geophysics. A suitable industry standard, such as BS5930 (BSI 2015), should be used when applying these techniques of ground investigation. The choice of methods for ground investigation is determined by the type of road project and the ground conditions observed or anticipated. Use of the terms 'observed' and 'anticipated' is critically important to this process. Construction projects have often run into difficulties when ground investigation is either not implemented at all or is implemented 'blindly', without observation and anticipation. If a ground investigation is to be effective, it must be carried out in a systematic way, using industry-standard techniques that are relevant to the soils and rocks to be investigated and cost-effective in yielding the required design information. For LVRs, investigations should employ standard and simple investigation techniques and basic soils testing wherever possible. More sophisticated and expensive procedures should only be employed when significantly problematic ground conditions are encountered. Under such circumstances it is advisable to seek specialist advice.

### 3.3.2 General requirements

Most ground investigations undertaken for LVRs are for subgrade assessment for pavement design purposes, as described in Section B.3.4. However, ground investigations will also be required for construction materials (see Section B.3.7) and wherever the bearing capacity and stability of the underlying soils and rocks needs to be investigated at depth, for example for the foundation design of bridges and other large drainage structures, retaining walls, high fills and the design of slope stabilisation works. Borehole and drill hole investigations will be necessary where:

- there is a requirement to assess ground conditions deeper that the practical depths of trial pit excavation;
- an indication of *in situ* density and shear strength of soils is required at depth using the Standard Penetration Test (SPT);
- undisturbed sampling is required for triaxial and oedometer tests; or
- investigations need to be made of the weathering, fracturing, strength and lithology of the underlying rock.

Borehole and drilling investigations will be required for the foundation design of bridges and other major structures, in areas of deep-seated unstable ground or where significant thicknesses of problematic soils are suspected. These investigations are not commonly used on LVR projects, and especially on road improvement schemes where existing structures should be retained wherever possible. Areas prone to significant landslides and slope instability should have been avoided in the route selection (see Chapter A.4), so deeper investigations are usually only required for bridge foundations on new roads or where previously unforeseen ground conditions need to be addressed during the design of an improved road. For example, it is often the case that the subtle effects of slow-moving landslides on gravel road performance go unnoticed, and it is only when such roads are sealed that the cracks become apparent. This outcome is usually the result of an inadequate site investigation during route selection. Further information on this important subject in hilly and mountainous areas is given in Section B.3.5.

## 3.4 Subgrade assessment

### 3.4.1 Purpose

The subgrade is the foundation layer for the pavement. An assessment of its condition in terms of the level of support provided to the pavement structure is therefore one of the most important factors, in addition to traffic loading, in determining pavement thickness design, composition and performance. This level of support, as characterised by subgrade strength or stiffness, is dependent on the soil type, density and moisture conditions at construction and during service. The purpose of subgrade assessment is therefore to estimate the support that the subgrade will provide to the pavement during its design life. The characteristics of the underlying soils and rocks are also important for assessing the stability of slopes, both naturally and in excavation, and for the design of earthworks in general. Characteristics that help determine the engineering behaviour of the ground are controlled by a number of factors, including:

- whether or not the subgrade is composed of rock or soil;
- if it is composed of rock, its lithology, structure and weathering condition;
- if it is composed of soil, its depth, whether it is derived from *in situ* weathering (eluvial soil), fluvial deposition (alluvial soil), or downslope transport under gravity (colluvial soil);
- if it is eluvial soil, its weathering grade, and its physio-chemical properties and whether it contains clay minerals that make the soil expansive;
- the grading of all soils (eluvial, alluvial and colluvial) and their *in situ* density, both of which help determine bearing capacity and compressibility under loading; and
- the groundwater regime, i.e. the depth to the water table during the wet and dry season and the natural moisture content of the soil.

### 3.4.2 Trial (test) pitting

The frequency or spacing of trial pits should be based on the apparent uniformity of ground conditions. Spacings should be decreased when the subsurface soils demonstrate more variability. Trial pits can be staggered either side of the proposed or existing centre line so as to cover the full width of the road formation.

Machine-dug trial pits can usually be excavated to depths of up to 3 m, depending on ground conditions and the depth of the water table. Health & Safety regulations prevent personnel from entering trial pits once they have advanced beyond a certain depth (usually 1 m) unless proper shoring is in place to prevent collapse. For a new road, the depth of trial pitting should not be less than 2 m, unless weathered rock is encountered. For upgrading and rehabilitation projects there is usually vehicular access and hence pits can be excavated using a backhoe through all the existing pavement layers or directly into adjacent original ground. The presence of pavement layers will reduce the depth to which trial pits can be excavated into the subgrade, though 1.5 m is the recommended minimum, unless weathered rock is encountered. The presence of large boulders within the top 1 m of the trial pit may not necessarily indicate equally strong materials at depth, as these materials may be underlain by softer, more clayey layers. A limited number of deeper pits may also be needed in order to confirm soil profiles, the bedrock interface and groundwater conditions at depth, though 5 m is the normal practical limit of machine-dug trial pit excavation. Where there is a distinct horizon change between loose and dense soils, or soil and the underlying rock, surface geophysics (mostly soil resistivity) can prove a cost-effective supplement once calibrated.

Undisturbed and disturbed (bulk) samples can be taken from trial pits for laboratory testing (see Section B.3.4.4). At least one sample should be taken from each soil horizon. The soil profile and its constituent materials should be logged carefully during the trial pitting investigation, in accordance with the descriptions provided in BS5930 or an alternative industry standard. This will allow an assessment of founding conditions as well as of the suitability of materials in excavations for use as embankment fill. The *in situ* shear strength of clay soils can be measured directly using a shear vane or hand penetrometer in the sides or base of trial pits (ASTM D2573).

Other portable equipment can be used for field testing, thus reducing the dependency on sample transport to often distant laboratories. Using portable field equipment can help to streamline the sampling and testing process. The portable kit developed by CSIR, for example, (Paige-Green 1988) includes a linear shrinkage mould, sieving and weighing equipment and a solar drying oven. Plasticity is taken as approximately twice the value for linear shrinkage. The California Bearing Ratio (CBR) can be approximated from plasticity index and percentage fines content using published linear regression relationships.

### 3.4.3 Hand-probing and auguring

Hand-operated probes, such as the DCP or the Mackintosh probe, can provide an important source of information on subgrade conditions. The DCP (ASTM 2006) was developed to assess the structural properties of unbound pavement layers (see Section B.6.3). However, it can also be used to assess the layer thicknesses and density of the subgrade. The penetration rate is inversely proportional to the resistance of the ground to the penetration of the DCP cone and may be related to the *in situ* CBR or soil density under field moisture conditions. A number of correlations exist that link the DCP penetration rate (mm/blow) to the subgrade strength parameters required for pavement design. These correlations are based on either soaked or unsoaked CBR values versus DCP penetration rates measured in different soil types. It is important to make sure that the correlation being used is the correct one for the purposes of the study. In general, the correlation should be between the DCP penetration rate and the actual CBR of the material being tested, i.e. the CBR at the density and moisture content of the material at that time. Soil moisture content significantly affects DCP penetration rate, especially in fine-grained plastic soils (Singh et al 2017). Another consideration is skin friction, the friction imposed by the confining layers on the DCP rod as the cone is advanced, though this is usually only significant if the DCP test is not performed vertically (Livneh 2014).

Use of the DCP to assess subgrade conditions for pavement design purposes for new roads is normally only applicable where they are to be located in flat terrain, where little cut or fill is envisaged.

The standard depth of investigation using the DCP is 850 mm, but this can be increased to 2 m using extension rods. The investigation depth can also be extended by probing into the floor of trial pits, assuming the necessary safety precautions are in place. The frequency or spacing of DCP tests will vary according to the observed variability in ground conditions, but quite often this cannot be determined solely by surface observation, and it will be necessary to experiment with the device to determine how uniform the soils in a given area appear to be. As a rule of thumb, one DCP test per hundred metres should be regarded as the norm, with higher frequencies applied to more changeable ground conditions.

The progress of the DCP test is inhibited by the presence of cobbles and boulders in the soil profile, and the validity of the test can be significantly compromised if cone penetration is controlled by large rock fragments within the soil matrix. For these reasons, investigations using the DCP in coarse-grained colluvial soils (i.e. stony ground) are not recommended.

The DCP can also be used to help determine the spacing and locations of trial pits by providing a rapid indicator in changing subsoil conditions within the study area. Used in combination, the DCP and trial pitting can usually provide sufficient information for subgrade assessment. A range of cone types and techniques are available and some of these are described, for example, in Mayne (2007). The Mackintosh probe (e.g. Fakher et al. 2006) is used principally for determining the depth of soft soils and can provide a means of assessing approximate undrained shear strength ($C_u$).

DCP test data have been correlated against SPT (Standard Penetration Test) N values derived during borehole investigations (e.g. Opuni et al 2017; Ampadu et al 2018). Opuni et al obtained a fairly good correlation between normalised value of SPT N and a DCP $N_{10}$ value (number of blows to achieve 10cm soil penetration) thus allowing the DCP to be used as a surrogate for the SPT in the determination of allowable bearing capacity for shallow foundations. Such an approach could be used for the foundation design of minor structures such as culverts and short-span bridges along LVRs. It is important however not to import relationships from other areas without calibration. Relationships must be derived from the same soils and moisture contents that exist on site.

Auguring can be carried out by hand or by machine. Hand auguring is suitable in self-supporting (firm to stiff) fine-grained soils down to a depth of a few metres but is unable to penetrate hard obstructions such as cobbles, boulders or rock. Machine auguring, particularly using hollow-stem augers, can be used to reach greater depths and to extract samples through the stem, but will also encounter similar difficulties in heterogeneous and large-sized material (Hunt and Hearn 2011).

## Laboratory testing

Soil tests commonly undertaken on disturbed samples include natural moisture content determination, Atterberg limits, particle size distributions and compaction testing of materials to be used as fill. These are described in Table B.3.3.

**Table B.3.3 Potential range of laboratory tests on recovered samples**

| Category | Test | Remarks |
|---|---|---|
| Classification tests (disturbed or undisturbed samples) | **Moisture content** | Together with plasticity can be helpful in estimating the undrained strength of cohesive materials. |
| | **Liquid and plastic limits, linear shrinkage and shrinkage limit** | For classifying the fine-grained fraction of soils, can be helpful for estimating both drained and undrained strength of cohesive materials. |
| | **Bulk density** | For undisturbed samples only, to use in the calculation of forces exerted by the soil. |
| | **Particle size distribution** | |
| | a) Sieving | a) to determine the grading of a soil coarser than silt size; with description of particle angularity can be used to estimate friction angle of granular soils. |
| | b) Sedimentation | c) to determine the relative proportions of silt and clay. |
| Compaction-related tests (disturbed samples) | **Dry density** | To determine the mass of solids per unit weight of soil. |
| | **Standard compaction tests** | To determine Maximum Dry Density and Optimum Moisture Content at which fill materials should be placed. Results used in the control of earthworks, e.g. embankments. |
| Soil strength tests (undisturbed samples) | **Triaxial compression** | To determine strength of cohesive soils. |
| | a) Unconsolidated undrained | a) undrained tests to assess undrained shear strength (cu). |
| | b) Undrained with measurement of pore pressure | b) and c) undrained or drained tests with the measurement of pore pressure to assess shear strength parameters in terms of effective stress (c′ and ϕ′). |
| | c) Drained | |
| | d) Multi-stage | d) useful if short of samples, but single stage tests usually more reliable. |
| | **Unconfined compression** | Only suitable for saturated, uniform fine-grained soils. |
| | **Laboratory vane shear** | Only for soft and firm clays. May not give representative results for remoulded samples; usually better to take in situ vane shear measurements if possible. |
| | **Direct shear box** | Cheaper alternative to undrained triaxial test. Drainage conditions cannot be controlled during testing. Samples can be orientated, residual strength can be determined. |
| | a) Multiple reverse shear box | To determine the residual shear strengths of cohesive soils. a) and b) use undisturbed or remoulded samples for use in the absence of a pre-formed shear surface. |
| | b) Ring shear | |
| | c) Triaxial test with pre-formed shear surface | c) Preparation and alignment of undisturbed samples for c) and d) test specimens can be very difficult. |
| | d) Shear box test with pre-formed shear surface | |
| Soil deformation tests | **One-dimensional consolidation test (oedometer)** | To determine the magnitude and rate of settlement of soft soil under loading. Samples to be undisturbed. |

*SOURCE: from Hunt and Hearn 2011*

NOTE: Soil classification and compaction tests are likely to fulfil most pavement design requirements on LVRs, unless slope stability and foundation design are being investigated

If a detailed investigation is deemed necessary, more specialized testing such as triaxial or shear box testing on undisturbed samples may be appropriate. However, undisturbed samples can only be obtained using more sophisticated boring/drilling techniques or by taking block samples from trial pits. Where embankments are to be constructed on compressible clay soils, located on valley floors for example, then oedometer tests to determine consolidation parameters may also be required to enable the magnitude and rate of settlement to be estimated. Undisturbed samples are also required to perform these tests.

An important consideration in the humid tropics and subtropics is the fact that aggregation of clay particles can occur in many residual soils. During sieve analyses these particles are recorded as silt-sized, but they can break down under loading or dynamic testing (such as Atterberg limit testing) into their constituent clay particles and behave quite differently (Fookes 1997).

## 3.5 Earthworks investigations in steep terrain

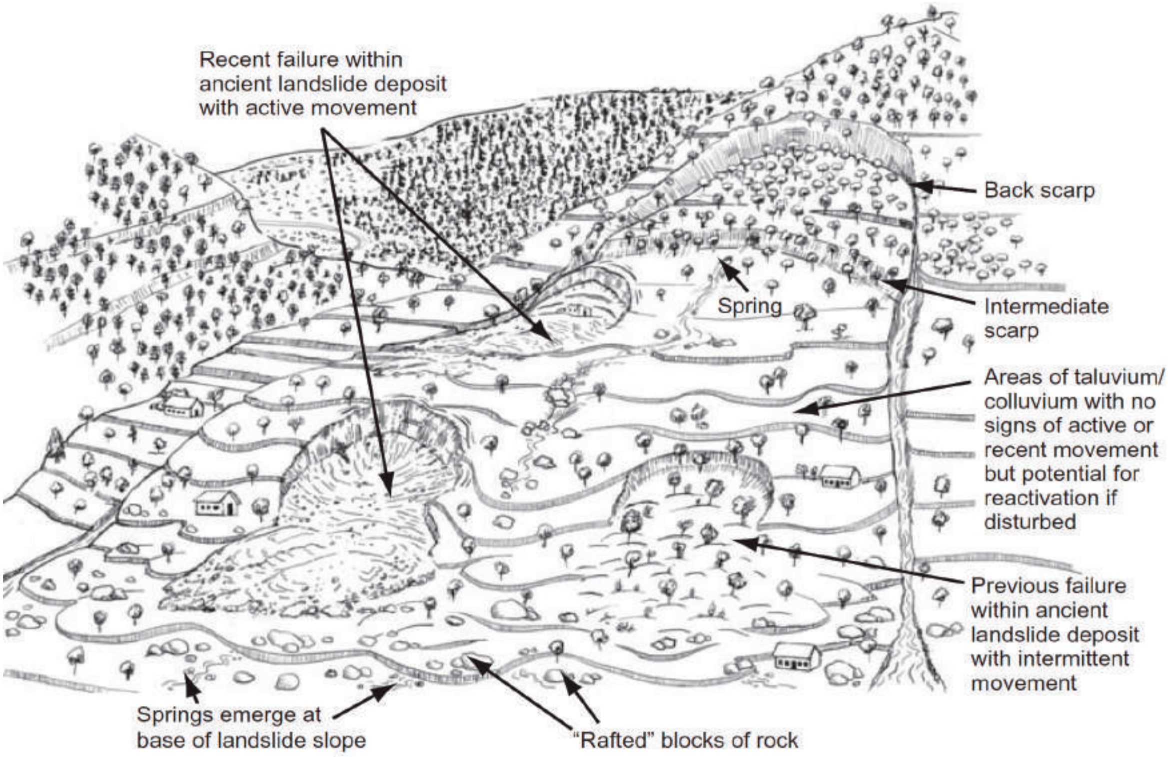
### 3.5.1 New road construction

In the case of new road construction, it will be necessary to rely on desk study (Section B.3.2.1) and field investigations to identify slopes susceptible to ground movement. Where these problems are judged to be particularly severe they should be avoided in the route selection (Section A.1.10). Field evidence for ground movement and slope instability can sometime be extremely subtle. Some of the key features to look out for include:

- an irregular or hummocky slope profile;
- the presence of a steep upper section, or back scarp, from which the material forming the slope below has become detached;
- the presence of tension furrows and tension cracks that usually run approximately perpendicular to the direction of maximum slope and along which ground rupture has taken place;
- the presence of an apparent toe or bulge on the lower part of the slope, often associated with springs and waterlogged ground;
- obvious distress and damage to structures located on the slope;
- the presence of boulders and displaced (‘rafted’) blocks of rock, and usually chaotic deposits of boulders and other debris; or
- a vegetation cover that is different from that found on adjacent slopes that cannot be accounted for by land use change (though the lack of a different vegetation cover does not preclude the presence of a landslide).

Figure B.3.1 illustrates some of the subtle and less subtle landforms associated with past and active ground movement. Specialist engineering geological advice should be sought when designing roads through hilly and mountainous terrain. Further guidance on landslide identification and the investigation of the main causes and areal extent of slope instability is given in Section B.3.7.

Road construction can trigger landslides and related slope instability that can become extremely difficult to manage as rainfall causes them to become more active and to enlarge in area on steep slopes. These ‘first-time failures’ can be difficult to predict, but usually occur where excavations expose rock with adverse bedding and jointing patterns or where thick colluvial deposits become disturbed by excavation, spoil disposal and road drainage effects. First-time failures can also be triggered by erosion below culverts or in river banks below an alignment, and it is important to employ specialist advice when designing roads through these areas. Some guidance is given in Table B.3.4 and Table B.3.5.




*SOURCE: Hearn (2011)*

**Figure B.3.1 Features commonly observed in landslide areas**

*Conditioning* factors are those that create a potentially unstable condition, such as weak soils, adverse jointing, steep slopes, etc. *Triggering* factors are those that actually precipitate slope failure, such as a heavy rainfall or erosional event

**Table B.3.4 Main factors controlling the stability of rock slopes**

| Rock slopes | |
|---|---|
| **Conditioning factors** | **Triggering factors** |
| ▪ Slope angle and height.<br>▪ Rock structure orientation, including discontinuity patterns, in relation to topography (slope direction and angle — kinematic feasibility).<br>▪ Rock mass strength and weathering grade*.<br>▪ Presence of weak horizons within the rock mass, either more closely jointed or softer (more clayey) layers.<br>▪ Presence of rock horizons/layers of varying permeability creating perched water tables. | ▪ Toe erosion by streams and rivers removing lateral support, or vertical support it undercut.<br>▪ When degree of weathering, particularly along discontinuities, reaches a critical level (strength).<br>▪ Earthquake acceleration. leading to increased driving forces.<br>▪ Heavy and/or prolonged rainfall. Increased water pressure along discontinuities.<br>▪ External influences including excavations, fills and spoil dumps, drainage changes. |

**Weathering grade increases with time but is taken here to be a constant factor over a short engineering timescale*

*SOURCE: Hearn (2011)*

**Table B.3.5 Main factors controlling the stability of soil slopes**

| Soil slopes | |
|---|---|
| **Conditioning factors** | **Triggering factors** |
| ▪ Slope angle and height.<br>▪ Soil depth and the presence of any adversely orientated relief structures that are derived from the original rock fabric (if *in situ* weathered soil) or previous failure surfaces (if taluvium/ colluvium).<br>▪ Presence of a distinct soil layer/rock head boundary along which failure takes place.<br>▪ Soil composition and strength, a function of grain size, particle arrangement and mineralogy, density and moisture content.<br>▪ Presence of weak horizons and permanent groundwater seepages. | ▪ Prolonged/heavy rainfall leading to a rise in groundwater level and reduction in strength.<br>▪ Intense (usually short-term) rainfall leading to saturation of surface soil layers and reduction in strength.<br>▪ Toe erosion by streams and removing lateral support.<br>▪ Earthquake acceleration, leading increased driving forces.<br>▪ Deforestation, and other land use changes, can lead to increased surface water runoff, erosion and slope instability.<br>▪ External influences, including excavations, fills and spoil dumps, drainage changes. |

*SOURCE: Hearn (2011)*

### 3.5.2 Road improvement projects

Along existing roads, any effects of significant ground movement on cut and fill slopes and sometimes the pavement itself are usually fairly apparent. Slopes may have been cut too steeply for the soil or rock structure exposed and the road improvement scheme will need to take this into consideration in the redesign of earthworks. Where the new scheme requires additional slope excavation, trial pitting should be undertaken in advance to determine the soil type and drainage condition of the slope prior to developing the cut slope design. Section C.2.2 should be consulted in this regard

Fill slope failures can be common along roads constructed through hilly and mountainous terrain. These failures usually occur because either the fill slope is too steep for the materials used or the natural ground underneath has not been properly prepared, or both. The causes of any observed fill slope failure should be thoroughly investigated before proceeding with the design. Trial pitting can assist in this process as well, though access restrictions may apply for machine-excavated trial pits. Carefully excavated hand-dug trial pits might therefore be considered.

The preparation of any road project, whether it be for a newly constructed road or an improved road, should entail carefully consideration of how excavation spoil is to be managed. Spoil is the excess excavation material that cannot be used in embankment and fill slope construction. In mountainous terrain, the volumes of spoil generated on LVRs can be significant because the ground is often too steep to form stable fill slopes. Suitable spoil disposal sites should be identified during the site investigation. Table B.3.6 sets out some general guidance on spoil disposal in mountainous terrain.

Investigations for high embankments should, as a minimum, consider:

- the range of materials and settlement potential;
- side-slope stability;
- groundwater and drainage condition;
- erosion potential;
- haul distances from fill source areas; and
- potential environmental effects.

**Table B.3.6 General guidance on spoil disposal in mountainous terrain**

| |
|---|
| The following guidelines are given with respect to the location and management of safe spoil disposal sites:<br>▪ Preference should be given to large level areas, such as river terraces (away from active river channels) or flat areas formed by structural control in the underlying rock. Spoil should be spread and compacted sufficiently by tracked vehicle to minimise any potential erosion or instability.<br>▪ In the absence of large level areas, a contractor will often resort to side-casting of spoil. Where there is no choice but to side-cast material, the designated locations should be such that slope stability and environmental impacts will be minimal. In unpopulated areas these locations might include the steeper slopes protected by resistant rock. It is normally preferable to utilise a large number of small areas rather than a small number of larger ones in order to minimise the risk of slope overloading. Side-casting represents a significant negative environmental impact, and so would normally need to be balanced by appropriate offset measures.<br>▪ Spoil checks should be obligatory, in the form of gabion walls or timber structures behind which small volumes of spoil are dumped. Timber is less preferable to gabion as a local forest source is usually required and the timber will rot quickly in warm, moist climates if not properly treated.<br>▪ Wherever possible, the natural vegetation cover should be maintained and spoil should be planted with appropriate grasses and shrubs to minimise erosion.<br>Locations that should be avoided include:<br>▪ areas where the spoil could impinge on springs, streams or river channels, resulting in impeded drainage, erosion, increased sediment load and downstream environmental impacts;<br>▪ slopes where the road alignment, adjacent housing or farmland might be at risk; and<br>▪ areas where past, active or potential future slope instability or erosion could be reactivated, exacerbated or initiated. |

*SOURCE: Hearn (2011)*

Settlement problems are unlikely if rock is encountered at a shallow depth. However, if the underlying foundation is formed in transported soils, problems are likely as the material may vary from soft colluvial clays to unconsolidated and collapsing silts and sands. Compressible peaty soils and expansive clays may also be present. As referred to in Section B.3.4, subgrade investigations that combine field mapping and trial pitting should be undertaken to log soil profiles and collect samples for laboratory testing.

If soils are predominately cohesive, the primary design issues will be bearing capacity, side slope stability, and long-term settlement. These will usually require the collection of undisturbed soil samples for laboratory strength and consolidation testing. Undisturbed sampling from trial pits can prove difficult and not to industry specification, so recourse may have to be made to shallow borehole or drilling operations with the facility to collect undisturbed samples. The vane shear test can provide valuable *in situ* strength data, particularly in soft clays.

Most embankment stability problems that occur at stream crossings and low-points in the terrain are a direct result of poor drainage and consequent high pore pressures. During the site investigation it is important that all sources of water along the alignment are identified and their impact on the design assessed. If groundwater is not identified and adequately addressed, it can significantly impair constructability, road pavement performance and earthworks stability. Claims related to unforeseen groundwater conditions often form a significant proportion of contractual disputes. Many of these claims originate from a failure to record groundwater during site investigation. Hydromorphic gleyed soils indicate high groundwater tables. Mottled soil colouration may indicate a fluctuating groundwater table.

## 3.6 Investigating the causes, areal extent and depth of slope failure

### 3.6.1 Causes

Table B.3.4 and Table B.3.5 list some of factors (both natural and man-made) that may cause a slope to fail. The most common of these is that a slope becomes too steep for the materials that form it to remain stable. This can occur naturally as a result of river erosion of the toe of a slope or, more typically, through excavation for roads and other infrastructure. High groundwater levels or the saturation of the upper layers of soil during heavy rainfall are also common causes of slope failure. Rainfall can be extremely localised and can cause slopes to fail even in locations where they had previously been stable for many years. Subtle changes to land use can also affect the drainage pattern of a slope, inducing slope failure. Although annual rainfall totals in Ghana are expected to reduce over the coming decades due to climate change, there may be a tendency towards more erratic rainfall with increase rainfall intensities, which may further induce slope instability.

For a slope to fail there must be a sliding surface. This surface is commonly formed along a weak soil horizon, between intact rock and overlying weathered rock or previously transported soil or along joints in a rock mass.

### 3.6.2 Areal extent and depth

Slope failures affecting roads through hilly and mountainous terrain are typically localised and shallow in depth. They often occupy cut slopes or fill slopes, but they sometimes extend into natural hillsides above or below. Most slope failures are up to 3 m in depth, though some can be deeper than this. The areal extent of a slope failure can be determined by mapping the extent of subsidence to the road and the extent of cracking to slopes and adjacent structures.

### 3.6.3 Recommended approach to investigation

The following investigations are recommended:

1. Consult local farmers and landowners about their knowledge of any ground movements or historical landslides.
2. In rocky terrain observe the orientation of joint sets in relation to the topographic slope (orientation and angle) to identify situations where rock jointing might be adverse to stability.
3. In rocky terrain look out for any areas that are obviously too steep (evidence of rock fall) or where there are boulders and rock outcrops that appear ‘precarious’.
4. Identify areas of cracking to existing road surfaces and or sections where the road surface appears to have subsided.
5. Inspect existing structures, including side drains if constructed in masonry or concrete, and retaining walls.
6. Identify sections of road where the side drain has been made narrow by sliding material in the adjacent cut slope – in extreme cases the road carriageway itself may be of restricted width due to movement in the cut slope.
7. Look out for areas with heavy water seepage, as saturated soils could have caused slope movement.
8. Where there is evidence of movement or failure in the cut slope climb above to see if there are any tension cracks. If there are tension cracks, identify their maximum extent, by either using a GPS or instructing a surveyor to survey them.
9. Where there is evidence of failure of fill slopes or natural ground below the road, its extent also needs to be identified and surveyed.
10. Carry out a trial pitting investigation to see if the slip materials and the slip surface itself can be identified. This will assist in developing remedial measures (see Section C.7).

Table B.3.7 provides further guidance on what to look out for in the identification of landslides and potentially unstable slopes during site investigation.

**Table B.3.7 Indicators of landslides and potentially unstable slopes**

| Indicators of landslides and potentially unstable slopes | Description and comments |
|---|---|
| Tension cracks | Often orientated in an arc with vertical displacement on the downslope side |
| Slip scarps | Steps across terraces and other slopes |
| Disturbed/displaced terracing | Lines of vertically/laterally displaced terracing often mark the margins of ground movement |
| Hummocky ground | Slope surface is irregular and often formed by a series of low amplitude hummocks |
| Cracking to structures and paved surfaces | Can be due to settlement of fill and foundations, so supporting evidence is required, unless extensive |
| Dislocation of drainage structures | Either directly observed or seen as seepages |
| Springs and seepages | Creating marshy ground |

| Indicators of landslides and potentially unstable slopes | Description and comments |
|---|---|
| Trees leaning or with curved trunks | Wind, steep slopes and slope movement can cause leaning tree trunks, so careful interpretation is required |
| Spoon-shaped landforms | Steep upper scarp often semi-circular, lower-angled, possibly tongue-shaped deposit |
| Chaotic debris forming landslide deposits | Boulders often protrude above the surface |
| Immature soil profile — indicates disturbed ground | Normal weathering profile is replaced by a structureless, and usually loose taluvium soil |
| Disturbed or uncharacteristic vegetation pattern | Could be related to land use, so needs to be interpreted with care |
| Waterlogged ground and marshy areas | Water is seen to collect, either from surface water or groundwater |
| Slopes underlain by adverse geological structures and rock types prone to failure | Smooth and persistent joint surfaces often forming slopes. These joints are usually repeated at depth forming potential failure surfaces if exposed during excavation or as a result of river downcutting |
| Slopes with high groundwater tables or ground saturation in deep low-density soils | Slopes where water is seen to collect, either from groundwater or surface water |
| Slopes prone to river or stream scour at their base | Should be directly observable |
| Flow deposits border main channel indicating future events are also possible | These deposits lack stratification, are predominantly boulders in matrix and often have low amplitude levees parallel to the direction of flow |

*SOURCE: Modified from Hearn (2011)*

## 3.7 Prospecting for construction materials

### 3.7.1 Introduction

Sources of road-building materials have to be identified within an economic haulage distance. They must be available in sufficient quantity and of sufficient quality for the purposes intended. Previous experience in the area may assist with this but additional survey is usually necessary. Among the more common reasons for construction cost escalation, once construction has started and material sources fully explored, is the case where materials are found to be deficient in quality or quantity. This leads to expensive delays while new sources are investigated, or the road is redesigned to take account of the actual materials available.

The construction materials investigation often requires an extensive programme of site and laboratory testing, especially if the materials are of marginal quality or occur only in small quantities. The site investigation must identify and prove that there are adequate and economically viable reserves of natural construction materials. The materials required are:

- common embankment fill;
- capping layer / subgrade replacement material;
- sub-base and road-base aggregate;
- road surfacing aggregate;
- paving stone (e.g. for cobblestone pavements);
- aggregates for structural concrete;
- filter/drainage material; and
- special requirements (e.g. rock fill for rock fill embankments and gabion baskets, dressed stone for masonry walls and granular backfill to retaining walls).

If the project is in an area where suitable quality construction materials are scarce or unavailable, alternative solutions that make use of the local materials should be considered to avoid long and expensive haulage. For example, consideration should be given to:

- modifying the design requirements;
- modifying the material (e.g. through mechanical or chemical stabilisation, though the latter is an expensive option for LVRs);
- material processing (e.g. crushing, screening, blending); or
- innovative use of non-standard materials (particularly important for LVRs).

The materials investigations should also consider any future needs of the road. This is particularly important in the case of gravel roads where gravelling is normally needed every few years to replace material lost from the surface.

Sources of good material could be depleted with the result that haul distances and costs will increase. Furthermore, construction material may be required at a later stage in a road's operational life when the standard needs to be improved to meet increased traffic demands.

The Design Engineer will need to ascertain the availability of sufficient quantities of suitable materials in the vicinity of the road alignment. A comprehensive list of the locations of potential borrow pits and quarries is needed, along with an assessment of their proposed use and the volumes of material available. Apart from quality and quantity of material, the borrow pits and quarries must be:

- accessible and suitable for efficient and economic excavation;
- close to the site to minimize haulage costs;
- of suitable quality to enable cost-effective construction with little or no treatment; and
- located such that their exploitation will not lead to any complicated or lengthy legal problems and will not unduly affect local communities or adversely affect the environment.

Exploration of an area to establish availability of materials has the following objectives:

- Determination of the properties of the material, and whether it is fit for the intended purpose;
- Determination of the depth, thickness, extent and composition of the strata of soil and rock that are to be excavated; and
- Assessment of the drainage condition, especially with regard to the potential for excavations to become flooded by groundwater or surface water.

Records and observations of roads already built in the vicinity of the project can be a valuable source of data, not only in terms of the location of construction materials but also on their excavation, processing, placement and subsequent performance.

### 3.7.2 Fill

In general, the location and selection of fill material for LVRs poses few problems. Exceptions include organic soils and clays with high liquid limit and plasticity. Problems may also exist in lacustrine and flood plain deposits where wet, fine-grained materials predominate that are difficult to work with and problematic to compact. Where possible, fill should be taken from within the road reserve, through balanced cut-fill operations. Fill should ideally not be obtained from areas of high agricultural yield, where land acquisition costs will be high and community livelihoods may be significantly affected.

### 3.7.3 Subgrade improvement and replacement

Where the subgrade soils are weak, improvement or replacement of compressible or expansive subgrade may be necessary. As far as possible the requirement to import material, from borrow areas, should be avoided, due to the additional haulage costs. However, import of strong (CBR>9) subgrade replacement materials can be cost-effective if it avoids the need for additional thicknesses to pavement layers. Where improvement is necessary or unavoidable, mechanical and chemical stabilisation methods can be considered.

### 3.7.4 Base course and sub-base material

Naturally occurring unprocessed materials should be selected for sub-base and road base for LVR construction. However, under certain circumstances, mechanical treatments may be required to improve the quality to the required standard. This often requires the use of specialist equipment and processing plant that are relatively immobile or static and expensive to deploy. For this reason, the borrow pits for base course and sub-base materials are usually spaced widely. Distances between these pits of about 50 km are not unusual. Weathered rock and coarse-grained fluvial deposits found in river terraces are a common source of sub-base material as are some lateritic soils. Sub-base materials must meet the requirements related to maximum particle size, grading, plasticity, and CBR, or the equivalent DCP Number (DN) value.

Naturally occurring aggregates can be suitable for use as base course, though crushed rock is commonly used where suitable naturally-occurring materials are in short supply.

The minimum thickness of a deposit normally considered workable for excavation of sub-base and base course is about one metre. Thinner horizons may be exploited if there are no alternatives. The absolute minimum depends on material availability and the thickness of the overburden.

### 3.7.5 Prospecting

Prospecting for construction materials is usually undertaken by an engineering geologist utilising all desk study data sources as detailed in Section B.3.2.1. Areas of river terrace and higher ground are inspected to examine any soils and rocks exposed on the surface. Where potential material sources are identified they should be mapped and investigated using trial pits. One or two initial excavations will indicate whether the subsurface materials appear suitable, and tests can be carried out on recovered samples to confirm suitability. A grid of trial pits is then excavated to confirm the uniformity of the material, its areal extent and thickness, and the depth of overburden. The quality and durability of borrow materials and crushed rock can be greatly affected by the degree of weathering or alteration. The standard weathering classification is described in Appendix B.1, which also considers the implications for the engineering behaviour of slope. It is important to ensure that the weathering condition of these materials is properly assessed so that their suitability for use in road construction can be confirmed.

Quantities can then be calculated, and additional samples taken for confirmatory testing. Shallow boreholes or drill holes may be required to prove the thickness of deeper soils or the quality of rock at depth in potential quarry areas.

Field tests can be carried out on potential sources of construction materials to determine aggregate strength. These include the Treton Impact Value test and the Aggregate Impact Value (AIV) test. Further information can be found in Paige-Green (1988).

### 3.7.6 Laboratory Testing

The quality of the testing programme is dependent on the procedures in place to ensure that tests are conducted properly using suitable equipment that is mechanically sound and correctly calibrated. The condition of test equipment and the competence of the laboratory staff are therefore crucial. There needs to be a robust Quality Assurance (QA) procedure (overseen by a competent materials engineer) in place that will reject data that does not meet acceptable standards of reliability. There should be no compromise on the QA procedure or quality of testing data. It is important that the relationships between *in situ* conditions and laboratory conditions are considered when designing and developing the test regime. Most testing programmes will be based on relatively simple classification tests that can be done quite quickly. Such tests are listed in Table B.3.8.

Given the usual variability in natural materials, it is important to ensure that samples are taken in sufficient number and volume so that a representative set of results is obtained. The sampling and testing programme should be sufficient to determine the variability of materials within the source area.

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

Projects with significant cut, fill and aggregate quantities will require mass-haul diagrams to be drawn that show the cumulative volume of cut vs fill with distance along the centre-line. These diagrams can be used to augment cost-benefit decisions with respect to the utilization of alternative materials or treatments using crushing and screening and mechanical and chemical stabilisation techniques.

**Table B.3.8 Laboratory tests to assess the suitability of materials for pavement**

<table>
<tr><th>TEST</th><th>Subgrade & Fill</th><th>Sub-base</th><th>Road Base</th><th>Surfacing Aggregate</th><th>Wearing Course Gravel</th></tr>
<tr><td>Index Tests</td><td></td><td></td><td></td><td rowspan="4"></td><td rowspan="4">✓</td></tr>
<tr><td>▪ Atterberg limits</td><td>✓</td><td>✓</td><td>✓</td></tr>
<tr><td>▪ Linear Shrinkage</td><td></td><td></td><td></td></tr>
<tr><td>▪ Particle Size Distribution</td><td>✓</td><td>✓</td><td>✓</td></tr>
<tr><td>Compaction and Strength Tests</td><td></td><td></td><td></td><td rowspan="3"></td><td></td></tr>
<tr><td>▪ Dry Density and Optimum Moisture Content</td><td>✓</td><td>✓</td><td>✓</td><td>✓</td></tr>
<tr><td>▪ CBR and CBR Swell</td><td>✓</td><td>✓</td><td>✓</td><td>✓</td></tr>
<tr><td>Particle Strength Tests (aggregate dependent)</td><td rowspan="8"></td><td rowspan="8"></td><td></td><td></td><td rowspan="8"></td></tr>
<tr><td>▪ Durability Mill</td><td>✓</td><td>✓</td></tr>
<tr><td>▪ 10% FACT, AIV, AV, LAA</td><td>✓</td><td>✓</td></tr>
<tr><td>▪ Glycol Soak</td><td>✓</td><td>✓</td></tr>
<tr><td>▪ Water absorption</td><td>✓</td><td></td></tr>
<tr><td>▪ Specific Gravity</td><td>✓</td><td>✓</td></tr>
<tr><td>▪ Flakiness and ALD</td><td></td><td>✓</td></tr>
<tr><td>▪ Affinity with bitumen — Immersion tray test</td><td></td><td>✓</td></tr>
</table>

NOTES:

- Tests on suitability of materials for chemical stabilization are not included as chemical stabilization is not commonly adopted for LVRs.
- Selection of rock-fill is based on field assessment not laboratory testing.

# 4. SUBGRADE CHARACTERISATION AND TREATMENT

## 4.1 Introduction

Subgrades are inherently variable and reflect a country's diverse geology, topography, soil type, climate, and drainage conditions. *In situ* subgrade materials form the foundation layer of the pavement structure and a key pavement design principle is that the subgrade provides an adequate support to the upper structural pavement layers. The assessment of the subgrade condition in terms of the level of support provided is therefore one of the most important factors in determining pavement thickness design, composition and performance.

The level of subgrade support, as characterised by subgrade strength or stiffness, is dependent on the soil type, density and moisture conditions at construction and during service. The evaluation of the *in situ* subgrade strength and assigning a design subgrade support value requires careful consideration of the quantity and quality of subgrade data available to the design engineer. The variability of subgrade support within a particular project road also needs to be taken into account and it may be necessary to divide the road into different sections, each with a different uniform characteristic subgrade type and strength.

The purpose of subgrade evaluation is therefore to assess and determine the support that the subgrade will provide to the pavement during its design life. A minimum CBR value of 15% or maximum DN value of 18 mm/blow (at 95% maximum dry density according to Test Procedure GHA S1) is generally required for flexible pavements. Therefore, *in situ* subgrades with a lesser bearing capacity than 15% CBR or DN value of 18 mm/blow, require subgrade improvement (see Section B.4.4).

Changes to the vertical and horizontal alignment also need to be taken into account. For an existing road, if the horizontal alignment changes then this needs to be identified and testing locations provided accordingly. If there is a change to the vertical alignment, there are also potential actions that are required during the field testing. Further information can be found in Section 4.2.6.

## 4.2 Assessment of subgrade strength

### 4.2.1 Subgrade testing

The subgrade can be characterised on the basis of a centre-line material testing survey. This can be done using the methods provided in Table B.4.1.

**Table B.4.1 Methods of subgrade testing**

| Subgrade Testing | | | | |
|---|---|---|---|---|
| **Method** | **Cost** | **Time** | **Representation** | **Laboratory testing** |
| CBR | Relatively expensive | Time consuming | Small sample of overall materials | Sampling of materials required. Transport of samples from site to laboratory needed. Testing of materials necessary. |
| DCP | Relatively inexpensive | Rapid testing possible | Good representation of materials at regular intervals | Test performed in situ, no sampling or transport of materials required. |

The subgrade is classified using the standard soaked CBR test to provide a strength index. It is not expected that the subgrades will become soaked in service except in exceptional circumstances and so the design catalogues show different thickness designs based on climate and drainage conditions for the same indexed subgrade class. A standard soaked CBR test is also used to evaluate the strength of the imported pavement materials.

Subgrades are classified on the basis of the laboratory soaked CBR tests on samples compacted to 97% of MDD determined in accordance with Test Method GHA S1. Samples are soaked for four days or until zero swell is recorded. On this basis, the soaked CBR is used to assign a design subgrade class.

The use of the Dynamic Cone Penetrometer allows the *in situ* subgrade strength to be determined at many points along the road and thereby provides an important set of subgrade strength data for a statistically reliable design to be produced. Allowance has to be made for the likely long-term subgrade strength under the completed road. The DCP Structure Number 450 ($DSN_{450}$) is the total number of blows required for the

DCP to penetrate to 450 mm and provides a broad measure of overall strength of the pavement, somewhat analogous to the AASHTO Structural Number. The $DSN_{800}$ is typically used on high volume roads (this is the number of blows required for the DCP to penetrate to 800 mm), but the $DSN_{450}$ is more relevant to LVRs where pavement structures are not required to be as deep as for high volume roads.

Since the subgrade strength is likely to vary along the project road, it is necessary to perform DCP testing at an appropriate interval such that the variability can be assessed. This will also improve the reliability of the design as the variability in the subgrade is better understood.

It is recommended that at least two DCP tests to 800 mm depth should be carried out per kilometre of road, alternating between the outer wheel tracks in each direction for an existing road and alternating with 2 m offsets to the left and right of the centre-line for a new road after removing the upper soil layer containing humus, vegetable matter or any other undesirable materials. If the subgrade conditions appear to be highly variable, the frequency of testing should be increased, even up to one test per 250 m if necessary. There should be at least 8 results for each uniform section for statistical validity, which may require more frequent DCP tests for shorter uniform sections.

Various correlation between the DCP and the CBR value have been determined, including the following, both of which apply to 60° cones:

*Log CBR = 2.48 – 1.057 Log DN* (Overseas Road Note 8, TRL, 1990)

*Log CBR = 2.632 – 1.28 Log DN* (Kleyn and van Heerden)

These correlations are shown graphically in Figure B.4.1. Conversion from DN to CBR needs to be approached with caution and with knowledge of the type of *in situ* materials at the DCP test locations. The DCP penetration rate is influenced by:

- **Grading**. Fine grained materials generally have consistent penetration rates, but coarse materials and materials containing larger stones can have erratic penetration rates which over-estimates actual bearing capacity.
- ***In situ* moisture conditions.** Certain material types such as calcretes and other pedogenic materials can exhibit very low penetration rates when dry, but may become very soft when saturated.
- **Plasticity and clayiness.** High plasticity may increase suction and friction along the shaft of the DCP thereby slowing penetration rates and overestimating actual bearing capacity, particularly deeper into the pavement or subgrade.

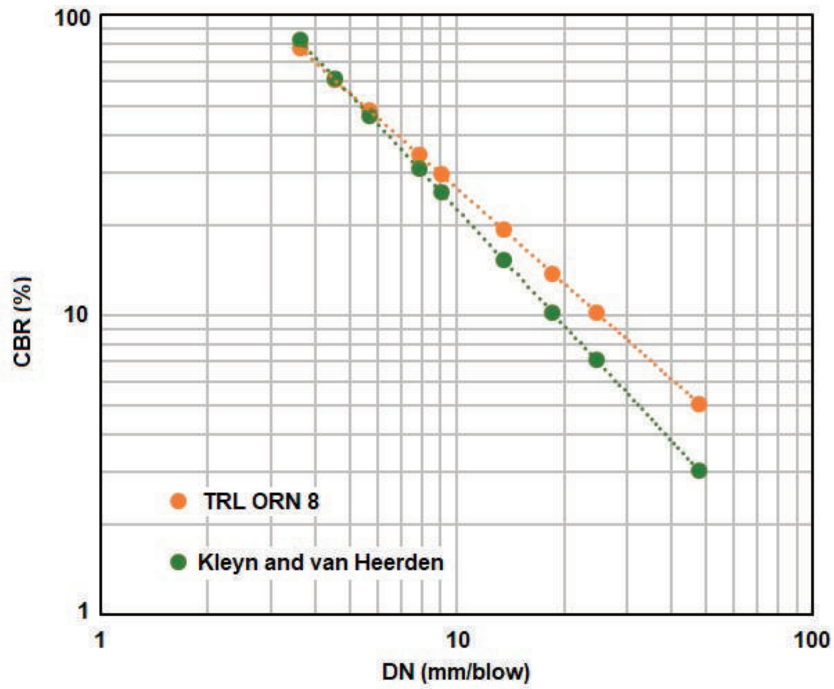



**Figure B.4.1 Correlation DN and CBR**

### 4.2.2 Delineation of subgrade uniform sections

The CBR test results and $DSN_{450}$ can be used to delineate the road into smaller uniform sections by using the cumulative sum (CuSum) method. The following formula is used to calculate the CuSum:

$$\mathrm{CuSum} = W\sum_{i=1}^{i=n} x_i - \bar{x}$$

Where $x_i$ is the current value;

$\bar{x}$ is the mean value; and

n is the total number of readings in the data set.

Table B.4.2 provides an example of a CuSum calculation. Figure B.4.2 provides a graphical representation of the CuSum calculations and also provides the basis of identifying uniform sections along the road. Significant changes to the gradient of the CuSum line indicates that the underlying properties are also changing. This suggests that a new uniform section can be introduced. The actual DCP readings also assist in determining boundaries between uniform sections. It must be kept in mind that the length of the uniform sections must be practical for analysis and construction purposes. The delineation of uniform sections is not an exact science and it is common for practitioners to differ somewhat in their interpretation of the same data set.

**Table B.4.2 Example of Cumulative Sum calculation**

| Location | $(x_i)$ $DN_{450}$ | $x_i - \bar{x}$ | CuSum* |
|---|---|---|---|
| 1 | 31 | 17,5 | 17,5 |
| 2 | 31 | 17,5 | 35 |
| 3 | 25 | 11,5 | 46,5 |
| 4 | 26 | 12,5 | 59 |
| 5 | 11 | -2,5 | 56,5 |
| 6 | 14 | 0,5 | 57 |
| 7 | 13 | -0,5 | 56,5 |
| 8 | 9 | -4,5 | 52 |
| 9 | 10 | -3,5 | 48,5 |
| 10 | 13 | -0,5 | 48 |
| 11 | 5 | -8,5 | 39,5 |
| 12 | 3 | -10,5 | 29 |
| 13 | 4 | -9,5 | 19,5 |
| 14 | 5 | -8,5 | 11 |
| 15 | 6 | -7,5 | 3,5 |
| 16 | 11 | -2,5 | 1 |
| 17 | 15 | 1,5 | 2,5 |
| 18 | 13 | -0,5 | 2 |
| 19 | 14 | 0,5 | 2,5 |
| 20 | 11 | -2,5 | 0 |
| Mean ($\bar{x}$) | 13.5 | | |

*Rounded

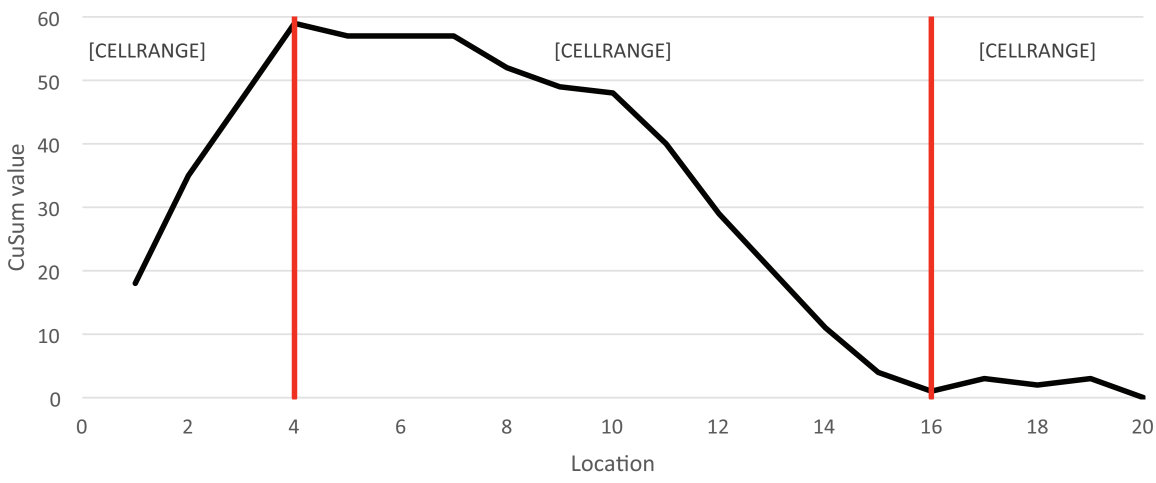



**Figure B.4.2 Example of a cumulative sum plot**

### 4.2.3 Determining the subgrade design CBR or $DSN_{450}$ value

The CBR and $DSN_{450}$ results obtained from the subgrade soils testing and DCP testing are used to determine which subgrade class should be specified for design purposes in accordance with Table B.4.3. In some cases, a variation in results may make selection unclear. In such cases it is recommended that:

firstly, the testing process is checked to ensure uniformity (to minimise inherent variation arising from, for example, inconsistent drying out of specimens); and

secondly, more tests are performed to build up a more reliable basis for selection.

**Table B.4.3 Design CBR/DN values related to Traffic Load Classification**

| Traffic Load Class | Design Reliability | Percentile |
|---|---|---|
| TLC 1.0 | 90% | 10th percentile |
| TLC 0.3 | 85% | 15th percentile |
| TLC 0.01 | 70% | 30th percentile |

Plotting these results as a cumulative distribution curve (S-curve), in which the y-axis is the percentage of samples less than a given CBR/DN value (x-axis), provides a method of determining a design CBR/DN value (Figure B.4.3). The percentiles to be used for determining the design subgrade CBR/DN value depend on the Traffic Load Class (determined as per Chapter B.2) and are indicated in Table B.4.3. For a design Traffic Load Class of TLC 0.3, the design CBR value should be the 15th percentile (i.e. the value exceeded by 85% of the CBR measurements) or if the DN value is used, the 85% percentile value (i.e. the value exceeded by 15% of the DN measurements). The cumulative distribution of the full data set of DN values as per the example presented in Table B.4.2 are shown in Figure B.4.3, but the same principles can be applied to determine the percentile value for each individual uniform section.

