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ETHIOPIAN ROADS AUTHORITY

DESIGN MANUAL FOR LOW VOLUME ROADS
PART D

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PART D

EXPLANATORY NOTES FOR LOW VOLUME ROAD DESIGN

Part D
Explanatory notes for low volume road design

- Explanatory notes for low volume road design general introduction
- Site investigation for route selection and design
- Roadside slope stabilisation
- Geometric design
- Drainage
- Materials and pavement design
- Surfacing

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1. GENERAL INTRODUCTION

This Part of the manual provides the following chapters of explanatory notes and supporting information that should be considered during the design process and provides background to the standards shown in Part B of the manual:

- Chapter 2: Site investigation for route selection and design
- Chapter 3: Roadside slope stabilisation
- Chapter 4: Geometric design
- Chapter 5: Drainage
- Chapter 6: Materials and pavement design
- Chapter 7: Surfacing

2. SITE INVESTIGATION FOR ROUTE SELECTION AND DESIGN

2.1 Introduction

Site investigation is an integral part of the location, design and construction of a road and provides essential information on the alignment soil characteristics, construction materials availability, topography, land use, environmental issues (including climate) and socio-political considerations for the client related to the following:

- Selection of the route/alignment of the road;
- Location of water crossings and drainage structures;
- Design information for the road pavements, bridges and other structures;
- Areas for specialist geotechnical investigation;
- Areas of potentially problematic soils requiring additional investigation and treatment;
- Location and assessment of suitable, locally available borrow and construction material.

This list indicates that the main component of site investigations is focussed on what is generally described as 'engineering' or, more precisely, 'geotechnical engineering'. However, various other types of survey are required. Hydrological surveys are required to determine the water flows that determine the drainage design of the road; traffic surveys are required to estimate the numbers of vehicles, both motorised and non-motorised, that will use the road; surveys are required to evaluate environmental impacts and how to control them; surveys are required in which the local communities are consulted about the road project; and so on. This chapter deals primarily with the engineering surveys. Surveys required for these specialist purposes are described in the chapters dealing with those topics.

Information captured during the site investigation is used by the design engineer to prepare and refine the detailed engineering design. This information is usually captured within a series of documents that are prepared by the design engineer, initially for consideration by the client and ultimately to develop the tender and draft contract documents. These documents would normally include separate volumes dealing with the following design aspects:

- Alignment Survey and Geometric Design;
- Traffic and Traffic Loading;
- Materials and Subgrade Design;
- Pavement Design;
- Hydrology, Drainage and Water Crossings;
- Ground Stability and Geotechnical Design;
- Environmental (EIA and outline EMP);
- Social and Complementary Activities;
- Engineer's Cost Estimate.

Road projects fall into one of the following categories:

- A new road following the general alignment of an existing track or trail;
- Upgrading a lower class of road to a higher class;
- A completely new road where nothing currently exists.

Some realignment, and therefore site investigation, will almost certainly be necessary when upgrading an existing road and considerably more will be required when converting a track into an all-weather route. Major site investigations are usually only needed when designing and building a completely new road. In all cases the extent and quality of any investigation has a strong influence on the selection of the most cost-effective route and road design.

Low volume roads, of all standards, require sufficient investigation to provide enough data and information that enables the engineer to optimise the design. In this respect, it is the job of the design engineer to ensure that a well-designed and organised site investigation is undertaken. The design engineer must therefore specify a site investigation programme for the site investigation teams (survey, materials, geotechnical, socio-environmental) that will provide adequate information and data to examine the feasibility of all the routes and designs under consideration.

The focus of this chapter is to provide guidance on the appropriate type and level of site investigation that is required for route selection and subsequent design of low volume roads. The chapter also provides practitioners with the necessary tools to develop suitable site investigation programmes and in-situ testing schedules and with assistance in interpreting the data obtained.

Site investigation techniques encompass a large range of methods and the amount and type of exploration that is needed for a specific road 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 site investigation techniques. Where information on the type, use and interpretation of site investigation techniques is needed, the reader is referred to the Site Investigation Manual (ERA 2011) that covers site investigation procedures for all roads. This complementary manual provides information on the general distribution of local materials in Ethiopia and an explanation of the physiography, geology, terrain, climate, and soil distribution in the country.

This Chapter only deals with the investigation of the site in terms of gathering relevant and appropriate engineering information for the selection of the most suitable route and the subsequent design of the road along that route. Site investigations for low level water crossings are dealt with in Part E of the manual. Where complimentary interventions are required to increase the positive impact of the road project under consideration, the design consultant should be familiar with the requirements in Part C of the manual.

2.2 Stages of Site Investigation

Some form of site investigation is required at all stages in the development of a road project. In general there are four stages leading up to and including Final Engineering Design. These are:

- Identification and general planning
- Pre-feasibility study
- Feasibility Study or Preliminary Engineering Design
- Final Engineering Design

These stages are described briefly in the following sections together with details of the site investigations associated with each stage. The final or detailed engineering design is dealt with in section 2.5. Not all stages will be required for all projects, particularly for projects for upgrading a road from a lower class to a higher class.

2.2.1 Desk Studies

Before any ground survey is carried out and, indeed, before such a survey can be planned and designed, it is vital to study all the relevant information that is available about the project area. This is done through a systematic desk study which entails the collection of detailed information for review and analysis. It allows checking the suitability of all environmental and engineering conditions along different route options. Studying existing documents, including site investigations from earlier project phases, and examining maps and aerial photographs often eliminates an unfavourable route from further consideration, thus saving a considerable amount of time and money. Topographic maps give essential information about the relief of an area, and whether or not there are any existing routes. Aerial photographs provide a quick means for preparing valuable sketches and overlays for reconnaissance/field surveys.

There are a number of very helpful sources of information in Ethiopia that can and should be used for this purpose. Table D.2.1 provides names of federal and other local agencies in Ethiopia where data relevant for site investigation can be obtained.

Table D.2.1: Existing data sources relevant to road construction

Information Resource	Functional Use	Location	Examples
ERA Design Manuals	▪ Additional data and information on aspects of geometry, drainage, pavement and materials and structural design of roads and bridges in Ethiopia	▪ Ethiopian Roads Authority (ERA).	▪ Site Investigation procedures detailed in ERA 2011 Site Investigation Manual; ▪ Data, Information and Maps on climate, terrain, soils, construction materials etc.
Road and Other Engineering Reports	▪ Previous road (and other engineering) investigations in the locality will provide a range of data and information that can supplement project design such as: soil and rock type, strength parameters, hydro-geological issues, construction materials etc; ▪ Information on local road performance and issues.	▪ Ethiopian Roads Authority (ERA); ▪ Regional / Rural Roads Authorities; ▪ Wereda and other local Administrations; ▪ Office of the Road Fund; ▪ Transport Construction Design Share Company (TCD) and other local agencies/ engineering firms.	▪ Road Design and Rehabilitation Reports; ▪ Maintenance Planning and Activity Reports may provide geological, hydrogeological, and geotechnical information for the general area that may reduce the scope or better target the nature of the site investigation.
Aerial Photographs	▪ Identifies manmade structures, potential borrow source areas; ▪ Provides geologic and hydrological information which can be used as a basis for site reconnaissance; ▪ Track site changes over time.	▪ Ethiopian Mapping Agency (EMA); ▪ Information Network Security Agency (INSA); ▪ Other International Agencies such as Quick Bird; IKONOS and Google Earth.	▪ Evaluating a series of aerial photographs may save time during construction material survey.
Topographic Maps	▪ Provides good index map; Allows estimation of site topography; ▪ Identifies physical features; Can be used to assess access restrictions	▪ Ethiopian Mapping Agency (EMA); ▪ Google Earth.	▪ Engineer identifies access areas and restrictions, identifies areas of potential slope instability; and can estimate cut and fill before visiting the site.
Geologic Reports and Maps	▪ Provides information on nearby soil and rock type and characteristics, ▪ Hydro-geological issues, ▪ Environmental concerns	▪ Geological Survey of Ethiopia (GSE); ▪ Ethiopian Mapping Agency (EMA).	▪ A report on regional geology identifies rock types, fracture and orientation and groundwater flow patterns.

Information Resource	Functional Use	Location	Examples
Soil maps	▪ Local soil types; ▪ Permeability of local soils; ▪ Climatic and geologic information.	▪ Ministry of Agriculture; ▪ Geological Survey of Ethiopia (GSE); ▪ Ethiopian Mapping Agency (EMA); ▪ Local soil conservation and research institutes.	▪ The local soil survey provides information on near-surface soils to facilitate preliminary borrow source evaluation.
Meteorological and Climatic data	▪ Mean Annual/Monthly; Rainfall and distribution ▪ Maximum and minimum temperatures; ▪ Evaporation rates; ▪ Weinert-N value and Thornthwaite Moisture Index.	▪ National Metrological Agency of Ethiopia.	▪ Climate controls the degree and type of weathering and influences the type of soils and materials in the locality.

Land use / land cover	▪ Distributional and type of :
	  ▪ Soils;
	  ▪ Road;
	  ▪ Drainage and water courses;
	  ▪ Agriculture and Forest.
		▪ Ministry of Agriculture and local Administrations;
		▪ Universities and research institutes.
			▪ Land use or land cover maps assist to identify the physical and biological cover over the land, including water, vegetation, bare soil, and artificial structures.

Local Knowledge	▪ Traffic classification, Seasonal traffic variation, road user demand, hazards and ground instability, local road performance and maintenance history, accident black spots, water sources, local weather conditions and drainage characteristics.	▪ Regional, Wereda and Kebele Administrations.	▪ Identification of specific problems and hazards along proposed alignment; ▪ Local sources of materials and previous performance.
Statistics and Future Plans	▪ Population data and demographics; ▪ Village distribution and pattern; ▪ Socio-Economic and household survey information; ▪ Development Plans.	▪ Regional, Wereda and Kebele Administrations; ▪ Central Statistical Agency.	▪ Statistical data, information and maps to optimise route alignment; ▪ Recognition of planned future activities within vicinity of planned road corridor.

2.2.2 Identification and General Planning

This is the stage at which the need for the project is identified and projects that do not meet selection criteria defined by the appropriate authorities are rejected. For DC1 and DC2 low volume roads this will usually be done at a relatively local level and will be the output of a planning process. It is likely that only a desk study and possibly a simple reconnaissance survey will be carried out.

2.2.3 Pre-feasibility Study

This is the stage where a broad economic and engineering assessment is made. It is at this stage that the main engineering problems and any other issues affecting the route are identified (for example, environmental and cultural issues) and likely corridors for the proposed road selected.

As part of the pre-feasibility study stage it is important to identify and investigate the major technical, environmental, economic and social constraints through a reconnaissance survey in order to obtain a broad appreciation of the viability of the competing alignment options. For low volume roads, one of the most important aspects of the pre-feasibility study is communication with the people who will be affected by the road. Their views are vital for the completion of a successful project and interacting with them is essential right from the outset.

A reconnaissance survey provides data that enables specialists to study the advantages and disadvantages of a variety of routes and then to determine which routes should be considered for further investigation. It is an opportunity for checking the actual conditions on the ground and for noting any discrepancies in the maps or aerial photographs. During this survey, it is necessary to make notes of soil conditions, especially potentially soil problems; availability of construction materials; unusual grade or alignment problems, water crossings and potential drainage problems; and requirements for clearing and grubbing. It is also very useful to take photographs or make sketches of reference points, structure sites, landslides, washouts, or any other unusual circumstances.

A desk study comprises the first step of the site investigations followed by a reconnaissance survey plus some additional testing to confirm the scale of any significant engineering problems within the potential corridors.

For the lower classes of roads (DC 1 and 2), predominantly unpaved earth or gravel roads, the information from the pre-feasibility study may be the only available data to assist in the design of the road due to financial constraints, hence, it is important to bear this in mind when designing the survey that is to be undertaken. The outputs of the pre-feasibility study for DC1 and DC2 gravel roads should be a single selected alignment for possible further investigation at the feasibility stage if required. For paved roads and most DC3 and DC 4 gravel roads, more than one viable alignment option should be available.

Although not covered in this part of the manual (see Part C), the importance of community participation at this stage, and throughout the project, cannot be over-emphasised as an input to the route selection, design and development of complimentary interventions related to a project.

2.2.4 Feasibility Study or Preliminary Engineering Design

At this stage sufficient data are required to identify the final choice of route and the structural design of the road. The feasibility study survey consists, essentially, of mapping the terrain along the centre-line of the viable route or routes identified at the pre-feasibility stage. Data are required that are sufficient to obtain likely costs to an accuracy of better than about 25%. General costs for similar roads that have been built recently may be used for much of the assessment but the costs for major structures such as bridges and major earthworks need to be estimated sufficiently accurately hence the extent of the site investigation programme is dictated by these requirements.

After the Feasibility Study there should be sufficient information for the final route alignment to be selected. Minor adjustments to the route alignment(s) may still be necessary during design, but the number of iterations needed to establish the best alignment and confirm the choice of the route should decrease significantly.

In some cases the choice of final route alignment might depend on factors other than just the engineering factors. Considerations such as environmental issues, numbers of people within a minimum distance from the road, proximity to historic, religious or other cultural sites and so on (also see Part C) might override the basic economic analysis. Decisions based on some form of multi criteria analysis are available and could be used by those responsible if required.

2.2.5 Scope of Investigations

Table D.2.2 summarises the level of detail generally required for the site surveys and other investigations for low volume roads at each stage of the road design process.

Table D.2.2: Summary of survey requirements for route selection and design

Stage of design	Study	DC1 and DC2	DC3 and DC4
Identification	Engineering	Very broad brush approach based on historic records. Usually a desk study only.	Probably a road or track in existence already. Very broad brush approach based on historic records. 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.	
	Environment	Assessment based on desk study information but concentrated on major issues such as land take and re-instatement.	
	Cost estimation	Historic data only. Based principally on terrain and number of structures. Accurate to only ±100%	
Pre -feasibility	Engineering	In most cases there will be only one route option identified through dialogue with the local community and the design team. Any major problem areas must be identified and avoided if possible.	Each option is broadly specified in terms of alignment, geometric and pavement design and structures. Limited geotechnical surveys may need to be undertaken together with historic surveys to identify basic designs and availability of materials. Limited evaluation of drainage conditions is also required to identify likely numbers and sizes of drainage structures.
	Social	Essential to engage local communities in dialogue concerning the impact of the road. Details now required of land-take and resettlement for each option	
	Environment	Many common environmental issues associated with major roads are unlikely to be significant for Low volume roads but attention must be paid to borrow and spoil areas and likely changes in drainage patterns.	
	Cost estimation	An accuracy of ±50% or better should be possible with the data available and historic information	Largely based on historic records but now supplemented with more detail about the scale (and therefore likely cost) of structures.

<table>
<tr><th>Stage of design</th><th>Study</th><th>DC1 and DC2</th><th>DC3 and DC4</th></tr>
<tr><td rowspan="4">Feasibility</td><td>Engineering</td><td rowspan="3">In most cases there will be only one option identified through dialogue with the local community hence no additional survey data are required to select options.</td><td>Detailed geotechnical surveys may need to be undertaken in unstable mountainous terrain and for major structures. A hydrological study may be needed if substantial rivers are to be crossed. The availability of materials should be confirmed. The data should be sufficient for the preferred option to be selected and specified in terms of alignment, geometric and pavement design and structures.</td></tr>
<tr><td>Social</td><td rowspan="2">The main social and environmental issues should have been identified and a preliminary assessment made. Any additional data should be obtained if required.</td></tr>
<tr><td>Environment</td></tr>
<tr><td>Cost estimation</td><td>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 are required</td><td></td></tr>
<tr><td>Final engineering design</td><td colspan="3">The pre-feasibility and feasibility studies 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 and the required supporting documents outlined in Section 2.1</td></tr>
</table>

2.3 Principal considerations for route selection

This section highlights most of the issues that require consideration when establishing and finalising the route alignment. For upgrading existing roads many of the points raised will not be relevant. For entirely new roads most of the issues should at least be considered and can act as a check list for the road designer.

2.3.1 General considerations and best practice

Socio-Economic:

- The road should be as direct as possible (within the bounds of the geometric standards for the particular class of road) between the cities, towns or villages to be linked, thereby minimising road user transport costs and probably minimising construction and maintenance costs as well.
- The preferred alignment should be one that permits a balancing of cut and fill to minimise borrow, spoil and haul.
- The road should be close to sources of borrow materials and should minimise haulage of materials over long distances.
- The road should not be so close to public facilities that it causes unnecessary disturbance. Cultural sites such as cemeteries, places of worship, archaeological and historical monuments should be

specifically protected. Although a road is designed to facilitate access to hospitals, schools and so on, it should be located at a reasonable distance away for safety and to reduce noise.

- Where the proposed location interferes with utility lines (eg over-head transmission cables and water supply lines), the decision between changing the road alignment and shifting the utility line should be based on a study of the feasibility and the relative economics.
- The road should, as far as possible, be located along edges of properties rather than through them to minimise interference to agriculture and other activities and to avoid the need for frequent crossing of the road by the local people.
- When the road follows a railway line or river, frequent crossings of the railway or river should be avoided.
- The location should be such as to avoid unnecessary and expensive destruction of trees and forests. Where intrusion into such areas is unavoidable, the road should be aligned on a curve so as to preserve an unbroken background.
- The road should be 'integrated' with the surrounding landscape as far as possible. Normally, it is necessary to study the environmental impact of the road and ensure that its adverse effects are kept to the minimum. (see Part C)

Engineering:

- The preferred alignment is one that is founded on strong sub-grades, thereby minimising pavement layer thicknesses. Therefore marshy and low-lying areas and places having poor drainage and weak materials should be avoided.
- Problematic and erosion susceptible soils should also be avoided.
- An important control point in route selection is the location of river crossings. The direction of the crossings of major rivers should be normal to the river flow.
- When an alignment passes near to a river, flood records for the past 50 years must be reviewed, if these are available. Areas liable to flooding and areas likely to be unstable due to toe-erosion by rivers should be avoided (see Chapter D.5)

Other:

- Where possible, the road should be located such that the road reserve can be wide enough to allow future upgrading to a wider carriageway.
- Areas of valuable natural resources and wildlife sanctuaries should remain protected.

