THE UNITED REPUBLIC OF TANZANIA

MINISTRY OF WORKS, TRANSPORT AND COMMUNICATION

LOW VOLUME ROADS

Manual

2016

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Preface

This **Low Volume Roads Manual (2016)** forms part of the Ministry's series of Road and Bridge Design documents. It applies specifically to the design of new, or upgrading of existing unpaved, roads which carry relatively low volumes of traffic, typically less than about 300 vehicles per day. Such roads would typically be classified as Class D5 to D8 (Collector, Feeder and Community roads) but, in particular cases, could also apply to higher classes of roads that carry relatively low volumes of traffic. Hence it al is a complementary document to the Pavement and Materials Design Manual (1999) which deals primarily with high volume roads, as well as to other existing manuals such as the Laboratory Testing Manual (2000), the Standard Specification for Road works (2000), the Field Testing Manual (2003) and the Road Geometric Design Manual (2011).

The purpose of the Manual is to serve as a nationally recognized document, the application of which is deemed to serve as a standard reference and ready source of good practice for the planning, investigation, design and construction of low volume roads (LVRs) in the country. In so doing, it will assist practitioners in developing the country's LVR network in a cost effective, environmentally optimized and sustainable manner.

The Manual was developed under the policy direction of a Roads Technical Committee comprising senior representatives from MOWTC, President's Office-Regional Administration and Local Government (PO-RALG), TANROADS and Road Fund Board (RFB). Technical guidance was provided by a Technical Working Group comprising representatives of the MOWTC, PO-RALG, TANROADS, Local Government Authorities, RFB and the local consulting industry.

The design standards set out in this Manual shall be adhered to unless otherwise directed by the MOWTC. However, it is emphasized that careful consideration to sound engineering practice shall be observed in the use of the Manual, and under no circumstances shall it waive professional judgment in applied engineering.

On behalf of the Ministry of Works, Transport and Communication, I would like to thank the UK Department for International Development (DFID) for their support of the development of the Manual through the African Community Access Programme (AFCAP) and the implementing organisation, Cardno Emerging Markets, UK. I would also like to extend my gratitude and appreciation to all of the roads sector stakeholders who contributed their time, knowledge and effort during the development of the Manual.

It is my sincere hope that this Manual will herald a new era in the more efficient and effective provision of low volume roads in Tanzania. In so doing, it will make a substantial contribution to the improved infrastructure of our country and, in the process, enhance economic growth and development and assist in poverty alleviation in the country.

Eng. Joseph M. Nyamhanga
Permanent Secretary (Works)
Ministry of Works, Transport and Communication

Foreword

The President's Office, Regional Administration and Local Government (PO-RALG), is responsible for providing national level coordination and support for programmes implemented at the local government level by local government authorities (LGAs). These LGAs are responsible for managing the classified district road network to an acceptable standard.

In order to cater specifically for the provision of Low Volume Roads (LVRs), i.e. those roads that typically carry less than about 300 vehicles per day - a category within which almost all the LGA roads fall - it was decided to develop a **Low Volume Roads Manual.** This Manual incorporates the most recent research and experience in the field of LVRs emanating from the African region and internationally. It promotes the use of locally available resources in an environmentally optimised manner, thereby offering considerable cost savings over conventional approaches. It also takes into account the needs of all road users, including non-motorised transport, with road safety being a primary consideration.

The aim of the Manual is to provide all practitioners with comprehensive guidance on the wide range of factors that need to be addressed in a holistic manner when undertaking the upgrading of unpaved roads to a paved standard. To this end, it serves as a standard reference and source of good practice for the planning, investigation, design and construction of low volume earth, gravel and paved roads.

PO-RALG expects all practitioners in the LGAs to adhere to the approaches set out in the Manual. This will ensure that a consistent, harmonized approach is followed in the provision of LVRs in the country.

The Manual will require periodic updating to take account of the dynamic nature of developments in LVR technology. PO-RALG would welcome comments and suggestions from any stakeholders as feedback on all aspects of the Manual during its implementation. All feedback will be carefully reviewed by professional experts with a view to amending future updates of the Manual.

Eng. Mussa I. Iyombe
Permanent Secretary
PO-RALG

Acknowledgements

The Ministry of Works, Transport and Communication wishes to acknowledge the support that was provided by the United Kingdom Department for International Development (DFID) for the preparation of the ***Low Volume Roads Manual.*** The project was carried out under the aegis of the Africa Community Access Programme (AFCAP) – a DFID-funded research programme that promotes safe and sustainable access for rural communities in Africa.

The development of the manual was guided by a Roads Technical Committee and a Technical Working Group comprising professionals from both public and private sector organizations. The manual was also reviewed by teams of experts from MOWTC and PO-RALG as listed below.

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Eng. Lukas Nyaki	Luptan Consult Ltd
Mr. Vincent Lwanda	PO-RALG

The contributions made by AMEND to the road safety aspects of the project, as well as the AFCAP peer review of the Manual undertaken by Jan Bijl are also gratefully acknowledged.

Project Management:
The project was managed by Cardno Emerging Markets, UK and was carried out under the general guidance of the AFCAP Technical Services Manager, Eng. Nkululeko Leta.

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

Mr. Michael Pinard (Team Leader)

Dr. John Rolt

Dr. Philip Paige-Green

Mr. Charles Overby

Eng. Abdul Awadh

Terminology

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

Pavement

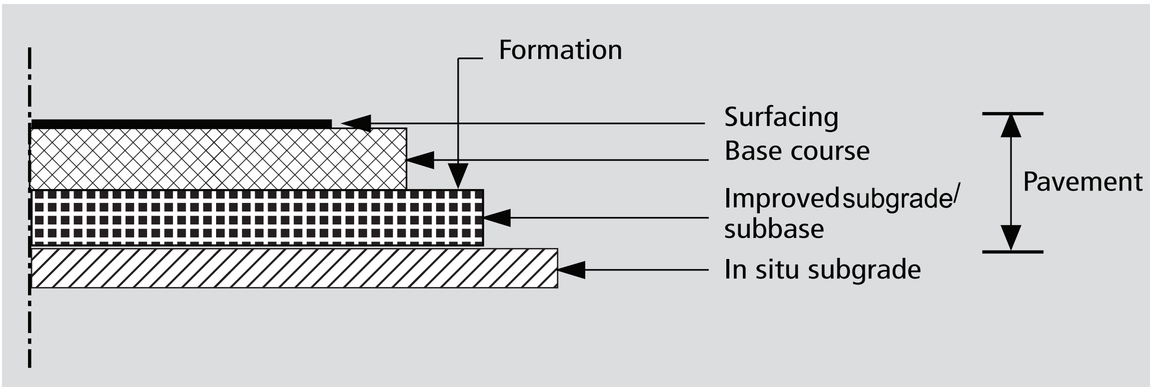

Figure 1: Main components of a LVSR pavement

Cross Section

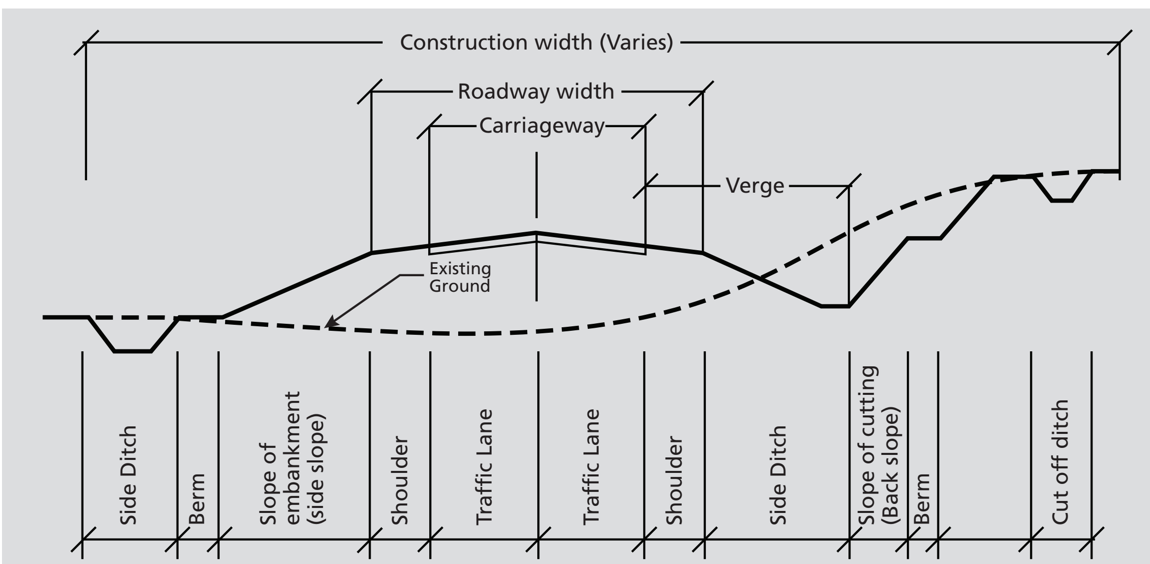

Figure 2: Road cross section

Drainage Elements

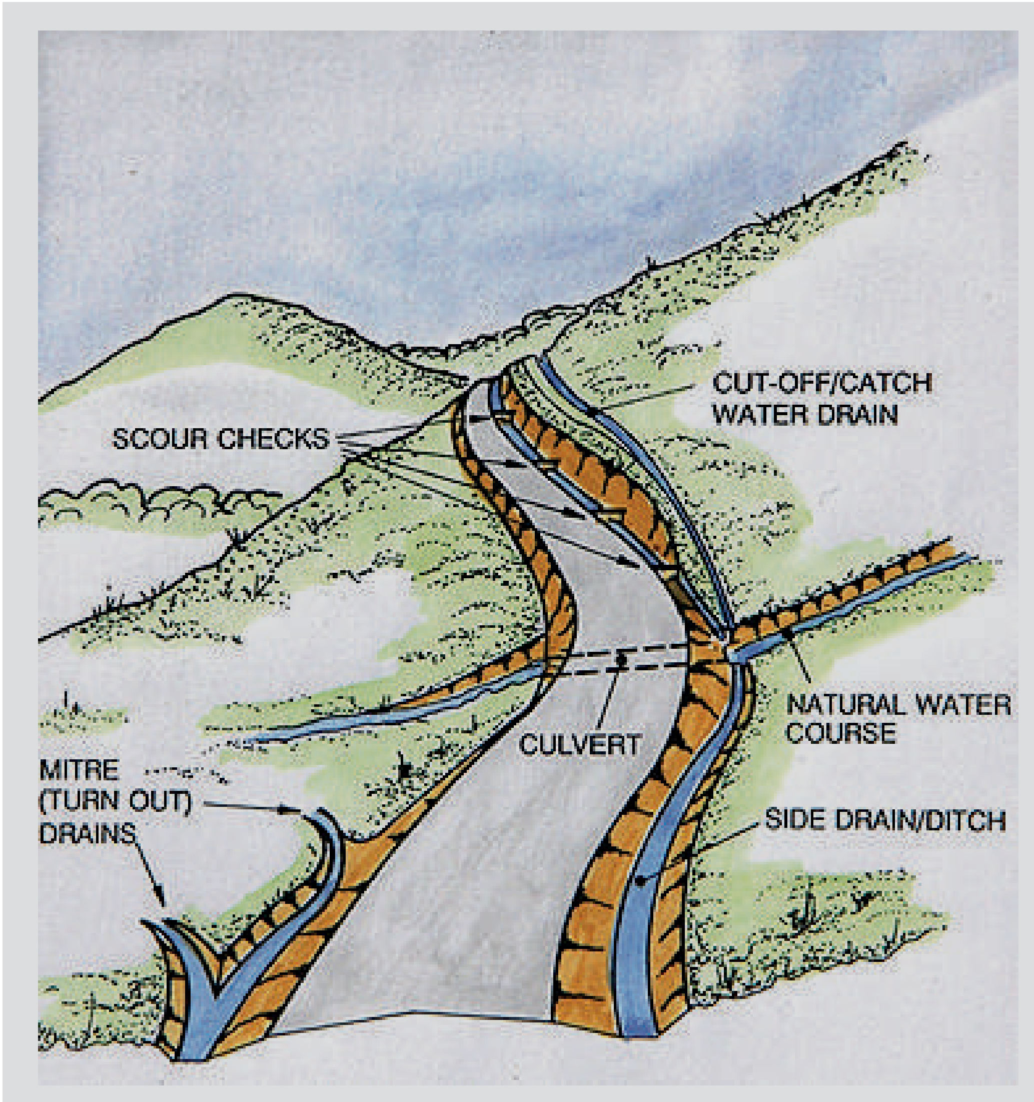

Figure 3: Main drainage elements

List of Symbols and Abbreviations

>	Greater than
<	Less than
%	Percentage
Ω	Resistance (ohms)
μsec	microsecond
μm	micrometre
$	US dollar

A

A	Area
AADT	Average Annual Daily Traffic
AFCAP	Africa Community Access Programme
AASHTO	American Association of State Highway and Transport Officials
ACV	Aggregate Crushing Value
ADT	Average Daily Traffic
AI	Accessibility Indicator
AIDS	Acquired Immune Deficiency Syndrome
AIV	Aggregate Impact Value
ALD	Average Least Dimension
ARL	Areal Reduction factor (for rainfall catchments)
ASIST	Advisory Support Information Services and Training
ASTM	American Society for Testing Materials

B

B	Width of a box culvert
BDS	Bid Data Sheet
BS	British Standard

C

°C	Degrees Celsius
C	Rainfall catchment run off coefficient
CB	Cemented Base
CBR	California Bearing Ratio
CEC	Cation Exchange Capacity
CESA	Cumulative Equivalent Standard Axles
CI	Complementary Intervention
CMA	Cold Mix Asphalt

CP	Collapse Potential
CPT	Cone Penetrometer Test
CML	Central Materials Laboratory
CS	Cape Seal
CSIR	Council for Scientific and Industrial Research
CUSUM	Cumulative Sum

D

D	Diameter of culvert or height of box culvert
DBM	Dry Bound Macadam
DC	Design Class
DCP	Dynamic Cone Penetrometer
DES	Discrete Element Surfaces
DESA	Mean daily Equivalent Standard Axles
DF	Drainage Factor
DFID	Department for International Development
DN	The average penetration rate in mm/blow of the DCP through a pavement layer
DOS	Double Otta Seal
DSD	Double Surface Dressing
DSN_{800}	Number of DCP blows required to penetrate top 800 mm of a pavement
DSS	Double Sand Seal

E

E	East
EF	Equivalence Factor
EIA	Environmental Impact Assessment
EIP	Environmental Impact Plan
EIS	Environmental Impact Study
EMP	Environmental Management Plan
ENS	Engineered Natural Surface
ESA	Equivalent Standard Axle (80 kN)
EOD	Environmentally Optimized Design
ESIA	Environmental and Social Impact Assessment
ESP	Exchangeable Sodium Percentage
ETB	Emulsion Treated Base
EVT	Equiviscous Temperature

F

F	Weighing Factor
FACT	Fines Aggregate Crushing Test
FHWA	Federal Highway Administration
FIDIC	International Federation of Consulting Engineers
FMC	Field Moisture Content

G

g	Gradient
G60	Gravel with CBR of 60%
GB	Granular Base
Gc	Grading Coefficient
GE	Grading Envelope
GM	Grading Modulus
GPS	Global Positioning System
GVM	Gross Vehicle Mass

H

H	Horizontal
He/D	Headwater depth in terms of culvert diameter, D
HDM-4	Highway Development and Management Model - 4
HGV	Heavy Goods Vehicle
hmin	Height minimum
HV	Heavy Vehicle
HIV	Human Immunodeficiency Virus
HPS	Hand Packed Stone
HVR	High Volume Road

I

I	Intensity of Rainfall
IKONOS	Commercial earth observation satellite
ILO	International Labour Organization
IRAP	Integrated Rural Accessibility Planning
ITB	Instructions to Bidder

K

k	Coefficient of permeability (nanometre/second)
K	Horizontal distance required to achieve a 1% change in grade on a crest curve

kg	kilogram
kg/m^3	kilogram per cubic metre
kJ	kilojoule
km	kilometre
km^2	Square kilometre
km/h	kilometre per hour
kN	kilonewton
kPa	kilopascal

L

LAA	Los Angeles Abrasion Value
LBM	Labour Based Methods
LCC	Life Cycle Cost
LGA	Local Government Authority
LGV	Light Goods Vehicle
LL	Liquid Limit
LVR	Low Volume Road
LVSR	Low Volume Sealed Road

M

m	metre
m^2	Square metre
m^3	Cubic metre
m^3/s	Cubic metre per second
MAASHTO	Modified AASHTO
MC	Moisture Content
MC	Medium Curing
MDD	Maximum Dry Density
MESA	Million Equivalent Standard Axles
Mg/m^3	Megagram per cubic metre
MGV	Medium Goods Vehicle
mm	millimeter
MN	Meganewton
MOID	Ministry of Infrastructure Development
MOW	Ministry of Works
MPa	Megapascal

m/s	metre per sec
MS	Medium Setting

N

n	Roughness coefficient of water courses
N	Design period in years
N/A	Not Applicable
NGO	Non-Government Organisation
NMT	Non-motorised Traffic
NPV	Net Present Value
NRC	Non-reinforced concrete
n/s	Not specified

O

O/D	Origin & Destination
OGL	Original Ground Level
OMC	Optimum Moisture Content
ORN	Overseas Road Note
OWP	Outer Wheel Path

P

P	Axle load
P075	Percentage material passing the 0.075 mm sieve
PCU	Passenger Car Unit
PDM	Pavement Design Manual
pH	Scale indicating how acidic or basic a substance is and ranges from 0 to 14
PI	Plasticity Index
PIARC	Permanent International Association of Road Congresses
PL	Plastic Limit
PM	Plastic Modulus
PMO-RALG	Prime Minister’s Office-Regional Administration and Local Government
PRA	Participatory Rural Appraisal
PS	Provisional Sum
PSD	Particle Size Distribution
PV	Present Value

Q

q	Flow of water in a channel (m^3/s)
Q	Water discharge volume flow rate
QA	Quality Assurance
QC	Quality Control

R

r	Used for growth rate as a percentage
R	Hydraulic depth
RC	Reinforced Concrete
RED	Roads Economic Decision Model (a World Bank Model)
RFP	Request for Proposal
RGDM	Road Geometric Design Manual
RRA	Rapid Rural Appraisal
RRPM	Retro-reflective Pavement Markers
RS	Rapid Setting
RSE	Road Safety Education
RTC	Roads Technical Committee

S

S	Hydraulic gradient
S2, S3	Subgrade Classes 2, 3 etc.
SA	South Africa
SADC	Southern African Development Community
SAR	Sodium Adsorption Ratio
SID	Spot Improvement Design
SLS	Single Slurry Seal
Sm^{-1}	Conductivity (Siemens/metre)
SME	Small and Medium Enterprise
SN	Structural Number
SNC	Structural Number with subgrade contribution
SNP	Adjusted Structure Number
SPT	Standard Penetration Testing
SOS	Single Otta Seal
Sp	Shrinkage Product
SS	Slow Setting

SSD	Single Surface Dressing
SSS	Single Sand Seal
STD	Sexually Transmitted Disease
SURF	Economic warrants for surfacing unpaved roads

T

t	time
T	Surfacing thickness (mm)
Tc	Time of Concentration (Seconds)
TLB	Tractor Load Bed
TLC	Traffic Load Class
ToR	Terms of Reference
TRL	Transport Research Laboratory
Tsh	Tanzanian Shilling
TWG	Technical Working Group

U

UCS	Unconfined Compressive Strength
UK	United Kingdom
UKAID	Development Assistance provided by the UK Department for International Development
URC	Unreinforced Concrete
USA	United States of America

V

v	Velocity (m/s)
V	Cross sectional average velocity (of water flow)
VEF	Vehicle Equivalence Factor
VHGV	Very Heavy Goods Vehicle
VOC	Vehicle Operating Costs
vpd	Vehicles per day

W

WA	Water Absorption

Contents

PART C: INVESTIGATIONS

PART D: DESIGN

PART E: CONSTRUCTION

Part A: Introduction

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1.1 BACKGROUND

Low volume roads (LVRs), defined as those roads which, over their design life, are required to carry an average of about 300 motor vehicles per day, and less than about 1.0 million equivalent standard axles (MESA) in one direction, comprise a substantial proportion of the road network in Tanzania (> 75%). The importance of this category of roads extends to all aspects of the economic and social development of the country, particularly in the rural areas of the country where a large percentage of the population live (> 80%) and where agriculture is the dominant economic activity. In such a situation, Tanzania's LVRs fulfil a critical function in that they generally provide the only means of access to these rural communities and provide for the essential mobility of people and movement of goods from the fields to the market place.

Whilst there are potentially significant life-cycle benefits to be achieved from upgrading unsurfaced LVRs roads to a paved standard, the cost of doing so following traditional standards and specifications can be prohibitive. This is because these approaches tend to be overly conservative and ill-matched to the dictates of the local road environment. As a result, they are generally far too costly for application to most rural road networks in Africa. This has led to a need to develop a new **Low Volume Roads Manual** in Tanzania that takes account of the many advances in LVR technology that have taken place in the region and internationally.

1.2 PURPOSE

The main purpose of this Manual is to provide practitioners with the requisite tools for undertaking a holistic, rational and affordable approach to the provision of LVRs in Tanzania. Such an approach is aimed at minimising the life-cycle costs of road provision by taking account of the many locally prevailing road environment factors that impact on the performance of LVRs.

The Manual draws on the outputs of a number of research and investigation projects that have been carried out in the region since the 1990s. The corroborative findings of these projects provide a wealth of performance-based information that has advanced previous knowledge on various aspects of LVR technology. This has allowed state-of-the-art guidance to be provided in the Manual which is expected to serve as a nationally recognized document, the application of which will harmonize approaches to the provision of LVRs in Tanzania. The Manual is intended for use by road authorities at central and local government level, as well as by private sector consultants.

1.3 SCOPE

The Manual consolidates in one document the latest approaches to the provision of LVRs that mirror the sequential activities that are typically undertaken in providing such roads, i.e. activities that progress from the planning stage through to investigations, design and implementation. In so doing, it complements and links to the latest versions of other relevant Manuals in Tanzania that include:

- Pavement and Materials Design Manual (1999).
- Laboratory Testing Manual (2000).
- Standard Specification for Road Works (2000).
- Field Testing Manual (2003).
- Road Geometric Design Manual (2011).

The Manual caters for a range of road types, from basic earth tracks to bituminous sealed roads, that are typically found in rural environments. The environmentally optimised approach to the design of such roads is a key feature of the Manual that can be applied to interventions that deal with individual critical sections, or to the total length of a road link. In the latter case this could comprise different design options along the total road length. The design of road pavements and surfacings in sub-urban/urban environments is also addressed, but that for road/sub-surface drainage, stormwater management systems and access to dwelling units in such built-up areas is outside the scope of the Manual.

1.4 DEVELOPMENT

The development of the Manual was overseen by a wide range of stakeholder organisations in Tanzania including representatives from the following organisations.

- Ministry of Works, Transport and Communication (MOWTC).
- TANROADS.
- President's Office – Regional Administration and Local Government (PO-RALG).
- Road Fund Board.
- Association of Consulting Engineers Tanzania.

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

1.5 STRUCTURE

The Manual is divided into five separate parts which follow the distinct stages of LVR provision as presented illustrated in Table 1-1.

Table 1-1: Structure and content of manual

Part	Chapter
A. Introduction	1. General Introduction 2. Low Volume Roads in Perspective 3. Physical Environment
B. Planning	4. Rural Accessibility Planning
C. Investigations	5. Site Investigations 6. Geotechnical Investigations and Design 7. Construction Materials 8. Traffic
D. Design	9. Geometric Design 10. Road Safety 11. Hydrology and Drainage Structures 12. Drainage and Erosion Control 13. Structural Design: Paved Roads 14. Structural Design: Unpaved Roads 15. Surfacing 16. Life-Cycle Costing
E. Construction	17. Construction, Quality Assurance and Control 18. Borrow Pit Management 19. Technical Auditing

1.6 BENEFITS OF USING THE MANUAL

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

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

1.7 SOURCES OF INFORMATION

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

1.8 UPDATING OF THE MANUAL

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

- Any proposed amendments should be sent to the Director of Roads, MOWTC motivating the need for the change and indicating the proposed amendment.
- Any agreed changes to the Manual will be approved by the MOWTC after which all stakeholders will be advised accordingly.

1.9 DEPARTURE FROM STANDARDS

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

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

The designer must submit all major and minor Departures from the Standards and his/her proposal for approval. If the proposed Departures from the Standards are acceptable, such departures will be given approval by the Director of Roads, MOWTC.

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CONTENTS

2.1 INTRODUCTION

2.1.1 Background

The traditional approaches to the provision of LVRs in many tropical and sub-tropical countries tend to be based on technology and research carried out in external environments that are not reflective of those that prevail in these countries. While these "standard" approaches might still be appropriate for much of the main trunk road network, they remain conservative, inappropriate and too costly for application on much of Tanzania's rural road network. Thus, in facing the challenges of improving and expanding the country's LVR more appropriate approaches need to be considered.

The approach to the design of LVRs follows the general principles of any good road design. However, there are a number of important differences from the traditional approaches that need to be appreciated by the designer in order to provide designs that will meet with the multiple social, economic and environmental requirements of Tanzania in a sustainable manner.

2.1.2 Purpose and Scope

The main purpose of this chapter is to place in broad perspective the various factors that affect the provision of LVRs. These factors include:

- The particular characteristics of LVRs.
- The LVR design philosophy.
- Various implementation considerations.

2.2 CHARACTERISTICS

2.2.1 General

Apart from the traffic volume and traffic loading characteristics indicated in *Chapter 1 – General Introduction,* there are a number of other specific characteristics of LVRs that affect the manner of their provision that need to be fully appreciated as follows:

- They are constructed mostly from naturally-occurring, often "non-standard", moisture-sensitive materials.
- Pavement deterioration is driven primarily by environmental factors, particularly moisture, with traffic loading being a relatively lesser influential factor, and drainage being of paramount importance.
- The alignment may not necessarily be fully "engineered", especially at very low traffic levels, with most sections following the existing alignment for which there must be careful attention to road safety.
- There is a need to cater for a significant amount of motorcycles, which commonly make up the vast majority (up to 90%) of all motorised vehicles, as well as non-motorised traffic, coupled with a focus on the adoption of a range of low-cost road safety measures.
- There will be variable travelling speeds that will seldom exceed about 80 km/h, as dictated by local topography.
- An appreciation that conventional economic analysis (focussing on consumer surplus or road user savings) often cannot fully justify the investment of public funds in the provision or improvement of LVRs and that it can be relatively difficult to quantify the many other benefits that are of a broad socio-economic and environmental nature.
- Environmental and sustainability factors are important components of economic analysis and life-cycle costing.

Based on the above characteristics of LVRs, it should be apparent that certain types of roads do not fall under the heading of LVRs as defined above. For example:

- A functionally classified trunk or regional road carrying less than 300 vpd and less than 1 million MESA over its design life would not necessarily be classified as a LVR. This is because the level of serviceability that it would be expected to provide would be dictated by its function (characterized by a relatively high design speed and corresponding geometric design) and the low risk of failure.
- A haul road serving, for example, an industrial, mining, agricultural or quarry area, in which heavy loads are transported for just a few months of the year during the rainy season even though the design traffic loading may be less than 1 MESA.

A holistic appreciation of the attributes that characterise LVRs will guide designers in producing more appropriate designs with an emphasis on using a fit-for-purpose, context sensitive, environmentally optimised approach to their design and construction. This will place an onus on the design engineer to provide an affordable road that meets the expected level of service at least life-cycle cost based on a full understanding of the local environment and its demands, and to turn these to a design advantage.

The unique characteristics of LVRs as described above challenge conventional engineering practice in a number of aspects, including materials and pavement design, geometric design, drainage, road safety and maintenance, to which particular attention should be paid in the development of new guidelines and manuals.

2.2.2 Earth and Gravel Roads (Unpaved Roads)

Surfacing material on earth road needs to resist wear and abrasion in dry weather and promote surface drainage and run-off in wet weather. Under traffic they need to resist whip off, dust generation and be stable enough when compacted to resist deformation. However, during the rainy season, even at relatively low traffic levels (<50 vpd) earth roads often become impassable due to weakening of the underlying soil coupled with erosion and scouring which may prevent access to any form of motorized traffic.

Gravelling of earth roads can improve passability during the rainy season. However, gravel is a non-renewable natural resource which is used as a sacrificial layer and must be replaced periodically. Moreover, gravel roads require a continuous cycle of reshaping and regraveling to maintain the required running surface and the desired level of service. These requirements place a major burden on road authorities and, where economically viable, a sealed road option is eminently preferable, as addressed in this Manual.

Dust is also a major problem on earth and gravel roads and adversely affects other road users, pedestrians, nearby houses and shops as well as crops near the road. Thus, dust can present significant and costly social (cleanliness), health (eye and respiratory hazards), environmental (crop and natural habitat damage) and economic (vehicle and equipment damage, pedestrian and vehicle safety) consequences. Approaches to alleviate dust problems, particularly in populated areas, through the application of appropriate types of surfacings are offered in the Manual.

Figure 2-1: Dust generation on gravel road

Figure 2-2: Crops adversely affected by dust

The major technical challenges associated with the provision of unpaved roads include: a need for durable and functional water crossings; surfacing with materials that provide the desired level of service; and effective maintenance management. These challenges are recognised in the Manual and options and solutions are offered to mitigate and manage potential problems.

2.2.3 Paved Roads

The approaches adopted for the provision of LVRs differ in a number of fundamental respects from roads carrying higher traffic volumes. In particular, the relative influences of road deterioration factors are significantly different for LVRs compared with higher volume roads. The appropriate design options for LVRs therefore need to be responsive to a wider range of factors captured in the road environment, the most critical being the internal and external drainage of the pavement.

In light of the above, the role of the design engineer is to take cognisance of the potentially cost-reducing developments that have taken place in LVR technology as a basis for producing environmentally optimised designs as described previously.

2.2.4 Road Classification

Based on the partly administrative and partly functional road classification system used in Tanzania, Table 2-1 shows those classes of roads which, for geometric design purposes, may be defined as low volume roads.

Table 2-1: Classification of low volume roads – geometric classes

Road Design Class	AADT at Mid Design Life (vpd)	Design Traffic Loading < 1.0 MESA			
		Functional Class			
		B	C	D	E
DC 5	200 - 400	*	**	**	**
DC 6	50 - 200	N/A	**	**	**
DC 7	20 - 50	N/A	**	**	**
DC 8	< 20	N/A	**	**	**

Notes: * Most unlikely to be upgraded to a Class D4 road in the foreseeable future.

** Less than about 20% of HGVs; 3 or less axles; GVM ≤ 15 tonnes.

As would be apparent from Table 2-1, and as illustrated in Figure 2-3, all District roads fulfilling a primarily access function and classified as Collector, Feeder or Community roads (functional Classes C, D and E) and some National roads, fulfilling a partly access/mobility function (functional Class B), would typically qualify as a LVR for design purposes.

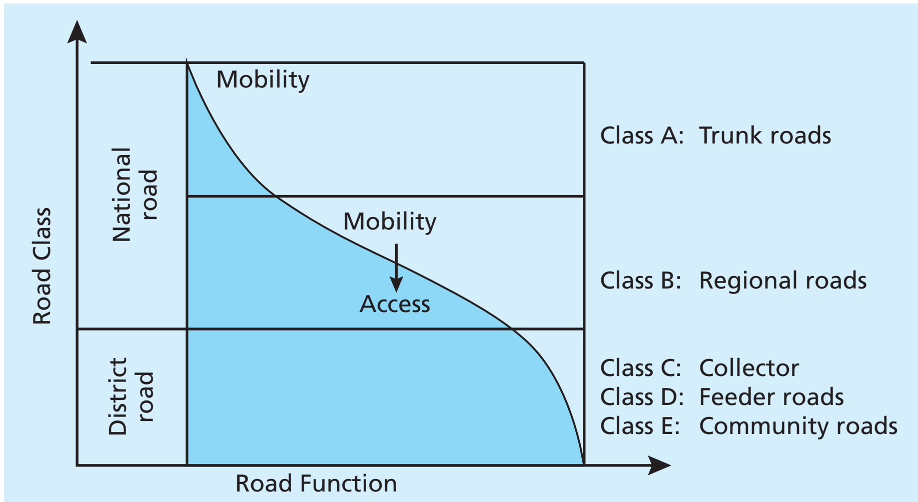

Figure 2-3: Relationship between road class and road function

Thus, a National road (functional Class A), even though the traffic loading over its design life may be less than 1 MESA, would not be classified a LVR, because the level of service that it is expected to provide (primarily a mobility function) is characterised by a relatively high design speed and related geometric design standard. Similarly, a haul road serving, for example, an industrial, mining or quarry area, in which heavy loads are transported for a few months of the year during the rainy season would not be classified as a LVR even though the road may be functionally classified as Class C, D or E and the design traffic loading may be less than 1 MESA.

For structural design purposes, the traffic load class, rather than road design class, is used with the relationship between these two parameters being shown in Table 2-2.

Table 2-2: Classification of low volume roads – structural classes

Traffic Load Class (TLC)	Cumulative traffic load during design life (MESA)
TLC 1.0	0.5 – 1.0
TLC 0.5	0.3 – 0.5
TLC 0.3	0.1 – 0.3
TLC 0.1	0.01 – 0.1
TLC 0.01	< 0.01

2.3 APPROACH TO DESIGN

2.3.1 General Approach

Whilst the approach to the design of LVRs follows the general principles of any good road design practice, the level of attention and engineering judgement required for optimal provision of such roads tends to be higher than that required for the provision of other roads. This is because optimising a design requires a multi-dimensional understanding of all of the project elements and in this respect all design elements become context specific. The design team therefore needs to be able to work outside their normal areas of expertise and to understand implications of their recommendations or decisions on all other elements of the design. This will require:

- A full understanding by the design engineer of the local environment (physical and social).
- An ability to work within the demands of the local environment and to turn these to a design advantage.
- Recognition and management of risks.
- Innovative and flexible thinking through the application of appropriate engineering solutions rather than following traditional thinking related to road design.
- A client who is open and responsive to innovation and practitioners who are able to explain and convince the client to adopt LVR practice as a cost-effective solution to road infrastructure development.

Design engineers are traditionally conservative and build in factors of safety that cater for their perceptions of risk. This approach prevents the application of innovation, uses scarce or inappropriate resources and results in high financial costs for the client and the country. There is also often a temptation to provide or upgrade roads to a future level of service not justified by the socio-economic or road user requirements. This type of approach unnecessarily absorbs available resources and prevents the provision of additional access to other constituents.

Generally, the design of LVRs will be guided by the client and will build on information and data collected during the planning stages of the project. The client will have a project in mind with an indicative budget, desired service level and route alignment. The client may also have views and guidance on how to apportion the works and other social, environmental and time constraints.

Ultimately, the design engineer is responsible for developing the project within and around the boundaries and limitations defined by the client, whilst at the same time highlighting any issues and problems that may limit or require adjustment of expectations. In so doing, the engineer will need to draw on his/her engineering skills, judgement and local experience to produce an appropriate design without incurring unacceptable levels of risk.

This Manual provides the design engineer with the requisite tools that will provide the client with an optimised design based on the financial, technical and other constraints that define the project.

2.3.2 Upgrading Strategy

The decision as to when a road should be upgraded to a higher (more expensive) standard (service level) is often not a simple choice between a paved and an unpaved road. In practice, a spot improvement strategy should be adopted as discussed in the Manual. Thus, over a period of time, a road will often undergo a number of improvements or upgrading iterations during its use. Figure 2-4 illustrates the various types of pavement 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 standard. Ultimately, a life-cycle cost analysis will be required to assist in determining the optimum upgrading option.