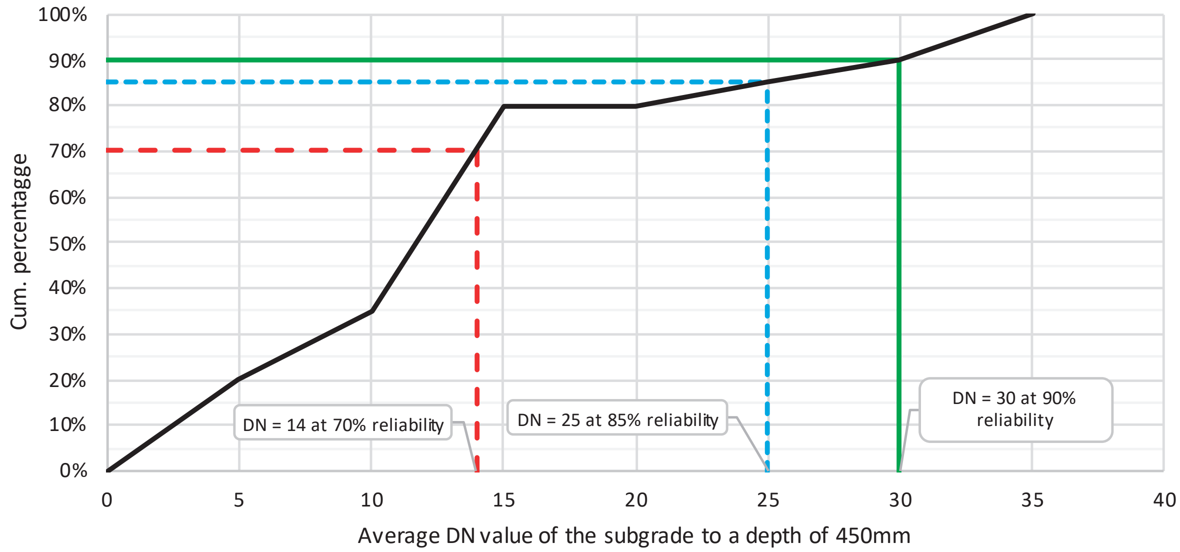



**Figure B.4.3 Example of a cumulative distribution plot**

## 4.2.4 Subgrade classification

The CBR structural design method described in Section B.6.4 of this Manual requires that the subgrade strength is categorized in subgrade classes. These classes are defined in Table B.4.4 based on CBR values and Table B.4.5 when using $DSN_{450}$ values. The CBR value for Table B.4.4 is obtained by testing a sample of the *in situ* material in the laboratory.

**Table B.4.4 Subgrade classes based on soaked CBR value**

| Subgrade Class | | | | |
|---|---|---|---|---|
| Subgrade Design Class | SC1 | SC2 | SC3 | SC4 |
| ORN 31 Subgrade strength class | S1 | S2 & S3 | S4 | S5 & S6 |
| CBR Range % | < 3 | 3 – 7 | 8 – 14 | ≥ 15 |

**Table B.4.5 Subgrade classes based on DCP-DN value**

| Subgrade Class | | | | |
|---|---|---|---|---|
| Subgrade Design Class | SC1 | SC2 | SC3 | SC4 |
| DCP-DN (mm/blow)<br>Laboratory - soaked<br>In situ (wet region) | <br>> 48<br>> 7 | <br>48 – 24.6<br>7 – 6.5 | <br>24.6 – 13.5<br>6.5 – 5.1 | <br>≤ 13.5<br>≤ 5.1 |
| $DSN_{450}$ | < 65 | 65 – 70 | 70 – 88 | > 88 |

The following should be noted:

- The subgrade classes as per Table B.4.4 and Table B.4.5 apply to the CBR design method and catalogue designs. When using the DCP-DN design method, the subgrade strength and support is assessed by means of the layer strength diagrams described in Section B.6.3 and subgrade design classes need not be assigned.
- Subgrades with a design CBR below 3% are not suitable as a foundation to the upper pavement layers due to insufficient bearing capacity. Moreover, the measurement of the bearing strength of such soft soils is generally very uncertain and CBRs below 2% are of little significance. For areas with Subgrade Design Class SC1 special treatment is required (see Section B.4.3).
- The use of Class SC2 as direct support for the pavement should be avoided as much as possible. Wherever practicable, such relatively poor soils should be excavated and replaced, or covered with an improved subgrade.

- Class SC4 covers all subgrade materials having a soaked CBR equal or greater than 15%, or i*n situ* DN value less than or equal to 5.1 mm/blow and can be used directly as sub-base when reworked and compacted provided the materials complies with the plasticity requirements for a G30 natural sub-base.

Since the combination of density and moisture content governs the CBR/DN for a given material, changes in moisture content alter the effective CBR/DN in the field. The *in situ* moisture regime must therefore be assessed when determining the subgrade strength. This moisture regime must be taken into account during the pavement design and the design subgrade strength must be adjusted to allow for very wet or very dry conditions.

Incorrect subgrade classification can have a significant effect on the performance of the pavement, particularly for poorer subgrade materials with CBR values of 5% and less. If the subgrade strength is seriously overestimated (i.e. the support is actually weaker than assumed), there is a high likelihood of local premature failures and unsatisfactory performance. Conversely, if the subgrade strength is underestimated (i.e. the support is stronger than assumed), then the pavement structure selected may be overdesigned and more expensive than necessary.

### 4.2.5 Material depth

It is of critical importance that the nominal subgrade strength is available to a reasonable depth in order that the pavement structure performs satisfactorily. The concept of "material depth" is used to denote the depth below the finished level of the road to which soil characteristics have a significant effect on pavement behaviour. In addition, the moisture regime may need to be controlled by, for example, the provision of adequate subsurface drainage and/or surface drainage. Below the material depth the strength and density of the soils are assumed to have a negligible effect on the pavement. Figure B.4.4 shows the material depth in relation to the main structural components of the road pavement.

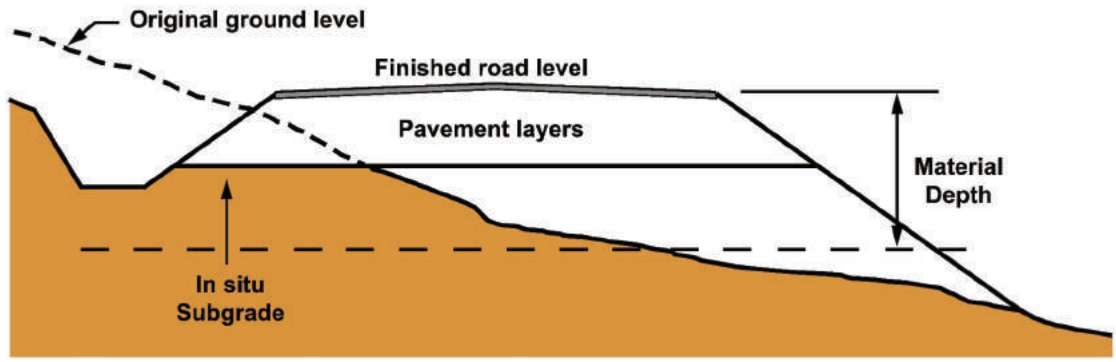



**Figure B 4.4 Material depth**

The material depth approximates to the cover required over *in situ* subgrade material less than 3% soaked CBR (i.e. Subgrade Class SC1). Table B.4.6 specifies typical material depths to be used for the different Traffic Load Classes for paved LVR's with a non-structural surfacing. It should be noted that the material depth is not necessarily the same as the thickness of the imported pavement layers. In most parts of Ghana the *in situ* subgrade material quality exceeds 3% soaked CBR and the *in situ* subgrade material can make up a significant part of the material depth. On the other hand, in certain special cases where "problem soils" occur (refer Section 4.4), the recommended material depth may still be insufficient, meaning that subgrade cover requirements need to be specifically checked.

**Table B.4.6 Recommended material depth for LVR's by Traffic Load Class**

| Traffic Load Class | Material Depth (mm) |
|---|---|
| TLC 1.0 | 700 |
| TLC 0.3 | 600 |
| TLC 0.01 | 450 |

NOTE: these material depths are based on the CBR cover design curve

The material depths indicated are not depths to which re-compaction and reworking is necessarily required. Rather, they are the depths to which the Engineer should confirm that the nominal subgrade strength is available. In general, unnecessary working of the subgrade should be avoided and limited to rolling prior to constructing overlying layers.

For the stronger subgrades, especially Class SC3 and higher, the depth check is to ensure that there is no underlying weaker material at shallow depths that could lead to detrimental performance.

It is recommended that the Dynamic Cone Penetrometer (DCP) be used during construction to monitor the uniformity of subgrade support to the recommended minimum material depths.

### 4.2.6 Upgrading roads with changes to the alignment

Improvements to the horizontal or vertical alignment of the road may result in sections of the road in cut or built on new fill. Ideally, the heights of fills and depths of cuts should be kept to a minimum to reduce costs, however other geometric and safety considerations also need to be made. These sections of cut and fill are treated as separate uniform sections in the upgrading design.

For sections of road in cut it is necessary to penetrate the DCP to a minimum depth of 450 mm below the final cut level. This provides the layer strength values for the subgrade which are required to determine the requirement for imported pavement layers (Steps 5 to 7 in Section 6.3.2). It should be noted that the DCP apparatus can be lengthened, to 2 m, by using extension rods. This will allow for investigations of deeper cuts if necessary. If the cut depth is beyond 1.5 m from the existing level, an alternative subgrade assessment method, as described in Section 3.4, must be followed.

On sections that require fill to be provided in order to raise the road level, the fill material is tested in the laboratory (see Appendix B.2) and an appropriate DN value selected depending on the expected in-service density and moisture content. The requirement for additional pavement layers of higher quality imported material is determined by following steps 5 to 7 in Section 6.3.2. During construction the DCP is used to verify the design DN value for the fill is achieved.

## 4.3 Improved subgrade layers

If the *in situ* subgrade is of class SC1 – SC3, subgrade improvement is generally required for paved roads. Only when the *in situ* subgrade is of Class SC4 can the upper pavement layers be placed directly on the *in situ* subgrade. For Subgrade Class SC4 the subgrade would only need to be prepared as roadbed, i.e. reworking to a minimum depth of 150 mm, shaping and compacting to 93% of mod AASHTO Density, provided that the requirements in terms of material depth described in Section B.4.2.5 are met. Subgrade improvement requirements for unpaved roads are less stringent and is generally only required on very poor subgrades of Class SC1.

There are many advantages to improving the strength of the *in situ* subgrade by constructing one or more improved layers. In principle, where a sufficient thickness of improved subgrade is placed, the overall subgrade bearing strength is increased to that of a higher class and the upper pavement layer thickness may be reduced accordingly. This often results in an economic advantage, as sub-base and base standard materials are generally more expensive than fill materials.

The use of improved subgrade layers also provides several other advantages, including:

- Provision of uniform subgrade strength;
- Protection of underlying earthworks;
- Improved compaction of layers above subgrade level;
- Provision of a more balanced pavement structure; and
- Provision of a running surface for the traffic during construction;

An improved subgrade placed on soils of any particular class must be made of a material of a higher standard than the underlying material. The decision to consider the use of an improved subgrade generally depends on the respective costs of sub-base and improved subgrade materials.

Table B.4.7 presents recommended actions for the different subgrade classes for paved roads only.

**Table B.4.7 Subgrade improvement actions for paved roads**

| Subgrade Class | Possible improvement actions |
|---|---|
| SC1 | ▪ Chemical / Mechanical stabilisation; or<br>▪ Remove and import new material; or<br>▪ Add additional cover to place poor quality material below material depth; or<br>▪ Use of geosynthetics (Section 4.5). |
| SC2<br>SC3<br>SC4 | The subgrade improvement action is dependent on the traffic class and in situ subgrade strength. Refer to Table B.6.3 and Table B.6.4 for details (CBR design method). |

## 4.4 Problem soils and mitigation measures

### 4.4.1 Definition

The cost of a road is integrally linked with subgrade conditions. The poorer and more problematic the conditions, the greater the thickness required to support the design loads. Sometimes certain special problems may arise in the subgrade below the material depth which requires individual treatment. Some of the common problems which need to be considered include:

- The excessive volume changes that occur in some soils as a result of moisture change (i.e. expansive soils and soils with a collapsible structure);
- The non-uniform support that results from wide variations in soil types over the road length;
- The presence of soluble salts which, under unfavourable conditions, may migrate upwards leading to several problems, including cracking of the surfacing; and
- The excessive deflection and rebound of highly resilient soils during and after the passage of a load (e.g. micaceous soils).

By virtue of their unfavourable properties, some subgrade materials fall into the category of "Problem Soils". When encountered, they would normally require special treatment before acceptance in the pavement foundation. This category of soils includes:

- Low-strength soils;
- Expansive clays;
- Collapsible sands;
- Dispersive / Erodible soils;
- Saline soils; and
- Micaceous soils.

Except for localised problem areas with expansive clays and micaceous soils, Ghana generally has stronger, better quality subgrade materials in comparison with Europe and North America.

This section focuses on typical measures that may be considered when dealing with problem soils. The investigation and testing of such soils to determine their engineering properties are dealt with in Chapter B.3.

### 4.4.2 Low-strength Soils

Soils with a soaked CBR of less than 3 % (< 2 % in dry climates) are described as low-strength soils. These soils may be extremely soft in their natural state or become extremely soft on soaking. They occur particularly in low-lying, swampy areas. They are easy to identify whether *in situ,* during site inspections or through laboratory testing of their soaked strengths. Typical treatment measures for such soils include:

- Removal and replacement with suitable material;
- Stabilisation – chemical modification with lime, or through mechanical means;
- Use of geo-synthetic products to provide mechanical stabilisation; and
- Raising of the vertical alignment to increase soil cover and thereby redefine the design depth within the pavement structure.

A method of treatment for low-strength soils needs to be established at the design stage. The appropriate measure depends on soil properties, site conditions, available equipment, available materials, experience from other sites with similar conditions and cost.

### 4.4.3 Expansive soils

Expansive soils, e.g. black cotton soils, are those which exhibit particularly large volumetric changes (swell and shrinkage) following variations in moisture contents. In the dry season they shrink and crack, becoming dusty. They are highly expansive and become very sticky when wet. In Ghana black cotton soils mainly occur in the plains of Accra, Ho and Keta. Deposits are also found in the vicinity of Wa, Grupe, Winneba and Yapei.

The mechanism of expansion illustrated in Figure B.4.5 is that of seasonal wetting and drying, with consequent movement of the water table. Soils at the edge of the road wet up and dry out at a different rate than do those under a paved surface, thus bringing about differential movement. It is this movement rather than the low soil strength (most expansive soils are often relatively strong at their equilibrium moisture content) which brings about failure. Such failure typically takes the form of associated longitudinal crack development, occurring first in the shoulder area and developing subsequently in the carriageway, as well as general unevenness of the pavement surface, arcuate cracking and settlement near trees and transverse humps and cracks at culvert sites (Figure B.4.6 and Figure B.4.7).

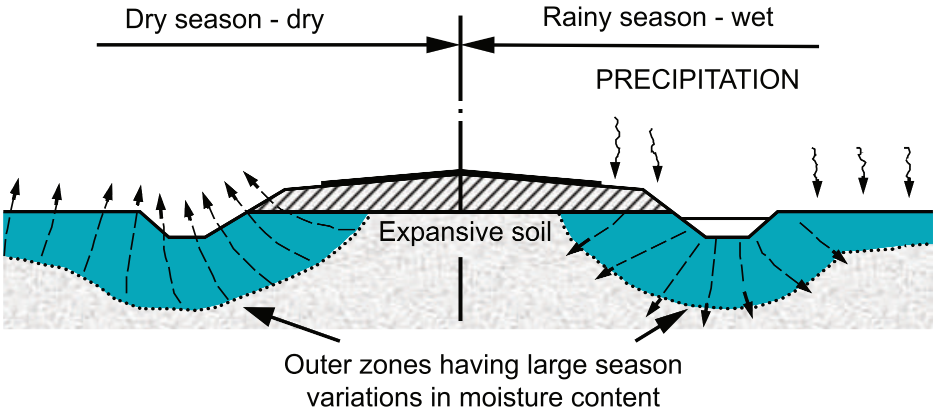



**Figure B.4.5 Moisture movements in expansive soils under a paved road**

Generally, all of the following conditions must be satisfied before significant movement occurs:

- The soil must be active; and
- The changes in moisture content must be sufficiently great; and
- The confining stresses must be sufficiently low.

When dry, some expansive soils present a sand-like texture and are prone to erosion to a much greater extent than what would be normally expected from their plasticity and clay content.

**Figure B.4.6 Expansive soil exhibiting wide shrinkage cracks**

**Figure B.4.7 Longitudinal cracking and deformation caused by expansive subgrade**

**Countermeasures for dealing with expansive soils:**

Expansive soils are often thick and laterally widespread which makes the implementation of countermeasures costly, particularly for LVRs. Measures for dealing with such soils need to strike a balance between the costs involved and the benefits to be derived over the design life of the road. Traditional countermeasures typically include one or more of the following elements:

- Placing an uncompacted pioneer layer(s) of sand, gravel or rock fill over the clay and wetting up, either naturally by precipitation or by irrigation;
- Pre-wetting (2-3 months) to induce attainment of the equilibrium moisture content before constructing the pavement;
- Partially or completely removing the expansive soil and replacing it with inert material;
- Modifying or stabilising the expansive soil with lime to change its properties;
- Increasing the height of the fill (surcharge) to suppress heave; and
- Minimizing or preventing moisture change using waterproofing membranes (Weston, 1980) and/ or vertical moisture barriers (Evans and McManus, 1999).

The scope, extent and hence cost of any set of countermeasures adopted should be appropriate for the class of road and the degree of expansiveness of the soil as defined in Chapter B.3. Table B.4.8 provides guidance on making such a choice. This is followed by a detailed description of what is meant by each of the four options. These are referred to as Countermeasures A, B, C1 and C2, each of which includes some of the elements listed above. The final choice of Countermeasure Option should be based on a life-cycle cost analysis of the options presented.

**Table B.4.8 Countermeasures for dealing with expansive soils**

| Expansiveness of soil | Alternative design and construction measures over expansive soils | |
|---|---|---|
| | Design Traffic TLC 0.01 and TLC 0.3 | Design Traffic TLC 1.0 |
| Low & Medium | Countermeasure A | Countermeasure A |
| High | Countermeasure A | Countermeasure B |
| Very high | Countermeasure B | Countermeasure C1 or C2 |

NOTE: Countermeasures C1 and C2 are normally not appropriate on unpaved roads.

**Countermeasure A**

General good construction practice for all LVRs on expansive soils adds little, if any, additional cost to construction works. Where possible:

- Remove vegetation during the dry season as long as possible in advance of construction.
- Construct any cuttings necessary, however shallow.
- Undertake construction when the *in situ* material is at equilibrium moisture content (i.e. at the end of the rainy season). If construction takes place in the dry season, the roadbed should be watered to saturation immediately prior to the placing of the backfill material.
- Extend side slopes of the embankment to 1:4 (TLC 0.3 roads) or 1:6 (TLC 1.0 roads). Utilise excavated material to flatten the side slopes of the embankment.
- Do not provide side drains unless this is necessary due to site conditions, in which case locate them 4 m from the toe of the embankment for TLC 0.3 roads or 6 m for TLC 1.0 roads as shown in Figure B.4.8.
- Seal the shoulders with an appropriate impermeable surfacing. (For low expansive soils, sealing shoulders is uneconomical and not recommended as a countermeasure for both traffic groups; sealing of shoulders does not apply to unpaved roads).
- Remove/do not plant trees within a distance of 1.5 times their mature height from the edge of the shoulder.

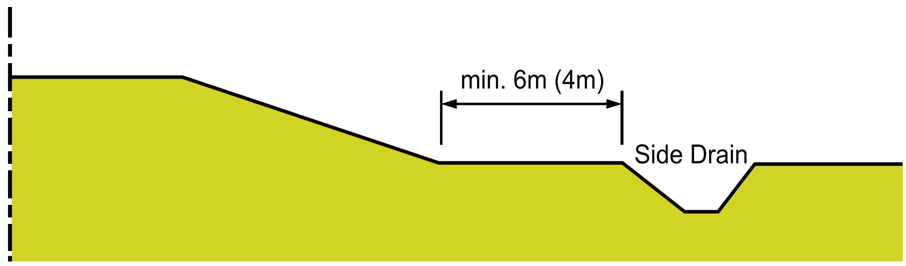



**Figure B.4.8 Location of side drains in expansive soils**

**Countermeasure B (Pioneer layer)**

These measures are recommended for the higher class of TLC 1.0 roads. They typically entail the use of a 'pioneer" layer (Figure B.4.9) as follows:

- Adopt countermeasures listed for Countermeasure A.
- Place a loose layer "pioneer" layer (about 100-200 mm in thickness) of permeable sand, gravel or rock fill over the clay to cover the full width of construction. It is essential that this layer should remain loose and permeable and must therefore not be compacted or trafficked.
- Allow the "pioneer" layer to stand through one full rainy season in order to pre-wet the roadbed as much as possible through the elimination of evapotranspiration, and the collection of rainwater. Prevent localised ponding of water.
- Compact the "pioneer" layer in advance of construction during the following dry season and utilize it as the first layer of fill.
- Ensure minimum earthworks cover of 0.6 m.

- Do not use active clay as fill.
- Replace clay under culverts to a depth equivalent to the reduction of surcharge caused by the culvert.
- Waterproof culvert joints.
- Prevent ponding of water at culvert inlets and outfalls and adjacent to road.

If it is not possible to apply the "pioneer" layer technique, the vegetation should be removed as far in advance of construction as is feasible. If the roadbed is to stand open during the rainy season, it will be advantageous to plough or scarify it to a depth of about 150 mm to promote the collection and ingress of rainwater.

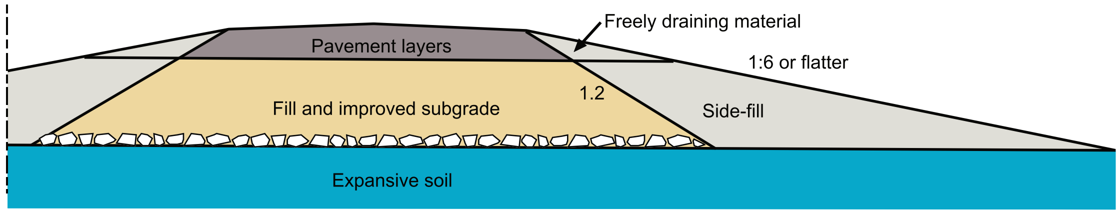



**Figure B.4.9 Construction on expansive soils (use of Pioneer layer)**

**Countermeasure C1 (embankment height < 2 m)**

These measures are recommended for TLC 1.0 roads on soils with very high expansiveness and embankment height < 2 m. They typically entail partial excavation of the road bed as follows:

- Adopt countermeasures listed for Measure A.
- Excavate expansive soil over width to toe 1:2 side slope and to depth of 0.6 m.
- Stockpile excavated material at sides for eventual grading on to fill side slopes.
- Backfill excavation with non-expansive fill. Ensure minimum earthworks cover of 0.6 m.
- Fill above ground level to be constructed with 1:2 side slopes.
- Grade and spread excavated expansive soil on fill side slopes to lengthen their slope to 1:6 or flatter, thereby extending the distance of the road over which transpiration will be reduced.
- Expansive material must not be used for the shoulder slope to the pavement – these must be constructed as wedges of permeable material as shown in Figure B.4.10.

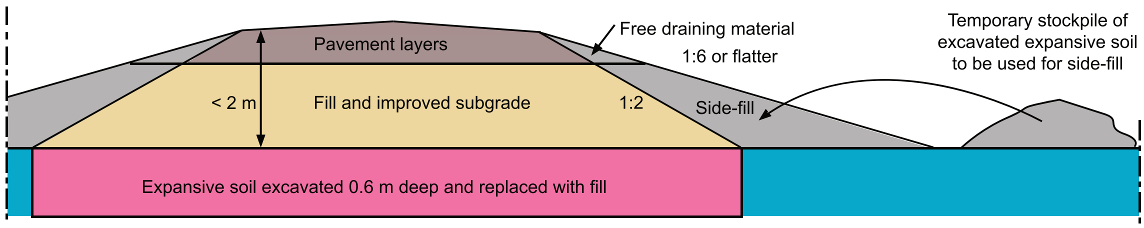



**Figure B.4.10 Construction on expansive soils (embankment height < 2m)**

**Countermeasure C2 (embankment height ≥ 2 m)**

These measures are recommended for TLC 1.0 roads on soils with very high expansiveness and embankment height ≥ 2 m. They typically entail partial excavation of the road bed. The same countermeasures as for Measure C1 are adopted except for the following:

- Excavate expansive soil only under the width of the 1:2 (or 1:1.5) side slopes (Figure B.4.11)

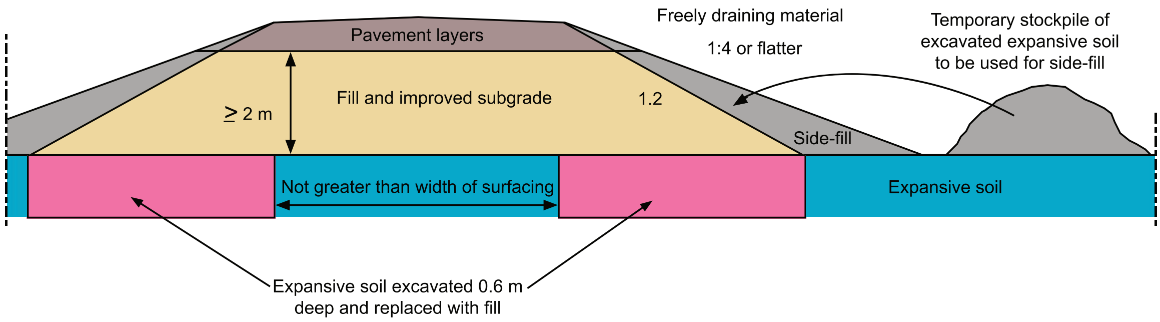



**Figure B.4.11 Construction on expansive soils (embankment height > 2 m)**

The other countermeasures mentioned above, including stabilisation, surcharging and use of waterproofing membranes would normally be ruled out on cost grounds for LVRs.

### 4.4.4 Collapsible soils

Collapsible soils exhibit a weakly cemented soil fabric which, under certain circumstances, may be induced to rapid settlement. A characteristic of these soils is that they are all unsaturated, and generally have a low dry density and a low clay content. At the *in situ* moisture content they can withstand relatively large imposed stresses, well in excess of the overburden pressure, with little or no settlement. However, without any change in the applied stress, but an increase in moisture content, additional settlement will occur as shown in Figure B.4.12 and Figure B.4.13. The rate of settlement will depend on the permeability of soil.

**Countermeasures for dealing with collapsible soils**

Methods for dealing with collapsible soils depend on the degree of collapse potential as described Chapter B.3. The countermeasures include:

- excavation of material to the specified depth below ground level; break down collapsible structure; replace in the excavation; and re-compact with conventional rollers in lifts typically not exceeding 250 mm;
- ripping of the road bed, inundation with water and compaction with heavy vibrating rollers; and
- use of high-energy impact compactors from the surface of the subgrade, with or without the use of water.

The risk of collapse occurring on LVRs, particularly in arid or semi-arid areas, is small. Thus, other than in exceptional circumstances, the above measures are unlikely to be economically justified for application to LVRs.

**Figure B.4.12 Collapse settlement in excess of 150 mm following impact compaction**

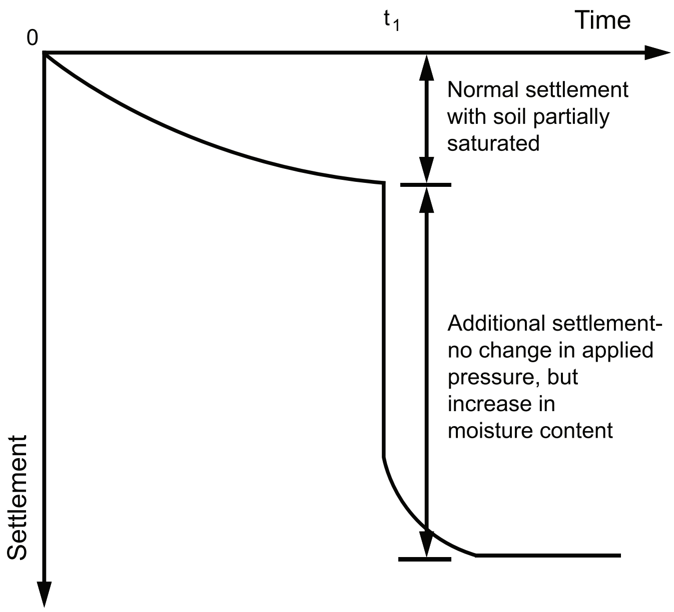



**Figure B.4.13 Manner of additional settlement due to collapse of soil fabric**

## 4.4.5 Dispersive/erodible soils

Although dispersive and erodible soils (see Figure B.4.14) are similar in their field appearance (highly eroded, gullied and channelled exposures), they differ significantly in the mechanisms of their actions and are differentiated as follows:

- Dispersive soils are those soils that, when placed in water, have repulsive forces between the clay particles that exceed the attractive forces. This results in the colloidal fraction going into suspension and in still water staying in suspension. In moving water, the dispersed particles are carried away.
- Erodible soils are those soils in which the cohesion (or surface shear strength when wet) is insufficient to resist the tractive forces of rain or runoff water flowing over them. Such soils tend to lose material as a result of flowing water over the material exceeding the cohesive forces holding the material together.

It is not normally important, or even easily possible, to quantify the actual potential loss of dispersive/ erodible material as the process is time-related and given enough time, all of the colloidal material could theoretically be dispersed and removed, leading to eventual loss of material on a large scale. However, it is important to identify the presence of dispersive/erodible soils so that necessary precautions can be taken if they affect the constructed pavement.

**Figure B.4.14 Example of severe erosion in erodible/dispersive soils**

**Countermeasures for dealing with erodible and dispersive soils**

Methods for dealing with **dispersive soils** include (Paige-Green, 2008):

- Avoid the use of such soils in fills and remove and replace it in the subgrade;
- Manage water flows and drainage in the area;
- Treat the soil with lime or gypsum to allow the calcium ions to replace the exchangeable sodium cations; and
- Compact the soil at 2 to 3% above optimum moisture content to as high a density as possible (Elges, 1985).

Methods for dealing with **erodible soils** include (Paige-Green, 2008):

- Ensure that the drainage in the area is well controlled;
- Cover the soils with non-erodible materials and vegetation; and
- Once erosion has occurred, back-fill the channels and gullies with less erodible material and redirect the water flows.

## 4.4.6 Saline soils

Saline soils occur mostly in the arid or semi-arid regions where the dry climate combined with the presence of saline materials and/or saline ground or surface water, create conditions that are conducive to the occurrence of salt damage. The presence of soluble salts in the subgrade or pavement materials can cause damage to the bituminous surfacings of roads. Such damage occurs when the dissolved salts migrate to the road surface, mainly due to evaporation, become supersaturated and then crystallize with associated volume change. This creates pressures which can lift and physically degrade the bituminous surfacing and break the adhesion with the underlying pavement layer as illustrated in Figure B.4.15 and Figure B.4.16. Generally, the thinner the surfacing layer is, the more likely the damage, primes being the most susceptible and thick, impermeable seals the least susceptible.

**Figure B.4.15 Severe distress to a road surfacing caused by salt attack**

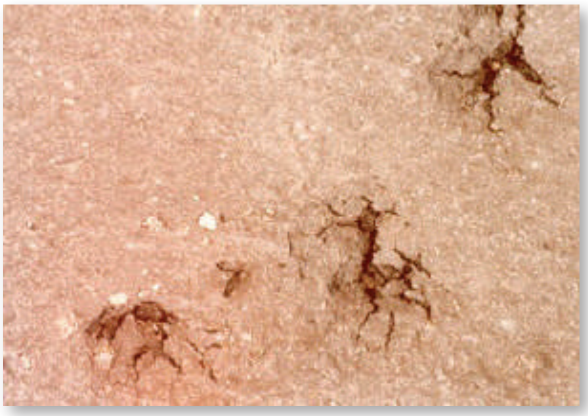


**Figure B.4.16 'Blistering', 'Heaving' and 'Fluffing' salt damage of wearing course**

Soluble salt problems arise from the accumulation and crystallization of the salts under the road surfacing and the upper base layer. The minimization of salts in the pavement layers can be achieved with the use of impermeable surfacings. Such surfacings prevent water vapour passing through it, and, as a result, crystallization will not occur beneath them (Netterberg, 1979). Construction should then proceed as fast as possible to minimize the migration of salts through the layers. The addition of lime to increase the pH to in excess of 10 will also suppress the solubility of the more soluble salts.

Guidelines for the prevention and repair of salt damage to roads and runways have been developed based on research work carried out in the southern African region (Botswana Roads Department, 2001). These guidelines provide guidance on methods of testing and measurement of salts as well as repair methods where damage has already occurred.

### 4.4.7 Micaceous soils

Micaceous soils contain large quantities of mica (muscovite) and occur in such materials as weathered granite, gneiss, mica schist and phylite materials. Micaceous soils occur in approximately 25% of the area of Ghana, while decomposed phylilites are encountered in the Western, Brong Ahafo, Central, Eastern and Ashanti Regions (DFR, Soils and Natural Gravels – A Guide for Area Engineers). These soils often cause problems with compaction because of the "spring action" of the muscovite materials which may prevent achievement of the intended density or, even if it is achieved initially, can cause rutting in the compacted layer at a later stage.

**Countermeasures for dealing with micaceous soils**

Methods for dealing with micaceous soils include:

- removing the micaceous soil layer to below the material depth in the subgrade; and
- stabilising the micaceous soil with lime or cement.

For LVRs, the loss of shape associated with micaceous generally must be accepted, unless the overlying pavement warrants the expense of the countermeasures indicated above.

## 4.5 Use of geosynthetics in subgrade strengthening

The primary uses of a geosynthetics in a pavement system are to serve as a construction aid over soft subgrades, improve or extend the estimated service life of the pavement, and reduce the thickness of the structural cross section for a given design period. These objectives are achieved through four functions (separation, reinforcement, filtration (drainage), and containment) as shown in Figure B.4.17.

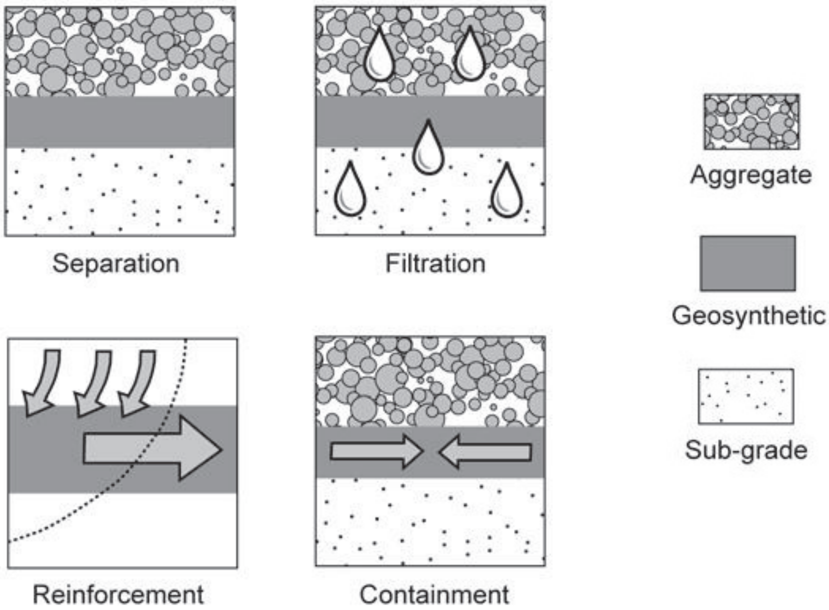



**Figure B.4.17 Main functions of geosynthetics in pavement systems**

Geotextile and geogrid materials are the most commonly used geosynthetics in pavement design. This is especially true when only the pavement itself is considered without fills and cut slopes, abutments, or drainage facilities. Stabilization using these materials is achieved through a combination of separation, filtration, and reinforcement. The separation function of a geotextile prevents the subgrade and the sub-base from intermixing (see Figure B.4.18), which might occur during construction and later in-service due to pumping of the subgrade by traffic loads. The filtration function is required because soils requiring stabilization are usually wet. By acting as a filter, the geotextile retains the subgrade without clogging, while allowing water from the subgrade to pass up into the sub-base, permitting the pore pressure to dissipate, and promoting strength gain due to consolidation.

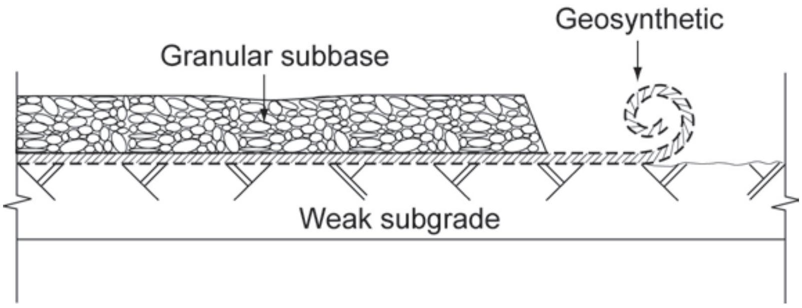



**Figure B.4.18 Use of geosynthetic to separate a sub-base from the subgrade**

Geosynthetics also provide some level of reinforcement by laterally restraining the base or sub-base and improving the bearing capacity, thus decreasing shear stresses on the subgrade. Soft subgrade soils provide very little lateral restraint (containment). Hence, when the granular material moves laterally, ruts develop on the surface and also in the subgrade. A geogrid with good interlocking capabilities or a geotextile with high frictional capacities can provide tensile resistance to such lateral aggregate movement. Geosynthetics also increase the system bearing capacity by forcing the potential bearing surface under the wheel load to develop along alternate, longer mobilization paths and, thus, higher shear strength surfaces.

Geotextiles serve best as separators, filters and, in the case of non-woven geotextiles, drainage layers, while geogrids are better at reinforcing. Geogrids, as with geotextiles, prevent the sub-base from penetrating the subgrade. When geogrids are used, either the sub-base must be designed as a separator or a geotextile must be used in conjunction with the geogrid, either separately or as a geo-composite material.

# 5. CONSTRUCTION MATERIALS

## 5.1 Introduction

Maximising the use of naturally occurring unprocessed materials is a central pillar of the LVR design philosophy. Traditional specifications do not include for or are silent on the use of many naturally occurring, unprocessed materials (natural soils, gravel-soil mixtures and gravels) in pavement layers and instead tend to favour more expensive imported gravel and crushed rock. However, research has demonstrated quite clearly that so-called “non-standard” materials[1] can often be used successfully and cost-effectively in LVR pavements provided appropriate precautions are observed as described in this Chapter.

The adoption of this approach provides the scope to consider a modification to the standard specification when considering particular material types within defined environments. Recognising whether a non-standard material is “fit for purpose” is central to assessing its appropriate use. However, the use of such materials requires a sound knowledge of their properties and behaviour in the prevailing environment.

Ghana’s diverse geology provides a variety of igneous, sedimentary and metamorphic rocks as well as transported and residual soils, many of which are potentially suitable for use as road construction materials. The selection of these materials for incorporation in a road pavement is generally based on a combination of such factors as availability, structural requirements, environmental considerations, method of construction, economics, previous experience and quality. These factors need to be evaluated during the pavement design process in order to select the materials that are most suitable for the prevailing conditions. An assessment of the durability of these materials is a key factor in the selection process.

## 5.2 Properties and characteristics of local materials

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

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

The relative dependence of a material, and the influence of moisture, on each of the above components of shear strength significantly influences how they can be incorporated within a pavement. Table B.5.1 summarises the typical, relative characteristics of unbound and bound materials that critically affect the way in which they can be incorporated into a pavement in relation to their properties and the prevailing conditions of traffic, climate, economics and risk.

[1] PIARC has defined non-standard and non-traditional materials as: *“...any material not wholly in accordance with the specification in use in a country or region for normal road materials but which can be used successfully either in special conditions, made possible because of climatic characteristics or recent progress in road techniques or after having been subject to a particular treatment.”* (Brunschwig, 1989).

**Table B.5.1 Pavement material categories and relative characteristics**

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

Denotes categories of particular significance for LVRs

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

- **Category 1:** These materials are highly dependent on soil suction and cohesive forces for development of shear resistance. The typical deficiency in hard, durable particles prevents reliance on inter-particle friction. Thus, even modest levels of moisture, typically approaching 60% saturation, may be enough to reduce confining forces sufficiently to cause distress and failure. Consequently, special measures must be taken to ensure that moisture ingress into the pavement is prevented. Otherwise, as shown in Figure B.5.1, suction forces and shear strength will be reduced, which could result in failures.
- **Category 2:** These materials have a moderate dependency on all forms of shear resistance (i.e. friction, suction forces and cohesion). They also have rather limited strength potential and therefore modest levels of moisture, typically 60-80% of saturation, may be sufficient to reduce the strength contribution from suction or cohesion, leading to premature distress and failure. This occurs at moisture contents lower than those necessary to generate pore pressures.
- **Category 3:** These materials have only minor dependency on suction and cohesion forces but have a much greater reliance on internal friction which is maximised when the aggregate is hard, durable and well graded. Very high levels of saturation, typically 80-100% are necessary to cause distress and this will usually result from pore pressure effects.
- **Category 4:** These materials rely principally on physio-chemical forces which are not directly affected by water. However, the presence of water can lead to distress under repetitive load conditions through layer separation, erosion, pumping and breakdown.

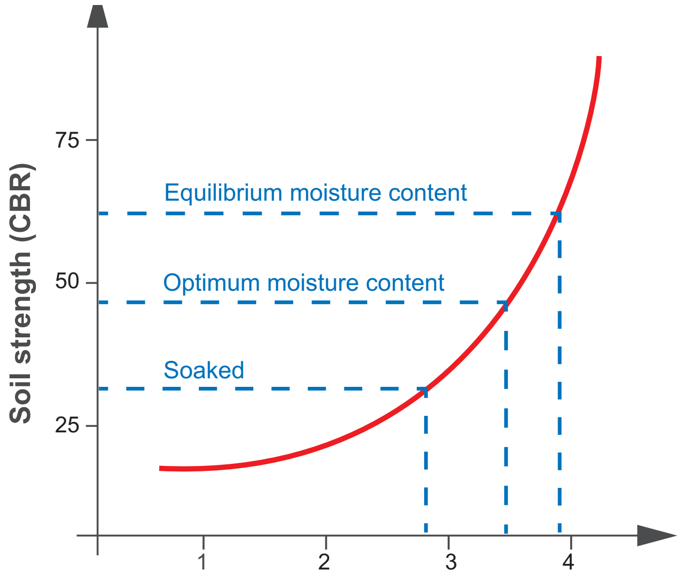



**Figure B.5.1 Illustrative soil strength/suction relationship**

The management of moisture during the construction and operational phases of a pavement affects its eventual performance, especially when unbound, unprocessed, often relatively plastic, materials, are used. Table B.5.2 shows the variation of strength, as measured by the laboratory CBR test, with moisture content expressed as the ratio of the *in situ* (or Field) Moisture Content (FMC) and the Optimum Moisture Content (OMC) for compaction (Emery, 1985). The Table illustrates the significant effect of drying out of materials of varying quality in terms of an increase in their strength (CBR) and, conversely, a loss in strength in their wetting up from a dry state.

As indicated, if such materials can be maintained in a relatively dry state in service, then they can be expected to perform satisfactorily at this "elevated" strength provided appropriate precautions are taken to avoid their wetting up.

**Table B.5.2 Variation of CBR with moisture content**

| Material Class | Laboratory Soaked CBR (%) | Laboratory Unsoaked CBR (%) at Varying FMC/OMC Ratios | | |
|---|---|---|---|---|
| | | 1.0 | 0.75 | 0.50 |
| G80 | 80 | 105 | 150 | 200 |
| G65 | 65 | 95 | 135 | 185 |
| G45 | 45 | 80 | 115 | 165 |
| G30 | 30 | 65 | 95 | 140 |
| G15 | 15 | 45 | 70 | 110 |
| G7 | 7 | 30 | 50 | 85 |

The nomenclature of the material classes presented in Table B.5.2 has the following meaning:

- The "G" refers to granular or gravel natural material. The term "natural material" includes., but is not limited to, lateritic gravel, quartzitic gravel, calcareous gravel, soft stone, conglomerate, sand or clayey sand or a combination of any of these materials. A natural material that can be processed using bulldozers and shovels without the need for blasting or crushing plant is also referred to as "gravel".
- The number behind the "G" indicates soaked laboratory CBR value of the material.

## 5.3 Construction material requirements

### 5.3.1 General

The different types of road construction materials required are:

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

Some of these materials require extensive processing so are costly. The road design should therefore be carefully planned to minimise use of the more expensive materials.

It should also be noted that most road designs prepared using this Manual relate to LVRs being upgraded from an existing earth or gravel road, which may have been in service for many years. The strength built up in the underlying material must be capitalised on and as little additional structure as possible should be constructed (refer to Chapter B.6).

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

### 5.3.2 Common embankment fill

In general, the identification of suitable fill materials for LVRs poses few problems. Materials require a minimum CBR value at their expected worst *in situ* moisture and density condition of 3%, equivalent to maximum DCP DN values of 48 mm/blow. The CBR swell should not exceed 1.5%. The nominal maximum aggregate size should be limited to two-thirds of the layer thickness. Unsuitable materials include organic soils and clays with high liquid limits and plasticity. Problems may also exist in lacustrine and flood plain deposits where very fine materials are abundant.

Where possible, fill should be taken from within the road alignment through balanced cut-fill operations, or by excavation of the side drains (except in areas of expansive soils). Borrow pits to provide fill materials should be avoided as far as possible and special consideration should be given to the undesirable impacts of winning fill from agriculturally productive land.

It is unusual to construct high fills for LVRs, other than in the case of approaches for bridges. In most cases the need is limited to having sufficient fill material to raise the pavement above the natural ground level to allow the placement of small cross drainage structures. Unless embankments cross naturally weak areas (swampy or black cotton soils), it is not generally necessary to raise them much higher than about 1 metre, and a low quality material is usually adequate. In areas of weak soils, it may be necessary to design lengths of fill such that they are drained (rock fill layers at their base, with or without geosynthetic layers) but the material quality above this does not need to be any higher than for the low fills described previously.

### 5.3.3 Imported (selected) subgrade

Where *in situ* and alignment soils are weak or problematic, it may be necessary to import higher quality selected subgrade material. As far the use of borrow areas for this purpose should be avoided due to the additional haulage costs. However, import of stronger (CBR > 15% or DN of ≤ 14 mm/blow, at expected worst moisture condition) subgrade materials can provide economies with regard to the pavement thickness design, although the cross section of the pavement must still allow for effective drainage. Where material improvement is necessary or unavoidable, mechanical and chemical stabilisation methods can be considered. Information about mechanical stabilisation is provided in Section B. 5.5.3.

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

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

Materials requirements for use in the top 300 mm below formation level are provided in Table B.5.3.

**Table B.5.3 Requirements for natural materials for selected subgrade**

| Material properties | Material Class | |
|---|---|---|
| | G15 | G7 |
| DCP-DN value (mm/blow) (max) | *13.5* | *24.6* |
| CBR (%)<br>CBR Swell (%) | 15<br>1.5 | 7<br>2.0 |
| Grading:<br>Maximum size (mm) | 2/3 layer thickness | 2/3 of layer thickness |
| Atterberg Limits<br>Plasticity Index PI (%) (max) | 25 | 30 |

*SOURCE: Modified from MoT (2007)*

NOTES:
- DCP-DN value to be determined on soaked samples at 93% of Maximum Dry Density
- Soaked CBR's to be determined at 93% of Maximum Dry Density
- All Atterberg limits will be determined using Test Procedure GHA6

## 5.3.4 Natural material sub-base

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

A wide range of local materials can be used successfully as base. These include lateritic, calcareous and quartzitic gravels, river gravels and other transported gravels, or granular residual materials resulting from weathering of rocks. Natural gravel sub-base materials are conventionally expected to meet requirements related to maximum particle size, grading, plasticity, and CBR as detailed in Table B.5.4. However, the DN value obtained through testing of samples with the DCP in the laboratory provides a composite measure of the adequacy of the material for use in the pavement layer and is increasingly replacing the conventional testing parameters including the CBR test for LVR design. Details of the DCP-DN approach to materials selection and the laboratory DCP-DN test are provided in Section B.5.4 and Appendix B.2.

**Table B.5.4 Requirements for natural gravel materials for sub-base**

| Material properties | Material Class | |
|---|---|---|
| | G40 | G30 |
| DCP-DN value (mm/blow) (max) | 6.8 | 7.8 |
| CBR (%)<br>CBR Swell (%) | 40<br>0.5 | 30<br>1.0 |
| Grading:<br>Grading Modulus (max)<br>Grading Modulus (min)<br>Maximum size (mm) | <br>2.5<br>1.5<br>75.0 | <br>2.6<br>1.25<br>2/3 of layer thickness |
| Atterberg Limits<br>Liquid Limit LL (%) (max)<br>Plasticity Index PI (%) (max)<br>Linear Shrinkage LS (%) (max)<br>Plasticity Modulus PM (max) | <br>30<br>14<br>7<br>250 | <br>35<br>16<br>8<br>250 |

*SOURCE: Modified from MoT (2007)*

NOTES:
- DCP DN value to be determined on soaked samples at 95% of Maximum Dry Density
- All CBR's to be determined at field density specified for the layer in which the material is used, usually 95% of Maximum Dry Density.
- All Atterberg limits will be determined using Test Procedure GHA6
- All grading specifications are applicable after placing and compaction
- Grading Modulus: $GM = \frac{300 - (P_{2.0} + P_{0.425} + P_{0.075})}{100}$

  with $P_{2.0}$ , $P_{0.425}$ and $P_{0.075}$ the percentage passing the 2.0 mm, 0.425 mm and 0.075 mm sieves respectively.
- Plasticity Modulus: $PM = PI \times P_{0.425}$

Mechanical treatments may in some circumstances be required to improve the quality of sub-base material to the required standard. Guidance on Material Improvement is provided in Section B.5.5.

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

### 5.3.5 Natural material base

A wide range of local materials can also be used successfully as base course. Natural gravel base materials are conventionally expected to meet requirements related to maximum particle size, grading, plasticity, and CBR as detailed in Table B.5.5. The DN value obtained through testing of samples with the DCP in the laboratory provides a composite measure of the adequacy of the material for use in the pavement layer and is increasingly replacing the conventional testing parameters (see Section B.5.4 and Appendix B.2).

For LVR's the use of crushed rock base material is normally not affordable and so is not recommended. The limited traffic loading on low volumes roads compared to high value roads does not justify the use of high-quality crushed stone bases. Selected natural gravel materials perform well as road base material when sealed and with good provision for drainage.

Sand has been used successfully as a road base and sub-base material in South Africa, Mozambique and Botswana. Detailed information regarding the use of sands in road construction can be found in the "Guide on the Use of Sand in Road Construction in the SADC Region" (InfraAfrica, 2014).

**Table B.5.5 Requirements for natural gravel materials for base**

| Material properties | Material Class | |
|---|---|---|
| | G80 | G60 |
| DCP-DN value (mm/blow) (max) | 3.62 | 4.54 |
| CBR (%) | 80 | 60 |
| CBR Swell (%) | 0.25 | 0.5 |
| Grading: | | |
| %Passing Sieve Size (mm) | | |
| 75 | 100 | 100 |
| 37.5 | 80 – 100 | 80 – 100 |
| 20 | 60 – 85 | 75 – 100 |
| 10 | 45 – 70 | 45 – 90 |
| 5.0 | 30 – 55 | 30 – 75 |
| 2.0 | 20 – 45 | 20 – 50 |
| 0.425 | 8 – 26 | 8 – 33 |
| 0.075 | 5 – 15 | 5 – 22 |
| Grading Modulus (min) | 2.15 | 1.95 |
| Maximum size (mm) | 53.0 | 63.0 |
| Atterberg Limits | | |
| Liquid Limit LL (%) (max) | 20 | 30 |
| Plasticity Index PI (%) (max) | 10 | 12 |
| Linear Shrinkage LS (%) (max) | 5 | 6 |
| Plasticity Modulus PM (max) | 200 | 250 |

*SOURCE: Modified from MoT (2007)*

NOTES:
- All CBR's or DCP-DN values to be determined at field density specified for the layer in which the material is used, usually at 98% of Maximum Dry Density.
- All Atterberg limits will be determined using Test Procedure GHA6
- All grading specifications are applicable after placing and compaction. Grading curves shall be smooth curves within the specified envelopes and approximately parallel to the envelopes.
- Grading Modulus: $GM = \frac{300 - (P_{2.0} + P_{0.425} + P_{0.075})}{100}$

  with $P_{2.0}$ , $P_{0.425}$ and $P_{0.075}$ the percentage passing the 2.0 mm, 0.425 mm and 0.075 mm sieves respectively.
- Plasticity Modulus: $PM = PI \times P_{0.425}$
- When lateritic material is used for the base, the maximum Plasticity Index is relaxed to 9 for traffic load classes TLC 1.0 and TLC 0.3 and to 12 for traffic load class TLC 0.01.

### 5.3.6 Gravel wearing course material

The wearing course material should be durable and of consistent quality to ensure it wears evenly. The desirable characteristics of such a material are:

- good skid resistance;
- smooth riding characteristics;
- cohesive properties;
- resistance to ravelling and scouring;
- wet and dry stability;
- low permeability; and
- load spreading ability.

For ease of construction and maintenance, a wearing course material should also be easy to grade and compact. The material properties having the greatest influence on these characteristics are the particle size distribution and the properties of the coarse particles.