2.3.2 Special Considerations in mountainous areas (see also Chapter D.3):

General principles:

- The location should, as far as possible, facilitate easy grades and curvatures.
- High fills should be avoided and special attention should be paid to the compaction of all fills.
- The alignment should minimise the number of hairpin bends. Where unavoidable, the bends and switchbacks should be located on stable ground. A series of hairpin bends on the same face of the hill should be avoided (Chapter D.5).
- In relatively stable slopes, half cut and half fill cross-sections should be adopted to minimise the disturbance to the natural ground.
- Natural terrain features such as stable benches, ridge-tops, and low gradient slopes should be utilized. If a ridge top is considered, roads should be located far enough above convergent gully headwalls or confluences to provide a buffer, otherwise a structure is needed to intercept moving sediment below the road.
- In crossing mountain ridges, the location should be such that the road preferably crosses the ridge at the lowest elevation.
- Needless rise and fall should be avoided, especially where the general purpose of the route is to gain elevation from a lower to a higher point.
- Locations along river valleys have the inherent advantage of comparatively gentle gradients, proximity to inhabited villages, and easy supply of water for construction purposes. However, there are also disadvantages such as the need for large number of cross-drainage structures and protective works against erosion.

- To minimise the adverse effect of moisture on the road environment, an alignment that is predominantly in sunlight should receive priority compared with one that is entirely or partially in the shade throughout the day.

Unstable terrain:

- If possible unstable slopes, areas having frequent landslide problems and benched agricultural fields should be avoided.
- Mid-slope locations on long, steep, or unstable slopes should be avoided. If an unstable area such as a headwall must be crossed, end-hauling excavated material rather than using side-cast methods should be considered.

Erosion potential:

- Erosion is a serious problem in much of Ethiopia. If possible, it is best to avoid areas of high erosion potential. If not, considerable attention is required to dissipate flow in road drainage ditches and culverts and reduce surface erosion (Chapter D.5). It is also advisable to consult local agricultural experts during the process of route selection to ensure that the selected alignment has a minimum potential for soil erosion and that the project design provides sufficient erosion control measures.
- In selecting the best location for the road, the engineering measures designed to minimise erosion will add to the construction costs but ongoing maintenance to deal with debris, blockage and siltation will be required and no erosion protection system is guaranteed.

2.4 Site investigation techniques

The choice of methods for site investigation is determined by the type of road project and the practical problems arising from site conditions, the terrain and climate. A wide variety of methods are used for site investigation. The ERA Site Investigation Manual (ERA 2011) describes the most frequently employed techniques for all aspects of the road design. Only those specialist techniques that can be used for site investigation on low volume roads are described in this chapter.

If an investigation is to be effective, it must be carried out in a systematic way, using techniques that are understood by the industry, relevant to the project in hand, reliable and cost-effective. For low volume roads, investigations should employ relatively standard and simple engineering methods. More sophisticated and expensive procedures should only be employed when a severe geotechnical problem is encountered. 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 bulk samples should be taken for laboratory testing in accordance with the appropriate standard.

Site investigation techniques for unpaved DC1 and DC2 roads can utilise relatively simple sampling and testing techniques. These include visual inspection and description of test pits along the proposed alignment, use of dynamic cone penetrometer testing to assign uniform sections and use of simple material testing kits to assess the grading and plasticity of in-situ soils and borrow materials.

The benefit of utilising the 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. Verification tests in the laboratory will also be required. Strength, compaction 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, reproduced in Annex D.2.

Site investigation techniques for DC3 and DC4 paved and unpaved roads should, in general, follow the traditional techniques used for site investigation on road projects as set out in the ERA Site Investigation Manual (ERA 2011).

2.5 Site investigation for final engineering design

The final engineering design requires sufficient design data for preparation of the tender and draft contract documents. This stage requires the most rigorous site investigation and considerably more data will 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 the contracts has been awarded. Detailed investigation will be required to provide technical data on the following:

- Topography
- Traffic count and loading
- Alignment soils and construction materials (fill, gravels, rock, potential aggregate, sand and water) – including potential haul and quantities
- Hydrology and drainage
- Ground stability, geotechnics and the characteristics of water crossings
- Socio-environmental considerations

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 DC1 and for many DC2 class low volume roads the design of the feasibility study should be such that most of the information obtained should be sufficient for final design. The data obtained at the feasibility stage will not be so comprehensive and will not be so robust from a statistical point of view as that obtained from site investigations for DC3 and DC4 roads. However, it should be adequate and reliable and sufficient to provide a competent design for DC1 and DC2 low volume roads. It is likely that some additional detailed survey will be required, particularly for water crossings, within areas of problem soils and unstable terrain.

The quality and level of the site investigation 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 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 sub-grade conditions.

An important assumption is that the topographic survey, based on the preliminary route alignment has been completed prior to the detailed site investigation. It is only against the topographic model that locations of structures and other features of the design can be fixed and estimates made of quantities, haulage and ultimately construction costs.

In general, the site investigation for final design will focus on sampling and testing of materials to provide information on the following reports:

- Characteristics of alignment soils and in-situ materials
- Location and characteristics of construction materials, volumes available and haulage
- Earthworks investigations – cut and fill
- Water crossings
- Water sources

2.5.1 Characterisation of alignment soils and in-situ materials (subgrade)

The subgrade can be defined, in terms of location, as the upper 600mm of the road foundation. The subgrade is required to resist repeated stressing by traffic and to be stable to the stresses imposed by varying climatic and moisture influence.

The character of the subgrade is determined by the geological and weathering characteristics of the rocks that produce the soil and the interaction with the local climate, moisture and drainage regime prevailing in the area. As a general "rule of thumb" better subgrades are found in well drained areas. Clayey soils often predominate in flat areas and along valley floors.

The design of a paved or unpaved road is very strongly dependant on the characteristics of the subgrade and, therefore, so is its potential performance. The desirable properties of a good subgrade include high strength, high stiffness, good drainage characteristics, ease of compaction and low compressibility. A good subgrade is strong enough to resist shear failure and has adequate stiffness to minimize vertical deflection. Stronger and stiffer materials provide a more effective foundation for the pavement layers and are more resistant to stresses from repeated loadings and environmental (moisture) conditions. Most importantly, the stronger the subgrade, the thinner the pavement layers above need to be. Unfortunately the designer usually has very little choice about the subgrade for most of the route.

Because the road design is so dependent on the subgrade, 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 (eg. improving drainage or stabilisation) or the road alignment must be altered to avoid such areas completely.

DCP surveys

The most cost effective method for obtaining sub-surface information to a depth of approximately 800mm is by using the DCP test. The use of the DCP helps to delineate homogenous subgrade sections along the road and to identify soft spots of the subgrade for further investigation using pits and trenches. The advantage of the DCP is that information can be gathered without disturbing the in-situ material. Using this test, strength characteristics of the subsurface soils at field moisture and density conditions can be obtained directly. The equipment is light and portable and is also useful for investigating the characteristics of all the pavement layers of existing roads for rehabilitation projects. In addition, for road widening and upgrading projects, DCP tests along the main pavement can be compared to those from shoulders. DCP tests can also be used for quality control during construction.

DCP tests are quick and simple. Tests can be carried out every few hundred metres along the chosen alignment to delineate uniform sections. Where obvious changes of surface conditions occur more frequently, 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. As a minimum standard, four DCP tests per kilometre should be used for DC1 and DC2 roads, whilst ten DCP tests per kilometre should be used for DC2 paved roads and all DC3 and DC4 roads.

A number of correlations exist to link the DCP penetration rate (mm/blow) to the subgrade strength parameters required for a pavement design. These correlations are based on either soaked or unsoaked CBR values versus DCP penetration rates measured in different soil types. It is important to make sure that the correlation being used is the correct one for the purposes of the study. In general, the correlation should be between the DCP penetration rate and the actual CBR of the material being tested (i.e. the CBR at the density and moisture content of the material at that time). In this way the in-situ strengths can be determined.

Test pits and trenches

Test pits and trenches are used to provide samples for testing and information on the in-situ subgrade soil and potential fill material.

The location, frequency and depth of pits and trenches for characterizing the subgrade depend on the type of the road and the general characteristics of the project area (the soil type and variability). In addition, the DCP testing carried out to assist delineation of uniform sections can be used to target areas for pitting and trenching. Spacing will decrease when the subsurface soils demonstrate more variability. In these areas, pits can also to be staggered left and right of the centerline to cover the full width of the road formation.

The depth of pits and trenches is determined by the nature of the subsurface. In pavement design, the depth of influence is related to the magnitude and distribution of traffic loads. Current AASHTO and many other standards limit this depth to 1.5m below the proposed subgrade level. 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 be increased to 1.5 m below the subgrade if required, but this will rarely be necessary for such projects.

For a new alignment, the depth of any pit should not be less than 2m unless a rock stratum is encountered. Some problem subgrade conditions may require deeper exploration. Greater depths may also be needed for high embankment design. This is also true for boulder identification as buried basaltic boulders are common in the highlands of Ethiopia. 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 preliminary 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 promote a proper assessment of the materials excavated in cuts 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 affected by foundation loads. 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 thick problem soils and soft deposits exist, and when 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
- **Index Tests:** Particle size distribution (AASHTO T88); Plasticity/Atterberg limits (AASHTO T89 and T90); Linear shrinkage
- **Compaction:** Density and Moisture relationship (Standard AASHTO T99)
- **Strength:** CBR and swell (AASHTO T99 or T180)

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

For the design of DC1 and DC2 low volume roads, a presumptive design CBR could be assigned on the basis of previous test data and the performance of soils in similar environments. 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.

2.5.2 Problem soils

Soils which can cause foundation problems and decrease the performance of roads are common in many parts of Ethiopia. These soils are collectively called problem soils and comprise among others; expansive; collapsible and compressible; and dispersive soils. The identification of such soils is crucial during site investigation so that appropriate designs can be established at the outset. Failure to recognise problem soils at the design stage could result in claims and overruns if identified later during construction or detrimental impact on the long term performance of the road.

Expansive soils

Problems are associated with the presence of expansive soils on the road alignment include:

- A very low bearing capacity when wet.
- Variable and seasonal moisture distribution leading to differential volumetric movement, settlement and cracking.
- Difficulty in locating any associated natural gravels for sub-bases and bases.

Expansive soils are typically clayey soils that undergo large volume changes in direct response to moisture changes in the soil. Although the expansion potential of a soil can be related to many factors (eg soil structure and fabric; and environmental conditions), it is primarily controlled by the clay mineralogy (eg smectites and montmorillonite).

Known as vertisols in agricultural soil classifications, expansive soils are found in the central, north-western and eastern highlands of Ethiopia; in the western lowlands around Gambella; and in some parts of the rift valley. Local deposits of these soils are also present throughout the country near rivers; water logged areas; and in drainage restricted localities. Damage caused by expansive clays is particularly prevalent around Addis Ababa.

Volume changes in expansive soils are confined to the upper few metres of a soil deposit where seasonal moisture content varies due to drying and wetting cycles. The presence of surface desiccation cracks or fissures in a clay deposit are indications of expansion. 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 becomes nearly constant is the limit of the active zone. This is also referred to as the depth of seasonal moisture change.

Several empirical relationships have been developed to identify expansive soils, although a standard classification procedure does not exist. Generally, soils with a plasticity index (PI) of less than 15% and liquid limit below 55% do not exhibit expansive behaviour. For soils with a plasticity index greater than 15%, the clay content of the soil should be evaluated in addition to the Atterberg limits.

Figure D.2.1 relates expansion potential and collapsibility to liquid limit and in-situ dry density. Additional tests for the qualitative assessment of expansion potential include percentage swell calculated from the CBR test (ASTM D4429), the free swell test, and the expansion index test (ASTM D4829). Such correlations are semi-empirical but can be used for an initial assessment of the expansion potential of a soil.

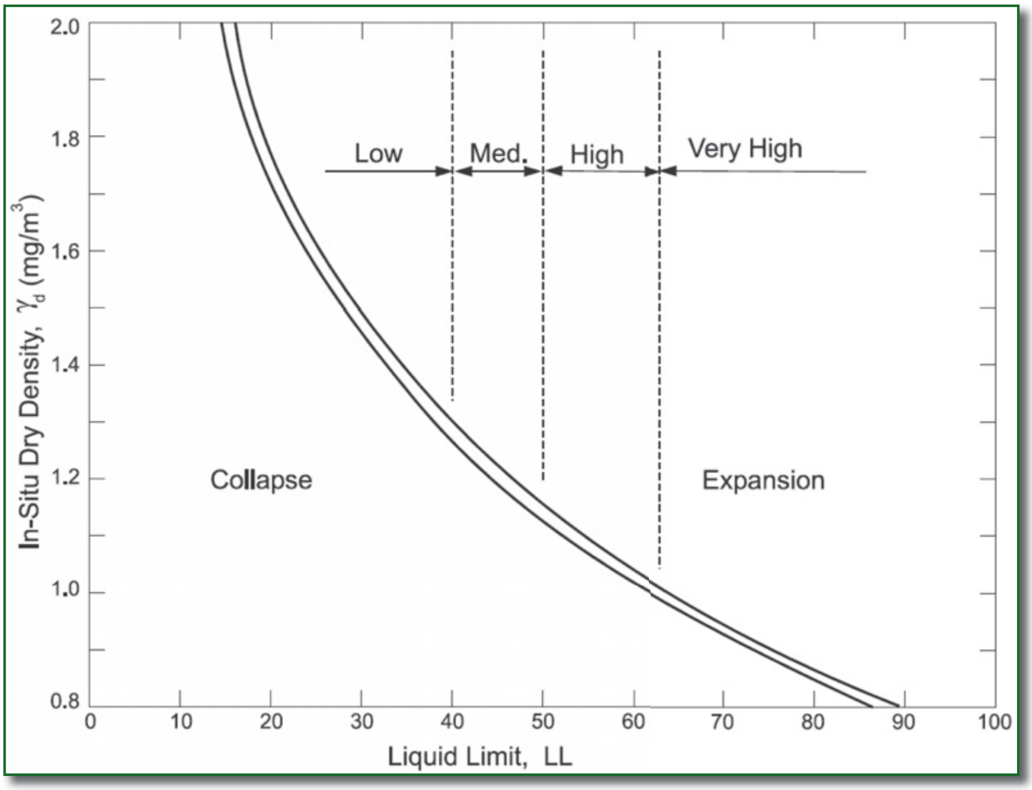

Figure D.2.1: Guide to collapsibility and expansion based on in-situ dry density and liquid limit (after Mitchell and Gardner, 1975 and Gibbs, 1969)

For classification purposes, the US Bureau of Reclamation developed a correlation between observed volume changes and colloidal content, plastic index, and shrinkage limit as shown in Table D.2.3. The measured volume changes were taken from oedometer swell tests using a surcharge pressure of 7 KPa from air-dry to saturation conditions.

Table D.2.3: Classification of expansive soils according to US Bureau of Reclamation

Colloid content % < 1µm	PI (%)	SL (%)	Potential expansion (%)	Degree of expansion
<15	<18	>15	<10	Low
13-23	15-28	10-16	10-20	Medium
20-31	25-41	7-12	20-30	High
>28	>35	<11	>30	Very high

Appropriate designs that can be used to reduce the effect of expansive soils on pavements are provided Chapter D.6. Treatment options and recommendations are also provided in the ERA Geotechnical Design Manual (ERA 2011).

Collapsible soils

Collapsible soils are generally described as soils that undergo a relatively significant, sudden and irreversible decrease in volume upon wetting. These types of soils predominantly consist of silt and sand with some clayey material. Deposits of collapsible soils are usually associated with regions of moisture deficiency, such as those in arid and semi-arid regions.

Collapsible soils are present in the southern part of the Omo River and in the central and southern part of the rift valley. Often, their existence around Zeway, Shashemene, and Awassa is manifested by the occurrence of ground cracks and a series of potholes after heavy rains in spring. In the Afar region, collapsible soils are present in the form of sand dunes.

Collapsible soils usually exist in the ground at very low values of dry unit weight (density) and moisture content. In their natural conditions, collapsible soils can support moderate loads and undergo relatively small settlements. They are also moderately strong and exhibit a slight but characteristic apparent cohesion. Usually, this cohesion is the result of calcareous clay binder that holds the silt particles together. The clay coating and the silt create a very loose soil structure with little true particle-to-particle contact. Upon wetting, however, the cohesion is lost and large settlements can occur even if the load remains constant.

For rapid identification, the liquid limit can be used. If under normal circumstances the void ratio of a given soil is higher than that at its liquid limit, on absorbing water the soil will lose strength. Before saturation is achieved, the soil will undergo considerable structural collapse accompanied by a reduction in volume. If this is the case, then laboratory testing of undisturbed samples should be performed to quantify the magnitude of volume reduction. Silts containing collapsible soils are often also characterized by being extremely erodible.

Typical pit excavation and disturbed sampling procedures can be used to obtain soil samples for sieve analysis, hydrometer, soil classification and Atterberg limits. For samples to be collected at shallow depths, it may be prudent to obtain block samples from trenches or test pits. Unlike expansive soils, where volume change occurs in the top few metres, the depth of collapse can be much higher and may involve loose soils in the region. In the rift valley, there are indications that the thickness of potential collapsible soils is greater than 8 m. In this case auger sampling or shallow boring can be considered.

For situations in which it is necessary to construct a road on collapsible soils, it is of primary importance to estimate the magnitude of potential collapse that may occur if the soil becomes wet. The amount of collapse normally depends on the initial void ratio, stress history of the soil, thickness of the collapsible soil layer and magnitude of the applied stress.