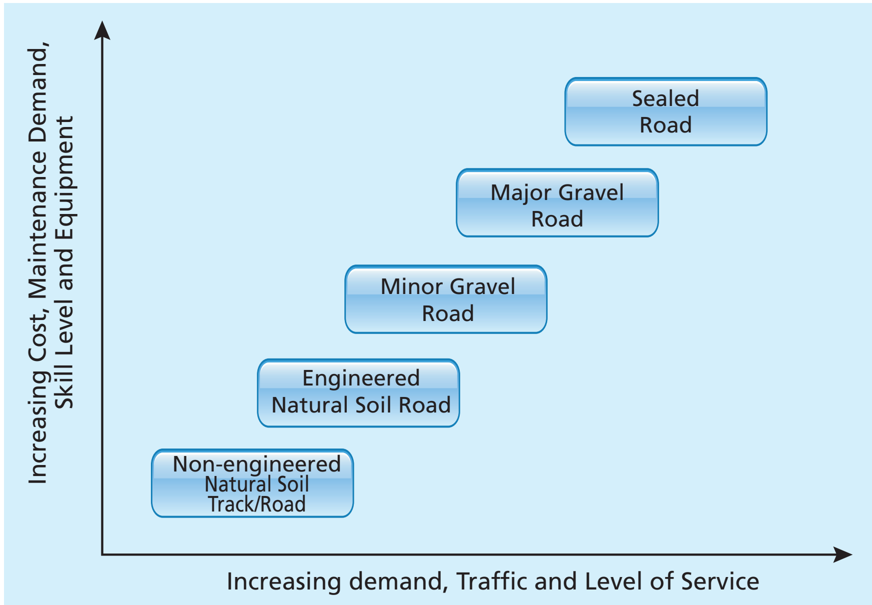

Figure 2-4: Upgrading stages of low volume roads

Figure 2-5: Non-engineered natural soil track/road

Figure 2-6: Engineered gravel road

Figure 2-7: Concrete strip road

Figure 2-8: Low volume paved road

2.3.3 Influence of Road Environment

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

Typical road environment factors that impact on the LVR provision process are presented in Figure 2-9 and are covered in more detail in various parts of the Manual.

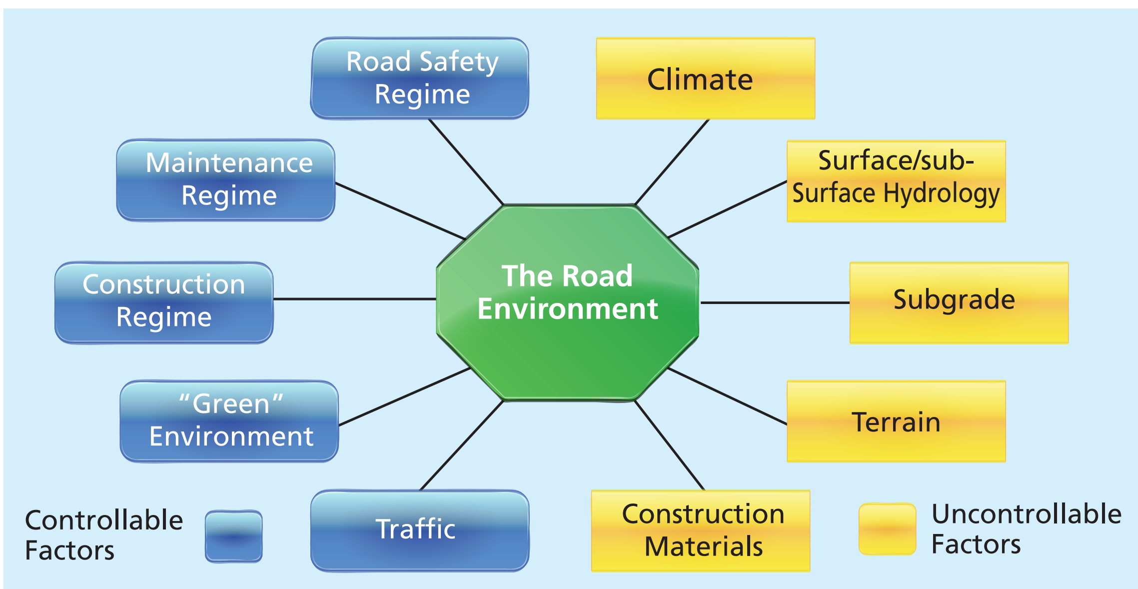

Figure 2-9: Various road environment factors affecting design

2.3.4 Road Deterioration Factors

Environmental factors - essentially in terms of moisture and temperature - have a profound effect on pavement performance, as illustrated in Figure 2-10. This is particularly the case with low volume roads for which pavement deterioration is influenced mainly by how the road responds to environmental factors, such as moisture changes in the pavement layers, fill and subgrade, rather than to traffic, as is the case with HVRs. Thus, particular attention needs to be paid to the influence of moisture in the design of LVR pavements and to the adoption of appropriate drainage measures to mitigate against the adverse effects of moisture ingress into the pavement structure. The environmentally optimised approach to LVR design thus concentrates on ensuring that the environmental factors are fully appreciated and carefully controlled during the design, construction and maintenance of the road

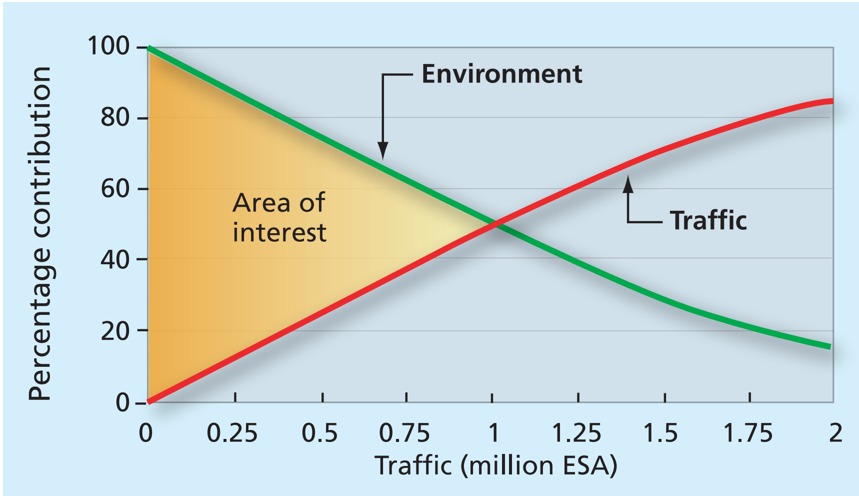

Figure 2-10: Traffic loading versus dominant mechanism of pavement distress (schematic only)

2.3.5 Environmentally Optimised and Spot Improvement 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 as steep gradients, wet and marshy areas as well as passage over easy terrain. This approach requires consideration of a range of options for improving or creating LVR access – from dealing with individual critical areas on a road link (Spot Improvement Design (SID), to providing a comprehensive set of interventions along the entire road link. which, in the latter case, could comprise different design options along its length.

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 to a uniform service level. 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 should be achievable with the materials, plant and contractors that are 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 chosen 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 2-11.

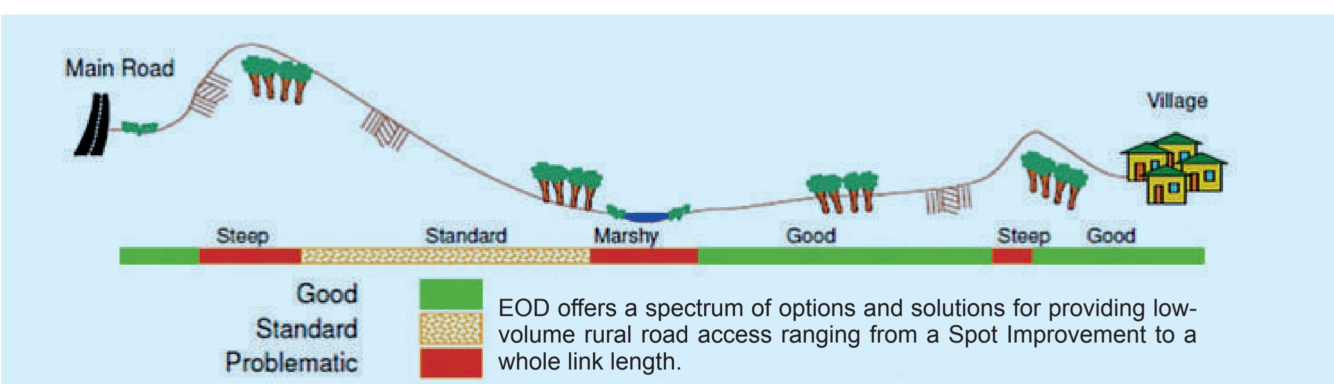

Figure 2-11: Environmentally optimised and spot improvement design

2.3.6 Surface Improvement Technology

Gravel and earth roads are particularly vulnerable to the effects of the road environment. A range of more durable surfacing options, other than gravel or earth are available for LVRs. These include thin bituminous surfacings, and non-bituminous surfacings such as cobblestone, hand packed stone and even thin concrete. The selection, design and use of the various surfacing options are described in detail in this Manual in *Chapter 15 – Surfacings.*

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

2.3.7 Design Reliability

There is an inherent uncertainty associated with the design of a LVR road pavement due to a number of factors including:

- Utilisation of mostly naturally occurring materials that exhibit widely varying properties and characteristics, with the performance of such materials being highly influenced by uncontrollable environmental factors, such as temperature and rainfall.
- The uncertain nature of the forces that cause the road to deteriorate, primarily weather, but also traffic.

Because of this uncertainty, an appropriate measure of the anticipated performance of the proposed pavement – its design reliability – must be ascribed to the pavement design process.This design reliability may be viewed as the probability that the pavement, when constructed, will reach its perscribed serviceability level, usually in terms of acceptable rutting, cracking and roughness, by the end of its design life.

Different levels of design reliability normally apply to different categories of road, with the chosen reliability levels, and related termuinal serviceability criteria, being commensurate with the standard and functional serviceability of the road. Thus, in the case of HVRs, non-attainment of the perscribed terminal service condition is likely to have relatively costly consequences in terms of disruption to traffic and increased vehicle operating costs. Consequently, the level of design relaibility would be relatively high, typically 90 to 95% for the highest classes of road.

In the case of LVRs, the consequences of non-attainment of the prescribed terminal service condition are seldom significant and a lower degree of design reliability, typically 50%, can be accepted as appropriate. This is because the terminal serviceability criteria can be lower, without adverse economic effects. To design a LVR with a higher level of reliability, and related higher terminal serviceability criteria, would not be cost effective as the consequences of undertaking the required maintenance over the affected length of the road would be significantly less than the cost of using more expensive pavement materials. This rationale assumes, of course, that reasonable maintenance will be carried out during the life of the LVR.

2.4 SUSTAINABILITY

2.4.1 Context Sensitivity

In addition to ensuring that the design developed is technically appropriate and is within the financial envelope, the design engineer needs to bear in mind other factors that could influence the success of the LVR design approach, its implementation and its long term sustainability. This requires a broadly focused, multi-dimensional and context sensitive approach in which a number of other influential factors are considered, as illustrated in Figure 2-12.

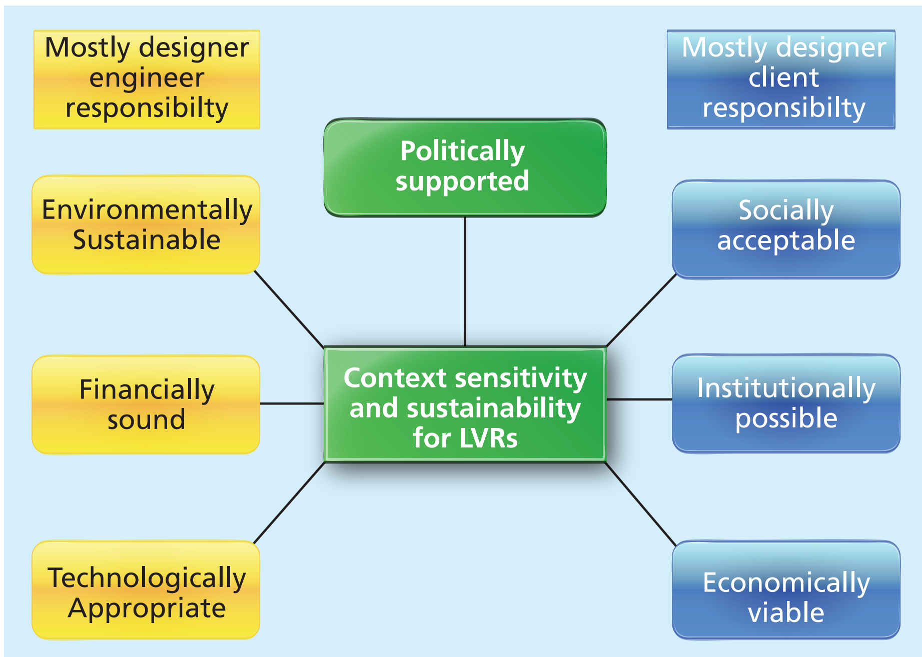

Figure 2-12: Framework for sustainable provision of LVRs

Political support

Demand for low volume road provision needs to be framed under a national policy driven by Government and should be supported at the highest level. The cross-sectoral influence of low volume road provision and its role in under-pinning other sectoral development strategies and poverty alleviation programmes should be highlighted, quantified and understood.

- The approach adopted for LVR provision should complement national plans, policies and strategies and should be responsive to wider needs and demands, including:
  - the social and economic goals of poverty alleviation and development;
  - increasing rural accessibility;
  - the use of appropriate technology, promotion of the domestic construction industry and employment creation;
  - protection of the environment;
  - Cost minimisation and improved efficiency.

There is a need to maintain dialogue with political and public stakeholders in order to highlight the advantages of design approaches and alternative, often unfamiliar, solutions selected for LVR provision. The language used for advocacy should be carefully chosen and should avoid negative connotations such as “low standard”; “low cost”, “marginal” and “relaxed”.

Social acceptance

Provision of low volume rural road networks should be managed in a way that:

- Ensures community participation in planning and decision making.
- Eliminates gender bias and promotes participation by women in the roads sector.
- Promotes activities and investment for sustainable livelihoods (including Complementary Interventions *(refer to Chapter 4 – Rural Accessibility Planning).*
- Promotes road safety in all aspects of LVR provision.
- Supports cost-effective labour-based and intermediate equipment methods of construction and maintenance.
- Minimises resettlement and mitigates unavoidable resettlement through appropriate compensation.

Institutional capacity

Road authorities and clients should:

- Promote institutional, economic and technical understanding in the provision and management of LVRs.
- Promote commercial management practices.
- Provide a conducive environment for the development of local contractors.
- Ensure that design, construction and maintenance approaches for LVRs are represented on all tertiary civil engineering training curricula.

Technology choice

Technologies for designing, constructing and maintaining LVRs should:

- Employ appropriate design standards and specifications.
- Take into account the users of the roads, in particular for roads with high numbers of motorcycles and non-motorised users.
- Utilise intermediate equipment technology options and reduce reliance on heavy equipment imports.
- Promote road construction and maintenance technologies that allow for community participation and creation of employment opportunities.
- Use types of contract that support the development of domestic contractors and consultants.
- Be robust to the vagaries of climate and recognise potential impacts of a changing climate.

Economic viability

Economic appraisal for LVRs should:

- Employ tools for LVRs that should be capable of quantifying social, economic and environmental costs and benefits.
- Ensure investment decisions are based on an assessment of whole life costs.

Financially sound

Sustainable provision of LVRs depends on the sustainable provision of funding to the sector in that:

- Roads should not be upgraded to engineered standards if funding is not in place for routine and periodic maintenance requirements.
- Designs should not be forwarded that require excessive allocation of maintenance resources.

Environmentally sustainable

The design and management of LVRs should:

- Minimise the physical impacts of construction and maintenance activities on the natural environment.
- Take account of socio-cultural impacts (community cohesion).
- Minimise the carbon footprint.
- Optimise resource management and allow for recycling of non-renewable materials.
- Minimise impacts and emissions that might contribute to climate change.

2.4.2 Risk Factors

In the design and construction of any road there are risk factors that need to be recognised and managed. Those for LVRs and those for HVRs differ in some respects but many are common to both. Those that are most pertinent to LVRs include:

- Quality of the materials (strength and moisture susceptibility).
- Construction control (primarily compaction standard).
- Environment (particularly drainage).
- Maintenance standards (drainage and surfacing).
- Vehicle loading.

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

2.5 IMPLEMENTATION

2.5.1 Level of Influence of Key Activities

The various LVR implementation activities addressed in this Manual, when aggregated into the main components of the project cycle – planning, investigation, design, construction and maintenance - all exert a decreasing level of influence on the quality, and hence cost efficiency of the final output in terms of the total life-cycle cost of LVR provision. This concept is illustrated in Figure 2-13.

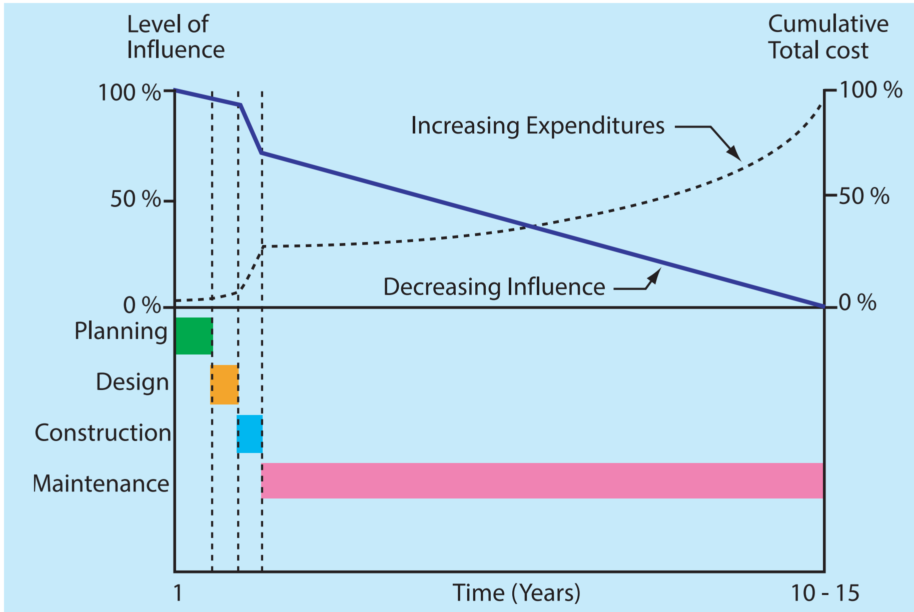

Figure 2-13: Level of Influence of activity in relation to life-cycle cost of LVR provision

2.5.2 Implementation Implications

The key message from Figure 2-13 is that at the beginning of the project cycle the roads agency controls all factors (100% influence) that affect the ultimate performance of the LVR in terms of its life-cycle costs. Thus, if the quality of decision making at the start of the project is poor, this will have an adverse, knock-on effect on all down-stream implementation activities and ultimately result in the provision of a LVR with relatively high life-cycle costs. Thus, it is of paramount importance that the scope and understanding of the project are adequately defined at the outset, i.e. at the planning stage, so as to exert a positive impact on the subsequent phases of LVR provision.

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3.1 INTRODUCTION

3.1.1 Background

The physical environment, in terms of the country's physical features and climate, affects many aspects of the design of LVRs in that such features impact, either positively or negatively, on the options open to the design engineer. By way of example, a soil through which a road may be located will be characterized by type, geotechnical properties, depth to bedrock, etc. Similarly, the design of drainage and anti-erosion systems depends largely on topography as well as the expected climatic conditions in terms of rainfall intensity, duration and frequency, while temperature may affect the choice of bituminous binders used in surfacings.

There is also a relationship between the physical features themselves which can assist the engineer to better understand the road environment and support the design process. For example, a knowledge of the type of vegetation might give some clue to the type of soil and the parent geology of an area interacts with rainfall and climate to produce certain types of soils. Thus, the designer would be expected to have a good knowledge of the various factors that comprise the physical environment to ensure that design solutions are compatible so as to avoid "working against nature".

3.1.2 Purpose and Scope

The purpose of this chapter is to highlight the various features of the physical environment that could affect the design of a LVR. The chapter discusses both the physical features and climate of Tanzania and indicates their potential impact on the design process. The various maps pertaining to the topography, geology, hydrology, vegetation and soils may be obtained from the respective government departments or found as an attachment to the Pavement and Materials Design Manual (MOW, 1999).

3.2 PHYSICAL FEATURES

3.2.1 General

The following physical features are considered in this section.

- Topography.
- Geology.
- Hydrology.
- Vegetation.
- Soils.

3.2.2 Topography

The country has the following four main topographic types:

- Lowlands (mainly the coastal plain below 200 m above mean sea level, with isolated hills up to 300 m in height).
- Broad nearly flat areas of inland drainage, notably the Malagarasi swamp.
- Plateau: 1000 – 1500 m above mean sea level; characterised by gently sloping plains broken by scattered hills and wetlands.
- Highlands: mountainous and steep rolling terrain.

The highlands include the following mountain ranges of altitudes generally between 1500 m and 3000 m:

- Northern Highlands - Usambara, Pare and the volcanic peaks stretching from Kilimanjaro (5895 m) westwards to the Serengeti plains.
- Central Highlands - stretching from Morogoro to the Iringa area.
- Southern Highlands - Tukuyu - Mbeya – Sumbawanga.
- Western Highlands - forming much of the western boundary of the country.

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

- Geometric design in terms of horizontal, vertical alignment and road cross-section.
- Drainage and anti-erosion measures.
- Traffic safety measures.
- Choice of road surfacing.

3.2.3 Geology

Metamorphic granitic rocks of the early Archaean age - more than 3000 million years old - occupy much of the central plateau of Tanzania, forming a large 'block' surrounded by younger fold belts, also of Precambrian age. Sedimentary rocks of the Karoo age - 220 to 140 million years old - occur to the north-east of Lake Nyasa.

Distinctive volcanic features of Neogene age are the recent volcanic centres in northern Tanzania and near Mbeya in the south. In the north, widespread volcanic activity that probably started 13 to 15 million years ago, stretches westwards from the Kilimanjaro peaks to Serengeti and into Kenya. Some volcanic centres in this area are moderately active today.

Younger marine deposits, associated with reef formation, are seen along the coast line and are in places raised by local warping to form low hills of reef limestone, commonly called coral rock whilst lake beds and Neogene deposits of limestone, sand, silts and clays that are formed in basins with restricted drainage, are widespread in the interior of the country.

Over the millennia, the climate of Tanzania has changed considerably and has led to the development of deeply weathered rock formations and thick overlying lateritic and related soil horizons. The long, evolved drainage history of the country has led to a complex series of fluvial, eluvial and alluvial deposits.

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

Large scale geological maps should be obtained in each district as a basis for planning site investigations and materials prospecting.

3.2.4 Hydrology

The hydrology of any project area requires very careful consideration in terms of determining not only the quantity of water that the drainage system must accommodate, but also with the detailed design of the internal and external drainage requirements that are crucial for the successful performance of a LVR.

3.2.5 Vegetation

The vegetation of Tanzania is characterised by large areas of woodland, bushland and thicket. However considerable stretches of savannah, grassland and cultivation are found in several locations and occupy large areas in the northern part of the country. Forests and swamps occur in comparatively small localised areas, and mangrove forests are found in the tidal zone along the coast line. The swamp areas are typically associated with the occurrence of expansive soils and impeded drainage conditions which require special attention in the design process.

The type of vegetation that exists is partly a function of the properties of the soil and this feature can often be used as an indicator of soil type.

3.2.6 Soils

The soil map of Tanzania shows the surficial materials (usually in the top 1 to 1.2 m below the surface) and the relative congruence between this and the geological map is notable. Use of the two maps in unison can give a strong indication of the types of potential construction material likely to be found in any area.

The coastal zone is mainly covered with deep, sandy to heavy textured soils. Most of the central and western plateau areas are mantled by sandy loams whilst drought-prone soils cover a great part of the northern portion of the country, including the Masai steppe and the southern plateau. Eroded land and deeply weathered soils, susceptible to erosion, occur on hill or mountain slopes and in the central highlands.

Volcanic soils with a high ash content are found in the northern rift zone and the volcanic areas in the northern and southern highlands. The soils of the western highlands are developed on basaltic or argillaceous rocks.

3.3 CLIMATE

3.3.1 General

Climate can have a considerable influence on road performance and should be taken into account by the design engineer in terms of the intensity, duration and frequency of rainfall and its impact on the design of drainage systems and anti-erosion measures.

Tanzania's climate is characterised largely by its rainfall pattern over the country which is driven mainly by the migration of the Inter-tropical Convergence Zone which migrates southwards through the country in October to March, reaching the south of the country in January and February, and returning northwards in March, April and May. This causes the north and east of Tanzania to experience two distinct wet periods – the short rains (or "Vuli") in October to December and the long rains (or "Masika") from March to May – while the southern, western and central parts of the country experience one wet season that continues from October through to April or May.

3.3.2 Rainfall

Altitude plays a large role in determining the rainfall pattern, with higher elevations receiving more precipitation. Country-wide, the mean annual rainfall varies from about 500 mm in the semi-desert central and north eastern part of the country to about 2500 mm in the north-eastern and southern Highlands of the country.

In the relatively high rainfall areas that occur in the mountainous and steep rolling terrain of the highlands, concentrated precipitation does occur. This can lead to high surface run-off and erosion of shoulders and side slopes, increased soil erosion, flash flooding and siltation of waterways by disturbance of soil. In such situations special attention must be paid to installing effective drainage systems and measures to control erosion.

3.3.3 Climatic Zones

For the purpose of pavement design, Tanzania can be divided into the following three climatic zones:

- A dry area in the interior.
- A large moderate zone.
- Several wet zones, mainly at high altitudes.

The three climatic zones are shown in Figure 3-1 and are demarcated on the basis of the number of months in a year with surplus of rainfall over potential evaporation, as presented in Table 3-1.

Table 3-1: Main climatic zones

Climatic zone	Number of months per year with higher rainfall than evaporation
Dry	Less than 1 month.
Moderate	1 to 3 months.
Wet	More than 3 months.

Figure 3-1: Main climatic zones in Tanzania

3.3.4 Temperature

Tanzania's tropical climate has regional variations due to topography. In the highlands, temperatures range between 10 and 20˚C during cold and hot seasons respectively. The rest of the country has temperatures rarely falling lower than 20˚C. The hottest period extends between November and February (25 - 31˚C) while the coldest period occurs between May and August.

The relatively high solar radiation levels that prevail in the dry zone of Tanzania, combined with extreme temperature conditions, result in relatively rapid ageing of the binders and reduced seal lives as well as surfacing problems such as bleeding of the binder and loss of aggregate. Thus, careful account should be taken of temperature conditions in the design of surface treatments to ensure maximum durability.

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Part B: Planning

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CONTENTS

4.1 INTRODUCTION

4.1.1 Background

The planning phase of a LVR project can rightly be viewed as the foundation on which the subsequent implementation phases are based. It is an activity aimed at considering a wide range of options with the objective of providing an optimal, sustainable solution, i.e. one which satisfies the multiple needs of stakeholders at minimum life-cycle costs. It should take full account of Government policies and strategies in the road transport sub-sector.

Planning should be undertaken in a context sensitive manner in which all dimensions of sustainability are addressed. This places more weight on multi-disciplinary planning in which teams of planners, engineers, environmentalists, etc., work together with stakeholders in order to reach optimal solutions in the most cost-effective manner. Such an approach provides the best chance of achieving long-term sustainability of projects and, in so doing, ensures that the available resources are used in the most cost-effective manner.

4.1.2 Purpose and Scope

The main purpose of this chapter is to outline the various procedures that are typically followed in the rural accessibility planning process in a manner that ensures that the outputs have the full support of the beneficiaries of LVR provision.

The chapter presents a generic framework for the planning and appraisal stages of road provision including aspects that are of particular significance to LVRs and that are often not considered in conventional approaches. The various planning tools that may be considered at the various stages of the project cycle are also presented. Finally, the chapter highlights the importance of stakeholder consultations in the planning process including the types of stakeholders that should be involved and the consultation techniques that may be employed.

4.2 PLANNING FRAMEWORK

4.2.1 General

Experience from successful programmes highlights a need to think of rural road planning in terms of a system comprising not only methodology and criteria, but also the process and procedures through which key constituencies are involved at various levels of decision-making. This points towards multi-tiered planning and programming systems based on locally acceptable criteria allowing participation of local communities.

In order to ensure active participation of stakeholders, the procedures need to be transparent, relatively simple to carry out, unambiguous, equitable and, above all, should not be too resource intensive.

The various stages/activities typically followed in the planning process are presented in Table 4-1. In principle, the process comprises structured activities which start from the general and work towards the particular in relation to both data and project ideas.

Table 4-1: Stages/activities in the planning process

Project Cycle	Evaluation Activity	Typical Evaluation Tools	Output
Identification	Selection	• Policy resource analysis. • Master Plans. • Local/regional plans.	Long list of projects.
Feasibility	Screening	• Livelihoods analysis. • Integrated Rural Accessibility Planning. • Network based planning.	Shorter list of projects.
Design	Evaluation	• Cost-benefit analysis. – consumer surplus (e.g. RED); – producer surplus; – compound ranking; – multi-criteria analysis.	Short list of projects.
Commitment and negotiation	Prioritisation	• Budget considerations. – ranking by economic or socio-economic criteria.	Final list of projects.

The main features of the planning and appraisal processes for new road projects are as follows:

- **Selection:** This is a multi-sectoral and multi-disciplinary process which should generate sufficient projects to ensure that no potentially worthwhile ones are excluded from consideration. The output is a long list of projects determined on the basis of an unconstrained policy resource analysis that satisfy national road transport policy.
- **Screening:** Defines the constraints within which specific planning solutions must be found, i.e. a constrained policy resource analysis. The output is a shorter list of projects that justify further, more detailed, analysis.
- **Evaluation:** Evaluates the shorter list of projects in more detail by subjecting them to a detailed cost-benefit appraisal for which various methods are available. The output is a final list of projects which satisfy a range of criteria – political, social, economic, and environmental – at least cost.
- **Prioritisation:** Ranks the "best" projects in order of merit up to a cut-off point dictated by the budget available

For existing roads which need to be rehabilitated or upgraded, it would not be necessary to undertake the identification and feasibility phases but, rather, to concentrate on the design and commitment and negotiation phases that lead to implementation of the project.

4.2.2 Planning Considerations

The procedures described in the planning and appraisal framework shown in Table 4-1 are common to any type of road project. However, there are aspects of it that are of particular significance in the planning and appraisal of LVRs that often do not emerge from conventional approaches. These are summarised in Table 4-2.

Table 4-2: Project cycle and related planning activities

Stage	Issues to be considered
Project identification • Project objectives	• Are the strategies being adopted supportive of government policy? (e.g. employment creation). • Are they relevant to the current and future needs of beneficiaries? • Are they cognisant of the multiple objectives and views of stakeholders? • Have effective communication channels with stakeholders been created? • Are they gender sensitive?
Feasibility • Design criteria • Cost-benefit analysis • Socio-economic assessment • Road safety assessment • Environmental assessment • Livelihoods	• Is there adequate participatory planning and consultation with public and private sector stakeholders? • Do the design criteria take full account of the specificities of LVRs, including non-motorised traffic? • Are appropriate evaluation tools being used? • Has a base line environmental survey been undertaken? • Has a road safety audit been incorporated in the project?
Design • Design standards • Pavement/surfacing design	• Are the geometric, pavement design and surfacing standards technically appropriate? • Are they environmentally sound? • Are specifications and test methods appropriate to local materials being used?
Commitment & negotiation • Contract documentation	• Do designs accommodate construction by labour-based methods? • Do they include environmental protection measures? • Have tender documents been prepared and contract strategies adopted that facilitate involvement of small contractors?
Implementation • Construction • Inspection and monitoring • Environmental mitigation	• Have labour-based rather than equipment based methods of construction been adopted where feasible? • Are environmental mitigation measures contained in the contracts? Are they enforceable? • Have specific measures been included in the contract to cater for health and safety matters such as HIV/AIDS?
Operations & maintenance • Performance evaluation • Maintenance operations	• Have the various indicators of socio-economic well-being been monitored and evaluated? • Are there adequate arrangements for community participation in road maintenance? • What are the lessons for the future?

4.2.3 Labour-based Projects

In view of the emergence of labour-based approaches as a viable alternative to the more traditional plant-based approaches, the planning of such projects merits special consideration as a means of providing much needed employment. Without appropriate technical and financial planning from the inception of a project, serious problems may ensue, which may ruin the initiative and bring into disrepute the practicability and objectives of labour-based projects.

Many items need to be investigated in terms of their suitability for labour-based methods of construction or maintenance. Contractual aspects need to be established and appropriate designs undertaken. Such planning must extend beyond engineering technology and the practicality of construction and also consider such factors as the financing and management of labour-based projects.

Comprehensive guidance on all aspects of labour based technology is provided by the Ministry of Works' Appropriate Technology Based Road Works Technical Manual (MOW, 2000) which sets out guidelines on policy, recommended standards and technical procedures to be followed by practitioners.

4.2.4 Complementary Interventions

Complementary Interventions (CIs) are a relatively new concept that may be considered in the early stages of LVR planning. They include actions or initiatives that are implemented through a roads project and that are targeted toward the communities that lie within the vicinity of the road. They are intended to optimize the benefits brought by the road and to extend the positive and mitigate the negative impacts of the project.

In its simplest terms CIs take advantage of the presence of the contractor on the road project to build in aspects that will enhance the social, environmental and safety situation of communities affected by the road. These are additional to the normal social, environmental and safety obligations of the contractor and do not replace or share the contractor's normal obligations.

Further information on the manner of undertaking CIs may be found in the Ethiopian Design Manual for Low Volume Roads: Part C – Complementary Interventions.

4.2.5 External Factors

There are a number of external factors, many of them of a non-technical nature, that directly or indirectly affect the planning process itself or the outcomes from that process. It is important to be aware of them when devising an appropriate planning procedure and, where possible, to take them into account. These various factors are listed in Table 4-3.

Table 4-3: External factors that affect the planning of LVRs

Environment	Factor	Implications on approach to LVSR provision
Political	• Government policy • Political perceptions • Political involvement	• Influences practice. Covers issues such as poverty alleviation, sustainable socio-economic development, technology choice, employment creation, standards, and sources of funding. • Tendency to favour conventional approaches and standards with perceived minimum "risk" attached to them. Need to communicate effectively, quantify and "sell" innovative approaches and appropriate, non-traditional standards. • To be expected. Will tend to influence decision-making. Highlight pros and cons of alternative solutions in a balanced, transparent manner and maintain continuous dialogue with stakeholders.
Social	• Poverty alleviation • Sustainable livelihood • Gender considerations	• Implies use of labour-based rather than fully plant-based methods where feasible. • Enhance local participation and resource mobilisation by involving the people who will ultimately benefit from the projects. • Understanding of community strengths and weaknesses, assets, vulnerability to shocks and constraints, governance issues and policies needed. • Eliminate gender biases by integrating the transport needs of women in the mainstream of policy and planning. • Promote participation by women in labour-based construction and maintenance programmes and training to assume supervisory roles.
Institutional	• Organisation	• Growing trend towards establishment of more autonomous central and local roads authorities. • Greater scope for generating accountability for results in road programmes and moving from force account to contracting out of work to the private sector.
Technological	• Technology choice	• Need for cost-effective strategies that utilise the dual output of road infrastructure provision whilst creating employment.
Economic	• Evaluation	• Road benefits often not limited to use of road, but also from the way in which the road is financed, designed, constructed and maintained. Need to capture monetary and non-monetary benefits in the evaluation framework.
Financial	• Funding • Sustainability	• Usually very scarce. Financing proposals must look increasingly at minimum standards, limited donor funding and local funding of recurrent maintenance costs. • Sustainability of funding has become a critical issue. Need to commercialise operations where possible and involve stakeholders in the maintenance of facilities.
Environmental	• Impact	• Need to capture social as well as environmental impacts in the evaluation of LVRs. • Address health-threatening impacts as a high priority.

4.3 PLANNING TOOLS

4.3.1 General

There are a number of tools that may be used to undertake rural accessibility planning at the identification and feasibility stages of the project cycle as shown in Table 4-1. Examples include:

- Policy analysis.
- Master plans.
- Livelihoods framework.
- Integrated planning techniques.
- Network based planning.

An overview of the above methods is presented below.

4.3.2 Policy Resource Analysis

The objective of the policy analysis is to define, in general terms, the constraints within which specific planning solutions must be found. Constraints may relate to such factors as Government policy on employment, provision of accessibility, income distribution and regional development as well as technical factors such as type of terrain and transport facilities, level of existing traffic, capacity and expertise of the local construction industry, availability of finance, etc.