**Performance-related specifications**: Performance related specifications for wearing course materials have been developed for southern Africa based on extensive sampling, testing and monitoring of a large number of test sections (Paige-Green, 1989). These specifications have been successfully implemented in a number of other African countries, including Ghana, and are appropriate for the Ghanaian environment. The specifications identify the most suitable materials in terms of two basic soil parameters, the Shrinkage Product and Grading Coefficient. These are determined from particle size distribution and linear shrinkage tests as shown in Table B.5.6 and Figure B.5.2.

**Table B.5.6 Recommended gravel wearing course material specifications**

| Material properties | Wearing Course Material Class | |
|---|---|---|
| | Type I | Type II |
| Maximum size (mm) | 37.5 | 37.5 |
| Oversize index ($I_o$)[a] | ≤ 5 % | 0 % |
| Shrinkage product (SP)[b (2)] | 100 - 365 | 100 - 240 |
| Grading coefficient (GC)[c (2)] | 16 - 34 | 16 - 34 |
| Soaked DCP-DN | ≤ 13.5 mm/blow | ≤ 13.5 mm/blow |
| Treton Impact Value (%) [(4)] | 20 – 65 | 20 – 65 |
| a. $I_o$ = Oversize index (percent retained on 37.5 mm sieve)<br>b. SP = (Linear shrinkage) x (Percent passing 0.425 mm sieve)<br>c. GC = (Percentage passing 26.5 mm - percentage passing 2.0 mm) x (percentage passing 4.75 mm)/100 | | |

NOTES:

1. Specifications are applicable after placement and compaction.
2. The Grading Coefficient and Shrinkage Product must be based on a conventional particle size distribution determination which must be normalised for 100% passing the 37.5 mm screen.
3. Only representative material samples are to be tested.
4. The Treton Impact Value (TIV) limits exclude those materials that are too hard to be broken with a grid roller (TIV < 20%) or too soft to resist excessive crushing under traffic (TIV > 65%).

Type II gravel wearing course material is recommended for gravel roads in built-up areas where there are a significant number of dwellings and local businesses. In comparison with the limits for Type I gravel wearing course material, the limits for the oversize index have been reduced to eliminate stones which compromise riding quality, whilst the shrinkage product has been reduced to a maximum of 240 to reduce the dust as far as practically possible. This lower SP limit reduces the probability of having unacceptable dust from about 70% to 40%. To reduce unacceptable dust further, a treated or sealed road surface should be considered.

Relaxation of the Type I specifications for the Shrinkage Product to a wider range of 50 – 450 and the Grading Coefficient to 15 – 45 is acceptable under certain circumstances. These wearing course materials are however more prone to ravelling and accelerated gravel loss. This needs to be accounted for when designing the gravel wearing course thickness. In general, a thicker wearing course layer of 200 mm is recommended for materials utilised at the relaxed specification.

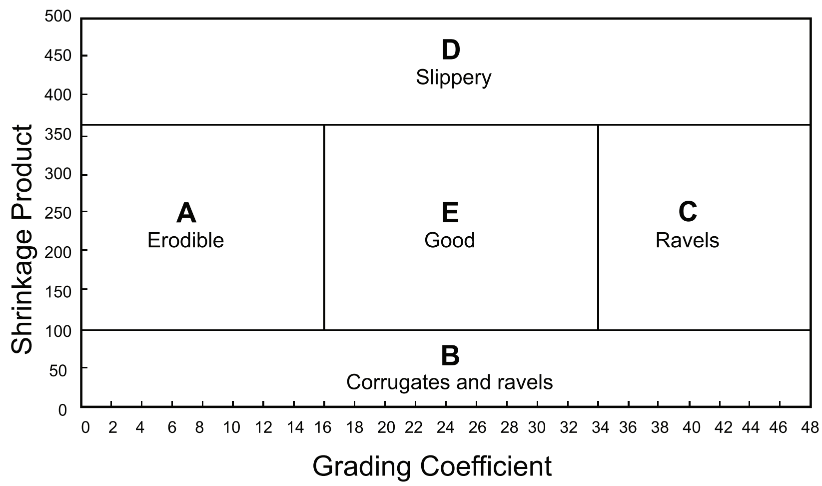



**Figure B.5.2 Gravel wearing course material quality zones**

The material quality zones define material quality in relation to their anticipated in-service performance. The combination of grading coefficient and shrinkage product of each material determines which material quality zone it falls into. The characteristics of materials in each zone are as follows:

**A:** Materials in this area generally perform satisfactorily but are finely graded and particularly prone to erosion. They should be avoided if possible, especially on steep grades and sections with steep cross-falls and super-elevations. Roads constructed from these materials require frequent periodic labour-intensive maintenance over short lengths and have high gravel losses due to erosion.

**B:** These materials generally lack cohesion and are highly susceptible to the formation of loose material (ravelling) and corrugations. Regular maintenance is necessary if these materials are used and the road roughness is to be restricted to reasonable levels.

**C:** Materials in this zone generally comprise fine, gap-graded gravels lacking adequate cohesion and a uniform grading, resulting in ravelling and the production of loose material.

**D:** Fine, plastic materials prone to slipperiness and excessive dust.

**E:** Materials in this zone perform well in general, provided the oversize material is restricted to the recommended limits.

### 5.3.7 Filter/drainage material

Filter materials have crucial roles in assisting in the prevention or in controlling the ingress of water and in the reduction of pore water pressures within both the earthworks and the pavement. Filter materials can account for a significant proportion of the construction material costs, particularly in wetter regions where road designs need to cater for the dispersion of large volumes of water, both as external drains and as internal layers within wet-fill embankments. As set out in Table B.5.7, the general requirements for filter material are a highly permeable mix comprising a durable aggregate that is resistant to chemical alteration.

**Table B.5.7 Basic requirements for filter/drainage materials**

| Key Engineering Factor | Material Requirement[1] |
|---|---|
| Permeability | The fundamental filter property is primarily a function of material grading. It is generally desirable for filter aggregates to be single-sized and equi-dimensional as this aids flow distribution and facilitates packing. It is also considered better to use material with rounded to sub-rounded rather than angular particles. |
| Strength | Aggregate particles need to be load resistant to abrasion and any loads imposed by the road design. |
| Resistance to Degradation | Aggregate particles need to be resistant to breakdown due to wetting and drying and we athering during construction and for the life of the project. |
| Resistance to Erosion | The as-placed material must be resistant to internal and external erosion. |
| Chemical Stability | Aggregate should generally be inert and resistant to alteration by groundwater. Weak surface coatings such as clay, iron oxide, calcium carbonate, gypsum etc. are undesirable. |
| Grading | D15 for filter material/d15 for adjacent subsoil ≥ 5 with minimum of 50% retained on 2 mm sieve. |

Note:

1. Actual requirements will depend on the individual situation and environment.

### 5.3.8 Rock-fill and gabion stone

It is often necessary to produce larger rock particles to fill gabion baskets, for rock fill or to provide erosion protection materials. These can be either hand-picked from suitable gravel materials or produced by breaking blasted stone from a quarry, the latter being significantly more expensive. Such materials need to be hard and durable with property requirements as shown in Table B.5.8.

**Table B.5.8 Basic requirements for rock used for fill and erosion protection**

| Key Engineering Factor | Material Requirement |
|---|---|
| Strength | Aggregate particles need to be load resistant to abrasion and any loads imposed by the road design. |
| Resistance to degradation | Aggregate particles need to be resistant to breakdown due to wetting and drying and weathering during construction and for the life of the project. |
| Resistance to erosion | The as-placed material must be resistant to internal and external erosion. |
| Chemical stability | Aggregate should generally be inert and resistant to alteration by groundwater. |
| Rock fill | Rock fill should comprise angular rock fragments interspersed with finer material to fill the voids and facilitate mechanical interlock of the rock. Tabular, flaky or elongated stones and boulders should not be used. The maximum size of rock which may be used in rock fill is 750 mm, and the layer thickness before compaction shall not be in excess of one and a half times the average actual size of the rock. |
| Dimensions and placing for gabion baskets (for retaining walls) | Gabions shall be hand-packed with clean, hard, durable, unweathered boulders or broken rock of 150 mm minimum dimensions and 300 mm maximum dimension. The sides shall be packed first in the form of a wall, using the largest pieces, with the majority placed as headers with broken joints to present a neat outside face. The interior of the gabion shall be hand packed with smaller pieces and the top layers shall be finished off with larger pieces. The whole interior and top layers shall be packed tight and hammered into place. |

### 5.3.9 Commercial products

Many commercially produced products are used in road construction. These include lime, cement, bituminous binders, and bitumen emulsions. Such products are normally procured from registered manufacturers or vendors and must comply with national or international (e.g. ASTM, BS) standards or national standards where they exist. Stabilisation of natural gravel materials is normally not required for LVRs due to the low traffic loading. The high costs of stabilisers can also be a prohibiting factor and the use stabilisation in the design of LVR's should only be considered in exceptional circumstances where no alternative materials are available in the project area.

The use of non-traditional commercial additives and stabilisers to LVRs requires careful design and is seldom cost-effective. Such products should only be considered when traditional stabilisers cannot be used effectively. No general guidance on the use of the chemicals can be given as the types, actions and uses of such products can differ widely. However, the following aspects should be considered before using any chemical stabiliser:

- Is the use of the chemical going to be cost effective and give some kind of financial, social or environmental benefit that is value for money? It may often be more cost-effective to import a better material from further away.
- Does the chemical consistently increase the strength of the material, if it is to be used as a stabiliser? This can be checked in a laboratory using traditional CBR testing. It has been found that the application rate of stabilisers is critical, as some materials react better with some chemicals. But this may vary considerably within a material source and ongoing testing of the compatibility between material and chemical must therefore be carried out.
- Products used for dust palliation are best tested on short sections of road before full-scale use. It is very difficult to test their effectiveness in the laboratory as a result of the speed and abrasion of vehicles that generate dust.
- Many commercially available chemical products are costly. It may be more cost-effective to place a bituminous surfacing on the natural material to conserve it for the full life of the road. Gravel loss may

be reduced through the use of a chemical stabiliser, but the road is still an unpaved road and will still be subjected to traffic and environmental erosion and associated material loss.

The South African Pavement Engineering Manual (SAPEM) provides guidance in selecting, assessing and utilizing non-traditional stabilizers. Refer to Section 6 of Chapter 3, Section 6.2 of Chapter 4 and Section 6.3 of Chapter 9.

## 5.4 The DCP-DN approach to the assessment of construction materials

### 5.4.1 General

A holistic approach is required in order to optimise the use of naturally occurring materials under the DCP-DN method. Attention must be given to compatibility between the pavement structure, the materials used, the type of surfacing, the construction process and, above all, control of moisture through effective drainage. Where some degree of risk in long-term performance can be accepted, then strict materials requirements may be relaxed and a wide range of naturally occurring non-standard materials may successfully be used. However, such practice demands careful attention to three factors:

- **Basic engineering principles**: there must be a careful evaluation of the in-service environment and a reasonable assurance that adequate internal and external drainage will be provided.
- **Compacted density and thickness**: there must be very good construction quality control (refer to Chapter B.11 – Quality Assurance and Control).
- **Probability of failure**: there must be an acceptance of a possibly higher risk of lesser performance.

The traditional approach to assessing material quality is based on the CBR design method. Although it is necessary to have a fundamental knowledge and understanding of the traditional properties of the materials in each layer, an alternative and potentially simpler approach using the DCP method is provided in this Manual. The design procedure does not use the traditional CBR-based approach but uses the DCP-DN design approach and related method of materials selection.

A fundamental feature of the DCP design method is that it utilises the existing road structure without disturbing its inherent strength derived from consolidation by traffic over many years. It requires the addition of a minimum thickness base (sometimes sub-base) layer of appropriate quality. The material quality is expressed in terms of the materials DCP resistance to penetration, i.e. its DN value, at the specified compaction density and expected in-service moisture condition – the parameter that serves as the criterion for selecting the materials to be used in the upper/base layer of the LVR pavement.

The DCP design approach and related method of materials selection differs markedly from the more traditional design approaches. In these traditional approaches, the pavement materials are traditionally evaluated using standard classification tests, such as grading and plasticity. However, research and investigations from the region and internationally have led to replacing these criteria with tests and specifications based on the composite measure of a material's ability to accept an imposed load without unacceptable deformation. More specifically, it has been shown that ***provided the design DN value is achieved*** - essentially a measure of a material's shear resistance to penetration at a given moisture and density - the in-service performance indirectly takes account of the actual grading and plasticity of the materials. These properties do not need to be separately specified for LVRs. Thus, a poorly graded, highly plastic material would not be expected to provide a relatively low DN value (high resistance to penetration) that is required for the base layer of a LVR.

By optimising the in situ material properties, many local materials can be used without increasing the risk of failure. A lower quality material (e.g. a G30 in place of a G60) can for example be utilised if the moisture content in the road will not exceed a certain percentage of the applicable OMC.

The three material parameters that need to be specified for the imported pavement layers are:

- **Density**: The density to which the material in the upper/base layer must be compacted should be the highest that is practicable, i.e. "compaction to refusal".
- **DN value**: The DN value of the materials to be used in the upper/base layer of the pavement at a specified density and moisture content. These values are determined as an output of the DCP design method.
- **Grading modulus:** The minimum GM (typically > 1.0) and maximum GM (typically < 2.25) of the material as a prerequisite for subsequent laboratory testing

These aspects are addressed in relation to pavement design in Chapters B.6 and B.7.

### Testing of materials in the laboratory using the DCP

Potential borrow pits are surveyed by trial pit excavation and sampling at the detailed design stage. The survey is required to prove sufficient quantities for all pavement layers. The minimum sampling frequency per DN test is indicated in Table B.5.9. Samples may also be taken from the existing road pavement for laboratory testing. The method for testing materials in the laboratory using the DCP is included in Appendix B.2.

**Table B.5.9 Minimum test frequency**

| Intended Use | Required volume ($m^3$)/ DN test |
|---|---|
| Base course | 5,000 |
| Sub-base | 10,000 |

The manner of dealing with oversize particles in the sample preparation for DN testing should be strictly in accordance with the following procedure:

- Screen the field sample on the 20 mm sieve.
- Crush the oversize material to pass the 20 mm sieve (maximum 30% by mass of the final sample).
- Add the crushed material to the minus 20 mm material from original sample and mix thoroughly.

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

## 5.5 Material improvement

### 5.5.1 General

Obtaining local materials that comply with the necessary grading and plasticity specifications for a gravel wearing course can be difficult. Materials can however be improved by mechanical modification. This includes reducing the amount of oversize or improving the grading by processing and improving materials properties by blending of two or more materials with varying properties. Under certain circumstances an armoring layer can be applied to a weak base course. These material processing and improvement measures are described below.

### 5.5.2 Mechanical modification – reducing oversize

In order to achieve the required wearing course properties a suitable Particle Size Distribution (PSD) can be obtained by breaking down oversized material to a maximum size of 50 mm or smaller. There are various measures for reducing oversize including the use of labour, mobile crushers, grid rollers or rock crushers. The choice of method depends on the type of project and the material to be broken down.

- **Hand labour**: This is quite feasible, especially on smaller, labour-based projects where material can either be hand screened and/or broken down to various sizes and stockpiled in advance of construction.
- **Mobile crushers**: The crushing of borrow pit materials may be achieved with a single stage crushing unit or, at the other extreme, a stage crushing and screening plant.
- **Grid rollers**: These are manufactured as a heavy mesh drum designed to produce a high contact pressure and then to allow the smaller particles resulting from the breakdown to fall clear of the contact zone, as illustrated in Figure B.5.3.
- **Rock crusher**: The “Rockbuster” is a patented plant item which is a tractor-towed hammermill. The hammermill action of the Rockbuster acts on the material that it passes over, breaking down both large and small sizes. There is the potential to “over-crush” a material and create too many fines in the product. It may be necessary to draw out only the larger particles in a material and process these with the Rockbuster, with the crushed material then blended back into the original product. The Rockbuster machine is illustrated in Figure B.5.4.

Figure B.5.3 Grid Roller

Figure B.5.4 Rockbuster

### 5.5.3 Mechanical modification – material blending

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

- Mixing of materials from various parts of a deposit at the source of supply;
- Mixing of selected, imported material with *in situ* materials; or
- Mixing two or more selected imported natural gravels, soils and/or quarry products on-site or in a mixing plant.

Such modification can achieve the following:

- Correction of grading associated with gap graded or high fines content gravels;
- Correction of grading and increasing plasticity of dune or river-deposited sands which are often single sized; and
- Correction of grading and/or plasticity in crushed quarry products.

The following methodology, using a ternary diagram (Figure B.5.5), and relevant example material data shown in Table B.5.10, has been developed for determining the optimal mix ratio for blending two or more materials to meet the required grading specification for unpaved roads but could be applied to improve the grading of any material:

2. Identify potential material sources that can be used to improve the available material.
3. Determine the PSD of the available material and that considered for addition or blending (wet sieve analysis recalculated with 100 per cent passing the 37.5 mm sieve).
4. Determine, for each source, the percentages of silt and clay (< 0.075 mm), sand (0.075 - 2.0 mm) and gravel (2.0 - 37.5 mm).
5. Plot the material properties of Material A and Material B on the ternary diagram as points A and B respectively (The points A and B shown in Figure B.5.5 are an example using two typical soils (the optimum grading is shown by the shaded area).
6. Connect the points. When the two points are connected, any point on the portion of the line in the shaded area indicates a feasible mixture of the two materials. The optimum mixture should be at point C in the centre of the shaded area.
7. The mix proportions of Material A to Material B are then the ratio of the line BC:AC. This can be equated to truck loads and dump spacing.
8. Once the mix proportions have been established, the Atterberg Limits of the mixture should be determined to check that the Shrinkage Product is within the desirable range (100 – 365 (or 240 if necessary)). The mix proportions should be adjusted until the required Shrinkage Product is obtained but ensuring that the mix quantities remain within the acceptable zone.
9. If the line does not intersect the shaded area at any point, the two materials cannot be successfully blended and alternative sources will have to be located, or a third source used for blending.

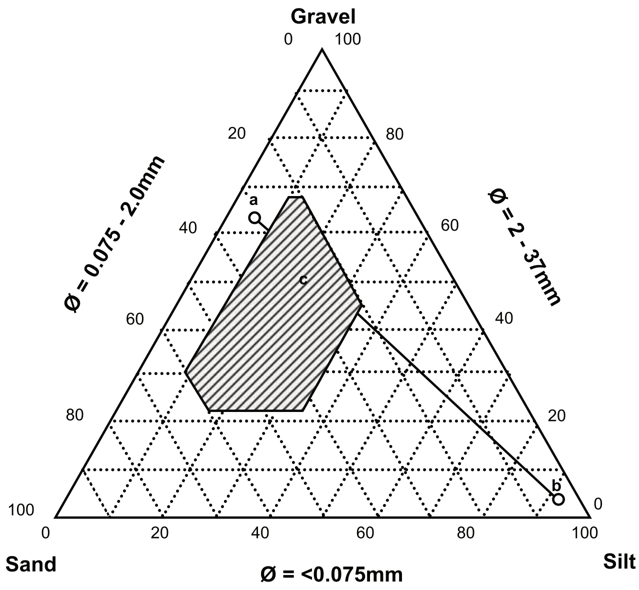



**Figure B.5.5 Use of ternary diagram for determining proportions for blending of materials**

**Blending example:** The material from Source A has a Grading Coefficient of 20 and a Shrinkage Product of zero. This material plots in Zone B of the wearing course material specification and is therefore likely to corrugate and ravel.

The material from Source B has a Grading Coefficient of 4 and Shrinkage Product of 470. This material plots in Zone D of the specification and would typically be dusty when dry and slippery when wet.

The relative proportions for each material are plotted onto the ternary diagram as points A and B which are then connected (Figure B.5.5). The midpoint of the line within the shaded area is located at point C. The mix proportions are thus the ratio of the line BC:AC. In this instance, the ratio is approximately 4:1, which indicates that one part of Material B should be mixed with four parts of Material A (i.e. one truck load of Material B for every four truck-loads of Material A). After blending, the grading coefficient and shrinkage product are 20 and 139 respectively, which falls within Zone E of the specification for gravel wearing course. The characteristics of the Materials A and B and the blended material are given in Table B.5.10.

**Table B.5.10 Characteristics of the blended materials**

| Parameter | Material | | |
|---|---|---|---|
| | A | B | Blend C (4A : 1B) |
| % passing screen size (mm) | | | |
| 37.5 | 100 | 100 | 100 |
| 26.5 | 85 | 100 | 100 |
| 4.75 | 42 | 97 | 88 |
| 2.0 | 38 | 96 | 50 |
| 0.425 | 20 | 94 | 35 |
| 0.075 | 7 | 92 | 24 |
| Atterberg Limits | | | |
| Linear Shrinkage | NP | 5 | 4 |
| Shrinkage Product | 0 | 470 | 139 |
| Grading Coefficient | 20 | 4 | 20 |
| Ternary grading proportions | | | |
| % silt/clay ($P_{0.075}$) | 7 | 92 | 24 |
| % sand ($P_{2.00}$ - $P_{0.075}$) | 31 | 4 | 26 |
| % gravel (100 - $P_{2.00}$) | 62 | 4 | 50 |

### 5.5.4 Armouring weak road bases

Armouring involves strengthening of the surface of a weak base course in order to increase shear resistance while enhancing the bond between the base and the surfacing. This has the effect of strengthening the interface between the base and the surfacing. It also provides a small increase to the strength of the base layer. By strengthening the interface between the base and the surfacing the ingress of moisture into the base is controlled, thus enhancing the performance of the base.

Coarse aggregate is used for armouring. If the base material is sand, the aggregate is mixed with the sand in the upper part of the base to a maximum depth of 50 mm before watering and compaction. If the base material is a clayey material, the aggregate is spread in a thin layer on top of the compacted base and then watered and rolled into the base.

# 6. PAVEMENT DESIGN FOR PAVED ROADS

## 6.1 Introduction

The main objective of design of a flexible pavement structure with a thin bituminous surfacing is to provide suitable structural layers that:

- distribute the loads applied by traffic (axle and wheel loads) so as to avoid overstressing the in situ subgrade conditions as illustrated in Figure B.6.1 and prevent excessive permanent subgrade deformation resulting from repetitive vertical compressive strains;
- have sufficient intrinsic material characteristics to prevent the shear stresses resulting from the three-dimensional stress state under repetitive loading from coming close to or exceeding the failure deviator stress ratio of the pavement layer material; and
- ensure sufficient overall pavement stiffness to minimise deflections and prevent excessive fatigue cracking in the bituminous surfacing resulting from repetitive horizontal tensile strains.

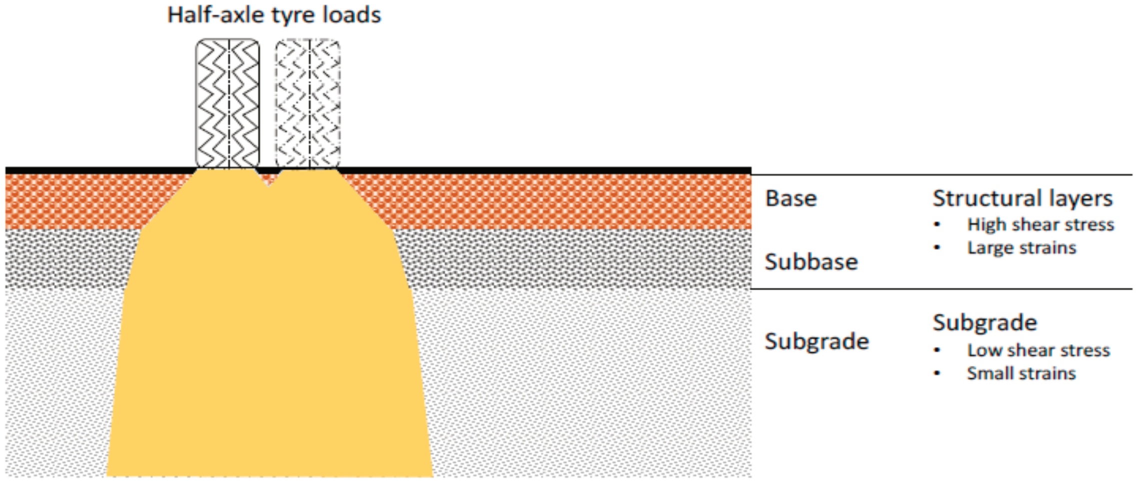



**Figure B.6.1 Use of stiff upper pavement layers to distribute stress on subgrade**

The design methods described in this Manual are adapted to make maximum use of the local *in situ* materials and any structural capacity that the existing pavement may already possess.

The surfacing of low volume sealed roads can consist of many different types of a thin flexible bituminous layer designed to produce a durable and waterproof seal. The structural design of LVRs with such different types of thin bituminous surfacings is identical because the surfacing adds no significant structural strength to the overall pavement.

Thicker surfacings that provide a structural component to the pavement may also be used for LVRs. The structural design for such roads is different from LVRs with a thin bituminous surfacing.

The purpose of this Chapter is to provide details of the structural design of LVRs that are paved. Guidance for the design of gravel and earth roads is provided in Chapter B.7. Two design methods are described in this Chapter; both are essentially ‘catalogue’ methods. The catalogue method is the most convenient, and indeed the most common, method of design for LVRs. For each type of pavement, designs have been produced based on experimental and empirical evidence for a range of subgrade strengths and a range of traffic loading levels. The Chapter describes how the designer must relate the key characteristics of subgrade strength (obtained as described in Chapter B.4), the expected traffic loading (obtained as described in Chapter B.2) and the properties of the available materials (some of which may be within an existing road or track) to determine the required pavement design.

Effective drainage is of critical importance to ensure the durability of the road pavement. The upper pavement layers must be kept as dry as possible. The user of this Manual should refer to guidance provided in Part A and Part C on appropriate road cross-sections and the design of Internal Drainage.

## 6.2 Design methods for bituminous surfaced roads

The current practice in Ghana for the pavement design of bituminous surfaced roads follows the design procedures of the Overseas Road Note (ORN) 31 Design Method (DFR Design Standards – Draft 2009). In essence this is an empirical design method using a design catalogue of pavement structures, based on design subgrade strength (CBR) and Traffic Load Classes.

The two different design methods presented in this Manual are based on research undertaken in several countries in southern Africa. The two methods are:

- The DCP-DN method; and
- The CBR method.

The CBR method is a traditional method similar to that used in ORN 31 (TRL; 1993). It is based on field material investigations and the determination of CBR properties of the existing or proposed pavement layer materials. Gourley and Greening (1999) further developed the ORN 31 design method for LVRs, applying it to lower traffic categories. The CBR method can be used in combination with dynamic cone penetrometer (DCP) testing of the *in situ* material, in which case the DCP results (DN values) need to be converted to CBR values and used as input into the design.

The DCP-DN method is in principle similar to CBR method, but instead of actual measured CBR values or DCP converted CBR values, the DN values are used for the design. The DCP-DN design catalogue is presented in terms of DN values. The DCP-DN methods is most suited for the design of upgrading of existing roads. The DCP-DN method is most suited for the design of upgrading of existing roads but may also be used for the design of new roads. For new roads in cut or fill the design DN values for the subgrade are determined as described in Section 4.2.6.

Figure B.6.2 summarises the design options.

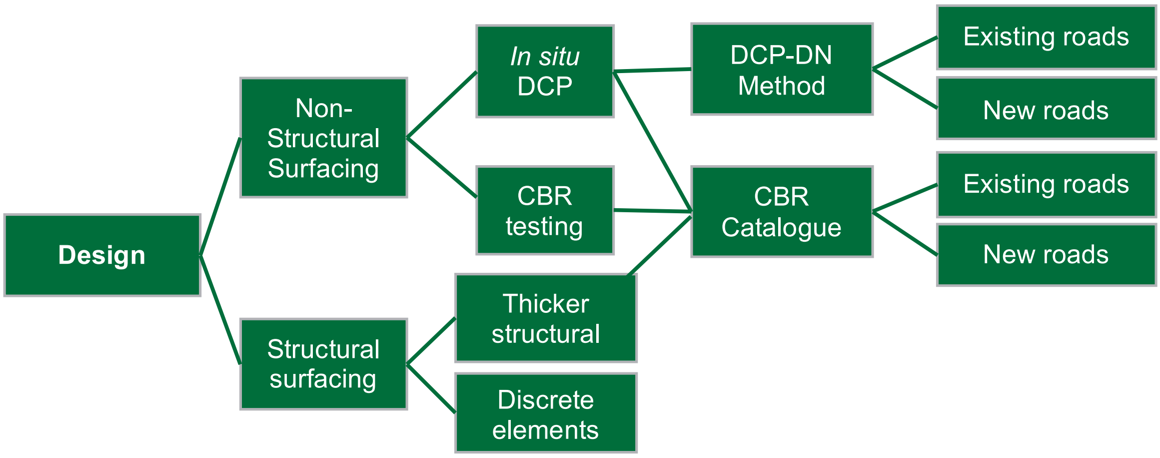



**Figure B.6.2 Design options**

## 6.3 DCP-DN method for upgrading of existing roads

### 6.3.1 Overview of the method

The DCP-DN is based entirely on using the DN values and does not require the engineer to convert the results to equivalent CBR values. The *in situ* DN values obtained from a survey of the road are plotted on a chart versus the depth and are compared directly with a related DCP-DN design catalogue. Any laboratory strength testing required for borrow pit materials is carried out with a DCP on specimens compacted in CBR moulds in the laboratory.

The flow diagram for the DCP-DN method for upgrading an existing road or track is shown in Figure B.6.3. For a new road the method is similar but there are no existing pavement layers. In this case only the subgrade properties are determined. The required pavement structures are then obtained directly from the catalogue of structures.

| | |
|---|---|
| **Step 1** | Select design period, collect and analyze traffic data, determine traffic class. |
| **Step 2** | Undertake DCP survey and subgrade sampling survey for moisture and strength measurements in the laboratory. |
| **Step 3** | For each test point determine the layer strength diagram (DN in mm/blow versus depth) using the computer program. The program analyses each DCP test point in 150 mm layers. |
| **Step 4** | Determine uniform sections from the analyzed DCP data using a CUSUMs approach. |
| **Step 5** | Option 1: Determine the representative layer strength profiles for comparison with the required profiles, based on the appropriate percentiles selected for the anticipated in-service moisture conditions relative to those at the time of the survey.<br>OR<br>Option 2: test the in situ materials in the laboratory to establish the appropriate DN value for the pavement design. |
| **Step 6** | Based on the results of the site and material surveys and the outcome of the planning process, determine the type of pavement that is the most cost effective and determine the required strength profiles from the design catalogue. |
| **Step 7** | Compare the existing profile and the required profile and determine how best to make up any deficiency. |

**Figure B.6.3 Flow diagram for the DCP-DN method**

The approach behind the DCP-DN design method is to achieve a balanced pavement design whilst optimising the use of the *in situ* material strength as much as possible. This is achieved by:

- determining the design strength profile needed for the expected traffic, and
- integrating this strength profile with the *in situ* strength profile.

To understand how best 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 result of each DCP test is a diagram of the strength of the existing pavement measured as DN values as a function of depth (Figure B.6.4).

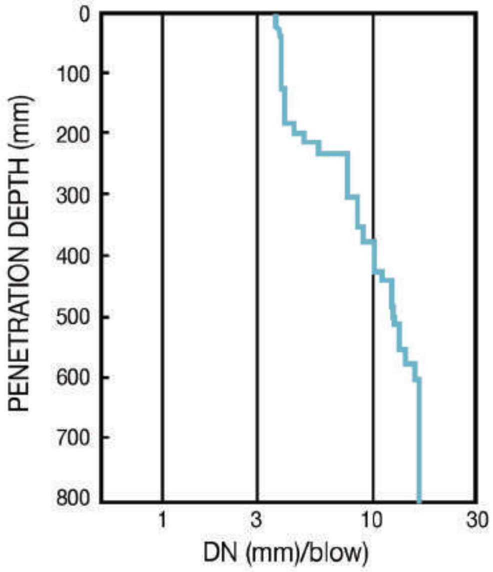



**Figure B.6.4 Typical DN with depth profile**

Useful parameters derived from the DCP analysis are the number of blows required to penetrate the top 150 mm of the pavement (this is known as the $DN_{150}$ value) and the number of blows required to penetrate from 150 mm to 300 mm ($DN_{150\text{-}300}$). These are the areas of the pavement that need to be the strongest and hence these parameters provide a quick appreciation of the likely need for strengthening. They are also useful for delineating uniform sections.

The rate of penetration of the DCP is a function of the *in situ* shear strength of the material *at the in situ moisture content and density of the pavement layers at the time of testing*. When *in situ* moisture contents are high, the bearing capacity of granular pavement materials is less (higher DN and lower CBR), while dry materials will behave with relatively greater strength (lower DN and higher CBR). Most methods of pavement design require an estimate of the values of strength that would be obtained under the worst possible conditions. It is therefore recommended that DCP testing is done at the height of the wet season. If this cannot be achieved, the *in situ* moisture condition at the time of the DCP testing should be measured and the DN values should be adjusted for wet season conditions. The relationships given in Table B.6.1 are used for this purpose.

**Table B.6.1 Percentiles of maximum DCP penetration rates by moisture conditions**

| Anticipated long-term in-service moisture content in pavement | Percentile to be used for design purposes (minimum *in situ* strength profile) | |
|---|---|---|
| | TLC 0.01 & TLC 0.3 | Design traffic TLC 1.0 |
| Drier than at time of DCP survey | 20 | 30 |
| Same as at time of DCP survey | 50 | 65 |
| Wetter than at time of DCP survey | 80 | 90 |

## 6.3.2 DCP-DN design process

Most LVR upgrading projects involve upgrading an existing engineered gravel road to a paved standard. No changes are made to the existing vertical or horizontal alignment. The main elements of the pavement analysis are summarised in the flow diagram, Figure B.6.3. The basic details are not repeated here, thus Steps 1 to 4 are assumed to have been completed.

**Step 5 Determining the *in situ* layer strength profile for each uniform section:**

**Option 1: Using *in situ* DN values**

The *in situ* layer strength profiles are obtained through a statistical analysis for each uniform section. The analysis is normally undertaken using the computer program (AfCAP LVR DCP[2]). The program uses the data from all of the DCP profiles included in that uniform section. The layer strength (DN) profiles for each uniform section are plotted as shown in Figure B.6.5 (all data) and Figure B.6.6 (average, maxima and minima).

[2] Available online at http://www.research4cap.org/SitePages/LVRDCPSoftware.aspx

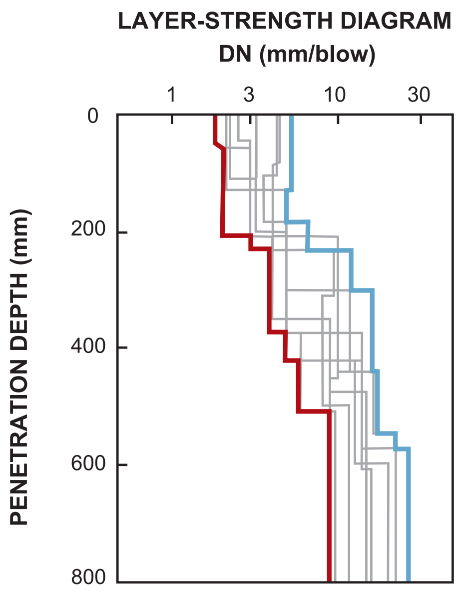



Figure B.6.5 Collective strength profile for a uniform section

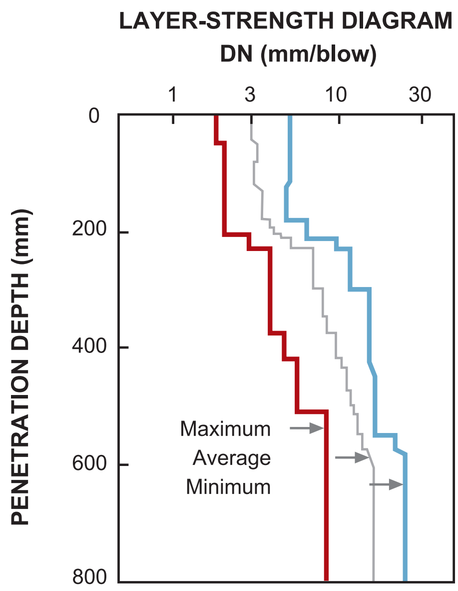



Figure B.6.6 Average and extreme DCP strength profiles for a uniform section

Various percentiles to be used in the design process can be selected in the computer program and the resultant DN value for each uniform section is computed automatically. Alternatively, this calculation can be done on a spreadsheet. The percentile to be used depend on the anticipated long-term in-service moisture content in relation to the actual moisture content at the time of the DCP survey and can be selected from Table B.6.1. The use of the respective 80$^{th}$ and 20$^{th}$ percentiles (Traffic Load Classes 0.01 and 0.3) or the 90$^{th}$ and 30$^{th}$ percentiles (Traffic Load Class 1.0) effectively results in an estimate of the expected in-service conditions.

**Option 2: Laboratory testing of samples from each uniform section**

A minimum of two bulk samples are taken from each uniform section. Each layer in the existing pavement is sampled separately. DN testing of the material is carried out in the laboratory as described in Appendix B.2. A DN value for the material in each layer in each uniform section is established based on the expected in-service density and moisture conditions (See Figure 2 in Appendix B.2).

If the existing road has only a thin gravel layer (< 75 mm) it is not necessary to test the gravel separately in the laboratory. It is assumed that the gravel will be mixed into the subgrade material during the scarifying process for preparation of the roadbed. Alternatively, a mixed sample of gravel and subgrade material can be taken to a depth of 150 mm and tested in the laboratory.

Option 2 is preferred to Option 1 wherever laboratory facilities are available for testing of the *in situ* materials.

**Step 6: The representative *in situ* strength profiles are now compared with the required strength profile.**

The required layer strength profile for each uniform section is determined from the DCP design catalogue which is shown in Table B.6.2 and illustrated in Figure B.6.7 for different traffic categories. It should be noted that the design catalogue is based on the anticipated, long-term, in-service moisture condition.

Table B.6.2 DN limits for different traffic classes

| Depth Range (Compaction requirement) | Traffic Class | | |
|---|---|---|---|
| | TLC 0.01 | TLC 0.3 | TLC 1.0 |
| 0- 150 mm Base (≥ 98% Mod. AASHTO) | DN ≤ 8 | DN ≤ 3.2 | DN ≤ 2.5 |
| 150-300 mm Sub-base (≥ 95% Mod. AASHTO) | DN ≤ 19 | DN ≤ 6 | DN ≤ 4.0 |
| 300-450 mm Subgrade (≥ 95% Mod. AASHTO) | DN ≤ 33 | DN ≤ 12 | DN ≤ 6 |
| 450-600 mm (In situ material) | DN ≤ 40 | DN ≤ 19 | DN ≤ 13 |
| 600-800 mm (In situ material) | DN ≤ 50 | DN ≤ 25 | DN ≤ 23 |

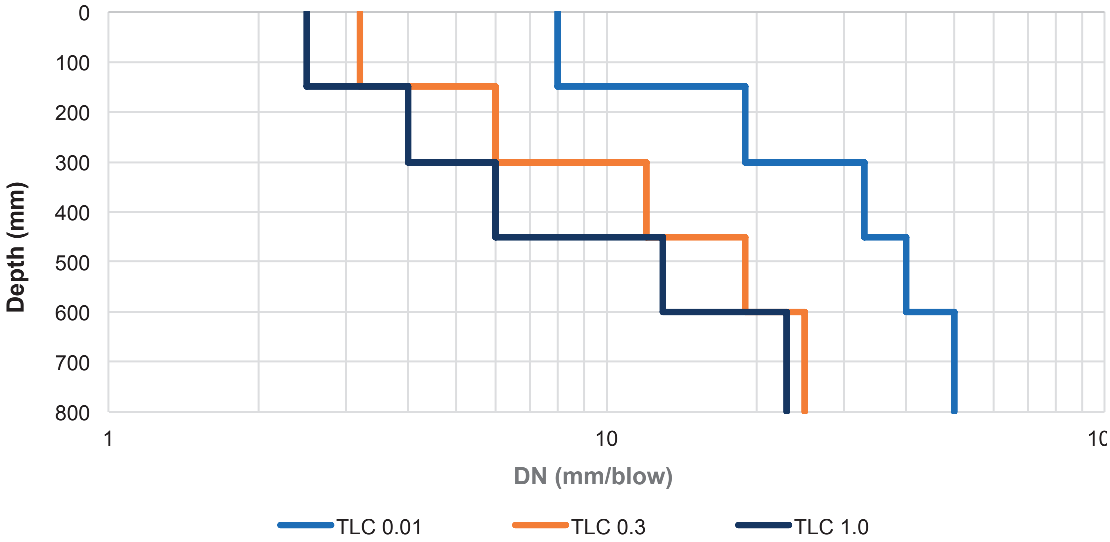



**Figure B.6.7 Layer strength profile for various traffic classes**

The required strength profile is plotted on the same layer-strength diagram on which the uniform section layer strength profiles were plotted as illustrated in Figure B.6.8. The comparison between the *in situ* strength profile and the required design strength profile allows an assessment of the adequacy of the various pavement layers for carrying the expected future traffic loading.

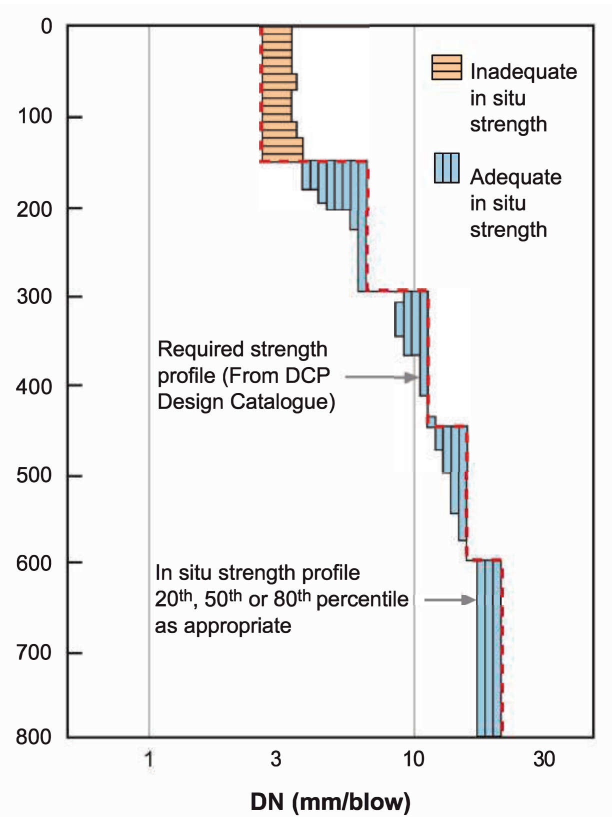



**Figure B.6.8 Comparison of required and *in situ* strength profiles**

**Step 7. Determining the upgrading requirements.**

**Option 1:** If the *in situ* strength profile of the existing gravel road complies with the required strength profile indicated by the DCP catalogue for the particular traffic class, the road only needs to be re-shaped, compacted and surfaced, assuming that the existing road is adequately above natural ground level to permit the necessary drainage requirements. If the improvement geometrics of the road requires that the road is in cut or fill the *in situ* strength profile would need to be adjusted accordingly. For example, if the road is in a 300 mm cut, then the *in situ* strength profile would be shifted upward by 300 mm.

**Option 2**: If the *in situ* strength profile of the existing gravel road does not comply with the required strength profile indicated by the DCP catalogue for the particular traffic class (as is the case in the upper 150 mm of Figure B.6.8) then the upper pavement layer(s) need to be:

- **reworked**- if only the density is inadequate and the required DN value can be obtained at the specified construction density and anticipated in-service moisture content;
- **overlaid –** if the material quality (DN value at the specified construction density and anticipated in-service moisture content) is inadequate, then appropriate quality material will need to be imported to serve as the new upper pavement layer(s);
- **mechanically stabilised –** as above, but new, better quality material is blended with the existing material to improve the overall quality of the layer; or
- **augmented –** if the material quality (DN value) is adequate but the layer thickness is inadequate, then imported material of appropriate quality will need to be imported to make up the required thickness prior to compaction.

If none of the above options produces the required quality of material, recourse may be made to more expensive options, such as soil stabilisation.

A fully worked example of the design DCP-DN method is included in Appendix B.3.

## 6.4 DCP-CBR method

### 6.4.1 Catalogue method for new roads

For roads with non-structural bituminous surfacings the flow diagram for the DCP-CBR design process is shown in Figure B.6.9 (for the design of bituminous surfacings refer to Part D of the manual). The design standards assume a flexible pavement with a granular base and sub-base. The pavement layer requirements are given in the design charts in Table B. 6.3 Bituminous pavement design Chart 1ables B.5.3 to B.5.5 provide the material specifications for the various structural layers.

| | |
|---|---|
| **Step 1** | Determine the climatic zone, select design period, collect and analyze traffic data, determine design Traffic Load Class (TLC). |
| **Step 2** | Undertake subgrade sampling survey for moisture and strength measurements in the laboratory OR carry out *in situ* DCP testing. |
| **Step 3** | Identify uniform sections based on the subgrade strength and determine design Subgrade Class (SC). |
| **Step 4** | Select Design Chart 1 or Design Chart 2 based on climate and sealed width. |
| **Step 5** | Select the pavement structure from the appropriate chart based on design Subgrade Class and design Traffic Load Class. |

**Figure B.6.9 CBR method - Flow diagram for designing a new road**

It is recommended that the Dynamic Cone Penetrometer (DCP) should be used for testing of the in situ material, because of its advantages as described in Section B.4.2.1. The DCP results (DN values) can either be used directly or may be converted to CBR values and used as input into the design.

Following testing of the subgrade material it is classified as described in Section 4.2.4. The road is divided into uniform sections and each section analysed separately as part of the Environmentally Optimised Design Approach.

The CBR design method typically utilises two design charts, each applicable to different climatic zones (i.e. wet climatic zone and moderate & dry climatic zones) and to the specific shoulder and drainage design adopted. The use of each chart also depends on the total sealed width, drainage provisions and the available materials as described below.

When the total sealed width is 7 metres or less, the outer wheel-track is within one metre of the edge of the seal. This affects pavement performance adversely because of seasonal moisture ingress. Relatively stronger pavements are therefore necessary in these situations. If the road width is sufficient for the outer wheel to be more than 1.5 metres from the pavement edge, and good drainage is ensured by maintaining the crown height, which is the vertical distance between invert of the side drain and the crown of the road, of at least 750 mm, an improvement in performance occurs and the requirements for the pavement materials may be relaxed.

The effect of drainage is reflected in the use of the design charts and the specified material properties where different sealed surface widths are treated separately (see Figure B.6.10). Thus, a wider sealed cross-section in a relatively wet environment allows a shift from Chart 1 to Chart 2. This allows the use of thinner pavement layers and a relaxation of the quality requirements for the base.

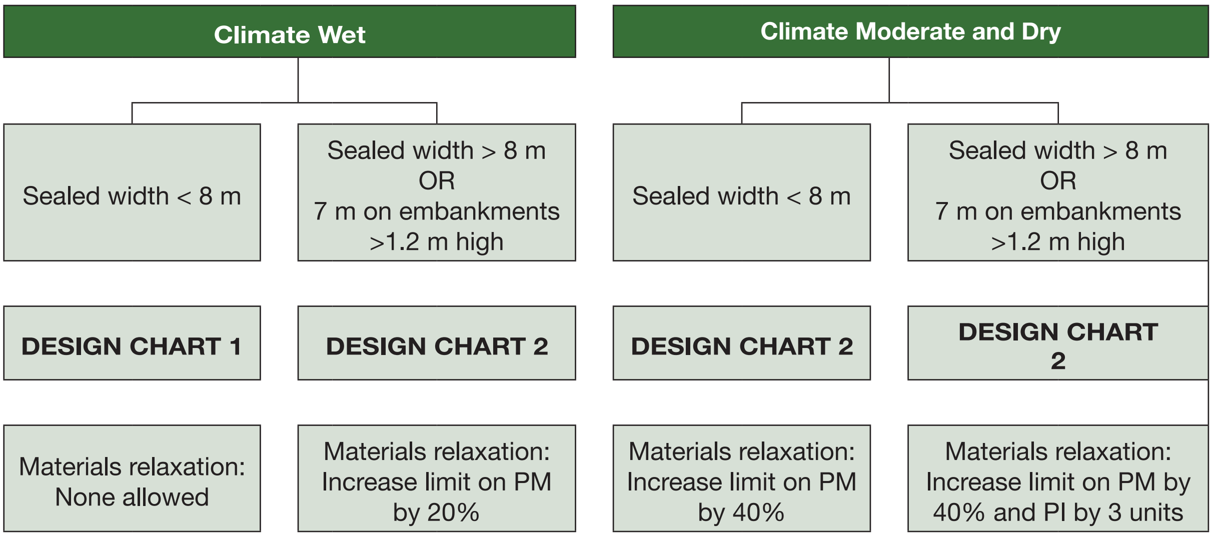



**Figure B.6.10 DCP-CBR pavement design flow chart**

When a road is on an embankment of more than 1.2 m in height, the material in the road base and sub-base stays relatively dry, even in the wet season. In this case, the design category can be relaxed, and a pavement with a 7 m total sealed width can be designed to the same criteria as for an 8 m seal.

**Wet climatic zone**

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

- Where the total sealed surface is 8 m or less, Pavement Design Chart 1 (Table B.6.3) should be used. No adjustments to the road base material requirements are required.
- Where the total sealed surface is more than 8 m, Pavement Design Chart 2 (Table B.6.4) should be used. The limit on the Plasticity Modulus of the road base may be increased by 20% (Table B.5.4 and Table B.5.5).
- 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 B.6.4) should be used. The limit on the plasticity modulus of the road base may be increased by 20% (Table B.5.4 and Table B.5.5). In the event that the road alternates between cuts and fills, these should be dealt with separately and optimised based on the situation at hand.

**Moderate and dry climatic zone**

In a moderate or dry climatic zone Pavement Design Chart 2 (Table B.6.4) should be used.

- Where the total sealed surface is 8 m or less, the limit on the Plasticity Modulus of the road base may be increased by 40% (Table B.5.4 and Table B.5.5).
- Where the total sealed surface is over 8 m or when the pavement is on an embankment in excess of 1.2 m in height, the Plasticity Modulus of the road base may be increased by up to 40% and the Plasticity Index by 3 units (Table B.5.4 and Table B.5.5).

When the project is located close to the boundary between the two climatic zones, the wetter value should be used. 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.

**Table B.6.3 Bituminous pavement design Chart 1**

| Subgrade CBR | Layer | Traffic Loading Class | | |
|---|---|---|---|---|
| | | TLC 0.01 | TLC 0.3 | TLC 1.0 |
| SC1 (< 3%) | | Special subgrade treatment required | | |
| SC2 (3-7%) | Base | 150 G60 | 125 G60 | 150 G80 |
| | Sub-base | 175 G40 | 150 G30 | 150 G40 |
| | Subgrade improvement (selected layer) | n/a | 200 G15 | 200 G15 |
| SC3 (8-14%) | Base | 100 G40 | 125 G60 | 150 G80 |
| | Sub-base | 125 G30 | 125 G30 | 150 G40 |
| | Subgrade improvement (selected layer) | n/a | 125 G15 | 125 G15 |
| SC4 (≥ 15%) | Base | 125 G40 | 175 G80 | 150 G80 |
| | Sub-base | n/a | n/a | 150 G40 |
| | Subgrade improvement (selected layer) | n/a | n/a | n/a |

**Table B.6.4 Bituminous pavement design Chart 2**

| Subgrade CBR | Layer | Traffic Loading Class | | |
|---|---|---|---|---|
| | | TLC 0.01 | TLC 0.3 | TLC 1.0 |
| SC1 (< 3%) | | Special subgrade treatment required | | |
| SC2 (3-7%) | Base | 150 G60 | 125 G60 | 125 G80 |
| | Sub-base | 150 G40 | 125 G30 | 150 G30 |
| | Subgrade improvement (selected layer) | n/a | 175 G15 | 200 G15 |
| SC3 (8-14%) | Sub-base | 100 G40 | 125 G60 | 150 G80 |
| | Base | 100 G30 | 125 G30 | 125 G30 |
| | Subgrade improvement (selected layer) | n/a | n/a | n/a |
| SC4 (≥ 15%) | Base | 125 G40 | 175 G80 | 125 G80 |
| | Sub-base | n/a | n/a | 125 G30 |
| | Subgrade improvement (selected layer) | n/a | n/a | n/a |

The design charts do not cater for weak subgrades (CBR < 3%) and other problem soils. These would first require localised subgrade improvement and mitigating measures as described in Chapter B.4 before the design of the upper pavement layers can then proceed. Once these measures have been taken, the subgrade should be reassessed, and a new subgrade class assigned. The Chart can then be used to determine the relevant layers required.