The collapse potential (CP) is calculated as the percentage collapse of a soil specimen using the change in void ratio before and after saturation. It is an index value used to compare the susceptibility of collapse. Table D.2.4 provides a relative indication of the degree of severity for various values of CP. Additional information is provided in the Site Investigation Manual - 2011.

Table D.2.4: Qualitative assessment of collapse potential

Collapse Potential (CP)	Severity of Problem
0 - 1%	None
2 - 5%	Slight
6 - 10%	Moderate
11 - 20%	Severe
> 20%	Very severe

Appropriate designs that can be used to reduce the effect of collapsible soils on pavements are provided Chapter D.6. Treatment options and recommendations are also provided in the ERA Geotechnical Design Manual (ERA 2011).

Dispersive soils

Soils in which the clay particles detach from each other and from the soil structure without a flow of water and go into suspension, are termed dispersive soils. These soils deflocculate in the presence of relatively pure water to form colloidal suspensions and are, therefore, highly susceptible to erosion and piping. Normally, they contain a higher content of sodium in their pore water than other soils. However, there are no significant differences in the clay contents of dispersive and non-dispersive soils although soils with high exchangeable sodium such as Na-montmorillonite clays tend to be more dispersive than others.

Dispersive soils tend to develop in low-lying areas with gently rolling topography and relatively flat slopes. Their environment of formation is also characterized by an annual rainfall of less than 850 mm. Dispersive soils have low natural fertility. Often, they are calcareous with a PH value of about 8. Suspicion of their presence is indicated by the occurrence of erosion gullies and piping.

In Ethiopia, dispersive soils exist in the rift valley, the southern and eastern lowlands, and Afar, Somali and Tigray regions. Isolated occurrences of these soils can also be found in other parts of the country.

It is difficult to identify dispersive soils using conventional engineering index tests such as Atterberg limits, gradation or compaction characteristics. Chemical properties can determine the dispersion potential of soils by measuring the dissolved sodium in the pore water.

The pinhole (BS1377 Part 5 – clause 6.2) and Crumb Test (BS1377 Part 5 – clause 6.3) provides a relatively simple way of identify dispersive soils in the field. The test starts by collecting soil aggregates (1-2 cm diameter) from each layer in the soil profile. The aggregates are dried in the sun for a few hours and placed in a small bowl of rain water. The aggregates are left in the water without shaking or disturbing for 2 hours. The samples are then observed for a milky ring around the aggregates and classified as shown in Figure D.2.2. The soil is highly dispersive if discoloration and cloudiness extends throughout the jar or bowl.

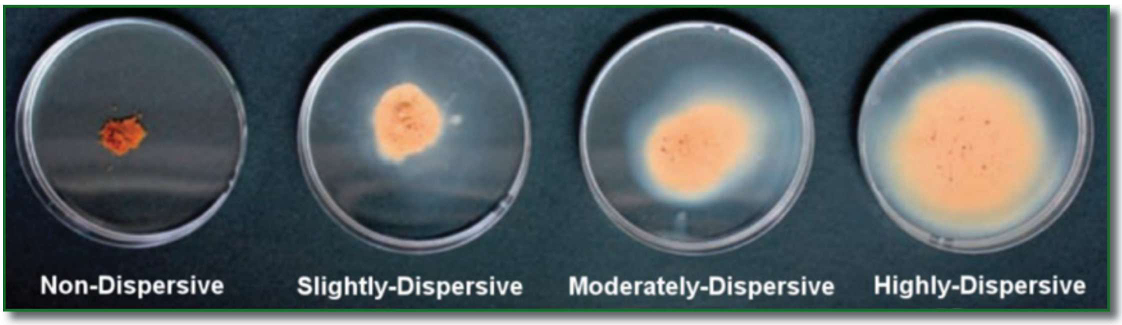

Figure D.2.2: A simple way of testing the dispersive nature of soils

While laboratory tests such as the crump, dispersion, pinhole tests are a useful way of identifying dispersive soils, much can be determined by observing the behaviour of the soils in the field. For instance, the occurrence of deep erosion gullies, 'worm channels', and piping failure in existing road embankments indicates the presence of dispersive soils. Erosion of road cuttings along ditches, gully lines or weathered rock joints; cloudy water after rains; and high turbidity in ponds are also linked to the effect of dispersive soils.

The geology of the area can also be a guide to the presence of dispersive soils. Many dispersive soils are of alluvial origin. Soils derived from shale and clay-stone in sedimentary areas and pyroclastic sediments in volcanic regions are also dispersive in nature.

2.5.3 Location and characteristics of construction materials, volumes available and haulage

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 this 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 the materials are of marginal quality or occur only in small quantities.

The site investigation must identify and prove that there are adequate and economically viable reserves of natural construction materials. The materials required are:

- Common embankment fill;
- Capping layer / imported subgrade;
- Sub-base and road-base aggregate;
- Road surfacing aggregate;
- Paving stone (eg for cobblestone pavements);
- Aggregates for structural concrete;
- Filter/drainage material;
- Special requirements (eg 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:

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

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.

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 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 of ERA, regional road authorities, or by road design consultants and construction supervising organisations 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 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 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, 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 with regards the pavement thickness design (see Part D -6). 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 road base 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 sub-base materials are usually spaced widely. In current practices, distances between these pits of about 50km are not unusual. Main sources of sub-base and base materials are rocky hillsides and cliffs, high steep hills, and river banks. In Ethiopia, sub-base materials have also been extracted from cinder cones and lateritic deposits. Sub-base materials are expected to meet requirements related to maximum particle size, grading, plasticity, and CBR.

The minimum thickness of a deposit normally considered workable for excavation for materials for subgrade, sub-base 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 300mm may be excavated.

Hard Stone and aggregate: The ERA Site Investigation Manual (ERA 2011) provides some details on the location and variety of rocks in Ethiopia 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.

Prospecting and testing: The earlier feasibility and pre-feasibility studies will have likely have 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.

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.

Site investigation activities will include detailed prospecting for materials through surface mapping, test pitting, boreholes, material sampling, and in situ testing. A variety of sub-surface sampling and investigation procedures appropriate for different materials are used to recover the samples needed for laboratory testing. The ERA Site Investigation Manual (ERA 2011) deals with the techniques required for materials prospecting.

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.

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, eg 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 (see Table D.2.5). 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

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 (eg crushed aggregate for surfacing).

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

The frequency of testing of borrow pit material needs to strike a balance between cost, time and statistical validity. Where possible, the location and testing of borrow pit material should be done by traditional methods using full laboratory facilities. In the absence of these facilities, testing using the test kit described in Appendix D.1 should be substituted for DC1 and DC2 earth and gravel roads.

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 low volume roads projects, 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 of the variation in material quality that can be expected during construction for process control.

Table D.2.5: Typical laboratory tests used to assess the suitability of alignment soils and pavement materials

	Subgrade & Fill	Sub-base	Road Base	Surfacing Aggregate	Wearing Course Gravel
Index Tests					
▪ Atterberg limits	✓	✓	✓		✓ (see note 1)
▪ Linear Shrinkage					✓ (see note 1)
▪ Particle Size Distribution	✓	✓	✓		✓ (see note 1)
Compaction and Strength Tests					
▪ Dry Density and Optimum Moisture Content	✓	✓	✓		✓
▪ CBR and CBR Swell	✓	✓	✓		✓
Particle Strength Tests (aggregate dependent)					
▪ Durability Mill			✓	✓	
▪ 10% FACT, AIV, ACV, LAA			✓	✓	
▪ Glycol Soak			(✓)	(✓)	
▪ Water absorption			✓		
▪ Specific Gravity			✓	✓	
▪ Flakiness and ALD				✓	
▪ Affinity with bitumen - Immersion tray test				✓	

Note 1:

1. Refer to appropriate test techniques and methods in Appendix D.1

Weathering

The quality and durability of borrow materials and crushed stones can be greatly affected by weathering or alteration. 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 dry areas, weathering is predominantly physical, and rock masses disintegrate by alternate heating and cooling, 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. Examples of differential weathering are:

- Resulting from compositional or textural differences;
- At contact zones associated with thermal effects within volcanic rocks;
- Along permeable joints, faults, or contacts where weathering agents can penetrate more deeply into the rock mass;
- Within a single rock unit due to relatively high permeability; and
- Resulting from topographic effects.

Table D.2.6 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 categories 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 D.2.6. 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.

Table D.2.6: Weathering grades for road design and construction

Grade	Descriptive term	Chemical weathering		Texture	General characteristics (strength, excavation, etc)
		Body of rock	Fracture surfaces		
W1	Fresh	No discoloration, not oxidized	No discoloration or oxidation	No change	Hammer rings when crystalline rocks are struck. Almost always excavation involves rock except for naturally weak or weakly cemented rocks such as siltstones or shales.
W2	Slightly weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull.	Minor to complete discoloration or oxidation of most surfaces	Preserved	Hammer rings when crystalline rocks are struck. Body of rock not weakened. With few exceptions, such as siltstones or shales, classified as rock excavation.
W3	Moderately weathered	Discoloration or oxidation extends from fractures, usually throughout; Fe-Mg minerals are "rusty," feldspar crystals are "cloudy."	All fracture surfaces are discoloured or oxidized	Generally preserved	Hammer does not ring when rock is struck. Body of rock is slightly weakened. Depending on fracturing, usually considered as rock during excavation except in naturally weak rocks such as siltstone or shales.
W4	Highly weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, see grain boundary conditions.	All fracture surfaces are discoloured or oxidized, surfaces friable	Texture altered by hydration	Dull sound when struck with hammer; can be broken with moderate to heavy pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures, rock is significantly weakened. Usually easy for excavation.
W5	Decomposed	Discoloured or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay	----	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete	Can be granulated by hand. Always easy for excavation. Resistant minerals such as quartz may be present as grains

2.5.4 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 stability and likely requirement for detailed surface and subsurface investigations.

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. Moreover, an understanding of the local geology is essential. 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 100m long slope cut. The trenches should be located at places where material changes are expected and range between 1m and 3m 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 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 Site Investigation Manual - 2011).

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.

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 many parts of Ethiopia. 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 greyed soils indicates a slope that is saturated for much of the year. Occasionally, mottles may appear above greyed 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 greyed 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 ERA Site Investigation Manual (ERA 2011) and ERA Geotechnical Design Manual (ERA 2011) can be referred 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 geotechnically fragile areas, increased earthworks can lead to an increased risk of landsliding.

2.5.5 Water crossings

Site investigation techniques needed for appropriate low level structures for water crossings are explained in Part E of this manual. Additional information is also provided in the ERA Site Investigation Manual (ERA 2011).

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 ERA Bridge Design Manual (ERA 2011) and thus the site investigation techniques and procedures described in the ERA Site Investigation Manual (ERA 2011) must be followed.

2.5.6 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. 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 Ethiopia, it is important to search for water sources, their yields and the distances from the construction site. In regions where water is scarce, such as the Afar and Somali regions, a separate and dedicated hydro-geological study may be needed. Alternatively dry compaction could be considered for some types of materials (see Chapter D.6). Data from the field reconnaissance can indicate if surface water is a critical problem.

In the rift valley, water sources for construction need to be chemically analysed to assess the concentration of chloride and sulphate, which could be deleterious to performance of concrete.

The Ethiopian Geological Survey has a collection of hydro-geological maps for various parts of Ethiopia. There are also some private and state agencies which are responsible for water resource studies in the country. Additional information can be obtained from these sources.

When water is a concern for the road construction, it is advisable to contact the Geological Survey of Ethiopia or the Regional Water Resources Bureau. It is also recommended that all water resource surveys are carried out by specialist practitioners.

2.6 References

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3. ROADSIDE SLOPE STABILISATION

3.1 Introduction

Unstable natural slopes, cuts and embankments in the highlands of Ethiopia often disrupt traffic flow and create a considerable problem to road users during rainy seasons. These slope failures or landslides typically occur where a natural slope is too steep, a cut slope in soil and/or weathered rock contains weak materials or adverse joints, or fill material is not properly compacted. The first two of these factors are interdependent and cannot be considered as separate entities. In all three cases, a rise in groundwater will lead to an overall reduction in stability and possibly failure. Once slope failures are initiated they can expand rapidly, causing even greater traffic holdups and maintenance costs.

Erosion can also take place on unprotected cut and fill slopes and in river channels, especially downstream of culverts, bridges and roadside turnouts. In addition, uncontrolled runoff can erode the roadside drain, road pavement and the edge of the road. Sediment derived from the erosion not only impacts on the road, but also on the wider environment. Large landslides can also contribute significant volumes of debris to watercourses, with significant downstream effects.

Slope stabilisation and erosion control usually employs a number of methods to reduce the causes of failure, together with measures to improve the stability of slopes. During design, it is important to select affordable methods that are relevant to the class of the road, the type of landslide, the materials involved and the extent of the slope instability problem. Techniques commonly used to prevent the occurrence of landslides and to stabilise the existing slope failures include earthworks (cuts and fills), retaining structures and revetments, surface and sub-soil drainage and bioengineering. Stabilisation methods that involve more substantial engineering works involve anchoring, piling and deep subsurface drainage, but these are rarely used on low volume roads. For LVRs, a combination of bioengineering, low cost retaining walls such as gabions, dry-stone and mortared masonry walls, and surface drainage structures is a cost-effective method to stabilise slopes.

It should be noted that whilst this chapter is based on international best practice, the recommendations may not be the only options needed to stabilise slopes and mitigate the effects of landslides in all regions of Ethiopia. The engineer should combine these recommendations with existing local practice.

For most minor slope failures, any remedial work will often rely on visual observation and the use of simple measures such as debris clearance, trimming, and the removal of overhangs. While this may be acceptable for many situations along low volume roads, there are occasions when additional measures are required. These include a combination of bioengineering, low-cost retaining structures, and drainage. For more complex slope stability and geotechnical issues, ERA's Geotechnical Design Manual should be consulted.

3.2 Roadside slope instability

Slope instability associated with roads is classified as that which occurs either above or below the road (Hunt et al, 2008). The slope failures above the road involve the road cut and the natural hill slope above it. Those below the road can affect both fill slopes and the valley slope and the natural ground below.

3.3 Slope instability above the road

3.3.1 Types of slope failure above the road

The types of slope failures and erosion processes commonly observed above a road alignment include (Figure D.3.1):

- Erosion of cut slope surface;
- Failures in cut slope;
- Failures in cut slope and hill-slope;

- Failures in hill-slope;
- Deep failure in original ground.

Once a cut has been formed the slope is exposed to erosion. The effect of erosion is normally to remove the weathered mantle. This may lead to the formation of gullies, and ultimately to slope failures. Erosion is especially common on cuts in weak rocks such as shale, marl, ash and tuff.

Failures in cut slopes usually occur due to a change in the external forces acting on the soil, a change in the geometry of the cut, or the introduction of water. Many of the failures in cut slopes occur in the form of minor slips. Sometimes, however, these small slips may develop into major failures and affect the entire road corridor. In cut slopes, these failures often develop at the toe of the slope and propagate upward. The majority of failures affecting the natural hillside above roads are caused by the removal of support in the road cutting, combined with the effects of groundwater rise during heavy or prolonged rainfall.

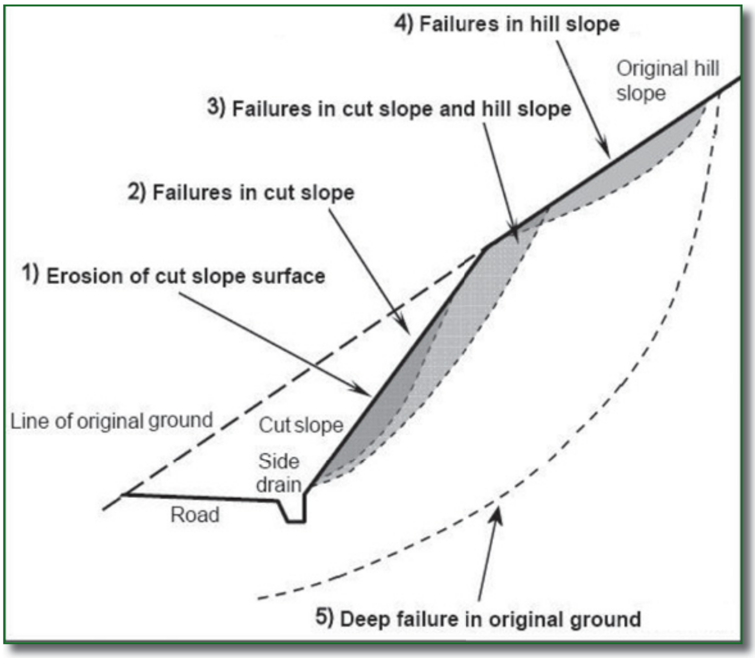


Figure D.3.1: Slope failures that occur above the road (modified from Hunt et al, 2008).

Failures in hill slopes usually develop over long periods of time through geological and geomorphological processes. These slopes are only stable if the soil has sufficient strength to resist gravitational forces. Changes in pore water pressure conditions, slope geometry and engineering works may cause these natural slopes to fail.

Deep-seated failures are defined, geologically, as those that involve bedrock. Generally in the tropics and sub-tropics they can be regarded as failures deeper than 5 metres. These landslides are usually caused by adverse joints in the underlying rock mass, or by the development of water pressures at depth in soils or within jointed rock. These types of failures often occur naturally on hillsides. If located on the slopes below a road they can quickly expand upslope until the road is affected. If a road is constructed across these landslides, it is likely that ground movements affecting the road will take place during rainfall. Sometimes, slope excavation for road construction can initiate deep-seated landslides that can reactivate later and result in frequent road blockage.

The techniques commonly used to protect and stabilise slopes and prevent the occurrence of landslides above a road alignment, especially along low volume roads, are summarized in Table D.3.1. The use of these techniques depends on site-specific conditions such as the size of the slide, soil type, road use, and the cause of failure.