4.3.3 Master Plans

Master plans or regional plans can be used to help in determining priorities for the future. These plans are not transport specific but relate to all sectors and help to identify investment requirements and priorities over a defined period. It is at this stage that road projects for rehabilitation and upgrading will first be identified.

During the preparation of a master plan it is important that transport planners liaise closely with other Ministries. In the rural context particular priorities will include education, health and agriculture. It is also important that extensive consultation is undertaken with local communities and opinion leaders.

4.3.4 Livelihoods Framework

"Livelihoods Analysis" is a useful approach to adopt in order to identify the ways in which any particular investment intervention will impact, benefit or disadvantage the local community. A rural livelihoods analysis provides a framework for understanding how any proposed changes will affect personal or community livelihoods in the longer term. It focuses directly on how the local community uses and develops its social, human, financial, natural and physical asset structure (Figure 4-1).

It is clear that transport interacts with many aspects and dimensions of a person's or a community's livelihood. Substantial benefits could therefore be obtained if the labour requirements involved in collecting water, firewood or taking crops to market could be reduced. The provision of all-season road access could reduce the vulnerability of rural communities by stopping seasonal isolation, and reducing transport costs and travel times to essential economic and social services.

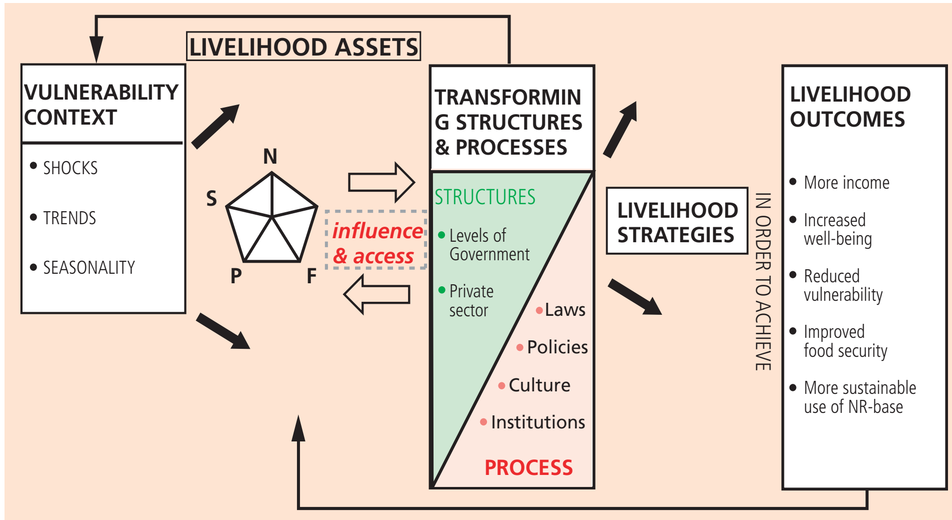

Figure 4-1: Sustainable livelihoods framework

Notes:

H: = human capital: the skills, knowledge, and ability to work and good health important to the ability to pursue different livelihood strategies.

P: = physical capital: the basic infrastructure (transport, shelter, water, energy and communications) and the production equipment and means that enable people to pursue livelihoods.

S = social capital: the social resources (networks, membership of groups, relationships of trust, access to wider institutions of society) upon which people draw in pursuit of livelihoods.

F = financial capital: the financial resources which are available to people (whether savings, supplies of credit or regular remittances or pensions) and which provide them with different livelihood options, and which provide them with different livelihood options.

N = natural capital: the natural resource stocks from which resource flows that are useful for livelihoods are derived (e.g. land, water, wildlife, biodiversity, environmental resources.

4.3.5 Integrated Planning Techniques

Integrated Rural Accessibility Planning (IRAP) has been developed by the International Labour Organisation (ILO) and is probably the most widely used planning technique of its type. IRAP has been used in many countries in the region, including Tanzania and Malawi. The approach integrates rural households' mobility needs, the siting of essential social and economic services, and the provision of appropriate transport infrastructure. Communities are involved at all stages of the planning procedure. It is based on a thorough but easy to execute data collection system that seeks to rank the difficulty with which communities access various facilities.

In the IRAP approach, an Accessibility Indicator (AI) is calculated for various facilities in each community. It is a function of the number of households (N), the average travel time to a facility (T), the target travel time (Tm) and the frequency of travel (F):

$AI = N * (T – Tm) * F$

Typical facilities included are health, education, water and fuel. The accessibility indicators are ranked in descending order and interventions are prioritised in this way. Results of this process are discussed at a participatory workshop and interventions identified which most effectively reduce time and effort spent.

4.3.6 Multi-Criteria Analysis

Multi-Criteria Analysis provides a means of prioritising investments in rural roads through the consideration of the current condition of each road link its economic and social importance. For rural roads such data typically include:

- Traffic on the road.
- The population served by the road.

- Agricultural output of the area served by the road.
- Existing social facilities such as schools and clinics along the road.

The condition of each road link is assessed and a score allocated. This is known as a “Condition Index”. Roads in poor condition have a high Condition Index. Priority factors are then determined for traffic, population, agriculture and social facilities, with weightings applied to each factor depending on their importance. A “Priority Score” for each road link can be then calculated by multiplying the Condition Index by the traffic, agriculture, social and population priority factors. The equation is as follows:

Priority Score=Condition Index ×TF ×AF ×SF ×PF

TF= Traffic factor
AF = Agriculture production factor
SF = Social facilities factor
PF = Population factor.

The result of this analysis is that roads in poor condition but with high social and economic importance are selected first for maintenance or improvement interventions. The Priority Score for each road link should be determined on the basis of weights and points allocated to each factor in a participatory and transparent manner. This will ensure that the outcome is accepted by all stakeholders.

4.3.7 Network-Based Planning

Traditionally, investments in roads are generally evaluated on a link by link basis with less consideration given to the connectivity or accessibility contributions of links to the entire network. Network-based planning enables contributions from the various links to be considered and it is particularly useful where a “core road network” needs to be identified in situations where funding is available to maintain only part of the total road network. However, there seems to be little information available in the form of manuals on network-based planning procedures.

Unfortunately, the situation confronting many countries is one in which funding is available for maintenance of only part of the road network. In such situations it has become necessary to identify a “core road network” which is reviewed periodically and will expand or contract depending on local circumstances. Such networks often include roads of different classes that are considered to be an essential part of the total network so that links are maintained between all the communities throughout the country.

Tools such as the World Bank’s Highway Design and Maintenance Standards Model (HDM-4) can be used for network-based planning purposes. However, the necessary data required is often not available at local level, making such models inappropriate. Thus, procedures that involve a

4.4 STAKEHOLDER CONSULTATIONS

4.4.1 General

The objective of consultation is to ensure that the road planning process is undertaken in an accountable and transparent manner. This is important for the overall benefit of the affected stakeholders and for the country at large. Consultations should be carried out throughout all stages of the project cycle and should be undertaken in such a manner as to allow full participation of the authorities and the public with the following typical aims:

- Establishing background information on the project from all possible sources.
- Identifying viable alternatives for the project.
- Taking on board the views of stakeholders at all stages of the project.
- Reaching a consensus on the preferred choice of project(s).

Decisions on transport planning and prioritisation are often taken without considering the transport requirements of the people being affected by the investment. Insufficient consultation can led to the inappropriate use of resources both in terms of their usefulness to rural communities but also in terms of their impact on social and cultural traditions. To rectify this shortcoming it should be ensured that:

- Local people are involved in the selection, design, planning and implementation of programmes and projects that will affect them.
- Local perception, attitudes, values and knowledge are taken into account.
- A continuous and comprehensive feedback process is made an integral part of all development activities.

4.4.2 Types of Stakeholders

Many people have an interest in road projects and all interested groups need to be identified and consulted in the road selection process. The primary stakeholders are those people whose social and economic livelihoods will be directly affected by the project and include:

- Rural communities.
- Farmers groups.
- Market traders.
- Road users and transport operators.

It is important to ensure that women's needs are heard and addressed as part of the stakeholder consultations indicated above.

Figure 4-2: Stakeholder consultation meeting

Some other interest groups are important in the decision-making process, even though their own lives may not be affected directly by the project. These include:

- District leadership.
- District's works departments.
- National roads agencies.
- Local and national politicians.

Because leaders' standpoints can differ significantly from the experiences of "average" village members, it is important for any consultation process to go beyond the leadership to the grass roots.

4.4.3 Consultation Techniques

There are a number of recognised participatory techniques for working with communities to determine their transport needs. These usually entail the use of trained facilitators to visually represent community livelihoods to identify constraints and needs. Typical techniques include:

- Participatory Rural Appraisal (PRA).
- Rapid Rural Appraisal (RRA).

The above methods include a range of activities with a common thread: enabling ordinary people to play an active and influential part in decisions which affect their lives. This means that people are not just listened to, but also heard; and that their voices shape outcomes.

In essence, PRA and RRA methods embody the following features:

- The information to be processed is collected by group members themselves.
- It is presented in highly visual form, usually out in the open and on the ground, using pictures, symbols and locally available materials.
- Once displayed, the information is “transparent rather than hidden” - all members can comment on it, revise it and criticize it. This assists in cross-checking and verifying collected data.

Other consultation techniques include public hearings through political leaders, and direct community consultation. Workshops are often a good way of undertaking initial prioritisation exercises, delivering key messages and receiving feedback. It is important that all consultation techniques are well organised, that all the relevant stakeholders have been invited and that the deliberations take place in and interactive and transparent manner.

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Part C: Investigations

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5.1 INTRODUCTION

5.1.1 Background

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

- Asses the condition, level of accessibility and inventory details of any existing road or track.
- Select the alignment of a road.
- Identify the best location of water crossings and drainage structures.
- Provide information for the design of the road pavement, bridges and other structures.
- Identify any areas that might require a specialist geotechnical investigation (refer to *Chapter 6 – Geotechnical Investigations and Design).*
- Identify areas of potentially problematic soils requiring additional investigation and treatment (refer to *Chapter 6 – Geotechnical Investigations and Design).*
- Identify and assess suitable, locally available borrow and construction material.

Not all projects will require the same detailed surveys. Road projects fall into one of the following categories:

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

Site investigations for an entirely new road are very comprehensive because none, or very little, of this information will be available beforehand and collecting it usually requires a range of skills. In contrast, for upgrading an existing road to an all-weather access standard, the required site investigations are considerably simpler because much of the required information is already available. For LVRs in Tanzania it is very unlikely that an entirely new road will be needed where no existing track or road already exists and therefore surveys for new roads are not included in this manual. For such a project the reader is directed to the Field Testing Manual (MOW, 2003).

5.1.2 Purpose and Scope

The purpose of this chapter is provide guidance on the appropriate type and level of site investigation that is required for upgrading an existing rural road or track to an all-weather road. It focusses on 'engineering' or, more precisely, 'geotechnical engineering'. However, other information is also required from what can generically be described as site surveys.

- Hydrological surveys are required to estimate the water flows that determine the drainage design of the road and the design of water crossings (refer to *Chapter 11 – Hydrology and Drainage Structures).*
- Traffic surveys are required to estimate future traffic (including non-motorised road users, number and type of vehicles and traffic loading) that will use the road over its design life (refer to *Chapter 8 –Traffic).* Surveys/studies are required to evaluate environmental impacts and how to control them.
- Surveys of local communities must be carried out to consult them about the road project to meet their requirements and to provide information to assist with the design and construction of the road (refer to *Chapter 4 – Rural Accessibility Planning).*

The chapter provides practitioners with the necessary tools to develop suitable site investigation programmes and to identify the need for more detailed geotechnical investigations, as described in Chapter 6, if they are required.

It is not the purpose of the chapter to describe individual site investigation techniques in detail. Where additional information on the type, use and interpretation of site investigation techniques is needed, the reader is referred to the appropriate manuals such as the Tanzania Field Testing Manual (MOW, 2003).

5.2 PRELIMINARY SITE INVESTIGATIONS

5.2.1 General

The choice of methods for site investigation is determined by the type of road project, practical problems arising from site conditions, terrain and climate. The Field Testing Manual (MOW, 2003) describes the most frequently employed techniques for all aspects of the road design. Only techniques appropriate for LVRs are described in this chapter. These will naturally overlap to some extent with the Field Testing Manual (MOW, 2003) but are included here for completeness.

5.2.2 Types and Scope of Site Investigation

In general there will be two main phases of site investigation namely:

- An Initial Investigation that will identify the main issues.
- A Detailed Investigation that will provide all the information needed for design and identify areas requiring specialised input.

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

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

The benefit of using materials test kits is that a large number of simple tests can be conducted in the field relatively quickly and cheaply and the frequency of testing will not be compromised. However, verification tests in the laboratory will also be required. Strength, compaction, swelling and other types of test can only be conducted by appropriate sampling and laboratory testing. A detailed explanation on the application and use of the test kits is provided in the ASIST Technical Brief Number 9: Material Selection and Quality Assurance for Labour-based Unsealed Road Projects.

5.2.3 Desk Study and Initial Survey

The desk study and initial survey should cover all of the aspects of site investigation required in the detailed survey but only to the extent required to plan the detailed survey that will provide all the information required for design. It should include the following stages summarised in Table 5-1:

- Desk study.
- Site visit for general assessment and visual assessment of road condition.

Although additional surveys may be required if significant problems are identified, for example, a new bridge crossing, the next stage after the initial assessment is essentially the final engineering design.

Table 5-1: Summary of requirements for the desk study and initial survey

Stage of design	Study	Comments
Desk study	Engineering	Probably a road or track is in existence already. Major engineering problems identified.
	Social	The need for the road will have been based on the current planning process at regional or local level. Social assessment based on desk study information and concentrated on major issues such as land take and resettlement, if any.
	Environment	Assessment based on desk study information but concentrated on major issues such as land take, re-instatement, existence of any conservation areas.
	Cost estimation	Historic data only. Based principally on terrain and number of structures. Accurate to only ±100%.
Site Visit (General)	Consultation with local people	Social and economic issues. Flooding (high water levels) and stream flows, adequacy of culverts, accident locations, weak sections of road that are impassable for part of the year, the nature of impassability, availability of materials etc.
	Engineering	The following aspects should be addressed: a) Confirm information obtained from consultation with local people. b) Assessment of defects visible on the road surface. c) Assessment of geometric characteristics. d) Assessment of road drainage, stream and river crossings and extent of flooding of water crossings and low-lying areas. e) Location of all possible bridge sites and water crossings requiring more than a small culvert. f) Slope stability and potential landslide problems. g) Other possible major hazard areas such as poorly drained soils, problem soils, springs, and erosion in river courses. h) Extent of erosion problems with road drainage requirements. i) Possible sources of water for construction. j) Possible sources of construction materials. k) Assessment of land acquisition/site clearance problems. l) Traffic.
	Environment	Many common environmental issues associated with major roads are unlikely to be significant for LVRs but attention must be paid to borrow and spoil areas and likely changes in drainage patterns plus possible effects of the road on biodiversity and ecology.
	Cost estimation	Largely based on historic records but now supplemented with more detail about the scale (and therefore likely cost) of the pavement and structures.

If there is an existing road or track, the basic route is already determined and potential problems will have already become apparent, for example:

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

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

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

Desk Study

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

Sources of information typically include:

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

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

5.2.4 Site Visit for General Assessment

The site visit for general assessment typically consists of the following activities:

Consultations with local communities

It is vital to involve the future users of the upgraded road, including the communities served by the road. Such persons can provide valuable information on various physical characteristics, such as the likelihood of flooding of certain sections of the road, adequacy of existing culverts, the location of weak pavement layers and accident black spots. Such consultations are best done through a series of meetings with a representative group of key stakeholders with a good gender balance (women's and children's perception of access problems is often different from men's).

Visual assessment of road condition

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

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

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

Drainage and erosion

It is important to ensure that the drainage system is functioning well. As the upgrading of major items of drainage structure such as bridges and large culverts is generally expensive, existing infrastructure should be used as much as possible. Where required, however, the necessary longitudinal and cross drainage capacity should be increased to an appropriate level because effective drainage of the road critically affects its performance and ultimate life. The initial survey should identify the problem areas that require more detailed analysis.

Geometric design and road safety

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

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

Materials

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

Traffic assessment

Assessment of traffic is described in detail in *Chapter 8 – Traffic.*

Climate

Characteristics of the climate, such as historical annual rainfall data, should be obtained, if available (from site visit as well as desk study). The rainfall data required depends partly on the method to be used for designing the drainage (refer to *Chapter 11 – Hydrology and Drainage Structures).*

Hydrological data

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

5.3 DETAILED SITE INVESTIGATIONS

5.3.1 General

Two further phases, at the most, are required, as follows:

- Feasibility study including any specialist investigations if difficult problems are identified.
- More detailed investigations to provide enough information for the Final Engineering Design.

The need for a feasibility study is likely to be an exception rather than a common requirement. The requirements for these more detailed investigations are summarised in Table 5-2 and details are provided in Section 5.3.2.

Table 5-2: Summary of requirements for detailed investigations

Stage of design	Study	Comments
Assessment of road condition	Visual assessment	Defects visible on the surface of the road measured and evaluated. Structural and non-structural deterioration identified. Adequacy of existing drainage structures.
	Structural Assessment	DCP survey and analysis. Structural and non-structural defects confirmed.
	Cost estimation	Data are required that are sufficient to obtain likely costs to an accuracy of better than about 25%. Usually based on historic costs of similar roads supplemented with any additional costs if any expensive structures or earthworks are required.
Final Engineering Design	Engineering	The initial surveys will have identified all major issues and should also have provided information on any additional data that might be required for completion of the final design. Analysis of laboratory samples completed and assessed. All data combined to expedite the detailed design.
	Cost estimation	Final cost estimate.
	Note that an inadequate design leads to inadequate contract documents which, in turn, lead to contractual disputes and expensive claims by the contractor. It is a false economy to be frugal at this stage of the project cycle.	

5.3.2 Feasibility and Detailed Design Study

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

(a) Structural assessment

A pavement structural assessment is required to ascertain the strength and bearing capacity of the existing road which is to be incorporated as part of the new pavement structure. A DCP survey is required as described in Section 5.4.2.

(b) Drainage and erosion

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

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

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

(c) Materials assessment and laboratory testing

Samples of the base material and the support layers in each uniform section, must be tested in the laboratory to provide information to aid construction and to ensure that the materials meet the relevant specifications (refer to *Chapter 6 – Geotechnical Investigations and Design*).

5.4 SITE INVESTIGATION METHODS

5.4.1 General

The engineering design requires sufficient design data for preparation of the tender and draft contract documents. The quality and level of the site investigations should not be compromised to provide cost savings nor should the level of investigation be necessarily reduced simply because the road is classified as a LVR. to reflect an anticipated low design class. This Chapter describes and summarises the principal methods available. Additional information can be found in Chapter 6 and in the Field Testing Manual (MOW, 2003).

Figure 5-1: Carrying out a DCP test on a paved road

5.4.2 Characterisation of Subgrade and In-Situ Materials

The design of a road is strongly dependent on the characteristics of the subgrade and, therefore, so is its potential performance. A good subgrade is strong enough to resist shear failure and has adequate stiffness to minimize vertical deflection. The stronger the subgrade, the thinner the pavement layers above need to be. The designer usually has very little choice about the subgrade for most of the route therefore it is vital that the characteristics of the subgrade along the alignment are measured in some detail and understood. In cases where the subgrade materials are unsuitable, either cost effective methods of improving the existing conditions must be identified (e.g. improving drainage or stabilisation) or the road alignment must be altered to avoid such areas completely.

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

DCP surveys

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

The frequency of the DCP measurements depends on the variability in road conditions and level of confidence required. Where obvious changes of surface conditions occur, the frequency of the tests should be modified to include the changes. Similarly, where surface conditions are uniform, the frequency of testing may be reduced. A guideline for the frequency of testing for upgrading an existing track or road to a paved standard is shown in Table 5-3.

Table 5-3: Frequency of DCP testing

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

A number of correlations exist to link the DCP penetration rate (mm/blow) to the more familiar CBR strength parameters. These correlations are based on 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). In this way the in-situ strengths can be determined.

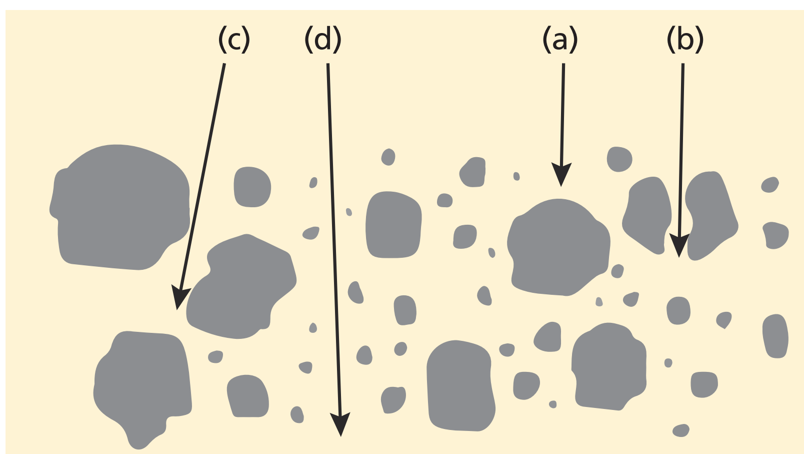
The structural assessment also requires test pits along the road, typically 2 – 3 per uniform section, to obtain samples for laboratory testing.The procedure for undertaking the structural assessment is as follows:

Step 1: Undertake a DCP survey along existing road

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

The tests should be staggered across the road at outer wheel-track, centre line, outer wheel-track. However, the variability of the road subgrade strength will only become fully apparent when the tests have been carried out. In order to ensure statistical reliability at least 10 tests should be taken in each uniform section hence additional tests may be required after analysing the first set.

Care must be exercised in carrying out the DCP survey by discarding any measurements which could produce anomalous results. Such results could arise, for example, where large stones occur in the pavement layer as shown in Figure 5-2.



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

Figure 5-2: DCP effects where large stones are present

Step 2: Analyse DCP data to obtain strength and thickness of pavement layers.

This can be done by hand but it is simpler to use a computer program designed for the task as discussed in Chapter 13.

The in situ strength of the material is strongly dependent on the prevailing moisture (and density) conditions. The results from the survey can be used directly to determine uniform sections along the road.

Step 3: Obtain moisture content beneath pavement at time of survey.

It is essential that an estimate of the in situ moisture condition is made at the time of the DCP survey for comparison with the expected moisture regime in service. At least 2 samples per kilometre should be obtained for moisture content and OMC determination from the outer wheel track of the road at depths of 0-150, 150-300 and 300-450 mm.

Step 4: Determine uniform sections

Following completion of Step 2 the road is often divided into uniform sections at this stage using a 'CUSUM' analysis. This will identify variations in in-situ strength (caused by variations in density, moisture content and/or material type) and variations in layer thicknesses.

The in situ strength of the material is strongly dependent on the prevailing moisture (and density) conditions therefore testing of in situ moisture conditions and standard laboratory testing of the properties of material excavated from the test pits are also required for designing the new pavement.

Test pits and trenches

Test pits and trenches are used to provide samples for testing to provide information on the in-situ subgrade soil conditions and potential fill material. Such testing is also valuable for verifying the DCP calibration and the results of the DCP survey.

Figure 5-3: Test pits for LVRs are commonly excavated using labour

The location, frequency and depth of pits and trenches for characterising the subgrade depend on the type of the road and the general characteristics of the project area (the soil type and variability). The DCP testing carried out to assist delineation of uniform sections can be used to target areas for pitting and trenching. Generally three such pits should be excavated in each section of road that is deemed to be uniform from other investigations, primarily the DCP survey and the road condition survey. Spacing will decrease when the sub-surface soils demonstrate more variability. In these areas, pits can also be staggered left and right of the centreline to cover the full width of the road formation.

The depth of pits and trenches is determined by the nature of the subsurface. For the purpose of sampling and description, pits should be dug to at least 0.5 m below the expected natural subgrade level. In cut sections, the depth can be reduced to 0.3 m. For upgrading and rehabilitation projects there is usually vehicular access hence pits can be excavated using a backhoe through all the existing pavement layers. In these circumstances the depth could usefully be increased to 1.5 m below the subgrade. Some problem subgrade conditions may require deeper exploration and greater depths may also be needed for high embankment design. A limited number of deep pits may also be needed to ascertain groundwater influence and irregular bedrock.

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

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

Standard laboratory testing

Samples collected from the test pits are used to provide the following basic information on the properties of the in-situ materials and subgrade along the alignment:

- Soil Profile: Overburden thickness, layer/horizon thickness, visual description, in situ moisture content and density.
- Index Tests: Particle size distribution (CML Test 1.7/1.8), Atterberg limits (CML test 1.2 and 1.3), Linear shrinkage (CML Test 1.4).
- Compaction: Density and Moisture relationship (CML Test 1.9).
- Strength: CBR and swell (CML Test 1.10).

Figure 5-4: Standard laboratory tests are used to characterise the material properties

Most of the subgrade test samples should be taken from as close to the top of the subgrade as possible (excluding material containing organic remains), extending down to a depth of 0.5 m below the planned subgrade elevation. Potential fill materials should be sampled to a greater depth.

Some regional road authorities have considerable experience and performance data on specific soil types in the local climate and topographic conditions. Use of this information can supplement and reduce (but not replace) the overall requirement for subgrade evaluation. The approach involves the assessment of subgrades on the basis of local geology, topography and drainage, together with regular routine soil classification tests.

5.4.3 Problem Soils

Soils which can cause foundation problems and decrease the performance of roads are common. These soils are collectively called problem soils and comprise among others; expansive; collapsible and compressible; and dispersive soils. The preliminary identification of such soils is crucial during the site investigation so that appropriate additional investigations can be included prior to the final designs being developed. Those areas with particularly problematic soils often require specialist investigations and testing and this is described in detail in *Chapter 6 – Geotechnical Investigations and Design.* Failure to recognise problem soils at the design stage could result in claims and cost overruns if identified later during construction, or have a detrimental impact on the long-term performance of the road.

Figure 5-5: Localised problem materials require special design and construction considerations

5.4.4 Location and Characteristics of Construction Materials

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

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

The investigation of construction materials often requires an extensive programme of site and laboratory testing, especially if the materials are of marginal quality or occur only in small quantities. The site investigation must identify and prove that there are adequate and economically viable reserves of natural construction materials. The uses of construction materials required are summarised below and discussed in more detail in *Chapter 7 – Construction Materials.*

- Common embankment fill.
- Capping layer / imported subgrade.
- Subbase and road-base aggregate.
- Road surfacing aggregate.
- Paving stone (e.g. for cobblestone pavements).

- Aggregates for structural concrete.
- Filter/drainage material.
- Special requirements (e.g. rock-fill for gabion baskets).

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

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

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

Figure 5-6: Sufficient construction material of the required quality must be located

A comprehensive list of the location of potential borrow pits and quarries is needed, along with an assessment of their proposed use and the volumes of material available. Apart from quality and quantity of material, the borrow pits and quarries must be:

- Accessible and suitable for efficient and economic excavation.
- Close to the site to minimize haulage costs.
- Of suitable quality to enable cost-effective construction with little or no treatment.
- Located such that their exploitation will not lead to any complicated or lengthy legal problems and will not unduly affect the local inhabitants or adversely affect the environment.

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

- Determination of the nature of the deposit, including its geology, history of previous excavation and possible mineral rights.
- Determination of the depth, thickness, extent and composition of the strata of soil and rock that are to be excavated.
- Analysis of the condition of groundwater, including the position of the water table, its variations, and possible flow of surface water into the excavation ground.
- Assessment of the property of soils and rocks for the purposes intended.

Records of roads already built with the material can be a valuable source of data, not only on the location of construction materials but also on their excavation, processing, placement and subsequent performance. Potential problems with materials can also be identified. Construction records are often available with regional road authorities, or by road design consultants and contractors.

Fill

In general, location and selection of fill material for low volume roads poses few problems. Exceptions include organic soils and clays with high liquid limit and plasticity. Problems may also exist in lacustrine (stratified deposits at the bottom of a lake) and flood plain deposits where very fine materials are abundant. Where possible, fill should be taken from within the road alignment (balanced cut-fill operations) or by excavation of the side drains (where materials meet the requirements). Borrow pits producing fills should be avoided as far as possible and special consideration should be given to the impacts of winning fill in agriculturally productive areas where land expropriation costs can be high.

Improved subgrade

The subgrade can be made of the same material as any fill. Where in-situ and alignment soils are weak or problematic, the import of improved subgrade may be necessary. As far as possible the requirement to import material from borrow areas should be avoided due to the additional haulage costs. However, import of strong (CBR>9) subgrade materials can provide economies because pavement thickness design can be reduced (refer to *Chapter 13 – Structural Design: Paved Roads)*. Where improvement is necessary or unavoidable, mechanical and chemical stabilisation methods can be considered.

Road base and subbase

Where possible, naturally occurring unprocessed materials should be selected for sub-base and base layers in paved low volume roads. However, under certain circumstances mechanical treatments may be required to improve the quality to the required standard. This often requires the use of special equipment and processing plants that are relatively immobile or static. For this reason the borrow pits for road base and subbase materials are usually spaced widely. The main sources of subbase and base materials are rocky hillsides and cliffs, high steep hills, river banks, naturally-occurring residual soil deposits and pedocretes, e.g. calcretes and laterites. The material quality requirements for base and subbase are addressed in *Chapter 6 – Geotechnical Investigations and Design.*

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

Hard stone and aggregate

The Field Testing Manual (MOW, 2003) provides details on the location and variety of rocks that can be used as material sources for concrete aggregate, bituminous road surfacing aggregate, masonry and cobble stone. In any area, a relatively fresh rock must be encountered at some depth as there is a gradual transition from one weathering state to the other. The recovery of a suitable material is, therefore, a matter of understanding the geological history and weathering profile at the quarry site.

Figure 5-7: Variable weathering in quarry

Locating and testing construction materials

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

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

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

The minimum frequency of testing of borrow pit material needs to strike a balance between cost, time and statistical validity as shown in Table 5-4. Where possible, the location and testing of borrow pit material should be done by traditional methods using established laboratory facilities. In the absence of these facilities, testing using an appropriate field the test kit could be used for DC8 and DC7 earth and gravel roads.

Table 5-4: Material testing frequency

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

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

5.4.5 Earthworks – Cut and Fill Investigation

Natural slopes, road cuts and existing embankment fill in the vicinity of the planned project provide evidence of expected ground instability and likely requirement for detailed surface and subsurface investigations. In many situations expert geotechnical assistance is likely to be required and this is described in Chapter 6.

Instabilities and settlement

These investigations should consider the types of materials in the cut, slope stability, and the different types of movements that may occur. Scars, anomalous bulges, odd outcrops, broken contours, ridge top trenches, fissures, terraced slopes, abrupt changes in slope or in stream direction, springs or seepage zones all indicate the possibility of past ground movements.

The first indication of possible instability problems can be obtained from a study of the topography. Topographic maps and aerial photographs provide useful data on whether instability is likely to occur or has occurred in the past. Slope failure along road cuts is often associated with pre-existing planes. Survey of the orientation and characteristics of joints and weak zones is therefore essential. In addition, the degree of weathering along these joints should be inspected.

When a visual survey is not enough, it is often useful to excavate a trench. In deep cuts, where interference with existing stability and groundwater conditions is expected, a trench across the face of the slope provides a better understanding of the geology of the area. Trenches are preferable to pits to inspect cuts because of their dimension. Depending on the geology and degree of weathering, up to five trenches are normally enough to investigate a 100 m long slope cut. The trenches should be located at places where material changes are expected and range between 1 m and 3 m in depth. Additional information on performance of slopes can also be obtained by inspecting soil and rock exposures along existing road cuts in the region.

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

High level embankment foundation investigation should, as a minimum, consider; the range of materials and settlement potential:

- Side-slope stability; groundwater; moisture regime and drainage requirements; erosion resistance; haul distance; and environmental impact.

Settlement problems are unlikely if rock is encountered at a shallow depth. However, if the underlying foundation is covered by transported soils, problems are likely to occur as the material may vary from soft alluvial clays to collapsing silts (sands) or expansive clays. It is therefore important to understand the particular transportation history and mechanism and the result that this has on the nature of the soil and its behaviour.

The type of field investigation will depend on the types of soils encountered. If soils are predominately cohesive, the primary design issues will be bearing capacity, side slope stability, and long-term settlement. These design issues will usually require the collection of undisturbed soil samples for laboratory strength and consolidation testing. The vane shear test can provide valuable in-situ strength data, particularly in soft clays (for more information on vane shear tests, see the Field Testing Manual (MOW, 2003).

Where embankments cross alluvial deposits, there will probably be a stream requiring a structure. Therefore investigations should assess the interaction between these structures, the embankment and the in-situ material. Most embankment problems at streams 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.

5.4.6 Groundwater

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

Groundwater is frequently encountered along road cuts. In areas where springs and seepages are present, there are several good indicators that may be used to determine the height that groundwater may rise in a slope and roughly how long during the year that the slope remains saturated. For example, in the highland areas where weathered basaltic lava flows are common, iron containing soils within the slope usually oxidize when in contact with groundwater and turn rusty red or bright orange and give the soil a mottled appearance. The depth below the ground surface where these mottled appearances first occur indicates the average maximum height that the fluctuating water table rises in the slope.

At locations where the water table remains relatively stable, iron compounds reduce chemically and give the soil a grey or bluish-grey appearance. The occurrence of these gleyed soils indicates a slope that is saturated for much of the year. Occasionally, mottles may appear above gleyed subsoil, which indicates a seasonally fluctuating water table above a layer that is subjected to a prolonged saturation. The practitioner should be aware of the significance of mottled and gleyed soils exposed during road construction. These soil layers give clues to the need for drainage or extra attention concerning the stability of the road cut.

The Field Testing Manual (MOW, 2003) can be referred to for more information on investigation of major road cuts and embankment. The basic principle for low volume road engineering design is to minimise cost. As far as possible this requires minimising the earthworks cut and fill operations. In some geo-technically fragile areas, increased earthworks can lead to an increased risk of landslides.

5.4.7 Water Crossings

Site investigation techniques needed for appropriate low level structures for water crossings are discussed in Chapter 6 of this manual. Additional information is also provided in the Field Testing Manual (MOW, 2003).

There is no compromise on the design of major structures, such as bridges, where these are placed on low volume roads. Design procedures follow the Bridge Management System for Tanzania (MOID, 2007) and thus the site investigation techniques and procedures described in the Field Testing Manual (MOW, 2003) must be followed.

5.4.8 Water Sources

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

In certain areas water may be scarce for construction purposes and, in particular, for providing proper moisture content during compaction of the soils and pavement layers. Since this problem is serious in some regions of Tanzania, it is important to search for water sources, their yields and the distances from the construction site. In regions where water is scarce, a separate and dedicated hydro-geological study may be needed. Alternatively, dry compaction could be considered for some types of materials (refer to *Chapter 6 – Geotechnical Investigations and Design*). Data from the field reconnaissance can indicate if surface water is a critical problem.