If the designer identifies that other risk factors (e.g. poor maintenance and/or construction quality) are high, then Pavement Design Chart 1 should be used regardless of the climate, sealed width and embankment height.

### 6.4.2 DCP-CBR Method for upgrading an existing road

The DCP-CBR method is an alternative design method based on the concept of the structural number (AASHTO 1993). Further details can be found in Appendix B.4. This design method is relatively complicated for LVRs. It is recommended that for the design of upgrading of existing roads, the DCP-DN method as described in Section 6.3 is used.

## 6.5 Non-bituminous surfaced roads

### 6.5.1 Non-bituminous surfacing options

Table B.6.5 lists the non-bituminous pavement options with their respective design tables.

**Table B.6.5 Non-bituminous pavement surfacing options**

| Non-bituminous pavement option | Initialisms | Table |
|---|---|---|
| Water-Bound and Dry-Bound Macadam | WBM and DBM | B.6.7 |
| Hand-Packed Stone | HPS | B.6.8 |
| Stone Setts or Pavé and<br>Mortared Stone Setts or Pavé | SSP and MSSP | B.6.9 |
| Cobblestone/ Dressed Stone<br>Mortared Cobblestone/ Dressed Stone | CS, DS & MCS, MDS | B.6.10 |
| fired Clay Brick<br>Mortared fired Clay Brick | CB, MCB | B.6.9 |
| Cast *in situ* Concrete | CIC | B.6.11 |

In Tables B.6.8 to B.6.10, unbound gravel material is used for capping, sub-base and road base. In many cases the specifications for the strength of these materials is flexible and, depending on the materials available, substitutions can be made. It is indicated in the tables where substitutions are allowed and where they are restricted. Table B.6.6 defines the allowable substitutions. This is used by simply taking the ratio of thicknesses of the material to be used and the material designated in the thickness designs in Tables B.6.7 to B.6.10 and scaling the thickness given in the tables appropriately. For example, if the thickness of a G40 material is given as 150 mm in the tables and a G80 material was more readily available the thickness required becomes: 150 x 65/80 = 122 mm.

**Table B.6.6 Substitution of pavement layer material**

| Material Designation | Material CBR (%) | Required thickness (mm) |
|---|---|---|
| G15 | 15 | 100 |
| G30 | 30 | 90 |
| G40 | 40 | 85 |
| G60 | 60 | 75 |
| G80 | 80 | 65 |

### 6.5.2 Water-Bound and Dry-bound Macadam (WBM and DBM)

A Macadam layer consists of a stone skeleton of single sized coarse aggregate (nominally 53 mm) in which the voids are filled with finer material (nominally 5 mm). Because it is made up of a single size large material, the stone skeleton contains considerable voids which are filled by fine aggregate which is washed or 'slushed' with water into the coarse skeleton. Dry-bound macadam is a similar technique to the original WBM. However instead of water and deadweight compaction being used in the consolidation of fine material, a small vibrating roller is used. WBM or DBM are commonly used as layers within a sealed flexible pavement, but in the appropriate circumstances may be used as an unsealed option with a suitably cohesive material being used as the fines component. The WBM or DBM may be constructed as a low cost, initial surface to be later sealed and upgraded as part of a 'stage construction' strategy. WBM is suitable for labour-based construction and should provide a relatively high-quality surface layer similar to a good quality natural gravel surface. However, like gravel, it is worn away by traffic and rainfall and therefore requires similar maintenance. The structural designs for WBM are similar to those required for a gravel road with the WBM itself acting as the wearing course. A capping layer and a sub-base are required as indicated but thicknesses can be reduced if stronger material is available.

**Table B.6.7 Thickness designs (mm) for WBM pavements**

| SG | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| | < 0.01 | 0.01-0.3 | 0.3-1.0 |
| SC2 | 150 WBM | 150 WBM | NA |
| | 150 G30 | 175 G20 | |
| | | 200 G15 | |
| SC3 | 150 WBM | 150 WBM | NA |
| | 100 G30 | 200 G30 | |
| SC4 | 150 WBM | 150 WBM | NA |
| | Note 1 | Note 2 | |

NOTES:

1. The capping layer of G15 material and the sub-base layer of G30 material can be reduced in thickness if stronger material is available (Table B.6.6).
2. On subgrades *CBR* > 15%, the material should be scarified and re-compacted to ensure the depth of material of *in situ* CBR >15% .

## 6.5.3 Hand-Packed Stone (HPS)

HPS paving consists of a layer of large broken stone pieces (typically 150 to 300 mm thick) tightly packed together and wedged in place with smaller stone chips rammed by hand into the joints using hammers and steel rods. The remaining voids are filled with sand or gravel. A degree of interlock is achieved and has been assumed in the designs shown in Table B.6.8. 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 Sand Bedding Layer (SBL). An edge restraint or kerb constructed, for example, of large or mortared stones improves durability and lateral stability.

**Table B.6.8 Thicknesses (mm) designs for Hand Packed Stone Pavement**

| Subgrade Class | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| | < 0.01 | 0.1-0.3 | 0.3-1.0 |
| SC1 | 150 HPS | 200 HPS | NA |
| | 50 SBL | 50 SBL | |
| | 175 G30 | 150 G30 | |
| | | 200 G15 | |
| SC2 | 150 HPS | 200 HPS | NA |
| | 50 SBL | 50 SBL | |
| | 125 G30 | 150 G30 | |
| | | 150 G15 | |
| SC3 | 150 HPS | 200 HPS | NA |
| | 50 SBL | 50 SBL | |
| | 100 G30 | 200 G30 | |
| SC4 | 150 HPS | 200 HPS | NA |
| | 50 SBL | 50 SBL | |
| | Note 1 | Note 2 | |

NOTES:

1. The capping layer of G15 material and the sub-base layer of G30 material can be reduced in thickness if stronger material is available (Table B.6.6).
2. On subgrades CBR > 15%, the material should be scarified and re-compacted to ensure the depth of material of *in situ* CBR >15% .

### 6.5.4 Stone Sett or Pavé Pavements (SSP)

SSP 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. Suitable structural designs are shown in Table B.6.9, which provides thicknesses for designs entailing Discrete Element Surfacings (DES). Such surfacings do not usually provide much structural strength in terms of load spreading because the interlock between the elements is poor. They are however very useful for areas of marketing and trading and often have the advantage that they can if necessary readily be uplifted and replaced.

**Table B.6.9 Thicknesses Designs (mm) for Discrete Element Surfacings**

| SG CBR | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| | <0.01 | 0.01-0.3 | 0.3-1.0 |
| SC2 (3-7%) | 100 DES | 100 DES | 100 DES |
| | 25 SBL | 25 SBL | 25 SBL |
| | 125 G80 | 125 G80 | 150 G80 |
| | 100 G30 | 125 G30 | 175 G30 |
| | | 150 G15 | 175 G15 |
| SC3 (8-14%) | 100 DES | 100 DES | 100 DES |
| | 25 SBL | 25 SBL | 25 SBL |
| | 150 G80 | 150 G80 | 175 G80 |
| | | 200 G30 | 225 G30 |
| SC4 (≥ 15%) | 100 DES | 100 DES | 100 DES |
| | 25 SBL | 25 SBL | 25 SBL |
| | 125 G80 | 150 G80 | 150 G80 |
| | | 125 G30 | 150 G30 |

NOTES:

1. The capping layer of G15 material and the sub-base layer of G30 material can be reduced in thickness if stronger material is available (Table B.6.6).
2. The road base layers (G80) must not be weaker.
3. The sub-base layers can be material stronger than G30 and laid to reduced thickness as shown in Table B.6.6.
4. On subgrades CBR > 15%, the material should be scarified and re-compacted to ensure the depth of material of *in situ* CBR > 15%

### 6.5.5 Cobblestone or Dressed Stone pavements (CS, DS)

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. Cobblestones 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 thickness designs are given in Table B.6.10. MCS and MDS refer to the mortared variants of CS and DS.

**Table B.6.10 Thicknesses Designs (mm) for Cobble Stone or Dressed Stone pavement**

| SG | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| | <0.01 | 0.01-0.3 | 0.3-1.0 |
| SC2 | 150 CS/DS | 150 CS/DS | 150 CS/DS |
| | 25 SBL | 25 SBL | 25 SBL |
| | 125 G80 | 125 G80 | 150 G80 |
| | 100 G30 | 125 G30 | 175 G30 |
| | | 150 G15 | 175 G15 |
| SC3 | 150 CS/DS | 150 CS/DS | 150 CS/DS |
| | 25 SBL | 25 SBL | 25 SBL |
| | 150 G80 | 150 G80 | 175 G80 |
| | | 200 G30 | 225 G30 |
| SC4 | 150 CS/DS | 150 CS/DS | 150 CS/DS |
| | 25 SBL | 25 SBL | 25 SBL |
| | 125 G80 | 150 G80 | 150 G80 |
| | | 125 G30 | 150 G30 |

NOTES:

1. The capping layer of G15 material and the sub-base layer of G30 material can be reduced in thickness if stronger material is available (Table B.6.6).
2. The road base layers (G80) must not be weaker.
3. The sub-base layers can be material stronger than G30 and laid to reduced thickness as shown in Table B.6.6.
4. On subgrades CBR> 15%, the material should be scarified and re-compacted to ensure the depth of material of *in situ* CBR >15%.

### 6.5.6 Fired Clay Brick pavement

Fired Clay Bricks (CB) 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 B.6.9 for TLC 0.01. Fired Clay Brick surfacings are not suitable for traffic classes TLC 0.3 and TLC 1.0.

### 6.5.7 Special application of mortared options

In some circumstances (e.g. on slopes in high rainfall areas and volume susceptible subgrade) it may be advantageous to use mortared options. This can be done with Hand-packed Stone, Stone Setts (or Pavé), Cobblestone (or Dressed Stone), and Fired Clay Brick pavements. The construction procedure is largely the same as for the un-mortared options except that cement mortar is used instead of sand for bedding and joint filling. The behaviour of mortared pavements is different to that of sand-bedded pavements and is more analogous to a rigid pavement than a flexible one. There is, however, little formal guidance on mortared option, although empirical evidence indicates that inter-block cracking may occur. For this reason, the option is currently only recommended for the lightest traffic divisions up to TLC 0.01 (Table B.6.9) unless other relevant local evidence is available.

### 6.5.8 Cast *in situ* Concrete

The Cast *in situ* Concrete (CIC) option for LVRs involves casting slabs 4.0 to 5.0 m long between formwork with load transfer dowels between them. The thickness of the concrete depends on the traffic and subgrade support as shown in Table B.6.11. The slabs are normally cast in half carriageway widths. A light reinforcing mesh may be included at the mid-depth of the concrete to help control cracking.

**Table B.6.11 Thicknesses designs (mm) for CIC pavement**

| SG | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| | <0.01 | 0.01-0.3 | 0.3-1.0 |
| SC2 | 160 CIC | 175 CIC | 190 CIC |
| | 150 G30 | 150 G30 | 150 G30 |
| SC3 | 150 CIC | 160 CIC | 180 CIC |
| | 100 G30 | 100 G30 | 100 G30 |
| SC4 | 150 CIC | 160 CIC | 180 CIC |
| | 100 G30 | 100 G30 | 100 G30 |

NOTES:

1. Concrete cube strength = 30 MPa at 28 days.
2. On subgrades > 30% no imported layer is required below the CIC, but the subgrade must be scarified and re-compacted.

Further details of the design approach for concrete LVRs are provided in Part D of this Manual. This constitutes a relatively high cost option that can generally only be justified in exceptional circumstances, such as on short steep slopes in areas prone to high rainfall.

# 7. PAVEMENT DESIGN FOR UNPAVED ROADS

## 7.1 Introduction

A significant portion of the road network in Ghana consists of unpaved roads. Although often rudimentary, these provide communities with access to important services (schools, clinics, hospitals and markets) and are the basis of a thriving market and social environment.

Unpaved roads are defined in this Manual as any road that is not surfaced with a “waterproof” surfacing, whether this be bituminous, concrete, interlocking blocks, cobbles or similar surfacings. Unpaved roads include earth roads and gravel roads.

Earth roads or tracks, on which vehicles, motorcycles, bicycles and persons travel directly on the *in situ* material are the simplest form of unpaved roads. The *in situ* material may in some cases be ripped, shaped and compacted (partially engineered), but generally earth roads developed through time by using traffic and the only compaction is that applied by vehicles moving over it. As such these are “un-engineered”.

There comes a point with these “roads” when passability is unduly affected by the weather and vehicles can no longer traverse the road during inclement conditions. This problem is best solved by applying a selected material with specific properties (gravel wearing course) over the *in situ* material to facilitate all-weather passability. The roads then become “gravel roads” and are considered to some extent to be “engineered”. Despite this, gravel roads may still occasionally become impassable as a result of flooding of parts of the road, in which case vehicles cannot pass because of deep water and not necessarily for any reason that can be attributed to the road surface.

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

The purpose of this Chapter is to provide guidance for the design of unpaved roads in an economic and sustainable manner such that the appropriate levels of quality are achieved. The Chapter covers the design of all levels of unpaved roads from earth roads making use of the *in situ* soil to engineered and treated gravel roads.

Unlike in the case of paved roads, any minor deformation of the support layers beneath a gravel wearing course does not unduly influence the performance of the road. The main objective of design of unpaved roads is to prevent shear failure and excessive deformation in the upper layer (wearing course) under loading as opposed to preventing cumulative subgrade deformation with time. Load distribution to protect the subgrade from high repetitive vertical compressive strains due to loads applied by traffic (axle and wheel loads) is therefore not the main concern (as it is for paved roads). Pavement distress related to subgrade deformation is addressed during routine maintenance and regravelling operation. Furthermore, since unpaved roads do not have a bituminous surfacing, limiting pavement deflections under loading is not a critical design parameter.

The need to invest in a series of structural layers is thus seldom warranted for unpaved roads. It should also be noted that a lower reliability of the design is acceptable as the repair of any possible failures on unpaved roads is much less disruptive than is the case for traditional paved road repairs.

## 7.2 Design of earth roads

Earth roads provide the cheapest, most basic form of access to rural communities for both non-motorised and motorised traffic. Such roads are normally the first stage in the construction of a more durable road and may be either “un-engineered”, “partially engineered” or “engineered” as described below:

**Un-engineered**: These are roads that typically consist of a track that is cleared of vegetation, but with no significant earthworks carried out. They are often not all-weather roads and can carry only very light traffic and then only in the dry weather or where the *in situ* soils are good (e.g. sand-clay or sand-silt-clay). Little if any drainage is generally provided. Where poor soils are encountered, such roads will generally be impassable in wet weather.

**Partially engineered**: Partially engineered or improved earth roads differ from the un-engineered earth roads described above in that the shape of the road structure is improved in order to improve drainage. The materials used are the same as the earth road, with additional *in situ* material excavated from the side of the road to form side drains (at least 150 mm below natural ground level) and added to the road to increase its height and provide a better drained road structure, as illustrated in Figure B.7.1. The road surface must be shaped to assist with water runoff and compacted to improve its strength, decrease its permeability and reduce maintenance requirements. When constructed with adequate quality materials, provided with a proper camber, adequately drained and properly maintained, the performance is enhanced to the point that it can carry higher volumes of traffic than un-engineered roads.

Partially engineered roads differ from fully engineered roads in that the geometric alignment is normally not designed and they are constructed without an imported gravel wearing course.

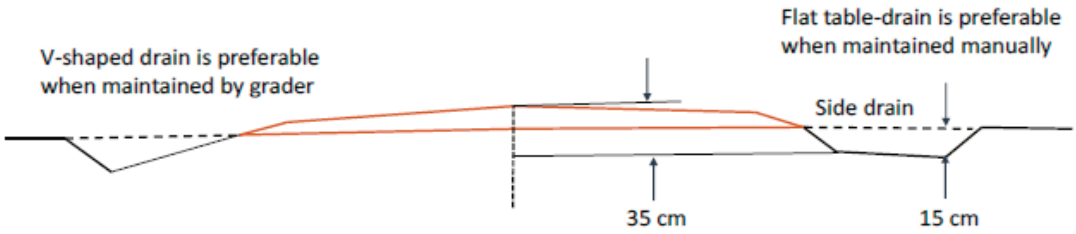



**Figure B.7.1 Cross section of typical partially engineered earth road**

The camber of carriageway and shoulders for earth roads shall be 4% to 6% depending on local conditions, to prevent potholes developing by ensuring rapid removal of water from the road surface and to ensure that excessive crossfall does not cause erosion of the surface. Although a maximum of 5% is normally recommended, it is often useful to construct a camber of 6%, as 1% or more is generally lost soon after construction.

The crown height of the improved earth road should be at least 35 cm above the bed of the side drains, which must be graded and lead into regular mitre drains to remove water from adjacent to the road as rapidly as possible.

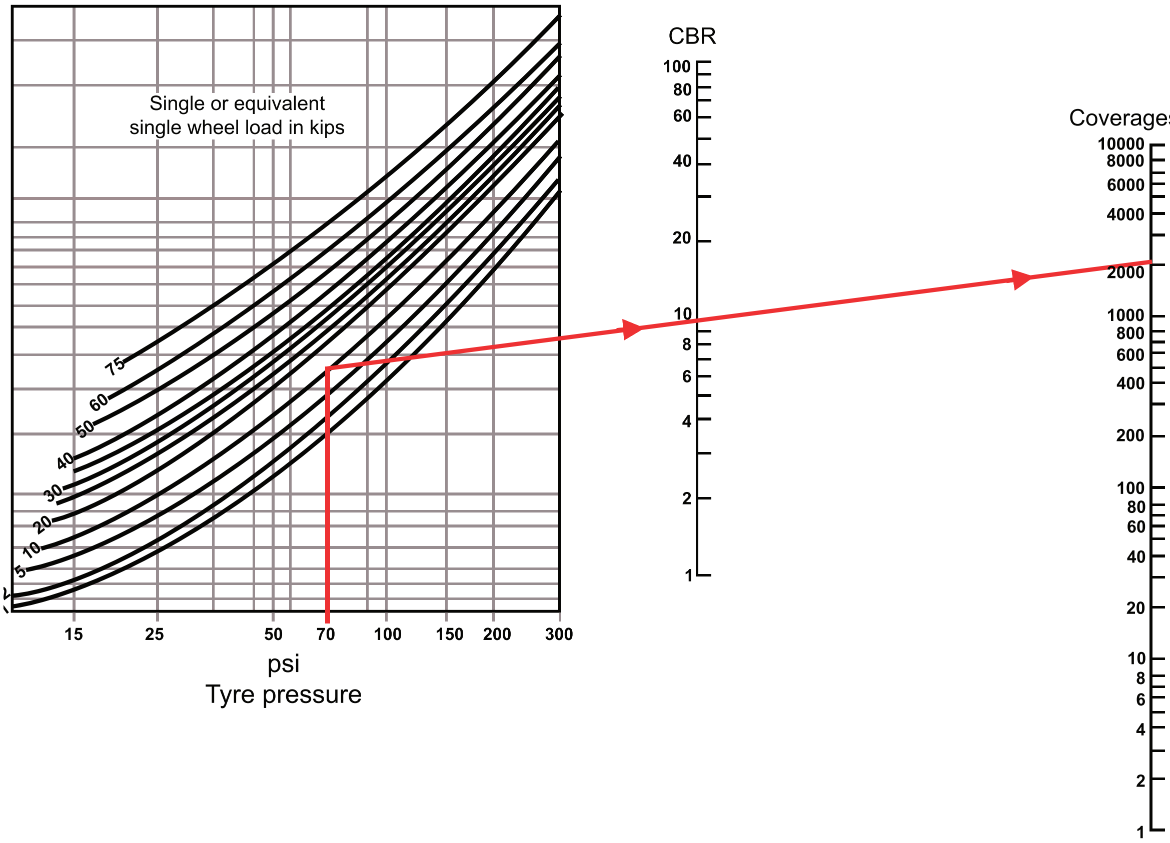
The performance of earth roads is constrained by the quality of the *in situ* materials, which in many cases is inadequate to provide good all-weather surfaces that require minimal maintenance. A knowledge of the past performance of local materials may, however, allow the use of these even though they do not comply with the required properties for good wearing course gravels. In general, no specific material requirements are applicable to earth roads but if the local materials comply with the requirements for gravel roads, a good performance can be expected.

Experience shows that a well cambered and drained earth road will usually quickly dry out after rain so that bearing capacity is rapidly restored. This suggests that partially engineered earth LVRs can be viable if communities and road users are aware of the implications and try to avoid significant trafficking, particularly by heavy vehicles, when wet. They also need to be aware of the importance of a well-maintained camber and drainage. Regular maintenance is crucial in order to maintain the performance of the pavement.

Estimating the likely performance of earth roads requires an assessment of the traffic-carrying capacity of the soils under varying environmental conditions. Research undertaken on the trafficability of soils in the United States of America (Alvin and Hammitt, 1975) provide some guidance on the traffic carrying capacity of earth roads from a knowledge of the bearing capacity (CBR) of the soil, the equivalent single wheel load of the vehicles and the tyre pressures (Figure B.7.2). If the strength of the earth road is known (in terms of its *in situ* CBR), the nomograph permits predictions to be made of the load carrying ability of the road. The definition of the terminal condition in this study was when the rut depth in the soil exceeded 50-75 mm.

As illustrated in the marked-up nomograph, an earth road with an i*n situ* CBR of 10% can be expected to provide approximately 2,000 coverages of vehicles with a single wheel load of 10 kips (4.54 tonnes) and a tyre pressure of 70 psi (482 kPa) before serious deformation is likely to occur. Since the wheel loads will not be concentrated on exactly the same path, but will wander slightly across the width of a road, one complete coverage is equivalent to the passage of 2.7 vehicles. Thus, 2,000 coverages is equivalent to 5,400 vehicles with the characteristics indicated above.

For a single lane road, the wheel loads will be restricted to narrower channels, as described in Table B.2.4 and therefore the coverages will be different. For example, for a narrow single lane road and using Table B.2.4, the number of vehicles that the earth road can accommodate before failure decreases to approximately 1350 vehicles (5400/4). For a route carrying 50 vpd and assuming 15% of them are relatively heavy (4.54 tonne wheels), this translates into a need to maintain, re-grade or reshape the surface about every 4 to 6 months. For soils with higher CBR this interval will be longer.




*SOURCE: Ahlvin and Hammitt (1975)*

**Figure B.7.2 Relationship between load, repetition, tyre pressure & CBR for unsurfaced soils**

Although earth roads can be constructed from *in situ* soils with an in-service CBR of less than 15%, the high maintenance requirements, costs, logistics and risk related to the lower strength soils mean that a soil of CBR 15% should normally be used as the minimum target *in situ* soil strength for a viable earth road. This will require a significant proportion of sand and gravel in the natural soil. In dry weather, and when surface water can run-off quickly, the *in situ* CBR is likely to be considerably higher and the capacity of the earth road increases rapidly. Conversely, in the saturated state its capacity will be very low.

It is important for both designers and road managers to appreciate that earth roads may have a low initial construction cost but that they require an ongoing commitment to regularly reshape/regrade the surface to keep it in a serviceable condition.

Areas that have specific problems (usually due to water or to poor subgrade materials) may be treated in isolation, by localised replacement of subgrade, gravelling, installation of culverts, raising the road way or by installing other drainage measures. This is the basis of a "spot improvement" approach and should be carried out to the best standard possible so that these areas will then be in a condition that is suitable for later upgrading to gravel road standard.

Given the long-term cost of regular maintenance required on earth roads, it is recommended that earth roads be upgraded to gravel roads where feasible.

## 7.3 Design of gravel roads

### 7.3.1 General considerations

A gravel road consists of a wearing course which covers the *in situ* material and, in some instances, a structural layer (sub-base) when the *in situ* subgrade has insufficient bearing capacity (*in situ* CBR < 8 % or DN > 31 mm/blow). The thickness of the wearing course will reduce with time under the influence of climate and traffic and therefore regravelling should take place before the wearing course thickness reduces to approximately 50 mm.

The performance of a gravel surfaced road depends on the quality of the materials, the location of the road (terrain and rainfall), and the traffic volume using the road. Where good quality *in situ* road building materials occur (*in situ* CBR ≥ 8% or DN 31 ≤ mm/blow), they can provide a strong enough pavement structure to carry the expected traffic for many years with no additional structural layers being required. It is, however, a prerequisite that suitable drainage must be provided, and the road prism and carriageway must be properly shaped and compacted.

To reduce the adverse impact of rainfall and water, the road must be constructed with an appropriate camber (typically 4-6%) to effectively shed surface water. 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 table drain inverts. Cross sections and geometric details are described in Part A of this Manual, but a schematic is shown for convenience in Figure B.7.3.

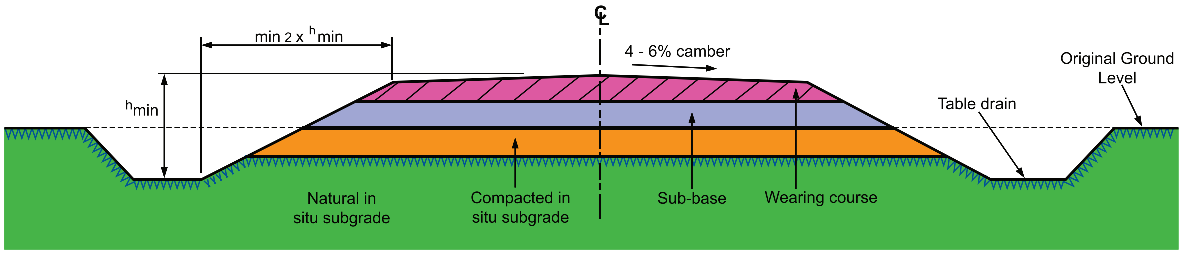



**Figure B.7.3 Typical gravel road cross section in flat terrain**

Roads described as gravel roads imply that a number of factors have been taken into account in their design and construction. These include:

- Material of a selected quality is used to provide an all-weather wearing course;
- The structure of the road and strength of the materials is such that the subgrade is protected from excessive strains under traffic loads;
- The shape of the road is designed to allow drainage of water (mainly precipitation) from the road surface and from alongside the road;
- The necessary cross and side drainage is installed; and
- The road is constructed to acceptable standards, including shape, compaction and finish.

Although an all-weather wearing course is provided, the road may not necessarily be passable at all times of the years as a result of low-level water crossings being flooded periodically. This, however, is not a function of the gravel roads design and is addressed in Part C of this Manual.

The critical aspect of gravel road design is the selection of the gravel wearing course material. The use of incorrect materials in the wearing course will result in roads that deform, corrugate, become slippery when wet, lose gravel rapidly and generate excessive dust. The required properties of good wearing course gravels are described in Section B.5.3.6.

Gravel roads that are likely to incur high maintenance costs in some circumstances, namely:

- When they carry relatively large traffic volumes (> 100 vpd);
- When the quality of the gravel is poor;
- Where no sources of gravel are available within a reasonable haul distance;
- On road gradients greater than about 6%; and
- In areas of high and/or intense rainfall.

In these circumstances spot improvements will almost certainly be justified. In some cases it may prove to be more economical to build a fully paved road at the outset.

### 7.3.2 The CBR design method

The CBR design procedure for new gravel roads consists of the following steps:

1. Determine the traffic volume and traffic loading (see Chapter B.2).
2. Determine the strength of the subgrade at the appropriate moisture condition (see Chapter B.4).
3. Establish the quality of the gravel that is to be used for the sub-base construction (see Chapter B.5). 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 sub-base (consisting of min. G15 quality material) that is necessary avoid excessive compressive stresses in the subgrade from Figure B.7.1 depending on the subgrade strength and traffic class.
5. Calculate the thickness of the wearing course based on the expected rate of gravel loss and a realistic choice of the frequency of regravelling.

**Table B.7.1 Sub-base thickness for gravel roads (G15 quality material)**

| Subgrade Strength Class | Traffic Load Class | | |
|---|---|---|---|
| | TLC 0.01 | TLC 0.3 | TLC 1.0 |
| SC1 | Special treatment required (refer Section B.4.3) | | |
| SC2 | N/A | 100 | 150 |
| SC3 & SC4 | Place GWC directly on prepared subgrade | | |

Where the in situ subgrade material has a soaked CBR < 3%, i.e. subgrade strength class SC1, realigning the road to avoid such material should be considered. Otherwise the material shall be excavated and backfilled with competent selected subgrade material of minimum soaked CBR 15% (DFR, 2007). The depth of backfill depends on the required material depth as described in Section 4.2.5.

### 7.3.3 The DCP-DN design method

For an **earth road** or an **existing gravel road** being upgraded the process below should be followed:

- Determine the DCP penetration rate for the upper 150 mm and the 150 - 300 mm layers of the existing structure ($DN_{150}$ and $DN_{150\text{-}300}$).
- Determine the DCP structural number ($DSN_{450}$ or number of blows to penetrate 450 mm).
- Plot the data using a cumulative sum (CUSUM) technique to determine uniform sections. If the uniform sections delineated by the three parameters ($DN_{150}$, $DN_{150\text{-}300}$ and $DSN_{450}$) differ significantly it is necessary to look at the individual DCP profiles and decide whether the differences are significant. Low $DSN_{450}$ values indicate weak support while low $DN_{150}$ values indicate that the upper 150 mm of the road is weak.

A similar process is carried out for **new roads** bearing in mind that the upper 150 mm layer will at least be ripped and re-compacted and additional formation material will usually be imported to raise the level of the road above natural ground level.

DCP testing is carried out at *in situ* moisture and density conditions. It is recommended that the testing is done at the end of the wet season (when the subgrade is probably at or close to its worst moisture condition), but some interpretation (judgement) may be required at the time of the DCP test survey regarding the moisture conditions. It must be noted how the subgrade conditions are expected to relate to their condition in service, i.e. whether the subgrade is likely to be in a similar state, wetter or drier in service than when the survey was carried out. Areas that are expected to be soaked or flooded periodically must also be noted.

On this basis, the *in situ* material condition should be divided into uniform sections with a characteristic subgrade strength for each section. The subgrade DN values will be determined as a percentile of the values determined for each uniform section.

Once the uniform sections have been identified, the subgrade can be classified in terms of its required strength to carry the expected traffic. This makes use of the following procedure:

- The characteristic subgrade strength for each uniform section is determined by assessing the $DN_{150}$, $DN_{150\text{-}300}$ and $DN_{450}$ values for each of the identified uniform sections. In order to achieve statistical validity, there should be at least 8 to 30 results for each uniform section.
- Determine the 80$^{th}$, 50$^{th}$ and 20$^{th}$ percentiles of the DN results for each uniform section in a similar manner to that described for paved roads.

**Table B.7.2 Suggested percentile of minimum *in situ* DCP penetration rates to be used**

| Site moisture condition during DCP survey | Percentile of strength profile (maximum penetration rate – DN) | |
|---|---|---|
| | Materials with strengths not moisture sensitive* | Materials with strengths that are moisture sensitive* |
| Wetter than expected in service | 20 | 20 – 50 |
| Expected in service moisture | 50 | 50 – 80 |
| Drier than expected in service | 80 | 80 – 90 |

NOTE:

* Moisture sensitivity can be estimated by inspecting and feeling a sample of the material – clayey materials (PI > about 12% can be considered to be moisture sensitive).

- Based on the moisture regime at the time of testing the percentiles of the data shown in Table B.7.2 shall be used to determine the design strength of the two upper layers. The mean (50th percentile) can be used for the less-critical underlying layers (below 300 mm).
- Compare the relevant subgrade strength profiles with the catalogue (Table B.7.3), or the layer strength diagrams Figure B.7.4 for the specified traffic categories.

Option 2 in Section B.6.3.2 provides an alternative method that may be used to assess the characteristics of the subgrade materia. A bulk sample is taken from each uniform section for testing gin the laboratory. It should be noted that the tests should be performed at the anticipated field density because it provides a more accurate assessment of the characteristics of the materials. However, it use must be balanced against the cost of sampling and transporting material to the laboratory. Correction for moisture content (Table B.7.2) is not required if bulk sampling and laboratory testing is carried out.

**Table B.7.3 Catalogue of support structures for different traffic categories (DN)**

| Traffic | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| | ≤ 2 heavy vpd | 6 – 20 heavy vpd | 20 - 60 heavy vpd |
| **Profile** | **DN (mm/blow)** | | |
| Formation or upper 150 mm ≥ 95% MDD | 25 | 14 | 14 |
| *In situ* 150-300 mm ≥ 93% MDD | 33 | 19 | 14 |
| 300-450 mm | 50 | 25 | 19 |
| 450-600 mm | 50 | 33 | 25 |
| 600-800 mm | 50 | 50 | 33 |

NOTE:

1. Heavy vehicles are defined as those vehicles classified as HGV and above (Classes 7 to 12 in Table B.2.1)

Only the upper two layers are critical; the underlying layers being given values to improve the pavement balance. The *in situ* strengths of the third layer (300 – 450 mm) and below range from 19 to 50 mm/blow, which is likely to occur in most situations. It should also be borne in mind that in most cases some formation material is likely to be placed on this *in situ* profile. This imported material will have an *in situ* DN value of between 14 and 25 mm/blow depending on the traffic.

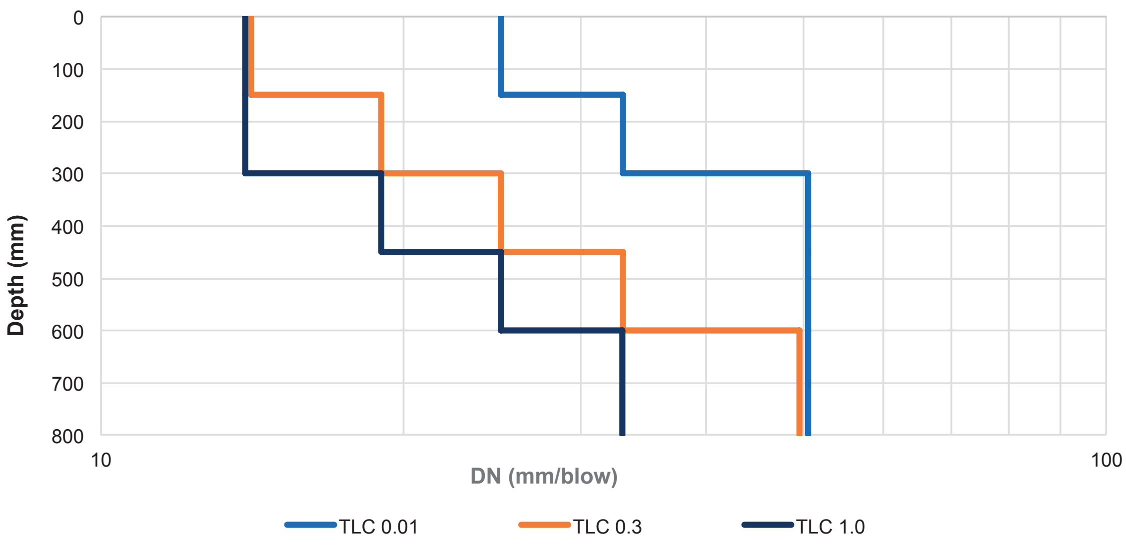



**Figure B.7.4 Layer strength diagrams of support layers for different traffic categories**

If the *in situ* profiles (selected percentiles) compare adequately with the layer strength diagrams, the wearing course layer can be placed on top. This would normally consist of 150 mm of specified material but, if the potential for delayed maintenance (i.e. regravelling) exists, an additional 50 mm should be added as a buffer layer. If the *in situ* profiles do not meet the layer strength requirements, then the recommendations provided in Option 2 of Step 7 of Section 6.3.2 should be followed.

The minimum strength of the support layer beneath the wearing course need not be very high. It becomes equal to the required minimum strength of the wearing course for higher traffic. This material may not have the necessary cohesive or grading properties to provide the necessary performance as a wearing course but must always be present. If this material complies with the requirements of Zone E in Figure B.5.2 the total thickness of the upper 150 mm formation and the wearing course can be reduced to 225 mm.

### 7.3.4 Wearing course thickness design

The design of the gravel wearing course thickness must take into account the fact that gravel will be lost from the road continuously. Other than the relatively high road user costs, this is the single most important reason why, in whole life cost terms,  gravel roads are expensive and often unsustainable especially when traffic levels increase.

Gravel loss (normally expressed in mm/year/100vpd) is a function of a number of factors including climate, traffic, material quality, road geometrics, maintenance frequency and type, etc. The appropriate wearing course thickness can be determined based on expected annual gravel loss, traffic and the number of years between regravelling operations.

Various studies and methods exist to estimate the expected rate of annual gravel loss. These include the HDM4 model, TRH20 model, ARRB model, and the Kenya Maintenance Study conducted by TRL. These models, however, often need regional calibration. A 2006 ILO study into the gravel loss experienced in Ghana shows that there is a link between the Plasticity Product ($PP = PI \times P_{0.075}$), traffic volumes and gravel loss. The PP can therefore be used to assist in determining the required wearing course thickness. The gravel losses shown in Table B.7.4 hold only for the first phase of the deterioration cycle lasting possibly two or three years. Beyond that period, as the wearing course is reduced in thickness, other developments, such as the formation of ruts or heavy grader maintenance, may also affect the loss of gravel material. However, the rates of gravel loss given above can be used as an aid to the planning for regravelling in the future.

**Table B.7.4 Typical gravel loss Ghana**

| Traffic regime | Adjusted gravel loss (mm/year/100vpd) | |
|---|---|---|
| | Low plasticity wearing course (PP < 225) | High plasticity wearing course (PP > 370) |
| Low trafficked roads (ADT < 20) | 20 | 10 |
| Higher trafficked roads (ADT = 100) | 40 | 15 |

*SOURCE: ILO (2006)*

A more accurate indication of gravel loss for a particular section of road can be obtained from periodic measurement of the gravel layer thickness. It should be noted that a wearing course of poor quality material will have to be regravelled more frequently that a material of higher quality. The economics of these decisions are addressed in Chapter B.1.

The ILO study (2006) also determined country specific HDM4 calibration factors for gravel loss ($K_{gl}$) and an overall calibration factor of 1.5 is recommended, with 2.4 for low plasticity wearing course material and 0.85 for high plasticity wearing course material.

Further guidance can be taken from Table B.7.5, which provides an approximate estimate of gravel loss based on the quality of the wearing course material.

The rates of gravel loss increase significantly on gradients greater than about 6% and in areas of high and intense rainfall. Spot improvements should be considered on these sections.

**Table B.7.5 Typical estimates of gravel loss**

| Material Quality Zone [1] | Material Quality | Typical gravel loss (mm/yr/100vpd) |
|---|---|---|
| Zone A | Satisfactory | 20 |
| Zone B | Poor | 40 |
| Zone C | Poor | 40 |
| Zone D | Marginal | 20 |
| Zone E | Good | 15 |

NOTE:

1. See Figure B.5.2 .

Regravelling should take place before the underlying layer is exposed. The regravelling frequency, R, is typically in the range 5 - 8 years.

The optimum wearing course thickness = R×AGL

Where: R = regravelling frequency in years; and

AGL = expected annual gravel loss = typical gravel loss (mm/yr/100vpd) x $\frac{ADT}{100}$

The wearing course thickness should be kept within a range of 150 mm to 250 mm for construction practicality and maintenance purposes.

# 8. LIFE CYCLE COSTING

## 8.1 Introduction

### 8.1.1 Background

There are always a number of potential alternatives available to the designer of new roads or the rehabilitation of existing ones, each of which is capable of providing the required performance. For example, as illustrated in Figure B.8.1, for a given analysis period, one alternative might entail the use of a relatively thin, inexpensive pavement which requires multiple strengthening interventions (Alternative B) whilst another alternative might entail the use of a thicker, more expensive pavement with fewer interventions (Alternative A).

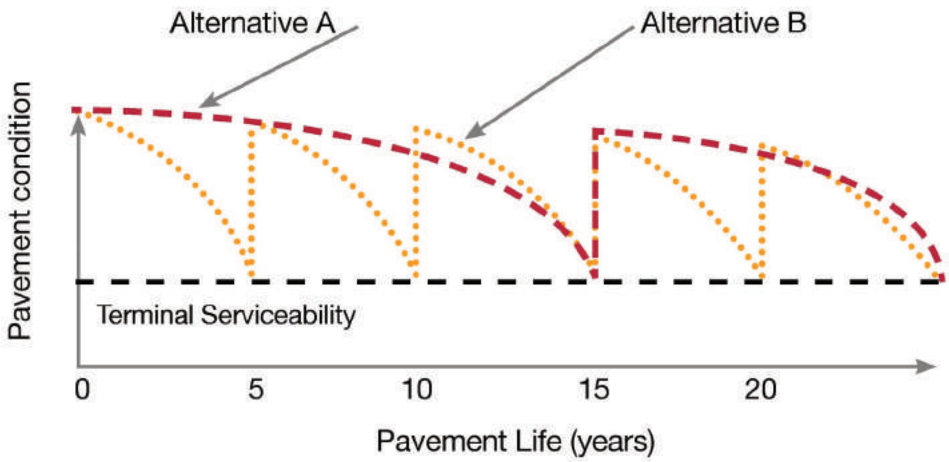



**Figure B.8.1 Effect of alternative design options on pavement condition**

In order to make the most effective use of the available resources, the designer is required to determine which alternative will serve the needs of road users for a given level of service at the lowest cost over time. Such a task can be achieved through the use of a life-cycle economic evaluation, often referred to as "life cycle" or "whole-of-life" costing.

### 8.1.2 Purpose and scope

The main purpose of this Chapter is to outline the procedure to be followed in undertaking a Life Cycle Cost (LCC) analysis to compare alternative pavement designs over their design lives in order to arrive at the most cost-effective solution. The Chapter outlines the methods of carrying out an LCC analysis and considers its necessary inputs including such factors as construction, maintenance and road user costs, residual value, discount rate and analysis period.

The focus of the Chapter is on LCC analysis of the upgrading of unpaved roads to paved standard. Though the principles of this analysis can also be applied to comparing road projects involving alternative alignments, or alternative maintenance strategies, such considerations are outside the scope of this Chapter.

## 8.2 Analysis methods

### 8.2.1 General

In the context of roads, an LCC analysis is defined as a process for evaluating the total economic worth of a project by analysing initial construction/rehabilitation costs and discounted future costs, such as maintenance, user and reconstruction costs over the life of the road or analysis period of the project. The analysis requires the identification and evaluation of the economic consequences of various alternatives over time, primarily according to the criterion of minimum total LCCs.

As illustrated in Figure B.8.2, the principal components of an LCC analysis are the initial investment or construction cost and the future costs of maintaining or rehabilitating the road, as well as the benefits due to savings in user costs over the analysis period selected. An assessment of the residual value of the road is also included so as to incorporate the possible different consequences of construction and maintenance strategies for the pavement/surface options being investigated.

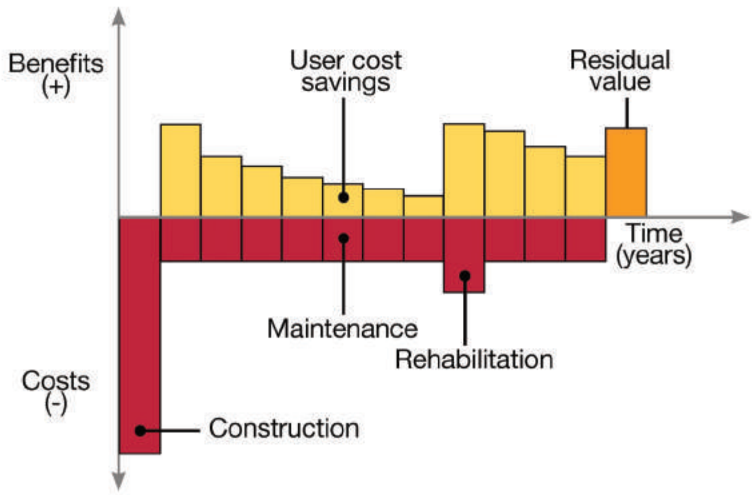



**Figure B.8.2 Distribution of costs and benefits during the life cycle of a road**

## 8.2.2 Method of economic comparison

In the life cycle analysis process, alternative pavement/surface options are compared by converting all the costs and benefits that may occur at different times throughout the life of each option to their present day values. Such values are obtained using discounted cash flow techniques involving the use of an appropriate discount rate, to determine the Net Present Value (NPV) of the pavement/surface options. Costs and benefits are usually estimated in constant local currency terms to eliminate the effects of inflation. Indirect taxes are usually excluded from costs and benefits.

The NPV can be calculated as follows:

$$NPV = \mathrm{C} + \mathrm{W}\sum_{i} M_i\,(1+r)^{-x_i} - S(1+r)^{-z}$$

Where:

| | | |
|---|---|---|
| NPV | = | present worth of costs |
| C | = | present worth of initial construction |
| $M_i$ | = | cost of the i[th] maintenance and/or rehabilitation measure |
| r | = | real discount rate |
| $x_i$ | = | number of years from the present to i[th] maintenance and/or rehabilitation measure, within the analysis period |
| z | = | analysis period |
| R | = | Residual value of the pavement at the end of the analysis period expressed in terms of present values. |

The NPV method is generally preferred over other methods of evaluating projects, such as the Internal Rate of Return (IRR). One of its main advantages is that it can be used to evaluate both independent and mutually exclusive projects whilst the IRR method cannot be relied upon to analyse mutually exclusive projects, so can lead to conflicts in the ranking of projects. In many cases, the project with the highest IRR may not be the project with the highest NPV. Nonetheless, the IRR, which is defined as the rate of discount which equates the present worth of the costs and benefits streams, may be computed by solving for the discount rate that makes the NPV of a project equal to zero. This may be done graphically or by iteration. On this basis, an independent project would be viable whose IRR is greater that the project cost of capital. Though this value gives no indication of the size of costs or benefits of a project it does serve as a guide to the profitability of the investment. The higher the IRR the better the project.

### 8.2.3 Components of an LCC analysis

The components of an LCC analysis associated with a particular design alternative are listed below and illustrated in Figure B.8.3.

- Analysis period.
- Structural design period.
- Construction/rehabilitation costs.
- Maintenance costs.
- Road user costs.
- Residual value.
- Discount rate.

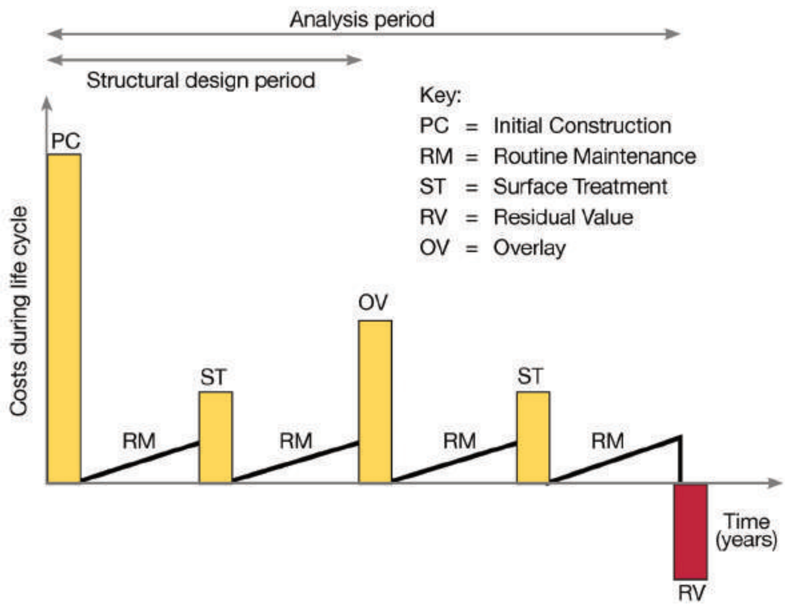



**Figure B 8.3 Components of a typical life cycle cost analysis**

**Analysis period**

This is the length of time for which comparisons of total costs are to be made. It should be the same for all alternative strategies and should not be less than the longest design period of the alternative strategies.

**Structural design period**

This is the design life of the road at which time it would be expected to have reached its terminal serviceability level and to require an appropriate intervention such as an overlay.

**Construction costs**

Unit costs for alternative pavement designs will vary widely depending on such factors as locality, availability of suitable materials, scale of project and road standard. Other factors that would typically warrant consideration include:

- land acquisition costs;
- supervision and overhead costs;
- establishment costs;
- the cost of accommodating traffic during construction; and
- the cost of any associated relocation of services.

**Maintenance costs**

The nature and extent of future maintenance is dependent on pavement composition, traffic loading and environmental influences. An assessment needs to be made of future annual routine maintenance requirements, periodic treatments such as reseals, and rehabilitation such as structural overlays.

**Road user costs**

These are the costs incurred by drivers using the road. They typically comprise vehicle operating costs (fixed costs, fuel, tyres, repair and maintenance and depreciation costs), the costs of accidents and congestion, and travel time costs. Vehicle operating costs (VOC) are related to the roughness of the road in terms of its International Roughness Index (IRI) and will change over the life of the road due to changes in surface condition and traffic. Relationships can be developed for main vehicle types which relate VOCs to variations in road surface conditions (IRI) under local conditions.

Road user costs are normally excluded from an LCC analysis that is confined to comparing alternative pavement/surfacing options, as the pavement options are considered to provide “equivalent service” during the analysis period. However, when evaluating the viability of upgrading a gravel road to a paved standard, the savings for the road user (primarily through reduced VOCs) on the latter versus the former option can be significant and are treated as benefits which should be incorporated as one of the components in the LCC analysis (ref. Figure B.8.2).

**Residual value**

The value of the pavement at the end of the analysis period depends on the extent to which it can be utilized in any future upgrading. For example, where the predicted condition of the pavement at the end of the analysis period is such that the base layer could serve as the sub-base layer for the subsequent project, then the residual value would be equal to the cost in current value terms for construction in future to sub-base level discounted to the evaluation year.

**Discount rate**

This rate must be selected to express future expenditure in terms of present values and cost. It is usually based on a combination of policy and economic considerations.

**LCC Procedure**

This is the procedure that is followed typically in undertaking an LCC analysis of mutually exclusive projects, i.e. when the selection of one project precludes selection of the other. The procedure is as follows:

1. Establish alternative project options
2. Determine analysis period
3. Estimate agency (construction and maintenance) costs
4. Estimate road user costs
5. Develop expenditure stream diagrams (similar to Figure B.8.3)
6. Compute NPV of both options
7. Analyse results, including sensitivity analysis, if warranted
8. Decide on preferred option, generally the option with the highest NPV

### 8.2.4 Selection of road design standard

The selection of an appropriate pavement design standard requires an optimum balance to be struck between construction/rehabilitation, maintenance and road user costs, so as to minimise total life cycle costs, as illustrated in Figure B.8.4. Such an analysis can be undertaken using an appropriate techno-economic model, such as the World Bank’s Highway Design and Maintenance Standards (HDM) model or, preferably, the LVR Economic Decision (RED) model which is better suited to the characteristics of LVRs.

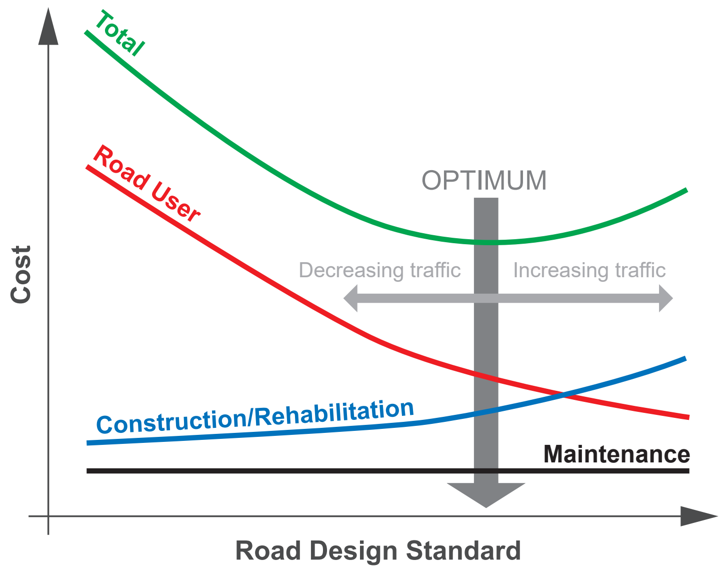



**Figure B.8.4 Economic analysis of optimum road design standard**

As indicated in Figure B.8.4, the optimum road design standard varies in relation to traffic level and the associated relative mix of construction, maintenance and user costs.

Similarly, as illustrated in Figure B.8.5, the optimum road design standard in terms of the pavement structural capacity for a relatively low traffic pavement would incur lower initial construction costs but, within its life cycle, this would be offset by higher maintenance and VOCs. Conversely, a higher traffic pavement would incur higher initial construction costs but lower maintenance and VOCs.