Table D.3.1: Stabilisation and protection measures for slope failures and erosion above the road (based on Hunt et al 2008)

Instability	Stabilisation options	Drainage options	Protection options
1. 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.
2. 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 is usually important to prevent surface erosion and increase the resistance of the surface soil.
3. 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 is usually important to prevent surface erosion and increase the resistance of the surface soil
4. 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 is usually important to prevent surface erosion and increase the resistance of the surface soil
5. 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 is usually important to prevent surface erosion and increase the resistance of the surface soil.

3.3.2 Cut slopes

A major cause of cut slope failures is related to the release of stress within the soil upon excavation. This includes undermining the toe of the slope and over-steepening the slope angle, or cutting into heavily over-consolidated clays. Cut slope failures also occur where adverse joints within the underlying rock mass become exposed in the excavation. Careful consideration should be given to prevent these situations by designing cut slope angles to be stable (ie avoiding over-steep angles), or using retaining walls (typically gabion or masonry) to provide the necessary support.

Sometimes cut slope failures occur where erosion in side drains removes sufficient support. This can be prevented by constructing lined drains, keeping the base of the slope as supported as possible, or by locating drainage ditches a suitable distance away from the toe of the cut. Consideration should also be given to establishing vegetation on the slope to prevent long-term erosion.

Minor cut slopes are generally designed in a prescriptive way based on past experience with similar soil and rock materials. Cut slopes greater than 3m in height usually require a more detailed engineering geological assessment depending on the complexity of the ground conditions. This would include an assessment of the strength of the soil or the orientation of joints in the rock. This assessment can be done in a descriptive way with plots of some representative sections.

The slope angles indicated in Table D.3.2 have been provided as a guide for LVRs. Note that these angles cannot be applied without due consideration of the ground conditions. In the design phase, the slope angles can be used to establish the arrangement between sections of road on the same slope (eg hairpin sections). During construction and maintenance, the slope angles are useful indicators to maintain the stability of slopes and reduce the possibility for the occurrence of landslides.

Cuttings in strong rocks can often be very steep where adverse joints are 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 type of geological materials exposed, the stratigraphy, and the variations in permeability between the different horizons. One of the simplest 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. 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.

Table D.3.2: Common cut-slope ratios for LVRs.

Soil and rock condition	(Horizontal : Vertical)
Most hard rocks (without adverse jointing)	1:4 – 1:2
Closely fractured rock	1:2 – 1:1
Well consolidated highly to completely weathered rock (residual soil)	1:2 – 1:1
Dense coarse granular soils	1:2 – 1:1
Loose coarse granular soils	3:2
Plastic clay soils	2:1 – 3:1
Low cuts (less than 3 m high)	2:1

In general, single-sloped cuts along LVRs should be limited to a height of 5m. For deeper cuts, 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.

In upgrading or rehabilitation projects, the remediation of cut slope failures usually requires the removal of failed material from the road and side drain and any overhangs and potentially unstable masses. A decision has to be made as to whether the slope can be stabilised through earthworks, or whether a retaining structure isrequired. In some cases the removal of slip debris can serve to undercut the slope above causing further failure. In such cases a gabion or masonry retaining wall can be constructed to support the slope. Most cut slope failures are shallow and the most common forms of treatment comprise removal, trimming and drainage. Bio-engineering should be considered as an integral part of the solution. Any cut slope where failure would result in large rehabilitation costs or would threaten public safety should be designed using more rigorous techniques. Situations that warrant more in-depth analysis include large cuts, cuts with complex geological structure (especially if weak zones are present), cuts where high groundwater or seepage pressures are likely, cuts involving soils with low strength, cuts in landslide debris, and cuts in formations susceptible to landslides.

3.4 Slope Instability below the road

3.4.1 Types of slope failures below the road

The type of slope failures and erosion processes commonly observed below a road, as shown in Figure D.3.2, can be categorized into five classes as follows:

- Erosion of fill slope surface;
- Failures in fill slope;
- Failures in fill slope and original valley slope;
- Failures in original valley slope;
- River under-cutting.

Most earth embankments or fill slopes face light to moderate erosion problems arising from rainfall splash, surface runoff from the road and agricultural activities. The problem along roads is especially the occurrence of surface runoff which results in sheet erosion and the formation of rills and gullies on poorly compacted and unprotected embankment shoulders and slopes. The degree of erosion is normally a function of material type and its compaction, rainfall or runoff intensity, slope angle, length of slope, and vegetation cover.

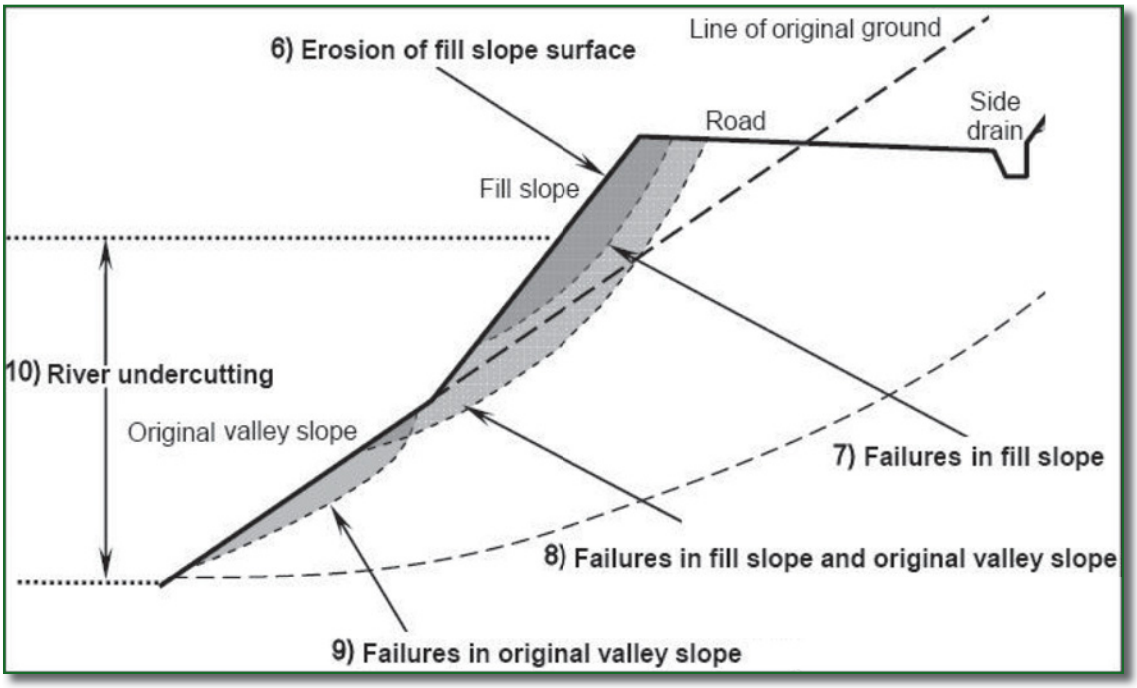


Figure D.3.2: Slope instabilities expected below the road (modified from Hunt et al, 2008).

Slope failures in fill slopes are often in the form of small-scale shallow translational slides, where the failure is contained entirely within the embankment side slopes and maximum depth of rupture does not exceed 2m. Generally, embankment stability is dependent on fill type and compaction, presence of water or drainage provision, shrink and swell cycles, vegetation, slope angle and height, construction method and type of foundation. Failure in embankments during and after construction can occur at the interface between the natural ground and the fill, if the natural ground is incorrectly prepared.

In some cases, fills and embankments can serve to overload the natural slope upon which they are constructed, thus creating instability that involves both the fill and valley slopes. This failure can extend for some distance down-slope, depending on topography and the underlying geology. High groundwater levels and the presence of perched or trapped water can often exacerbate the situation.

Sometimes when valley slopes are exposed to high stresses exerted by fill materials or rock wastes, they may fail. This is especially true with unbalanced cut and fills, where excess material from the cut is dumped onto the slopes below. The accumulation of this material together with any ponding in the rainy season creates conditions that can cause valley slopes to fail.

When rivers erode their side-slopes, an undercut or over-steepened condition occurs, and the process may give rise to slope failure, including the failure of any retaining walls or fill embankments constructed on the slope.

Table D.3.3 gives slope stabilisation and protection options appropriate for fill and valley slopes below a road. As with measures recommended for failures above the road, designs to stabilise and protect slopes below the road are also typically site specific and may require local information from engineers.

Table D.3.3: Stabilisation and protection measures for slope failures below the road (Hunt et al., 2008)

Instability	Stabilisation options	Drainage options	Protection options
1. Erosion of the fill slope surface	▪ None	▪ Ensure road-side drainage is controlled	▪ Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
2. 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 is usually important to prevent surface erosion and increase the resistance of the surface soil.
3. 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 is usually important to prevent surface erosion and increase the resistance of the surface soil.
4. 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 is usually important to prevent surface erosion and increase the resistance of the surface soil.
5. 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

3.4.2 Fill slopes

Slope failures in fill slopes occur when there are changes in slope profile that add driving weight at the top of the fill, or reductions in resisting load at the base. Stability is also reduced as a result of an increase of pore water pressure resulting in a decrease in frictional resistance in cohesion-less soils, or swell in cohesive soils. Water can also cause a progressive decrease in shear strength due to weathering, erosion, leaching, opening of cracks and fissures, and softening and overstressing of the foundation soil. Usually, short-term stability of embankments on soft cohesive soil is more critical than long-term stability, because the foundation soil gains shear strength as the pore pressures dissipate. It may be necessary to check the stability for various pore pressure conditions.

A wide range of slope stabilisation measures is available to address slope stability problems in fill slopes. However, in most cases, the construction of more gentle slopes requires adequate compaction and improved drainage to eliminate routine instability problems.

In general, for LVRs, fill slopes 3m or less in height with 2H:1V or flatter side slopes, may be designed based on the strength characteristics of a compacted free draining granular fill and engineering judgment. This is provided there are no known problem soils such as expansive or collapsible soils, organic deposits, of soft and loose sediments.

Fill slopes over 3m in height or any embankment on soft soils, in unstable areas, or those comprised of light weight fill require site-specific engineering geological assessment depending on specific ground conditions. Embankments with side slope inclinations steeper than 2H:1V also require special attention. Moreover, any fill placed near or against a bridge abutment or foundation, or that can impact on a nearby structure, will likewise require stability analysis by a specialist engineer.

When deciding on the fill slope batters for design, it is advisable to consider the type of material that is going to be used. For example, over-steep fill slopes formed in loose side-cast materials may continue to ravel with time. Similarly, flat fill slopes formed by soft materials may be exposed to flooding and settlement. Design should also consider the stability of existing fill slopes in the surroundings of the project site. Observation of existing slopes should include vegetation, in particular the types of plants that may indicate wet soil. Subsurface drainage characteristics may be indicated by the vegetative pattern. It is also advisable to assess whether tree roots are providing anchoring of the different types of soils.

Generally, fill slope batters commonly used in different materials are summarized in Table D.3.4. Ideally, fill slopes should be constructed with a 2H:1V or flatter slopes to promote vegetation growth. A suitable combination of plants can help to stabilise fill slopes that are prone to very shallow translational slides. Failures deeper than 1m would be out of the zone of influence of most plant rooting systems. Besides, terraces, benches or bunds are desirable on large fill slopes to break up the flow of surface water.

Table D.3.4: Common fill slope batters for LVRs

Soil and rock condition	(Horizontal : Vertical)
Soft clays or fills on wet areas and swamps (>3m)	2:1 - 3:1
Fills of most soils (free draining granular fill >3m)	3: 2 - 2:1
Fills of hard, angular rock (rock fill >3m)	3:2 – 5:4
Low fills (less than 3m high)	2:1 or flatter (for vegetation)

In addition to the selection of appropriate fill slope batters and the use of vegetation, the construction of toe berms can also improve the stability of fill slopes by increasing the resistance along potential failure planes. Toe berms are typically constructed of granular materials with relatively high shear strength that can be placed quickly with minimum compaction. As the name indicates, toe berms are constructed near the toe of slopes where stability is a concern. The sides of toe berms are often flatter than the slopes.

In general, toe berms increase the shearing resistance by:

- Adding weight and thus increasing the shear resistance of granular soils below the toe area of the fill slope;
- Adding high strength material for additional resistance along potential failure surfaces that pass through the toe berm;
- Creating a longer failure surface, thus more shear resistance, as the failure surface must pass below the toe berm.

It is generally preferable to avoid the construction of fill slopes in swampy areas. When this cannot be avoided, fill slopes through swampy areas must be constructed on a stable foundation. This helps reduce problems which may occur within a short time after construction. The manner in which this is accomplished, and the problems to overcome, depends largely on the type and depth of materials that exist in the swamp.

In upgrading and rehabilitation projects, it may be more difficult to determine accurately the depth of failure surface in fill slopes. Careful site inspection is therefore required. Failed fill material should be excavated and stockpiled while the ground beneath is prepared. Organic, weathered and weak materials should be removed, benched profiles created for a shear key, etc.) The fill should then be replaced and compacted in layers until the final slope profile is achieved. Planting and drainage may be necessary.

During operation and maintenance, should embankments fail, they can either be:

- Excavated and re-compacted if traffic flow and safety allows;
- Excavated and replaced with granular materials if traffic flow and safety allows;
- Supported by retaining walls where foundation stability and allowable bearing pressures permit;
- Isolated from the road by construction of a road edge retaining wall founded beneath the zone of movement.

3.5 Drainage

Slope instability is controlled to a large extent by water. Rainfall infiltrating through the ground surface can concentrate within the slope. Excess water may decrease pore suction in the underlying soil and create pore water pressure, reducing the effective stress and hence the stability of the slope. Hence, the construction of surface and sub-surface drainage structures is vital to ensure that this excess water can be intercepted and conveyed to a safe location where it will not create further instability problems. If water flow in the slope cannot be controlled properly, then retaining structures may be ineffective.

Several surface drainage systems can be constructed on cuts, fills and natural slopes depending on slope geometry and ground materials. The engineering design of surface drainage systems on the slope is based on the amount of run-off. The runoff is a function of the catchment area, concentration time, rainfall intensity, slope geometry, and surface conditions.

The types of surface drainage systems that can be used in low volume roads include open ditches (rectangular, U-shaped, trapezoidal and semi-circular) and gravel filled trench drains. Trench drains are in almost all cases rectangular. Open ditches should collect runoff from the catchment area and convey water as efficiently as possible. Abrupt changes in flow direction may cause splashing, turbulence, and erosion of ditch walls. Attempts should be made to avoid this. Wherever the flow velocity is high, energy dissipators should be placed inside the channel.

On most slopes, it is common to construct down-slope channels with steps (stepped channels or cascades). Like slope ditches, these stepped channels can be constructed in different shapes. Although these channels are divided into different steps, they should be constructed as a single unit. Otherwise, when sections are built independently, the flow kinetic energy can cause openings between sections, which may result in water infiltration and differential settlements. It is worthwhile to note that the steps may not provide an efficient energy dissipator when the volume of discharge is high. Hence, stepped channels should not be built on slopes steeper than 50o because shooting (high-speed) flow will occur whereby individual steps are bypassed. Where these drainage structures are constructed in slopes steeper than 50%, energy dissipators such as up-stands should be introduced.

Cut-off drains intercept surface and shallow subsurface flow and are normally placed at the crest of the slope. They are internally filled with granular materials, perforated pipes, wrapped in geosynthetic materials for drainage and filtering purposes. The draining material must collect and conduct the water and have adequate permeability to allow flow to take place along the line of the drain rather than across it. Free-draining inert material is used as backfill. Filter fabric is designed to allow water to penetrate, but prevent fines from washing through. It must also be designed to prevent clogging.

A cut-off drain can be excavated with a minimum width of 0.5m at the bottom and 0.6m at the top. Its depth normally varies between 1.0 and 1.5m. For deeper excavation, care should be taken with the stability of walls. The choice of filling material and pipe perforations must satisfy filter design criteria. When small quantities of seepage are to be removed, a single layer of well-graded moderately permeable material meeting grain-size requirements may serve the dual purpose of filter and drain. But when large quantities of seepage are to be removed, a filter layer is usually needed for the prevention of piping, and a coarse layer for the removal of water. In areas where erosion is high, a graded filter material at the top and an impermeable liner at the base could be needed. The rest of the drain is filled with free-draining granular material. A slotted pipe at the base facilitates drainage, and it may need to be filter-wrapped.

In general, the use of surface drainage structures together with bio-engineering techniques and retaining walls may suffice to reduce the occurrence of erosion and landslides on LVRs. Where there is an indication that slope failure is partially or entirely due to groundwater, then the use of subsurface (sub-soil) drainage systems such as deep cut-off and horizontal drains may be considered. Refer to the ERA Geotechnical Design Manual for explanations on sub-surface drainage systems and their use in slope stabilisation.

3.6 Retaining walls

3.6.1 Design of Retaining walls

Retaining walls are used to resist the lateral pressure of soil where there is a desired change in ground elevation that exceeds the angle of repose of the soil. This normally occurs when it is necessary to gain roadway space without making large cuts into the hillside or constructing wide fills below the road.

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 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 weight 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.

This manual covers the design and construction of gravity retaining walls, including gabion walls, dry stone walls and mortared stone walls. These options are described below. Construction details are given in Part E of the manual.

3.6.2 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.

There are two types of gabions with different uses:

- Gabion boxes are the heavier, more rigid form, with larger stones used in bank protection, aprons, and retaining walls. Usually 1.0m high boxes are used for these purposes but sometimes 500mm boxes may be used for rigid aprons.
- Gabion mattresses are thinner using smaller stones and mesh and, therefore, more flexible so that they will fold down to protect scour holes. The maximum depth of the box for this purpose is often 300mm.

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 6m 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 road support retaining walls 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.

3.6.3 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 5m in height.