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

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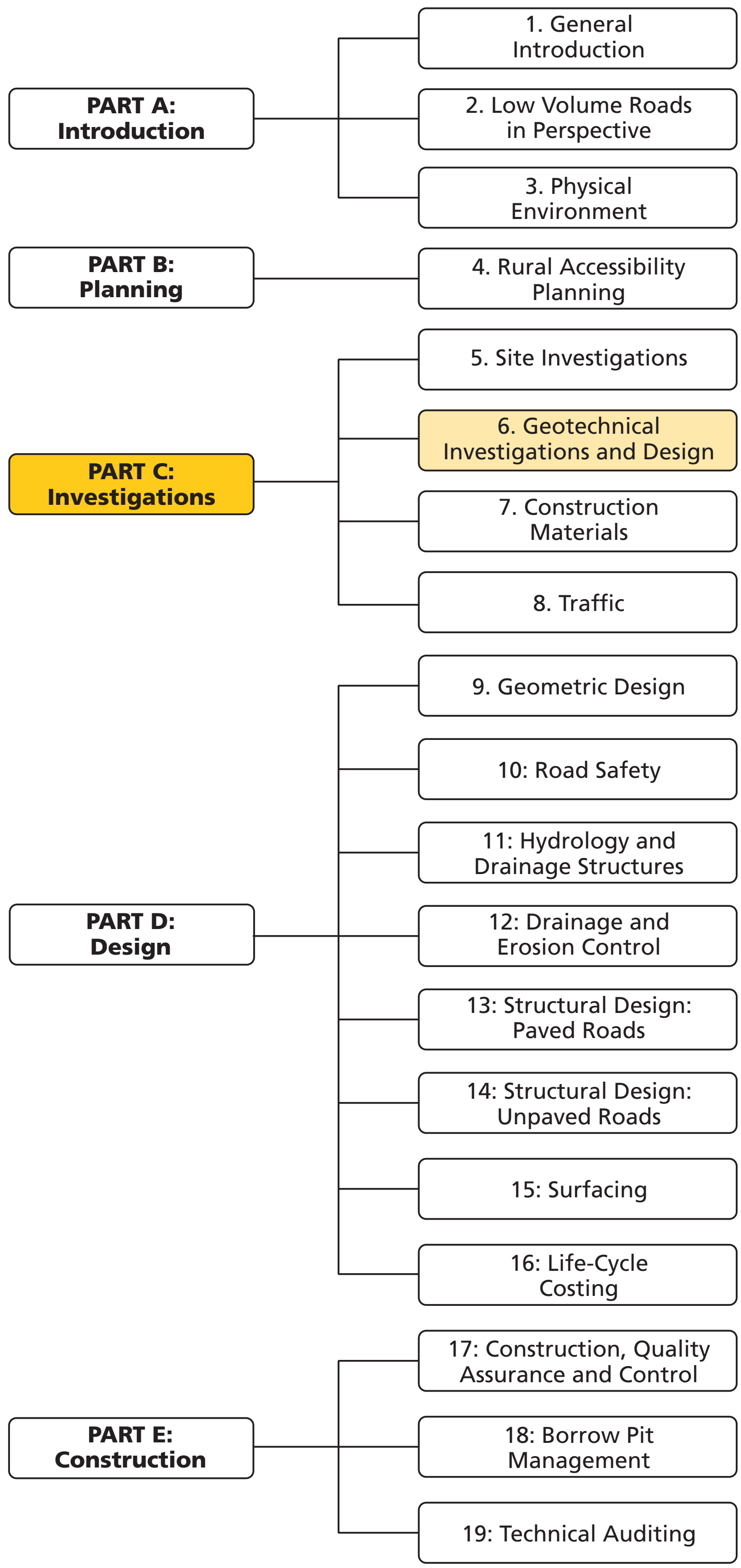
Low Volume Roads Manual



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6.1 INTRODUCTION

6.1.1 Background

Prior to the design of a road, the site and geotechnical investigations need to be carried out. The site investigation is done early in the project as described in *Chapter 5 – Site Investigations*, and should provide sufficient information to locate new roads and stream crossings, identify the optimum road structure and preliminary sources of suitable construction materials and identify potential subgrade problems.

The geotechnical investigation described in this chapter is a more sophisticated exercise than that described in *Chapter 5 – Site Investigations*, during which aspects such as the types and extent of excavations, foundation works and control of problem subgrades is assessed. This usually requires sub-surface investigations and the development of geological and geotechnical models related to the road and structures. The information will allow the engineer to:

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

There is a fine line between site investigation and geotechnical investigation. In many cases, especially smaller projects, the geotechnical investigation may actually be included as an integral part of the site investigation as discussed in *Chapter 5 – Site Investigations,* whereas in other situations (mountainous regions, areas with particularly poor subgrade or drainage conditions, additional construction materials, etc.), independent and more comprehensive geotechnical investigations may be required. Because of this possible overlap between the two activities discussed in Chapters 5 and 6, some duplication of philosophies may occur.

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

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

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

6.1.2 Purpose and Scope

The purpose of this chapter is to provide guidance on the appropriate type and level of geotechnical investigations for low volume roads, the necessary tools to develop such investigation programmes and in-situ and laboratory testing schedules. Some assistance in interpreting the data obtained and applying them to design solutions is also provided.

The scope of the chapter includes the identification and mechanisms for dealing with subgrade problems, foundations for bridges and large structures and an assessment of the stability of cuts and fills. The main component of the geotechnical investigations is therefore focused on providing the engineer with the information necessary to carry out the most cost-effective and safe structural and foundation designs.

6.2 APPROACH TO GEOTECHNICAL INVESTIGATIONS

6.2.1 General

Geotechnical influences can have a major impact on the serviceability, cost and effectiveness of low volume roads. However, the degree of investigation and sophistication of the design should be related to the class of the road. It is unreasonable to expect the same investigation to be carried out or preventative measures to be implemented for a DC8 road as for a DC5 road. However, should a geotechnical investigation be required for roads in the lower road categories, this should be carried out with the same diligence as for any other geotechnical investigation.

It is unlikely that significant geotechnical investigations will be carried out for DC8 and DC7 roads but it is important that the design engineer is able to identify potential problems and know when to get expert geotechnical advice.

6.2.2 Need for specialist input

Table 6-1 summarises the impacts of the various potential geotechnical problems discussed in this chapter on the different classes of road and provides an indication of when specialist geotechnical input should be sought. The table shows the road class against various geotechnical problems and identifies their potential importance/impact on the road design and the likely degree of geotechnical input necessary.

The cells highlighted in red indicate high importance/impact, those in orange have moderate impact and those in green only have minor impact. The cells marked with an x indicate that the design engineer can in most cases make the decision, based on the details in the text of this chapter, while those marked with ✓ indicates that a geotechnical specialist should be involved.

Table 6-1: Relationship between road class, importance of problem and necessity for specialist geotechnical input

Geotechnical problem	Road Class			
	DC 5	DC 6	DC 7	DC 8
Subgrade:				
Expansive clay	✓	✓	X	X
Collapsible soil	✓	X	X	X
Dispersive soil	✓	✓	X	X
Erodible soil	X	X	X	X
Slaking soil	X	X	X	X
Saline soil	✓	✓	✓	X
Soft clay	✓	✓	✓	X
Wet area	✓	✓	X	X
Cut slope	✓	✓	✓	✓
Embankment	✓	✓	X	X
Foundations:				
Slab	✓	✓	X	X
Footing	✓	✓	X	X
Pile	✓	✓	✓	✓

6.3 GEOTECHNICAL INVESTIGATION METHODS

6.3.1 General

The size of most of the structures and excavations for LVRs is generally limited by the cost of construction. However, there is always a possibility that some medium to large structures will be necessitated by the topography. Descriptions of how to carry out appropriate investigation for these are discussed in this chapter. Geotechnical design is a specialised (and often expensive) activity and in many cases it should be done by a geotechnical specialist. This section presents the thought process to be followed before making a decision to employ a specialist.

Geotechnical investigation techniques encompass a wide range of methods. The type and extent of exploration required for a specific project will depend on the nature of the proposed project and the environment in which it is to be built. It is not the purpose of this chapter to explain individual investigation techniques. Detailed information on the type, use and interpretation of geotechnical investigation techniques can be found in the Field Testing Manual (MOW, 2003).

There is a fine line between site investigation and geotechnical investigation. In many cases, especially smaller projects, the geotechnical investigation may actually be included as an integral part of the site investigation as discussed in *Chapter 5 – Site Investigations,* whereas in other situations (mountainous regions, areas with particularly poor subgrade or drainage conditions, etc.), independent and more comprehensive geotechnical investigations may be required. Because of this possible overlap between the two activities discussed in Chapters 5 and 6, some duplication of philosophies may occur.

For appropriate design of structures (including earthworks), it is important to determine the nature and location of the different soil types occurring along the route alignment to appropriate depths for the structures involved. This is determined during the geotechnical investigation.

6.3.2 Techniques

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

A wide variety of techniques is used for geotechnical investigations as presented in Table 6-2, but relatively simple and standard techniques should be used as much as possible. More sophisticated and expensive techniques should only be employed when a significant geotechnical problem is encountered with potentially severe consequences should failure occur. Under such circumstances, it is advisable to seek specialist assistance as indicated in Table 6-1. Ground investigations need to be carefully planned and must take into account the following:

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

Results from the Desk Study and the initial assessment (walk-over survey) as described in Chapter 5 should be used in the planning of cost-effective ground investigations.

Figure 6-1 outlines the key objectives of ground investigations which may be undertaken using a variety of sampling and testing techniques, outlined in Table 6-2.

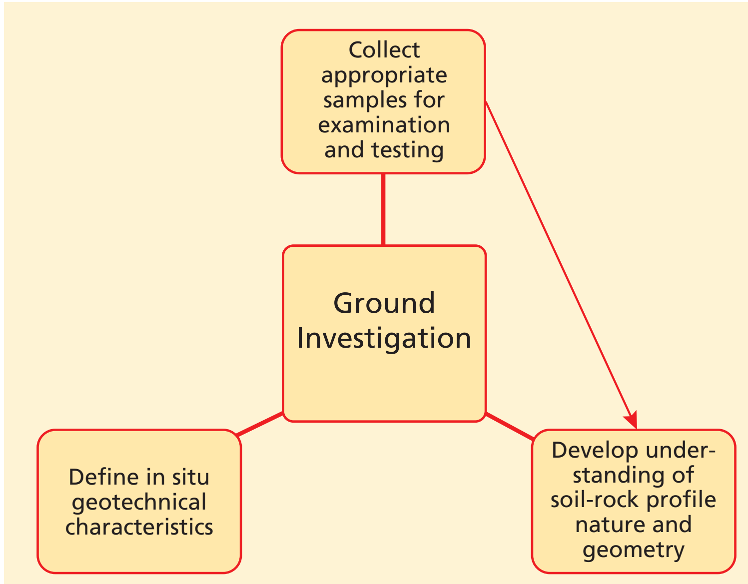

Figure 6-1: Key ground investigation objectives

Table 6-2: Standard ground investigation techniques

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

6.3.3 Scope and Extent of Investigations

The scope and extent of the site investigation for final engineering design will depend on the characteristics of the alignment and the type of road under consideration. For many low volume roads the design of the initial site investigation, as described in *Chapter 5 – Site Investigations,* should be such that most of the information obtained should be sufficient for final design. The data obtained at the site investigation stage will not be so comprehensive and will not be as robust from a statistical point of view as that obtained from site investigations for higher volume roads. However, it should be adequate and reliable and sufficient to provide a competent design for the specific low volume roads. It is likely that some additional detailed survey (geotechnical investigation) will be required later, particularly for water crossings, within areas of problem soils and unstable terrain.

The quality and level of the site and/or geotechnical investigations for final design should not be compromised to provide cost savings nor should the level of investigation be necessarily reduced to reflect an anticipated low design class.

The final engineering design requires sufficient data for preparation of the tender and draft contract documents. This stage requires the most rigorous site investigation and considerably more data may be required than hitherto. An estimate of the requirement for detailed site investigation should be made as part of the feasibility study. The entire process of project design should now be completed with sufficient accuracy to minimise the risk of changes being required after contracts have been awarded.

6.3.4 Material Inspection and Testing

Ground Investigations involve the physical examination, sampling and in situ testing of the soils and rocks underlying and adjacent to the route corridor in order to determine geotechnical and engineering properties relevant to the appropriate design of LVRs. Ground investigations should provide a description of ground conditions relevant to the proposed works and establish a basis for the assessment of the geotechnical and road engineering parameters relevant for all stages of the Project Cycle. Ground investigations for construction materials determine the nature and extent of proposed construction materials sources as well as their relevant geotechnical parameters. They may also be required to provide relevant information on groundwater needed for geotechnical design and construction. Specialist investigations may be required to collect information about identified geo-hazards.

The information required for design is usually obtained by analysing representative samples taken from the route alignment and bridge locations, and by testing the samples for basic properties (grading and consistency), strength, and compressibility. On rare occasions, it may be necessary to carry out cable percussion boring or rotary drilling to obtain samples. However, this equipment is unlikely to be available on most low volume road projects.

Test pitting is a cheap and simple method of subsurface exploration. The pit is normally dug by hand, but a mechanical excavator can be used if available to remove the bulk of the material before the sides and bottom are squared and cleaned for examination. Test pits supply excellent data on subsurface conditions and enable a clear picture of the stratification of the soils to be obtained as well as the presence of any lenses or pockets of weaker material and the level of the water table.

A test pit should be at least 1 m square at the bottom. The maximum practical depth to which a pit can be excavated is about 3 m. It is important to ensure, though, that adequate support is provided below a depth of about 1.5 m or that the sides are battered back to provide a safe working environment, particularly in moist and wet materials.

Nearby excavations and quarries can reveal soil and rock types and their stability characteristics. There may also be buildings and other structures in the vicinity of bridge sites that have a settlement history due to the presence of compressible or unstable soils.

A specialist service provider who can undertake boring, sampling, field and laboratory testing, and soil mechanics analysis is necessary for sites where problems may be expected. It is advisable to use a single organisation to undertake the whole investigation, as this provides continuity between the field, laboratory and office work. The description of the soil profiles should follow a systematic standard process, typically used in Tanzania. Details in this regard are included in the Field Testing Manual (MOW, 2003).

The engineering properties necessary for design are best obtained from laboratory testing of soil samples recovered during subsurface explorations. Testing programmes vary greatly in size and scope depending on the type and phase of the road project and associated works. Testing should not be commissioned on an arbitrary or ad hoc basis but should be part of a rationally designed programme to fulfil clear objectives in order to avoid unnecessary costs: for instance, carrying out unconsolidated-undrained triaxial tests which are deemed to be the cheapest and quickest triaxial tests but give results that are effectively meaningless on unsaturated soils. The relationships between in situ conditions and the in-service performance of the sampled and tested material need to be carefully considered when designing and developing the test regime and the effects of sample disturbance must always be considered.

In the majority of cases no single test procedure will satisfy all requirements and a battery of tests will be needed. An appropriate test programme will include a logical selection and sequence of tests that are a function of the geotechnical environment, the nature of the investigation and the road design requirements.

It is necessary that testing requests be clear and sufficiently detailed. The owner should require that all testing be performed in accordance with appropriate specifications for laboratory testing as in the Laboratory Testing Manual (MOW, 2000).

6.4 PROBLEM SOILS AND SUBGRADE INVESTIGATIONS

6.4.1 General

It is important to delineate the subgrade conditions as these will dictate the pavement structure required to carry the design traffic over its design life. Different procedures are required for new roads and roads that are being upgraded from gravel to paved standard. The objective is to understand the underlying materials and to identify any possible problem soils.

For new roads, the objective is to define the condition and quality of the underlying materials through a centre-line survey. This is usually done by excavating test pits to a depth of at least 800 mm below the “top soil” along the proposed centre-line of the road, which are described and sampled as necessary. The frequency of testing depends on the terrain and variability of the geology/materials, but typically the recommendations in Table 6-3 should be followed.

Table 6-3: Materials testing frequency

Project	Indicator tests (No per km)	CBR tests (No per km)	DCP tests (No per km)	Minimum number of DCP tests per uniform section	
				Min for statistical validity	Absolute minimum
New road	≥ 3	≥ 2	≥ 5	8	5
Existing gravel road	≥ 2	≥ 1	≥ 5	8	8

For existing gravel roads, test pits should also be excavated (at lower frequencies than for new roads) and a full DCP survey should be carried out along the road. The objective here is to determine the in situ strength of the existing road structure. Minimum frequencies of testing are also shown in Table 6-3. The

DCP tests should follow a pattern of left outer wheel track, centre-line, right outer wheel track, centre-line, left outer wheel track, etc.

The main objective of the centre-line survey is to identify uniform sections so that the materials can be brought up to formation level such that the material at this level will provide a uniform layer of the required strength. For an environmentally optimised design, this is typically at an in situ DCP DN value of less than 14 mm/blow (CBR of about 15%) at the expected density and design (expected worst) moisture condition. Any pavement structure above this would then have similar support and provide a similar response to traffic.

Certain subgrade materials, if not identified timeously, can lead to ongoing road performance problems. Such materials need to be located and some form of remedial action taken. These materials and the actions necessary to minimise their effects are described below and additional information can be found in the Materials and Pavement Design Manual (MOW, 1999).

6.4.2 Expansive Clays

Causes

Expansive clays are widespread internationally and of major economic significance. Typical damage to roads includes longitudinal unevenness and bumpiness, differential movement near culverts and longitudinal cracking. The presence of trees alongside the road often results in localised moisture extraction by their roots with the development of sporadic subsidence and arcuate cracking. Expansive clay damage to roads usually affects their serviceability more than their structural integrity, provided cracking and surface distress is timeously and effectively repaired. Damage is generally restricted to areas that have significant seasonal rainfall or poor surface water drainage.

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

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

Recognition

The simplest way of identifying the presence of expansive soils is through field observations where the surface expression of cracking in dark grey, black or sometimes red soils is evident as shown in Figure 6-2. However, the presence of a thick non-expansive transported or topsoil cover can sometimes mask these cracks and excavation of a test pit, in which cracking and slickensiding of the material will be observed is necessary. The identification of smectite in subgrade soils is best done using X-ray diffraction.

Figure 6-2: Cracking on expansive clay

By their nature, smectites will tend to be more plastic than other clay minerals and a measure of the plasticity index, or better still the activity (ratio of plasticity index to clay fraction) is a good indication of the presence of smectites. This is one of the earliest methods of indicating potentially expansive soils using Figure 6-3 based on the clay fraction of the soil (minus 2 µm) and the standard Plasticity Index (PI), which remains very useful for the preliminary identification of expansive soils. It should be noted that the estimates for the degree of swell using this technique do not take into account the initial moisture content of the material, assuming that they move from a state of dryness normally used in the laboratory to wet. It is known that an equilibrium moisture content develops under a road structure and the moisture fluctuation in this zone is minimal. However, from beneath the outer wheel track of roads with unsealed shoulders to the edge of the fill, significant and variable moisture fluctuations occur. It is unlikely that the initial moisture content in these zones is, however, particularly dry. An alternative method of estimating

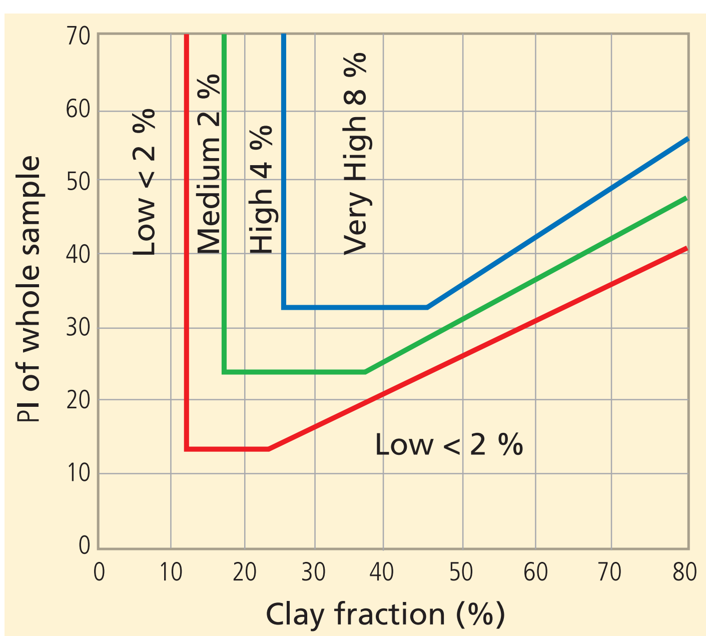

Figure 6-3: Identification of expansive clay soils and estimate of expansion (Van der Merwe, 1976)

potential expansiveness of a soil using the weighted shrinkage and plastic limits is provided in Appendix 6 of the Materials and Pavement design Manual (MOW,1999) and this can also be used. It is often useful to apply a number of methods and base the design on more than one value.

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

Countermeasures

Although the estimation of potential heave is imperative for structures on expansive clay, it is not as critical for subgrades under roads. It is more important to identify the possible existence of the problem and the potential for differential heave along the road and take the necessary precautions. These will generally be based on the expected degree of swell determined from Figure 6-4.

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

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

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

1. Flattening of embankment side slopes (between 1V: 4H and 1V:6H).
2. Remove expansive soil and replace with inert material (between 0.6 and 1 m depending on depth of clay).
3. Retain the road over the clay as an unpaved section.
4. Pre-wetting prior to construction of the fill or formation (to OMC).
5. Placing of uncompacted pioneer layers of sand, gravel or rockfill over the clay and wetting up, either naturally by precipitation or by irrigation (100 to 500 mm depending on clay thickness and potential swell).
6. Lime stabilization of the clay to change its properties (expensive – up to 6% lime may be required).
7. Blending of fine sand with the clay to change its activity (blend ratio to be determined by laboratory experimentation).
8. Sealing of shoulders (not less than 1 m wide).
9. Compaction of thin layers of lower plasticity clay over the expansive clay to isolate the underlying active clays from significant moisture changes.
10. Use of waterproofing membranes and/or vertical moisture barriers, which are generally geosynthetics (only limited success has been achieved using these methods.

Figure 6-4 provides a preliminary indication of possible counter-measure options (numbered as above) as a function of potential expansiveness. It should be noted that usually a combination of these is most effective and all should go together with careful design and construction of side-drains, which should preferably be sealed.

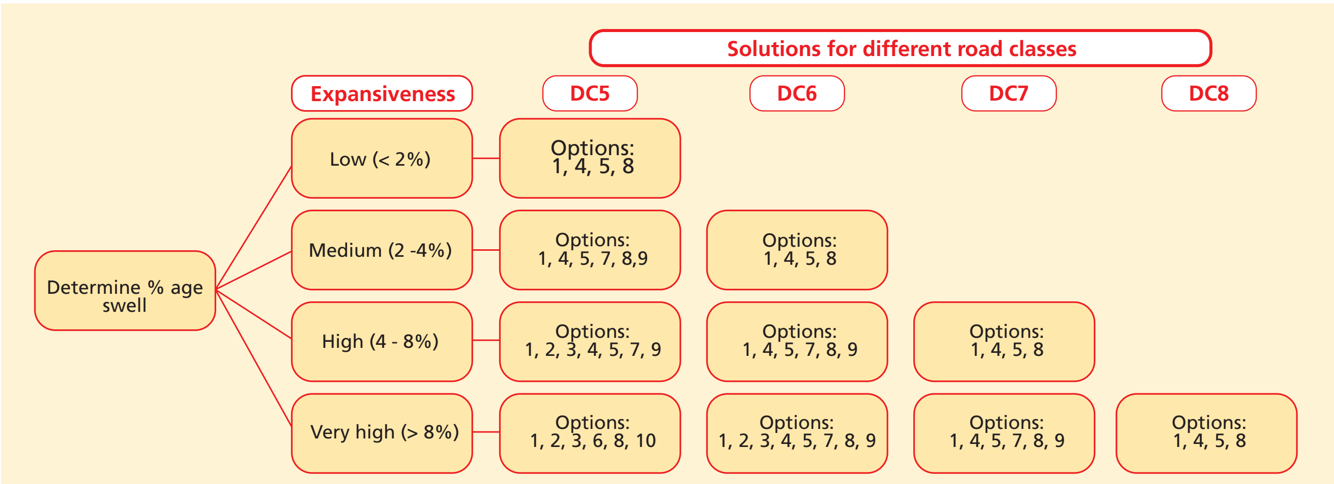

Figure 6-4: Possible solutions for the construction of roads on active clays related to potential expansiveness

In many cases for DC8 class roads, it may be more economic to retain the road as a gravel road over the expansive clay sections and apply the necessary maintenance.

One of the most important considerations is to try and minimise the zone of seasonal moisture movement beneath the road, as shown in Figure 6-5, and to increase the zone of moisture equilibrium. A combination of slope flattening, material replacement, sealed shoulders and lined side drains as shown in Figure 6-6 is usually the most cost-effective means of achieving this, but the design of counter-measures needs to be specific to any situation.

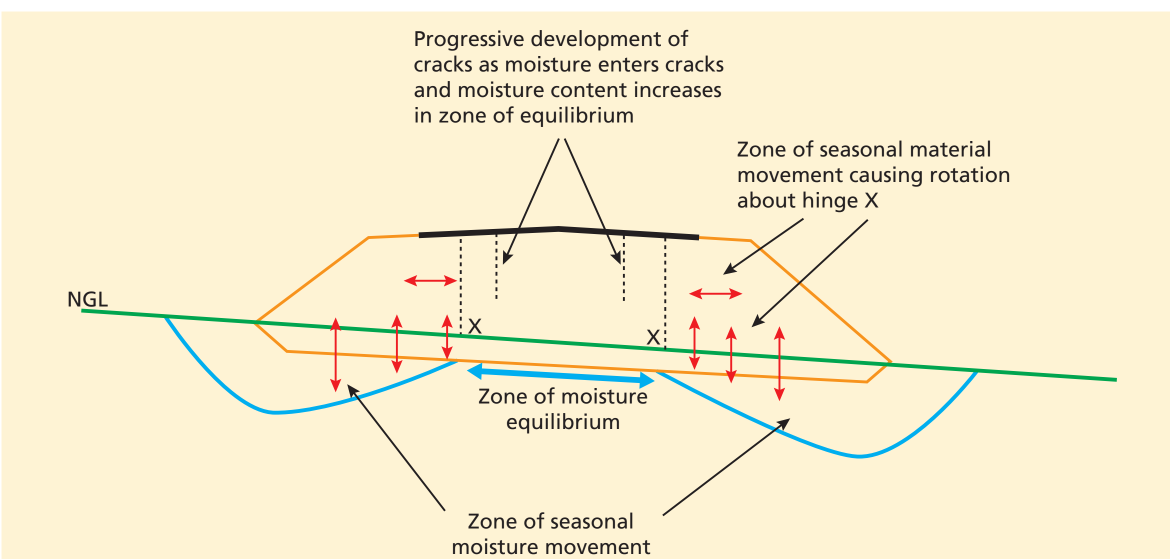

Figure 6-5: Typical moisture movement regime under roads on expansive clays

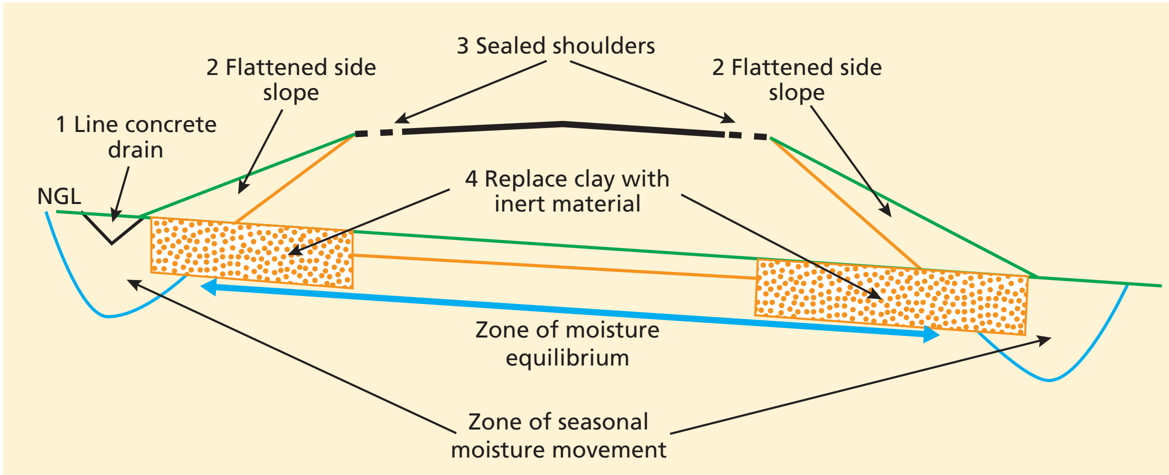

Figure 6-6: Some countermeasures used to increase zone of moisture equilibrium

Expansive clays are often thick and laterally widespread and this makes the implementation of countermeasures costly. The most successful technique for counteracting subgrades susceptible to high movement is to remove the expansive clay beneath the road structure and replace it with a raft of inert material. This would typically involve the excavation and removal of between 600 and 1500 mm (or even deeper in some cases) of material over the entire footprint of the road prism (or at least beneath shoulders and side slopes) combined with drainage structures that remove all water from adjacent to the fill slopes and culverts. Removal of material results in the reduction of the swell potential as well as slightly increasing the load on the expansive subgrade with a usually denser, better compacted material. Unfortunately, this is often impracticable or uneconomic for LVRs, unless the problem is localised. More frequently, expansive materials cover a wide area and the importation of substitute material involves the haulage of large quantities of inert material over long distances.

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

Particular attention should be paid to culverts. The clay beneath them must be replaced with an inert material, all joints must be carefully sealed to avoid leakage and inlets and outlets well graded to avoid ponding of water. It is essential, however, that a proper understanding of the potential moisture movements in and around the road is obtained and this is related to the swell potentials of the various pavement materials (fill, shoulders, subgrade, etc.).

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

6.4.3 Dispersive/erodible/slaking material

Causes

Dispersive, erodible and slaking materials are similar in their field appearance (highly eroded, gullied and channelled exposures), but differ significantly in the mechanisms of their actions. Fortunately for road builders, only the (probably less common) dispersive soils present problems of any consequence. Figure 6-7 shows a typical dispersive soil in Tanzania with definite evidence of piping.

Figure 6-7: Dispersive soil showing formation of "pipes"

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 as shown in Figure 6-8. In moving water, the dispersed particles are carried away. This obviously has serious implications in earth dam engineering, but is of less consequence in road engineering except when used in fills. Dispersive soils often develop in low-lying areas with gently rolling topography and relatively flat slopes. Their environment of formation is also usually characterised by an annual rainfall of less than 850 mm.

Figure 6-8: Dispersive soil in water (crumb test) showing suspension that does not settle out

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

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

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

Recognition

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

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

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

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

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

Countermeasures

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

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

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

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

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

6.4.4 Saline Soils

Causes

Unlike dispersive soils that are affected by the presence of excessive cations of sodium attached to clays, saline materials are affected by the combination of specific cations and anions in the form of soluble salts, independent of clays. These can be a major problem on road projects where migration of soluble salts to beneath bituminous surfacings (Figure 6-9) leads to weakening of the upper base and blistering and disintegration of the surfacings. Soluble salts, particularly sulphates, and their acids can also have a serious detrimental effect on the stability/durability of chemically stabilized materials and concrete.

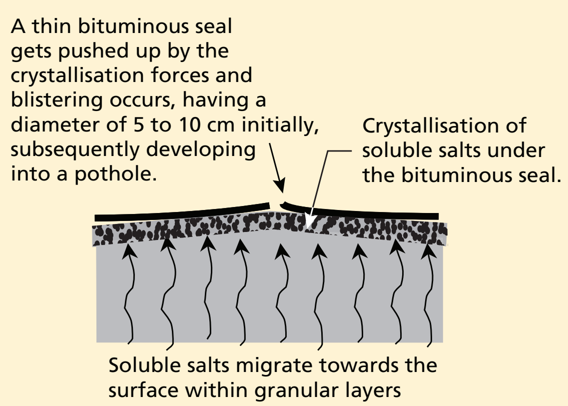

Figure 6-9: Mechanism of soluble salt damage to bituminous surfacings

Soluble salt damage to roads has been reported primarily from arid, semi-arid and warm dry areas. Salts can originate from the in situ natural soils beneath the structures as well as from imported material for the pavement layers or from saline construction water. Only the presence of soluble salts in subgrade materials is considered in this report as the materials for other layers can be controlled provided the problem is identified timeously.

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

Recognition

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

In the conventional road engineering context, the identification of possible soluble salt problems is based on the pH and conductivity of the materials. Most roads departments do not differentiate between the subgrade materials and the imported layer materials.

It should be noted that the results of the electrical conductivity and pH tests can vary significantly depending on the pre-treatment, the moisture content at which the measurements are made and particularly on the material size fraction tested.

Limits for the use of saline materials are generally based on work in specific countries and their applicability to other areas is unknown. In general, an electrical conductivity on the passing 6.7 mm fraction in excess of 0.15 Sm-1 (or an electrical resistance of less than 200 Ω on the minus 2 mm fraction) should raise concern and indicate the need for further investigation. Similarly, soluble salt contents in excess of 0.5% should be a cause for possible concern and lead to additional investigations.

Countermeasures

The following measures should be considered:

- As soluble salt problems arise from the accumulation and crystallization of the salts under the road surfacing and in the upper base layer, minimisation of salts in the pavement layers and subgrade should be attempted.
- If the surfacing is sufficiently impermeable (coefficient of permeability, k in nanometre/second)/ surfacing thickness, T in mm or k/T < 30 (μsec)$^{-1}$) to avoid water vapour passing through it, crystallization will not occur beneath the surfacing.
- Construction should proceed as fast as possible to minimise the migration of salts through the layers. Only impermeable primes should be used, e.g. bitumen emulsions. Figure 6-10 provides an indication of the allowable delay between priming and sealing for various material subgrade salinity.
- The addition of lime to increase the pH to in excess of 10.0 will also suppress the solubility of the more soluble salts.

Even for the lowest classes of road (DC8 and DC7), the effects of excessively saline materials can lead to a rapid and total loss of the bituminous seal and precautions should thus be taken for all road classes. The use of non-bituminous surfacings should be considered over saline materials.

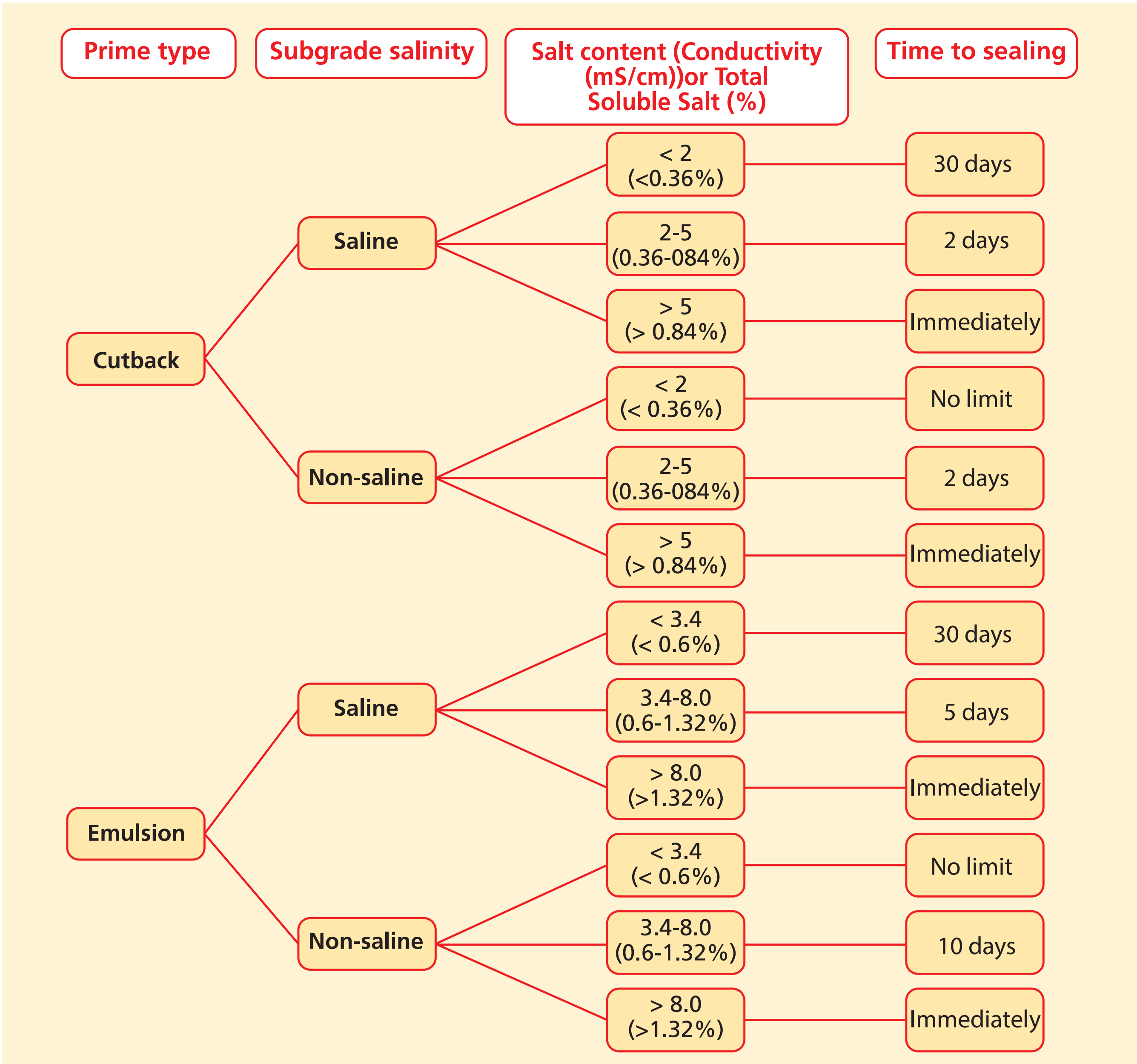

Figure 6-10: Permissible intervals between priming and surfacing related to subgrade salinity

6.4.5 Soft Clays

Causes

Widespread problems, mostly in estuarine (lagoon) and marshy areas result from the presence of very soft alluvial clays in these areas. Deep soft clays in estuarine areas are formed mostly as a result of periodic fluctuations in seal level. Inland soft clays tend to be much shallower having been deposited in marshy areas. Soft clays are generally, but not necessarily saturated and normally consolidated to lightly over-consolidated (as a result of fluctuating water tables). The materials thus have low shear strengths, are highly compressible and their low permeabilities result in time-related settlement problems. In addition, the frequent occurrence of organic material in the clays affects their behaviour and the determination of their properties.