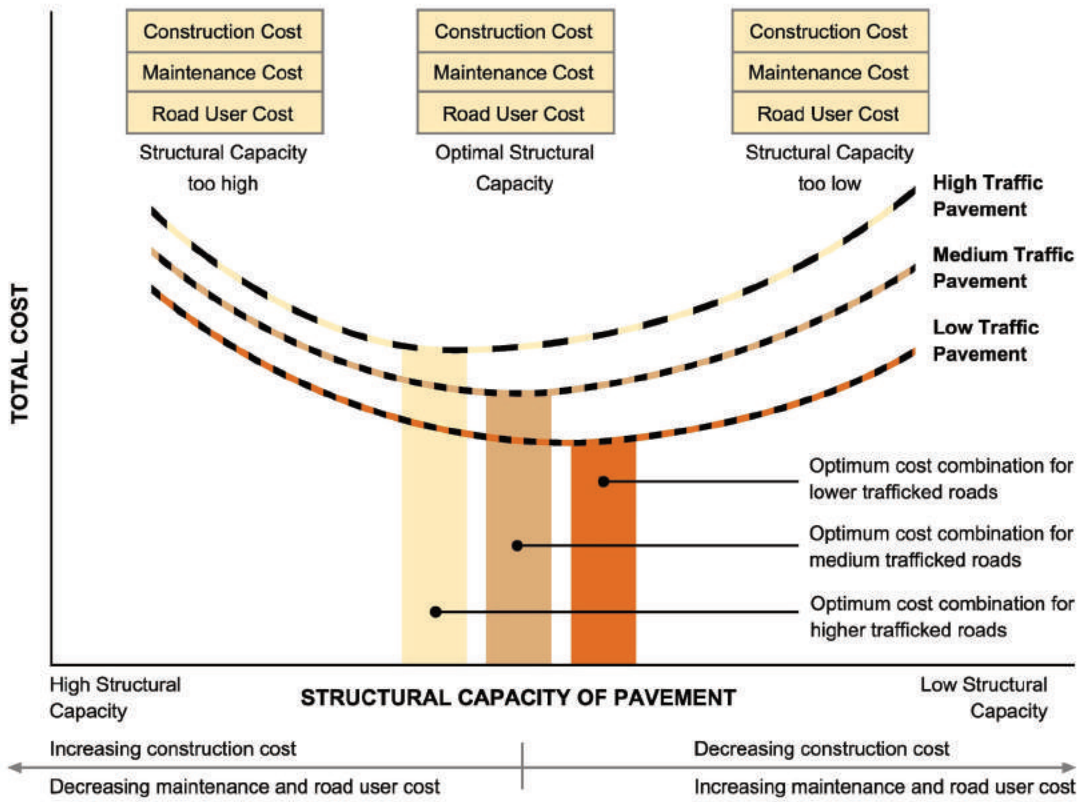



**Figure B.8.5 Combined cost for various pavement structure capacities**

## 8.2.5 Gravel versus paved road comparison

A typical situation faced by a road agency is to determine when it is economically justified to upgrade a gravel road to a paved standard. The gravel and paved road options illustrated in Figure B.8.6 and Figure B.8.7 would have a have a different relative mix of construction, maintenance and road user costs. In such a situation, an LCC analysis can be undertaken to determine the viability of upgrading a gravel road to a paved standard.

**Figure B.8.6 Gravel road**

*Lower construction costs, higher maintenance and road user costs*

**Figure B.8.7 Paved road**

*Higher construction costs, lower maintenance and road user costs*

The typical components of an LCC analysis are illustrated in Figure B.8.8. This could be undertaken using an appraisal model such as RED in which the VOC relationships may need to be calibrated for local conditions. The option with the higher NPV would generally be the preferred one.

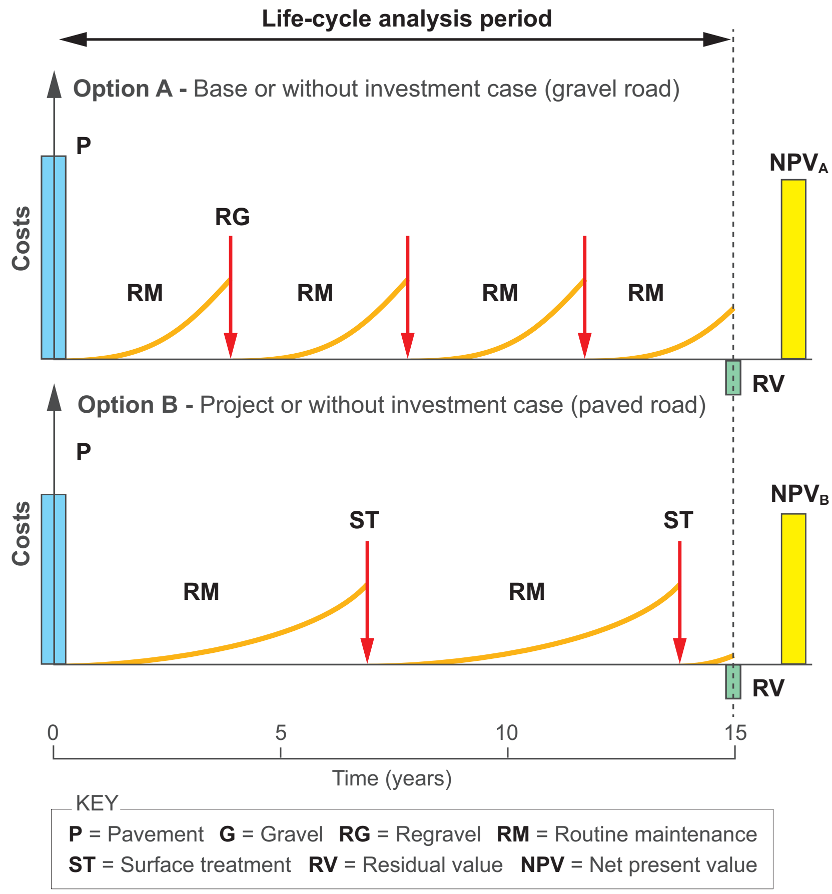



**Figure B.8.8 Typical components of a LCC:- gravel versus paved road**

In very general terms, the upgrading of a gravel road to a paved standard would be economically justified when the Net Present Value (NPV) of the sum of savings in VOCs and maintenance costs, relative to the well-maintained gravel road, is at least as great as the NPV of upgrading costs to paved standard. Where not captured in the investment appraisal model, the inclusion of socio-economic benefits would need to be evaluated separately after the economic appraisal has been carried out. In so doing care must be taken to avoid any inadvertent double-counting of benefits.

## 8.2.6 Selection of surfacing option

An LCC analysis can also be undertaken to determine the most cost-effective type of surfacing to use on a low volume sealed road. Such an analysis entails comparing the construction and maintenance costs of alternative surfacings during the life of the road. The main inputs to the analysis would typically include the:

- assumed service life of surfacing;
- construction cost for surfacing options;
- maintenance cost for surfacing options; and the
- discount rate.

Such an analysis assumes that the vehicle operating costs imposed by the various options are similar, as a result of the minor nature of differences in their associated roughness levels.

Figure B.8.9, Table B.8.1, and Table B.8.2 illustrate the manner of undertaking an LCC analysis for two typical types of bituminous surfacings by comparing the Present Value (PV) of all costs and maintenance interventions that occur during a given analysis period. The example is a hypothetical one used for illustrative purposes only and does not necessarily reflect a real-life situation.

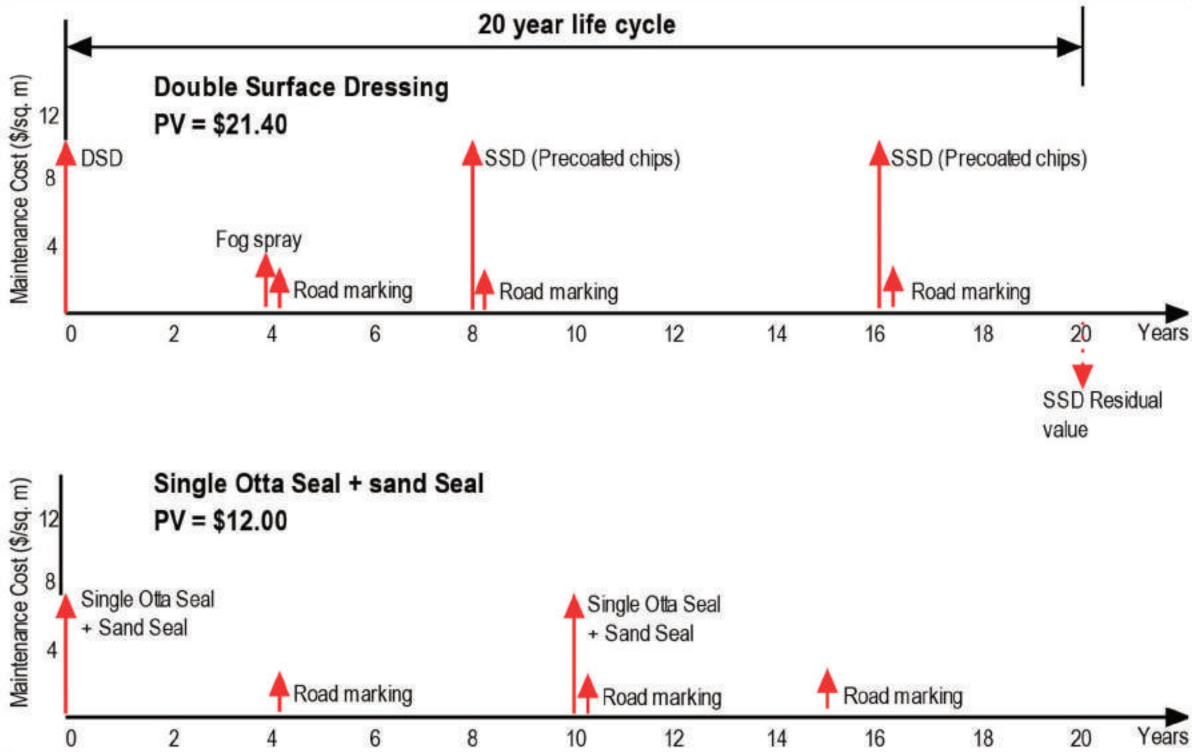



**Figure B.8.9 Cost components of an LCC comparison between an Otta seal & SS with a DSD**

**Table B.8.1 Life cycle cost analysis for Double Surface Dressing**

| Activity | Years after construction | Base Cost/ m² ($) | 8% Discount Factor | PV of costs/ m² ($) |
|---|---|---|---|---|
| 1. Construct Double Chip Seal | - | 10.00 | 1.0000 | 10.00 |
| 2. Fog spray | 4 | 02.00 | 0.7350 | 1.47 |
| 3. Road marking | 4 | 00.96 | 0.7350 | 0.71 |
| 4. Single Chip Seal (pre-coated) | 8 | 10.00 | 0.5403 | 5.40 |
| 5. Road marking | 8 | 00.96 | 0.5403 | 0.52 |
| 6. Fog spray | 12 | 2.00 | 0.3971 | 0.79 |
| 7. Road marking | 12 | 00.96 | 0.3971 | 0.38 |
| 8. Single Chip Seal (pre-coated) | 16 | 10.00 | 0.2919 | 2.92 |
| 9. Road marking | 16 | 00.96 | 0.2919 | 0.28 |
| 10. Residual value of surfacing | 20 | (5.00) | 0.2145 | (1.07) |
| **Total** | | | | **$21.40/m²** |

**Table B.8.2 Life cycle cost analysis for single Otta Seal and Sand Seal**

| Activity | Years after construction | Base Cost ($) | 8% Discount Factor | PV of Costs ($) |
|---|---|---|---|---|
| 1. Construct single Otta Seal + sand Seal | - | 7.25 | 1.00 | 7.25 |
| 2. Road marking | 5 | 0.96 | 0.6806 | 0.65 |
| 3. Single Otta reseal | 10 | 7.25 | 0.4632 | 3.36 |
| 4. Road marking | 10 | 0.96 | 0.4632 | 0.44 |
| 5. Road marking | 15 | 0.96 | 0.3152 | 0.30 |
| Assume life span of 20 years, thus no residual value. | | | | 0.00 |
| **Total** | | | | **$12.00/m$^2$** |

NOTE: It is assumed in the above examples that the underlying pavement structures are identical.

# 9. ROAD CONSTRUCTION

## 9.1 Introduction

### 9.1.1 Background

The construction processes required for a LVR does not, in principle, differ markedly from that adopted for other types of road. However, LVRs are much more sensitive to the social, economic and technical context in which they are built. Variations can be significant with regard to the choice of construction method, type of resources available and type of construction materials being used. This requires the adoption of an appropriate construction strategy which, on some projects, may require providing incentives for the contractor to generate productive employment through the use of well managed labour-based methods of construction.

### 9.1.2 Purpose and scope

The purpose of this Chapter is to highlight the typical range of activities that a contractor will need to consider in undertaking the construction of a LVR. These include the adoption of an appropriate construction strategy as well as appropriate techniques for undertaking earthworks and pavement construction operations using locally available materials. The importance of compaction is highlighted, as are problems that may be encountered in dealing with poor soils, such as expansive, collapsible and dispersive soils. Finally, precautions are identified that should be adopted in undertaking shoulder construction and surfacing of LVRs.

## 9.2 Construction Strategy

### 9.2.1 General

Of particular interest in the construction of LVRs is the utilisation of labour-based and intermediate equipment technology. The objective of this approach is, where it can be justified on technical, economic and practical grounds, to maximise the number of job opportunities per unit of expenditure. This approach involves using a combination of labour and light equipment instead of heavy plant, without compromising the quality of the end product. It optimises the use of labour and employs equipment only for those activities which are difficult for a labour force alone to undertake efficiently and cost-effectively.

### 9.2.2 Labour-based construction

Despite the substantial potential benefits offered by labour-based construction, a number of myths and perceived problems concerning this technology still prevail in the minds of many. These need to be fully understood and addressed if the labour-based approach is to be successfully deployed on low volume projects.

Common myths about labour-based technology include:

- Standards should be lowered to allow for labour-based methods.
- Labour-based construction is out-of-date and incompatible with the modern world.
- Labour-based methods can be used for any construction activity.
- Labour-based construction is only for welfare relief schemes.
- Ill-educated contractors will never understand tender procedures.
- Voluntary labour can be used to keep costs down.

Typical challenges faced include:

- A lack of appreciation that competitive labour-based construction is management-intensive.
- A lack of suitable documentation for the management of labour-based contracts.
- Employers not being open to considering a labour-based approach for new projects.
- Employers not being able to process payment for labour and materials fast enough to keep a labour-based contract operating smoothly.
- Non-availability of suitable skilled and semi-skilled labour in rural communities serviced by LVRs.

**Countermeasures**

Following a major capacity-building programme for small contractors in the 1980s, Ghana has longstanding experience of the practical use of labour-based methods on LVRs. However, this experience typically relates to relatively small contracts awarded by DFR under its Maintenance Performance Budgeting System (MPBS), and for minor works at the community or very local level. As such, the full potential of the technology may not have been realised, particularly in areas of high labour availability. It is therefore necessary to continue the process of raising awareness amongst stakeholders of the benefits labour-based construction. This requires the active refinement and propagation of appropriate strategies by key Ministries of Government, based on established policy directives.

**Practitioner's guide to Rural Roads Improvement and Maintenance**

Developed with technical support from the ILO, this Labour Intensive Public Works Manual was prepared in 2012 by the Ministry of Local Government and Rural Development (MoLGRD) through the National Coordinating Office of Ghana Social Opportunities Project in close collaboration with the Ministry of Roads and Highways, the Ministry of Employment and Labour Relations, and the Ministry of Gender, Children and Social Protection. Though the manual is focused on labour-intensive (maximising labour content) rather than labour-based (optimising such content) approaches, the detailed underlying technical guidance remains valid for both, and draws extensively on experience from DFR and Koforidua Training Centre. As such this guide constitutes a sound reference document for practitioners of labour-based works of any type.

**Suitability of construction activities for labour-based works**

Activities such as site clearance/bush clearing and ditch excavation are well suited to labour-based methods, while other activities such as the bulk compaction of pavement layers or haulage of materials over long distances (typically > 5 km) are not. Some construction activities, for example the handling of heavy precast sections, are not possible without the help of the right machinery. Emphasis is given in this Chapter to those activities than can be effectively undertaken by labour-based methods, supported where appropriate by suitable light equipment.

Labour-based projects usually employ a relatively large number of labourers. In such a situation, the site management staff require to be particularly good "people-managers" with strong managerial as well as technical skills. They need to be familiar with local traditions and social structures in order to avoid disputes on site that could threaten the progress of construction and the sustainability of the project.

**Figure B.9.1 Screening aggregate**

**Figure B.9.2 De-stumping of trees**

## 9.2.3 Equipment-based construction

Some projects, especially large ones, may require heavy plant and equipment for various reasons:

- Large volumes of earthworks may need to be moved.
- Haul distances are long and large quantities of fill and pavement materials may be required.
- Large volumes of materials need to be won from borrow-pits or quarries which have to be excavated and adequately rehabilitated after completion.
- Heavy watering and compaction may be required to achieve specified *in situ* densities.
- Crushing of pavement and surfacing materials, where specified, may be required.
- Large quantities of concrete or asphalt may be required.
- Labour availability may be low.

Generally, the overall size of the project and the quantities of materials to be moved within a fixed construction period are the governing factors when determining whether labour-based or plant-intensive methods are to be used. However, even the largest plant-intensive projects can accommodate many labour-based tasks within the works. The designer should always try to incorporate these into the contract documents, where required, to assist with the government's aim of employment generation, skills development and poverty reduction.

## 9.3 Construction equipment

### 9.3.1 General

The choice of the most appropriate type of equipment for a particular project is normally dependant on the following major factors:

- Site conditions.
- Type of operations.
- Size of the project.
- Soil conditions and material types being used.
- The degree to which manual labour is used in the operation.

Equipment in current use for the construction of LVRs varies from heavy equipment for major highways to the light plant such as tractor-trailer combinations used for labour- based methods. It is often not appropriate to use high-capacity, heavy equipment on LVR sites due to smaller quantities of materials and dimensions of the works. Use of manual labour for major construction operations requires flexible solutions that entail the use of many smaller units of equipment.

### 9.3.2 Equipment used with labour-based methods

Labour-based methods include the use of hand tools for excavation and spreading of material and equipment such as wheelbarrows and animal drawn carts for very short-haul transport. In addition, hand-operated compactors may be used for compaction. These compactors require the use of specific methods to be effective, such as the construction of maximum layer thickness of 75 mm. They are unlikely to be effective in operations where pavement materials require compaction on a large scale. Heavier compaction equipment may also be required for the compaction of pavement layers for sealed roads. Penetration macadam, emulsion treated base and thin reinforced concrete pavements can all be constructed entirely by labour-based methods, whereas densely graded materials can require the use of plant-based methods in order to consistently achieve the required specifications.

**Labour-adapted equipment - tractor units**

Construction units that use agricultural tractors as a power unit provide flexibility in the use of equipment in small units. This approach suits operations where manual labour is a major part of the resource input. The uses of agricultural tractors in key operations include:

- **Loading/transport:** A few tractors can operate many small trailers intermittently, thereby giving labourers sufficient time to load the trailers and maximising the utilisation of the mechanical units. Such trailers usually have a practical height for manual loading. Otherwise, it may be necessary to use the bench method for loading by hand (see Chapter B.10 on Borrow Pit Management).
- **Spreading/shaping:** Towed graders are available in several sizes to carry out these operations, although spreading and shaping can also be done by hand.
- **Watering:** Towed water bowsers are a flexible resource.
- **Mixing on the road:** Towed agricultural disc harrows drawn by a large tractor are very effective at mixing.
- **Compaction**: There are towed versions of vibrating, grid or tamping rollers. Vibrating rollers on labour-based works are often hand controlled self-propelled units that can be used effectively in relatively restricted spaces.
- **Surface reparation:** Towed mechanical brooms can be effective.
- **Bitumen operations:** Towed bitumen sprayers can be used for priming and binder application in conjunction with suitable heating and pumping plant. Emulsions are generally preferred to hot binders on labour-based sites to avoid the need for heating to high temperatures, and to limit associated risks to the workforce.
- **Surfacing aggregate:** Aggregate can be spread by hand from towed trailers; tractors may be used for towing chip spreader units.

Figure B.9.3 Tractor-towed grader

### 9.3.3 Advantages of using tractor units.

Tractor based units have the following advantages over heavy equipment when used by emerging contractors and for operations in remote areas:

- **Plant operation**: Relatively few mechanical items are in use and units are simple to maintain with local mechanical skills. Access to spare parts is relatively easy.
- **Plant availability**: It is often relatively easy to find locally available tractors outside the ploughing season, thereby offering flexibility in fleet management.
- **Utilisation:** Utilisation rates of agricultural tractors are generally higher than for heavy plant.
- **Economic:** There is added potential Income to the local economy outside the ploughing season.

Figure B.9.4 The use of labourers and small tractor-drawn trailers

### 9.3.4 Heavy equipment units

The use of construction units based on conventional equipment, as opposed to tractor-based units, is common in Ghana. Units of this kind typically have the following features in the context of the construction of LVRs in remote areas by small local contractors:

- **Bulldozers for stockpiling:** These have generally been replaced with more economical excavators. Caterpillar D8 or larger bulldozers are difficult to utilise economically where material sources are small, scattered and of variable quality within each borrow pit. Caterpillar D7 or smaller models are normally better suited. Bulldozers require regular preventive maintenance, typically every 250 hours.
- **Front-end loaders:** Front-end loaders come in a variety of sizes. Those mostly used for loading gravel for layer works are the Cat 936/950/966 Loaders or similar. This depends on the size of tipper trucks available. For labour-based construction, a tractor loader backhoe is suitable and can load a 6 cubic metre truck in 5 minutes.
- **Scraper operations:** These are effective where earthworks quantities are large and where material quality is not critical. The control of materials quality is very difficult when using scrapers. The advantage of scrapers is that they can be used for cutting the road way, excavating drains, filling, spreading and to some degree compacting with a single machine. However, motor scrapers typically incur very high investment and operational costs with high requirements for utilisation and mechanical skills for their maintenance. They are expensive to operate and where more flexibility is required tend to be replaced by a combination of other plant.
- **Motor graders** are versatile and are typically used to level tipped heaps, spread gravel, break down oversize material, mix in water, place gravel layers for compaction, cut levels, shape the road prism, shape cut-off berms and cut mitre drains. Most operations carried out by motor graders can be undertaken by labour-based methods. However, on higher trafficked roads, it may be preferable for good riding quality to cut the final levels with a motor grader. This can be carried out as a one-off operation whenever a sufficient length (say 20 km) of base has been placed by hand.

**Figure B.9.5 Motor graders are versatile for processing materials on the road**

- **Excavators:** Large excavators can carry out earthmoving operations of both a bulldozer and a front end loader in the road way and in the borrow pits. This is often an economical option. Selection of material quality is difficult using excavators and such operations can therefore only be used only where material quality in the borrow pit is uniform, or the material can be mixed (e.g. for bulk earthworks).
- **Articulated dump trucks:** These incur high investment and operational costs with stringent requirements for mechanical skills in their maintenance. They can be efficient in high capacity operations and provide both an off-road and an on-road driving capability where the units can legally use public roads. Loading of dump trucks normally requires the use of mechanised plant.
- **Tipper trucks:** Ordinary tipper trucks are often favoured by emerging contractors because they can be used for other transport purposes and are readily available on the second-hand market, generally with readily available spare parts. The skills required for their mechanical maintenance are moderate.

## 9.3.5 Compaction equipment

### Types of plant

In addition to conventional rollers for compaction there are some types of equipment that give particular benefits in the construction of earthworks and pavement layers for LVRs. These include:

- **Grid roller**: This is a static roller towed at a relatively high speed of 15 km/hour for breaking down oversize and 8 km/hour for compaction. In this manner the material is better utilised and problems due to oversize particles are avoided. Good results are generally obtained with the use of this plant for compaction of pavements constructed with natural gravel and of fill layers with lower quality materials which can sometimes be difficult to compact to the full layer depth.

The grid roller allows compaction of the layer to take place in several smaller lifts at the same time as the graders spread the material. The pattern of the surface of the roller ensures that compaction is achieved without forming laminations and shear planes within the layer.

- **Very heavy towed pneumatic rollers**: This type of roller can weigh up to 50 tonnes on a single axle and has been used successfully for compaction and proof rolling of the roadbed, especially in thick single-sized graded sand. Its advantage is in the provision of a uniform and sound foundation for the pavement, achieved by collapsing and densifying any soft areas.

**Figure B.9.6 Grid roller**

**Figure B.9.7 Towed pneumatic roller**

**Impact compactors:** These are non-circular high-energy ‘rollers’, typically three-, four- or five- sided. Large-wheeled tractors are used for pulling the compactors at operational speeds of 12 – 15 km/h producing a series of high amplitude/high impact blows delivered to the soil at a relatively low frequency (90 – 130 blows per minute). The energy per blow varies between 10 and 25 kilojoules, depending on the mass and amplitude of the compactor.


**Figure B.9.8 Three-sided impact compactor**

Due to their very high energy density per blow, the main advantage of impact compactors over conventional compaction plant is their depth effectiveness, typically of the order of one metre of fill or *in situ* layers, thereby producing deep, well-balanced, relatively stiff pavement layers. These rollers are well suited for densifying collapsible soils. They have been successfully used in low-cost road systems and, when appropriately specified and used on relatively large-scale works, offer a cost-effective option for LVR construction.

**Selection of compaction plant**

Figure B.9.9 provides a broad guide to the selection of compaction equipment. Each roller has been positioned in its economic zone of application. The exact positioning of the zones can vary with differing material conditions.

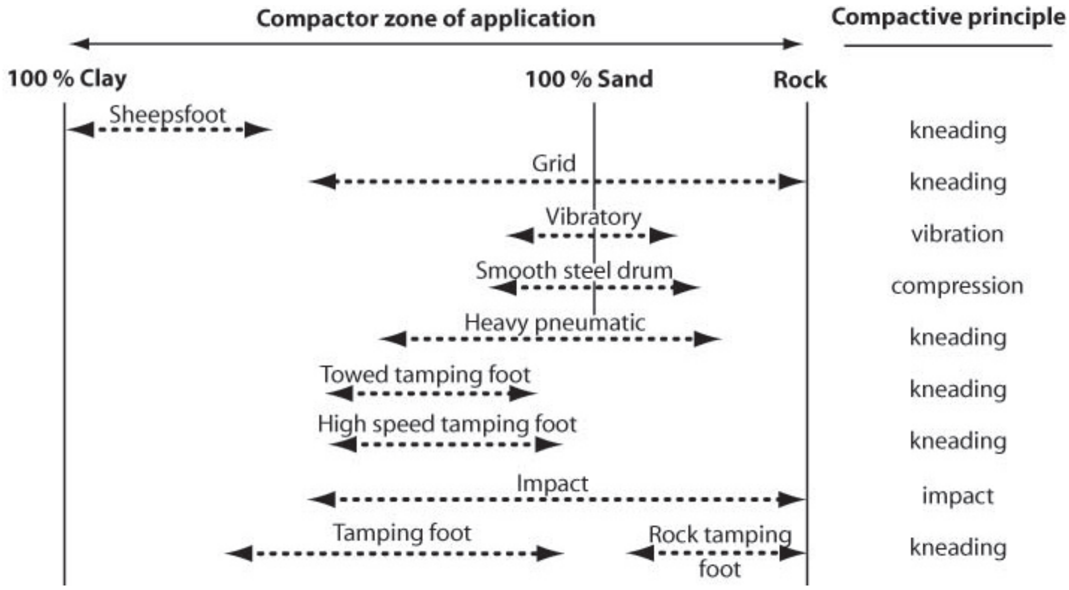



**Figure B.9.9 Compaction equipment guide**

## 9.4 Construction issues

### 9.4.1 General

There are a number of issues that must be dealt with before construction starts. These are varied in nature and may include:

- land acquisition and utilities;
- environmental studies including:
  - temporary environmental issues caused by the construction process itself;
  - overall environmental issues potentially caused by the new road in the longer term;
  - environmental Impact Assessments leading to an Environmental Impact Mitigation Plan and an Environmental Improvement Plan in accordance with Environmental Protection Agency (EPA) requirements; and
  - associated Complementary Interventions, as described in Part A.
- traffic logistics and control during construction; and
- social issues.

All such issues should to some extent have been considered by the contractor during the bid preparation process. Prior to the start of the construction works, the successful contractor should achieve a more thorough understanding of all applicable environmental and social management stipulations. Depending on the scope of the works, these could typically potentially include:

- Activity schedule and list of standard reports and checklists;
- Agenda for environmental management meetings;
- Standard report formats for the Environmental Management Plan;
- Checklist for environmental issues and associated implementation measures;
- Agenda for environmental/social progress report and meetings;
- Standard report format, Environmental Audit Report; and
- Environmental Clearance Certificate.

### 9.4.2 Labour versus equipment based construction

The approach to construction of the works will be in accordance with the Contractor's construction strategy. Although both labour-based and equipment-based methods are feasible for executing the LVR works, there may be a preference for utilising the former approach in economies, such as in Ghana, with generally low wage levels (with the exception of some high wage areas related to mining) and a shortage of foreign exchange. In such an environment, serious consideration should be given to the use of the abundant resource of labour and to limit the use of scarce foreign exchange for imported plant and equipment.

Conditions that favour the use of labour in construction in Ghana are well documented in MoLGRD's 2012 Practitioners' Guide. These include:

- Sufficient numbers of under- or unemployed persons in the areas where the work is required plus local availability of construction materials;
- low wage levels (under US$ 4.00 per day according to World Bank studies);
- Shortage of conventional construction equipment and high capital costs;
- Government commitment to the development of employment and generation of income in the rural areas;
- Small contractors skilled in labour-based technology and capable of managing the work efficiently;
- Competence of the public sector agencies responsible for rural infrastructure works in the areas of contracting and supervision of contractors' performance;

In the following sections various aspects of LVR construction are described in which both labour-based and equipment-based methods of construction are considered.

### 9.4.3 Utilising natural gravels and soils

In areas where natural gravel and soils are available for road building purposes, these materials constitute the most valuable resource in the construction of LVRs. Every effort should therefore be made to use them in an optimal and potentially creative manner. High quality materials should not be used for lower layers, since such materials will be treated only as subgrade for any future upgrading of the road.

### 9.4.4 Materials issues on site

A good appreciation of the properties of natural gravels is required if they are to be used successfully in the construction of LVRs. For example, some of these materials often include some weak larger particles. When such materials are compacted, these larger particles may break down, changing the properties of the material as a whole. An assessment of the consequences of this is required in order to establish whether or not the material adequately meets the specifications following construction.

As is the case with all materials used in LVR pavements, the level of performance is directly related to successful construction methods and workmanship. Aspects of materials utilisation that require particular attention are described in the sub-sections that follow.

### 9.4.5 Materials management

Proper management of the material sources is essential to ensure that the best qualities of available material are used in the top layers of the pavement structure. Efforts made to locate the best quality of locally available (and often scarce) materials for road base are of no avail if, as a result of poor management, this material ends up in earthworks layers. This is far more critical for LVRs than in the construction of more highly trafficked roads, where high quality processed material is used.

Many naturally occurring materials are found in thin seams, so the utmost care is required not to indiscriminately mix different quality materials during their extraction. This issue is addressed in more detail in Chapter B.10.

### 9.4.6 Dealing with variability

Natural gravels are inherently variable. The mixing of two different materials to achieve a quality that exceeds that of the two individual sources is the most common, and probably one of the best methods of improving the engineering properties of natural gravels. For example, mixing fine graded materials with sources that lack fines, such as some volcanic tuffs, can create a material with less potential for breaking down under compaction. The resulting material may have a higher density after compaction due to improved grading, and improved stability and workability.

### 9.4.7 Stockpiling

This forms an important part of materials management by promoting appropriate selection of materials and providing the opportunity to perform testing before transportation of the materials to the road. Some of biggest threats to good materials management arise when borrow pit operations are not kept sufficiently ahead of the construction.

The following sequence of procedures ensures good management of the material resources:

1. Initial investigation of material sources by trial holes;
2. Stockpiles clearly marked;
3. Materials allocated for specific layers on specific sections of the road after stockpiles are completed. Associated laboratory testing should be conducted if possible;
4. Loading from stockpiles according to allocation for transportation to site (avoiding segregation); and
5. Re-testing for suitability of material after breaking down and compaction.

The steps outlined above ensure that acceptance or rejection of materials is carried out at the source, before the material has been transported to the road. However, this requires sufficient plant for opening of borrow pits to avoid the construction demand exceeding material supply from the borrow pits. In cases where opening of borrow pits cannot keep ahead of construction, there is a considerable risk that materials selected for base-course, for example, will end up in the lower layers of the pavement. This causes pressure on material supply when base course materials are needed at a later time.

### 9.4.8 Mixing technique on the road

Motor graders should be used for mixing two types of material on the road. Graders may be used in combination with disc harrows to achieve a homogeneous mix. As illustrated in Figure B.9.10, the method should typically be as follows:

- Dump gravel A on the road in the required quantity then flatten the heaps and spread the gravel over half the width of the layer;
- Dump gravel B on top of spread material A and then spread also over the same half width; and
- Mix the material as normal with the blading of both material A and material B.

**Figure B.9.10 Mixing of materials on road with a motor grader**

## 9.5 Roadbed preparation

### 9.5.1 Clearing, grubbing and topsoil removal

It is particularly important to take account of environmental issues at the early stages of construction so that sensitive operations such as clearing and grubbing are conducted as carefully as possible. Damage to the vegetation cover should be minimised, shifting of soil and associated damage due to erosion avoided, and that any mitigation measures set out in the Environmental Impact Mitigation Plan fully observed.

All topsoil that is stripped should be stockpiled for use in areas that are being reinstated for farming purposes or to promote re-vegetation. Any tree limbs or stumps being removed should be handled and stockpiled in such a manner that the wood can be of benefit to the local community, e.g. as firewood.

**Suitability for labour-based operations:** Clearing and grubbing is suitable for labour-based operations where the required speed of construction and availability of labour makes it possible. Labourers may experience problems in achieving the required result as described in specifications due to the need for ripping, depth of grubbing, size of roots, etc. In such cases it is advisable to review the specifications in the light of the requirements of a LVR and to ascertain whether there is a realistic risk of damage to the pavement due to reduced standards of grubbing.

**Roadbed operations**

After clearing and grubbing, any unsuitable materials (such as clays, black cotton soils, dispersive soils, etc.) should be removed to appropriate depths and the roadbed graded.

The roadbed should then be compacted, either to a specified percentage density or by using a method specification. A method specification usually entails watering the roadbed and applying a specified number of roller-passes to the roadbed at the *in situ* natural moisture content. A trial-section should be prepared and the *in situ* compaction measured after each pass of the roller. This can be done through density tests or by using DCP measurements.

Once the required compaction has been achieved, the same number of passes should be used thereafter for the rest of the roadbed compaction, provided soil conditions are homogeneous. Method specifications are very practical and time-saving and their use should be encouraged where appropriate

If collapsible soils are found beneath the roadbed, it is necessary to pre-collapse them before commencing the earthworks. There are a number of ways of achieving this, but the recommended methods are those that minimise the amount of water required by using heavy impact rollers or heavy vibrating rollers.

**Underlying Hard stratum**

Where there is a hard (rock) stratum below the roadbed drainage problems can arise, particularly in cut areas. The choice of appropriate solution depends on a number of factors including the proximity of the stratum to the finished road level, the thickness/hardness of the stratum, and whether the road is in cut or fill. Remedial measures are:

**Relatively thin ( ≤ 1 m) stratum**. Breaking up the rock layer using jack hammers or by blasting in order to provide a vertical drainage path to an underlying pervious stratum. Providing lined drains to minimise seepage of water under the road pavement.

**Relatively thick ( > 1 m) stratum**: Raising the road embankment and/or providing lined drains to minimise seepage of water under the road pavement.

## 9.6 Compaction

### 9.6.1 General

Compaction is a vital aspect of LVR construction. Good compaction results in all-round improvements of soil properties and in their performance as a pavement supporting layer. A well-compacted subgrade possesses enhanced strength, stiffness and bearing capacity, is more resistant to moisture penetration and less susceptible to differential settlement.

### 9.6.2 Compaction to refusal

When using natural gravels it is important to maximise their strength and increase their stiffness and bearing capacity through effective compaction. This can be achieved, not necessarily by compacting to a pre-determined relative compaction level, as is traditionally done, but by compacting to the highest uniform level of density possible without significant degradation of the particles. This compaction to refusal is illustrated in Figure B.9.11. It results in a significant gain in density, strength and stiffness, the benefits of which generally outweigh the costs of the additional passes of the roller.

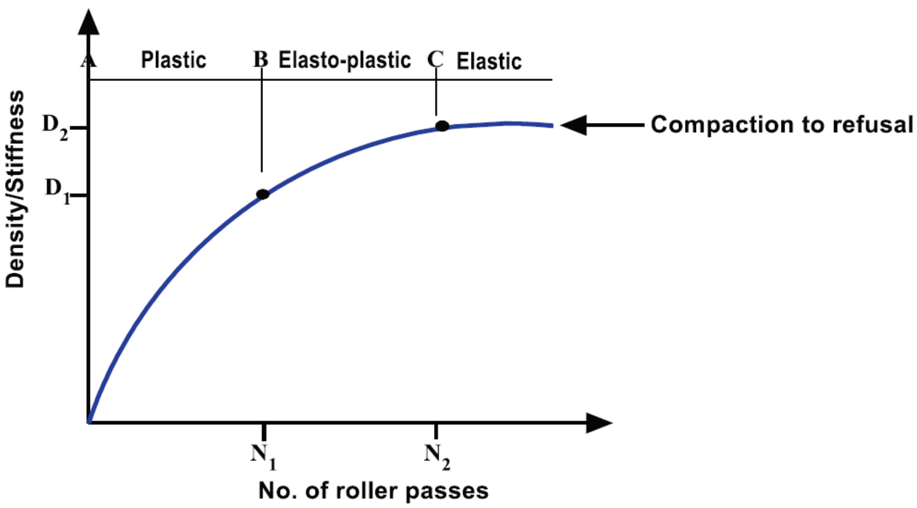



Figure B.9.11 Illustration of concept of compaction to refusal

To ensure optimal performance from the road, a maximum allowable moisture content during construction should be specified and proper precautions taken for surface and sub-surface drainage (where required).

In general, the effectiveness of the compaction process depends on three important, inter-related factors, namely:

- Soil moisture content during compaction;
- Soil type; and
- Type and level of compactive effort.

Different types of soils respond to compactive effort in different ways. It is therefore important to ensure that the compaction plant being used is appropriate for the type of soil being compacted and the purpose intended. For example, sand or sandy soils are most efficiently compacted with high frequency vibrating rollers whereas cohesive soils are most efficiently compacted by static pressure, high amplitude compaction plant. Furthermore, if the requirement is to compact and produce a good riding quality of base course, this is unlikely to be achieved with a very heavy roller that compacts to a great depth and, in the process, disturbs the surface.

### 9.6.3 Subgrade compaction

Effective subgrade compaction is one of the most cost-effective means of improving the structural capacity of pavements. A well compacted subgrade possesses enhanced strength, stiffness and bearing capacity, is more resistant to moisture penetration, and less susceptible to differential settlement. The higher the density, the stronger the subgrade support, the lesser the thickness of the overlying pavement layers and the more economical the pavement structure. There is therefore every benefit to achieving as high a density and related strength as economically possible in the subgrade.

### 9.6.4 Pavement layers compaction

Compaction of the pavement layers (and the subgrade) is specified as a percentage of the maximum dry density (MDD). Such a requirement should desirably be that obtained at “refusal” density, for the reasons described above. To achieve a well-balanced pavement, the compactive effort is increased in each layer as the pavement prism is built, and the quality and therefore the physical strength of material is greater in each ascending layer, thereby making the pavement stiffer at the top than at the bottom. Whatever level of compaction is specified it is important to achieve it on site for each layer by ensuring that the material is appropriately watered and mixed before starting its compaction. The higher the density in the underlying layer, the stiffer and more deeply balanced the pavement structure will be.

### 9.6.5 Moisture for compaction

Thorough mixing of water with soil over the full width and depth of the layer and at the OMC of the admixture is essential for achieving the required density and an even surface finish. The OMC determined in the laboratory is a good guide for determining the amount of water required in the field compaction process, bearing in mind that modern compaction plant normally requires a lower moisture content than the optimum indicated from laboratory compaction methods.

Natural gravels used in LVR pavements often have a high fines content and therefore require much larger amounts of water for compaction than crushed or coarser, well-graded, materials. Effective mixing is therefore of particular importance when utilising these materials. Mixing equipment such as ploughs or large disc harrows (see Figure B.9.12) greatly reduces the required time for mixing water into the material compared to blade mixing with grader, though blade mixing is more effective.


Figure B.9.12 Use of disc harrow for effective mixing of water in material

**Drying-out of layers**

On hot days, natural gravels may need to be brought near to saturation moisture content for efficient compaction. However it is also good practice to allow a significant amount of drying back to occur before sealing takes place. This is particularly beneficial for fine-grained materials that rely on suction and cohesion as their predominant source of shear strength.

### 9.6.6 Water usage, evaporation and temperature variations

Before deciding upon the required moisture content, the *in situ* moisture content must be determined. If evaporation is taken into account, then the water investigations should aim to provide at least 50% more water than the amount required to compact all materials at OMC, particularly in dry areas.

High daytime temperatures can result in an extremely high evaporation rates leading to excessive loss of water. In order to prevent such losses it is prudent to programme watering and mixing activities very early in the morning or late in the afternoon. For LVRs the cost of transporting water may constitute a substantial part of the total cost, so any means of reducing this cost should be considered.

### 9.6.7 Quality attainment

LVR design procedures assume that both the material properties and levels of density specified are achieved in the field. However, in order to attain the specified densities, it is essential to ensure, as far as practicable, the uniform application of water, the uniformity of mixing and uniformity of compaction at or near OMC.

It is also important to note that layers below the one being compacted should be of sufficient density and strength to facilitate effective compaction of the upper layer(s).

Granular materials which are well graded are easier to compact than poorly graded ones. It may therefore be more economical to get the gradation right (e.g. by mechanical stabilisation) before wasting time and energy with excessive rolling. Improved grading is also likely to improve the material strength to an extent where a sub-base quality material could become eminently suitable for road base.

Whilst it is necessary for natural gravels to be brought to OMC for efficient compaction, it is also necessary to ensure that premature sealing does not lock in construction moisture. This can be achieved by allowing a significant amount of drying out to occur before sealing takes place.

The variability of natural gravels is a significant factor in the reliability of performance of the pavement. However, various measures can be taken during construction to reduce such variability. These include:

- careful selection during the winning process;
- processing of stockpiled material; and
- quality control and assurance.

### 9.6.8 Single and multi-layer construction

Single layer construction is usually specified for pavement layers from 75 mm to 200 mm thick. The most common layer thickness is 150 mm. However, when a pavement layer is specified at a thickness greater than 200 mm it will be necessary to compact it in more than one layer of the same thickness in order to achieve uniform specified density throughout. The maximum allowable size of any constituent layer is 2/3 of the total layer thickness.

### 9.6.9 Finishing of base course

If the operation of mixing, spreading and compaction is not completed before drying out of the surface takes place, then a loose upper "biscuit" layer will result. If this happens, the bituminous surfacing will not have a hard surface on which to bond resulting in base course failures due to shearing from wheel loads. Such failures may appear to be the result of insufficient material strength, but studies of construction records, and evidence of good performance under similar conditions in base course layers of poorer material qualities, indicate that premature failure of the uppermost layer of the base course can often be linked to poor finishing of this layer. Careful finishing of the base course layer is vital and decisive for good performance of LVRs.

Figure B.9.13 illustrates a recommended procedure for finishing off base courses made of natural gravel. The advantage of this method is the speeding up of the processing of the base course to prevent drying out of the surface, whilst ensuring that full attention is given to surface finish rather than to minor irregularities of geometric levels. Trimming of the surface should be confined to the action of cutting off gravel to side spoil or offloading it for use in subsequent sections. Attempts to spread loose material over the surface in a thin layer is unacceptable. This is likely to prevent a firm finish of the layer and inhibit the bond with the bituminous surfacing.

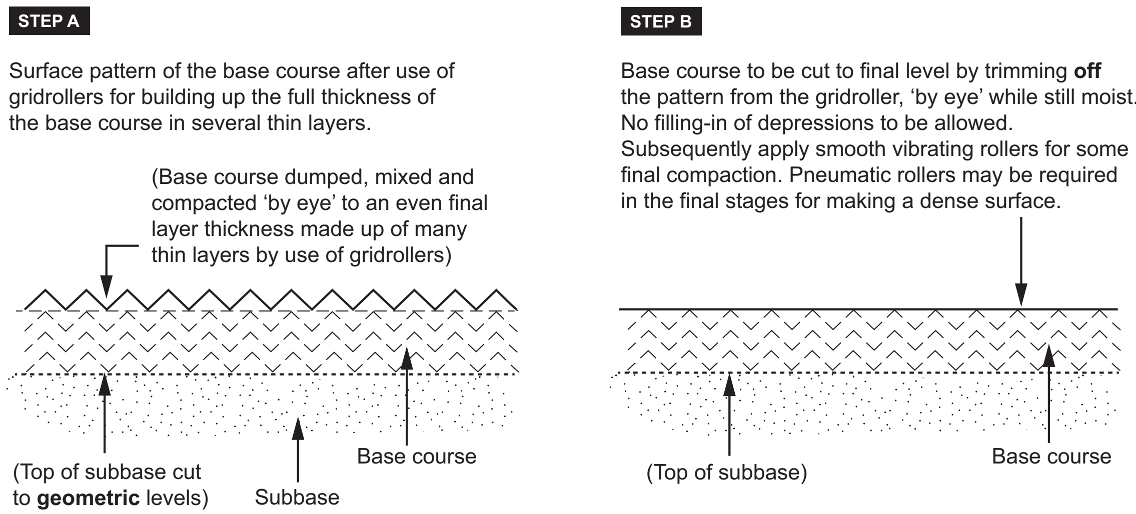



**Figure B 9.13 Procedure to finish off base course constructed from natural gravel**

## 9.7 Surfacing

### 9.7.1 Construction Procedure

All types of sprayed surfacings, such as surface dressings, Otta Seals and sand seals, follow a similar construction procedure, as follows:

1. Priming of the base. This can sometimes be omitted for some surfacings.
2. Base repair (chip & spray by hand using emulsion) to even out the occasional rut caused by a stone under the motor grader blade.
3. Spraying of bituminous binder.
4. Spreading of aggregate. Chip spreading requires uniform aggregate cover; a drag broom can assist this process when required over large areas.
5. Rolling. This is preferably carried out with pneumatic rollers but can also be achieved through trafficking.
6. If applying a double layer, repeat 2 to 5.
7. An emulsion "fog spray" is sometimes applied to chip seals.

In slurry seals and cold mix asphalt, crusher dust or aggregate, bitumen emulsion, water and cement filler are premixed either in a specialised "mix and spread" machine or in a concrete mixer for spreading by hand with squeegees. Mixing by hand may be used but is not encouraged.

Detailed approaches to the design of various types of bituminous and non-bituminous surfacings are presented in Chapter B.8 as well as in the Department of Feeder Roads – Surfacing and Pavement Options for Low-Volume Roads Manual (2007).

### 9.7.2 Need for good construction practice

There are a several potential problems associated with the construction of surface treatments, particularly sprayed seals such as surface dressings and Otta Seals. The majority of instances of poor performance are related to the following:

- Poor transverse distribution of binder;
- Poor joint construction; and
- Over or under spray.

**Figure B.9.14 Verifying transverse distribution of binder**

As illustrated in Figure B.9.16, non-bituminous surfacings (e.g. cobble stones) offer potentially fewer construction problems than bituminous seals because they avoid the use of imported bitumen and relatively expensive construction equipment. In addition, they offer the advantage of promoting and utilising local industry in areas where stone such as granite or limestone may be won and shaped easily by local entrepreneurs, potentially with hand tools. Thus, in a small-scale contractor environment, and in appropriate circumstances, they may offer an attractive alternative to bituminous surfacings. Cobble stone surfaces are however generally much rougher than bituminous seals.

**Figure B.9.15 Cobble Stone paving**

### 9.7.3 Use of Labour-Based Methods

The various types of surfacing provide varying scope for the use of labour-based methods (LBM) in a practical, and economic, manner. Table B.9.1 provides an assessment of the suitability of each surfacing type for the use of manual labour in production of aggregate and construction respectively.

**Table B.9.1 LBM suitability of various surfacing types**

| Activity/Output | | Suitability for labour-based methods (Good – Moderate – Poor) | | | | |
|---|---|---|---|---|---|---|
| | | Surface dressing[1] | Otta seal[2] | Sand seal | Slurry[3] | AC[4] |
| Production of aggregate | Quality | Poor | Good | Good | Good | Poor |
| | Output | Poor | Good | Good | Poor | Poor |
| Construction of surfacing | Quality | Moderate | Good | Good | Good | Poor |
| | Output | Good | Good | Good | Moderate | Poor |

NOTES:
1. Hand-crushing of aggregate for surface dressing tends to produce flaky chippings with some rock types.
2. Oversize and fines can be removed by hand screening of natural gravel aggregate for use with Otta seals.
3. Output of aggregate production for slurry (crusher dust) depends entirely on availability on the commercial market.
4. Although included for comparison with other seal types, AC would not normally be used on a LVR.

As indicated in Table B.9.1, all seal types offer varying scope for labour-based operations, both through production of aggregate and through construction on site. However, the uniform binder spray rates required for surface dressing are more difficult to achieve with labour-based methods. Where labour-based methods are being considered, seal types that are most suited for this type of construction should therefore be given priority. It is also noted that all seals, except the slurry seal, need rolling, and therefore require some form of machine-based equipment for this purpose. Where traffic volumes are sufficiently high, it may be possible to rely on traffic for rolling, though at the risk of an inferior result.

Examples of surfacing-related operations that can be adequately undertaken by labour-based methods are illustrated in Figure B.9.16 and Figure B.9.17.

Figure B.9.16 Screening aggregate by LBM

Figure B.9.17 Spreading gravel using LBM

## 9.7.4 Concrete

There are several elements of LVR construction that require the production of concrete. They include concrete strip and slab surfacings as well as various types of drainage elements, such as head and wing walls, box culverts, etc. The integrity of these elements will depend on the quality of concrete used. Appropriate measures must be observed to ensure that the strength of the concrete complies with the specified requirements.

# 10. BORROW PIT MANAGEMENT

## 10.1 Introduction

The identification and development of good sources of road construction material at regular intervals along the length of a LVR is essential for achieving cost effective construction and ongoing maintenance operations.

Approximately 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 savings and other benefits that can be achieved by implementing improved borrow pit management procedures and material supply strategies.

Proper management of material sources is essential to ensure that the best quality materials available are used in the top layers of the pavement structure. Too often, borrow pit excavation is carried out with only the plant operator present and with inadequate supervision. In many cases this results in good quality gravel becoming contaminated and having to be discarded. Good management of materials (including adequate supervision during all operations in the borrow pit) is therefore a critical operation in LVRs construction.

The purpose of this Chapter is to provide the basic principles of good borrow pit management, including the application of appropriate management strategies with the objective of encouraging cost-effective selection and use of natural resources for LVR construction. In so doing, the Chapter:

- encourages adequate record keeping relating to material resources and their utilisation. In particular, it provides guidelines on the establishment of material resource inventories and databases and promotes the benefits that may result from the use of such databases to improve materials management and design of material supply strategies;
- highlights environmental and social issues associated with road material source development; and
- reviews possible negative impacts and provides guidelines in terms of gravel sources that may be implemented to prevent or reduce adverse effects on local populations and the environment.

An overview is provided of the importance of materials and borrow pit management with a focus on borrow pit planning and preparation, material extraction (both by the use of labour-based and plant-based methods), stockpiling, processing and control. Associated record-keeping and materials data management are also briefly addressed. The Chapter does not deal with the investigations required prior to the establishment of a borrow pit, such as prospecting, resource quantity estimation and borrow pit evaluation. These are addressed in Chapter B.3.

## 10.2 Environmental and social considerations

### 10.2.1 General requirements

The Environmental Protection Agency (EPA) of Ghana has issued guidance for the registration, screening, scoping and implementation of Environmental Impact Assessments for development projects, including projects in the road sector. The establishment of borrow pits require a mandatory Environmental Impact Assessment. The EPA document (EPA 2011) provides guidance on the issues that need to be addressed and the programming of investigations in relation to project phases of pre-feasibility study, feasibility study and preliminary design, detailed design and construction, as part of an Environmental Management Plan. If, for example, borrow pits are to be opened up or extended as part of a road improvement project, then it is necessary to include adequate provision for the assessment and mitigation of adverse impacts as part of the project planning, investigation, design and implementation programme. It is essential to liaise with the EPA throughout this process.