3.6.4 Mortared masonry walls

As with gabion walls and dry stone walls, a mortared masonry wall design uses its own weight 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 75mm diameter and placed at 1.5m centres with a slope of 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 permit free drainage of water.

3.7 Bio-engineering methods

In the design of LVRs, both cut and fill slopes should be constructed so that they can be vegetated. Hill and valley slopes should be protected using plants. Certain types of plants, arranged in particular configurations, can be used to control erosion and reduce the likelihood of shallow landslides. Vegetation is unlikely to have a significant impact on slope stability where slip planes are deep, due to the shallow rooting nature of many species. However, the condition of the upper layer, stabilised by plant roots, may be sufficient to maintain the stability of the slope.

Different types of plants and planting materials give rise to a variety of rooting patterns, with the result that the surface layer of soil will be bound together and have its resistance to deformation, failure and landslides increased.

Grasses are very quick growing plants that offer a dense protective ground cover. Since their stem is at the ground level, moderate harm to the area by humans or domestic animals does not cause lasting damage to them. In many cases, grass re-grows quickly if damaged. Grasses with their dense network of shallow roots are useful in protecting sites from surface erosion. However, some species such as vetiver have very deep root systems and can offer greater potential for shallow slope stabilisation.

Herbs have little or no woody tissue. They tend to grow closer to the ground providing a dense ground cover with shallow root systems. Together with other plants such as grasses and shrubs, they can protect the slope from erosion.

Woody plants and shrubs are other types of vegetation that can increase stability in slopes. A woody plant has a perennial woody stem and supports vegetative growth. Shrubs are defined as low-growing woody plants with multiple stems. Shrubs can vary in height from 0.2m to 6m. In areas where visibility is essential, such as road curves, shrubs are preferred to trees as they do not grow as large and are easier to control and maintain. Although root systems may not spread as deep and as far as tree root systems, the tensile strength developed in soils as a result of the presence of shrubs is normally sufficient to hold soil materials together.

Trees are perennial woody plants having a main stem and root. Tree roots can extend to considerable depth and over a large area if sub-soil stratigraphy permits. Therefore, trees are often considered suitable for reinforcing slopes with deep soil profiles. Trees have been classified as having three main root system types: plate, heart and tap. Plate root systems have large lateral roots and vertical sinkers, heart systems possess many horizontal, oblique and vertical roots, and tap systems have one large central root and smaller lateral roots. Trees having heart and tap root systems have been classified as being the most resistant to uprooting. Individual roots within a system may be further classified into sub-groups depending on their morphology and function. Extensive roots are those which grow to large depths and spread over large areas, while intensive roots are short, fine roots and tend to be much more localised.

3.7.1 Selection of appropriate plant species

The main factors to be addressed when selecting the particular species for use in bio-engineering works are as summarized as (Hunt et al, 2008):

- The plant must be of the right type to undertake the bio-engineering technique that is required. The possible categories include:
  - A grass that forms large clumps,
  - A shrub or small tree that can be grown from woody cuttings,
  - A shrub or small tree that can grow from seed in rocky sites,
  - A tree that can be grown from a potted seedling,
  - A large bamboo that forms clumps.
- The plant must be capable of growing in the location of the site.

Other points to be considered are:

- There is no single species or technique that can resolve all slope protection problems. It is always advisable to use local species which don't invade and harm the environment, and were able to protect the slope from sliding in the past.
- Large trees are suitable on slopes of less than 3H:2V or in the bottom 2m of slopes steeper than 3H:2V. Maintaining a line of large trees at the base of a slope can help to buttress the slope and reduce undercutting by streams.
- Grasses that form dense clumps generally provide robust slope protection in areas where rainfall is intense. They are usually best for erosion control, although most grasses cannot grow under the shade of a tree canopy.
- Shrubs (ie woody plants with multiple stems) can often grow from cuttings taken from their branches. Plants propagated by this method tend to produce a mass of fine, strong roots. These are often better for soil reinforcement than the natural rooting systems developed from a seedling of the same plant.
- In most cases the establishment of full vegetation cover on unconsolidated fill slopes may take one to two rainy seasons. Likewise, the establishment of full vegetation on undisturbed cut slopes in residual soils and colluvial deposits may need 3 to 5 rainy periods. Less stony and more permeable soils have faster plant growth rates, and drier locations have slower rates.
- Plant roots cannot be expected to contribute to soil reinforcement below a depth of 500mm. Plants cannot be expected to reduce soil moisture significantly at critical periods of intense and prolonged rainfall.
- Grazing by domestic animals can destroy plants if it occurs before they are properly grown. Once established, plants are flexible and robust. They can recover from significant levels of damage (eg flooding and debris deposition).

3.7.2 Site preparation

Before bio-engineering treatments are applied, the site must be properly prepared. The surface should be clean and firm, with no loose debris. It must be trimmed to a smooth profile, with no vertical or overhanging areas. The object of trimming is to create a semi-stable slope with an even surface, as a suitable foundation for subsequent works.

- Trim soil and debris slopes to the final desired profile, with a slope angle of between 30° and 60° (in certain cases the angle will be steeper, but review this carefully in each case). Trim off excessively steep sections of slope, whether at the top or bottom. In particular, avoid slopes with an over-steep lower section, since a small failure at the toe can destabilise the whole slope above.
- Remove all small protrusions and unstable large rocks. Eradicate indentations that make the surrounding material unstable by trimming back the whole slope around them. If removing indentations would cause an unacceptably large amount of work, excavate them carefully and build a buttress wall. Remove all debris from the slope surface and toe to an approved tipping site. If there is no toe wall, the entire finished slope must consist of undisturbed material.
- When materials form the lower parts of slopes to be trimmed, the debris can be used for backfilling. In this case, compact the material in layers, by ramming it thoroughly with tamping irons. This must be done while the material is moist.

3.7.3 Recommended techniques

Table D.3.5 provides the different types of bio-engineering techniques recommended for various kinds of slopes and soil materials above the road structure. Similarly, the techniques useful for slopes below the road are summarized in Table D.3.6. The types of plants commonly used for erosion control at any site in the surroundings of the right-of-way of the road are given in Table D.3.7. Many of these techniques, as they are compiled by Hunt et al (2008), have been used successfully in many countries notably in Asia.

In Ethiopia, different regions use different types of plants for erosion control and slope stabilisation. These plants, among many others, include Chrysopogon zizanioides (Vetiver grass), Agave sisalana, (Sisal, Kacha), Aloe vera (Eret), Euclea schimperi (Dedeho), Calpurnia subdecandra (Laburnum, Digita), Juniperus procera (Cedar, Tid), Hyparrhenia rufa (Senbelet), and Oxytenanthera abyssinica (Lowland Bamboo).

Table D.3.5: Recommended bio-engineering techniques for slopes above the road.

Site characteristics	Recommended techniques
Cut slope in soil, very highly to completely weathered rock or residual soil, at any grade up to 1H:2V	Grass planting in lines, using slip cuttings.
Cut slope in colluvial debris, at any grade up to 1H:1V (steeper than this would need a retaining structure)	
Trimmed landslide head scarps in soil, at any grade up to 1H:2V	
Roadside lower edge or shoulder in soil or mixed debris	
Cut slope in mixed soil and rock or highly weathered rock, at any grade up to about 1H:4V	Direct seeding of shrubs and trees in crevices.
Trimmed landslide head scarps in mixed soil and rock or highly weathered rock, at any grade up to about 1H:4V	

Chrysopogon zizanioides (Vetiver grass) is a perennial grass with short rhizomes and massive, finely structured root system that grows very quickly in some places. Its root depth reaches 3-4 m in the first year. The deep root makes vetiver grass extremely drought tolerant and very difficult to dislodge when exposed to a strong water flow and subsequent erosion. It is also highly resistant to pest, disease and fire.

According to Mekonnen (2000), vetiver grass was first introduced to Ethiopia from India at the start of the 1970s by the State Coffee sector and the Ethiopian Institute of Agricultural Research. Since then, the grass is increasingly used for soil and water conservation and stabilisation of steep slopes on different slope classes. The upper slope limit lies between 40% and 45%. The vertical intervals between units vary from one slope class to the other. The vertical classes recommended in the country for slope classes between 3-15%, 16-25%, and greater than 25% are 2m, 1.5m and 1m, respectively. For very steep slopes, it is advisable to plant grass units closer right after the rainy season. The effective elevation limit for its use is about 2,800 m.

Agave sisalana (Sisal, locally called Kacha) is a plant that yields a stiff fibre used in some countries to make twine, rope and dartboards. Depending on context, the term may refer either to the plant or the fibre. Sisal plants consist of a rosette of sword-shaped leaves about 1.5m to 2m tall. They are common in the northern, western and eastern part of Ethiopia. In the east, sisal resists the semi-arid and arid climate where the average rainfall is around 500mm per year and grows on rocky slopes. In Tigray, sisal is used on road cuts and natural slopes to control erosion and landslides. Sisal is valued for making geotextiles because of its strength, durability, and ability to stretch. These textile fabrics are used in geotechnical engineering for soil erosion control. Biodegradable products create a stable temporary slope environment in which plants can develop and then break down to add nutrients to the soil.

Table D.3.6: Bio-engineering methods useful to improve slope stability below the road

Site characteristics	Recommended techniques
Fill slopes and backfill above walls without a water seepage or drainage problem; these should first be re-graded to be no steeper than 3H:2V	Brush layers (live cuttings of plants laid into shallow trenches with the tops protruding) using woody cuttings from shrubs or trees
Debris slopes underlain by rock structure, so that the slope grade remains between 1H:1V and 4H:7V	Palisades (the placing of woody cuttings in a line across a slope to form a barrier) from shrubs or trees
Other debris-covered slopes where cleaning is not practical, at grades between 3H:2V and 1H:1V	Brush layers using woody cuttings from shrubs or trees
Fill slopes and backfill above walls showing evidence of regular water seepage or poor drainage; these should first be re-graded to be no steeper than about 3H:2V	Fascines (bundles of branches laid along shallow trenches and buried completely) using woody cuttings from shrubs or trees, configured to contribute to slope drainage
Large and less stable fill slopes more than 10m from the road edge (grade not necessarily important, but likely eventually to settle naturally at about 3H:2V)	Truncheon cuttings (big woody cuttings from trees)
The base of fill and debris slopes	Large bamboo planting; or tree planting using seedlings from a nursery

Like sisal, *Aloe vera* (Eret) is commonly found in warm, fertile regions of Ethiopia where it is capable of withstanding very long periods of drought. There are also many species of Aloe vera in the dry parts of Ethiopia that are commonly used for erosion control. The plant takes four to five years to reach maturity, at which time its leaves, which grow from a short stem, are about 60cm in length and 8-10cm wide at the base. Being perennial in behaviour, Aloe vera has a lifespan of about 12 years, and if planted early, it may protect cut and fill slopes throughout most of the design life of the road.

Euclea schimperi (Dedeho) is a widespread Afromontane shrub or small tree common in overgrazed rolling hills or fallow land. It is seldom consumed by domestic animals and can be used to stabilise cuts and fills as its roots can hold soil particles together. Similarly, Calpurnia subdecandra (Laburnum, Digita), and Juniperus procera (Cedar, Tid) are also traditionally used for protection of slopes from erosion. Besides, hill slopes and the toe of road cuts are covered and stabilised by Hyparrhenia rufa (Senbelet).

Oxytenanthera abyssinica (Lowland Bamboo) is a clump forming, solid stemmed plant that is widely distributed in the dry regions of the western part of Ethiopia. Traditionally, it has been used as a raw material for building and making numerous household utensils, basketry, and handicrafts. Bamboo is very drought resistant, sustains itself with minimal rainfall, and has a very economical water uptake. It thrives in very poor, shallow soils unsuitable for most trees. This characteristics and its capability to grow in steep areas makes it ideal to stabilise slopes. Besides, its ability to resist high tensile stresses makes the stem useful for building retaining walls.

Table D.3.7: Plants useful for erosion control in the surroundings of roads

Site characteristics	Recommended techniques
Stream banks where minor erosion is possible	Local plants including grasses, shrubs and bushes, bamboos, etc
Gullies or seasonal stream channels with occasional minor discharge	Live check dams using woody cuttings of shrubs and trees
Gullies or seasonal stream channels with regular or heavy discharge	Stone pitching, probably vegetated. Gabion check dams may also be required
Other bare areas, such as on the land above landslide head scars, on large debris heaps and stable fill slopes	Tree planting using potted seedlings from a nursery

When reinforcing soils against shallow slope instability, the most important criteria to consider are the number, diameter, and tensile resistance of roots crossing the slip surface. Therefore, root systems composed of deep taproots and sinker roots are ideal for this purpose. In contrast, as root tensile strength is greater in thin woody roots, a large number of small diameter roots would provide a root-soil matrix that resisted shear better. Vetiver grass is often used for vegetating shallow slope failures, due to its relatively deep and fibrous root system. It is advisable to consult local agricultural experts to select plants appropriate for different bio-engineering purposes.

At the top or toe of the slope, it is also be necessary to have roots that provide lateral reinforcement. Thus, the ideal root morphology in shrubs and trees would be a heart root system, with deep sinkers and wide-spreading lateral roots. In planar slides, tap-rooted species could be planted as the slip surface would most likely be parallel to the slope. In many embankments and cuts, the slip surface is circular and the root network should have sufficient depth to interact with the slip surface.

In active rock-fall corridors, mechanical properties of stem are more useful than root system morphology for determining tree resistance. Nevertheless, if trees are well-anchored with a deep taproot, they will less likely to uproot when hit by a failing rock.

3.8 Useful dos and don'ts

The following advice is given when considering slope stability problems and solutions for low volume roads:

- Always determine the cause and extent of the slope instability or erosion problem;
- Prioritise each alignment and alignment section in order to identify critical areas;
- Regularly inspect side slopes, culvert outlets and side ditches or drains to identify potential problems;
- Identify areas where land use activities are adversely affecting engineering performance and take steps to rectify the situation;
- Identify areas where engineering works are adversely affecting land use;
- When scheduling retaining structures in unstable areas try to locate rock or firm ground for foundation purposes;
- Ensure adequate bearing capacity for all structures using in situ tests (eg DCP);
- When building gabion (or masonry) structures in stream channels or on colluvial deposits, ensure adequate foundation beneath potential scour depths or loose stratum and key structure adequately into channels banks or side slopes;
- Drainage control is a major factor in the maintenance of natural and man-made slopes. Drainage must be diverted away from vulnerable areas and every attempt made to slow down but not impede or pond drainage;
- Select appropriate areas for spoil disposal.

3.9 References:

Hunt T., Hearn G., Chonephetsarath X., and Howell J. (2008). Slope maintenance manual. Ministry of Public Works and Transport, Roads Administration Division, Laos.

Mekonen A., 2000. Erosion control in agricultural areas: an Ethiopian perspective, IFSPEthiopia. Proceedings 2nd International Vetiver Conference (ICV), Thailand.

4. GEOMETRIC DESIGN

4.1 Introduction

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. The geometric standards are intended to meet two important objectives namely to provide minimum levels of safety and comfort for drivers by provision of adequate sight distances, coefficients of friction and road space for manoeuvres; and to minimise earthworks to reduce construction costs.

Geometric design covers road width; cross-fall; horizontal and vertical alignments and sight lines; and the transverse profile or cross-section. The cross-sectional profile includes the design of the side drainage ditches, embankment heights and side slopes, and is a vital part of geometric design for low volume roads. The cross-section essentially adapts the pavement to the road environment and is part of the drainage design. For example, wide, sealed shoulders and high camber or cross-fall can significantly improve the operating environment for the pavement layers by minimising the ingress of surface water. Sub-surface water is a problem in low-lying flood-prone areas and whenever the road is in cut. Again, the height of embankment and the depth and type of drainage ditch have very significant effects. Some of these aspects are dealt with in the Drainage Chapter (Chapter 5).

4.1.1 Principal factors affecting geometric standards

The principal factors that affect the appropriate geometric design of a road are:
- Cost and level of service;
- Terrain;
- Safety;
- Pavement type;
- Traffic volume and composition;
- Roadside population (open country or populated areas);
- Soil type;
- Climate;
- Construction technology; and
- Administrative or functional classification.

The cost of a road is usually the most critical factor. It is also the most difficult to include in the setting of the design standards. The standard of a road is essentially an index of its 'service level' but 'service level' is a rather imprecise term that means different things to different people. However, most would agree that its main components include; speed of travel, safety, comfort, ease of driving, stopping and parking, and reliable trafficability or passability. The chosen service level is directly associated with traffic volume and, hence, is not treated as a separate variable. The standards for service level simply increase from the lowest road class to the highest, remaining relatively constant within each class. (see Table A.3.1 in Part A)

Since these factors differ for every road, the geometric design of every road could, in principle, be different. This is impractical and it is therefore normal practice to identify the main factors and to design a fixed number of geometric standards to cope with the range of values of these key factors.

For LVRs in Ethiopia four basic standards are defined based on traffic levels (see Part B - Chapter 3). These are then modified, sometimes quite considerably, to cater for the other key factors. The most important of these are:
- Terrain;
- Traffic composition (including pedestrians and non-motorised vehicles);
- Roadside land use – activities and population density;
- Safety;
- Pavement type (paved or unpaved).

Varying standards of geometric design do not exist to cater specifically for climate and soil type. However, these factors are taken into account in the design of the drainage features of the road (Chapter D.5) and affect the road cross-section thereby contributing to the geometric design.

The designer, therefore, has a very wide range of standards from which to choose, ensuring that a suitable standard is available for almost all situations. However, there will be cases where it is impossible to meet any of the standards, often because of extremely severe terrain conditions. Under such circumstances the standards must be relaxed and road users must be warned of the reduction with suitable and permanent signage.