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

Recognition

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

Countermeasures

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

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

6.4.6 Wet Areas/High Water Tables

Causes

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

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

Various moisture indices such as Thornthwaite's Moisture Index or water surplus maps can provide very useful information on potential problems in this regard. Many of the problems encountered in roads are common to specific moisture zones, and these have been highlighted under their respective headings in this document.

Recognition

It is usually easy to recognise potential wet conditions, which are characterised by areas of standing water, specific types of vegetation (reeds, papyrus grasses, etc.), localised muddy conditions and often the presence of crabs and frogs.

Countermeasures

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

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

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

6.4.7 Others

Collapsible Soils

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

Collapsible materials can occur on both residual and transported materials but are not widespread in Tanzania. Many granites and feldspathic sandstones when weathered result in the feldspar altering to kaolinite with the quartz particles staying intact. This forms a honeycomb type of structure, which, when wetted up and loaded, results in shearing or “collapse” of the clay bridges and a settlement or reduction in volume of the material. Certain basalts and dolerites with dry densities of 1200 to 1300 kg/m^3 have also shown collapse potential.

Indications of the possibility of collapsible materials are:

- A very low density, because of the large number of voids separating the quartz framework.
- Densities of less than about 1600 kg/m^3 (mostly in the range 1000 to 1585 kg/m^3).
- The presence of “pinholing” or voiding observed during the soil profiling.
- Usually more than 60% of the mass of the material lies in the 0.075 to 2 mm range and less than 20% is finer than 0.075 mm.
- When the material excavated from a pit is insufficient to fill the pit again (the collapse structure will be disturbed and the material will decrease in volume).

If potentially collapsible soils are identified, specialist assistance should be use for DC5 roads to avoid excessive rutting. The deformation that is likely to affect lower classes of roads will seldom have a major impact on their performance.

The result of collapse of the subgrade is mostly manifested by the development of a deeply rutted and often uneven road surface and significant deterioration of the riding quality of the road as shown in Figure 6-11.

Sinkholes

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

Figure 6-11: Typical manifestation of collapsible subgrade

6.5 GEOTECHNICAL INVESTIGATIONS

6.5.1 General

As discussed earlier, there is a wide range of techniques available for carrying out geotechnical investigations, many of which are specifically suited to certain applications. The art of carrying out a good geotechnical investigation is in being able to decide which techniques will provide the most useful information at the lowest cost. This chapter covers the investigations and designs for dealing with deep cuts and high fills and foundations for structures. It assumes that the need for these investigations has already been identified during the site investigation stage of the project. The full geotechnical investigation would fall under the design phase.

6.5.2 Hilly and Mountainous Areas

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

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

These investigations should consider;

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

Scars, anomalous bulges, odd outcrops, broken contours, ridge top trenches, fissures, terraced slopes, bent trees and misaligned fences, abrupt changes in slope or in stream direction, springs or seepage zones all indicate the possibility of past ground movements.

These areas often require carefully designed drainage structures (usually in conjunction with the cuts and fills) in order to convey concentrated water flows down to lower altitudes and these flow paths need to be identified. Evidence of active erosion and the transportation of large boulders usually make such water courses obvious.

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

Initial investigations for cuts and excavations should concentrate on identifying those areas where additional specialist investigation is necessary. A simple flow-chart can be used for this purpose as shown in Figure 6-12.

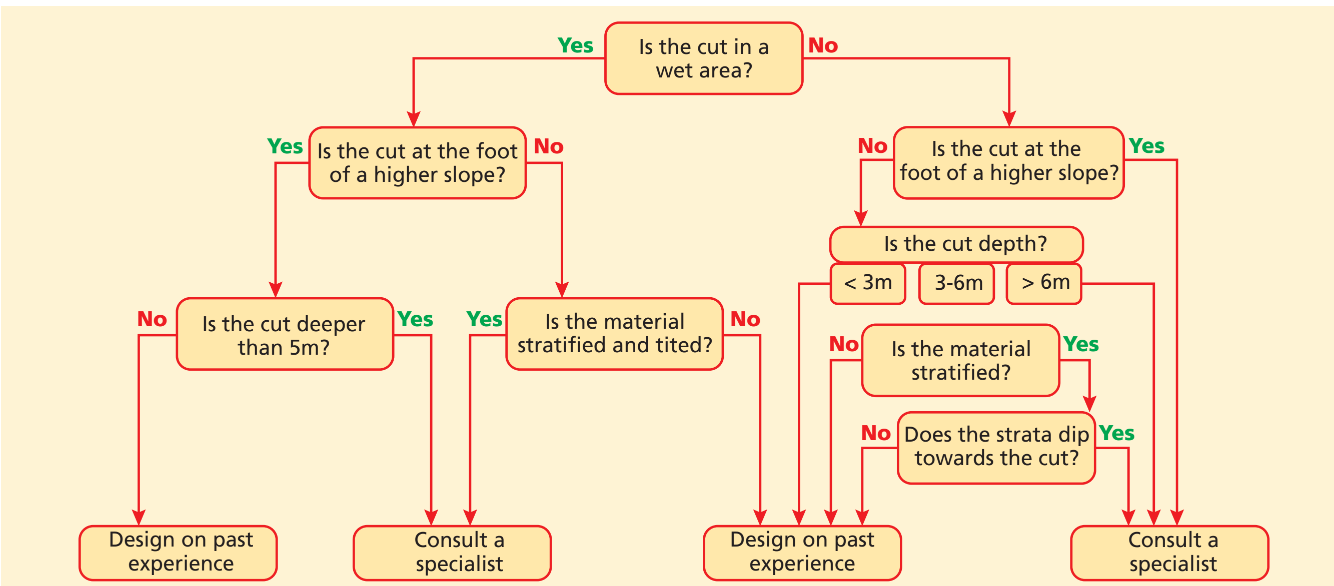

Figure 6-12: Decision chart for design of road cuttings

The question as to whether the cut is at a foot of a higher slope is extremely important. If the cut does not undercut a higher slope, failure will have minimal consequences as shown in Figure 6-13a. However, where the slope continues above a cut, failure of the cut will usually result in large quantities of material higher up the slope becoming unstable and moving onto the road as shown in Figure 6-13.

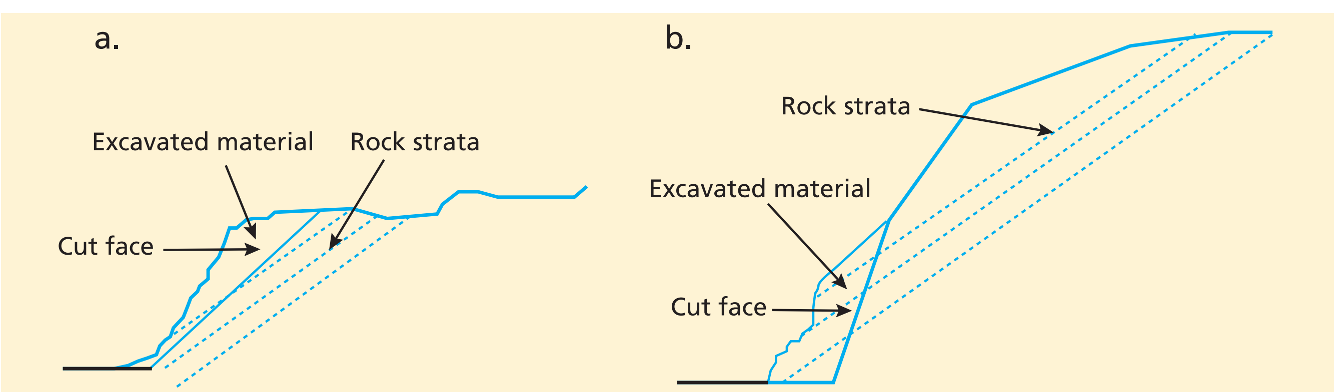

Figure 6-13: Effect of topography on volume of failed material

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

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

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

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

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

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

6.5.3 Foundations

Bridge Foundations

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

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

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

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

The number of trial pits that should be dug will depend on the complexity of the structure and the uniformity of the soil. Table 6-4 gives a guide to the number and depth of trial pits that should be dug for different structures. If the ground conditions are known to vary over the proposed site, or two trial pits show markedly different results, then further trial pits should be dug as appropriate. The trial pit depth is only given as a guideline figure. If the soil conditions are very poor, it may be necessary to increase their depth or carry out deeper investigations using boring or drilling. Where bedrock exists close to the ground surface this offers the best foundation. MOW (2003) should be used as a guide for additional information.

Table 6-4: Trial pits: requirements and locations

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

If the ground conditions are poor at the proposed or expected level of the structure’s foundation it will be necessary to continue excavation to firm material that can provide sufficient bearing capacity.

For larger structures, a range of foundations could be used depending on the materials at site. It is useful to carry out a geophysical survey (seismic or resistivity) to identify the general strata in the area and to provide a basis for siting further exploratory points. This can substantially reduce the number of boreholes or deep auger holes required. Detail can be found in MOW (2003).

Information on aspects such as the depth to bedrock, the strength of overlying soils and the underlying bedrock, scour resistance, etc. need to be identified so that the most cost-effective and appropriate foundation solutions can be designed. Such structures are normally founded on spread footings if acceptable materials are present in the upper 2 or 3 m of the underlying soil profile or piles if weak materials extend to greater depths.

The ground underneath the proposed structure should have an adequate bearing capacity to support the load of the structure itself and the vehicles, which pass over it. If the soil has insufficient strength it will compress and the structure will subside, possibly causing failure. The bearing capacity will depend on a range of different factors including; the proportions of sand clay; organic and other material in the soil; the mineralogy of the clay materials; and the level of the water table. As the type of soil may change with depth it is necessary to dig trial pits at the proposed site to determine the bearing capacity at the proposed foundation level. By identifying and sampling the material excavated from different depths of the trial pits the bearing capacity of the soil can be determined. Bearing capacities are particularly important in the design of structures where large localised loads are expected, (e.g. bridge abutments and piers) as the soil must have a high bearing capacity to support these loads.

The geotechnical investigation should thus provide sufficient information for the engineer to make this decision on the most appropriate design, as the strength and durability of any structure will be determined by the quality of its foundation in relation to the bearing capacity of the underlying material.

Scour

Scour is the erosion of material from the river sides and bed due to water flow as shown in Figure 6-14. Damage due to scour is one of the most likely causes of structural failure. Minimising or eliminating the effects of scour should therefore receive adequate attention when designing any structure. Scour can occur during any flow but the risk is generally greater during floods. There are three major types of scour to be considered and the potential for these should be assessed during the geotechnical investigation:

1. River morphology: these are long-term changes in the river due to bends and constrictions in the channel affecting the shape and course of the channel.
2. Construction scour: this is the scour experienced around road structures where the natural channel flow is restricted by the opening in the structure. The speed of the water increases through the restriction and results in more erosive power, removing material from the banks and bed.
3. Local scour: occurs around abutments and piers due to the increased velocity of the water and vortices around these new unnatural obstructions.

Figure 6-14: Scour at base of bridge pier foundation

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

6.5.4 Construction Materials

The site investigation will usually determine potential construction material sources. However, it is often necessary during the geotechnical investigation to carry out additional investigations, particularly for any hard rock materials required. This is usually related to the excavation (or drilling) of additional pits or holes to determine the extent of the materials and to obtain samples on which to carry out additional laboratory testing for confirmation of the material properties or to assess variability.

6.5.5 Construction Water

As for the construction materials, if insufficient water for construction was identified during the site investigation, or that determined during the wet season is no longer available if construction is carried out in the dry season, additional investigations will be necessary.

The location of water is a specialised activity and groundwater specialists should be consulted if the search is to be for groundwater. The search for groundwater must be planned on a scientific basis and random drilling for water is not recommended, as the process is expensive and in many cases water will not be located.

6.6 GEOTECHNICAL DESIGN

6.6.1 General

Geotechnical design is not a conventional text-book exercise. Every pavement or structure differs in terms of its design and foundation/support requirements and the design must take all of the local factors and the properties of the structure into account. The following section outline various issues to be considered during the design process and highlight certain design criteria.

6.6.2 Earthworks

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

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

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

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

Cut-slopes

Where possible, LVR cut slopes are generally designed on precedent or modified precedent principles (i.e. past experience) as discussed in Section 6.4, based on past experience with similar soil and rock materials. Cut slopes greater than 3 – 6 m in height may require a more detailed engineering geological assessment depending on the complexity of the ground conditions. This would include an assessment of the type of the soil-rock materials and their mass structure as shown in Figure 6-13.

The slope angles indicated in Table 6-5 have been provided as a general guide for LVRs and should be used in conjunction with Figure 6-13. Note that these angles cannot be applied without due consideration of the ground conditions.

Table 6-5: Suggested Cut-Slope Gradients

<table>
<tr><th colspan="2" rowspan="2">Soil/Rock Classification</th><th colspan="3">Slopes (V:H) for Various Cut Heights</th></tr>
<tr><th>< 5 m</th><th>5-10 m</th><th>10-15 m</th></tr>
<tr><td colspan="2">Hard rock (without adverse structure)</td><td colspan="3">1:0.3 – 1:0.8</td></tr>
<tr><td colspan="2">Soft rock</td><td colspan="3">1:0.5 – 1:1.2</td></tr>
<tr><td rowspan="3">Sand</td><td>Loose, poorly graded</td><td colspan="3">1:1.5</td></tr>
<tr><td>Dense or well graded</td><td>1:0.8 – 1:1.0</td><td>1:1.0 – 1:1.2</td><td>-</td></tr>
<tr><td>Loose</td><td>1:1.0 – 1:1.2</td><td>1:1.2 – 1:1.5</td><td>-</td></tr>
<tr><td rowspan="2">Sandy soil, mixed with gravel or rock</td><td>Dense, well graded</td><td colspan="2">1:0.8 – 1:1.2</td><td>1:1.0 – 1:1.2</td></tr>
<tr><td>Loose, poorly graded</td><td colspan="2">1:1.0 – 1:1.2</td><td>1:1.2 – 1:1.5</td></tr>
<tr><td>Cohesive soil</td><td></td><td colspan="2">1:0.8 – 1:1.2</td><td>-</td></tr>
<tr><td colspan="2">Cohesive soil, mixed with rock or cobbles</td><td>1:1.0 – 1:1.2</td><td>1:1.2 – 1:1.5</td><td>1:1.2 – 1:1.5</td></tr>
</table>

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

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

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

Embankments

Embankments may be required to:

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

Embankment design must accommodate two related elements; the design of the embankment itself using available materials and the strength or compressibility of its foundation. Embankment slopes should be designed taking into account both elements; typical angles for embankment fill on sound foundations are presented in Table 6-6.

Table 6-6: Suggested Fill Slope Gradients

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

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

For embankments founded on soft soils the most usual design option in low-cost road engineering is recommend excavation down to satisfactory strength materials where possible. Where this is not feasible then detailed geotechnical analysis will be required. The options of access route alignment to avoid soft soils areas is the most suitable course of action.

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

Cut-fill Cross Sections

Cut-fill cross sections are a combination of excavation into hillside above the alignment and placement of the excavated fill on the “down” side. Although the cut-fill option is attractive in terms of cut-fill balance and is a common situation in many hill or mountainous access routes, it also a frequent cause of access failure unless adequate design and construction precautions are adopted as shown in Figure 6-15.

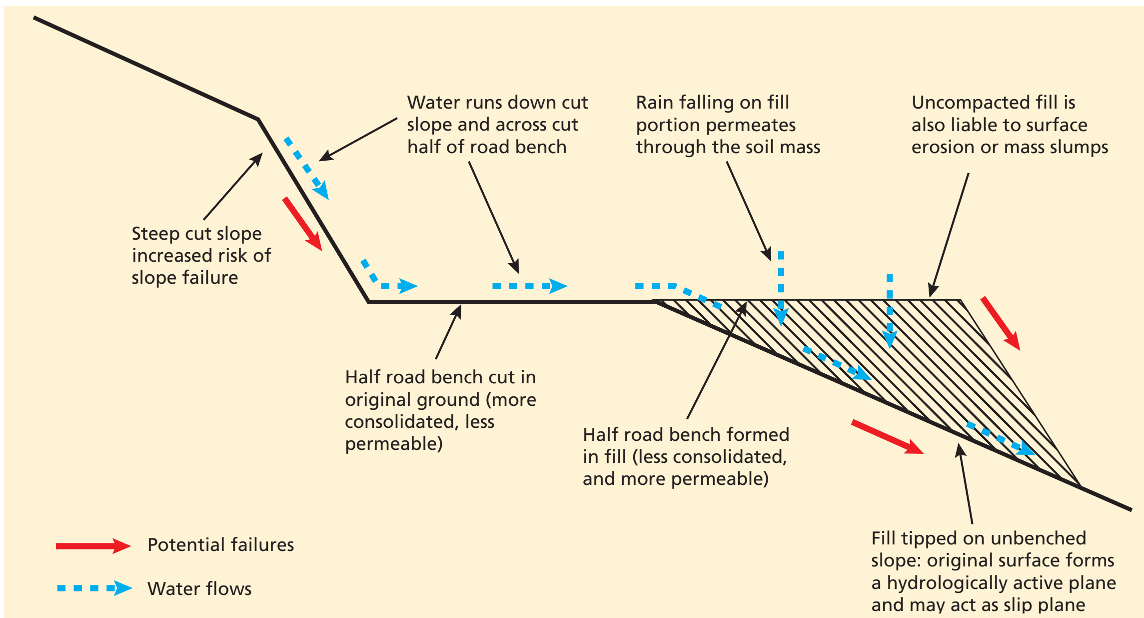

Figure 6-15: Cut-Fill situations

Key requirements for an adequate design are:

- Suitable cut slope excavation, as shown in Figure 6-15.
- Fill section key-in to natural slope.
- Adequate drainage to prevent pore pressure build-up or lubrication of the cut-fill interface and erosion or permeation of water into the slope.
- Specification for compaction of fill in layers and not simply dumped over the alignment edge.
- Specification of complete removal of vegetation and organic material prior to construction.
- Construction of embankments on loose spoil material derived from earlier excavations.
- Prevention of erosion on slopes immediately below the embankment.

Slope Protection and Stabilisation

Various techniques are used to protect and stabilise slopes associated with roads and prevent the occurrence or recurrence of landslides, especially along low volume roads. These are summarised in Tables 6-7 and 6-8.

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

Table 6-7: Stabilisation options for problems above the road

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

Table 6-8: Stabilisation options for problems below the road

Instability	Stabilisation options	Drainage options	Protection options
Erosion of the fill slope surface.	None.	Ensure road-side drainage is controlled.	Bio-engineering a key option.
Failures in fill slope.	Re-grade or remove, replace and compact fill; Before replacing fill, cut steps in original ground to act as key between fill and original ground. A new road retaining wall may be the only option.	Ensure road-side drainage is controlled.	Bio-engineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation.

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

Slope drainage

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

Principal earthwork drainage options include the following measures which are discussed in more detail in *Chapter 12 – Drainage and Erosion Control.*

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

6.6.3 Retaining walls

Retaining walls must be designed to withstand the pressure exerted by the retained material attempting to move forward down the slope due to gravity. The lateral earth pressure behind the wall depends on the angle of internal friction and the cohesive strength of the retained material. Lateral earth pressures are smallest at the top of the wall and increase towards the bottom. The total pressure may be assumed to be acting through the centroid of a triangular load distribution pattern, one-third above the base of the wall. The wall must also withstand pressure due to material or other loads placed on top of the fill behind the wall ("surcharge").

Groundwater behind the wall that is not dissipated also exerts a horizontal hydrostatic pressure on the wall and must be taken into account in the design. Dissipation of ground water is normally achieved by constructing horizontal drains behind the wall with weep-holes.

Gravity walls depend on their mass to resist pressures from behind the wall that tend to overturn the wall or cause it to slide. A factor of safety of 1.5 should be applied to the calculations of overturning and sliding. Gravity walls are normally designed with a slight “batter” to improve stability by leaning the wall back into the retained soil. The foundations should be wide enough to ensure that excessive pressure is not applied to the ground.

The design and construction of gravity retaining walls, including gabion walls, dry stone walls and mortared stone walls are briefly introduced below.

Gabion walls

Gabion walls are built from gabion baskets tied together. A gabion basket is made up of steel wire mesh in a shape of rectangular box. It is strengthened at the corners by thicker wire and by mesh diaphragm walls that divide it into compartments. The wire should be galvanized, and sometimes PVC coated for greater durability. The baskets usually have a double twisted, appropriate size, hexagonal mesh, which allows the gabion wall to deform without the box breaking or losing its strength.

Gabion walls are cost effective because they employ mainly locally available rock and local labour. Gabion structures are commonly used for walls of up to 6 m high. Gabion walls are usually preferred where the foundation conditions are variable, the retained soils are moist, and continued slope movements are anticipated.

Because of their inherent flexibility, they are not preferred as retaining walls immediately below and adjacent to sealed roads due to the likelihood of movement of the backfill behind the wall and subsequent pavement cracking. Where gabion walls are used to support a sealed road, care should be taken to locate the base of the wall on a good foundation, in order to reduce the potential for movement.

Gabion walls have the following advantages:

- Gabions can be easily stacked in different ways, with internal or external indentation to improve the stability of the wall.
- They can accommodate some movement without rupture.
- They allow free drainage through the wall.
- The cross section can be varied to suit site conditions.
- They can take limited tensile stress to resist differential horizontal movement.

Their disadvantages include:

- Gabion walls need large spaces to fit the wall base (this base width normally occupies about 40% to 60% of the height of the wall).
- The high degree of permeability can result in a loss of fines through the wall.

For retaining walls supporting roads this can result in potentially problematic settlement behind the wall, although this can be prevented by the use of a geo-textile (filter fabric) between the wall and the backfill.

Dry-stone walls

Dry-stone walls are constructed from stones without any mortar to bind them together. The stability of the wall is provided by the interlocking of the stones. The great virtue of dry stone walls is that they are free-draining. The durability of dry-stone walls depends on the quality and amount of the stone available and the quality of the work. In a slope management situation, they are useful as revetments for erosion protection and as a means of supporting soil against very shallow movement. Dry stone walls should not exceed 5 m in height.

Mortared masonry walls

As with gabion walls and dry stone walls, a mortared masonry wall design uses its own mass and base friction to balance the effect of earth pressures. Masonry walls are brittle and cannot tolerate large settlements. They are especially suited to uneven founding levels but perform equally well on a flat foundation. Mortared masonry walls tend to be more expensive than other gravity wall options. If the wall foundation is stepped along its length, movement joints should be provided at each change in wall height so that any differential settlement does not cause uncontrolled cracking in the wall.

Mortared masonry walls require the construction of weep-holes to prevent build-up of water pressure behind the wall. Weep holes should be of 75 mm diameter and placed at 1.5 m centres with a slope of at least 2% towards the front of the wall. A filter of lean concrete or geo-textile should be placed at the back of the weep holes to avoid clogging and ensure free drainage of water.

6.6.4 Foundations

The geotechnical investigation for foundations should provide sufficient information for the engineer to make a decision on the most appropriate design, as the strength and durability of any structure will be determined by the quality of its foundation in relation to the bearing capacity of the underlying material.

Small structures such as culverts will normally be constructed on a concrete slab or “raft” and have low bearing pressures requiring no particularly strong support. It is important, however, that the underlying material is volumetrically stable otherwise some degree of reinforcement in the concrete may be necessary to avoid cracking caused by differential movement of the slab.

The design of foundations for bridges will depend on the nature and dimension of each structure. The first decision is whether spread footings or piled foundations are required and this will depend on the depth to material with a suitable bearing capacity.

TThe ground underneath the proposed foundation should have an adequate bearing capacity to support the load of the structure itself and the vehicles, which pass over it. If the soil has insufficient strength it will compress and the structure will subside, possibly causing failure. The bearing capacity will depend on a range of different factors including:

- the proportions of sand and clay;
- organic and other material in the soil;
- the mineralogy of the clay materials;
- the level of the water table.

As the type of soil may change with depth it is necessary to assess the bearing capacity at the proposed foundation level, whether it be for footings or piles (end bearing or friction). By identifying and sampling the material excavated from different depths through the soil profile, the nature and strength of the material can be assessed and the bearing capacity of the soil can be determined. Bearing capacities are particularly important in the design of structures where large localised loads are expected, (e.g. bridge abutments and piers) as the soil must have a high bearing capacity to support these loads.

Small bridges are normally founded on spread footings if acceptable materials are present in the upper 2 or 3 m of the underlying soil profile or piles if weak materials extend to greater depths. Materials subject to scour must be protected as far as possible using a foundation and pier or abutment design that will minimise the scouring effects of fast flowing water, especially turbulence caused by the shapes of the structures.

Larger structures on weak support materials will usually be founded on piles. Two types of pile can be considered, depending on the foundation conditions. If strong material exists at moderate depths (5 – 10 m) an end-bearing pile can be considered where the pile is supported totally by this layer. Enlargement of the pile end (under-reaming) can be considered to reduce the pile loads. Where strong material exists at depth below about 10 m, a friction pile, making use of the friction between the pile sides and the soil can be used. It is often, however, economic to use a combination of end-bearing and friction properties to minimise the length of the pile.

A common problem is the junction between the approach fills and the bridge deck. The bridge structure is normally founded such that little if any settlement movement will occur. The approach fills, however, are placed on the in situ material, (often alluvial silts and clays) that are subject to slow settlement with time. This is difficult to compensate for during construction and usually results in a sag in the road as the bridge deck is approached. It is common practice to repeatedly fill this sag with asphalt as it develops or use a run-on concrete slab.

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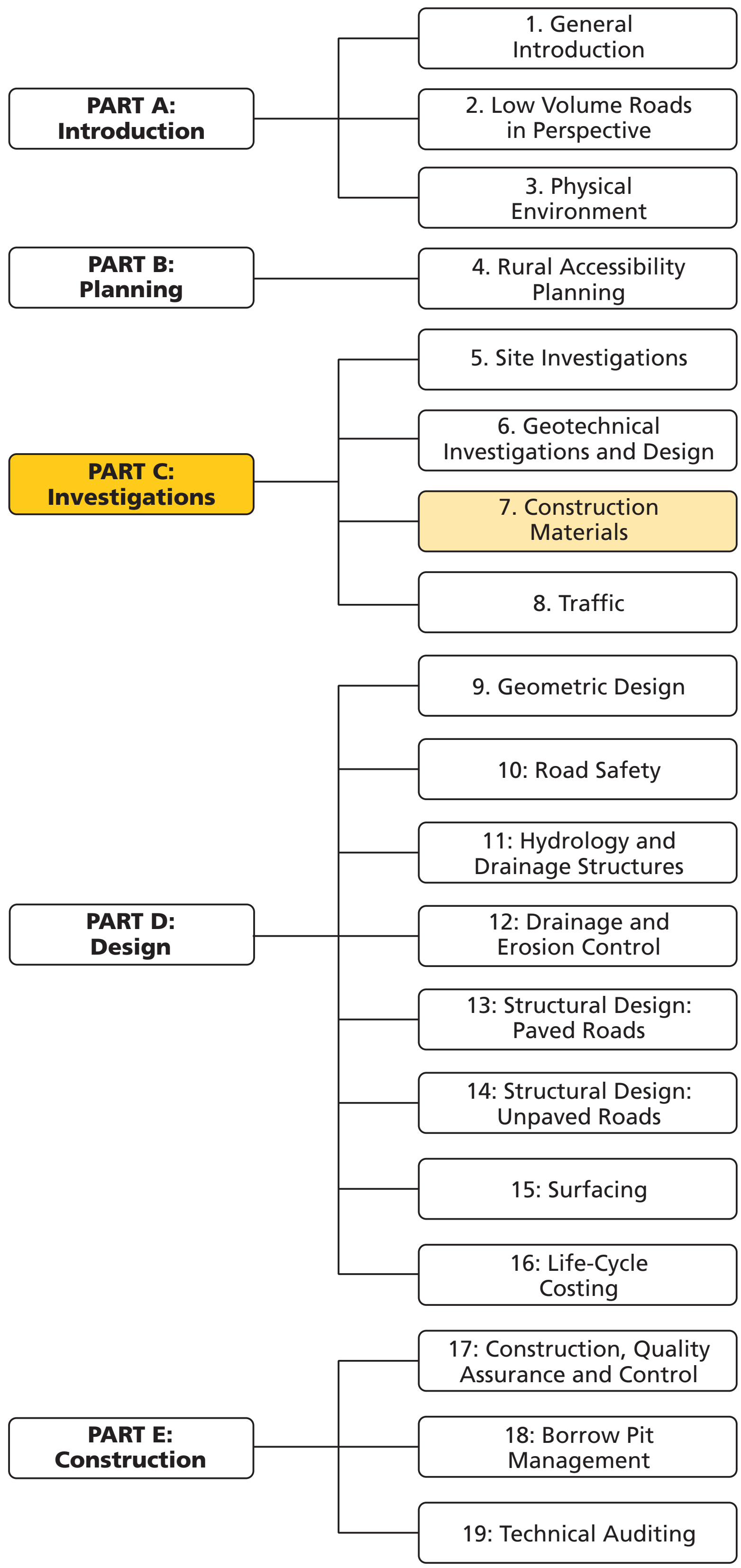
Low Volume Roads Manual



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7.1 INTRODUCTION

7.1.1 General

A large proportion of the total cost of construction of roads is directed towards the location, excavation and processing of materials for the structural layers of the pavements. This is applicable to both unpaved and paved roads and by optimising the use of local materials, significant savings can be achieved.

Unpaved roads, by their nature make use of a sacrificial layer of wearing course material that is lost over a period, typically between 3 and 5 years depending on the traffic, environmental conditions, construction quality and maintenance. Materials most suitable for unpaved roads need to comply with certain requirements for optimum performance and as these are directly related to the design of unpaved roads, they are discussed in detail *Chapter 14 - Structural Design: Unpaved Roads.*

Suitable materials for the different layers in paved roads with the required quantities and qualities must be identified within an economic haulage distance of the road. Previous experience in the area often assists with this but additional investigations for material location are usually essential although efforts should be made to make maximum use of in situ and nearby materials.

The materials usually utilised for low volume roads consist of gravels derived from the weathering of rock, either occurring in situ or transported to a new location. The use of the geological and soil maps, together with a knowledge of the rainfall and climate as discussed in Chapter 2 can assist in identifying potentially useful construction materials.

7.1.2 Purpose and Scope

The purpose of this chapter is to provide the background for the general understanding of the selection and use of materials for the construction of low volume roads in an economic and sustainable manner such that good levels of quality are produced. The chapter has been developed so as to harmonise with the relevant sections described in the Pavement and Design Manual (MOW, 1999) and the Standard Specification for Road Works (MOW, 2000) as far as possible. Innovations have been introduced where considered appropriate.

The scope of the chapter covers the range of construction materials required for all classes of low volume road, which are obtained from borrow sources and hauled for use on the roads. The benefits of using local materials are highlighted and the relevant properties are discussed. Means of locating and improving local material are also briefly introduced.

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

7.2 MATERIAL TYPES

7.2.1 General

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

7.2.2 Materials in Tanzania

Tanzania has a wide range of geological material types as shown on the 1:1 000 000 Geological Map of Tanzania. These rocks vary in age from the Dodoma Series that is more than 3 billion years old to the recent river and coastal sands south of Dar es Salaam which are only thousands or tens of thousands of years old. Over the millennia, these rocks have been acted on by the climate with consequent alteration and weathering to form residual gravels and soils, which are mostly considered in this chapter.

A preliminary indication of the local materials can be obtained from soil maps. These are available at different scales from the generalised 1:1 000 000 to larger scale maps.

7.2.3 Weathered and Residual Materials

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

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

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

Table 7-1 presents a system for describing and classifying the states of weathering in borrow or quarry materials. In this table, the degree of weathering is divided into grades that reflect definable physical changes that could result in modified engineering properties. The general descriptions cover ranges in bedrock conditions and are intended for a rapid assessment of the use of borrow and quarry materials for different purposes in pavement construction.

Weathering tables may generally be applicable to all rock types. However, they are easier to use in igneous and metamorphic rocks that contain ferromagnesian minerals. Weathering in many sedimentary rocks will not always conform to the criteria in Table 7-1. In addition, weathering descriptions and categories may have to be modified to reflect site-specific conditions, such as fracture openness and filling, and the presence of groundwater.

The properties of the final residual material will, however, depend on the mineralogy of the parent rock and with the diverse range of rock types present in Tanzania, a wide range of natural gravels can thus be expected. These can vary from non-plastic quartzitic gravels through to highly plastic and expansive swelling clays. Only some of these are suitable as selected road construction materials as discussed in Section 7-4.

Table 7-1: Weathering classification system

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

7.2.4 Transported Materials

Surficial soils can be moved to different locations by wind, rain, rivers, ice or gravity. During this process the properties of the materials change as large particles are broken down, finer materials are removed and sorting of different size fractions may occur. Many of these transported materials are also suitable for construction of roads.

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

A common rock used for construction in the coastal areas is the so-called “coral rock”. This is a fossilised coral reef consisting of calcium carbonate, which has been cemented together by calcium carbonate to form a material similar in properties to many calcretes. This material can often be ripped as a graded aggregate. However, one of the problems with it is the presence of solution channels or pockets (caused mainly by dissolving of the carbonate by acidic rainwater) that are filled with transported material as shown in Figure 7-1. This obviously results in variable properties and thus the material derived from these coral rocks differs in properties (particularly grading and plasticity) depending on the number and content of these pockets.

Figure 7-1: Transported sandy material (red brown) in dissolution channels/pockets in coral rock

7.2.5 Pedogenic Materials

Pedogenic materials are a unique type of soil in which the existing material is “fully or partially cemented” by certain minerals. Typical cementing materials include iron and aluminium oxides, calcium carbonate and to a lesser extent in Tanzania, silica. These materials can be formed by:

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

These materials, collectively known as pedocretes, can be exceptionally good construction materials, although they frequently do not comply with existing material specifications. Experience with their testing (their unique properties usually require special sample preparation methods) and use will allow a good understanding of their properties and a knowledge of how best to use these materials.

The dominant pedocrete in Tanzania is laterite, which is generally a good construction material, for all layers up to basecourse, if the material properties are tested correctly and understood. A large number of factors control how a particular type of laterite is developed and the material tends to exhibit both vertical and lateral variability within a deep and irregular weathering profile as shown in Figure 7-2.