There are no formal specifications for the inclusion of environmental assessment and mitigation for the feeder road network, although outline recommendations for the location of borrow pits are provided in DFR (2004). Furthermore, the Site Supervision Pocketbook (DFR 2005) contains the following recommendations for borrow pit operation and reinstatement, that are based principally on environmental considerations:

The contractor must provide a method statement including how he will open and operate borrow pits. This must take account of environmental issues.

- The contractor must organise a Public Consultation to discuss the location of any borrow pits.
- Do not allow the Contractor to begin to open a borrow pit until after the Public Consultation.
- Do not issue handing over certificates until all borrow pits are reinstated.

Chapter 11 of the Practitioner's Guide to Rural Roads Improvement and Maintenance (MoLGRD 2012) contains some general recommendations for borrow pit restoration with respect topsoil and vegetation reinstatement.

The Standard Specification for Road and Bridge Works (Ministry of Transport (2007) includes comprehensive reference to the management of borrow areas. These should be equally applicable to feeder roads, as a poorly located, operated or restored borrow area along a feeder road will potentially have similar adverse environmental effects.

According to the Ministry of Transport (2007), the Contractor shall ensure that the location of his camps, quarries, borrow pits, stockpile, and spoil areas are not located in, and do not impact in any way on, Environmentally Sensitive Areas as defined in EPA Regulation 30 920 of LI 1652. Such areas include:

- All areas declared by law as national parks, watershed areas, wildlife reserves and sanctuaries including sacred groves;
- Areas with potential tourist value;
- Areas which constitute the habitat of any endangered or threatened species of indigenous wildlife (flora and fauna);
- Areas of unique historic, archaeological, or scientific interests;
- Areas which are traditionally used by cultural communities;
- Areas prone to natural disasters (geological hazards, floods, rainstorms, earthquakes, landslides, etc);
- Areas prone to bushfire;
- Hilly areas with critical slopes;
- Areas classed as prime agricultural lands;
- Recharge areas of aquifers;
- Water bodies characterized by one of any combination of the following conditions:
  i. Water tapped for domestic purposes
  ii. Water within the controlled and/or protected areas
  iii. Water which supports wildlife and fishery activities
- Mangrove areas characterized by one or any combination of the following conditions:
  iv. Areas with pristine and dense growth
  v. Areas adjoining the mouth of major rivers systems
  vi. Areas near or adjacent to traditional fishing grounds
  vii. Areas which act as natural buffers against shore erosion, strong winds or storm floods.

The Engineer's approval may be withheld if the quarry, borrow pit, spoil or stockpile area or access into them:

- Is less than 3 km from the next quarry, borrow pit, spoil or stockpile area;
- Will have a significant visual impact, or will likely result in excessive erosion, sedimentation, slope instability or other environmental impact that will be difficult to control, or has been excluded in the Environmental Impact Assessment and Environmental Management Plan;
- Will incur relatively high land acquisition costs or would be very difficult to acquire;
- Is in or near an urban centre;
- Will require an access road which is excessively long;
- Has excessively thick layers of overburden;
- Covers too large an area;
- Would constitute a danger to the public;
- Is an excessive distance from the location where the material is to be used;
- Is less cost-effective to use than a source of suitable material closer at hand;
- Is located within the road reserve;
- Is located in or near a village (where near is defined in the Special Specification or by the local stakeholders);
- Is located within 200 m of a drinking water intake (stream or water pump);
- Is in or adjacent to a Sensitive Area as defined by the EPA; or
- Cannot be rehabilitated in accordance with the requirements of the Environmental Management Plan.

The Contractor will need to provide the following documentation prior to obtaining the necessary approvals:

- A plan at 1:250 scale in ink on a stable transparent material or a paper and electronic copy giving details of:
  i. plot boundaries;
  ii. owners' names and addresses, and, if appropriate, ID numbers;
  iii. the District, Location, Registration Section and Number for each plot;
  iv. local details such as buildings, fences, graves, types and areas of cultivation and services, all agreed with the land owners;
  v. areas to be used for working areas, stockpile areas, blasting safety zones etc.
- Cadastral maps covering the areas to be acquired;
- Details of the proposed access road route;
- Results of investigation and laboratory tests
- Documentation confirming acquisition of right of way where necessary;
- Details of measures for controlling runoff and sediment from the site during operations; and
- A plan for site rehabilitation as detailed in the Environmental Management Plan.

Additional clauses are provided concerning entry upon land, access roads, obtaining borrow materials, opening and working borrow pits and haul roads, removing topsoil, clearing and grubbing, excess overburden, excavating borrow material, control at borrow pits, protecting borrow pits, stockpiling the material, site clearance, reinstatement, including drainage, shaping, removal of rubbish and landscaping. Provisions are also made concerning community reuse of borrow pits once worked out, and the manner in which borrow pits should be returned to the landscape through drainage, reforming and topsoil replacement.

The development of borrow pits and quarries to supply road construction material may impose significant negative impacts on the local environment and its inhabitants. Appropriate precautions must therefore be taken in the planning, development and operation of all borrow pits. There is a need to be aware of both the potential negative and positive aspects that may be associated with different types of borrow pit or quarry development. This is particularly relevant when there may be a choice of developing one or other type of resource.

Environmental impacts may be associated with extraction of the following main types of road building materials:

- River bed gravels;
- Near-surface natural gravels (laterites, residual and transported granular soils);
- Alluvial terrace deposits;
- Hill-slope material (weathered and/or closely fractured rocks); and
- Hard rock.

### 10.2.2 River bed gravel borrow pits

The development of river bed gravel sources is a sensitive environmental issue. Negative impacts are typically associated with over-exploitation and careless extraction. The intermittent extraction of small quantities of sand and gravel from a large dry river bed is probably the least damaging form of material supply. This is because no productive land is lost and the deposits will be replaced during future high-water flows. Problems arise when the quantities of material extracted greatly exceed nature's ability replenish the supply or where the extraction either alters the flow pattern of the river or removes protection to engineering structures such as bridge piers and river training works. Sand and gravel should never be extracted at a distance of less than 300 m downstream of bridges. If there is any doubt about acceptable excavation volumes or potential impacts of extraction on nearby structures or the downstream environment, then alternative sources should be found. Expert advice should always be obtained in any case.

River gravels are typically non-cohesive and rounded and may be prone to ravelling if used as wearing course gravel. These materials are usually most suitable for forming embankments and for use as sub-base.

### 10.2.3 Near-surface natural gravel borrow pits

LVR construction may rely heavily on winning construction materials from relatively thin and discontinuous, near-surface gravel deposits (see Figure B.10.1). As illustrated in Figure B.10.2, these deposits include lateritic materials, residual quartzite gravels and alluvial gravels. Easily extracted deposits close to existing unpaved roads can in many area become rapidly exhausted, increasing pressure to exploit marginal quality deposits close to the road, or better-quality materials at further distance from the road.

**Figure B.10.1 Shallow surface deposit extraction results in high land take – volume ratios**

**Figure B.10.2 Typical lateritic and quartzite gravels found at or close to the ground surface**

Working thin deposits involves a poor ratio between land take and resource quantity. This can become environmentally unacceptable in the following situations:

- In populated and cultivated areas, where pit development may result in permanent loss or down grading of productive land;
- In areas of landscape value and in habitats of conservation value; and
- In areas where topsoil is thin and cannot be salvaged to enable adequate pit reinstatement and prevention of soil erosion.

In areas of high erosion potential, consideration must be given not only to initial economics of extraction, but also to long-term economic and environmental consequences. Hauling material longer distances from pits with less adverse environmental impact should to be considered.

### 10.2.4 Hillslope borrow pits

Borrow pits developed in mountainous and hilly terrain can have significant damaging effects on the local environment if they are not carefully located and operated in an environmentally sensitive way. Most hillside borrow pits exploit weathered and fractured rock materials. Topsoil is usually thin and stony and as a result difficult to salvage and replace. Excavation of natural gravel from steep slopes can cause serious slope stability hazards that may endanger the workforce, road users and people living downslope. Slope failures on valley sides can result in heavy sediment pollution of rivers. Carefully constructed benched excavations are usually required.

In Ghana, borrow areas in weathered rock are not only confined to hilly areas;  weathered rock can be found close to the surface in gently undulating and plateau areas and provide sources for construction materials, though this is often poor quality. These materials are, by definition, weathered and therefore weaker and much more easily broken down.

On some hill roads there is a desire to open a large number of small pits (less than 3,000 $m^3$) at regular intervals. This can be very destructive in the short and long term. It is generally better to identify a few well located borrow sites with relatively large potential resource volumes at larger intervals. In rolling or hilly terrain, the benefit of short haul distances during construction should not be a factor that overrides environmental considerations.

If borrow pits are to be opened adjacent to the road, efforts should be made to locate sites where extraction will improve the road alignment, for example material from a spur or blind spot in the road.

### 10.2.5 Hard rock quarries

Environmental considerations are particularly important at the planning stage when construction materials need to be obtained from hard rock quarry sources. The following guidelines apply to quarry planning:

- Hard rock deposits rarely occur in isolation. It is therefore necessary to check the geological maps and consider environmental effects when looking for the best site for development;
- Quarry sites should be located as far away from settlements as possible. Quarry operations will produce noise, vibrations and dust that could impact on nearby residents even if controls are imposed;
- Steep quarry faces are a hazard to people and livestock, so fencing and site security measures are essential;
- The arrangement of the quarry operations should be designed to cause least possible visual impact on the landscape and to allow for future reinstatement. Natural vegetation (trees and bushes) should be preserved around the quarry;
- Quarry site development costs are high and negative environmental impacts potentially significant. Quarry sites should therefore be fairly widely spaced and located so that there is the potential to supply large quantities of material over a long period of time to various sites. When natural gravel deposits are not available, then the haulage of aggregates from hard rock quarries is usually economically and environmentally justified for distances of greater than 20 km.
- Crushing and screening plant and stockpile areas need not be located directly adjacent to the actual quarry excavations. Visual intrusion may be significantly limited, at no great additional cost, by processing aggregates at a more concealed location a short distance from the outcrop.

### 10.2.6 Borrow pit location

Borrow pits and quarries should be located in such a way that they cause a minimum of environmental damage and impact on the local environment. Typically, the following guidelines apply, subject to the overriding requirements of the EPA stipulations and guidelines:

- Borrow pits should not be within 500 m of a watercourse or human habitation;
- If possible, pits should be on land that is not suitable or currently used for cultivation and is not forested;
- Areas of local historical or cultural interest should be avoided and borrow pits should not be located within 25 m of grave sites;
- Wherever possible, borrow pits should not be visible from the road. Development should be designed to minimise visible damage to the landscape;
- There should only be one agreed access road to each borrow pit site;
- Borrow areas should not be on steeply sloping ground if it can be avoided; and
- Borrow pits should have minimum overburden material.

### 10.2.7 Land take

Land used for material sources should always be minimised as far as possible and fair compensation should be paid to the owner. This applies equally to both permanent and temporary land take. No land should ever be used without formal authorisation.

### 10.2.8 Borrow pit working plan

Borrow pits should never be opened in an uncontrolled manner. A proper working plan must be in place before any excavation begins. The plan must include:

- Arrangements for consultation with affected parties;
- A compensation agreement signed with the land user/owner, including agreed borrow pit access arrangements;

- Clear demarcation on the ground of the extent of each borrow pit/quarry, and possible future extension;
- An outline of the direction, timing and depth of working area, including suitable locations for stockpiling the topsoil and overburden materials;
- Provision for adequate drainage and implementation measures for sediment control, topsoil replacement and re-vegetation;
- Provisions for safety of workers and nearby residents at and around the borrow pit area; and
- A reinstatement plan which gives details of the final shape of the borrow pit and the method of achieving it, including possible Complementary Intervention plans (see Part A) for the affected local residents.

### 10.2.9 Borrow pit operation

Borrow pit operation should commence with vegetation clearance and stockpiling of topsoil for eventual reinstatement. Topsoil is the organic soil typically occurring as a surface layer, normally 150 to 200 mm thick. The future productivity of the restored land is totally dependent on careful replacement of the topsoil layer. Failure to adequately return topsoil materials will have a long-term damaging effect on the environment and on the future ability of the land user/owner to earn a living from their land. Any overburden soil (soil that rests on the gravel deposit and under the topsoil) should be stockpiled separately and overburden stockpiles located where they will not interfere with future pit extensions. They should be shaped in order to best resist the erosive actions of rainfall.

Restoration of land used for borrow pits and quarries should be considered from the start of excavation. The borrow pit layout should be designed to enable easy reinstatement. Unnecessarily high and steep slope faces should be avoided both to reduce reinstatement problems and to minimise danger to people and livestock. Reduction in the visibility of an excavation can often be achieved by identifying the best orientation of working faces in relation to receptors. However, despite the environmental desirability of a particular direction of working, the geological structure may determine the safest and most effective method of excavation and this will sometimes be the more important factor.

The extraction and processing of pit or quarry materials can have several adverse effects on the local environment. The most significant of these is the creation of noise, airborne dust and the pollution of water courses, as follows:

- Noise may be generated by the excavation process. It is important to limit noise as far as possible both for the local residents and the workforce who should be provided with ear protectors;
- Dust generated during the extraction of materials can be a health hazard causing respiratory diseases, it can cause of accidents in the pit and can inhibit the growth of plants. Care should therefore be taken to minimise dust emissions. The workforce should be provided with dust filter masks. The main sources of dust and appropriate methods of reducing emissions are:
  i. Drilling - dust suppressers can be fitted to drilling rigs
  ii. Movement of traffic - dusty access/haul roads should be watered regularly during dry weather using bowsers (water trucks)
  iii. Dumping of dry aggregates in stockpiles - material processing plants should be fitted with water sprays when stockpiling. This will not only suppress dust but also prevent aggregate segregation;
- Water course pollution may be associated with sediment entering streams from the borrow pit excavation. This can be prevented by constructing bunds to divert surface water away from the excavation and by ensuring that any water leaving the excavation passes through a settlement pond. Any re-fuelling or other plant maintenance activities carried out in the borrow pit should be controlled to avoid spills and water contamination. Any accidental spills should be cleared up immediately and disposed of safely;
- Safety aspects such as access to borrow pits and quarries with steep and potentially dangerous working faces must be controlled to avoid accidents involving local people and livestock. This may require construction of fences with warning notices and the posting of guards. For the safety of the work force, dangerous loose faces should be made stable. Workers 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. Special care must be taken when blasting takes place in quarries. Quarries should not be located close to settlements. Only suitably trained and qualified staff should handle explosives. Storage of explosives must comply with internationally recognised standards of practice in terms of security and safety.

### 10.2.10 Borrow pit / quarry reinstatement

During the planning phase (before appointment of a contractor), the possible use of the borrow pit or quarry once extraction is complete should be discussed with the local community. For instance, many communities like to use these areas as collectors of water for both human use and animal consumption.

Further consideration on the community management of water ponding areas in the road corridor is provided in Chapter A.3 (Complementary Interventions) and in EMU (2011). Whether the borrow areas should be fenced or not needs to be discussed and agreed, as the stored water can be hazardous, particularly for children, on account of the risk of drowning. Ponded water (Figure B.10.3) also increases the potential for mosquito-borne diseases. Before any use by the community, the borrow pit must be cleaned up properly and shaped according to its intended use. The work required must be adequately described in the tender documents by way of a Bill item so that the contractor can price the work accordingly.

**Figure B.10.3 Localised ponding in undrained low point of a borrow pit**

Before a borrow pit or quarry Is reinstated, the need for materials in future road maintenance should be assessed and appropriate quantities of gravel stockpiled for this purpose.

The environmental damage caused by inappropriate extraction and rehabilitation practices can extend over a wide area and may only become apparent after project completion. Hence, the borrow pit or quarry preparation work must be carried out carefully and in such a way that the topsoil is stockpiled separately from overburden soils and shaped in such a way as to minimise any loss by erosion, wind and rainwater. Reinstatement should follow simple procedures which minimize the limitations the construction activity has placed on future use of the land. This entails:

- shaping mounds and steep banks to a slope that is found naturally in the landscape (the steepest should be 1V:3H);
- spreading the topsoil evenly back into the pit in order to promote growth of vegetation; and
- ensuring that the area is self–draining.

## 10.3 Borrow Pit Preparation

### 10.3.1 Introduction

The preparation and arrangements that are typically necessary prior to material excavation, including administrative/planning activities and physical site preparations, are often neglected, especially for LVRs. Borrow pits for the supply of materials for LVRs are usually not in continuous use. In the case of gravel wearing course sources, regravelling operations will typically recur every 3-5 years and paved roads will usually require rehabilitation after 10 years. This should be taken into account during planning. The area stripped should only be large enough to provide aggregate for the immediate needs, with some stockpiling of maintenance materials.

### 10.3.2 Site survey

In the case of small borrow pits in remote uncultivated areas, it may be sufficient to prepare pit sketches with important dimensions determined by tape measure or coordinates using a handheld GPS. When a borrow pit is likely to affect cultivated land and local inhabitants then an accurate site survey should usually be made. This includes determining the locations of proximal dwelling and plots of cultivated land and marking them up on a map prepared at a scale of about 1:500 to 1:1000 depending on the size of the proposed workings and nature of the site.

### 10.3.3 Borrow pit working plan

Arrangements should be made for consultation with affected parties and a compensation agreement reached with the users/owners of the land. Access and working arrangements should also be agreed. Borrow pits must be opened and operated in a controlled manner, including:

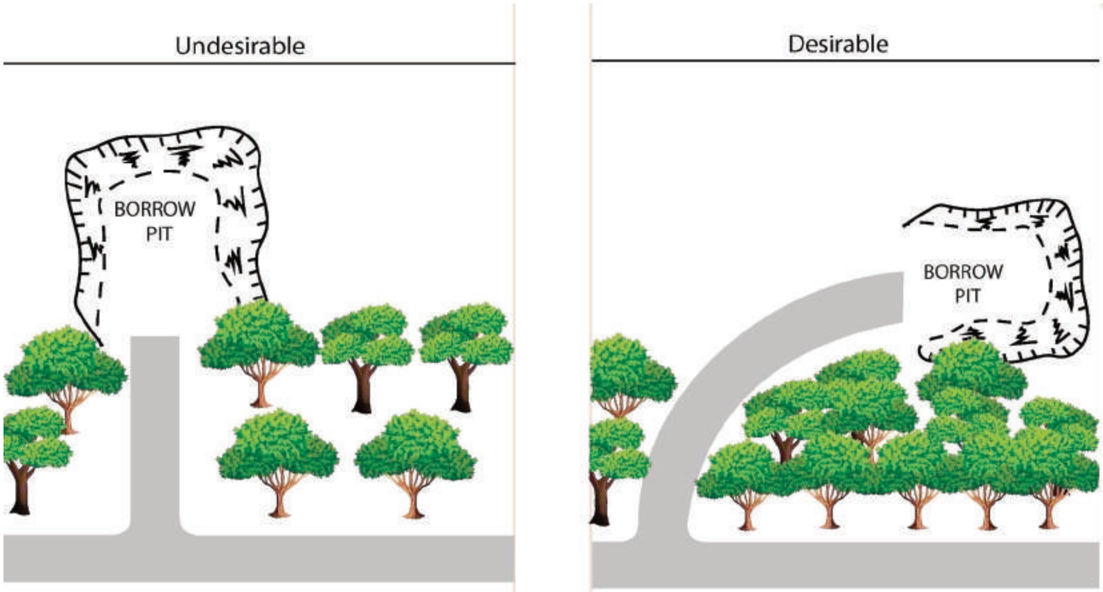
- undertaking a survey for record purposes to define agreed limits of working area and time;
- defining the direction, timing and depth of working; and
- ensuring that borrow pits are excavated to a regular width and shape, with all existing trees, hedges, fences preserved if possible.

### 10.3.4 Access roads

The following recommendations relate to the provision of access roads to borrow pits:

- Access roads should be designed to be strong enough to carry the expected haulage traffic without significant deformation or maintenance. Economies of construction may easily be outweighed by increased haulage costs.
- Adequate provision should be made for drainage in order to prevent soil erosion, sediment pollution or road closure due to flooding.

Borrow pit access roads should be aligned so as to minimise disturbance to the local population and the environment. They should be located at a safe distance from permanent dwellings. If necessary, fencing should be provided to protect local people and livestock, and prevent them from entering. Figure B.10.4 shows a sketch of undesirable and desirable access road layouts.




*SOURCE: RI (2000)*

**Figure B.10.4 Undesirable and desirable access road layout**

### 10.3.5 Site clearance

Great care needs to be taken during the site clearance operations to expose gravel or fractured rock. Otherwise effective pit reinstatement may not be possible and significant environmental damage may result. The following are examples of bad practice:

- Bush clearing carried out by burning, prior to topsoil stripping. This practice removes organic matter and kills useful bacteria in the soil that assist in producing the required soil nutrients. If uncontrolled, extensive damage due to fire may affect the surrounding countryside.

- The removal of topsoil and residual soil is carried out as one operation. This results in complete destruction of the fragile topsoil layer;
- In mountainous terrain, disposal of overburden soils is achieved by side-tipping downslope. This removes vegetation cover and topsoil and frequently leads to erosion and slope instability.

**Removal of vegetation**

Vegetation removal should be confined to the area of the proposed borrow pit and the operational areas. Unnecessary removal of vegetation should be avoided. During site clearance any shrubs that would be suitable for any later use should be identified and protected. Transplanting or taking cuttings of the shrubs will preserve them for future replanting.

**Removal of topsoil, subsoil and other unsuitable material**

Guidelines on appropriate procedures for topsoil removal, overburden soil stripping and pit reinstatement have been prepared by TRL (1999). Figure B.10.5 shows the recommended procedure diagrammatically. The process includes careful removal of topsoil and its stockpiling for subsequent reinstatement of the borrow pit as supplementation from other sources is normally not possible nor environmentally friendly. This is followed by extraction of identified sub-soil layers separately into stockpiles followed by borrow pit extraction.

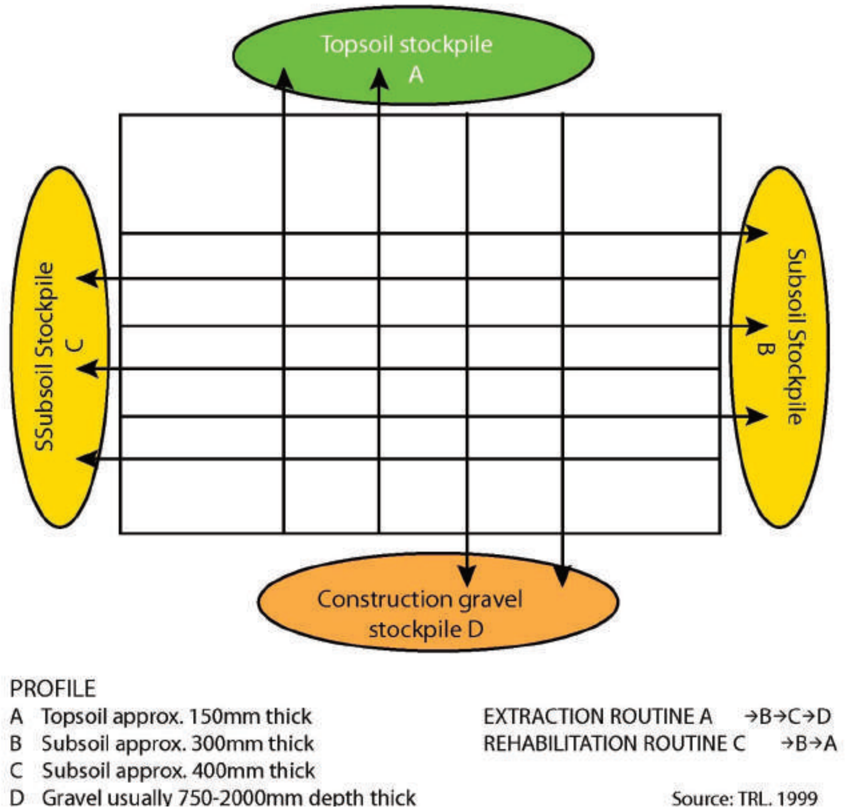



**Figure B.10.5 Recommended procedure for removal of overburden and stockpiling**

The average thickness of overburden soils should be accurately known from site investigations and must be shown in the borrow pit plans. The overburden should be stockpiled in such a way that it does not interfere with the drainage of the adjacent land area. Where possible the overburden material should be used for construction works such as embankment fill. In mountainous terrain where there is limited space for storage of overburden materials, the soil should be hauled to a suitable disposal site or to a stockpile area located in stable terrain.

## 10.3.6 Layout of shallow borrow pits

Whether the borrow pit is operated by labour-based methods or machine-based methods, the working of relatively thin near-surface deposits involves a poor ratio between land take and resource size. Attention given to borrow pit layout and the method of borrow pit operation can help to reduce the potential for this to create significant adverse effects on the environment.

**Shallow gravel borrow pits using labour-based methods**

The layout of a borrow pit exploiting near-surface deposits is strongly influenced by the associated choice of technology. The following considerations should be taken into account when planning borrow pit development using labour-based methods:

- The optimum height of a face to be worked with a pick is about 700 mm. The borrow pit layout should avoid unnecessary double handling of excavated material. When possible, excavation bays should be about 3.5 m wide so that trailers/trucks can be backed in for loading.
- Sufficient space should be provided to allow tractors and trailers or trucks to manoeuvre in and out of loading positions without difficulty.
- It may be desirable to have both access and exit routes into the working area, while still maintaining a single access road into the borrow area from the main road.
- The borrow pit layout should, as far as possible, be self-draining and not subject to the accumulation of water and the development of soil erosion problems.
- In all borrow pits, emphasis should be given on allowing easy loading of the material and ensure the safety of the workers.

**Shallow gravel borrow pits using machine-based methods**

When mechanised extraction methods are adopted the main influences on the borrow pit layout may be somewhat different from using labour-based methods, as follows:

- The working face should be arranged in such a way that the excavation plant can operate efficiently. For example, bulldozers work best down a slightly inclined face, while backhoes usually operate most efficiently in a near vertical face several metres high (subject to face stability).
- The need for mixing (blending) or to avoid mixing of different soil layers, may influence both borrow pit layout and the selection of appropriate excavation plant.
- Stockpiling of excavated materials is more readily accomplished using plant-based methods.
- Careful consideration needs to be given to the location of stockpile areas. They should not interfere with future development or extension of the pit and need to be arranged so that there is sufficient space for the efficient operation of loading plant and trucks.
- If processing of the excavated materials is required (such as when blending material from two different stockpiles) careful consideration must be given to the area required for this as such operations will require considerable space.
- Fencing is required to prevent the local residents and livestock from entering dangerous parts of the pit and to reduce the risk of accidents with operating plant.

## 10.4 Borrow pit material extraction using labour-based methods

### 10.4.1 Geological considerations

Efficient labour-based material extraction and supply is generally only viable when:

- Un-cemented gravels occur beneath a relatively thin overburden cover.
- Exploitable deposits occur at frequent intervals close to the road, so suit the use of tractor-drawn trailers that can readily be loaded by hand. When haul distances exceed 5-10 km tippers become more cost-effective for haulage, but in most circumstances are not well suited to being loaded by hand.
- Labour-based methods are most successful in areas where:
  i. There is a widespread occurrence of near-surface lateritic gravel or weakly-cemented laterite deposits;
  ii. There are frequent exploitable river bed or river terrace gravel deposits.

### 10.4.2 Environmental considerations

Labour-based methods are sometimes used in steep terrain where there are roadside occurrences of suitably fractured rock. However, as detailed in Section B.10.2.3, the development of frequent small borrow pits in terrain which is prone to soil erosion and slope instability is not good practice. Large borrow pits and longer haulage distances then favour mechanised extraction and loading and tipper truck haulage.

### 10.4.3 Resources and work methods

Labourers working in a borrow pit require picks, crowbars, hoes, shovels and sledge hammers for effective excavation. In addition, they should be provided with drinking water, first aid facilities and head, hand and foot protection. Careful planning and preparation of borrow pit operations is particularly important in the case of material extraction using labour-based methods. The plan should indicate some of the most important aspects of work to be considered when excavating and stockpiling in the pit. The number of labourers deployed will depend on the following:

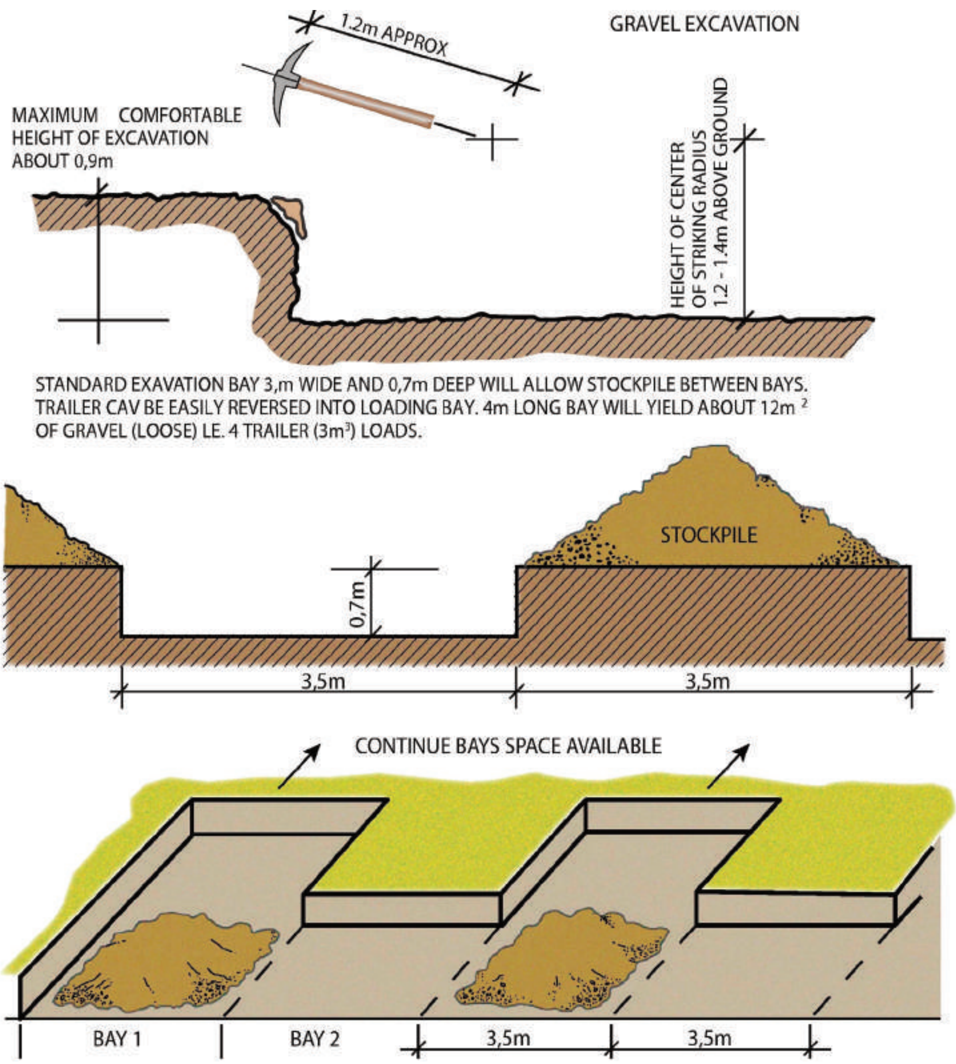
- Size of the borrow pit site and quantity of overburden to be cleared. Large borrow pit operations may require between 40 and 60 labourers, both for the site preparation and the subsequent excavation.
- Availability and capacity of the equipment used for hauling.
- Productivity rates, which will be influenced by the hardness of the material to be excavated.

### 10.4.4 Gravel excavation and stockpiling

During gravel excavation and stockpiling activities, the following issues should be considered in order to optimise the borrow pit operations:

- Gravel should be excavated, stockpiled and confirmed for use (through quality control testing) before it is required to be hauled. Usually, seven days are required to carry out the complete set of quality control tests.
- Gravel should be excavated and stockpiled alongside the loading bays to allow easy loading and avoid unnecessary repeat handling.
- Where possible, loading bays should be constructed to allow trailers to be backed in for loading.
- Ramps into the loading bays must not be too steep for tractors hauling loaded trailers.

Figure B.10.6 shows details of a borrow pit development on flat ground.




*SOURCE: RI (2000)*

**Figure B.10.6 Borrow pit development on flat land**

In hillside borrow pits, material should be excavated such that loading is made easier and to ensure that working conditions are safe. It is also important to ensure that the labourers have sufficient space to work comfortably and safely. Figure B.10.7 shows a schematic arrangement for the progress of excavation in sloping ground. The numbers indicate the sequence of excavation to be adopted.

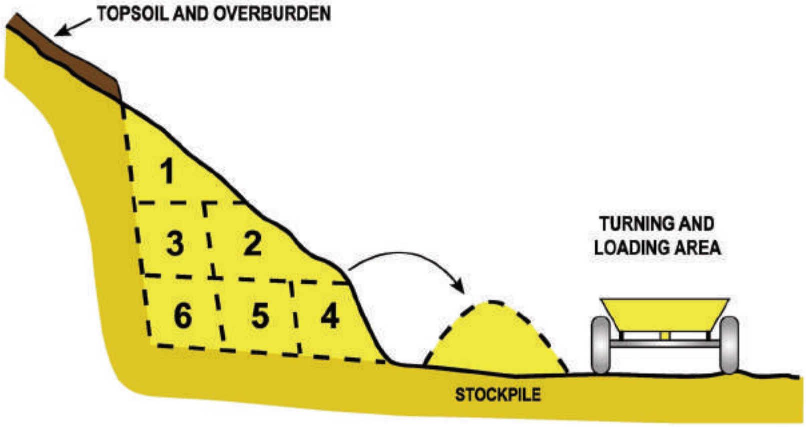
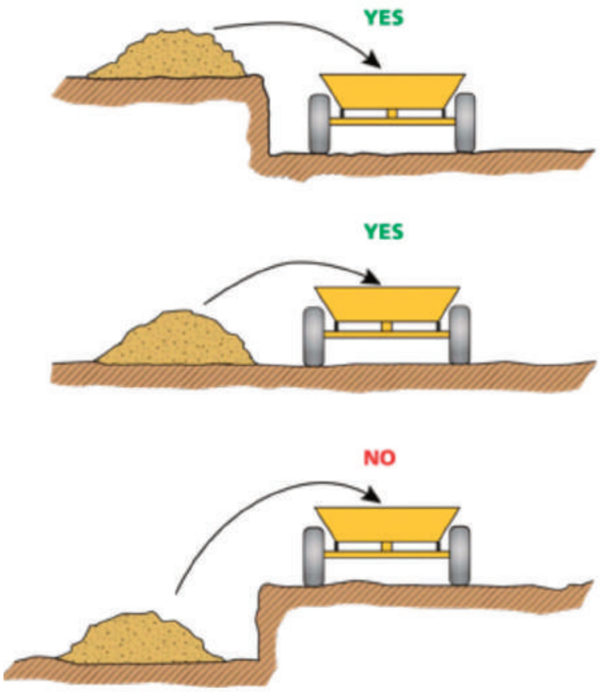



**Figure B.10.7 Borrow pit development on sloping ground**

Where possible, trailers should be parked at the same height as, or preferably below, stockpiles for ease of loading. The loading gang should be divided into groups of 4 to 6 workers and these groups load the empty trailers in the same order as they arrive in the borrow pit. All of the trailers must be loaded to the correct load line (capacity) to facilitate uniform dumping of material. Figure B.10.8 shows details of the trailer loading height.



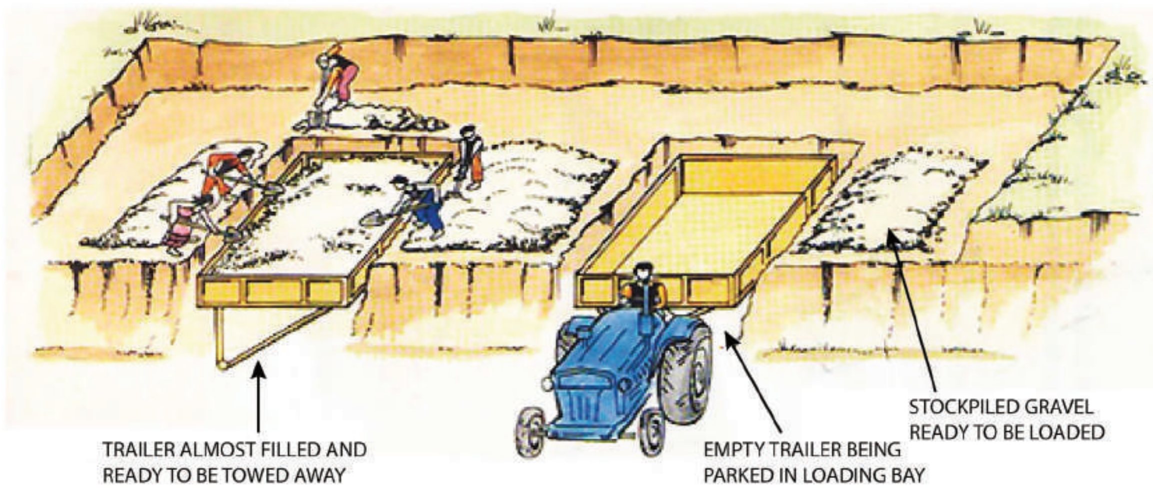

SOURCE: RI (2000)

**Figure B.10.8 Trailer loading height**

Dumping and spreading of the material should commence from where the borrow pit access joins the road to be regravelled/constructed. Initially the road should be gravelled/constructed away from the borrow pit access in both directions simultaneously. With short hauls this reduces congestion at the loading sites. When hauls exceed about 1 km, gravelling/construction should continue only in one direction at a time. The advantages of this are that:

- the tractors and trailers compact the material as they haul over the laid material;
- damage to un-gravelled road formation is minimised;
- gravelling does not interfere with reshaping activities;
- gravelling can recommence sooner after rainfall; and
- one tractor can work with multiple trailers to minimise standing time.

Figure B.10.9 shows an ideal borrow pit arrangement using labour-based methods during trailer loading. For sealed road construction without diversion roads, it may be necessary to start construction of the base away from the borrow sources, back towards the sources, to minimise damage caused by haul vehicles on the newly constructed base.




*SOURCE: RI (2000)*

**Figure B.10.9 Borrow pit arrangement using labour-based methods**

## 10.5 Borrow pit material extraction using mechanised plant methods

### 10.5.1 Introduction

The economic extraction and production of borrow pit materials for construction purposes using plant depends on the appropriate selection of equipment for the project and the careful programming of the works. Mechanised plant used for borrow pit excavation may only be suitable for larger projects or when the haul distances are long.

### 10.5.2 Excavation planning and plant selection

The types of plant suitable for heavy, medium and light excavation work in gravel and rock are summarised in Table B.10.1.

**Table B.10.1 Suitability of plant for gravel/rock extraction**

| Activity | Type of excavation | | | Comments |
|---|---|---|---|---|
| | Heavy | Medium | Light | |
| Drill and Blast | ✓ | | | Pneumatic. Top-hammer rotary percussive methods can be used for drilling small diameter blast holes. |
| Bulldozer Ripping | ✓ | ✓ | | Single tine used for very heavy ripping (of poorly fractured rock) and multiple tines for medium ripping (of fractured or weak rock). The correct selection of tine, ripper arrangement and method of use will each affect the efficiency of excavation and the characteristics of the excavated material. |
| Bulldozer | | ✓ | ✓ | Blade excavation of fractured rock may reduce oversize associated with ripping (when feasible). However, when ripping is not required then use of plant that can excavate and load is desirable. |
| Grader | | | ✓ | Typically not efficient for excavation, but may be required for mixing material in the pit or on the road. |
| Excavator | | ✓ | ✓ | Versatile method of excavation and loading. Large selection of plant available. Face excavation may allow effective mixing of beds. |
| Tractor Backhoe | | | ✓ | Rate of production may be limited, but might be adequate particularly if material is stockpiled. |
| Drag-line Excavator | | | ✓ | May be required for excavating gravel from below the water table |

| Activity | Type of excavation | | | Comments |
|---|---|---|---|---|
| | Heavy | Medium | Light | |
| Wheel Loader | | | ✓ | Ideal for excavating and loading loose gravel (after pit preparation) |
| Crawler Loader | | | ✓ | Usually not ideal due to lack of manoeuvrability on the tracks. |
| Scraper | | | ✓ | Best suited for large scale earthworks operations. will rarely be economical in low cost road construction. |

*RI (2000)*

Factors to consider when planning material excavation and use of mechanised plant include:

- Choose a method of extraction that produces the best quality "as dug" materials (i.e. does not generate a large proportion of oversize material that will be spoiled);
- If borrow pit materials are variable or inter-bedded, use plant and excavation methods that can produce a suitably mixed aggregate;
- Select plant that achieves an acceptable rate of material production (alternatively, programme stockpiling ahead of aggregate supply);
- Where possible, use plant that can both excavate and load the aggregate;
- If aggregates are likely to deteriorate/segregate in the stockpile, try to combine excavation and supply activities;
- Carefully programme plant and borrow pit activities that require more than one type of plant (i.e. the dozer to strip overburden and the excavator to dig gravel for loading or stockpiling);
- Programme activities so that the plant does not stand idle in a borrow pit;
- Ensure adequate supervision in the borrow pits so that appropriate extraction methods and procedures are followed by all concerned;
- Ensure satisfactory plant utilisation/output in the borrow pit by enforcing the following requirements:
  i. All plant should be in a sound mechanical condition and well-maintained;
  ii. All operators should be adequately trained and experienced; and
  iii. All items of plant should be operated within the normal limits of their capacity. The plant on site should not be overworked.

### 10.5.3 Efficient use of plant

Short descriptions follow of each item of equipment that is most commonly used for borrow pit extraction.

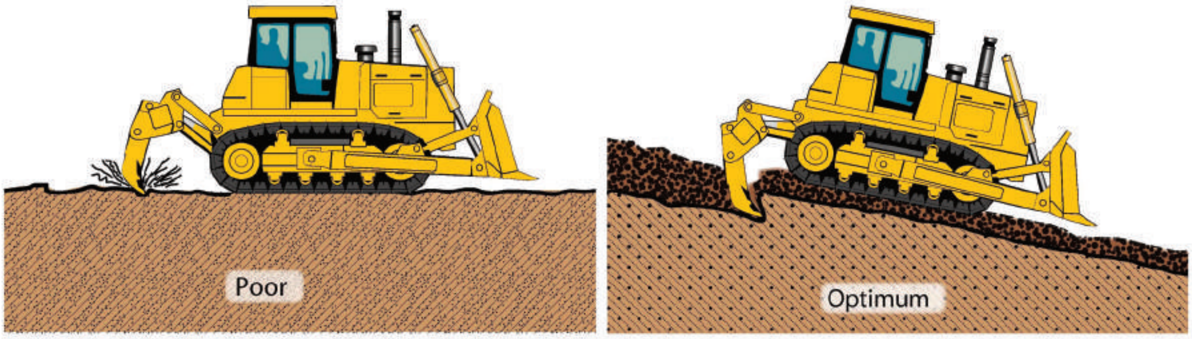
**Bulldozers**

These are the most commonly used plant to extract gravel from borrow pits. However, it is not necessarily always the most suitable. With softer materials, the use of a dozer may be an "overkill' as the material tends to get crushed under the tracks. Transport of dozers between sites is slow and this may incur lost time and extra cost. Crawler "dozers" are required when weak rock or cemented gravels are to be extracted. Cutting and pushing downhill invariably improves the operating efficiency of bulldozer excavation, because the mass of the machine assists the process. Bulldozers may be used to push material up to about 150 m into a stockpile. However, their use is unlikely to be efficient for moving material over distances greater than this.

Bulldozers may be fitted with one or more ripper tines (shanks). The correct selection of tine, ripper arrangement and method of use affects the efficiency of excavation and the characteristics of the stockpiled material. There are two basic types of ripper and several types of ripper tine (shank) and tip, namely radial ripper and parallelogram ripper. Refinements to these types of ripper are the adjustable radial and the adjustable parallelogram. These rippers have hydraulic controls to enable the operator to vary the tip angle while ripping. In very hard ground a single tine parallelogram ripper is usually the most effective arrangement.

Ripper tines and tips consist of two basic types, the straight and the curved. Straight tines provide the lifting action needed in thinly-bedded sedimentary rocks or cemented layers plus the ripping ability in more thickly-bedded material. Curved tines work well in less dense material and require less ripping distance. In order to maximise efficiency in ripping to produce road construction material, the following points should be considered, as illustrated in Figure B.10.10:

- Ripping should proceed downhill as far as is possible to obtain the benefit from the weight of the machine;
- Care should be taken to arrange for ripping to make optimum use of natural fracture orientation in the material. Ripping will be most effective when carried out in the direction of inclined fracture planes (or bedding) as this tends to pull the tine into the ground;
- If the materials contain vertical joints that run parallel to the cut, it is sometimes necessary to rip across the cut to obtain proper material break-up. Ripping along the laminations may only produce deep channels in the material;
- Ripping as deep as possible will loosen the maximum amount of material, but ripping to partial depth may reduce the proportion of oversize material produced;
- Never remove all the ripped material before ripping deeper. Leave a layer of at least 100 mm to 150 mm of ripped material to provide better traction. This also reduces track wear and minimises crushing of the underlying materials.




*SOURCE: RI (2000)*

**Figure B.10.10 Poor and optimum arrangement for effective ripping**

### Excavators

Excavators are versatile digging machines that are produced in a great variety of configurations. All excavators have boom-arm hydraulically operated digger buckets and are turntable-mounted on either a crawler track or with a wheeled chassis. The turntable can be horizontally rotated on the fixed chassis by a full circle. The reach of the boom arm for digging operations, either upwards or downwards, may be up to 6 m. In order to maximise efficiency in excavator digging and loading operations the following guidelines apply:

- Greatest efficiency is achieved if borrow pit materials can be loaded for haulage as they are excavated. The excavator and trucks should be arranged so that the operating cycle is minimised;
- Ensure that there are sufficient trucks so that the excavator does not have to wait for loading. However, in highly variable materials this step often eliminates stockpiling, which can be a good homogenising process. It also removes the opportunity to test the material before placing it;
- The size of the bucket should be appropriate for the particular conditions on site, such as the quantity and type of material to be loaded and the trucks used for haulage;
- The rake or angle of the bucket should be adjusted to suit the particular material. For easy digging and low cuts, maximum rake should be used. For harder digging and higher faces a smaller rake should be adopted.

### Backhoes

A backhoe comprises a bucket or shovel mounted on a hydraulic boom and attached to the rear of a crawler or rubber-tyred tractor. Backhoes are well suited to excavating relatively loose material from above or below the level of its wheels into trucks. The reach for digging and loading is controlled by the length of the boom, but is usually up to 4 m. Backhoes are easy to move between material sources and can also be useful for carrying out trial pit investigations during prospecting for material sources and borrow pit extension.

### Graders

In borrow pits, graders are normally only used for the following:

- Greatest efficiency is achieved if borrow pit materials can be loaded for haulage as they are excavated. The excavator and trucks should be arranged so that the operating cycle is minimised;
- Topsoil and loose overburden stripping;
- Mixing excavated materials (by windrowing);

- Maintenance of haulage and access roads; and
- Reinstatement of topsoil.

For mixing purposes, the blade is leaned forward at the top edge to enable the material to flow and rise freely.

Graders are frequently fitted with tines to allow hard road surfaces to be ripped to shallow depth, but graders are not generally efficient for borrow pit excavation work. Use of the ripping tines to aid extraction of borrow pit materials is liable to overwork the machine and also result in serious tyre wear.

**Front-end loaders**

A front-end loader is a bucket mounted on the front of a rubber-tyred or crawler tractor. Loaders are often used to excavate loose materials, such as river gravel. Tyres are likely to suffer excessive wear when excavating hard materials. Their main use in borrow pits is usually to load from the stockpiles into trucks. Loaders may also be used to transport small quantities of material over short distances (up to 200 or 300 m). Loaders may be used to transport ripped materials to the grizzly feed of a screen or crusher. For the efficient use of front-end loaders, the following apply:

- Loaders are usually not efficient at winning material and loading at the same time. Loading from stockpiles is their normal use;
- It is important to match the size and capacity of the loader to the trucks that they are working with;
- Material should be stockpiled in such a way that a full bucket can easily be achieved.

## 10.6 Stockpiling

### 10.6.1 Introduction

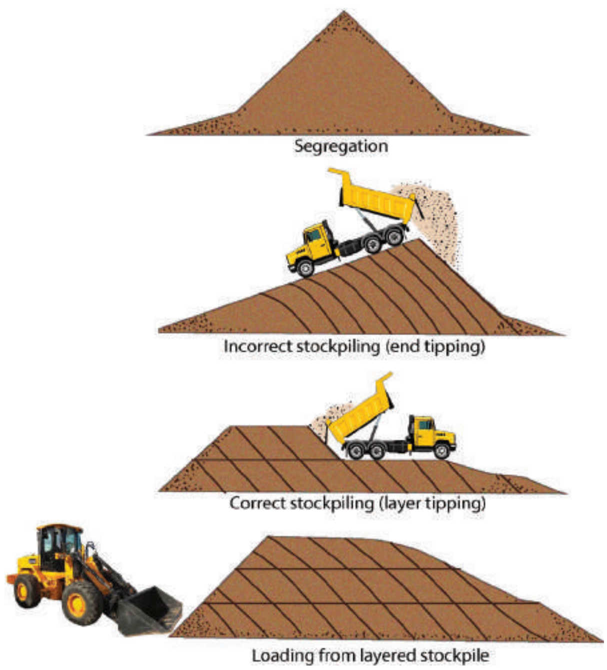
It is often necessary to extract and store quantities of material either in the borrow pit or at a location close to the section of the road that is to be constructed or maintained. Handling and stockpiling of aggregate needs to be undertaken with care to minimise wastage of material and avoid particle segregation. When there are different types of materials in the borrow pit, stockpiling is an efficient way of managing material variability and the following actions should be considered:

- Create different stockpiles according to the properties of the material;
- Mixing differing materials in the borrow pit; and
- Mixing differing materials on the road.

#### 10.6.1.1 Segregation

Segregation is the separation of materials according to particle size and results in pockets of coarse material with no fines in some places and pockets of fine material in others. In order to avoid segregation in the borrow pit the following method is recommended (Figure B.11.1).

During transportation of the materials, care should be taken to ensure that the loaded material does not segregate. On longer hauls, more segregation takes place and watering of the material is an option to avoid excessive segregation (as well as loss of dust during haulage). Segregation may also occur during end tipping of material on the road which may trigger a need for thorough mixing during the processing operations.




*SOURCE: RI (2000)*

**Figure B.10.11 Correct stockpiling to avoid segregation**

### 10.6.2 Mixing

Mechanical blending involves the mixing of two different materials to achieve a composition or grading that satisfies the intended end use that could not be achieved by using either material in isolation. It is often the most cost-effective option for increasing the quantity of an acceptable material. However, careful mixing of the various components being blended is essential. The commonly-used procedure shown in Figure B.10.12 is seldom effective and is not recommended for a borrow pit.


SOURCE: RI (2000)

**Figure B.10.12 Non-recommended method for blending in borrow pit**

When materials are mixed in the borrow pit the proportions will normally be less accurate than when mixed on the road. When mixing on the road materials are loaded from two different stockpiles in the borrow pit as shown in Figure B.10.13.


SOURCE: RI (2000)

**Figure B.10.13 Loading from two separate stockpiles in the borrow pit**

When materials are mixed on the road it is important that the correct quantity of the dominant material (i.e. the largest proportion) is evenly spread on the road and lightly compacted before spreading the subordinate material in the correct quantity on top. The two materials must then be carefully blended on the road before final compaction.

## 10.7 Material processing and control

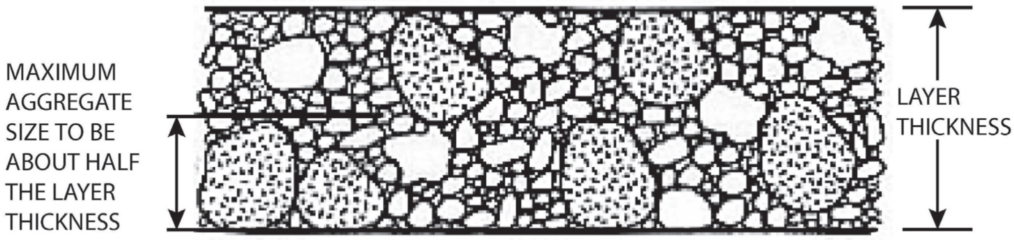
### 10.7.1 Introduction

There are various methods of improving the quality of "as dug" gravel road construction materials. Common material defects include the presence of oversize particles and too much or too little fine-grained "binder" material. Various procedures and treatments can be used to improve the engineering characteristics of "as dug" materials. The selection of the appropriate treatment will be strongly influenced by relative costs and benefits as well as by the severity of the problem. Typically, it is cost effective to use the best-quality materials that are available. Good performance of the material on the road will result in significant cost savings that will nearly always outweigh the expense associated with processing. Unfortunately, some of the benefits are not easy to quantify and to take account of in a cost/benefit analysis.