4.1.2 How the Standards are used

A national 'standard' is not a specification, although it could, and often is, incorporated into specifications and contract documents. Rather, a standard is a specific level of quality that should be achieved at all times and nationwide. Amongst other things this ensures consistency across the country. For the geometric standards, this means that road users know exactly what to expect. Drivers, for example, are not 'surprised' by unexpected changes in quality. Thus they will not unexpectedly find that a road is too narrow, or that they have to alter their speed drastically to avoid losing control of their vehicle. Thus standards are a guarantee of a particular quality level and, for roads, this is vital for reasons of safety.

It is important to note that there is no reason why a higher standard than the standard appropriate to the traffic and conditions should not be used in specific circumstances. For example, for reasons of national and international prestige or for strategic or military reasons, a road may be built to a higher standard than would normally be justified eg a road to an international sports facility (where the traffic is low for most of the time but can be quite high for short periods), the road to an airport, and roads to military establishments. Thus higher standards can be used if required but lower standards should not be used except in exceptional circumstances, for example, in particularly difficult terrain.

Figure D.4.1 shows how the appropriate geometric standards are selected.

Step 1: The first step is to determine the basic traffic level because this defines the road class (see Section 4.2.1 and 4.2.3). At this point, the proportion of heavy vehicles in the traffic stream is also determined. This step is not specific to the geometric design and will usually have been done by the time it is necessary to determine the geometric characteristics of the road. However, more details of the traffic are required for the geometric design in terms of the other road users such as pedestrians, bicycles, motor cycles, motor cycle taxis and animal drawn vehicles. These are taken into account in Step 5.

Step 2: The numbers and characteristics of all the other road users are considered (see Section 4.2.4). It is here that the road layout may be altered and additional widths provided for safety and to improve serviceability for all road users (eg reduce congestion caused by slow moving vehicles).

Step 3: The terrain class; flat, rolling, mountainous and escarpment is determined (see Section 4.2.5).

Step 4: The 'size' of the villages through which the road passes is evaluated to determine whether they are large enough to require parking areas and areas for traders (see Section 4.2.6).

Step 5: For most road classes there are options for road type and therefore the next step is to decide which type will be built (Section 4.2.7) In many cases the adoption of an EOD policy will mean that different parts of the road may be designed with a different surfacing. The choice of road type is described in Chapter D.6.

Step 6: From the available data the widths of carriageway and shoulders should be determined (see Section 4.3.7). At this stage additional factors that affect the geometric standards are also considered such as additional road safety features and the construction technology to be employed.

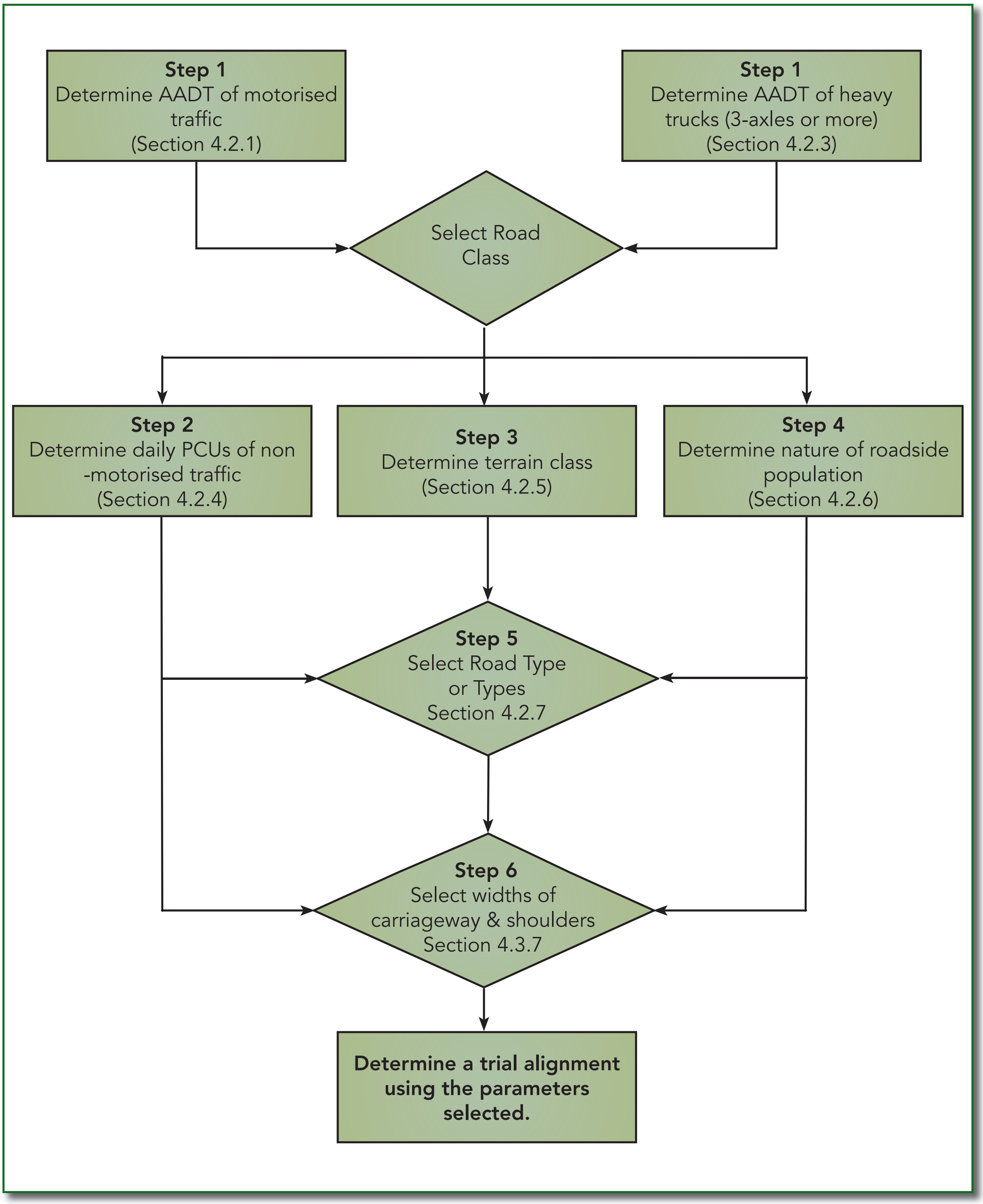


Figure D.4.1: Selection procedure for appropriate geometric standards

Once the basic parameters have been determined, the appropriate Table from Tables B.3.11 to Table B.3.17 is selected. This provides details of the other geometric factors that are needed to carry out the geometric design.

The completion of the process is the design of a trial alignment as a check to ensure that all the standards have been met. If not, alternative alignments should be tried. In extreme conditions it may not be possible to adhere to all the standards at all points along the road. In such cases engineering judgement or additional technical advice may be needed.

The pre-feasibility study should have shown that the costs of the road are likely to be acceptable. However, at this stage it may be found that the engineering problems are more costly to solve than anticipated. This needs to be checked and a final alignment selected. If the costs are too high then the project will need to be reviewed.

4.2 The principal factors determining the choice of geometric standard

4.2.1 Traffic volume

Roads are designed to provide good service for many years and therefore the traffic level to be used in the design process must take into account traffic growth. Designing for the current traffic will invariably lead to inadequate standards in the future unless the traffic growth rate is extremely low. To deal with these uncertainties it is generally expected that there is a strong correlation between traffic level, traffic growth rates and the functional classification of a road and therefore such a classification is often seen as a suitable alternative to represent traffic.

However, although traffic levels often increase in line with the functional classification, this is not always true and, furthermore, the traffic levels and growth rates are likely to differ considerably between different areas and different regions of the country. For example, the traffic on a 'collector' road in one area of the country might be considerably more than on a 'main access' road in another area. The design of the road, and therefore the standards adopted, should reflect the traffic level. In addition, traffic growth rates are often expected to be considerably higher on roads connecting district (wereda) centres than on roads connecting villages but this is not always the case.

In general it is expected that growth rates on roads that do not have 'through' traffic (essentially feeder roads) will have lower traffic growth rates than the higher classes of road but each situation should be treated on its own merits taking into account any expected future developments.

For geometric design it is the daily traffic that is important. The approach recommended for estimating the traffic for geometric design purposes is based on the estimated traffic level at the middle of the design life period and this therefore requires an estimate of the traffic growth rate This method eliminates the risk of under-design that may occur if the initial traffic is used and the risk of over-design if traffic at the end of the design life is used. A design life of 15 years is recommended for paved roads and 10 years for unpaved roads.

Normally a general growth rate is assumed or is provided by government based on the growth in registered vehicles during previous years. However, local development plans may indicate higher growth rates in some places.

Where there is no existing road, estimating the initial traffic is difficult and estimating future traffic even more so. However, in many cases where a new road is proposed there is likely to be pedestrian traffic and therefore some information on the likely vehicular traffic after the road is constructed. In some cases an economic evaluation may have been carried out to justify the road in the first place. This will have provided an estimate of the amount of goods transported by pedestrians and the likely amount that will be carried by vehicles. In the unlikely event that there is no information available, the lowest class of engineered road (DC1) should be designed. Historical growth rates of similar roads in any specific area should be considered if available.

It should be noted that the issue of road classification to determine the standards to be applied is not difficult. A maximum of four different standards are defined for LVRs (DC1-DC4 – Tables B.3.1 and B.3.2) and each will be applicable over a specific traffic range. These ranges are therefore quite wide and little difficulty should normally be experienced in assigning a suitable standard to a new road project. Where the expected traffic is near to a traffic boundary, it would be prudent to use a higher classification.

4.2.2 The design vehicle

For geometric design it is the physical dimensions of a vehicle that are also important. A truck requires more space than a motorcycle, for example, and this does not depend on whether the truck is empty or fully loaded.

The way that vehicle size influences the geometric design of low and high volume roads is fundamentally different. When the volume of traffic is high, the road space occupied by different types of vehicle is an essential element in designing for capacity (ie the number of vehicles that the road can carry in a unit of time -vehicles per hour or per day). For example, at the highest traffic levels, when congestion becomes important, traffic volume dictates how many traffic lanes need to be provided.

For LVRs the volume of traffic is sufficiently low that congestion issues do not arise from traffic volume but from the disparity in speed between the variety of vehicles and other road users which the road serves. In other words the traffic composition is the key factor; traffic capacity is not the problem. Nevertheless it is the size of the largest vehicles that use the road that dictates many aspects of geometric design. Such vehicles must be able to pass each other safely and to negotiate all aspects of the horizontal and vertical alignment. Trucks of different sizes are usually used for different standards – the driver of a large 5 or 6-axle truck would not expect to be able to drive through roads of the lowest standards.

In some countries the truck population in rural areas is predominantly one or two types and sizes of vehicle. This makes it relatively easy to select a typical vehicle for setting geometric standards. Conversely, some countries have a wide variety of truck sizes and selecting a suitable truck size for geometric design is more difficult.

Good information on the vehicle fleet in Ethiopia is lacking but, in view of the low density of roads and, hence, lack of alternative routes, together with the limited choice of vehicle for many transporters, it is prudent to be conservative in choosing the design vehicle for each class of road so that the maximum number of vehicle types can use them. In Ethiopia four different design vehicles have been used as shown in Table D.4.1 and Table D.4.2. However there is very little difference between design vehicles DV2 and DV3. Roads designed for the single unit truck will be suitable for the bus provided the front and rear overhangs of the bus are taken into account when designing curves; and this can be done with suitable curve widening where required as described later. The standard for only the lowest class of road is insufficient for DV2 and DV3.

Diagrams showing the full minimum swept out path of the design vehicle are shown in ERA's Geometric Design Manual (2011).

Table D.4.1: Design vehicle characteristics

Design vehicle	Designation	Height (m)	Width (m)	Length (m)	Front overhang (m)	Rear overhang (m)	Wheelbase (m)	Minimum turning radius (m)
4x4 Utility	DV1	1.3	2.1	5.8	0.9	1.5	3.4	7.3
Single unit truck	DV2	4.1	2.6	11.0	1.5	3.0	6.5	12.8
Single unit bus	DV3	4.1	2.6	12.1	2.1	2.4	7.6	12.8
Truck and semi-trailer	DV4	4.1	2.6	15.2	1.2	1.8	4.8+8.4 =13.2	13.7

Table D.4.2: Design vehicle for each LVR class

Design standard	Design vehicle
DC4	DV4
DC3	DV3
DC2	DV3
DC1	DV1

4.2.3 Traffic composition – proportion of heavy vehicles

The density of roads in Ethiopia is quite low and one of the consequences of this is that the proportion of heavy vehicles in the traffic stream on LVRs is often quite high. Design standards DC2, DC3 and DC4 include a modification to cater for this.

For DC4, if the number of 'large' vehicles, defined as 3-axled (or more) trucks with GVWs (Gross Vehicle Weights) potentially greater than 12 tonnes, is greater than 40 the width of the paved surface is increased to 7.0m and the shoulders reduced to 1.0m. If there are more than 80 large vehicles then the standard for DC5 (as defined in the Geometric Design Manual – 2011 series) should be used instead of DC4.

For DC3, if the number of large vehicles is greater than 25, design standard DC4 should be used and, for DC2, if the number of large vehicles exceeds 10 then DC3 should be used.

4.2.4 Traffic composition - use of Passenger Car Units (PCUs)

In order to quantify traffic for normal *capacity* design the concept of equivalent PCUs is often used. Thus a typical 3-axle truck requires about 2.5 times as much road space as a typical car hence it is equivalent to 2.5 PCUs. A motor cycle requires less than half the space of a car and is therefore equivalent to 0.4 PCU's.

The PCU concept is very useful where traffic congestion is likely to be a problem and it was not originally intended for use in the geometric design of LVRs. However, vehicles that move slowly cause congestion problems because of their speed rather than because of their size. In effect, they can be considered to occupy more road space than would be expected from their size alone. The actual PCU rating of a vehicle is affected by the function of a road (i.e. the nature of the other traffic) and varies as the traffic mix varies and as the traffic volume and traffic speeds vary. Nevertheless, in situations where the number of slow moving vehicles, both motorised and non-motorised, is significant, in order to retain the level of service appropriate to the traffic level of motorised vehicles, the road standard should be improved by reducing congestion and this is best done by widening the shoulders. Thus when the PCU level *of the slow moving and intermediate forms of transport* reaches a certain level, shoulder widening is justified.

The PCU concept is also useful for identifying the need for additional safety features where the numbers of pedestrians and slow moving vehicles are high.

The PCU values for Ethiopia are shown Table D.4.3. Motorcycle taxis (eg bajaj) are becoming popular in urban situations and it is only a matter of time before these spread to more rural areas and become adapted for freight as well as for passenger transport.

Table D.4.3: PCU values

Vehicle	PCU value
Pedestrian	0.15
Bicycle	0.2
Motor cycle	0.25
Bicycle with trailer	0.35
Motor cycle taxi (bajaj)	0.4
Motor cycle with trailer	0.45
Animal drawn cart	0.7
Bullock cart	2.0
All based on a passenger car = 1.0	

4.2.5 Terrain

Terrain has the greatest effect on road costs therefore it is not economical to apply the same standards in all terrains. Fortunately drivers of vehicles are familiar with this and lower standards are expected in hilly and mountainous terrain.

Four categories have been defined which apply to all roads as follows:

Flat 0-10 five-metre contours per km. The natural ground slopes perpendicular to the ground contours are generally below 3%.

Rolling 11-25 five-metre contours per km. The natural ground slopes perpendicular to the ground contours are generally between 3 and 25%.

Mountainous 26-50 five-metre contours per km. The natural ground slopes perpendicular to the ground contours are generally above 25%.

Escarpment Escarpments are geological features that require special geometric standards because of the engineering problems involved. Typical gradients are greater than those encountered in mountainous terrain.

The following information is particularly relevant to LVR design:

Hilly terrain

An important aspect of geometric design concerns the ability of vehicles to ascend steep hills. Roads that need to be designed for very heavy vehicles or for animal drawn carts require specific standards to address this, for example, special climbing lanes. Fortunately the technology of trucks has improved greatly over the years and, provided they are not grossly overloaded (which is a separate problem) or poorly maintained, they do not usually require special treatment. On the other hand, animal drawn vehicles are unable to ascend relatively low gradients and catering for them in hilly and mountainous terrain is rarely possible. Climbing lanes cannot be justified on LVRs and nor can the provision of very low maximum gradients. The maximum gradients allowable for different road classes are shown in Tables B.3.11 to Table B.3.17.

Mountainous and escarpment terrain
In mountain areas the geometric standard for LVRs takes account of the constraints imposed by the difficulty and stability of the terrain. This design standard may need to be reduced locally in order to cope with exceptionally difficult terrain conditions. Every effort should be made to design the road so that the maximum gradient does not exceed the standards shown in Tables B.3.11 to Table B.3.17 but where higher gradients cannot be avoided, they should be restricted in length. Gradients greater than 12% should not be longer than 250m and relief gradients are also required as indicated in the Tables. Horizontal curve radii of as little as 13m may be unavoidable, even though a minimum of 15m is specified.

4.2.6 Roadside population (open country or populated areas)

The more populated areas in village centres are not normally defined as 'urban' but in any area having a reasonable sized population or where markets and other business activities take place, the geometric design of the road needs to be modified to ensure good access and to enhance safety. This is done by using:

- A wider cross section;
- Specifically designed lay-byes for passenger vehicles to pick up or deposit passengers;
- Roadside parking areas.

The standards are specified in the Geometric Design manual - 2011 and are not specific to LVRs. However, they are repeated here for completeness.

The additional width depends on the status of the populated area that the road is passing through. If the road is passing through a Wereda seat or a larger populated area, an extra paved carriageway of 3.5m width and to the same structural design as the main carriageway, is provided in each direction for parking and for passenger pick-up and a 2.5m pedestrian footpath is also specified. The latter is essentially the shoulder. In addition, the main running surface is paved and is 7.0m wide. Thus the road in such areas is similar to Class DC4 but with an additional wide parking/activities carriageway and a footpath on each side. The pavement structure of the wide parking should be identical to the pavement of the running surface.