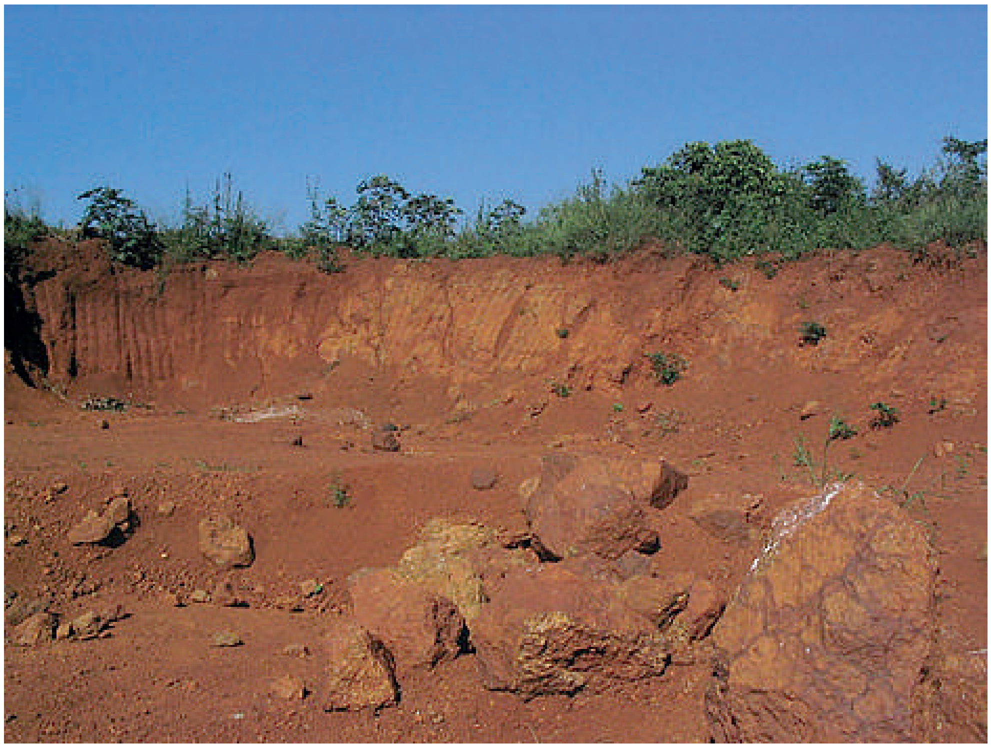

Figure 7-2: Typical laterite borrow pit showing variable nature of the material

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

The requirements for selection and use of lateritic gravels for bases are different to those given for other natural gravels and this needs to be taken into account during their testing. Conventional testing using oven drying for instance, can have a major effect on the test results. Other aspects such as mixing times for the Atterberg limits can also affect the results. For these reasons, the actual properties of laterites in terms of conventional specifications may be quite meaningless.

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

7.2.6 Other Materials

Many other materials occur over limited areas in Tanzania such as the volcanic tuff or scoria in the Kilimanjaro and Mbeya areas and various ultrabasic and basic rocks (eclogite, kimberlite, meta-gabbro, etc.), which can have unusual properties. Such materials need to be investigated individually for specific uses as construction materials according to the standard specifications.

Specific attention should be given to the volcanic ash in the Kilimanjaro area. Much of this material is a natural pozzolan and as such, when activated by lime will result in cementation, producing relatively high strengths. The addition of ash to certain material could have a beneficial effect, which needs to be tested in the laboratory for each ash source.

Table 7-2 summarises the typical properties of the residual gravels obtained from the weathering of various rock types. This should only be seen as a guide, as many local conditions (e.g. perched water tables, good drainage conditions) could affect the actual individual properties.

Table 7-2: Typical properties of residual materials derived from various rock types

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

7.3 THE USE OF LOCALLY AVAILABLE MATERIALS

7.3.1 General

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

Although many naturally occurring materials do not meet conventional specification criteria they, nonetheless, can still provide satisfactory performance. Their choice must therefore be based on locally developed selection criteria, non-standard testing, and attention to construction technique. In addition, it is important to recognise that the specifications for materials must be coupled to the pavement design method being used - either the DCP design method or the traditional or other CBR design methods.

7.3.2 Local materials and low volume roads

The maximum use of naturally occurring unprocessed materials is a central pillar of the LVR design philosophy. Current specifications tend to limit the use of many naturally occurring, unprocessed materials (natural soils, gravel-soil mixtures, gravels and pedocretes) in upper pavement layers in favour of more expensive crushed rock, because they often do not comply with traditional requirements. However, recent research work has shown quite clearly that so-called “non-standard” materials can often be used successfully and cost-effectively in LVR pavements provided appropriate precautions are observed. These precautions include effective drainage of the pavement structure and good construction practices as discussed in other chapters.

The adoption of this approach provides the scope to consider a reduction in specification standard when considering particular material types within defined environments.

Recognising the material’s “fitness for purpose” is central to assessing the appropriate use of non-standard materials. However, the use of such materials requires a sound knowledge of their fundamental properties and behaviour in the prevailing environment.

A key objective in sustainable LVR construction is to best match the available construction material to the road task and the local environment. The benefits of utilising locally available materials arise from: a reduction in haulage costs; less damage to existing pavements from extended haul; stimulation of the local economy and local enterprise; road designs compatible with local maintenance capabilities and, generally, reduced whole life costs.

When reserves are limited or of marginal quality, their relevant usage is a priority and it is important to use materials to ensure that they are neither sub-standard nor wastefully above the standards demanded by their engineering task. Hence the necessity of deriving locally relevant specifications and either adapting designs or modifying materials to suit. Further guidance on the use of marginal materials is contained in Chapter 13 of this document.

Figure 7-3 illustrates the potential benefits of innovatively using alternative materials for road construction.

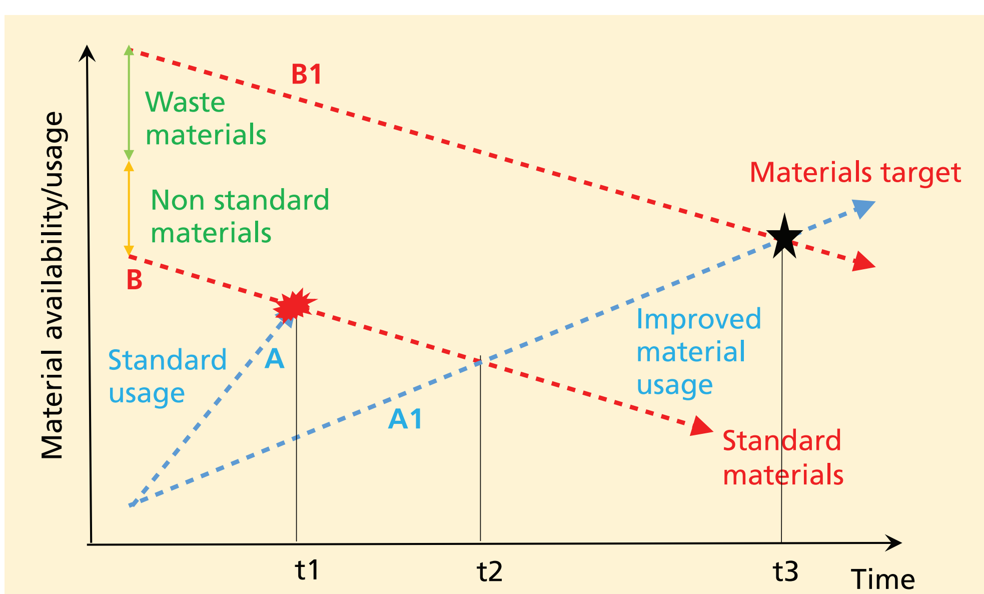

Figure 7-3: Impact of increasing the use of non-traditional road construction materials

The potential benefits of innovatively using natural and alternative materials for road construction are illustrated in Figure 7-3, the potential benefits of innovatively using natural and alternative materials for road construction. It can be seen that continued use of existing material standards (standard usage – blue line A) and standard materials (red line B) will result in depletion of potential sources at time t1. The increased availability of natural materials can only be achieved by changing the required materials standards to those more appropriate for the road category, i.e. not using material that is suitable for high quality roads in lower volume roads that do not require such high material standards, and will reduce the cost of construction materials. This will result in the potential to build more roads (shown as improved material usage in Figure 7-3 shown by blue line A1). By doing this, the life of the existing natural material sources can be extended to time t2 in Figure 7-3. However, by increasing the use of alternative materials (by-products or what are now considered to be waste materials as well as materials not complying with current specifications) the availability of construction material resources will be significantly increased (red line B1) and the time before usage exceeds availability will be extended to time t3.

If the project is in an area where good quality construction materials are scarce or unavailable, consideration should be given to:

- Modifying the design requirements.
- Modifying the material (eg mechanical or chemical stabilisation).
- Material processing (eg crushing (Figure 7-4), screening, blending).
- Innovative use of non-standard materials (particularly important for LVRs).

Figure 7-4: Coarse river gravels can be crushed and used for concrete and/or surfacing works

Many material manuals classify construction materials into various categories, primarily dependent on the material strength (e.g. G80 or G60 (MOW, 1999) with CBRs of 80% and 60% respectively). These categories, however, also frequently have additional criteria for other properties such as grading, plasticity and particle strength. Many local materials may comply with the material strength criterion but not the other requirements and would thus be rejected. In low volume roads, it is the strength (or ideally stiffness) that is critical and provided that this is mobilised (and retained under the expected prevailing conditions in the short and long term), there is no reason why the material should not perform satisfactorily. In most cases the strength categories are based on the soaked CBR, which is not necessarily the optimum design requirement, particularly in moderately dry to dry areas and where the pavement drainage is properly designed, installed and maintained.

7.3.3 Beneficial characteristics of local materials

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

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

The relative dependence of a material, and the influence of moisture, on each of the above components of shear strength will significantly influence the manner in which they can be incorporated within a pavement. In this regard, Table 7-3 summarises the typical relative characteristics of unbound and bound materials that critically affect the way in which they can be incorporated into a pavement in relation to their properties and the prevailing conditions of traffic, climate, economics and risk.

Table 7-3: Pavement material types and characteristics

Parameter	Pavement Type			
	Unbound			Bound
	Unprocessed	Moderately Processed	Highly processed	Very highly processed
Material types	Category 1 As-dug gravel.	Category 2 Screened gravel.	Category 3 Crushed rock.	Category 4 Stabilised gravel.
Variability	High	Decreases		Low
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

Of particular significance to LVSRs

Unprocessed materials (Category 1) such as laterite are highly dependent on suction and cohesion forces for development of shear resistance which will only be generated at relatively low moisture contents. Consequently, special measures have to be taken to ensure that moisture ingress into the pavement is prevented, otherwise suction forces and shear strength will be reduced (Figure 7-5) which could result in failures.

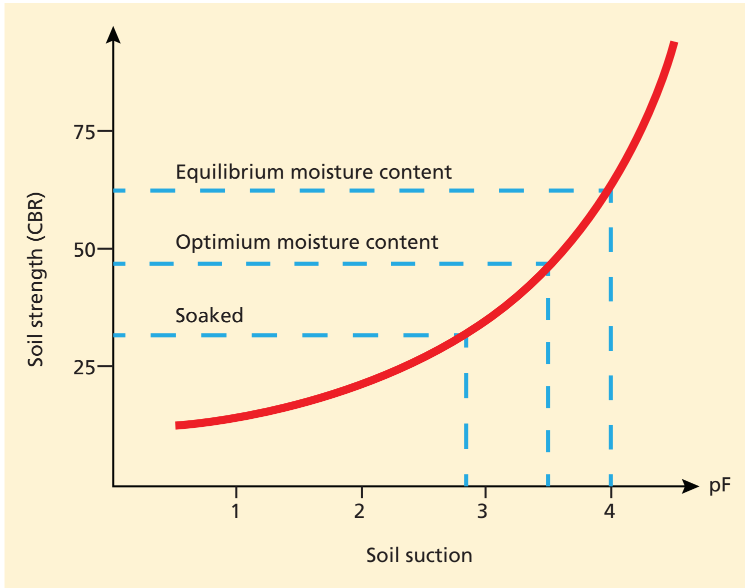

Figure 7-5: Illustrative soil strength/ suction relationship

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

- **Category 1 materials:** are highly dependent on soil suction and cohesive forces for development of shear resistance. The typical deficiency in hard, durable particles prevents reliance on inter-particle friction. Thus, even modest levels of moisture, typically approaching 60% saturation, may be enough to reduce confining forces sufficiently to cause distress and failure.
- **Category 2 materials:** have a moderate dependency on all forms of shear resistance – friction, suction forces and cohesion. Because these materials have rather limited strength potential, concentrations of moisture, typically 60-80% saturation may be enough to reduce the strength contribution from suction or cohesion sufficiently to cause distress and failure. This would occur at moisture contents lower than those necessary to generate pore pressures.
- **Category 3 materials:** have only minor dependency on suction and cohesion forces but have a much greater reliance on internal friction which is maximised when the aggregate is hard, durable and well graded. Very high levels of saturation, typically 80-100% will be necessary to cause distress and this will usually result from pore pressure effects.
- **Category 4 materials:** rely principally on physio-chemical forces which are not directly affected by water. However, the presence of water can lead to distress under repetitive load conditions through layer separation, erosion, pumping and breakdown.

The management of moisture during the construction and operational phases of a pavement affects its performance, especially when unbound, unprocessed, generally relatively plastic materials are used. It is therefore very clear that emphasis should be placed on minimising the entry of moisture into a LVR pavement so as to ensure that it operates as much as possible at an unsaturated moisture content. The beneficial effect of so doing is illustrated in Table 7-4 which shows the variation of a material's strength (CBR) with moisture content.

Table 7-4: Variation of CBR with moisture content

Laboratory Soaked CBR (%)	Approximate Laboratory Unsoaked CBR (%) at varying FMC/OMC Ratios		
	1.0	0.75	0.50
80	105	150	200
65	95	135	185
45	80	115	165
30	65	95	140
15	45	70	110
10	35	60	100
7	30	50	85

The field moisture to optimum moisture content (FMC/OMC) ratio is a significant contributory factor related to the performance of a LVR. If, through effective drainage, the materials in the road pavement can be maintained at a field moisture content that does not rise above OMC in the rainy season, then more extensive use can be made of local, relatively plastic materials that might otherwise not be suitable if they were to become soaked in service.

7.3.4 An Alternative Approach to Assessment of Construction Materials

Although a modified approach to the traditional CBR design method is described in Chapter 13, an alternative and potentially simpler approach using the DCP method is also provided. In order to optimise the use of naturally occurring materials in this method, a holistic approach is required in which attention is paid to the compatibility between the pavement structure, the materials used, the type of surfacing, construction processes and, above all, control of moisture through effective drainage. Moreover, where some degree of risk in long-term performance can be accepted, then strict requirements may be relaxed and a wide range of naturally occurring non-standard materials may successfully be used. However, such use 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 17 – Construction, Quality Assurance and Control).*
- **Probability of failure:** there must be a realistic acceptance of a possibly higher risk of lesser performance.

Although it is necessary to have a fundamental knowledge and understanding of the traditional properties of the materials in each layer, the design procedure will not make use of these. By following the proposed paradigm shift in the use of material properties, it is easier to dissociate the knowledge and any conservative philosophies of the past from the new techniques discussed.

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 and requires the addition of a minimum thickness base (sometimes subbase) layer of appropriate quality. Such 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 differ markedly from the more traditional design approaches. In these latter approaches, 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, which 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 might be specified for the base layer of a LVR.

Using the optimised in situ material properties, many local materials can be utilised in the road without increasing the risk of failure. Essentially, a lower quality material can be utilised, e.g. a G25 in place of a G45 for instance, assuming that the moisture content in the road will not exceed a certain percentage of the optimum moisture content for that material. This assumption is taken care of in the design (refer to Chapter 12 and Chapter 13).

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 will be 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.

The above aspects are discussed in relation to pavement design in Chapter 13.

7.4 ASSESSMENT AND SPECIFICATION OF MATERIAL QUALITY

7.4.1 General

Any material proposed for use in roads should be tested to determine its basic properties. The normal classification tests (Atterberg limits, grading, compaction characteristics and strength (CBR)) carried out on samples from the road or proposed borrow sources will give an early indication of the potential suitability of the material. The methods and traditional specification limits for the use of these materials for various applications/layers in roads are summarised in the Pavement and Materials Design Manual (MOW, 1999).

These specifications are applicable to materials for all classes of road including low volume roads, although some simplifications and “relaxation” of specifications (or more correctly the use of more appropriate specifications) are applicable to low volume roads. It is, however, useful to carry out the traditional testing for the engineer to get a feel for the material in relation to past experience.

7.4.2 Material testing

The testing of materials should be such that they can be classified according to the standard Tanzanian classes (refer to Tables 5.4, 7.2 and 7.3 in the Pavement and Materials Design Manual (MOW, 1999)). These classes are based on a wide range of material properties, but mostly related to the CBR. It is notable that not only the soaked CBR but also the unsoaked (tested at optimum moisture content) CBR is permitted in dry zones. A relaxation of the Atterberg limits is also allowed for certain materials (coral rock, calcrete or other calcified materials). One of the problems with a multi-criteria classification system such as this is that many potentially suitable materials can be excluded from use because they do not comply with one or more of the less relevant criteria.

For low volume roads, it is usually not critical if the standard specifications for grading, Atterberg limits, and aggregate hardness are not fully complied with provided that the material strength requirements are met. An understanding of the effects of these variations on the properties of the material will assist in making a decision as to whether to use the material or not, remembering that the proposed sealed low volume road is unlikely to be subjected to excessive heavily loaded vehicles.

The following two sections focus on the use of materials related to two low volume road design methods, the Dynamic Cone Penetrometer (DCP) test method and a modified CBR method (taking into account the local environmental conditions) and their relevant material characterisation processes. The design procedures are discussed in later sections (Chapters 13 and 14).

7.4.3 Materials for the DCP DN Design Method

Materials selected for use in the upper layers of a sealed low volume road should exhibit the following attributes at the specified density and anticipated in-service moisture content:

- adequate bearing capacity under any individual applied load;
- adequate bearing capacity to resist progressive failure under repeated individual loads;
- the ability to retain bearing capacity under various environmental influences such as climate, drainage and moisture regime.

The criterion to be used for selecting a material for use in a LVR should thus be based primarily on its strength as measured by resistance to penetration, or DN value. A proper evaluation of the suitability of the materials for incorporation in the pavement will require a knowledge of the DN/moisture/density relationship as discussed below.

- Strength – The required strength of the material is determined in terms of a laboratory DCP DN value at a specified moisture and density.
- The strength/density/moisture relationship: The moisture and density dependence of the strength of the materials to be used in the imported upper/base layers of the new road must be evaluated carefully so that a full understanding is obtained of the potential performance of the material under the possible moisture conditions which may occur in service.

Achievement of the above will require that a normal borrow pit investigation is carried out with representative samples being obtained for laboratory testing to determine the DN value at varying moisture contents and densities. This should always be carried out on the traditional maximum dry density test moulds as shown in Figure 7-6.

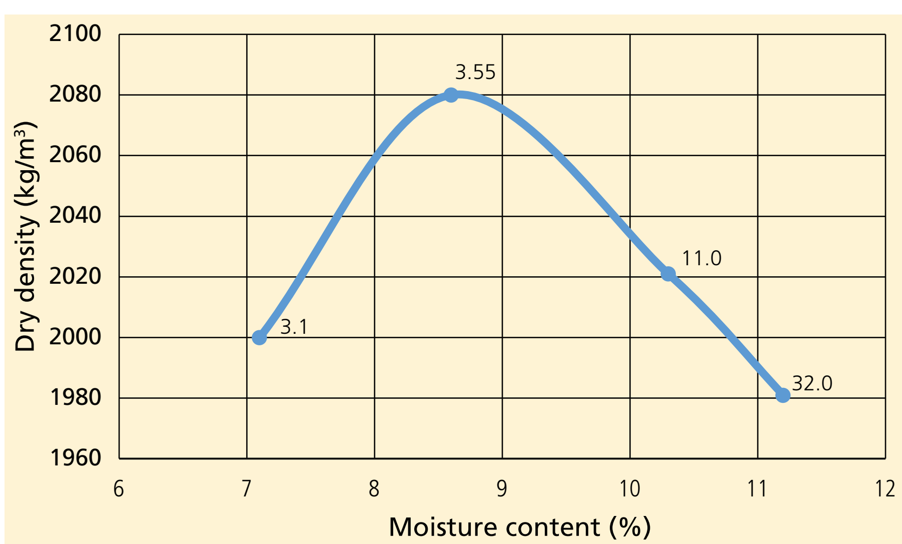

Figure 7-6: MDD curve with DN values (mm/blow) determined on each mould

Figure 7-7 shows the typical relationships between DN, density and moisture content for a naturally occurring material which illustrates two critical factors that crucially 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. This will result in a stronger material with lower voids and a reduced permeability, enhancing the overall properties of the material. Compaction to refusal is indicated by the point at which no additional density is achieved for any specific compaction effort, but any additional compaction effort may result in breakdown of individual particles of the material.

b) The need to ensure that the moisture content in the outer wheel track of the road does not rise above OMC. This will require careful attention to drainage, as discussed in *Chapter 11 – Hydrology and Drainage Structure.*

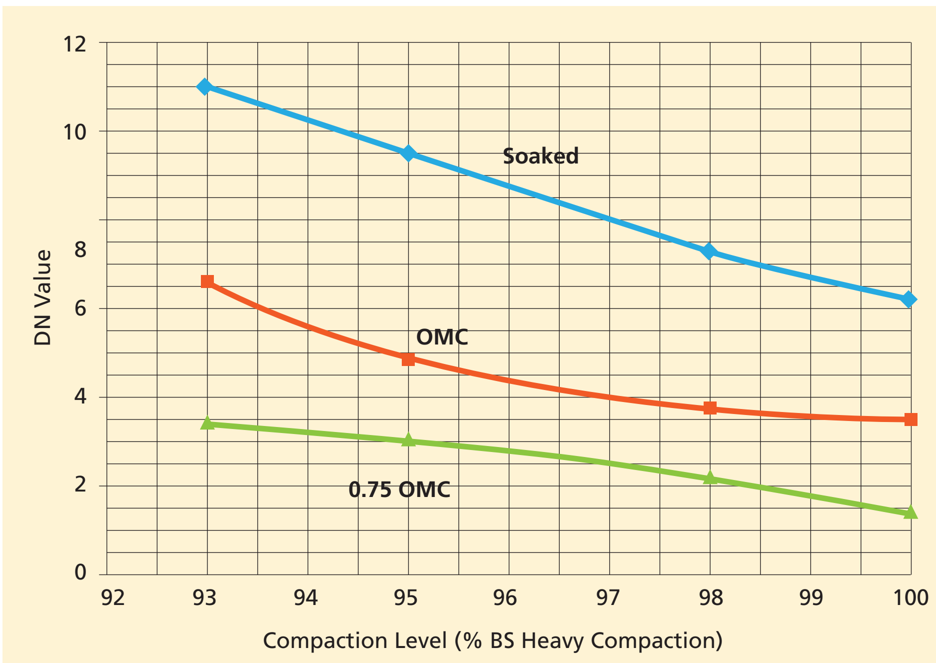

Figure 7-7: DN/density/moisture relationship

(1) Grading modulus (GM): This parameter is expressed by the relationship:

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

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

The inclusion of GM as a specification criterion is to avoid the unnecessary testing of materials that are patently unsuitable for use in a pavement layers in terms of their grading and/or plasticity, e.g. very fine, plastic soils or very coarsely/poorly graded gravels. It should be noted that certain sands (typically with a GM of about 1.0) can perform well in roads and should be assessed carefully using their grading, plasticity on the 0.075 mm fraction and strengths before excluding them as a construction material.

Testing of Materials

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

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Table 7-5: Sampling frequency

Intended Use	Required volume (m^3)/ DN test
Base course	5 000 m^3
Subbase	10 000 m^3

The manner of dealing with oversize particles in the sample preparation for DN testing should be strictly in accordance with the CML Method 1.9 (Procedure for coarse materials using CBR mould) which may be summarized as follows:

- Screen field sample on 20 mm sieve.
- Crush oversize material to pass 20 mm sieve (maximum 30%).
- Add to – 20 mm material from original sample and mix thoroughly.

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

(a). Thoroughly mix and split each borrow pit sample into nine sub-samples for DN testing in a CBR mould at three moisture contents, viz:

- Soaked.
- OMC.
- 0.75 OMC .

and three compaction efforts, viz:

- BS Light.
- BS Intermediate.
- BS Heavy as summarised in Table 7-6.

Table 7-6: 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

(b). The compacted samples should be allowed to equilibrate for the periods shown below before DN testing is carried out to dissipate pre-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.

Each of the nine specimens should be subjected to DCP testing in the CBR mould as summarised below and illustrated in Figure 7-8.

(a) Secure the CBR mould to the base plate and compact the sample in standard CBR 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 as shown. 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 middle 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 (An example is shown in Table 7-7).

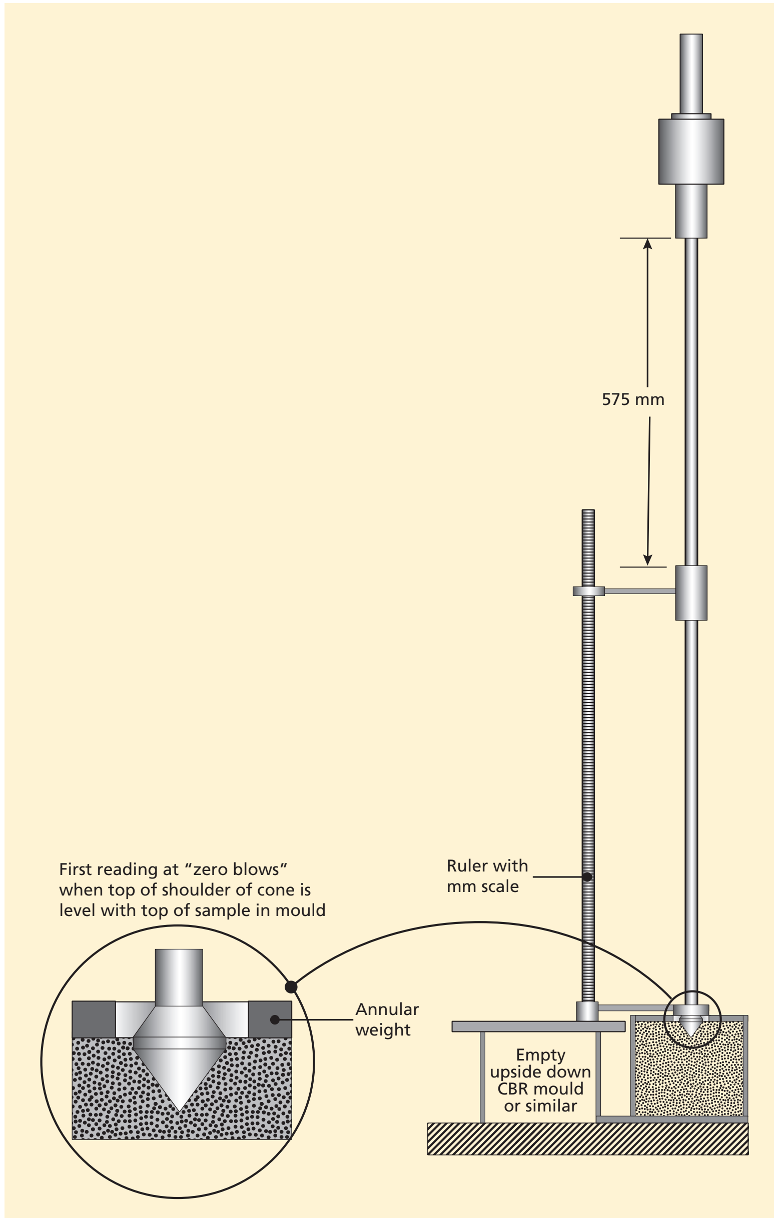

Figure 7-8: Determination of laboratory DN value

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:

$$DN = \frac{\sum \, (Avg\ DN\ per\ blow\ x\ DN\ per\ n\ blows)}{Penetration\ depth}$$

   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.
3. Carry out at least 2 more tests on the same material and calculate the average DN for the three (or more) tests.
4. Assess whether the material satisfies the design criteria from the DCP Design Catalogue (refer to *Chapter 13 – Structural Design: Paved Roads*).

Table 7-7: Determination of laboratory DN values at varying moisture contents 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	20.0	5	137	8	1.6	5	141	18	3.6
1	167	17	17.0	5	149	12	2.4	5	151	10	2.0
1	180	13	13.0	5	164	15	3.0	5	165	14	2.8
1	190	10	10.0	5	178	14	2.8	5	178	13	2.6
1	215	25	25.0	5	194	16	3.2	5	190	12	2.4
					216	22	4.4	5	206	16	3.2
								2	214	8	4.0
Penetration depth		85				87				91	
Average DN			17.0				2.90				2.94
Weighted Average DN			18.62				3.15				2.96

Table 7-8 shows the relationship between the gravel classification of materials based on their laboratory CBR (MOW, 1999) and the equivalent DCP DN values determined in the field or on CBR moulds in the laboratory for comparative purposes. However, the actual in situ strength properties are of more importance.

For many decades it has been known that 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 following laboratory DN values, as shown in Table 7-9, are necessary to provide the required field DN value for the strengths required for base and subbase materials as shown in Table 13-2.

Table 7-8: Relationship between CBR and DCP DN values for various material classes (MOW, 1999)

Material class/property	G80	G60	G45	G25	G15/S15	G7/S7	G3/S3
CBR (%) (wet areas)	≥ 80	≥ 60	≥ 45	≥ 25	≥ 15	7 -14	3 - 6
Equivalent DN (mm/bl)	≤ 3.62	≤ 4.54	≤ 5.7	≤ 9.05	≤ 13.5	24.5–14.3	48.0–28.0
CBR (%) dry areas	≥ 60	≥ 45	≥ 25	≥ 15	≥ 7	3 - 14	2 - 6
Equivalent DN (mm/bl)	≤ 4.54	≤ 5.7	≤ 9.05	≤ 13.5	≤ 24.5	48.0–14.3	66.0–28.0

Note: Equivalent DN values based on Kleyn relationship (Kleyn, 1982).

Table 7-9: Relationship between required field DN and laboratory DCP DN values for various base and subbase materials

Max Field DN value (mm/bl)	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/bl)	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 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. These fundamental specification principles are the basis of the DCP DN design method described in *Chapter 13 – Structural Design: Paved Roads.*

Note Regarding CBR/DCP Testing

There are significant differences between the results of the CBR carried out using the CML (similar to BS) and the South African (SA) test methods. These can have a significant effect on the correlation between the CBR and the DN value as discussed below. All discussion is related to the development of the DN-CBR correlation by Kleyn in the 1970s.

The standard method of sample preparation used in South Africa for the CBR test was to scalp the material at 19.0 mm, crush all of the material retained and add this back to the sample for testing. This differs significantly from the CML method which has a number of alternatives, but generally uses either the full grading up to 20 mm (for the 1 litre mould) or else replaces the fraction retained on the 37.5 mm sieve with an equivalent mass of the 20 to 37.5 mm material from the original sample for the larger mould. In both methods the grading is severely interfered with, depending on the quantity of material larger than 20 mm.

The compaction effort applied by the South African method is also much less than that applied by the CML method. The so-called Mod AASHTO effort is about 10 % less than that of the CML heavy effort (246,039 versus 272,401 kg/m^2 respectively) and the SA Proctor compaction is about 25% less than the CML light compaction effort (54,172 versus 67,500 kg/m^2 respectively).

In the South African test, after compaction the CBR sample is inverted and soaked and then penetrated on the lower (most densely compacted) side unlike the CML method, which penetrates the top (least compacted side) of the sample – there is an option to penetrate the base of the sample as well, but this is not standard.

The SA method places a 5.56 kg surcharge on the specimen before penetration, while the CML method places a variable (selected by the engineer) number of 2 kg surcharges before penetration.

The SA method calculates the CBR from the penetration load at 2.54 mm penetration (using a factor of 13.344 kN of the California standard) while the CML method uses the highest of the 2.54 or 5.08 mm penetration resistances and a factor of 13.2 kN.

There are thus so many (often contradictory) differences that it is difficult to compare the results directly, despite the value of 80% being accepted internationally as the minimum for a conventional base course materials.

In general, however, it can be concluded that a material with a CBR of 80% determined in South Africa is as a result of the different compaction efforts and test procedures, would be somewhat better quality than the same material tested using the CML method. The effect of the interference with the grading is difficult to quantify or compare.

The correlation between the CBR and DN value developed by Kleyn was based on standard material testing protocols in South Africa at the time. The above discussion shows the possible differences between the SA and CML test methods and indicates that prediction of the CML CBR from the DN value using the Kleyn model is probably rather inaccurate.

It is for this reason that it is better when using the DCP in Africa that the DN value is only considered, and that all the design catalogues are thus related only to the DN values based on past experience and not on converted CBR values.

7.4.4 Materials for the DCP CBR Design Method

The DCP CBR design method is based on research from southern Africa in which a series of design catalogues were developed by Gourley and Greening (1999). This has subsequently been extended to determination of the material properties using the DCP. The material classification used for the DCP CBR design method and discussed more fully in Chapter 13 are summarised in Table 7-10.

It will be noted that there are more groups (codes) in this classification than in the national standard (MOW, 1999) to allow the use of intermediate materials. The specifications also include a grading requirement as given in Table 7-11.

Table 7-10: Pavement material types and abbreviated nominal specifications used in the DCP-CBR design method

Code	Material	Abbreviated Specifications
G80	Natural gravel	Min. BR: 80% @ 98/100% BS heavy compaction and 4 days soaking. Max. Swell: 0.2%. Max. Size and grading: Max size 37.5 mm, grading as specified. PI: Dependent on material type, traffic and subgrade strength or as otherwise specified.
G65	Natural gravel	Min. CBR: 65% @ 98/100% BS heavy compaction and 4 days soaking. Max. Swell: 0.2%. Max. Size and grading: Max size 37.5 mm, grading as specified. PI: Dependent on material type, traffic and subgrade strength or as otherwise specified.
G55	Natural gravel	Min. CBR: 55% @ 98/100% BS heavy compaction and 4 days soaking. Max. Swell: 0.2%. Max. Size and grading: Max size 37.5 mm, grading as specified. PI: Dependent on material type, traffic and subgrade strength or as otherwise specified.
G45	Natural gravel	Min. CBR: 45% @ 98/100% BS heavy compaction and 4 days soaking. Max. Swell: 0.2 %. Max. Size and grading: Max size 37.5 mm, grading as specified. PI: Dependent on material type, traffic and subgrade strength or as otherwise specified.
G30	Natural gravel	Min. CBR: 30% @ 95/97% BS heavy compaction & highest anticipated moisture content. Max. Swell: 1.0% 1.5% @ 100% BS heavy compaction. Max. Size and grading: Max size 63 mm or 2/3 layer thickness . PI: Dependent on material type, traffic and subgrade strength or as otherwise specified.
G25	Natural gravel	Min. CBR: 25% @ 95/97% BS heavy compaction & highest anticipated moisture content. Max. Swell: 1.0% 1.5% @ 100% BS heavy compaction. Max. Size and grading: Max sixe 63 mm or 2/3 layer thickness. PI: Dependent on material type, traffic and subgrade strength or as otherwise specified.
G15	Gravel/soil	Min. CBR: 15% @ 93/95% BS heavy compaction & highest anticipated moisture content. Max. Swell: 1.5% 1.5% @ 100% BS heavy compaction. Max. Size: 2/3 of layer thickness. PI: Dependent on material type, traffic and subgrade strength or as otherwise specified.
G7	Gravel/soil	Min. CBR: 7% @ 93/95% BS heavy compaction & highest anticipated moisture content. Max. Swell: 1.5% @ 100% BS heavy compaction. Max. Size: 2/3 layer thickness. PI: Dependent on material type, traffic and subgrade strength or as otherwise specified.
G3	Gravel/soil	Min. CBR: 3% @ 93/95% BS heavy compaction & highest anticipated moisture content. Max. Swell: N/A. Max. Size: 2/3 layer thickness .

Note: Two alternative minimum levels of compaction are specified. Where the higher densities can be attained in the field (from field measurements on similar materials or other established information) they should be specified by the Engineer.

Source: Modified from Gourley and Greening, 1999.