## 10.7.2 Dealing with oversize material

**Manual removal or breakage**

The maximum aggregate size in a gravel wearing course should be no greater than about half the layer thickness, with two-thirds of the layer thickness generally the absolute limit specified. This ensures that all large particles can be bound tightly in an interlocking structure as shown in Figure B.10.14. The presence of oversize material in unpaved roads results in rough roads that are difficult to maintain.




*SOURCE: RI (2000)*

**Figure B.10.14 Maximum aggregate size within layer**

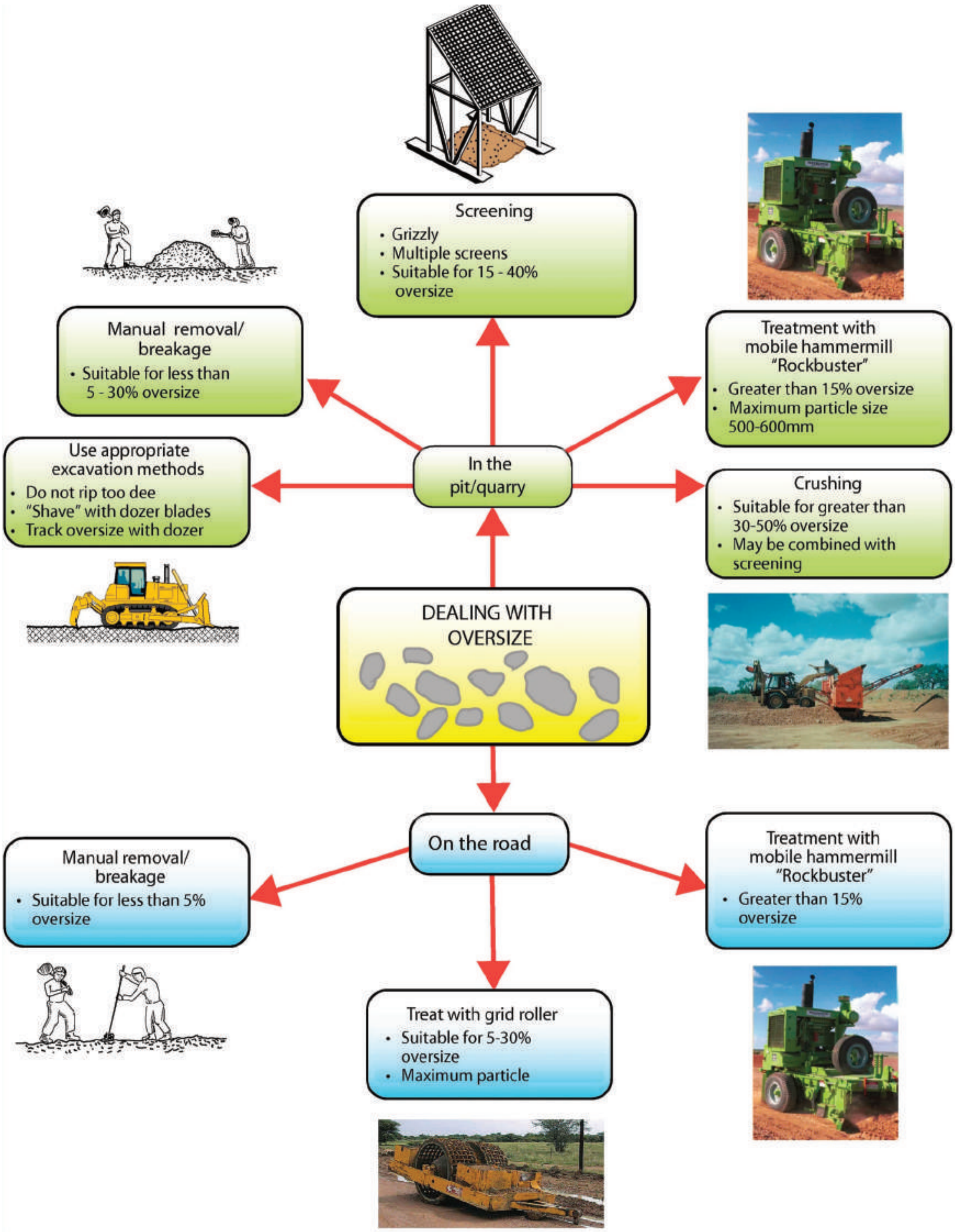
The various methods of dealing with oversize material either in the borrow pit or on the road (during material placement) are shown in Figure B.10.15. Each method is briefly reviewed below.

Where the proportion of oversize material is relatively small it may be treated effectively by manual removal or crushing either at the borrow pit or on the road. It may be beneficial to crush the oversize material and add it to the stockpile to optimise material usage and minimise wastage and spoil.

Field experience has indicated that manual treatment of oversize (removal or breaking) may not be successful where the proportion exceeds about 20%, even when large teams of labourers are employed for this purpose. However, the upper limit will depend on the diligence of the labourers, the way they are supervised and the ease with which the particles can be broken down. In the case of weaker materials, any large particles that are not manually removed will be broken down during compaction. The removal of hard oversize fragments at the construction site leads to considerable wastage and potential obstruction of side drains. It is therefore recommended that, whenever possible, oversize material is treated or removed at the borrow pit. Special plant is required to break down the large particles during construction, for example a mobile hammer mill or grid roller may be used for this purpose during pavement laying.

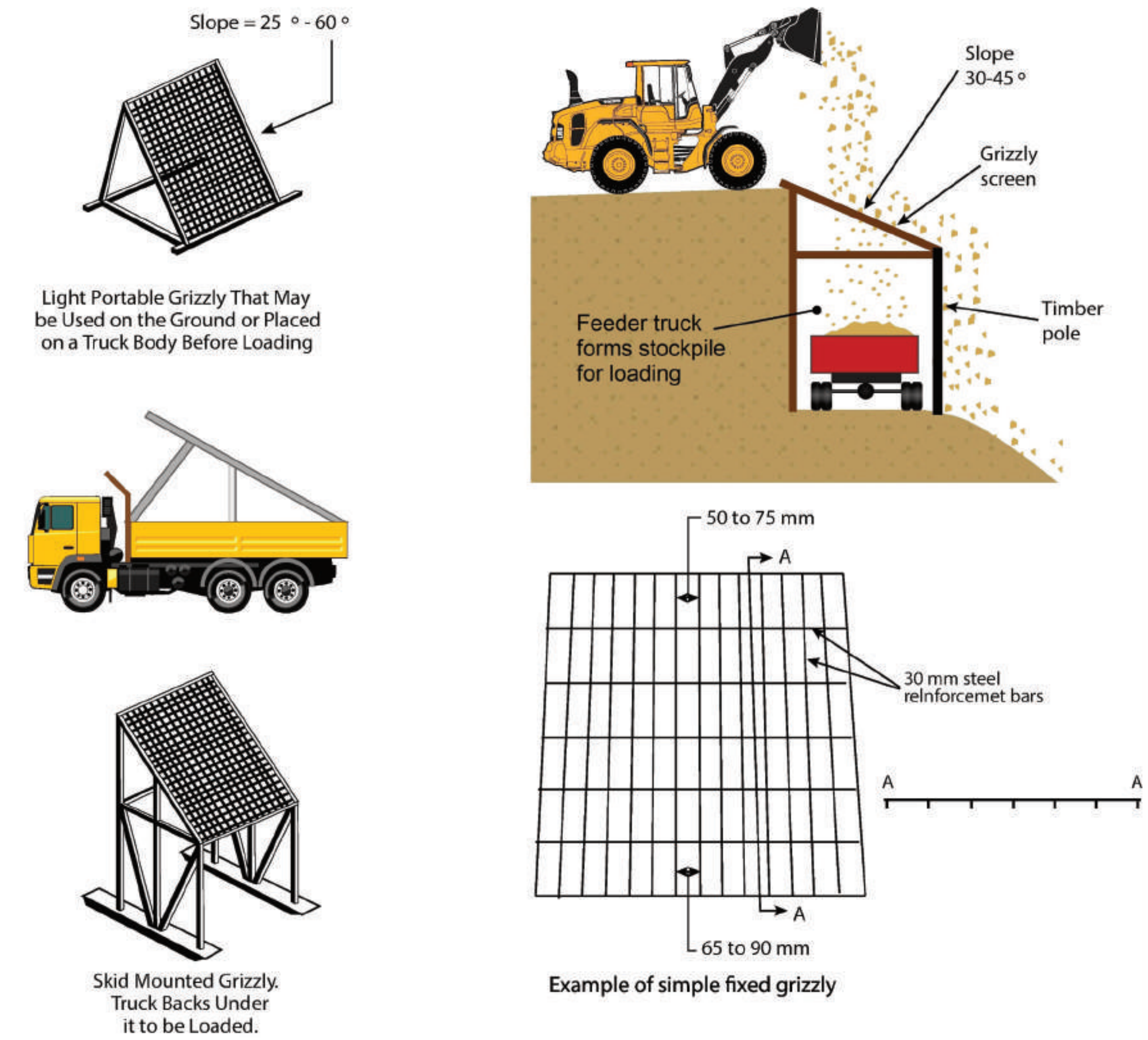
**Screening**

Screening to remove oversize particles at the borrow pit can be a low-cost solution when the proportion of oversize is in the range 15% to 40%. The screen comprises a frame supporting a mesh or slotted panel with an aperture designed to prevent large particles passing through. The oversize material removed by screening is rejected unless crushing plant is available. Various methods of screening exist and each method is appropriate for use in different situations. Screening to remove oversize aggregates is an important and cost-effective material processing technique. Figure B.10.16 illustrates the various types of simple screens.




*Source: RI (2000)*

**Figure B.10.15 Dealing with oversize material**




*SOURCE: RI (2000)*

**Figure B.10.16 Examples of simple fixed grizzly screen**

### Crushing

Crushing of oversize aggregate is expensive so is normally reserved for surfacing and concrete works and not used for gravelling or pavement layer construction. Even primary crushing of rippable materials with a small mobile crushing machine may lead to aggregates that cost more than three times as much as natural occurring materials. As a result, only the large-scale production of high value crushed gravel road surfacing materials may be viable. Factors that should be considered before committing to the production of crushed aggregates include the:

- cost of hauling natural gravels from outside the area compared with the cost of producing crushed local materials;
- relative quality of crushed stone compared with alternatives;
- alternative of using a mobile hammer mill rather than conventional crushing equipment;
- viability of stabilising local fine-grained materials with, for example, lime or cement;
- influence of climate and topography on the viability of using a particular material;
- possible environmental benefits of the crushing of oversize material; and
- use of mobile crushers, which can be very effective.

In steeply sloping ground with high rainfall, such as occurs in many parts of Ghana, well-graded angular gravels stand up well to scour and traffic abrasion, because of the higher friction associated with particle interlock. Hence, the use of crushed river gravel, as opposed to rounded river gravel, may be justifiable for a gravel wearing course due to the resulting reduction in maintenance costs. However, both river gravel and crushed gravel tend to lack plastic fines and may be unsuitable for use as wearing course gravel, unless blended.

When natural gravel is not easily accessible and the only option is to produce crushed aggregate, consideration should then be given to constructing a bituminous surfacing. This may be a more economical solution in the long term (life cycle costing), and allow for conservation of local materials.

Stabilizing gravel either with lime or cement can also be used to produce a suitable wearing course gravel, and may be cheaper than providing a bituminous surface. However, chemical stabilisation is not commonly used in Ghana.

## 10.8 Excavation and testing

### 10.8.1 Introduction

The quality of materials produced and the quality of the road constructed are dependent, to a large extent, upon the following:

- Careful selection of suitable material and avoidance of contamination with overburden or underlying unsuitable deposits;
- Continuous monitoring and supervision of any processing activities; and
- Appropriate stockpiling methods.

A borrow pit supervisor should be appointed to control all extraction and processing operations. This is particularly important if the materials are variable or if the plant operators are changed frequently.

### 10.8.2 Sampling and testing

Prior to any gravelling/construction operations, laboratory testing should be carried out to determine:

- The characteristics of the excavated materials in all borrow pits that may be required to supply the section of road to be regravelled or constructed;
- Appropriate materials processing methods (if required); and
- The expected characteristics and uniformity of the processed materials.

The recommended type and frequency of tests are shown in Table B.10.2.

**Table B.10.2 Recommended Testing of Borrow Pit material**

| Tests | Frequency (every) | Comments |
|---|---|---|
| Atterberg Limits (PL, LL, LS) | 2,000 $m^3$ | Increase frequency if variable or marginal suitability |
| Grading Analyses | 2,000 $m^3$ | Increase frequency if variable or marginal suitability |
| Compaction and CBR | 4,000 – 6,000 $m^3$ | Dependent on uniformity of material |
| Particle Strength (AIV & ACV) | 4,000 – 6,000 $m^3$ | Dependent on uniformity of material |

## 10.9 Materials management

### 10.9.1 Introduction

Material supply strategies are often determined by field supervisors from undocumented knowledge (local experience). This may result in the exploitation of a diminishing number of local traditional natural gravel pits. In many areas, these existing material sources are rapidly becoming exhausted. The need for material prospecting, adequate borrow pit evaluations and material processing is very often not fully recognised. As a result, the best road construction materials are not always used and expensive over-haul is required due to perceived construction material deficiencies. It is recognised good practice to establish a national or regional borrow pit/quarry inventory that assembles and stores data concerning the location and engineering properties of road building material resources. There can however be challenges in keeping such inventories coordinated between agencies, accurate, complete, and up to date.

### 10.9.2 Record keeping

Records concerning the actual use of material should be prepared following completion of a project. Observations concerning in-service performance of materials should be documented so that the quality rating of the material may be assessed. The following data should be recorded:

- The actual source of materials used to supply each section of road;

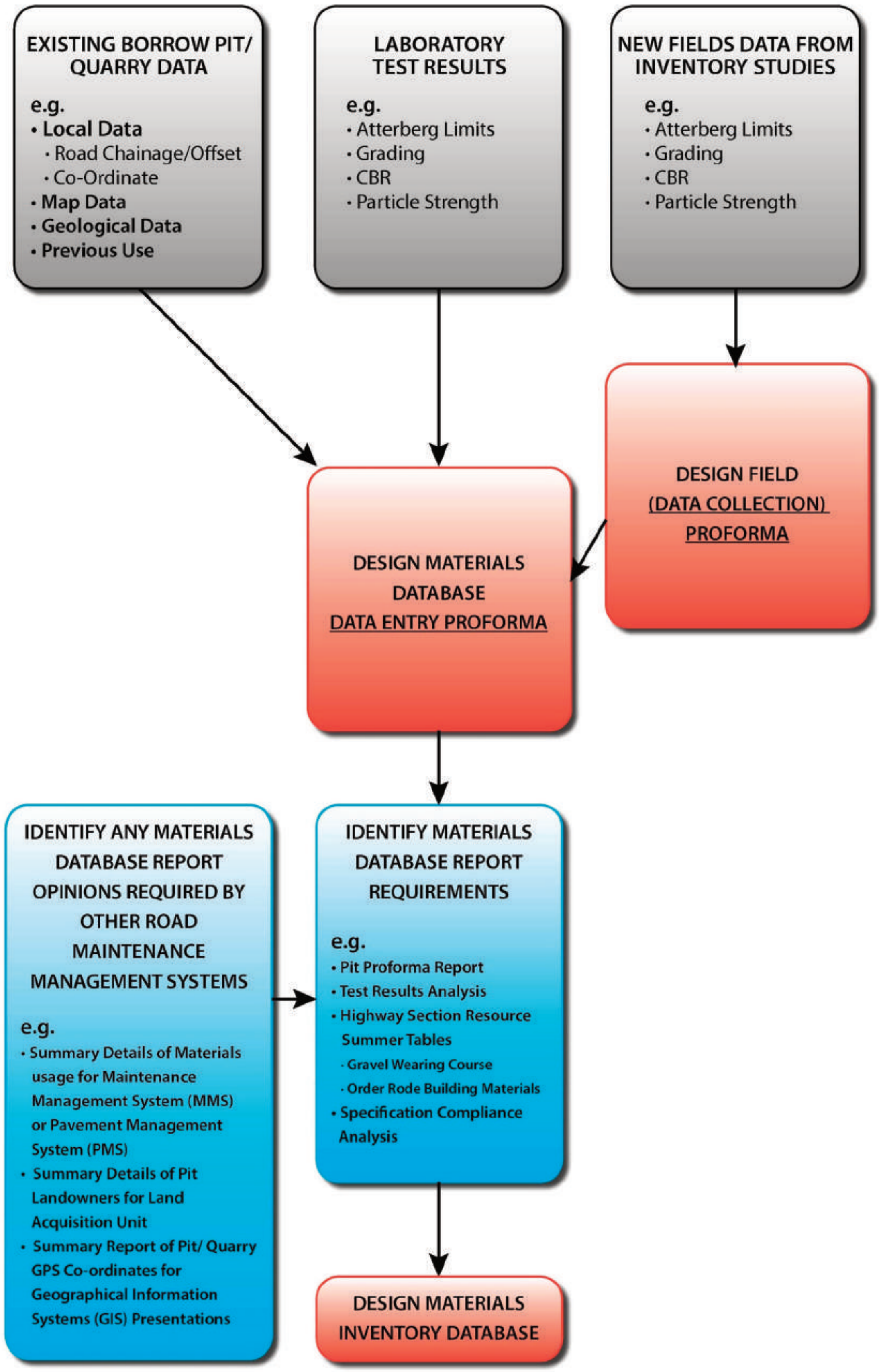
- The characteristics of the materials used to supply each road section;
- The cost per $m^3$ of material at the road side and the haulage cost per tonne-km; and
- An estimate of the quantity of exploitable material remaining in each source after completion of construction.

### 10.9.3 Materials database

The purpose of a materials database is, by consistently assembling existing records over time, to ensure that valuable data are preserved, centralised and made readily available to all interested parties for future use. Paper records are bulky and can easily be mis-filed, lost or destroyed. The development of a computerised database is an ideal and cost-effective solution to the need to preserve existing borrows pit records and enable all new information to be linked with historical data for analysis and evaluation.

In its simplest form a computerised materials resource database can be established using a spreadsheet programme such as MS EXCEL. This is encouraged as an intermediate measure, for gathering information on a road by road basis prior to the establishment of a more powerful and appropriate database management system. Figure B.10.17 shows an example of such a database.

The location of borrow pits and the type and quantity of material resources remaining in each could be usefully interfaced and integrated into the Road Database operated by the Department of Feeder Roads (DFR 2006).




*SOURCE: RI (2000)*

**Figure B.10.17 Example of a design materials resource database**

# 11. QUALITY ASSURANCE AND CONTROL

## 11.1 Introduction

Quality Control (QC) is about detecting and correcting defects. By contrast, Quality Assurance (QA) is about having systems in place to help prevent defects arising in the first place.

Good quality control/quality assurance (QC/QA) practices are essential to obtain satisfactory results on any road project. This is particularly the case for LVRs where naturally occurring, inherently variable materials are being used and it is essential that the underlying design assumptions are achieved on site. This includes critical factors such as use of materials of acceptable quality and attainment of the minimum compaction requirements and pavement layer thicknesses specified. Unless these and other specified requirements such as an adequate quality surfacing to waterproof the pavement structure and effective internal and external drainage, are met, then premature failure of the LVR is likely.

In developing a QC/QA system, it should be borne in mind that LVR projects in Ghana are often relatively small in size and widely scattered in remote areas with limited facilities. Moreover, the speed of construction can be relatively slow and the available resources and skills available to small contractors may be at a relatively low level. It is therefore necessary that such constraints are borne in mind while developing a suitable QC/QA system for construction. The types of quality control and their frequency must be judiciously selected so as to be achievable under the prevailing conditions.

The purpose of this Chapter is to set out a simplified approach to QA/QC that will achieve an acceptable end product in a typical rural environment. To achieve the best possible results in any given circumstance, such an approach will often require innovative solutions and a focus on a QA/QC system that is simple but robust.

This Chapter covers the general approach to QA/QC that is typically adopted in the construction of a LVR. It then focuses on the manner of undertaking QC with reduced resources and the priority that should be placed on QA.

## 11.2 Approach to QA/QC

### 11.2.1 General

There are various means of ensuring that an acceptable quality of the final LVR is achieved. Each is separate and each has an important role to play. Together they consist of a suite of procedures that work together to ensure good quality, but are often not fully understood. A clear distinction should be drawn between the following:

- Quality Plan (QP);
- Quality Assurance (QA);
- Quality Control (QC);
- Production Control PC); and
- Acceptance Control (AC).

The differences and functions of each component of the Total Quality Management System (TQMS) are now explained.

### 11.2.2 Components of a TQMS

**Quality Plan (QP)**

This refers to a written plan prepared by the contractor and submitted to the client / supervising engineer for review and approval. This document clearly describes how the contractor will control the processes used during construction in order to meet the requirements set out in the technical specifications. The QP will typically include the sequence of tests (QC tests) to be performed on the materials. Intended for use at a prescribed frequency, these have the objective of demonstrating that the intent of the specification is being satisfied.

The tender documents should include the requirement that the contractor must present the project Quality Plan that he/she intends to follow during the working process.

**Quality Assurance (QA)**

Quality Assurance refers to the component of the TQMS and associated work procedures within the construction site that help to ensure that correct quality of the final road is consistently attained. The

QA system includes the documentation required to show that the contractor is following the QC plan. It incorporates standard procedures and standard methodologies and applies to all site activities. The QA Plan should be followed by everyone on site and checked by both the construction supervisor and the contractor. QA activities are determined before construction work begins, and the activities are performed throughout construction. Components of a QA system include process checklists, project audits and construction methodology (contractor's work plan).

The tender documents should include the requirement that the contractor must present his project Quality Assurance Plan that he/she intends to follow during the working process.

**Quality Control (QC)**

Quality Control refers to measured quality-related attributes associated with the construction of specific elements of the project. QC is generally concerned with measuring properties and checking that specifications are being met consistently. It does not in itself create higher quality outputs. Examples of quality control activities include site inspections, field and laboratory testing. Such activities are performed after the work has been completed.

**Production control**

Production control is carried out by the contractor for the purpose of satisfying himself that chosen methods and materials meet the specified standards. Production control serves as an early warning for the contractor and helps reduce his risk of failure and associated additional cost to himself of remedial work. The contractor may be obliged to submit results from the production control to the supervising engineer and in some cases these may be taken as part of the acceptance control.

**Acceptance control**

Acceptance control is carried out by the supervising engineer to check compliance with the specified standards and thus to enable payments to be made. Acceptance control makes use of confirmed QA and QC testing. Results from acceptance control will normally form part of the as-built data which provides the basis of the road inventory kept by the responsible road agency.

Quality control supervision therefore comprises two principal elements namely site inspection and laboratory and *in situ* testing. A significant proportion of the latter is focused on compaction control and testing.

**Site Inspection**

The works are inspected visually to detect any deviation from the specified requirements. Visual assessment is an essential element of pavement layer approval, particularly in the identification of oversize material in lower pavement layers or in a gravel wearing course. Physical measurements of thickness, widths and crossfall are an essential element of this assessment. This activity is supplemented by simple *in situ* checking of specified procedures such as those for checking the temperature of bitumen, associated spray rates, and concrete slump test results.

**Laboratory and *in situ* testing**

Materials as well as the finished product are subject to laboratory testing for such characteristics as density and strength. On larger projects it may be possible for the contractor to set up and maintain a basic field laboratory for routine tests for quality control testing required on a day to day basis. The field laboratory will normally have test equipment that does not require an electric power supply and is relevant to the project specifications. There are also portable field test kits, such as the Gravel Test Kit supplied by CSIR of South Africa. This includes simple equipment for basic control tests.

**Field compaction control**

One of the most critical quality control activities is the field density compaction test, the outcomes of which could have a significant bearing on the performance of the road. The Sand Replacement Test, sometimes used in conjunction with Nuclear Density testing, is commonly used for compaction control on LVR projects. However, on a practical level, these tests are time consuming. They may be replaced for quality control purpose by the easier-to-perform DCP test, initially in conjunction with *in situ* density testing and moisture content testing for correlation purposes.

Compaction control is typically based on absolute requirements and spot tests. However, the number of such tests is often too low for a high level of statistical significance and therefore does not necessarily ensure a well-defined quality of the product. It is for this reason that a statistical approach to quality control should be adopted, particularly for the larger LVR projects.

### 11.2.3 Benefits of operating a TQMS

The operation of a TQMS offers the following major potential advantages:

- Ensures that the work is done correctly the first time;

- Achieves quality by focusing on preventing problems or errors rather than reacting to them;
- Ensures that errors are detected and corrected as early as possible. This requires quality controls, which include checking and back-checking procedures, to be implemented during all phases of the work;
- Eliminates the causes of the errors as well as the errors themselves. Removing the cause serves to improve the quality application process;
- Streamlines the inspection and release process for acceptance of constructed projects for handover and associated qualification for subsequent maintenance.

## 11.3 Quality Control Issues

### 11.3.1 General

The resources available for effective quality control in the construction of LVRs are sometimes limited. These vary with the size of the contract, with the risk of low QC capacity being particularly high when use is made of new and emerging contractors. It is therefore important to utilise whatever means are available as efficiently as possible and to combine conventional control methods with other practical procedures as follows:

- **Stockpiling** as a means of ensuring that the quality of the materials being used can readily be assessed. Such stockpiling is preferably carried out in the borrow pit rather than on the road;
- **Good management procedures** to ensure that materials are used to their full potential and to minimise any rejection of material after its transportation to the road;
- **Control** by observation of construction procedures by an experienced practitioner;
- **Proof rolling** (e.g. with loaded trucks) to test the stability of layers before proceeding with further construction;
- Use of methods (e.g. probing methods such as DCP and others) for **direct strength measurements** by correlation with known parameters ;
- Laboratory testing for '**calibration'** of method specifications; and
- Laboratory testing of typical material sources for '**calibration**' of visual observations.

In addition to the above, the following quality control activities, which are not expensive to undertake, can make a significant difference to the quality of the constructed LVR:

**Bituminous surfacing:**

- Visually inspect all surfacing equipment;
- Check that spray nozzles are not blocked;
- Check that bitumen spray temperature is correct;
- Ensure that spray rate is correct;
- Ensure that sheets for start and end points are in place;
- Ensure that longitudinal joint is correctly done; and
- Ensure that rolling is timely and correctly carried out.

**Base finishing:**

- Ensure that base is not too rough and/or too open;
- Ensure that 'biscuit' layers are not present. The use of a geological hammer/pick can be used to identify such flaws; and
- When priming is carried out, ensure that light watering takes place prior to spraying prime.

**Pavement layers and subgrade:**

- Test stockpile prior to transporting material to site;
- Ensure that the tested stockpile(s) are the only one used;
- Prepare a method specification for compaction control using i.e. a DCP; and
- Prepare a watering plan for adding water to the material.

### 11.3.2 Frequency of laboratory testing

The frequency of testing of borrow pits needs to strike a balance between cost, time and the statistical validity of the results. This will depend on the variability of the material: the more homogeneous the material the less testing will be necessary. Table B.11.1 provides guidance on the typical frequency of various laboratory tests carried out on samples collected after stockpiling of the material in the borrow pit in order to verify the quality of the material which will be hauled and used on site.

**Table B.11.1 Typical frequency of materials sampling and laboratory testing**

| Type of test | Frequency of the sampling and testing requirements |
|---|---|
| **Tests on Soils** | ▪ Samples shall be collected from each stockpile.<br>▪ Collect 5 samples from different parts of each stockpile.<br>▪ Each sample shall be 50 kg or more.<br>▪ Collect 2 samples per stockpile if the material is sand. |
| Grading tests | ▪ Two sieve analyses per material source. |
| Atterberg limits | ▪ 2 tests for each sample. |
| Determination of laboratory dry density and OMC. | ▪ Mix the material for each stockpile and carry out at least 2 similar tests, separate tests on the mixed material to check for accuracy of the test results. |
| Determination of CBR or DN value (soaked, OMC and 0.75 OMC (optional) | ▪ Carry out 2 similar tests on the mixed samples for each stockpile. |
| **Tests for concrete** | |
| Slump test on fresh concrete | ▪ One test for every 3 batch mixes. |
| Cube strength tests | ▪ Six cubes for every pour (every lift or continuous pour). |
| **Test on aggregate in the laboratory** | ▪ Samples shall be collected from 3 or more different positions in the truck or stockpile and the samples shall be tested separately. |
| Grading | ▪ Minimum of 3 tests and plot a grading envelop for each delivery on site. |
| ACV | ▪ Minimum of 3 tests and record and take an average of the values. |
| 10% FACT | ▪ Minimum of 3 tests and record and take an average of the values. |
| Water absorption | ▪ Minimum of 3 tests and record and take an average of the values. |
| **Bitumen tests** | ▪ Collect samples of each delivery and test separately. |
| Penetration (suitable for straight run and cutback bitumen, 60/70 pen, 80/100 pen, 150/200 pen, MC3000, MC800) | ▪ Minimum of 3 tests and record and take an average of the values. |
| Softening point (suitable for straight run and cutback bitumen, 60/70 pen, 80/100 pen, 150/200 pen, MC3000, MC800) | ▪ Minimum of 3 tests and record and take an average of the values. |
| Viscosity tests (suitable for straight run and cutback bitumen, 60/70 pen, 80/100 pen, 150/200 pen, MC3000, MC800) | ▪ Minimum of 3 tests and record and take an average of the values. |
| Bitumen content (suitable emulsions SS60, SS70) | ▪ Minimum of 3 tests and record and take an average of the values. |
| **Compaction tests** | |
| Testing of field compaction using sand replacement or Troxler. | ▪ 9 tests per 200 m (standard).<br>▪ 9 tests per 400 m for remote projects.<br>▪ 9 tests per km and DCP tests every 50 m with some of the tests carried out within 0.5 m of the sand replacement or Troxler tests for very remote projects.<br>▪ The average values of density and the standard deviation shall be plotted on a compaction judgment chart. |

## 11.3.3 Compaction quality control

### General

Compaction is a crucially important aspect of road construction. It substantially influences the long-term durability and performance of the pavement structure and hence the whole-life cost of the road. This aspect of the construction process must therefore be carefully controlled on site to ensure that the specified densities are met in a consistent manner. The main aspects that need to be considered, other than routine material control testing, are the field density attained, the layer thickness and the surface finish. In Ghana, inadequate moisture content is commonly encountered as an underlying cause of poor compaction on LVRs.

### Compaction control

The pavement layers (or gravel wearing course) should be compacted preferably to refusal or to the minimum specified density assumed in the pavement design. This requires that the materials be processed at or about OMC and that rollers of adequate mass are used. Trial sections should be constructed using the materials and plant that will be used on site, and density increase monitored for each pass of the roller. Compaction control can make use of any density determination method (nuclear, sand replacement, etc.) but the most practical method is to use a DCP which is quick and easy to carry out and which will provide adequate assurance that satisfactory compaction has been achieved.

### Compaction trial using the DCP

Provided the material is tested at or close to OMC, a reliable Target DCP DN value can be obtained from the compaction trial for the subsequent compaction control of the road works. This Target DN value can be determined as follows:

1. Following completion of the first 3 roller passes, undertake three DCP measurements after every successive pass of the roller up to about 8 passes.
2. Calculate the average DN value after each successive pass of the roller.
3. Plot the average DN values against the number of roller passes as shown in Figure B.11.1

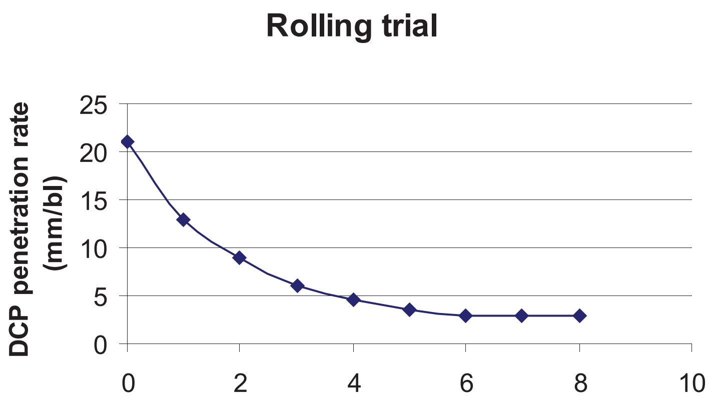



**Figure B.11.1 Typical results obtained from compaction trials**

The average DN value will normally decrease with an increase in the number of roller passes until the curves flattens out to a point where the decrease in DN value becomes negligible, in this illustrative example after 6 passes. This DN value becomes the Target DN for compaction control. This value is valid as long as the moisture content and the rollers do not change. Where possible the Target DN value should be correlated with the density obtained using the conventional density determination methods (sand replacement or nuclear gauge).

### Compaction control procedure using the DCP

The procedure to control compaction during construction is as follows:

1. Determine from a compaction trial the optimum number of roller passes and Target DN value (which is deemed to be at "compaction to refusal" at or close to OMC (+1%/-2%) as described above;
2. For each lot do a minimum of 10 DCP tests in a staggered pattern illustrated as illustrated in Figure B.11.2 with 3 tests on each side and 4 tests along the centre line; then
3. The offset from CL for the LHS/RHS tests shall be varied and no tests shall be done closer to the start/end and to the outer edge of the lot than 0.2 m.

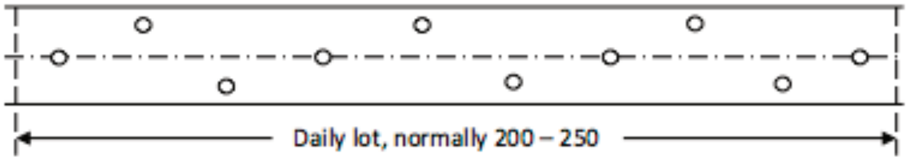
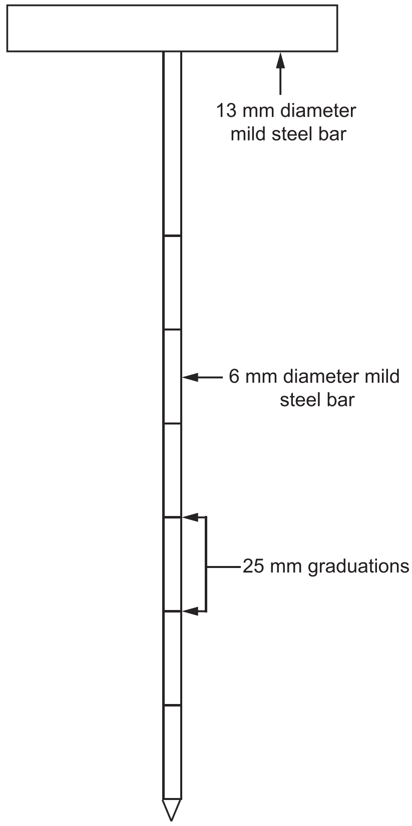



Figure B.11.2 Pattern for DCP compaction control tests

Further guidance on quality control of compaction may be obtained from the AfCAP Guideline for Compaction Quality Control on Low Volume Roads Using the DCP (2015).

### 11.3.4 Thickness control




It is important that the required thicknesses of material are placed. Layers that are too thin will require premature regravelling and those that are too thick will lead to an increased cost and possibly a lack of compaction in the lower part of the layer. Layer thickness can be easily controlled using a simple tool as shown in in Figure B.11.4. This tool can be easily made and is pushed into the ground until an obvious change in resistance is felt (the underlying, usually dried out layer). The thickness of the layer is then read off the scale. The tool can also be used to assess the uncompacted thickness before rolling, which together with knowledge of the bulking factor, can indicate whether the loose material is adequately thick. The layer should be slightly thicker than required as some thickness will be lost during final trimming and shaping, particularly developing the camber as described below.

Figure B.11.3 Probe for assessing layer thickness

### 11.3.5 Final finish

The final finish is critical as this will affect the riding quality (and thus the vehicle operating costs) as well as the drainage from the road surface. Good shape of the road is essential with a central crown and a cross-fall of between 4% and 5%. In order to ensure this, proper control using stakes and string lines must be carried out during construction.

In addition, it is essential to ensure that all oversize material has been removed. Any oversize material near the surface will be plucked during the final trimming and dragged along the surface leaving grooves. An excess of these grooves indicates that the oversize material has not been removed properly and the layer should be reconstructed after removing the large particles. Failure to remove them will result in excessive roughness, difficult, expensive and ineffective maintenance and excessive vehicle operating costs.

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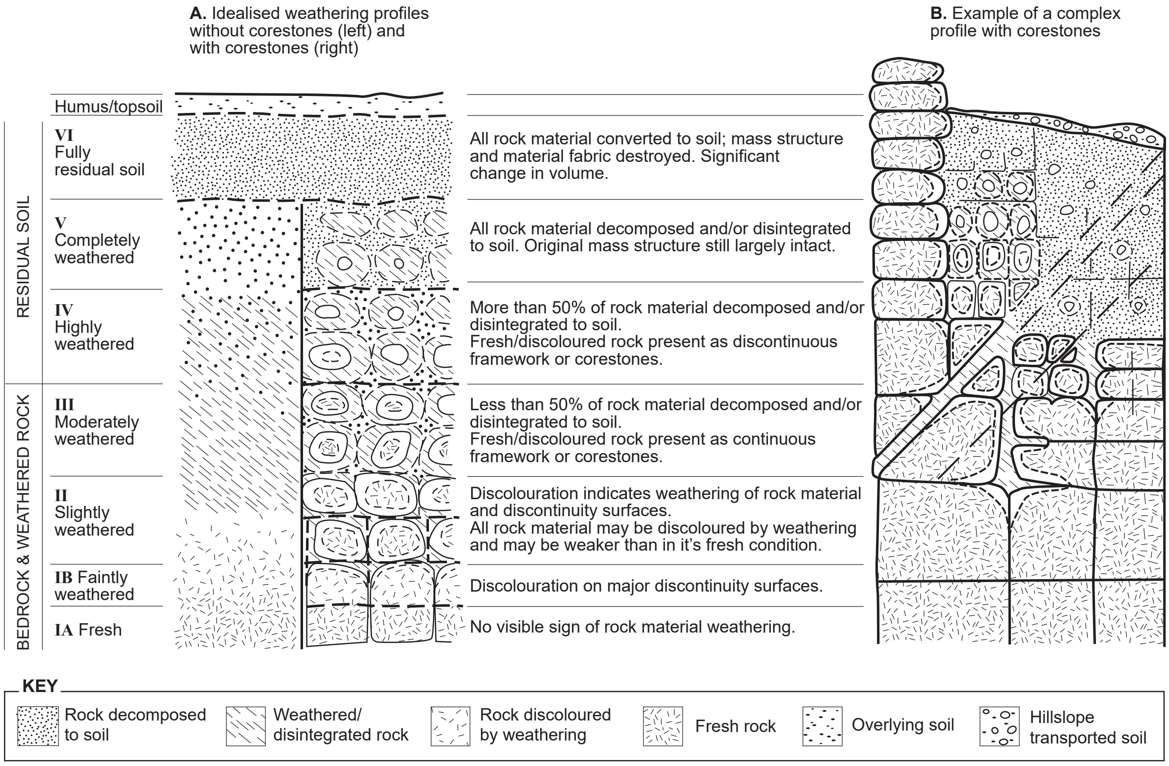
# Appendix B.1: Properties of soils

Eluvial soils are formed from *in situ* weathering of rock and are widespread in Ghana. Their behaviour is often linked to their weathering grade, as shown in Figure Appx B.1.1 and Table Appx B.1.1. Rock that is fresh or slightly weathered retains the same properties of the parent rock, whereas rocks that have undergone advanced weathering have developed into soils. These soils have varying composition according to the parent rock type and the degree of weathering, but they are often dominated by silts and clays with some gravels and cobbles. Residual soil is the name given to soil that is the product of complete weathering. It has a higher density than reworked soil of the same grading. Weathering and leaching processes often result in the precipitation of iron, aluminium and manganese compounds in the soil. Iron and aluminium precipitation imparts an orange to red-brown colour to the soil. These soils are described as ferralitic and they commonly occupy areas of higher ground. Ferralitic soils are widespread in Ghana (CSIR 1971). Given the drained topographic locations in which these soils are usually found, their short-term behaviour is often controlled by negative pore pressures (suctions or tensions between interparticle surfaces) that impart an increased apparent effective cohesion to the soil. These soils often have a density and cohesion that allow them to stand vertically in cuttings unless or until their strength is reduced by groundwater rise or surface water penetration. The precipitation of iron and aluminium oxides and hydroxides has the effect of welding the soil together so that ultimately it forms a hard pedogenic rock-like material, referred to as laterite. Soils that are intermediate in this weathering process often contain nodular laterite as gravel-sized clasts within a yellow-brown and orange brown silt-clay matrix. These soils are commonplace in the higher ground and sloping ground.




*SOURCE: Hearn (2011), modified from Fookes (1997)*

**Figure Appx B.1.1 Weathering grade classification**

**Table Appx B1.1 Weathering grade classification and its engineering implications**

**Weathering Grade I**
Rock is fresh with no visible signs of rock material weathering.

**Weathering Grade II**
Rock is slightly weathered: there has been some loss of material strength; > 90% of materials remain as competent rock; < 10% of materials have soil properties; more weathered, weaker materials are located along joints; joint shear strength is typically markedly lower than for joints in fresh rock; rock mechanics principles should be applied to excavation design; potential for kinematic (joint-controlled) failure may exist; blasting required for excavation; excavated materials behave as clean, competent, essentially free-draining rockfill.

**Weathering Grade III**
Rock is moderately weathered: *in situ* rock framework controls mass strength and stiffness; in excess of 50% of the material forms clasts that cannot be broken by hand but which may break down / degrade over time; shear strength along joints is typically markedly lower than for slightly weathered rock; combination of rock mechanics and soil mechanics principles to be applied to excavation design; potential for kinematic failure may exist; combination of ripping and blasting required for excavation depending on percentage of materials weathered to soils and joint pattern; when excavated described as boulders or cobbles, with some (5 to 20%) or much (20 to 50%) fines; behaves as a "dirty" rockfill which requires careful screening of fines and moisture control during placement and compaction.

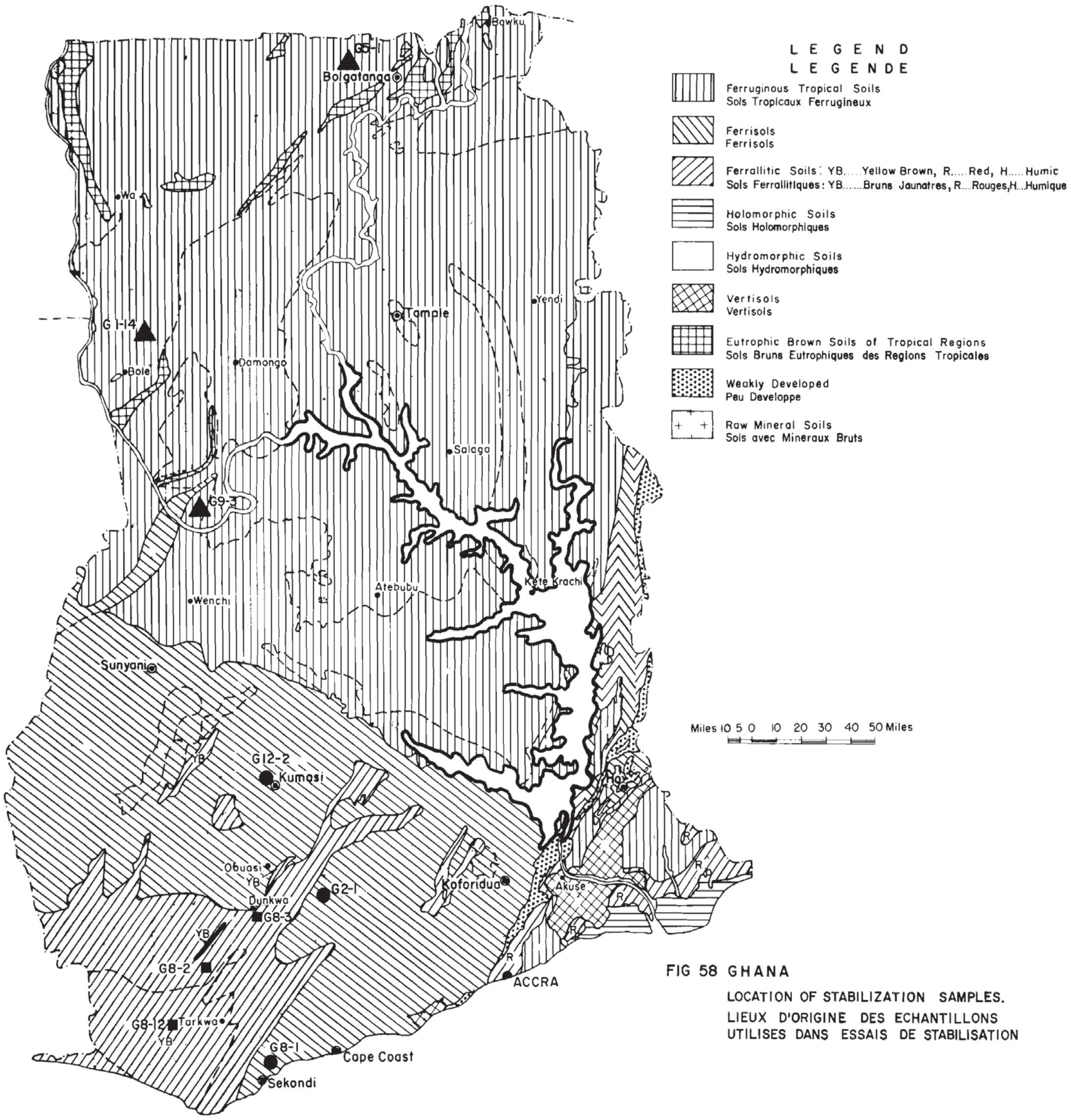
**Weathering Grade IV**
Rock is highly weathered: *in situ* rock fabric or texture contributes to mass strength; matrix or weathering products control stiffness; more than 50% of the material is decomposed or disintegrated to soil; remainder forms clasts that cannot be broken by hand and do not readily disaggregate or slake when a dry sample is immersed in water, but which may break down / degrade over time and are present as a discontinuous framework or core stones "floating" in a soil matrix; combination of soil mechanics and rock mechanics principles to be applied to excavation and foundation design; typically rippable during excavation but potentially problematic due to presence of boulders / core stones within the soil matrix (blasting of large remnant blocks may be required to break them down to a size that can be excavated and transported); when excavated, described as fine material with some (5 to 20%) or many (20 to 50%) boulders or cobbles; may not suitable as fill due to gap grading i.e. boulders in a fine matrix. A 'mixed fill' category might be required.

Weathering Grade V
Rock is completely weathered: all rock material is decomposed and / or disintegrated to soil; original mass structure still largely intact; considerably weakened compared to weathering grade IV material; slakes when wet; weathering products and relict structure control strength and stiffness; soil mechanics principles to be applied to excavation design, with a kinematic check required due to the relict structure; rippable during excavation; when excavated described as fine material; depending on soil characteristics excavated materials treated as common fill (if suitable), treated fill (where removal, mixing or blending is required to allow usage as fill) or unsuitable (cannot be used as fill due to susceptibility to erosion (unless protection is provided), too high a clay content or too low a plasticity); moisture control required during placement; potential for loss of structural strength during excavation, haulage, placement and compaction, and potential for loss of strength on wetting. High plasticity clay soils are not uncommon in tropical residual soil profiles and these will have low friction and may be subject to long-term softening as a result of loss of effective cohesion.

**Weathering Grade VI**
Residual soil: all rock material converted to soil; mass structure and material fabric destroyed; behaves as a soil; soil mechanics principles to be applied to excavation design; rippable during excavation; when excavated described as fine material; depending on soil characteristics excavated materials treated as common fill (if suitable), treated fill (where removal, mixing or blending is required to allow usage as fill) or unsuitable (cannot be used as fill due to susceptibility to erosion (unless protection is provided) due to too high a clay content or too low a plasticity); moisture control required during placement. High plasticity clay soils are not uncommon in tropical residual soil profiles and these will have low friction and may be subject to long-term softening as a result of loss of effective cohesion.

*SOURCE: Hearn (2011)*

Figure Appx B.1.2 shows the approximate distribution of the main soil types of Ghana. Table Appx B.1.3 then provides an outline definition of these various soil types.




*SOURCE: Redrawn from AID (1971)*

**Figure Appx B.1.2 Distribution of principal soil types in Ghana**

**Table Appx B.1.2 Summary definitions of soil types shown on Figure 2**

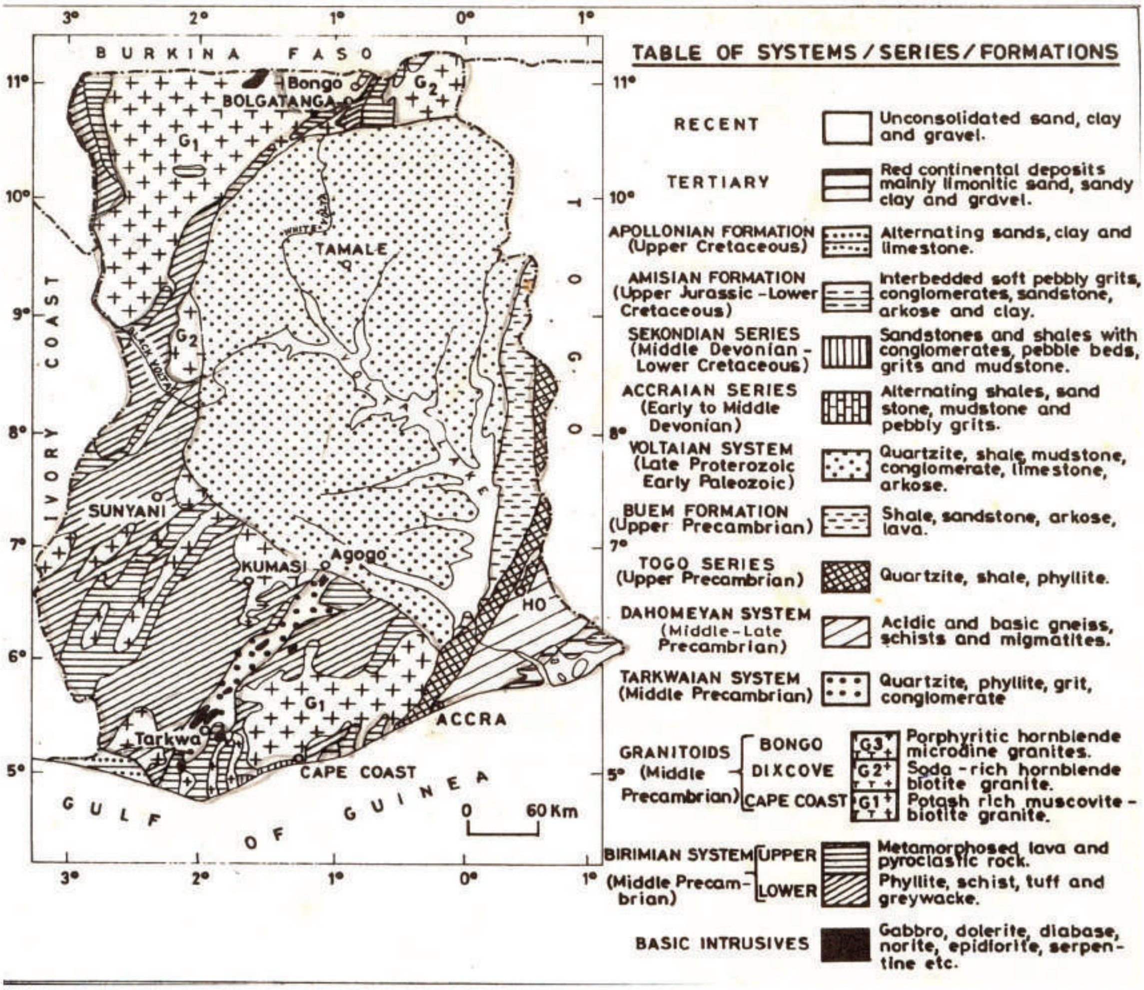
| Soil type | Definition | Comments |
|---|---|---|
| 1) Ferruginous | Residual soil of the sub-tropics (20-25 °C; 1000-1500 mm rainfall) | Partially chemically-altered residual soil (muscovite and orthoclase remain, and soil profiles are usually up to 3 m thick. |
| 2) Ferrisol | Transitional between ferruginous and ferrallitic types | |
| 3) Ferrallitic | Residual soil of the hot (> 25 °C) humid (> 1500 mm) tropics | Deep weathering profiles, all primary minerals except quartz are weathered by hydrolysis, kaolinite-rich, bauxite (Al2O3) and laterite (Fe2O3) duricrusts |
| 4) Holomorphic | Saline soils (chlorides, sulphates, carbonates and bicarbonates) | Adverse effects on road pavements and asphalt surfacings, sulphate attach on concrete |
| 5) Hydromorphic | Soils formed under seasonally waterlogged conditions – gley soils | Can be associated with compressible subgrades and can develop into acid sulphate soils that yield sulphuric acid upon contact with oxygen |
| 6) Vertisol | Expansive black cotton soil | Shrink-swell cycles cause heave and settlement to road pavements |
| 7) Eutrophic | Nutrient-rich soils | No known adverse effects on roads, depends on geochemistry |

*SOURCE: Some definitions taken from Fookes et al (1997)*

Soil types 1) to 3) in Table 3 are usually formed in well-drained areas, generally occupying higher ground. They represent long periods of geological stability in the landscape. Given the drained topographic locations in which these soils are usually found, their short-term behaviour is often controlled by negative pore pressures (suctions or tensions between interparticle surfaces) that impart an increased apparent effective cohesion to the soil. These soils often have a density and cohesion that allow them to stand vertically in cuttings until their strength is reduced by groundwater rise or surface water penetration. The precipitation of iron and aluminium oxides and hydroxides has the effect of welding the soil together so that ultimately it forms a hard pedogenic rock-like material, referred to as laterite in ferralitic soils. Soils that are intermediate in this weathering process often contain nodular laterite as gravel-sized clasts within a yellow-brown and orange brown silt-clay matrix. These soils are also common and can be less stable when excavated, counter-intuitively.