When passing through a Kebele seat a 2.5m paved shoulder is specified but no additional footpath; although one could easily be provided if required. The carriageway is also increased to 7.0m and therefore the standard is very similar to DC4 but with wider shoulders.

These standards are not justified for the lower traffic levels of DC2, which is a single carriageway, unless the road is passing through a particularly well populated area that is not classified as a Kebele or Wereda seat but where additional traffic may be expected. In such circumstances the shoulders should be widened to 2.5m for the extent of the populated area.

4.2.7 Pavement type

For a similar 'quality' of travel there is a difference between the geometric design standards required for an unsealed road (gravel or earth) and for a sealed road. This is because of the very different traction and friction properties of the two types of surfaces and the highly variable nature of natural materials. Higher geometric standards are generally required for unsealed roads. A road that is to be sealed at a later date should be designed to the higher, unsealed, geometric road standards.

4.2.8 Soil type and climate

Soil type affects the ideal geometric design, principally in terms of cross-section rather than in terms of the width of the running surface or road curvature. With some problem soils the cross-section can be adjusted to minimise the severity of the problem by, for example, minimising the speed of water flow; minimising the likelihood of excessive water inundation or penetration into the carriageway; and/or moving problems areas further away from the carriageway itself. These aspects are dealt with in the drainage section (Chapter 5) of the manual and in the pavement design section (Chapter 6).

Ideally maximum gradients for unpaved roads should also depend on soil types but this is usually impracticable because, in most climatic regions, almost any gradient causes problems for unpaved roads. Recent research has demonstrated that gravel-surfaced roads are unsustainable in many more situations than has been thought previously and this applies equally to earth roads. Consequently every effort is being made to introduce or to develop more sustainable surfacings for use where unpaved roads deteriorate too quickly. Such surfacings cannot usually be justified for long stretches of road where they are not essential hence the concepts of spot improvements and environmentally optimised design (EOD) are being developed and refined.

4.2.9 Safety

Experience has shown that simply adopting 'international' design standards from developed countries will not necessarily result in acceptable levels of safety on rural roads. The main reasons include the completely different mix of traffic, including relatively old, slow-moving and usually overloaded vehicles; a large number of pedestrians, animal drawn carts and, possibly, motorcycle-based forms of transport; poor driver behaviour; and poor enforcement of regulations. In such an environment, methods to improve safety through engineering design assume paramount importance.

Although little research has been published on rural road safety in Ethiopia, the following factors related to road geometry are known to be important:

- Vehicle speed;
- Horizontal curvature;
- Vertical curvature;
- Width of shoulders.

These factors are all inter-related and part of geometric design. In addition, safety is also affected by:

- Traffic level and composition;
- Inappropriate public transport pick-up/set-down areas;
- Poor road surface condition (eg potholes);
- Dust (poor visibility);
- Slippery unsealed road surfaces.

The last three factors are related to structural design covered in Chapter 5.

Conflicts between motorised vehicles and pedestrians are always a major safety problem on many rural roads where separation is generally not economically possible. The World Bank Basic Access document (World Bank, 2001) considers that there are sound arguments based on safety for keeping traffic speeds low in mixed traffic environments rather than aiming for higher design speeds, as is the case for major roads. The use of wider shoulders is also suggested. These considerations have been incorporated into this manual.

Traffic level and composition are both considered. A considerable number of conflict situations can arise when the number of PCUs of non-motorised traffic is large even though the number of two (or more)-axled motorised traffic is quite low. Furthermore, the proportion of heavy vehicles on the LVRs of Ethiopia can also be high, leading to more serious conflict situations. The overall traffic class standards are based on the number of two (or more)-axled motorised vehicles but additional safety features are based on:

- the number of PCUs of non 2-axled motorised vehicles and pedestrians; and
- the proportion of heavy vehicles in the motorised stream.

Pedestrians (and draft animals) find it very uncomfortable to walk on poorly graded gravel shoulders containing much oversized material, especially in bare feet. They usually choose to walk on a paved running surface, if available, despite the greatly increased safety risk. Thus, provision of a wider unsurfaced shoulder does not ensure greater safety. On the approaches to market villages, where the pedestrian traffic increases greatly on market days, provision of a separate footpath is the best solution provided that the soil is suitable.

A checklist of engineering design features that affect road safety is given in Figure D.4.2. Not all are suitable for rural roads but the general philosophy of design for safety is emphasised.

The following factors should be considered when designing for safety:

- Wherever possible, non-motorised traffic should be segregated by physical barriers, such as raised kerbs (through villages and peri-urban areas).
- Designs should include features to reduce speeds in areas of significant pedestrian activity, particularly at crossing points. Traffic calming may need to be employed (see Section 4.8.1).
- To minimize the effect of a driver who has lost control and left the road, the following steps should be taken.
    - Steep open side-drains should be avoided since these increase the likelihood that vehicles will overturn. (See Section 4.3.1).
    - Trees should not be planted immediately adjacent to the road.
- Guard rails should only be introduced at sites of known accident risk because of their high costs of installation and maintenance.
- Junctions and accesses should be located where full safe stopping sight distances are available (see Section 4.4).

4.2.10 Construction technology

In a labour-abundant economy it is usually beneficial to maximise the use of labour rather than rely predominantly on equipment-based methods of road construction. In such a situation the choice of technology might affect the standards that can be achieved, especially in hilly and mountainous areas. This is because:

- Maximum cuts and fills will need to be small;
- Economic haul distances will be limited to those achievable using wheel-barrows;
- Mass balancing will need to be achieved by transverse rather than longitudinal earth movements;
- Maximum gradients will need to follow the natural terrain gradients;
- Horizontal alignments will need to be less direct.

The standards in hilly and mountainous terrain are always lower than in flat terrain but this reduction in standards need not necessarily be greater where labour-based methods are used. Following the contour lines more closely will make the road longer but the gradients can be less severe. Every effort should be made to preserve the same standards in the particular terrain encountered irrespective of construction method.

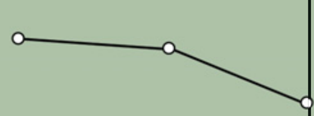
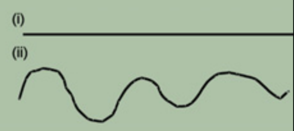
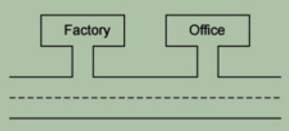
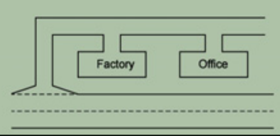
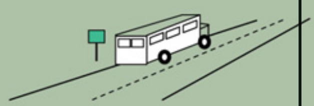
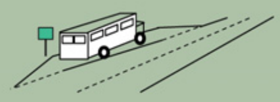
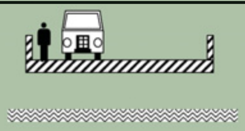
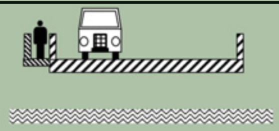
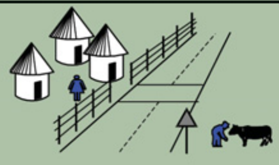
	Undesirable	Desirable	Principle applied
Route location			Major routes should by-pass towns and villages
Road geometry			Gently-curving roads have lowest accident rates
Roadside access			Prohibit direct frontal access to major routes and use service roads
			Use lay-bys or widened shoulders to allow villagers to sell local produce
			Use lay-bys for buses and taxis to avoid restriction and improve visibility
Segregate motorised and non-motorised traffic			Seal shoulder and provide rumble divider when pedestrian or animal traffic is significant
			Construct projected footway for pedestrians and animals on bridges
			Fence through villages and provide pedestrian crossings

Figure D.4.2: Examples of effects of engineering design on road safety

4.2.11 Administrative function

In many countries it is necessary to take account of the administrative or functional classification of roads because a certain standard may be expected for each functional class of road irrespective of the current levels of traffic. Generally the hierarchy of administrative classification broadly reflects the traffic levels observed but anomalies are common where, for example, traffic can be lower on a road higher in the hierarchy. It is recommended that the standards selected should be appropriate to the task or traffic level of the road in question but a minimum standard for each administrative class can also be defined if it is policy to do so.

4.2.12 Matrix of standards

For each of the basic standards based on traffic level there are four standards to cope with terrain (flat, rolling, mountainous and escarpment), a further two standards (for DC4, DC3 and DC2) to cater for roadside population/activities and three standards to cater for traffic composition, essentially the number of PCUs of non-motorised traffic (including pedestrians) and the percentage of heavy vehicles in the traffic stream. These additional standards for traffic composition and roadside activities are essentially standards to enhance safety.

Once these factors have been taken into account, safety alone no longer affects the number of road standards because an acceptable level of safety must be applied to each road class. This will differ *between* classes (greater safety features for higher traffic) but not within classes. The administrative classification does not add to the number of standards either. If the traffic level indicates that a lower standard than would normally be acceptable based on administrative classification is sufficient, the road can be built to the minimum standard appropriate to its administrative classification.

Aspects of geometric design outlined in the following sections require particular consideration because they have a major influence on the life-cycle costs of rural roads. The basis for developing the standards is also discussed in these sections.

In contrast to the judgements required for quantifying traffic, the standards themselves are largely dictated by the selected design speed and form a continuous range as design speed increases.

4.3 Cross sections

The cross section of a road is essentially a geometric design feature but is also intimately related to drainage issues as well as slope stability and erosion problems in hilly and mountainous areas. The cross section includes the shape and size of the running surface; shoulders; the side slopes of embankments; side slopes to drainage ditches; drainage ditches themselves; and slopes to the batter.

The basic cross sections for LVRs are shown in Part B, Section 4.5. Some aspects of cross sectional design are concerned with drainage and further details concerning this aspect are discussed in Chapter D.5.

The cross-section of a road may need to vary over a route but it is essential that any changes take place gradually over a transition length. Abrupt and isolated changes lead to increased hazards and reduced traffic capacity.

A common situation arises at bridge and water crossing points where the existing structure is narrower than desired. In such situations warning signs must be erected to alert drivers. Fortunately many such crossings are visible well in advance but if not, extra signage may be required.

4.3.1 Slopes of shoulders, side slopes, embankments

Side slopes should be designed to ensure the stability of the roadway and to provide a reasonable opportunity for recovery if a vehicle goes out of control across the shoulders. In addition, the position of the side drain invert should be a reasonable distance away from the road to minimise the risk of infiltration of water into the road if the drain should be full for any length of time.

Figure D.4.3 illustrates the general cross section and defines the various elements.

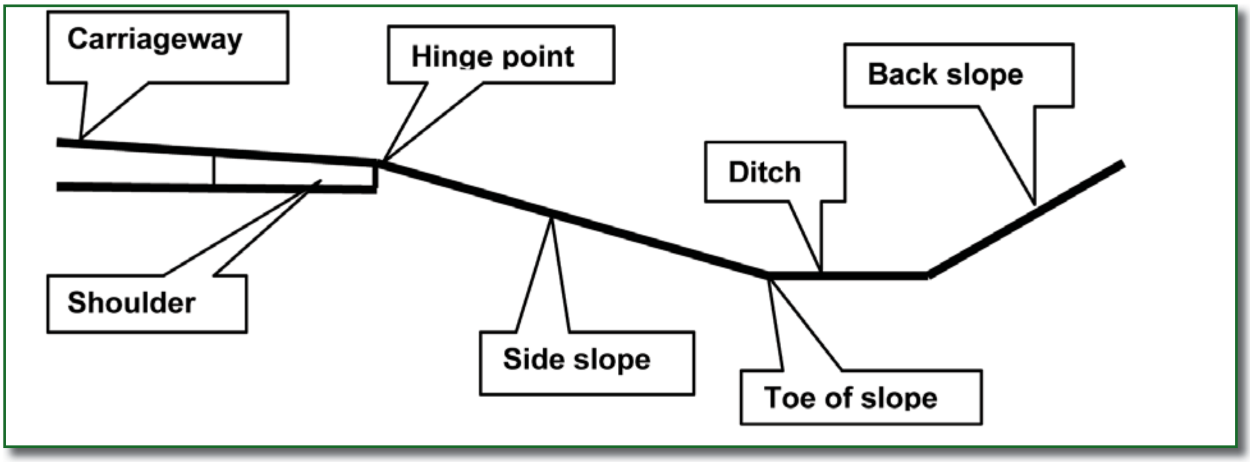


Figure D.4.3: Details of the road edge

The side slope is defined as 'recoverable' when drivers can generally recover control of their vehicles should they encroach over the edge of the shoulder. Side slopes of 1:4 or flatter are recoverable. Research has also shown that rounding at the hinge point and at the toe of the slope is also beneficial.

A non-recoverable slope is defined as one that is traversable but from which most drivers will be unable to stop safely or return to the roadway easily. Vehicles on such slopes can be expected to reach the bottom. Slopes of between 1:3 and 1:4 fall into this category.

A critical slope is one on which the vehicle is likely to overturn and these will have slopes of greater than 1:3.

The selection of side slope and back slope is often constrained by topography, embankment height, height of cuts, drainage considerations, right of way limits and economic considerations. For rehabilitation and upgrading projects, additional constraints may be present such that it may be very expensive to comply fully with the recommendations provided in this manual.

Table D.4.4 indicates the side slope ratios recommended for use in the design based on the type of material and the height of fills and cuts. This Table should be used as a guide only. Where slope stability problems are likely, slope configuration and treatment should be based on expert advice.

Table D.4.4: Safety of slopes (ratios are vertical:horizontal)

Material	Height of slope (m)	Side slope		Back slope	Safety classification
		Cut	Fill		
Earth(1)	0.0-1.0	1:4	1:4	1:3	Recoverable
	1.0-2.0	1:3	1:3	1:2	Not recoverable
	>2.0	1:2	1:2	1:1.5	Critical
Rock	Any height	Dependant on costs			Critical
Expansive clays(2)	0-2.0	n/a	1:6		Recoverable
	>2.0	n/a	1:4		

Notes:

1. See Section
2. Certain soils may be unstable at slopes of 1:2. Geotechnical advice require
3. The drainage ditch should be moved away from the embankment

4.3.2 Roadside ditches

Detailed information concerning roadside ditches is provided in Chapter D.5, Section 5.4.4.

Side drains should be avoided when the road traverses areas of expansive clays. Water should be discharged uniformly along the road. Where side drains cannot be avoided they should be a minimum distance of 4m from the toe of the embankment and should be trapezoidal in shape.

4.3.3 Clear zones

The discussion in Section 4.3.1 has highlighted the safety aspects of embankment side slopes. However, many accidents are made more severe because of obstacles that an out-of-control vehicle may collide with. The concept of clear zones identifies these obstacles and attempts to eliminate such hazards.

The most common hazards are headwalls of culverts and road signage. The clear zone defined for high volume roads is substantial (15m is typical) but for LVRs this is impractical. Ideally it should extend at least to the toe of the embankment and should always be greater than 1.5 m from the edge of the carriageway. At existing pipe culverts, box culverts and bridges the clear zone cannot be less than the carriageway width. If this criterion cannot be met, the structure must be widened. New pipe and box culverts must be designed with a 1.5m clearance from the edge of the shoulder. Horizontal clearance to road signs and marker posts must also be an absolute minimum of 1.5m from the edge of the carriageway.

4.3.4 Right-of-way

Right-of-way (or the road reserve) is provided to accommodate road width and the drainage requirements; to enhance safety; to improve the appearance of the road; to provide space for non-road travellers; and to provide space for upgrading and widening in the future. The width of the right-of-way depends on the cross-sectional elements of the highway, topography and other physical controls; plus economic considerations. Although extended rights-of-way are convenient, right-of-way widths should be limited to a practical minimum because of their effect on local economies.

Rights-of-way are measured equally each side of the centre line. Road reserve widths applicable for the different road classes are shown in Table D.4.5. In mountainous terrain where large cuts are required, the total width can exceed the right-of-way width.

Table D.4.5: Right of way widths

Road Class	Total Right of Way (m)
DC4	50
DC3	30
DC2	30
DC1	20

4.3.5 Shoulders

The shoulders of a road must fulfil the following functions:

- A structural function;
- Allow wide vehicles to pass one another without causing damage to the shoulder;
- Provide safe room for temporarily stopped or broken down vehicles;
- Allow pedestrians, cyclists and other vulnerable road users to travel in safety;
- Allow water to drain from within the pavement layers;
- Reduce the extent to which water flowing off the surface can penetrate into the pavement (often done by extending a seal over the shoulder).

Shoulders have an important structural function which is often overlooked in the provision of LVRs. They act as edge supports to contain the running carriageway; without adequate shoulders the road will move laterally and deform. Therefore, there is a minimum width of shoulder that is required to perform this function. Depending on the properties of the material and the traffic, this can range from 0.5 to 1.5m.

Shoulders also have to perform an important traffic carrying function for non-motorised vehicles and pedestrians. Wider shoulders are required when this traffic is high enough. In addition, wider shoulders are provided for some classes of road when the proportion of heavy vehicles in the traffic stream exceeds certain values.

When the road passes through denser areas of population, additional width is provided for parking and for other roadside activities. This widening may be considered to be shoulder widening although the need to provide access to shops and market areas means that the construction is usually of an extra carriageway.

Where the carriageway is paved, the shoulder may be gravel or may be sealed with a bituminous surface treatment. The structural advantages of a sealed shoulder are discussed in Chapter 6, Section 6.18. However, sealing the shoulders whenever the numbers of non-motorised traffic exceeds a critical value is recommended in order to encourage the travellers to use the shoulders rather than the carriageway. On the approaches to villages and towns the local traffic builds up quite quickly and therefore consideration should be given to extending the sealed shoulders for considerable distances each side of the town/village. No standard guidance can be given; each situation should be treated on its merits.