Table 7-11: Particle size specification for natural gravel road bases

Test Sieve size	Per cent by mass of total aggregate passing test sieve				
	Envelope A Nominal maximum particle size			Envelope B	Envelope C
	37.5 mm	20 mm	10 mm		
50 mm	100			100	
37.5 mm	80-100	100		80-100	
20 mm	55-95	80-100	100	55-100	
10 mm	40-80	55-85	60-100	40-100	
5 mm	30-65	30-65	45-80	30-80	
2.36 mm	20-50	20-50	35-75	20-70	20-100
1.18 mm	-	-	-	-	-
425 µm	8-30	12-30	12-45	8-45	8-80
300 µm	-	-	-	-	-
75 µm	5-20	5-20	5-20	5-20	5-30
Envelope D 1.65 < GM < 2.6					

Source: Gourley and Greening, 1999.

The strength and plasticity specifications vary depending on the traffic level and subgrade class as outlined in Tables 7-12 and 7-13. The soaked CBR test is used to specify the minimum road base material strength.

Table 7-12: Plasticity specifications for natural gravel road base materials

Subgrade Class(1)	Property	Limit of design traffic class				
		0.01 M	0.1 M	0.3 M	0.5 M	1 M
S2	PI	<12	<9	<6	<6	<6
	PM	400	150	120	90	90
	Grading	B	B	A	A	A
S3	PI	<15	<12	<9	<6	<6
	PM	550	250	180	90	90
	Grading	C(2)	B	B	A	A
S4	PI	Note (3)	<12	<12	<9	<9
	PM	800	320	300	200	90
	Grading	D(4)	B	B	B	A
S5	PI	Note (2)	<15	<12	<12	<9
	PM	n/s	400	350	250	150
	Grading	D(4)	B	B	B	A
S6	PI	Note (2)	<15	<15	<12	<9
	PM	n/s	<550	<500	<300	<180
	Grading	D(4)	C(2)	B	B	A

Notes:

(1) S2 to S6 are the subgrade classes defined by their CBR values (@100% BS light compaction as shown in Table 7-12).
(2) Grading 'C' is not permitted in wet environments or climates grading 'B' is the minimum requirement.
(3) Maximum PI = 8 x GM.
(4) Grading 'D' is based on the grading modulus 1.65 < GM < 2.65.

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

PI Plasticity index.
PM Plasticity modulus.
n/s Not specified.

Source: Gourley and Greening, 1999.

Table 7-13: Subgrade class definitions

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

Source: Gourley and Greening, 1999.

A maximum plasticity index of 6% is specified for higher traffic classes and also on weaker subgrades. For designs in dry environments, the plasticity modulus for each traffic and subgrade class can be increased depending on the crown height and whether unsealed or sealed shoulders are used as described in Chapter 13.

The requirements for the selection and use of lateritic gravels for bases, as shown in Table 7-14, are slightly different to those given for other natural gravels, as shown in Table 7-11. A maximum PI of 9% has been specified for some of the higher traffic levels (0.3 – 0.5 MESA) and weak subgrades (S2). For design traffic levels greater than 0.3 MESA, a requirement is set that the liquid limit should be less than 30%. Below this traffic level, this requirement is relaxed to a liquid limit of less than 35%. Where sealed shoulders over one-metre wide are specified in the design, the maximum plasticity modulus may be increased by 40 %. A minimum field compacted dry density of 2.0 Mg/m^3 is required for these materials.

Table 7.14: Specifications for lateritic gravel roadbase materials

Subgrade Class	Property	Limit of design traffic class (MESA)				
		<0.01	**0.1**	**0.3**	**0.5**	**1.0**
S2	PI	<15	<12	<9	<9	<6
	PM	<400	<150	<150	<120	<90
	GE	B	B	A	A	A
S3	PI	<18	<15	<12	<9	<6
	PM	<550	<250	<180	<120	<90
	GE	B	B	B	A	A
S4	PI	<20[1]	<15	<15	<9	<9
	PM	(800	<320	<300	<200	<90
	GE	GM 1.6-2.6	B	B	B	A
S5	PI	<25[1]	<18	<15	<12	<9
	PM	n/s	<400	<350	<250	<150
	GE	GM 1.6-2.6	B	B	B	B
S6	PI	<25[1]	<18	<18	<15	<12
	PM	n/s	<400	<400	<300	<180
	GE	GM 1.6-2.6	B	B	B	A

Notes:
(1) PI maximum = 8 x GM
n/s = not specified
Unsealed shoulders are assumed
PI = plasticity index
PM = plasticity modulus
GE = grading envelope
GM = grading modulus

Source: Gourley and Greening, 1999.

7.4.5 Specific materials

The use of residual basic igneous rock (including basalt and dolerite) gravels could result in significant savings provided the characteristics of the material are good enough to serve as a roadbase material. The following are indicative limits[1]:

- Maximum secondary mineral content of 20 % (determined from petrographic analysis).
- Maximum loss of 12 or 20 per cent after 5 cycles in the sodium or magnesium sulphate soundness tests, respectively (ASTM C88-13).
- Clay index of less than 3 in the methylene blue absorption test (ASTM C837-09).
- Durability Mill index of less than 125 (SATCC, 1994).

In the moderate and dry zones of Tanzania, residual basic igneous rock can be used unmodified up to a maximum plasticity index of 10%. However, it is suggested that the materials should not be used in wet areas unless chemically modified. The risk of using the material can be minimised if consideration is given to:

- The variability of the material deposit, with good selection and control procedures in place for the operation of the pit and on site.
- The provision of good drainage conditions (many materials used for LVRs are particularly sensitive to moisture).
- The adequacy of the pavement design (refer to *Chapter 13 – Structural Design: Paved Roads)*.
- The use of double surface treatments or similar.

With careful selection, cinder gravels can be used for lightly trafficked paved roads in accordance with the requirements of the specific pavement requirements (refer to *Chapter 13 – Structural Design: Paved Roads)*.

If well compacted and kept dry, natural sands can exhibit high load bearing capacities and can be successfully used as basecourse materials. Experience has shown that red sands with an appropriate grading and plasticity on the minus 0.075 mm fraction typically have relatively high CBR values (40 to 70%) and can be used for both subbase and base courses in paved roads. It should be noted that this CBR is measured at the expected in situ moisture conditions and the specimens should be prepared by vibrating hammer to 100% BS heavy MDD (AFCAP, 2013).

7.5 MATERIAL IMPROVEMENT

7.5.1 General

The purpose of blending is to improve the properties of one or more materials. The materials to be blended must complement each other, for example, non-plastic sand with plastic calcrete has proved to be a very successful blend for both gravel roads and as bases of paved roads. Figure 7-9 shows the effect on the CBR of adding various percentages of sand to four different calcretes.

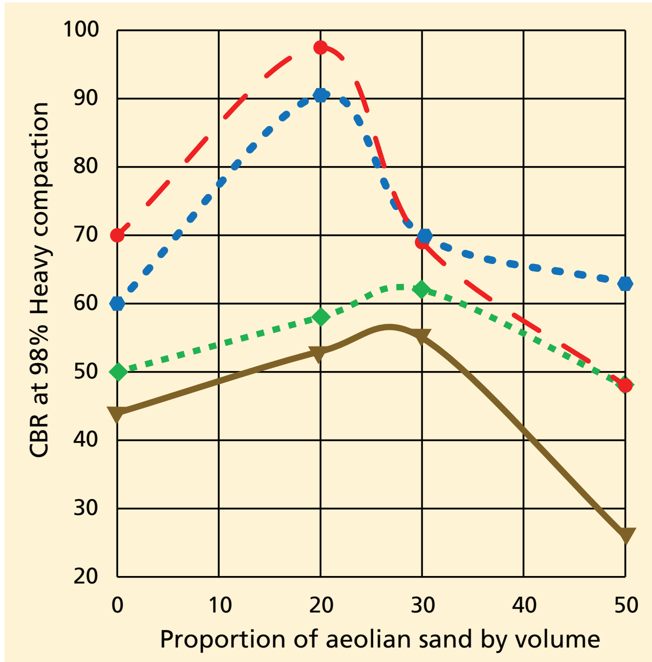

Figure 7-9: Improvement of CBR strength of calcrete gravel with the addition of sand

The properties of the final blended material must meet the requirements of the design catalogue for granular materials for the layer in which it is intended for use. The blend ratio that meets these requirements in the most economical way should be used.

7.5.2 Armouring Weak Roadbases

Armouring is the application of a layer of aggregate on top of a relatively weak base to strengthen the interface between the base and the surfacing as shown in Figure 7-10. It also increases the strength of the base layer but by a small margin. The strengthening of the interface prevents failure caused by the poor bond between a fine material and the surfacing.

Figure 7-10: Crushed stone armouring on laterite

Coarse aggregate is used for this purpose. If the material used to build the base is sandy it is possible to mix the sand and the aggregate only in the upper part of the base to a maximum depth of 50 mm. If, on the other hand, the material is clayey the aggregate shall be applied on top and rolled into the base. This minimises contamination of the interface with plastic materials which are, by their nature, deleterious to surfacing because they cause delamination or debonding of the surfacing from the base.

7.5.3 Stabilised bases and subbases

In certain parts of Tanzania, good gravels for the construction of roads do not exist. In most of these cases, it is not possible to obtain material of acceptable quality even through blending. Chemical stabilization with lime or cement could be considered in these situations but only as a last resort as these operations can prove extremely expensive. If chemical stabilization is considered, the conventional requirements for stabilized materials should be implemented (MOW, 1999).

If cement or lime are found to be ineffective the possibility of using bitumen emulsion or foamed bitumen for stabilisation can be considered. These are generally more costly than conventional chemical stabilisation but can often be more effective.

Numerous proprietary chemical stabilisers are available on the market, and these too can be considered for possible use. They require extensive testing with various application rates and different additives to determine whether they are suitable for any material. Should the required CBR strength be achieved using a chemical, the cost-effectiveness should then be assessed and if any chemical is found to be cost effective, the product can be considered for use.

7.6 CONSTRUCTION MATERIAL REQUIREMENTS

7.6.1 General

The different types of road construction materials required are:

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

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

It should also be noted that the majority of low volume road designs for which this document is relevant will be upgraded from an existing earth or gravel road, which may have been in service for many years. The strength built up in the underlying material must be capitalised on and as little additional structure as possible should be constructed (refer to *Chapter 13 – Structural Design: Paved Roads)*. Other aspects such as shape, drainage and the repair of localised problem areas have also usually been attended to over the years.

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

7.6.2 Common Embankment Fill

In general, location and selection of fill materials for low volume roads poses few problems with materials requiring CBR values at their expected worst in situ moisture and density condition of 3 to 5% (DCP DN values of 33 – 48 mm/blow). Exceptions include organic soils and clays with high liquid limits and plasticity. Problems may also exist in lacustrine and flood plain deposits where very fine materials are abundant.

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

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

7.6.3 Imported (Selected) Subgrade

Where subgrade soils are weak or problematic, import of higher quality selected subgrade material may be necessary. As far as possible the requirement to import material from borrow areas should be avoided due to the additional haulage costs. However, import of stronger (CBR>15% or DN of ≤ 14 mm/blow, at expected worst moisture condition) subgrade materials can provide savings with regard to the pavement thickness design, although the cross section of the pavement must always allow for effective drainage as discussed later in *Chapter 11 – Hydrology and Drainage Structure.* Where material improvement is necessary or unavoidable, mechanical and chemical stabilisation methods can be considered.

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

For very weak subgrades (in situ CBR less than 3% or DN value of > 48 mm/blow, i.e. weaker than S2), it would normally be inappropriate to lay a pavement on such soils. However, for unpaved roads to be upgraded, 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 they could be and for such materials, special treatment is required.

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

7.6.4 Base and subbase

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

A wide range of local materials including lateritic, calcareous and quartzitic gravels, river gravels and other transported gravels, or granular residual materials resulting from weathering of rocks can be used successfully as base. Subbase and base materials are conventionally expected to meet requirements related to maximum particle size, grading, plasticity, and CBR. However, under certain circumstances, mechanical treatments may be required to improve the quality to the required standard. This often requires the use of special equipment and processing plants that are relatively immobile or static. For this reason, the borrow pits for base and subbase materials are usually spaced widely. In current practices, distances of about 50 km between borrow pits are not unusual, but the shorter the haul distances, the better. Main sources of subbase and base materials are rocky hillsides, high steep hills, and river banks.

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

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

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

Subgrade Class S6 covers all subgrade materials having a soaked CBR greater than 30% and which comply with the plasticity requirements for natural subbase. In such cases, no subbase is usually required.

However, it should be noted that the strength of both the subbase and base will depend on the traffic, the environment (in situ moisture conditions predominantly) and the underlying support and unlike the traditional CBR design method, no standard strength for the subbase and base are specified using the DCP-DN design method.

7.6.5 Road Surfacing Aggregate

The general requirements for aggregate to be used in a bituminous surfacing layer are that it must be durable, strong and should also show good adhesion with bituminous binders. It should also be resistant both to the polishing and abrasion action of traffic. The main qualities for surfacing aggregate are summarised in Table 7-15.

Table 7-15: Basic requirements for surfacing aggregate

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

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

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

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

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

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

The specification of aggregates for surface seals should comply with the requirements of the Pavement and Materials Design Manual (MOW,1999) for conventional seals as far as possible. Where more appropriate surfacings such as Otta seals or Sand Seals are considered, the requirements discussed in *Chapter 15 – Surfacing* should be applied.

7.6.6 Block or Paving Stone

Paving stones (or blocks or cobbles) can be produced by cutting or breaking large natural boulders. Each stone should be a strong, homogenous, isotropic rock, free from significant discontinuities such as cavities, joints, faults and bedding planes. Rocks such as fresh granite, basalt and crystalline limestone have proven to be suitable materials. Quartzite rock is not suitable, nor is any rock that polishes or develops a slippery surface, or abrades under traffic *(*refer to *Chapter 15 – Surfacing).*

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

7.6.7 Clay bricks and cement blocks

Burnt clay bricks and concrete blocks are potentially useful surfacing materials. Both of these can provide good riding quality and high skid resistance and are highly labour intensive in their construction. Problems due to poor construction or insufficient support can be easily maintained with only the localised areas showing distress requiring removal and resetting, after correcting the causes of the problems. It is important that the blocks/bricks have adequate strengths and are durable (*Chapter 15 – Surfacing).*

7.6.8 Aggregates for Structural Concrete

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

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

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

7.6.9 Filter/Drainage Aggregate

Filter materials have crucial roles in assisting in the prevention or in controlling the ingress of water and in the reduction of pore water pressures within both the earthworks and the pavement. Filter materials can account for a significant proportion of the construction material costs, particularly in wetter regions where road designs need to cater for the dispersion of large volumes of water, both as external drains and as internal layers within wet-fill embankments. The general requirements for filter material are a highly permeable mix comprising a durable aggregate that is resistant to chemical alteration (Table 7-16). Additional detail is provided in MOW (1999).

Table 7-16: 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 weathering during construction and for the life of the project.
Resistance to Erosion	The as-placed material must be resistant to internal and external erosion.
Chemical Stability	Aggregate should generally be inert and resistant to alteration by groundwater. Weak surface coatings such as clay, iron oxide, calcium carbonate, gypsum etc. are undesirable.
Grading	D_{15} for filter material/d_{15} for adjacent subsoil ≥ 5 with minimum of 50% retained on 2 mm sieve(2).

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

7.6.10 Special Materials

Natural

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 costly. Such materials need to be hard and durable with property requirements as shown in Table 7-17.

Table 7-17: 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. Weak surface coatings such as clay, iron oxide, calcium carbonate, gypsum etc. are undesirable.

Commercial Products

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

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

7.7 MATERIAL LOCATION

7.7.1 General

The location of construction materials is a science that is acquired with time. In order to minimise the haulage of materials over long distances sources of borrow materials should be as close to the road as possible.

Sources of road-building materials thus have to be identified within an economic haulage distance and they must be available in sufficient quantity and of sufficient quality for the purposes intended. Previous experience in the area may assist with material location but additional survey is usually essential. Two of the most common reasons for construction costs to escalate once construction has started and material sources fully explored, are that the materials are found to be deficient in quality or quantity. This leads to expensive delays whilst 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 local materials are of marginal quality or occur only in small quantities.

As discussed in Chapters 5 and 6, the site and/or geotechnical investigation must identify and prove that there are adequate and economically viable reserves of natural construction materials.

If the project is in an area where good quality construction materials are scarce or unavailable, alternate solutions that make use of the local materials should be considered to avoid long and expensive haulage.

For example, consideration should be given to:

- Modifying the design requirements.
- Modifying the material (e.g. mechanical or chemical stabilization).
- Material processing (e.g. crushing, screening, blending).
- Innovative use of non-standard materials (particularly important for low traffic roads).

The cost of haulage over long distance should be compared with that of treating the local materials and the most cost-effective option chosen.

The materials investigations should also take into account any future needs of the road. This is particularly important in the case of gravel roads where re-gravelling is normally needed 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, good quality material may be required at a later stage in the road's life when the standard needs to be improved to meet increased traffic demands.

The design engineer will need to ascertain the availability of sufficient suitable materials in the vicinity of the road alignment. A comprehensive list of the location and 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.
- Located such that their exploitation will not lead to any complicated or lengthy legal problems and will not unduly affect the local inhabitants or adversely affect the environment.

7.7.2 Material exploration

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

- Determination of the nature of the deposit, including its geology, history of previous excavation and possible mineral rights.
- Determination of the depth, thickness, extent and composition of the strata of soil and rock that are to be excavated.
- Analysis of the condition of groundwater, including the position of the water table, its variations, and possible flow of surface water into the excavation ground.
- Assessment of the property of soils and rocks for the purposes intended.

The process is summarised in Figure 7-11 and additional information is provided in the Field Testing Manual (MOW, 2003).

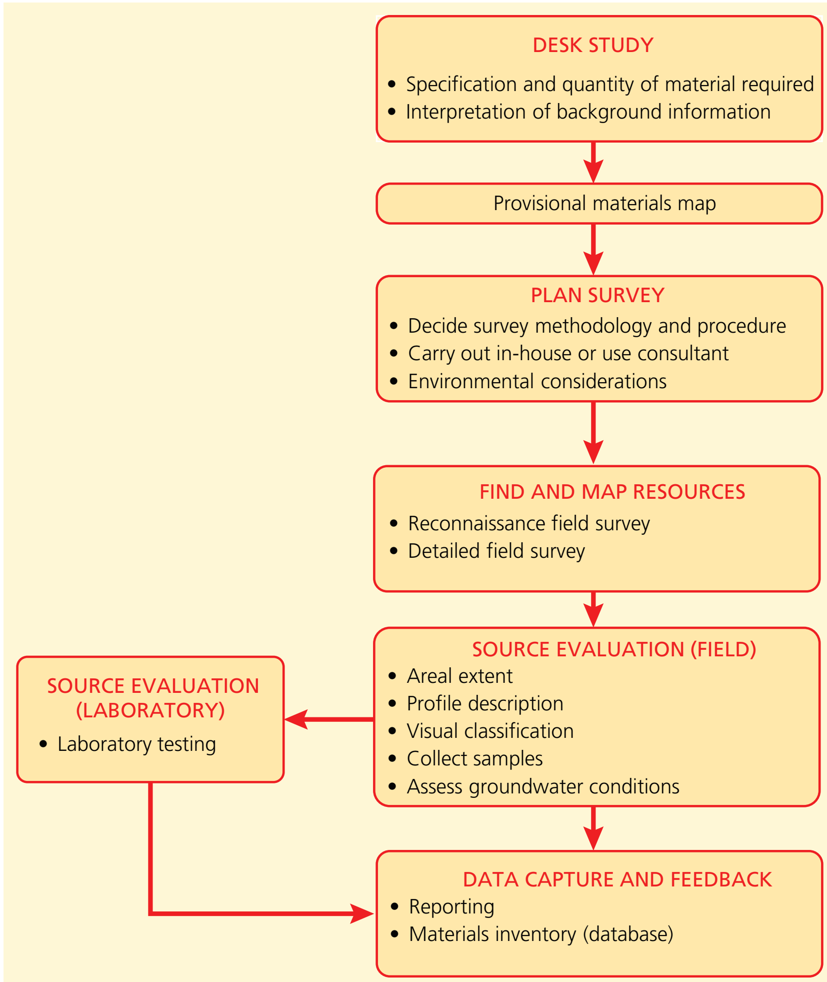

Figure 7-11: Flow diagram of prospecting procedure

Records of roads already built can be a valuable source of data, not only on the location of construction materials but also on their excavation, processing, placement and subsequent performance. Potential problems with materials can also be identified. Construction records are kept by different departments within the Tanzanian road hierarchy, regional road authorities, or by road design consultants and construction supervising organisations and contractors. These and any other materials-related reports should be consulted to assist with material location.

The recovery of a suitable material is, therefore, a matter of understanding the geological history and weathering profile at the quarry site.

Prospecting and sampling: The earlier pre-feasibility and feasibility studies will have likely used desk studies (topography, geology, soils, hydrology, vegetation, land-use and climate in the area), field survey and possibly also laboratory testing programmes to make preliminary identification and location of potential construction materials. This information will guide the verification process undertaken by the design engineer in preparation of the detailed design.

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

The geomorphology is also a strong indicator of potential materials. Specific features such as pans, depressions, ridges or trenches can indicate material differences. Flat lying areas tend to have deeper weathering profiles (or transported soils) than more steeply inclined areas.

It can also be useful to make use of termite hills and animal burrows to investigate subsoil conditions and possibly isolate suitable prospecting areas.

Site investigation activities will include detailed prospecting for materials through surface mapping, test pitting, boreholes and material sampling. A variety of sub-surface sampling and investigation procedures appropriate for different materials is used to recover the samples needed for laboratory testing. It is important that adequate representative samples of each material are obtained for testing. Figure 7-12 indicates the required quantity of material for routine testing as a function of the maximum particle size.

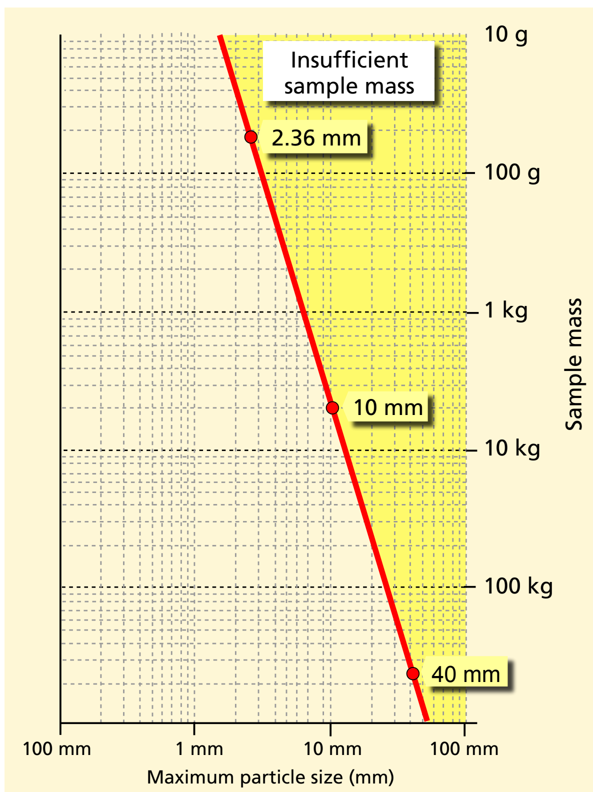

Figure 7-12: Guide to estimating mass of sample required for routine testing based on maximum particle size

Laboratory Testing

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

The laboratory testing programme should be part of a rational programme designed by the engineer to give all of the information needed to adequately define the nature, use and volumes available of construction materials. Typically, the material grading, Atterberg limits, compaction characteristics and strength are determined as the primary laboratory tests. It is important that the CML methods (based on the BS 1377 standards) are followed. Only if no CML or BS method for a specific test exists should an

ASTM or AASHTO method be followed. There are often significant differences in results obtained by the different test methods. The biggest differences in the test results are obtained for the Atterberg limits and the compaction characteristics/strength tests – other test methods have relatively insignificant differences in the results.

Maximum use should be made of data and information compiled during earlier parts of the project design. The construction materials used for low volume roads and the design philosophies that are adopted in this manual, mean that it is important that the relationships between expected/in situ conditions and laboratory conditions are considered when designing and developing the test regime.

Early phases of the laboratory test programme will generally concentrate on gaining clues to unusual soil behaviour, e.g. swelling or collapse potential. Bearing in mind the difficulties of sample recovery, statistical sample sizes and the cost of laboratory testing, most testing programmes will be based around relatively simple classification tests that can be done quite quickly. More sophisticated tests will only be used if absolutely necessary.

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

7.8 MATERIAL PROCESSING

7.8.1 General

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

In order to achieve suitable wearing course properties a suitable particle size distribution (PSD) can be obtained by breaking down oversized material to a maximum size of 50 mm or smaller. Atterberg limits may be modified by granular/mechanical stabilisation (blending) with other materials. These material processing/ improvement measures are discussed briefly below:

7.8.2 Reducing Oversize

There are various measures for reducing oversize including the use of labour, mobile crushers, grid rollers or rock crushers. The choice of method will depend on the type of project and material to be broken down:

- **Hand labour:** This is quite feasible, especially on relatively small, labour-based projects where material can either be hand screened and/or broken down to various sizes and stockpiled in advance of construction.
- **Mobile crushers:** The crushing of borrow pit materials may be achieved with a single stage crushing unit or, in the other extreme, multi-stage crushing and screening plant.
- **Grid rollers:** These are manufactured as a heavy mesh drum designed to produce a high contact pressure and then to allow the smaller particles resulting from the breakdown to fall clear of the contact zone, as shown in Figure 7-13. It is, however, essential that the material is re-mixed after breaking down with a grid roller to separate broken particles before compaction commences.

- **Rock crusher:** The “Rockbuster” is a patented plant item which is basically a tractor-towed hammer mill, as shown in Figure 7-14. The hammer mill action of the Rockbuster will act on the material that it passes over, breaking down both large and small sizes. There is the potential to over-crush” a material and create too many fines in the product. It may be necessary to windrow out only the larger particles in a material and process these with the Rockbuster, with the crushed material then blended back into the original product.

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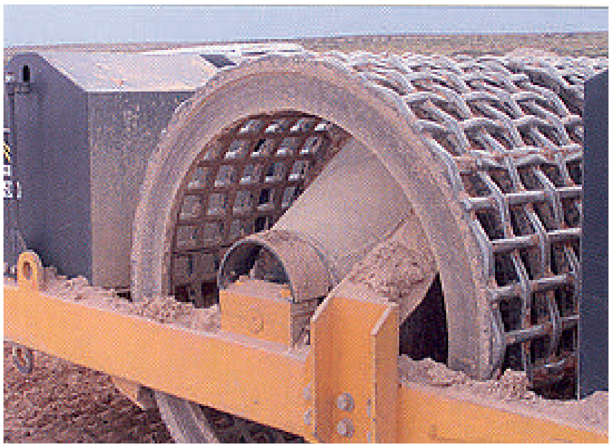

Figure 7-13: Grid roller

Figure 7-14: Rockbuster

7.8.3 Material blending

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

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

The following methodology, using a ternary diagram, is shown in Figure 7-15, has been developed for determining the optimal mix ratio for blending two or more materials to meet the required grading specification for unpaved roads (the optimum grading is shown by the shaded area) but could be applied to improve the grading of any material. The points A and B in the figure shown are an example using two typical soils, the gradings of which are summarised in Table 7-18.

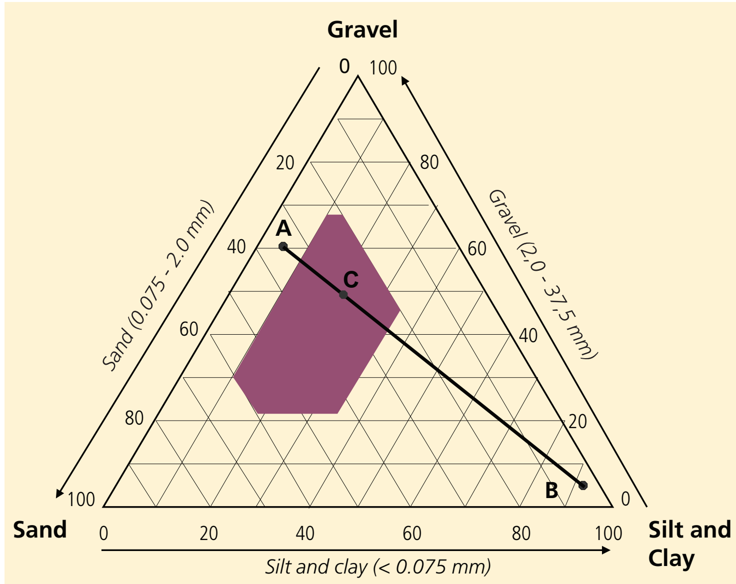

Figure 7-15: Example of use of ternary diagram for determining proportions during material blending

Table 7-18: Gradings of materials used for blending in Figure 7-15

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

1. Identify potential material sources that can be used to improve the available material.
2. Determine the particle size distribution of the available material and that considered for addition or blending (wet sieve analysis recalculated with 100 per cent passing the 37.5 mm sieve).
3. Determine the percentages of silt and clay (<0.075 mm), sand (0.075 - 2.0 mm) and gravel (2.0 -37.5 mm) for each source.
4. Plot the material properties on the ternary diagram as points A and B respectively (see example in Figure 7-15).
5. Connect the points. When the two points are connected, any point on the portion of the line in the shaded area indicates a feasible mixture of the two materials. The optimum mixture should be at point C in the centre of the shaded area.
6. The mix proportions are then the ratio of the line AC:BC (in this case 9.5 to 37 or 3.9, i.e. 4 loads of material A will be blended with 1 load of material B). This can be equated to truck loads and dump spacing.

7. Once the mix proportions have been established, the Atterberg Limits of the mixture should be determined to check that the shrinkage product is within the desirable range (140 – 400 (or 260 if necessary). The quantity of binder added should be adjusted until the required shrinkage product is obtained, but ensuring that the mix quantities remain within the acceptable zone. If the line does not intersect the shaded area at any point, the two materials cannot be successfully blended and alternative sources will have to be located, or a third source used for blending.

7.8.4 Hard Stone and aggregate

The sourcing of hard stone for concrete aggregate, bituminous road surfacing aggregate, masonry and cobble stone is usually done from quarries. These can be available commercial quarries or quarries set up for the project. Many portable crushers are now available that make the setting up of a temporary crushing plant cost effective and quick. A crushing and screening combination tailored to the material properties and aggregate requirements needs to be installed.

Blasting of the material is usually necessary to produce feed-stock for the crusher. Outcrops of suitable material are normally easily visible as cliffs or hills but their properties still need to be approved for use. In flat terrain, it is necessary to drill to locate underlying rock, where relatively fresh rock will nearly always be encountered at some depth as there is a gradual transition from one weathering state to the other. This type of quarry is, however, expensive to develop, as there is often significant overburden and all material has to be hauled uphill.

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8.1 INTRODUCTION

8.1.1 Background

Reliable data on traffic volumes and characteristics are essential for both geometric and pavement structural design and also assist in the planning of road safety measures in the manner summarised 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 frequency of individual axle loads.
- ***Road safety:*** The volume, type and characteristics of the traffic using the road all influence the type of road safety measures required to ensure a safe road environment.

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

8.1.2 Purpose and Scope

The purpose of this chapter is to outline the procedures to be followed in determining the traffic loading over the design life of the road as a basis for designing the road pavement, choosing an appropriate road cross section and planning road safety measures.

Normally, the geometric design of a road is very dependent on traffic flow, but for a LVRs where the geometric design consists of relatively minor improvements of an existing road, the traffic flow details do not require high accuracy. The main issue is to determine whether the traffic limit for a LVR is likely to be exceeded and the road then designed to one of the higher road classes where a full geometric design is required.

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

8.2 SURVEYS

8.2.1 General

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

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

8.2.2 Traffic Surveys

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

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

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

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

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

Usually motorised traffic volumes will fall in the wet season to, typically, 80% of their dry season level. However, on poor quality roads this difference is even more marked and the wet season traffic can fall to 35% of dry season traffic levels as shown in Figure 8.1. For the purposes of this manual it can be assumed that roads have traffickability problems when wet season traffic levels fall below about 60% of dry season levels. It is also possible that dry season traffic may be lower than wet season traffic, e.g in coastal areas where sands tend to become loose and less traversable in the absence of ground moisture.

Thus, the timing, frequency and duration of traffic surveys should be given very careful consideration in terms of striking a balance between cost and accuracy. As indicated in Figure 8-2, short duration traffic counts in low traffic situations can lead to large errors in traffic estimation.

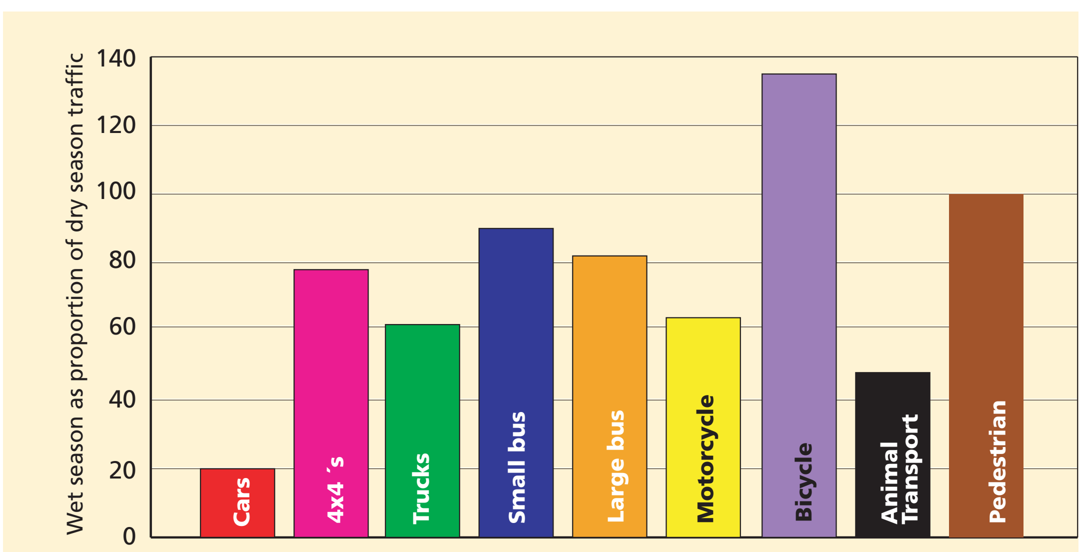

Figure 8-1: Difference in wet season and dry season traffic levels on poor quality roads

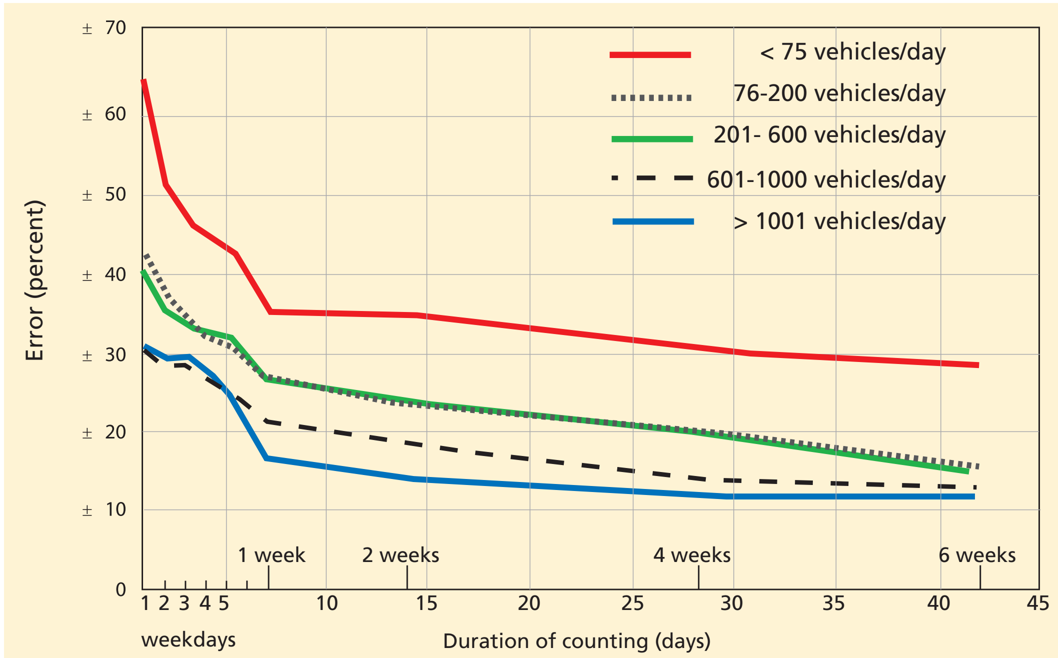

Figure 8-2: Possible errors in ADT estimates from random counts of varying duration
Source: Howe (1972)

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

- Counting for seven consecutive days.
- On some days counting for a full 24 hours, preferably with one 24-hour count on a weekday and one during a weekend; on other days, 16 hour counts (typically 06.00 – 22.00 hours) should be made and expanded to 24-hour counts using a previously established 16:24 hour expansion ratio.
- Avoiding counting at times when road travel activity increases abnormally; for example, just after the payment of wages and salaries, or at harvest time, public holidays or any other occasion when traffic is abnormally high or low. However, if the harvest season is during the wet season (often the case, for instance, in the timber industry), it is important to obtain an estimate of the additional traffic typically carried by the road during these periods.
- Repeating the seven-day counts several times throughout the year.