Other soil types found in Ghana also reflect the underlying rock types from which they have been formed. Expansive (*black cotton*) soils (vertisols) are high in smectite clay mineral content. These ordinarily develop on volcanic rocks (especially basalts), though in Ghana they are also developed on some gneissic rocks (Gidigasu and Gawu 2014). According to these authors and DFR (2006) these soils can be found on the plains of Accra, Ho-Keta and Winneba Plains, i.e. in the Volta River delta region. Their approximate distribution is also shown on Figure 2. Although not shown on this Figure, soils high in mica content can pose problems of compressibility and erosion potential and, according to DFR (2006), such soils predominate in the coastal areas of the Central Region and southern parts of Ashanti Region.

Micaceous soils tend to be derived from the weathering of high-grade metamorphic rocks, such as schists and gneiss. Schists, for example, are found in the far southeast of the country and the south-west. Soils that have high clay and silt contents are often derived from the weathering of mudstones, shales, phyllites and conglomerates, and these are likely to predominate in the Volta River basin. These soils can be prone to instability in excavations and may be unsuitable for use as fill. Mudstone and shale rock itself are also often unstable when excavated, especially if adverse bedding is exposed. Granites occur in the north-west of the country and in the south, and to the north-west of Accra. They can be expected to be characterised by irregular weathering profiles. In fact, abrupt changes in rock and soil profiles are common in the humid tropics, including in Ghana, where strong rock might be encountered next to residual soil, either horizontally or vertically. This can make it difficult to predict the volumes of rock and soil in earthworks, as illustrated in Figure Appx B.1.4. Trial pitting can help to identify such anomalies in the shallow excavation depths that are commonly encountered along LVRs




*SOURCE: Yendaw (undated)*

**Figure Appx B.1.3 Geology of Ghana**

In most cases, and especially in the case of eluvial soils, the density and grain size of soil increases with depth, though this can be complicated in lateritic soil profiles where hardpan crusts have developed, either on the surface or within the soil profile. Ordinarily, the materials exposed in the top metre or so might be considered to be weaker than the soil at depth and this may be important for earthworks design. In alluvial deposits the soil profile will be complicated by the depositional history of the material, and in localised areas of Ghana compressible organic and peaty (hydromorphic) soils may be present. These soils are usually bluish-greyish in appearance due to a *reducing* (oxygen-deficient) weathering environment and are termed gley soils by pedologists. They are subject to water saturation and seasonal alternation between waterlogging and drainage, and are often found in low-lying areas, valley floors and in the coastal belt. These soft and compressible soils pose significant difficulties as subgrade for road construction and should be avoided wherever possible. Investigations to determine the depth of these soils will allow an assessment to be made as to whether subgrade replacement is practical, or whether drainage and expensive chemical stabilisation are viable options.

Abrupt interface between WG V and WG I-II, i.e. rock that requires blasting is found directly beneath residual soil

WG II-III overlies WG V due to increased weathering at depth caused by groundwater flow. Creates weaker materials with depth

Marked lateral change from WGI on left to WG V on right with no topographic indication

Weathering grade changes laterally as well as with depth (WG decreases in direction of arrows). Note the infilled vertical joints

*SOURCE: Hearn (2011)*

**Figure Appx B.1.4 Implications of departures from standard weathering profiles**

Colluvial soils are usually low-density and often contain both fine-grained and coarse-grained clasts. There is sometimes a distinct boundary between these soils and underlying eluvial soils or weathered rock. Colluvial soils are prone to instability, especially where water tables are high in sloping ground.

It is unlikely that the locations and thicknesses of any of these soils have yet been mapped in detail, and it will be up to geological and materials staff to identify and investigate these soils ahead of any engineering design (see Sections B.3.5.1 to B.3.5.3).

Ground investigation for subgrade assessment generally comprises a combination of trial pitting, *in situ* probing and sampling, and associated laboratory testing of recovered samples. The location and composition of these investigations should be based on an initial assessment, from surface observation, of the distribution of the different soil and rock types that make up the study area. Published geological mapping is usually too small scale, and so it is usual to employ an engineering geologist or materials engineer to carry out a project-specific field mapping exercise. Mapping can be undertaken by recording field observations onto prints of published topographical (and geological) maps enlarged to scales of 1:10,000 for pre-feasibility studies or onto alignment drawings or strip maps for detailed design. The composition and variability in surface materials can often be assessed by inspecting exposures in cut slopes along existing roads, in river banks and as exposed rocks and soils on slopes. Where there are insufficient such exposures to make an initial assessment, or where there are no apparent material changes within the study area, ground investigation locations should be regularly-spaced, but with sufficient density to be able to detect any subtle, but potentially important, changes that may not be apparent from the surface.

# Appendix B.2: Testing Materials in the laboratory using the DCP

Thoroughly mix and split each borrow pit sample into nine sub-samples for DN testing in a CBR mould at three moisture contents: (a) soaked, (b) at OMC and (c) at 0.75 OMC and at three compaction efforts: (a) BS Light, (b) BS Intermediate and (c) BS Heavy as summarised in Table Appx B.2.1.

**Table Appx B.2.1 Scope of compaction testing**

| Compaction effort | Moisture content | | |
|---|---|---|---|
| BS Light | Soaked | OMC | 0.75 OMC |
| BS Intermediate | Soaked | OMC | 0.75 OMC |
| BS Heavy | Soaked | OMC | 0.75 OMC |

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

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

The procedure to be followed for determining the DN value of a material is similar to that for the more traditional CBR test except that a DCP is used to penetrate the CBR mould instead of the CBR plunger, (Figure Appx B.2.1).

Each of the nine specimens should be subjected to DCP testing in the CBR mould as summarised below.

a) Secure the CBR mould to the base plate and compact the sample as indicated above.
b) Place the full mould on a level floor and place the annular weight on top of the mould.
c) Place an empty CBR mould upside down next to the full mould. Alternatively use bricks or cement blocks to provide a firm platform for the base of the DCP ruler level with or slightly higher than the top of the full mould.
d) Position the tip of the DCP cone in the centre of the CBR mould, hold the DCP in a vertical position, knock it down carefully until the top of the 3 mm shoulder is level with the top of the sample and record the zero reading.
e) Knock the cone into the sample with "n" number of blows and record the reading on the ruler after every "n" blows as shown in the example. At OMC and 0.75 OMC "n" may be 5. At 4-days soak "n" may be 1 or 2. "n" does not have to be the same number for all readings.
f) Continue until just before the tip of the cone touches the base plate and stop in order not to blunt the cone (the last reading minus the "zero blows" reading must be less than the height of the mould 115 mm).
g) Determine the weighted average DN value, as described below.

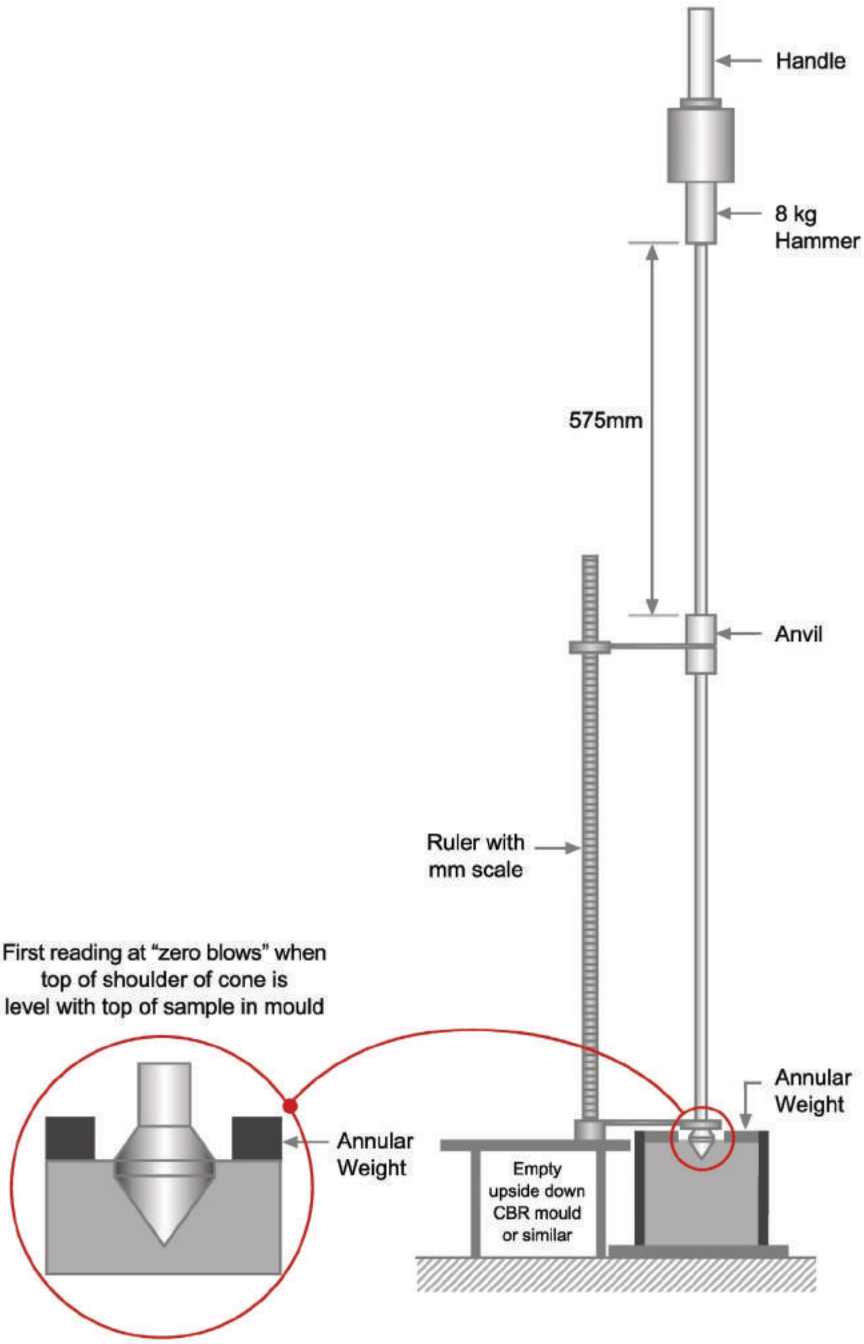



**Figure Appx B.2.1 Determination of laboratory DN value**

Procedure for calculating weighted average DN value for DCP lab test:

1. Record the readings as shown and calculate the DN per "n" blows and Average DN per blow.
2. Calculate the Weighted Average DN for the whole test using the formula:

$$DN = \frac{\sum(\text{Average DN per Blow} \times \text{DN per n Blows})}{\text{Penetration Depth}}$$

*Note that the Weighted Average DN is different from the Average DN which is not representative for the sample and is only to illustrate the difference.*

1. Carry out at least 2 more tests on the same material and calculate the average DN for the three (or more) tests.
2. Assess whether the material satisfies the design criteria from the DCP Design Catalogue (refer to Chapter 13).

An example of the determination of the weighted DN value for each of the nine sub-samples tested is shown in Table Appx B.2.2.

**Table Appx B.2.2 Determination of lab DN values by moisture content and specific density**

| 4 Days Soaked | | | | OMC | | | | 0.75 OMC | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 98% BS Heavy | | | | 98% BS Heavy | | | | 98% BS Heavy | | | |
| No of blows n | DCP Reading | DN per n blows | Avg. DN per blow | No of blows n | DCP Reading | DN per n blows | Avg. DN per blow | No of blows n | DCP Reading | DN per n blows | Avg. DN per blow |
| 0 | 130 | | | 0 | 129 | | | 0 | 123 | | |
| 1 | 150 | 20.0 | 20.0 | 5 | 137 | 8.0 | 1.6 | 5 | 141 | 18.0 | 3.6 |
| 1 | 167 | 17.0 | 17.0 | 5 | 149 | 12.0 | 2.4 | 5 | 151 | 10.0 | 2.0 |
| 1 | 180 | 13.0 | 13.0 | 5 | 164 | 15.0 | 3.0 | 5 | 165 | 14.0 | 2.8 |
| 1 | 190 | 10.0 | 10.0 | 5 | 178 | 14.0 | 2.8 | 5 | 178 | 13.0 | 2.6 |
| 1 | 215 | 25.0 | 25.0 | 5 | 194 | 16.0 | 3.2 | 5 | 190 | 12.0 | 2.4 |
| | | | | 5 | 216 | 22.0 | 4.4 | 5 | 206 | 16.0 | 3.2 |
| | | | | | | | | 2 | 214 | 8.0 | 4.0 |
| Penetration depth | | 85.0 | | | | 87.0 | | | | 91.0 | |
| Average DN | | | 17.0 | | | | 2.9 | | | | 2.9 |
| Weighted Average DN | | | 18.6 | | | | 3.2 | | | | 3.0 |

Figure 2 shows the typical relationships between DN, density and moisture content for a naturally occurring material. This illustrates two critical factors that affect the long-term performance of the road:

a) The need to specify the highest level of density practicable (so-called "compaction to refusal") by employing the heaviest rollers available.

b) The need to ensure that the moisture content in the outer wheel track of the road does not rise above OMC. This requires careful attention to drainage.

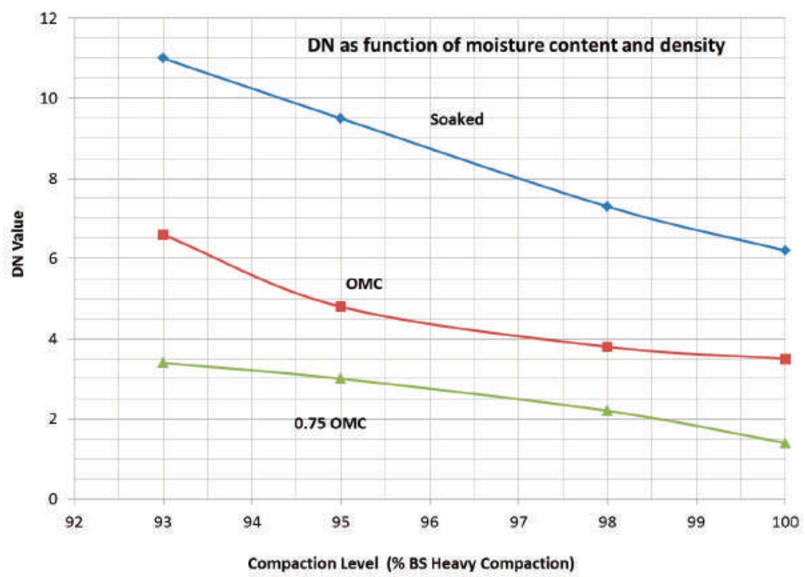



**Figure Appx B.2.2 DN/density/moisture relationship**

### Interpretation of Laboratory DN values

The effect of confinement in the steel CBR mould increases the strength of the material compared with that in a pavement layer. This needs to be taken into account when interpreting laboratory DN values. In order to account for this, the laboratory DN values shown in Table Appx B.2.3 are necessary to provide the required field DN value for the materials for use in the DCP design catalogue.

**Table Appx B.2.3 Relationship between required field DN and laboratory DCP DN values**

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Max Field DN value (mm/blow)** | 19.0 | 14.0 | 9.0 | 8.0 | 6.0 | 5.9 | 4.6 | 4.0 | 3.2 | 2.6 | 2.5 |
| **Max Laboratory DN value (mm/blow)** | 17.0 | 12.0 | 7.2 | 6.2 | 4.7 | 4.4 | 3.4 | 2.9 | 2.2 | 1.8 | 1.7 |

*SOURCE: Adapted from Kleyn, 1982*

In the DCP DN design method, the material strength in terms of the DCP resistance to penetration through is the primary design parameter specified and this is related to the required *in situ* strength, which depends on the traffic class of the road.

# Appendix B.3: Pavement Design Example using the DCP-DN method

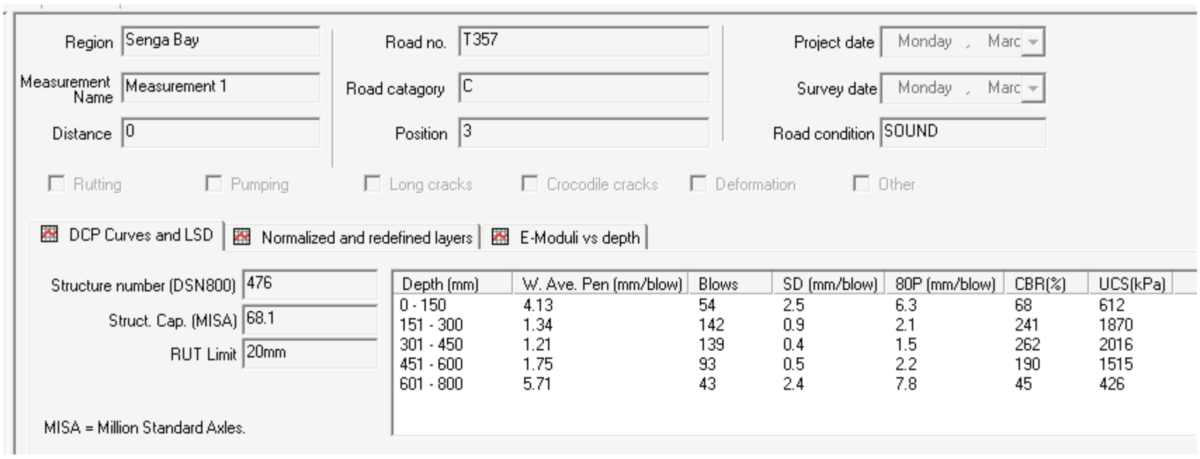
**The Project**

- A new paved road is to be built on the alignment of an existing gravel road to carry a cumulative design traffic of 0.3 MESA.
- A DCP survey was carried out under expected in-service conditions (wet) and the data were analysed using AfCAP DCP Design program.
- In all, 87 DCP tests were carried out, one every 100 m over the total length of the road of 8.6 km.

The following design procedure was followed:

**Step 1:**

Each DCP test was analysed using the program's "single measurement analysis". From the outputs (Figure Appx B.3.1), the $DSN_{450}$, and weighted average DN values for the upper three 150 mm layers of each DCP test were determined.


Region Senga Bay
Measurement Name Measurement 1
Distance 0
Road no. T357
Road catagory C
Position 3
Project date Monday , Marc
Survey date Monday , Marc
Road condition SOUND
Rutting Pumping Long cracks Crocodile cracks Deformation Other
DCP Curves and LSD | Normalized and redefined layers | E-Moduli vs depth
Structure number (DSN800) 476
Struct. Cap. (MISA) 68.1
RUT Limit 20mm
MISA = Million Standard Axles.

| Depth (mm) | W. Ave. Pen (mm/blow) | Blows | SD (mm/blow) | 80P (mm/blow) | CBR(%) | UCS(kPa) |
|---|---|---|---|---|---|---|
| 0 - 150 | 4.13 | 54 | 2.5 | 6.3 | 68 | 612 |
| 151 - 300 | 1.34 | 142 | 0.9 | 2.1 | 241 | 1870 |
| 301 - 450 | 1.21 | 139 | 0.4 | 1.5 | 262 | 2016 |
| 451 - 600 | 1.75 | 93 | 0.5 | 2.2 | 190 | 1515 |
| 601 - 800 | 5.71 | 43 | 2.4 | 7.8 | 45 | 426 |

**Figure Appx B.3.1 Typical output of DCP Program**

The accompanying plot of the DCP penetration rate with depth is shown in Figure Appx B.3.2. The individual results are then tabulated in a spreadsheet.

**Step 2:**

These results were then used to identify uniform sections using a cumulative sum technique. Prior to this all obvious outliers based on $DSN_{800}$ (very high or very low) were removed from the dataset (14 readings out of 87). The majority of these were particularly high, probably the result of stones within the layer. It is important, however, to check on site the actual reasons for the very high or very low readings as far as possible. Removal of the outliers only results in a smoothing of the curves and does not affect the actual "change points".

Figure 3 shows a part of the spreadsheet used to calculate the "cumulative sums" and Figures 4 and 5 are plots of the CUSUM curves for the different parameters. The CUSUM for the $DSN_{800}$ values is calculated by obtaining the average of all of the $DSN_{800}$ values (Column D in Figure 4) and then subtracting this from each of the $DSN_{800}$ values (column E). The results are then added together (Column F). These values are then plotted against distance.

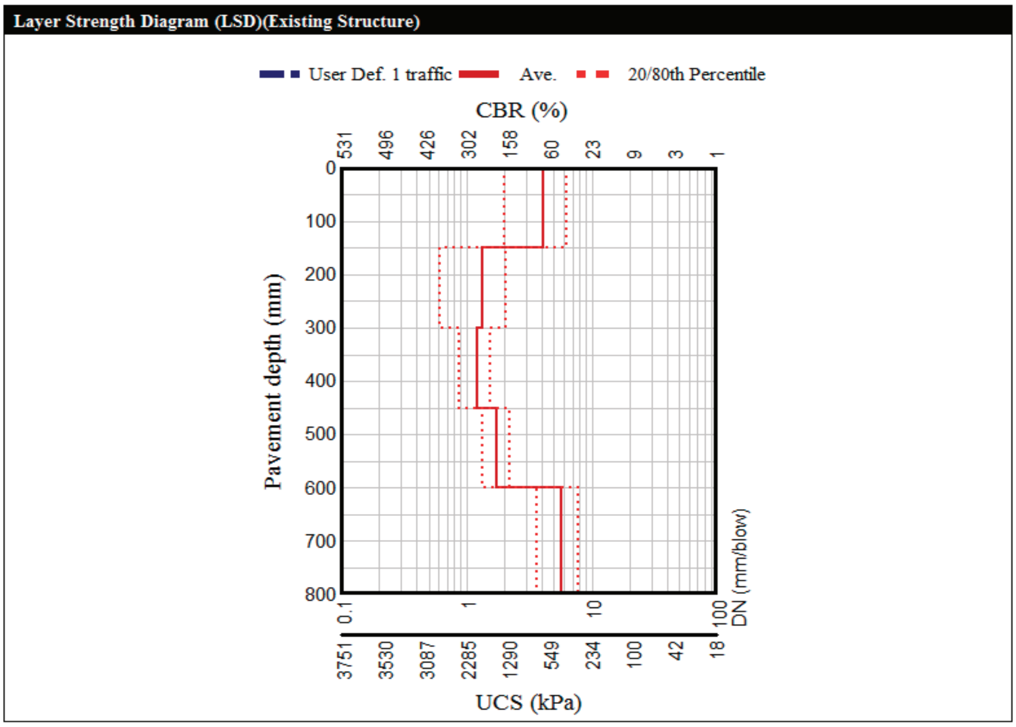



**Figure Appx B.3.2 Plot of penetration with depth for Test 1 (0.1 km)**

| | A | B | C | D | E | F | G | H | I | J | K | L | M | N | O |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Cusum Analysis E1641 | | | | | | | | | | | | | | |
| 2 | Test | | | DSN800 | | | 0-150 mm | | | 151-300 mm | | | 301-450 mm | | |
| 3 | no | Chainage | Position | DSN | DSN-Avg | Cusum | DN | DN-Avg | Cusum | DN | DN-Avg | Cusum | DN | DN-Avg | Cusum |
| 4 | 2 | 0.100 | RHS | 198 | 5.59 | 5.59 | 4.09 | -1.77 | -1.77 | 2.49 | -2.40 | -2.40 | 5.76 | 0.15 | 0.15 |
| 5 | 4 | 0.300 | RHS | 169 | -23.41 | -17.82 | 3.84 | -2.02 | -3.79 | 3.44 | -1.45 | -3.84 | 4.77 | -0.84 | -0.70 |
| 6 | 6 | 0.500 | RHS | 134 | -58.41 | -76.23 | 4.22 | -1.64 | -5.43 | 6.80 | 1.91 | -1.93 | 7.87 | 2.26 | 1.56 |
| 7 | 8 | 0.700 | RHS | 206 | 13.59 | -62.63 | 2.49 | -3.37 | -8.80 | 3.44 | -1.45 | -3.37 | 5.77 | 0.16 | 1.72 |
| 8 | 9 | 0.800 | LHS | 207 | 14.59 | -48.04 | 3.45 | -2.41 | -11.21 | 2.11 | -2.78 | -6.15 | 4.90 | -0.71 | 1.00 |
| 9 | 10 | 0.900 | RHS | 164 | -28.41 | -76.45 | 3.41 | -2.45 | -13.66 | 4.39 | -0.50 | -6.64 | 5.41 | -0.20 | 0.80 |
| 10 | 11 | 1.000 | LHS | 146 | -46.41 | -122.86 | 3.92 | -1.94 | -15.60 | 5.14 | 0.25 | -6.39 | 7.65 | 2.04 | 2.83 |
| 11 | 13 | 1.200 | LHS | 188 | -4.41 | -127.27 | 3.80 | -2.06 | -17.66 | 3.30 | -1.59 | -7.97 | 4.39 | -1.22 | 1.61 |
| 12 | 15 | 1.400 | LHS | 194 | 1.59 | -125.68 | 3.90 | -1.96 | -19.62 | 2.94 | -1.95 | -9.92 | 4.49 | -1.12 | 0.49 |
| 13 | 16 | 1.500 | RHS | 230 | 37.59 | -88.08 | 5.51 | -0.35 | -19.97 | 2.11 | -2.78 | -12.69 | 4.14 | -1.47 | -0.99 |
| 14 | 17 | 1.600 | LHS | 163 | -29.41 | -117.49 | 5.11 | -0.75 | -20.72 | 3.47 | -1.42 | -14.11 | 3.80 | -1.81 | -2.80 |
| 15 | 18 | 1.700 | RHS | 210 | 17.59 | -99.90 | 3.09 | -2.77 | -23.49 | 3.51 | -1.38 | -15.48 | 4.08 | -1.53 | -4.33 |
| 16 | 19 | 1.800 | LHS | 113 | -79.41 | -179.31 | 5.72 | -0.14 | -23.63 | 4.26 | -0.63 | -16.11 | 11.80 | 6.19 | 1.85 |
| 17 | 20 | 1.900 | RHS | 191 | -1.41 | -180.72 | 9.01 | 3.15 | -20.48 | 4.87 | -0.02 | -16.12 | 5.69 | 0.08 | 1.93 |
| 18 | 21 | 2.000 | LHS | 169 | -23.41 | -204.13 | 3.59 | -2.27 | -22.75 | 6.24 | 1.35 | -14.77 | 21.20 | 15.59 | 17.52 |
| 19 | 22 | 2.100 | RHS | 217 | 24.59 | -179.54 | 2.66 | -3.20 | -25.95 | 4.11 | -0.78 | -15.54 | 5.68 | 0.07 | 17.58 |
| 20 | 23 | 2.200 | LHS | 272 | 79.59 | -99.94 | 2.57 | -3.29 | -29.24 | 2.94 | -1.95 | -17.49 | 3.83 | -1.78 | 15.80 |
| 21 | 24 | 2.300 | RHS | 271 | 78.59 | -21.35 | 4.20 | -1.66 | -30.89 | 2.47 | -2.42 | -19.90 | 3.15 | -2.46 | 13.33 |
| 22 | 25 | 2.400 | LHS | 258 | 65.59 | 44.24 | 2.90 | -2.96 | -33.85 | 3.15 | -1.74 | -21.64 | 3.40 | -2.21 | 11.12 |
| 23 | 28 | 2.700 | RHS | 253 | 60.59 | 104.83 | 5.68 | -0.18 | -34.03 | 4.93 | 0.04 | -21.59 | 2.54 | -3.07 | 8.05 |

**Figure Appx B.3.3 Part of the spreadsheet showing the CUSUM calculation**

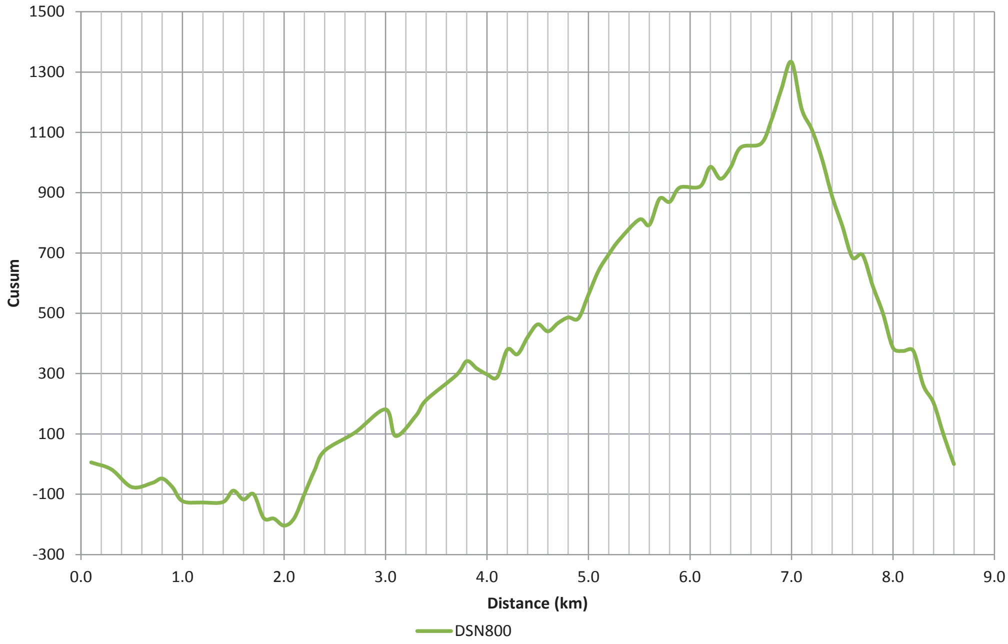



**Figure Appx B.3.4 Plot of CUSUM versus distance for the $DSN_{800}$ results**

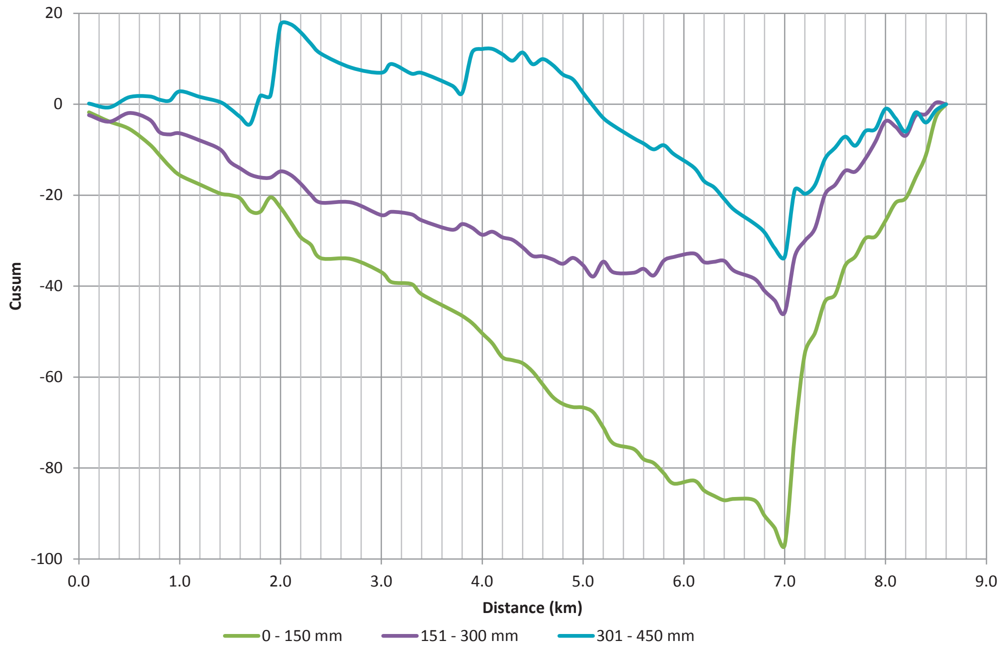



**Figure Appx B.3.5 CUSUM vs distance for the $DN_{150}$, $DN_{151\text{-}300}$ and $DN_{301\text{-}450}$ results**

From the plots of $DSN_{800}$ and $DN_{301\text{-}450}$ it is evident that there are significant changes in the subgrade support at about km 2.0 and km 7.0. The change at km 7.0 is also reflected in the $DN_{150}$ and $DN_{151\text{-}300}$ plots. It is thus possible to derive 3 distinct uniform sections from these plots: 0 to 2.0 km, 2.0 to 7.0 km and 7.0 to 9.0 km.

**Step 3:**

The data for each of these uniform sections are then analysed individually. The outliers can be retained or removed and generally have little impact on the final result. By retaining the outliers, the average $DN_{150}$ is 4.59 compared with a value of 4.34 obtained when they are excluded (Figures Appx B.3.6 and Appx B.3.7).

Region: Senga Bay | Road no.: T357 | Project date: Monday , Marc
Measurement Name: Average Analysis | Road catagory: C | Survey date: Monday , Marc
Distance: --- | Position: --- | Road condition: SOUND

☐ Rutting ☐ Pumping ☐ Long cracks ☐ Crocodile cracks ☐ Deformation ☐ Other

DCP Curves and LSD | Normalized and redefined layers | E-Moduli vs depth

Structure number (DSN800): 197
Struct. Cap. (MISA): 3.2
RUT Limit: 20mm

| Depth (mm) | W. Ave. Pen (mm/blow) | Blows | SD (mm/blow) | 80P (mm/blow) | CBR(%) | UCS(kPa) |
|---|---|---|---|---|---|---|
| 0 - 150 | 4.59 | 46 | 0.6 | 5.1 | 59 | 544 |
| 151 - 300 | 3.98 | 57 | 0.7 | 4.5 | 71 | 638 |
| 301 - 450 | 7.04 | 37 | 1.0 | 7.9 | 34 | 337 |
| 451 - 600 | 8.47 | 26 | 0.4 | 8.8 | 27 | 274 |
| 601 - 800 | 7.70 | 31 | 0.4 | 8.1 | 31 | 305 |

MISA = Million Standard Axles.

Figure Appx B.3.6 "Average points analysis" for uniform Section 1 including all points

DCP Curves and LSD | Normalized and redefined layers | E-Moduli vs depth

Structure number (DSN800): 178
Struct. Cap. (MISA): 2.3
RUT Limit: 20mm

| Depth (mm) | W. Ave. Pen (mm/blow) | Blows | SD (mm/blow) | 80P (mm/blow) | CBR(%) | UCS(kPa) |
|---|---|---|---|---|---|---|
| 0 - 150 | 4.34 | 47 | 0.7 | 4.9 | 63 | 579 |
| 151 - 300 | 3.90 | 48 | 0.7 | 4.5 | 73 | 652 |
| 301 - 450 | 6.78 | 29 | 0.9 | 7.5 | 36 | 352 |
| 451 - 600 | 8.15 | 22 | 0.5 | 8.6 | 29 | 286 |
| 601 - 800 | 7.66 | 31 | 0.5 | 8.0 | 31 | 307 |

MISA = Million Standard Axles.

**Figure Appx B.3.7 "Average points analysis" for uniform Section 1 excluding "outliers"**

The data can be analysed using either the average point analysis function in the computer program or using the spreadsheet developed for the CUSUM analysis. The layer strength diagram output of the computer program analysis is shown in Figure 8. It can be seen that the average (note that this is not the 50th percentile) and the range between the 20th and 80th percentiles is very small.

The required percentiles of the measured DN values for the pavement design can be calculated on the initial spreadsheet using the Excel percentile function.

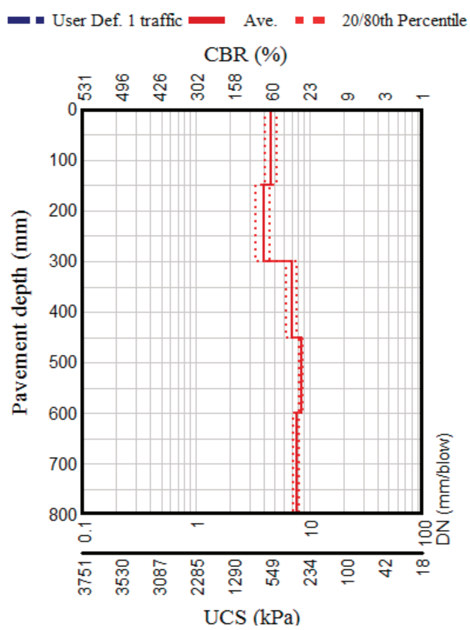



**Figure Appx B.3.8 Plot of average analysis for uniform section 1**

**Step 4:**

The process is repeated for each of the uniform sections. The results for each uniform section are summarised in Table Appx B.3.1. From the results it is evident that the upper layer in all cases is inadequate. A single design solution for uniform sections 1 and 2 requires that the upper layer be improved. It is noted that the material is only marginally inferior and it may be possible to improve its quality to the required specification through blending, better compaction or stone removal. If improvement of the existing material is not possible, the layer should be overlaid with a new 150 mm layer of selected material.

**Table Appx B.3.1 Summarised DN values for each layer and uniform section**

| Design class LE 0.3 | Spec. DN mm/blow | Uniform Section 1 km 0+000 - 2+000 50th percentile | Uniform Section 2 km 2+000 - 7+000 50th percentile | Uniform Section 3 km 7+000 - 8+600 50th percentile |
|---|---|---|---|---|
| 0-150 mm | 3.2 | 4.01 | 3.68 | 10.07 |
| 151-300 mm | 6 | 3.46 | 3.66 | 7.68 |
| 301-450 mm | 12 | 5.16 | 3.83 | 7.81 |
| 451-600 mm | 19 | 6.88 | 3.89 | 8.96 |
| 601-800 mm | 25 | 7.06 | 4.57 | 11.04 |
| DSN800 | 100 | 198.00 | 237.00 | 109.00 |

Uniform section 3, on the other hand, is particularly poor. Neither of the two upper 150 mm layers are adequate. The addition f a single 150 mm layer would not prove adequate and in this case, the upper 150 mm layer needs to be removed and discarded. The underlying layer (150 – 300 mm) should be assessed to see if it could be improved by blending or some other treatment. If not, this section of the road requires the addition of 300 mm of material after removal of the upper 150 mm.

The analysis is illustrated for uniform section 1 in Figure Appx B.3.9 showing a comparison of the *in situ* layer strengths with the layer strengths required for the selected traffic category. The areas shaded green are adequate and the yellow area is deficient. Note that although the average value is used in this example, both the 80th and 20th percentiles for all but the upper layer would have proved adequate. In the upper 150 mm layer, none of the percentiles would allow the material to be used in its current condition.

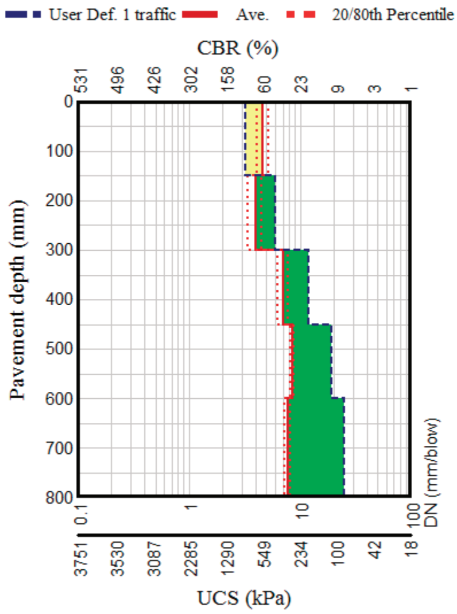



**Figure Appx B.3.9 Layer strength diagram - material strengths and traffic requirements**

**Verification of *in situ* materials properties through laboratory testing**

It is recommended that two bulk samples of the *in situ* material should be taken from each uniform section and the DN value at different moisture contents and densities established in the laboratory (see Appendix B.1). An in-service DN value for the material is read off Figure Appx B.3.2 depending on the likely moisture conditions and density.

The laboratory testing should include deriving the Atterberg Limits for the material in each layer to assess its likely sensitivity to moisture.

# Appendix B.4: The Structural Number method of pavement design

## 1. Introduction

In order to make optimum use of the existing layers in a road pavement, this method makes use of the structural number concept (AASHTO 1993). Using this method, the difference between the structural number of the existing road and that required for the upgraded road, obtained from the catalogue of structures, defines the additional requirements for upgrading, rehabilitation or reconstruction. The flow diagram for upgrading an existing road is shown in Figure Appx B.4.1.

| | |
|---|---|
| **Step 1** | Select design period, collect and analyse traffic data, determine traffic class. |
| **Step 2** | Undertake a DCP survey plus a sampling survey for moisture and strength measurements in the laboratory. |
| **Step 3** | For each test point determine the layer strength diagram (DN in mm/blow versus depth and CBR versus depth) and the layer boundaries using the computer program(1). |
| **Step 4** | Determine the in situ SN values for each layer and the total SN (and SNP) for the pavement at each test point using the DCP program(1, 2). |
| **Step 5** | Convert the CBR values (and hence the SN values) from in situ to soaked conditions for comparison with the design catalogue. |
| **Step 6** | Determine the required SN values at each test point from the catalogue appropriate to the climate, traffic and subgrade strength. |
| **Step 7** | Compare the in situ with the required SN to determine the structural deficiency at each test point. |
| **Step 8** | Identify areas (a) where the structural deficiency is large and (b) areas where layers are very weak and unlikely to meet the specifications for the layer that they will become in the upgraded design. (3) |
| **Step 9** | Identify uniform sections based on the structural deficiencies at each test point using a CUSUM method. |
| **Step 10** | Determine the appropriate percentile (4) of the structural deficiencies for each uniform section and design the upgrading requirements in terms of additional layers and/or layer processing. |

**Figure Appx B.4.1 DCP-CBR method - Flow diagram for upgrading an existing road**

Notes:

1. These calculations can be done by hand using a spreadsheet but the WinDCP 5.1/AfCAP LVR DCP program makes this easy and straightforward. *The software is available on the ReCAP web site www.research4cap.org.*
2. The SNP values are useful when the sub-base and/or subgrade comprise a number of different layers of varying strengths.
3. Normally the consolidated layers should not be disturbed. The *in situ* strength of the material as measured with the DCP is a better guide to their likely future behaviour than disturbed samples tested in the laboratory. Such layers will often be satisfactory as sub-base or even road base. However, if they are weak or of insufficient thickness the upgrading will need to take this into account in the design of any additional layer (or layers) that need to be added to provide the new road pavement.
4. Depends primarily on traffic level.

The DCP survey provides the thicknesses and *in situ* strengths of the layers of the existing road along the entire alignment. The analysis of the DCP data, preferably using the DCP program, provides the overall strength of the pavement at each test point based on the structural number approach.

## 2. Design procedure

To design the upgrading or rehabilitation of a road, it is first necessary to measure the structural number at each test point. The calculation of SN for design purposes is the AASHTO method which is based on the value of the soaked CBR of the layers. 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 OMC, and the use of Figure Appx B.4.2. The *in situ* moisture condition is obtained from the samples collected for laboratory analysis during the DCP survey. A minimum of three samples per kilometre 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 small (less than 14 days).

The relationship between soaked and *in situ* strength (CBR) depends on the characteristics of the materials. However, for the level of accuracy required, Figure Appx B.4.2, which is based on extensive research, is adequate.

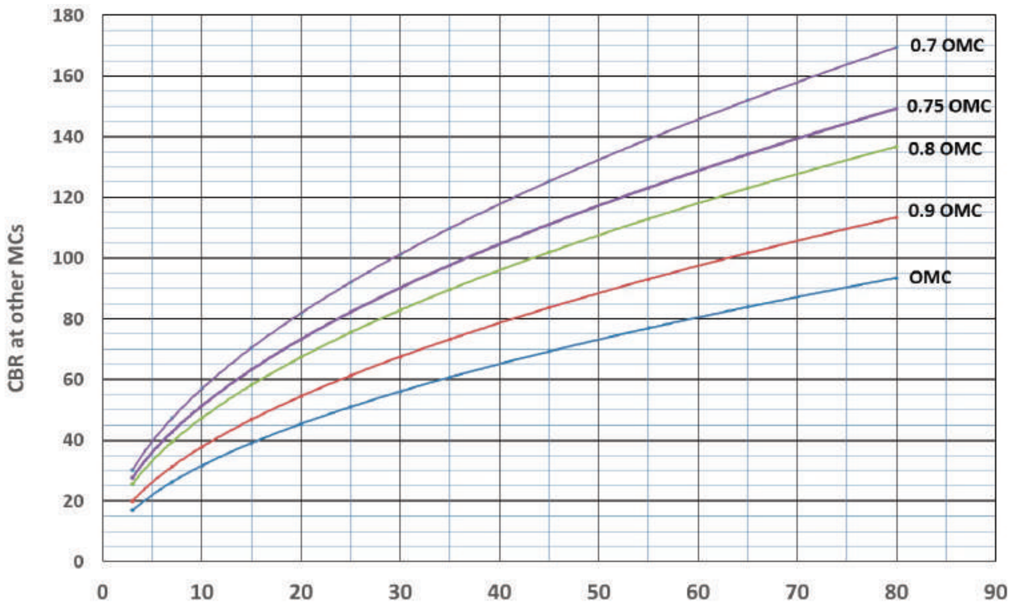



**Figure Appx B.4.2 Soaked CBRs at different moisture contents**

### Modified Structural Number

The effect of different subgrades can also be included in the structural number approach. The Modified Structural Number, including the subgrade contribution, is defined as follows:

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

Where:

SNC = Modified Structural Number of the pavement

SN = Structural number of the pavement (without subgrade contribution)

CBRs = *In situ* CBR of the subgrade

The Modified Structural Number (SNC) has been used extensively over the past 20 or 30 years and forms the basis for defining pavement strength in many pavement performance models. It should be used to identify the overall strength of each DCP test point in the old road if the subgrade is particularly variable.

**Table Appx B.4.1 Pavement layer strength coefficients**

| Layer Type | Condition | Coefficient |
|---|---|---|
| *Surface treatment* | | $a_i = 0.2$ |
| Granular unbound roadbase | Default | $a_i = (29.14\ CBR - 0.1977\ CBR^2 + 0.00045\ CBR^3)\ 10^{-4}$ |
| | CBR > 100% | $a_2 = 0.145$ |
| | CBR = 100% | $a_2 = 0.14$ |
| | CBR = 80% | |
| | With a stabilised layer underneath | $a_2 = 0.135$ |
| | With an unbound granular layer underneath | $a_2 = 0.13$ |
| | CBR = 65% | $a_2 = 0.12$ |
| | CBR = 55% | $a_2 = 0.107$ |
| | CBR = 45% | $a_2 = 0.1$ |
| Bitumen treated gravels and sands | Marshall stability = 2.5 MN | $a_i = 0.135$ |
| | Marshall stability = 5.0 MN | $a_i = 0.185$ |
| | Marshall stability = 7.5 MN | $a_i = 0.23$ |
| Cemented | Equation | $a_i = 0.075 + 0.039\ UCS – 0.00088(UCS)$ |
| | CB 1 (UCS = 3.0 – 6.0 MPa) | $a_i = 0.18$ |
| | CB 2 (UCS = 1.5 – 3.0 MPa) | $a_i = 0.13$ |
| Granular unbound sub-bases | Equation | $a_j = -0.075 + 0.184(\log_{10} CBR) – 0.0444(\log_{10} CBR)^2$ |
| | CBR = 40% | $a_3 = 0.11$ |
| | CBR = 30% | $a_3 = 0.1$ |
| | CBR = 20% | $a_3 = 0.09$ |
| | CBR = 15% | $a_3 = 0.08$ |
| | CBR = 10% | $a_3 = 0.065$ |
| Cemented | (UCS = 0.7 – 1.5 MPa) | $a_i = 0.1$ |

NOTE: Unconfined Compressive Strength (UCS) is stated in MPa at 14 days.

### Target Structural Numbers

When designing upgrading or rehabilitation it is necessary to determine the existing effective SN as described above at each test point and the required SN to carry the new design traffic. Tables Appx B.4.2 to Appx B.4.5 show the target values of SN and SNC for different subgrade conditions and for different traffic levels calculated from the design charts for roads with a thin bituminous surfacing. The difference between the required structural number and the existing structural number is the deficiency that needs to be corrected.

The final step is to determine uniform sections based on the strengthening requirements using a CUSUM method. For each uniform section the following percentiles of the strengthening requirements should be used:

1. Median for TLC 0.01
2. Upper 75th percentile for TLC 0.3
3. Upper 90th percentile for TLC 1.0

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 Appx B.4.6 is a guide to the treatments.

**Table Appx B.4.2 Structural Numbers (SN) for bituminous pavement design Chart 1**

| Subgrade CBR | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| SC1 (< 3%) | Special subgrade treatment required | | |
| SC2 (3-7%) | 1.05 | 1.95 | 2.30 |
| SC3 (8-14%) | 0.8 | 1.55 | 1.85 |
| SC4 (≥ 15) | 0.7 | 1.25 | 1.5 |

NOTE: These values exclude a contribution from the surfacing.

**Table Appx B.4.3 Structural Numbers (SN) for bituminous pavement design Chart 2**

| Subgrade CBR | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| SC1 (< 3%) | Special subgrade treatment required | | |
| SC2 (3-7%) | 1.05 | 1.80 | 2.15 |
| SC3 (8-14%) | 0.7 | 1.35 | 1.6 |
| SC4 (≥15) | 0.6 | 1.05 | 1.3 |

NOTE: These values exclude a contribution from the surfacing.

**Table Appx B.4.4 Required Modified Structural Numbers (SNC) for Chart 1**

| Subgrade CBR | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| SC2 | 1.1 | 2.0 | 2.35 |
| SC3 | 1.85 | 2.6 | 2.9 |
| SC4 | 2.2 | 2.75 | 3.05 |

**Table Appx B.4.5 Required Modified Structural Numbers (SNC) for Chart 2**

| Subgrade CBR | TLC 0.01 | TLC 0.3 | TLC 1.0 |
|---|---|---|---|
| SC2 | 1.1 | 1.85 | 2.2 |
| SC3 | 1.75 | 2.4 | 2.65 |
| SC4 | 2.1 | 2.55 | 2.8 |

**Table Appx B.4.6 Structural Deficiency Criteria**

| Structural deficiency based on appropriate percentiles | Action | Notes |
|---|---|---|
| 0.2 or negative | Maintain with a surface treatment (e.g. a surface dressing) | A thin granular overlay can be used to correct other road defects |
| 0.2 – 1.2 | New granular layer. The existing layers must be checked for quality (sub-base or road base). The minimum thickness of new road base should be 50 mm. | Some localised remedial works can be expected. A surface treatment is required. |
| 1.2 – 1.8 | The existing road base is likely to be only of sub-base quality and should be checked. Additional sub-base and a new road base are required. | Some localised remedial works will be needed. A surface treatment is required. |
| > 1.8 | The existing layers are likely to be less than sub-base quality, hence a new sub-base and road base are required. Chemically stabilising existing material should be considered. | Localised remedial treatment and a surface treatment are required. |