Care should be exercised in selecting appropriate locations for conducting the traffic counts to ensure a true reflection of the traffic using the road and to avoid under- or over-counting. Thus, locations such as within villages or market places should be avoided

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

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

Adjustments for season

An appropriate, weighted average adjustment will need to be made according to the season in which the traffic count was undertaken and the length of the wet and dry seasons, as illustrated in Figure 8-3.

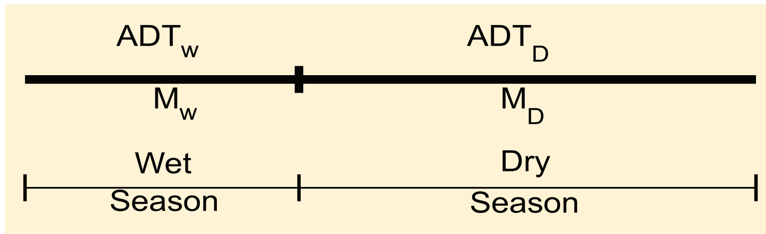

Figure 8-3: Basis for traffic count adjustment in relation to seasonal characteristics

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

$$\text{Weighted average ADT} = \frac{(ADT_W \times M_W)}{12} + \frac{(ADT_D \times M_D)}{12}$$

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

Vehicle Classification

Table 8-1 shows the vehicle classification system used for compiling the results of the traffic survey described above.

Table 8-1: Vehicle classification system

Class	Type	Axles	Description	Use
A	Car	2	Passenger cars and taxis.	Capacity analysis for geometric design.
B	Pick-up/4-wheel drive	2	Pick-up, minibus, Land Rovers, Land Cruisers.	
C	Small bus	2	≤ 25 seats	
D	Large bus/coach	2	> 25 seats	Capacity and axle load analysis for pavement design.
E	Light Goods Vehicle	2	≤ 3.0 tonnes empty weight	
F	Medium Goods Vehicle (MGV)	2	> 3.0 tonnes empty weight	
G	Heavy Goods Vehicle (HGV)	3/4	> 3.0 tonnes empty weight	
H	Very Heavy Goods Vehicle VHGV)	≥4	> 3.0 tonnes empty weight	
I	2-axled trailer	2	Trailers towed by MGVs HGVs or VHGVs.	
J	3-axled trailer	3		
K	4-axled trailer	4		
L	Tractor	2		
M	Motorcycles, motor cycle taxis			Capacity analysis for geometric design.
N	Bicycles			
O	Other NMT			
P	Pedestrians			

8.2.3 Origin-Destination Surveys

Origin-Destination (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 be carried out as part of a regional planning exercise rather than for an individual road project. These surveys, which can be quite labour-intensive, serve a number of useful purposes including a quantitative assessment of the amount of traffic likely to be affected by the proposal and the consequent impacts on various elements in the road system.

8.2.4 Axle Load Surveys

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

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.
- 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 control performed. In general, the higher the quality of the data, the greater will be the resources required to collect it. Details for carrying out axle load surveys can be found in the Field Testing Manual (MOW, 2003).

There is an almost inevitable 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. Thus, the choice of analysis procedures may also involve trade-offs.

Ultimately, an appropriate choice of equipment should be made in relation to such factors as:

- Accessibility to back-up support (technical and maintenance).
- Ease of installation and use.
- Accuracy of measurement required.
- Acquisition and operational cost of equipment.

It is also important that axle load surveys are carried out in a systematic and standardised manner. Axle load information can also be obtained from weighbridge data but it is important to note that if vehicle operators believe that the purpose of weighing is to enforce the legal limits, they very quickly reduce the loads on their vehicles whilst the weighing exercise is going on but quickly increase them again afterwards, hence very inaccurate axle load information is obtained.

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.
- 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 that are passing in one direction at a time, both empty and loaded.

Methods of acquiring axle load data

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

Assuming that an axle load survey is not being carried out, information about the vehicle loading can be obtained by observation during the traffic counting survey. The enumerator merely records, for every heavy vehicle in the heavy vehicle classes, the state of loading (full, partial or empty) and the type of load (heavy, medium, or light). Based on data recorded during axle load surveys in Tanzania the number of ESAs per vehicle can be estimated based on Table 8-2 and using guidance on types of load in Table 8-3. Table 8-2 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.

Table 8-2: Average equivalency factors for different vehicle types

Class	Type	Axles	Average ESA per vehicle - all loaded	Average ESA per vehicle - half of the vehicles loaded[1]
A	Car	2	-	-
B	Pick-up/4-wheel drive	2	-	-
C	Small bus	2	0.3	0.15
D	Large bus/coach	2	2.4	1.2
E	Light Goods Vehicle	2	1.5	0.75
F	Medium Goods Vehicle (MGV)	2	4	2
G	Heavy Goods Vehicle (HGV)	3	4.5	2.25
H	Very Heavy Goods Vehicle (VHGV)	≥4	7	3.5
I	2-axled trailer	2	8	4
J	3-axled trailer	3	10	5
K	4-axled trailer	4	12	6

Note 1 It is common that vehicles will have no back-load hence half the vehicles are likely to be empty, or nearly so.

Table 8-3: Approximate ESA values to be used only when no data are available

Dense goods for which the average ESAs in Table 8.2 apply	Light goods for which the load bearing axles can be assumed to contribute only 0.5 x the ESA values in Table 8.2
Quarry products and ore	Household products excluding white goods
Sheet or rod metal	NA
Bulk liquids	NA
Logging	NA
Bulk agricultural products	NA
Machinery	NA

8.3 PROCEDURE FOR DETERMINING DESIGN TRAFFIC

8.3.1 General

The procedure for determining the traffic loading for pavement design purposes is summarized in Figure 8-4.

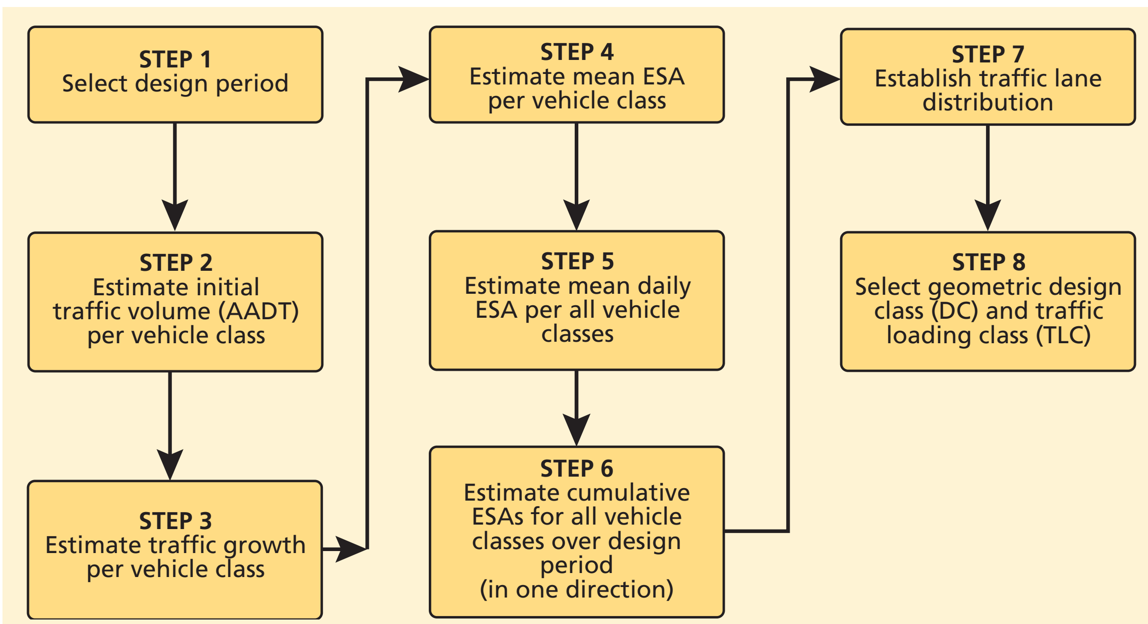

Figure 8-4: Procedure for establishing design traffic class

8.3.2 Select Design Period – Step1

A structural design period must be selected over which the cumulative axle loading is determined as the basis of designing the road pavement. The design period is defined as the time span in years considered appropriate for the road pavement to function before reaching a terminal value of serviceability after which major rehabilitation or reconstruction would be required. Such a level of terminal serviceability is defined in terms of surface condition as approximately 12 IRI (International Roughness Index) which is equivalent to a roughness of about 10,000 mm/km as measured with a calibrated Bump Integrator or as a Present Serviceability Rating of about 0.5. At such a level of deterioration drivers would rarely exceed a speed of 50 km/hr. The designs described in Chapters 13 and 14 are based on an acceptable level of service provided that adequate maintenance, both routine and periodic, is carried out in order to meet the design life.

Various factors that influence the choice of design period include:

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

Based on the above factors, Table 8-4 provides guidance on the selection of the structural design life. Choosing a relatively short design life reduces the problem of long-term traffic forecasting whilst choosing a relatively long design life requires greater care in estimating the design traffic loading if over/under-design of the pavement, and the related cost implications, are to be avoided.

Table 8-4: Structural design period

Importance/level of service	
Low	**High**
5 - 10 years	10 – 15 years

8.3.3 Estimate Initial Traffic Volume per Vehicle Class – Step 2

Based on the traffic surveys described in Section 8-2, the initial traffic volume for each vehicle class can be determined. For structural design purposes, it is only the commercial vehicles in classes D to K inclusive (refer to Table 8-2) that will make any significant contribution to the total number of equivalent standard axles. In contrast, for geometric design purposes it is necessary to count non-motorised and intermediate means of transport including pedestrians, bicycles, motorcycles, tractors and trailers and, possibly, animal transport.

8.3.4 Traffic Growth per Vehicle Class – Step 3

Following the establishment of the baseline traffic, further analysis is required to establish the total design traffic based on forecast of traffic growth in each vehicle class. To forecast such growth, it is first necessary to sort traffic in terms of the following categories (refer to Figure 8-5):

- **Normal traffic** - Traffic that would pass along the existing road in the absence of any upgrading to a higher standard.
- **Diverted traffic** – Traffic that changes from another route to the project road, but still travels between the same origin and destination points. Unless origin-destination surveys have been carried out this can only be estimated based on judgement of the traffic on nearby roads that could benefit from a shorter or more comfortable route.
- **Generated traffic** – Additional traffic that occurs in response to the new or improved road. This traffic is essentially ‘suppressed’ traffic that does not currently exist because of the poor state of the existing road. Local historic precedent can sometimes assist in estimating this, otherwise a rule of thumb is that generated traffic is typically 20% of the existing traffic but it can be considerably higher.

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

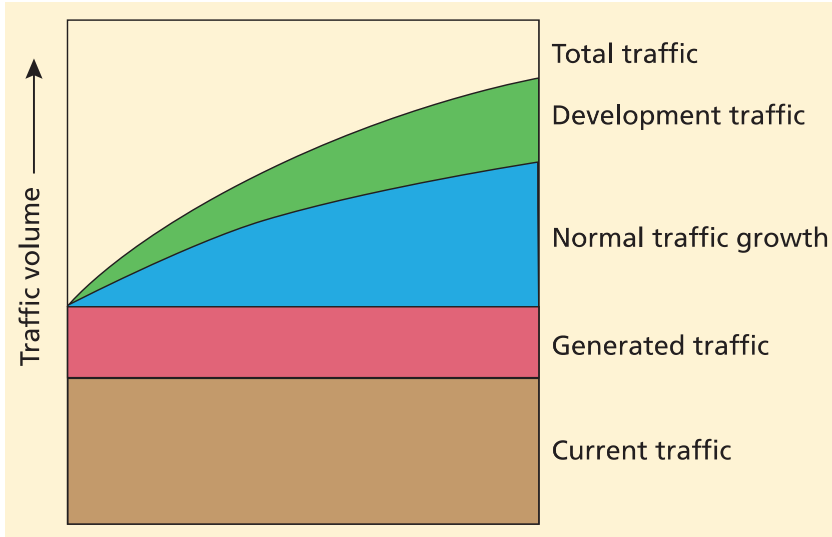

Figure 8-5: Traffic development on an improved road

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

The growth rate of each traffic class may differ considerably. Motor cycles and motor cycle driven vehicle modes, for example, are usually growing at a much faster rate than for other classes. This should be taken into account.

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 is a good guide as to what to expect.

Government predictions of economic growth

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

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

The AADT in both directions in the first year of analysis consists of the current traffic plus an estimate of the generated and diverted traffic. Thus, if the total traffic is denoted by $AADT_0$ 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:

$$AADT_x = AADT_0\ (1+r/100)^x$$

This is illustrated in Figure 8-6 which shows the multiplier for the AADT in the first year of analysis to obtain the AADT in any other year.

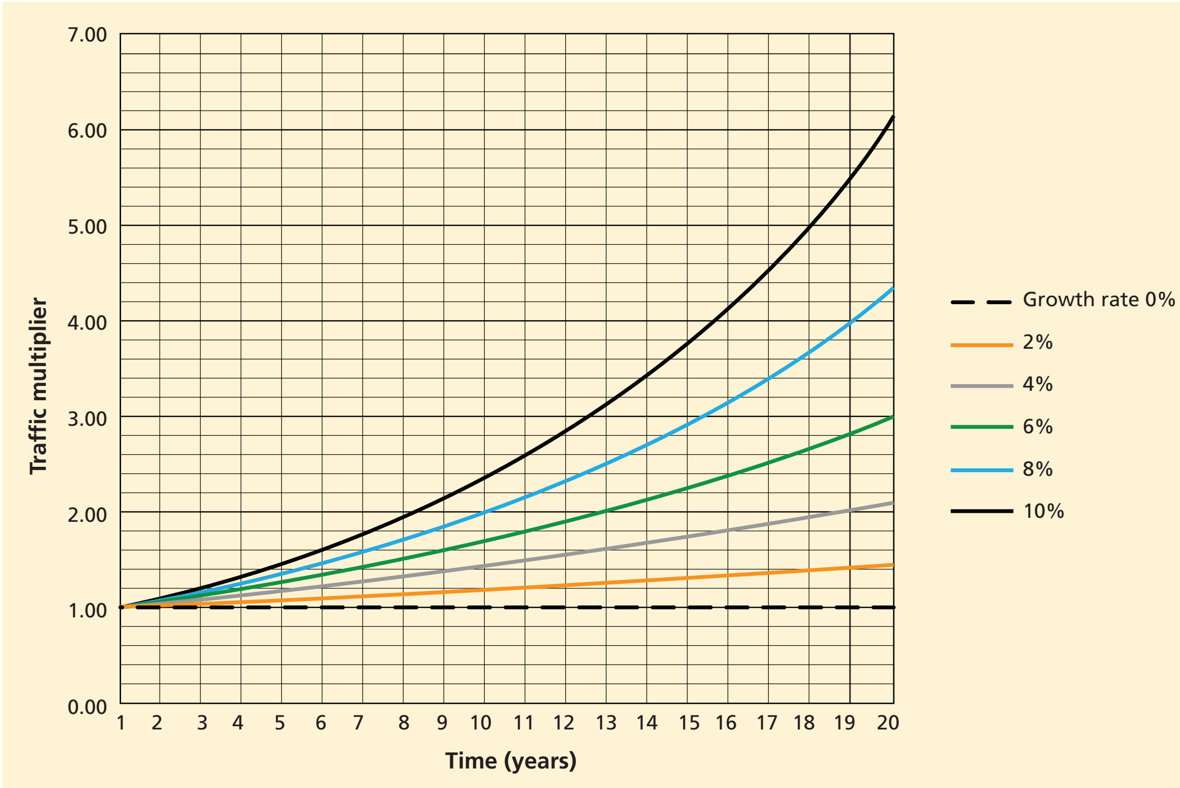

Figure 8-6: Multiplier to obtain AADT in any year for different growth rates

8.3.5 Mean ESA per Vehicle Class –Step 4

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

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

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

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

Where: P = axle load (in kg, tonnes or kN)
n = power exponent (lies between 2.5 and 4.5. A value of 4 is recommended for LVRs)

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

Guidance on the likely average VEF for different vehicle classes derived from historical data in Tanzania is given in Table 8-3. However, data from any recent axle load survey on the road in question or a similar road in the vicinity is better than using countrywide averages.

8.3.6 Mean Daily ESA for all Vehicle Classes – Step 5

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

DESA = AADT x VEF

8.3.7 Cumulative ESA (CESA) for all Vehicle Classes over the Design Period – Step 6

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

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

Where: CESA = mean daily ESAs for each vehicle class in the first year (each direction) (From Step 5).
r = assumed annual growth rate expressed as a decimal fraction. (Different traffic categories may have different growth rates).
N = design period in years (from Step 1).

Figure 8-7 shows the multiplier for the CESA in the first year to calculate the CESA after any other number of years up to year 20.

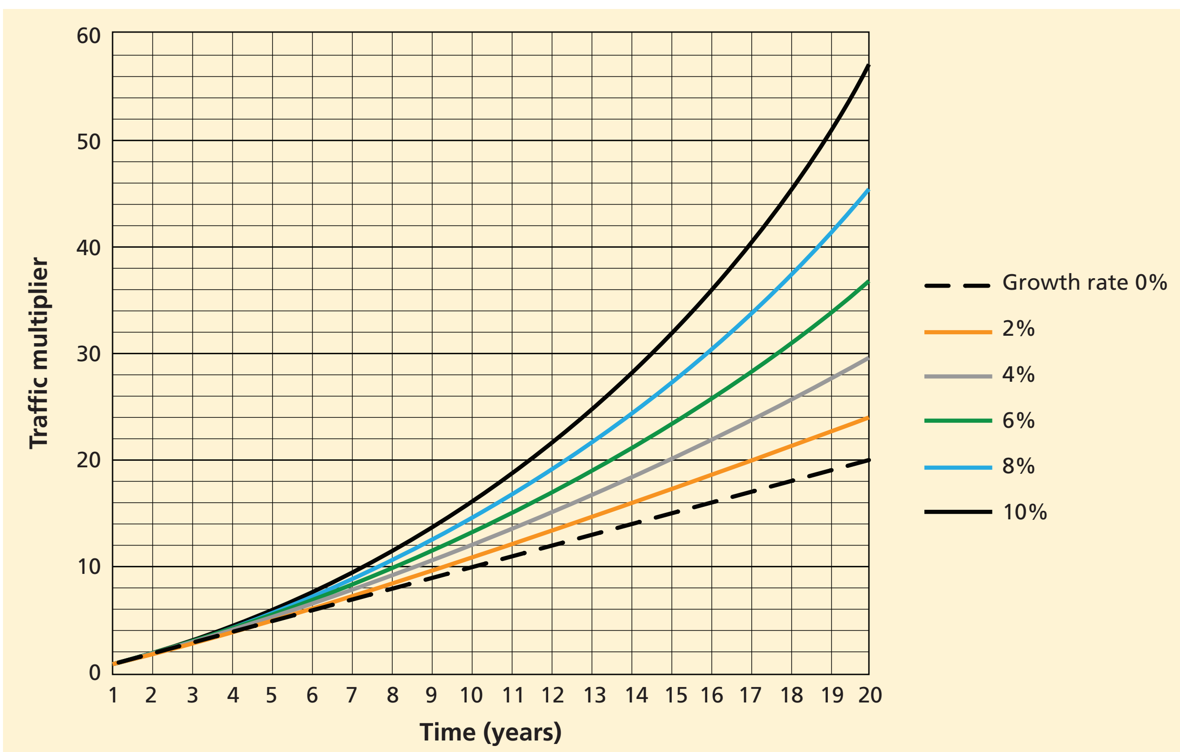

Figure 8-7: Multiplier for the first year CESA to calculate the CESA after any number of years

For geometric design: Four different basic geometric standards (DC5-DC8) are defined for LVRs based on the number of 4-wheeled (and more) vehicles defined in Table 8-2. The traffic level is the sum for both directions and is estimated at the middle of the design life period. When the expected traffic is within 10% of a traffic class boundary, the higher classification should be adopted.

8.3.8 Traffic Lane Distribution – Step 7

The actual design traffic loading (ESAs) needs to be corrected for the distribution of heavy vehicles between the lanes in accordance with Table 8-5.

Table 8-5: Lane width adjustment factors for design traffic loading

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

The traffic classes for structural design are shown Table 8-6.

Table 8-6: Traffic classes for structural design

Traffic Load Class	Cumulative traffic load during design life (MESA)
TLC 1.0	0.5 – 1.0
TLC 0.5	0.3 – 0.5
TLC 0.3	0.1 – 0.3
TLC 0.1	0.01 – 0.1
TLC 0.01	< 0.01

8.4 DESIGN EXAMPLE

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

A. Design inputs

1. Design life = 15 years.
2. Road width = 5 m (surfaced shoulder breakpoint to shoulder breakpoint).
3. A 7-day traffic count summary (AADT of commercial vehicles in both directions) is as follows:

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

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

5 Vehicle equivalence factors. Figures used are for illustration purposes only.

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

6. Power exponent = 4

B: Design calculations

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

-	Large bus	1 x 2.4	=	2.4
-	Small bus	3 x 0.3	=	0.9
-	LGV	5.5 x 1.5	=	8.25
-	MGV	1 x 4.0	=	4.0
-	HGV		=	0
	Total ESA/day		=	15.55 (direction 1)

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

-	Large bus	1 x 2.4	=	2.4
-	Small bus	3 x 0.15	=	0.45
-	LGV	5.5 x 0.75	=	4.12
-	MGV	1 x 2.0	=	2.0
-	HGV		=	0
	Total ESA/day		=	8.97 (direction 2)

3 Daily traffic loading for design for a 5 m wide road

From Table 8-5 the traffic loading for design for a 5 m carriageway is 80% of the ESAs in both directions.

Traffic loading =0.8 x (15.55 + 8.97) = 19.62

(N.B: For roads 6 m wide and wider, the total ESAs in the heaviest loaded direction is required).

4. Estimation of Cumulative ESAs (CESA) per all vehicle classes over design life

The design CESA can be computed from the following equation:

CESA = 365 * DESA* [(1 + r)N – 1]/r

= 365 x 19.6 x [(1 + 0.045)15 – 1]/0.045

= 365 x 19.6 x [(1.045)15 – 1]/0.045

= 365 x 19.6 x [1.935- 1]/0.045

= 365 x 19.6 x 20.78

= 148,660 ESA

= 0.15 million equivalent standard axles (MESA)

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Part D: Design

Low Volume Roads Manual

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- **PART B: Planning**
  - 4. Rural Accessibility Planning
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CONTENTS

9.1 INTRODUCTION

9.1.1 Background

Geometric design is the process whereby the layout of the road through the terrain is designed to meet the needs of all the road users. However, the needs of road users where the roads provide a relatively low speed access function are quite different from road users on more highly trafficked trunk roads which provide a relatively high speed mobility function. Accordingly, the standards and levels of service on access roads are generally lower than on mobility roads. Such standards are intended to meet two important objectives, namely to provide a minimum level of service as well as an acceptable level of safety and comfort for all road users.

The geometric design process is a complex task and goes beyond merely applying values extracted from a table of standards. The unthinking application of charts, tables and figures is unlikely to lead to an optimal design outcome. Good design requires creative input based on a sound understanding of the principles involved.

The major challenge is to rehabilitate and upgrade to acceptable standards the existing rural access road network comprising many thousands of kilometres of unpaved roads and which, in large part, are in very poor condition and impassable during the rainy season. Thus, the over-riding objective of the LGAs is to provide reliable, all-weather access to communities at least cost (in terms of total life-cycle costs) for the locally prevailing means of transport that includes a high proportion of non-motorized traffic. Achievement of this objective is not based on constructing entirely new roads on new alignments but, rather, on improving the existing roads, as far as possible on the existing alignments.

There is now increasing realisation that it would be inappropriate to design the lowest design classes of roads on the basis of traditional geometric standards. Instead, a wider approach needs to be taken in which the over-riding criterion of acceptability is the achievement of an appropriate level of all-year access at “least cost” while at the same time ensuring the LVRs are “fit for purpose” in terms of user requirements and road safety. Given scarce resources, this approach will enable Tanzania to improve the LVR rural access network for as many people as possible, and as rapidly as possible. By so doing, this will foster faster economic and social development which is linked to provision of adequate rural accessibility.

National standards for geometric design are presented in the Tanzania Road Geometric Design Manual (MOW, 2011). However, these standards follow traditional, international design principles and standards which do not cater adequately for the specific requirements of the LVRs as described above and, if applied, are likely to lead to high-cost designs. Thus, a fresh look at the design philosophy and strategy for LVR provision is required as described in this chapter.

9.1.2 Purpose and Scope

The main purpose of this chapter is to provide an appropriate approach for the design of LVRs in a manner that is context sensitive and that emphasises the economic aspects of geometric design whilst taking due account of the road safety aspects. Flexibility in application of the guidance given in this chapter is encouraged so that independent designs tailored to particular situations can be developed.

The chapter firstly presents the general approach to the design of LVRs in a manner that is appropriate to the local physical, social and financial environment. The classification system used for LVRs is then presented, followed by consideration of the many factors that affect the geometric design process. Finally, various design controls and cross section details are presented.

9.2 FUNDAMENTAL DESIGN PRINCIPLES

9.2.1 General

The design approach adopted in this manual is holistic in that, in addition to the traditional aspects of geometric design, it considers other aspects that are unique to the design of LVRs in a constrained budgetary situation.

9.2.2 Design Principles

A geometric standard represents a service level that is deemed appropriate for the particular road environment. Typically, this service level increases with traffic and is relatively high for major, highly trafficked roads and has a clear connection with transport efficiency and economic benefits. For LVRs the benefits of a high service level are less tangible in economic terms and, as a result, a compromise has to be reached between service level and costs. Thus, in order to produce an economic standard, a balance needs to be struck between the cost of improving the alignment, both horizontally and vertically, and the benefits to be derived from so doing – an approach that emphasizes the economic aspects of geometric design which needs to be applied with appropriate understanding of economics and flexibility.

9.2.3 Design Options

There are two main options for the geometric design of LVRs as follows:

- **Option A – Alignment engineered for fulfilling an access function**
  This option adopts most of the existing alignment except in problem areas where safety may be an issue. The use of this option means that “the existing alignment fixes the travel speed”. It will result in variable cross section widths (because the width of most of the existing road need not be changed) and travel speeds but will not incur significant earthworks costs. This option is appropriate in situations where:
  - The road is unlikely to change its function over its design life.
  - The road is likely to be used mostly by local people and seldom by other users who are not familiar with the characteristics of the alignment.
  - Problem areas such as very steep curves or grades or other potentially hazardous black spots are addressed by sound engineering solutions.

In many cases, based on the least cost criterion discussed above, Option A is the most economic standard in that it will result in an alignment that is “fit for purpose” and provide an appropriate level of access at minimum costs. Thus this option is recommended for the geometric design of many LVRs in Tanzania. However, the adoption of this option will require some good engineering judgement to be exercised by a design engineer with experience with LVR design.

- **Option B – Alignment engineered for fulfilling a mobility function**
  A fully engineered alignment is one in which the design speed determines the alignment. For each road class this option uses a consistent cross-section width throughout and a fixed design speed that determines many of the geometric requirements such as passing and stopping sight distances, engineered curvature, both horizontally and vertically, etc. These are the design principles and specifications contained in the Road Geometric Design Manual (MOW, 2011) which should be used when Option A is not appropriate.

Whenever an entirely new road is to be designed and constructed, it is most likely to be in the higher road classes and Option B is then the natural choice, but elements of Option A could still be applied.

9.3 SELECTING GEOMETRIC DESIGN STANDARDS

9.3.1 General

Four different basic geometric standards (DC5-DC8) are defined for LVRs based on the daily number of 4-wheeled (and more) vehicles. The traffic level is the sum of traffic in both directions and is estimated at the middle of the design life period. The vehicle definitions and assessment of traffic is dealt with in *Chapter 8 –Traffic.*

With the provision for varying the geometric design in accordance with the design principles outlined above, the recommended basic geometric standards for the LVR classes are retained for new roads as described and specified in the Road Geometric Design Manual (MOW, 2011) and shown in Table 9-1.

Table 9-1: Recommended basic geometric standards

Road Class	Design Traffic Flow (AADT) (Mid-life)	Surface Type	Right of way (m)	Width (m)	
				Carriageway	Shoulder
DC5 (2)	200 – 400	Paved Unpaved	60	6.50	Varying(1)
DC6	50 – 200	Paved Unpaved	40	6.00	Varying(1)
DC7	20 – 50	Paved(3) Unpaved	30	5.50	Varying(1)
DC8	< 20	Paved(3) Unpaved	20	4.00	n/a

(1) Normal width 1.0 m where required and feasible, but width varying with terrain. See 9.4.2.
(2) DC5 roads with < 400vpd and not likely to change to a higher functional classification within the design period.
(3) On steep sections.

9.3.2 Factors Affecting Design

Factors that will affect the design of each of the four basic geometric standards include:

a) Administrative and functional classification.

b) Traffic composition and accommodation of all road users.

c) Topography or terrain.

d) Nature of the road side population.

e) Road surface type (or structure).

f) Land use and physical features.

g) Construction technology.

h) Economic and financial considerations.

(a) Administrative and functional classification

The existing network is classified in accordance with the Road Act of 2007 into National Roads and District Roads and road classes based partly on administrative aspects of the facility and partly on functional aspects. For LVRs the classifications are:

- **Class B: Regional roads**
  - The secondary national routes connecting a trunk road and district or regional headquarters in a region; or connecting regional and district headquarters.
- **Class C: Collector roads**
  - A road linking a district headquarters and a division centre.
  - A road linking a division centre with any other division centre.
  - A route linking a division centre with a ward centre.
  - A road within an urban area carrying through traffic which predominantly originates from the town and links with either a regional or a trunk road.
- **Class D: Feeder roads**
  - A road within an urban area that links a collector road and other minor road within the vicinity and collects or distributes traffic between residential, industrial and principal business centres of the town.
  - A village access road linking wards to other ward centres.
- **Class E: Community roads**
  - A road within a village or a road that links a village to a village.

To ensure a satisfactory functioning of a new road, a range of geometric design standards may be applicable to several functional classes as shown in the Table 9-2.

Table 9.2: Linkage between road design class and functional class

Road design class	AADT [veh/day] in the design year	Functional class				
		A	B	C	D	E
DC 5	200 – 400		M	✓		
DC 6	50 – 200			✓	✓	
DC 7	20 – 50			✓	✓	✓
DC 8	<20				✓	✓

Notes: ✓ applies to roads in flat to rolling terrain.
M Minimum standard for the appropriate functional class.

The road design standards apply specifically for new roads but might rarely be required for an upgrading project.

(b) Traffic composition and accommodation of all road users

The safe and comfortable accommodation of road users is closely related to the width of the roadway and the travelling speed of motorised traffic. At high travelling speeds of cars, more space is needed for other road users to feel safe. Conversely, wide roads tend to encourage high speeds thereby reducing the level of road safety, both real and perceived. Speed is universally recognised as being closely related to risk of road accidents, hence the LVR design must aim at keeping travelling speeds at acceptable levels.

The typical traffic situations on a DC5 road with less than 400vpd and low percentage, typically < 15%, of heavy traffic, are illustrated in Figures 9-1 to 9-4.

Cars travelling towards the middle with space for pedestrians and motorbike/cyclists on either side, as shown in Figure 9-1.

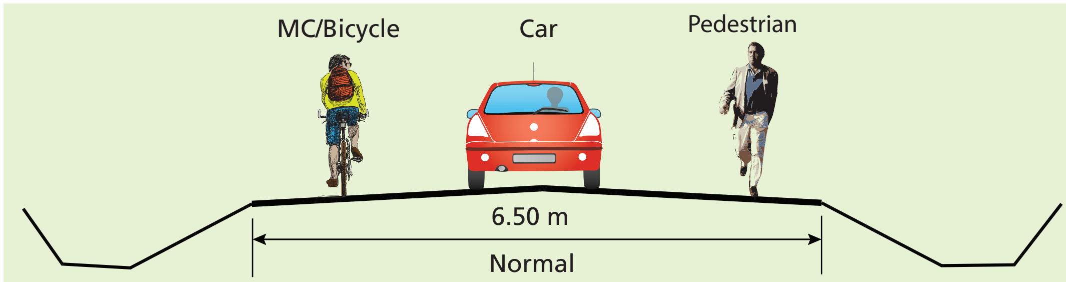

Figure 9-1: Typical traffic situation (D5 standard – schematic)

Two cars meeting (or one being an animal cart) will still leave adequate space for pedestrians, as shown in Figure 9-2.

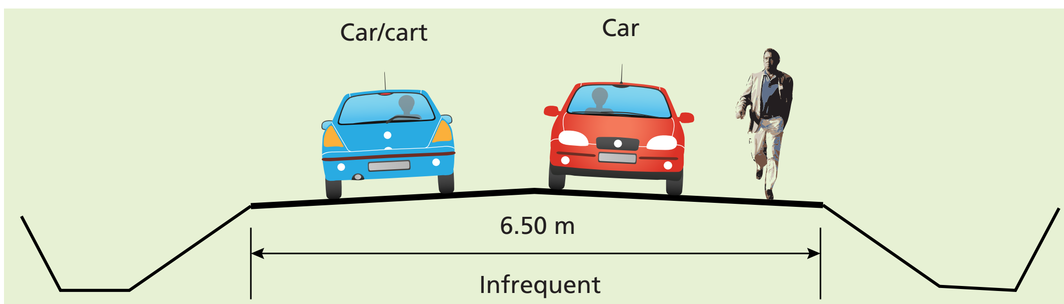

Figure 9-2: Infrequent traffic occurrence (D5 standard - schematic)

Cars and buses meet rarely, as shown in Figure 9-3.

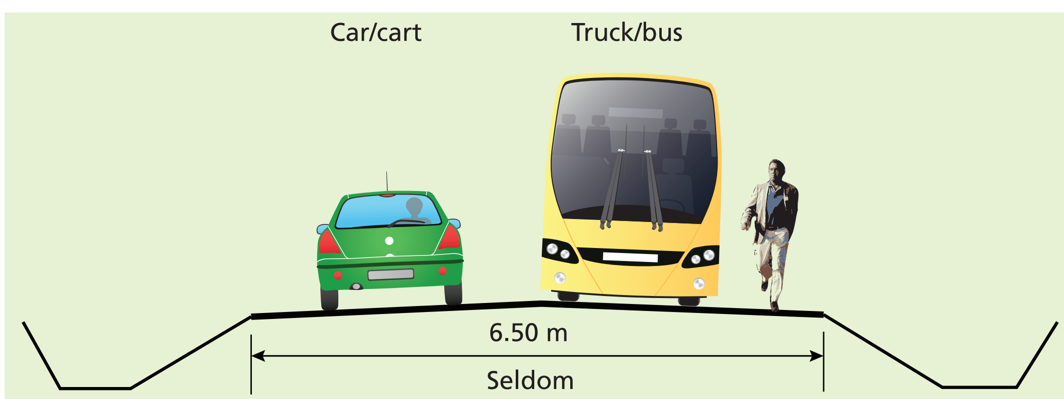

Figure 9-3: Very infrequent traffic occurrence (D5 standard - schematic)

Two buses or a bus and a truck meet very rarely indeed, as shown in Figure 9-4.