Shoulders constructed with the same material as the carriageway (earth or gravel) should have the same cross-fall as the carriageway. If the shoulders are gravel and the carriageway is paved the cross-fall of the shoulder should be 1.5-2.0 % steeper than that of the carriageway.

Shoulder widths in mountainous terrain and escarpments are reduced to minimise the high cost of earthworks. Usually the design of the overall cross-section in such terrain will include significant drainage and erosion control features and the shoulder will form an important component of this (Chapter 6, Section 6.18.3.

4.3.6 Single lane roads and passing places

There is good agreement internationally about the recommended carriageway width for single-lane roads, namely 3.0m. Passing places will be required, depending on the traffic level and provision for other traffic and pedestrians will need to be introduced (wider shoulders) if the numbers of other road users exceed specified levels. The increased width should allow two vehicles to pass at slow speed and hence depends on the design vehicle.

Passing places should normally be provided every 300m to 500m depending on the terrain and geometric conditions. Care is required to ensure good sight distances and the ease of reversing to the nearest passing place, if required. Passing places should be built at the most economic places rather than at precise intervals provided that the distance between them does not exceed the recommended maximum. Ideally, the next passing place should be visible from its neighbour.

The length of passing places is dictated by the maximum length of vehicles expected to use the road, indicating the need to define a design vehicle. The design vehicle DV3 is 12.8m long therefore passing places of twice this length should be provided. In most cases, a length of 25m will be sufficient for rural roads.

A suitable width depends upon the width of the road itself. The criterion is to provide enough overall width for two design vehicles to pass each other safely at low speed. Therefore, a total traffickable minimum width of 6.3m is required (providing a minimum of 1.1m between passing vehicles). Allowing for vehicle overhang when entering the passing bay, a total road width of 7.0m is suitable.

4.3.7 Width standards

Road width (running surface and shoulders) is one of the most important geometric properties since its value is very strongly related to cost and to safety.

A review of international standards showed that some countries have adopted road widths that are intermediate between single lane and two lane requirements. Such roads are considered to be dangerous because vehicles try to pass each other at speed. Since there is not enough room to do so they are forced onto the shoulder area and dangerously close to the road edge. If the road is paved, the edge of the paved area becomes damaged very quickly.

The standards for Ethiopia do not include such intermediate widths. However, for all standards except DC1, shoulders are widened if the number of road users other than 2-axled (and more) motorised vehicles exceeds levels that cause too much interaction with the motorised traffic or if the proportion of heavy vehicles in the traffic stream is high. For DC1 the traffic levels are so low that dangerous interactions will be rare and drivers will expect other road users to have priority.

The basic vehicle classification in terms of traffic is shown in Table B.3.1.

Tables B.4.2 to B.4.8 show the standard road widths for each road class with the widths of running surface and shoulder given for the paved road classes. The road width of unpaved roads shown in the tables includes the widths of the shoulder as the surfacing of gravel or earth for these roads spreads across the whole surface to the edge of the road making it difficult to define the shoulder for these roads. The shoulder widths for paved roads are also varied for different terrains; for roadside population/activities; and for traffic composition.

The lowest class (DC1) is a single lane road and the shoulder is effectively 0.75m wide. For DC2 the minimum shoulder width is not specifically defined for the unpaved option but, for this class, the traffic is less than 5 vehicles per hour in each direction which will invariably travel down the centre of the road unless another vehicle is seen approaching. Therefore, the effective shoulder width for most of the time is 1.5m. For this class, two vehicles can normally pass each other safely using the shoulder but if one of the largest vehicles is involved the vehicles may need to slow down. If there are sufficient of these larger vehicles, then class DC3 should be used.

A similar argument applies to DC3. The unpaved option for DC3 is wider than DC2 and allows easier passing but the level of service is not commensurate if there are a large number of the larger vehicles in the traffic stream. In this case, the next higher class (DC4) is recommended. For DC4 the shoulders are at least 1.0m wide.

Additional shoulder widths are provided if there is a high number of PCUs of non-motorised vehicles (defined as more than 300 PCUs per day on average).

Other variations are also sometimes needed, for example, where certain problem soils are encountered or in areas that are particularly wet and where the road is likely to be inundated and needs to be raised on a higher embankment. These variations are discussed in Chapter D.2 Section 2.5.2 and Chapter D.6 Section 6.19.

Where spot improvements are made which involve a short length of paved surfacing (eg on a steep incline) then the width used should be that shown in the respective Tables for paved surfaces.

The width standards for each classification are summarised in Part B, Section 4.3. For some of the cells in the Tables the values quoted will never be a limitation. For example, in flat terrain there will be no need to be concerned about the criteria for maximum gradient and paved road sections will be rare at the lowest traffic levels. Nevertheless, for completeness all the cells have been filled.

4.4 Design speed and geometry

Design speed is defined as effectively the maximum (actually the 85th percentile) safe speed that can be maintained over a specified section of road when conditions are so favourable that the design features of the road govern the speed. Design speed is used as an index that essentially defines the geometric standard of a road, linking many of the factors that determine the road's service level, namely traffic level; terrain; pavement type; safety/population density; and road function, to ensure that a driver is presented with a consistent speed environment.

The concept of design speed is most useful because it allows the key elements of geometric design to be selected for each standard of road in a consistent and logical way. For example, design speed is relatively low in mountainous terrain to reflect the necessary reductions in standards required to keep road costs to manageable proportions. The speed is higher in rolling terrain and highest of all in flat terrain.

In practice the speed of motorised vehicles on many roads in flat and rolling terrain will only be constrained by the road geometry over relatively short sections but it is important that the level of constraint is consistent for each road class and set of conditions.

In view of the mixed traffic that occupies the rural roads of Ethiopia and the cost benefit of selecting lower design speeds, it is prudent to select values of design speed towards the lower end of the internationally acceptable ranges. The recommended values are shown in Table D.4.6.

Changes in design speed, if required because of a change in terrain, should be made over distances that enable drivers to change speed gradually. Thus changes should never be more than one design step at a time and the length of the sections with intermediate standards (if there is more than one change) should be long enough for drivers to realise there has been a change before another change in the same direction is encountered (ie considerably more than one single bend). Where this is not possible, warning signs should be provided to alert drivers to the changes.

Table D.4.6: Design speeds

Design standard	Design speed (km/h)				
	Flat	Rolling	Mountain	Escarpment	Urban
DC 4	70	60	50	40	50
DC 3	70	60	50	30	50
DC 2	60	50	40	30	50
DC 1	50	40	30	20	40

4.4.1 Stopping sight distance

In order to ensure that the design speed is safe, the geometric properties of the road must meet certain minimum or maximum values to ensure that drivers can see far enough ahead to carry out normal manoeuvres such as overtaking another vehicle or stopping if there is an object in the road.

The distance a vehicle requires to stop safely is called the stopping sight distance. It mainly affects the shape of the road on the crest of a hill (vertical alignment) but if there are objects near the edge of the road that restrict a driver's vision on approaching a bend, then it also affects the horizontal curvature.

The driver must be able to see any obstacle in the road hence the stopping sight distance depends on the size of the object and the height of the driver's eye above the road surface. The driver needs time to react and then the brakes of the vehicle need time to slow the vehicle down, hence stopping sight distance is extremely dependant on the speed of the vehicle. The surface characteristics of the road also affect the braking time so the values for unpaved roads differ from those of paved roads, although the differences are small for design speeds below 60km/h.

The stopping distance also depends on the gradient of the road; it is harder to stop on a downhill gradient than on a flat road because a component of the weight of the vehicle acts down the gradient in the opposite direction to the frictional forces that are attempting to stop the vehicle.

Full adherence to the required sight distances is essential for safety reasons. On the inside of horizontal curves it may be necessary to remove trees, buildings or other obstacles to obtain the necessary sight distances. If this cannot be done, the alignment must be changed. In rare cases where it is not possible and a change in design speed is necessary, adequate and permanent signage must be provided.

Recommended stopping sight distances for paved and unpaved roads at different design speeds are shown in Table D.4.7.

Table D.4.7: Stopping sight distances (m)

Design speed (km/h)	20	30	40	50	60	70	80
Unpaved roads(1)	20	30	50	70	95	125	160
Paved roads(1)	18	30	45	65	85	110	135

Note:
1. In rolling and mountainous terrain these volumes should be increased by 10%.

4.4.2 Stopping sight distance for single lane roads (meeting sight distance)

For single lane roads, adequate sight distances must be provided to allow vehicles travelling in the opposite direction to see each other and to stop safely if necessary. This distance is normally set at twice the stopping sight distance (Table D.4.6) for a vehicle that is stopping to avoid a stationary object in the road. An extra safety margin of 20-30 metres is also sometimes added.

Although a vehicle is a much larger object than is usually considered when calculating stopping distances, these added safety margins are used partly due to the very severe consequences of a head-on collision; and partly because it is difficult to judge the speed of an approaching vehicle, which could be considerably greater than the design speed. However, single lane roads will have a relatively low design speed, hence meeting sight distances should not be too difficult to achieve.

4.4.3 Intersection sight distance

Intersection sight distance is similar to stopping sight distance except that the object being viewed is another vehicle that may be entering the road from a side road or crossing the road at an intersection. The required safe sight distance for trucks in metres is about 3 times the vehicle speed in km/hr. On straight sections of road many vehicles will exceed the road's design speed but, being straight, sight distances should be adequate.

4.4.4 Passing sight distances

Factors affecting the safe sight distances required for overtaking are more complicated because they involve the capability of a vehicle to accelerate and the length and speed of the vehicle being overtaken. Assumptions are usually made about the speed differential between the vehicle being overtaken and the overtaking vehicle but many road authorities have simply based their standards on empirical evidence.

For single lane roads, overtaking manoeuvres are not possible and passing manoeuvres take place only at the designated passing places. On the lower classes of 2-lane roads, passing sight distances are based on providing enough distance for a vehicle to safely abort a passing manoeuvre if another vehicle is approaching. The recommended values are shown in Table D.4.8.

Table D.4.8: Passing sight distances (m)

Design speed (km/h)	30	40	50	60	70	80
Recommended values	75	110	160	205	260	320

4.4.5 Camber and cross-fall

Camber and cross-fall are essential to promote surface drainage. Ponding of water on a road surface quickly leads to deterioration. There is general agreement that camber or cross-fall should be 3% on sealed LVRs (2.5% is sometimes advocated but this is insufficient).

Drainage is less efficient on rough surfaces and therefore the camber or cross-fall needs to be higher on earth and gravel roads. However, if the soil or gravel is susceptible to erosion, high values of camber or cross-fall can cause erosion problems. Values that are too high can also cause driving problems but, on the lower standards of rural roads where traffic is low and the road is a single carriageway, vehicles will generally travel in the middle of the road. Therefore, high levels of camber are not as much of a problem for drivers as high levels of cross-fall. The design of LVRs makes use of this fact so that higher camber is used where appropriate. As a result, the optimum value of cross-fall/camber varies considerably but it normally lies between 4% and 7% with 6% being the usual recommendation in the absence of additional information concerning the erosion potential of the soil/gravel.

Shoulders having the same surface as the running surface should have the same slope. Unpaved shoulders on a sealed road should have shoulders that are about 2% steeper, in other words 5% if the running surface is 3%.

4.4.6 Adverse cross-fall

Adverse cross-fall arises on curves when the cross-fall or camber causes vehicles to lean outwards when negotiating the curve. This affects the cornering stability of vehicles and is uncomfortable for drivers, thereby affecting safety. The severity of its effect depends on vehicle speed, the horizontal radius of curvature of the road and the side friction between tyres and road surface. For reasons of safety it is recommended that adverse cross-fall is removed where necessary (see Table D.4.9) on all roads regardless of traffic.

Table D.4.9: Adverse cross-fall to be removed if radii are less than shown

Design speed (km/h)	Minimum radii (m)	
	Paved	Unpaved
<50	500	700
60	700	1000
70	1000	1300
85	1400	
100	2000	

Some cross-fall is necessary for drainage and hence flat sections are not allowed. Instead, a single value of cross-fall is designed in the proper direction (ie all camber is removed as shown in Figure D.4.4) such that the cross sectional shape of the road is straight with the cross slope being the same as that of the inner side of the cambered two-lane road (usually 3 or 4% for sealed roads). For unpaved roads the recommended cross-fall should also be the same as the normal camber or cross-fall value of 6%.

To remove adverse cross-fall the basic cambered shape of the road is gradually changed as the road enters the curve until it becomes simply cross-fall in one direction at the centre of the curve.

For sealed roads the removal of adverse camber may not be sufficient to ensure good vehicle control when the radius of the horizontal curve becomes too small. In such a situation additional cross-fall may be required. This is properly referred to as super-elevation but it has become common practice to refer to all additional elevation as super-elevation and this convention will be used here.

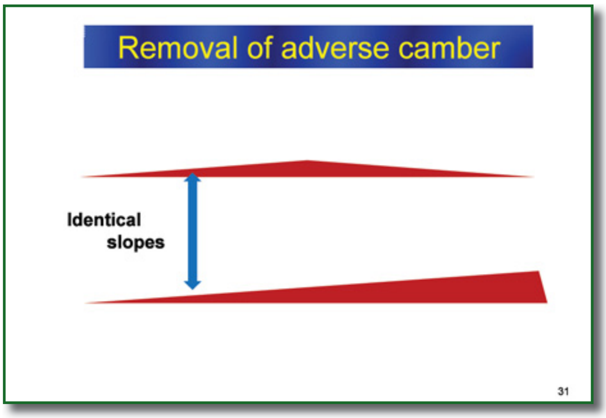


Figure D.4.4: Removal of adverse camber

4.4.7 Super-elevation

Super-elevation on unsealed LVRs is not necessary. This is because it is recommended that adverse cross-fall or camber is always removed on horizontal curves below 1000m radius. Since the recommended cross-fall or camber is 6%, the effective 'super-elevation' when adverse cross-fall is removed will also be 6% and this therefore determines the minimum radius of horizontal curvature for each design speed in the same way as for genuine super-elevation. In practice it may not be possible to maintain such a value of cross-fall during the life of an unsealed road and therefore it is recommended that minimum radii are based on the lower level of 4% cross-fall.

For sealed roads the removal of adverse cross-fall will result in an effective super-elevation of 3% and this should be used to determine minimum radii of curvature for such roads. However, if these radii are difficult to achieve, genuine super-elevation of up to 7% (or, in exceptional circumstances, up to 10%) can be used with a resulting decrease in horizontal radius of curvature.

The change from normal cross-section on straight sections of road to a super-elevated section should be made gradually. The length over which super-elevation is developed is known as the super-elevation development length. Two-thirds of the development length should be provided before the curve begins. The development depends on design speed as shown in Table D.4.10. Between 50% and 75% of the super-elevation should be achieved by the tangent point. 66% is usually used.

Table D.4.10: Super-elevation development lengths

Design speed (km/h)	Development length (m)
30	25
40	30
50	40
60	55
70	65
80	80

4.5 Horizontal alignment

The horizontal alignment consists of a series of straight sections (tangents) connected to circular curves. The horizontal curves are designed to ensure that vehicles can negotiate them safely. The alignment design should be aimed at avoiding sharp changes in curvature, thereby achieving a safe uniform driving speed. Transition curves between straight sections of road and circular curves whose radius changes continuously from infinity (tangent) to the radius of the circular curve (R) are used to reduce the abrupt introduction of centripetal acceleration that occurs on entering the circular curve. They are not required when the radius of the horizontal curve is large and are normally not used on the lower classes of road. In Ethiopia their use is confined to roads where the design speed is 80km/hr or greater and therefore they are not required for LVRs.

In order for a vehicle to move in a circular path an inward radial force is required to provide the necessary centripetal acceleration or, in other words, to counteract the centrifugal force. This radial force is provided by the sideways friction between the tyres and the road surface assisted by the cross-fall or super-elevation.

The sideways friction coefficient is considerably less than the longitudinal friction coefficient. Its value decreases as speed increases but there is considerable disagreement about representative values, especially at the lower speeds. For paved roads it ranges from between 0.18 and 0.3 at 20km/h down to between 0.14 and 0.18 at 80km/h. For unpaved roads it can be considerably less. The design speed is therefore one of the main design parameters. Values for each class of road under each of its operating conditions have been set as shown in Table D.4.6.

For both sealed and unsealed roads there are also constraints on the maximum cross-fall, as described in Section 4.4.5 and 4.4.6. These constraints translate directly into minimum values of horizontal radii of curvature.

The recommended values of horizontal curvature are shown in Table D.4.11 and Table D.4.12. As indicated in the Tables, the use of a higher value of super-elevation makes it possible to introduce a smaller horizontal curve based on the same design speed. This can be used for paved roads but not for unpaved roads.

Table D.4.11: Recommended minimum horizontal radii of curvature: paved roads (m)

Design speed (km/h)	20	30	40	50	60	70	80
Minimum horizontal radius for SE = 4%	15	25	50	85	135	195	270
Minimum horizontal radius for SE = 7%	15	25	45	75	120	170	235
Minimum horizontal radius for SE = 10%	15	20	40	70	105	150	205

Table D.4.12: Recommended minimum horizontal radii of curvature: unpaved roads (m)

Design speed (km/h)	20	30	40	50	60	70	80
Minimum horizontal radius for SE = 4%	17	35	70	110	175	245	340
Minimum horizontal radius for SE = 6%	15	30	60	100	155	215	300

4.5.1 Curve length

For reasons of safety and ease of driving, curves near the minimum for the design speed should not be used at the following locations:

- On high fills, because the lack of surrounding features reduces a driver's perception of the alignment.
- At or near vertical curves (tops and bottoms of hills) because the unexpected bend can be extremely dangerous, especially at night.
- At the end of long tangents or a series of gentle curves, because actual speeds will exceed design speeds.
- At or near intersections and approaches to bridges or other water crossing structures.

There are conflicting views about curve lengths. One school of thought maintains that the horizontal alignment should maximise the length of road where adequate sight distances are provided for safe overtaking. Overtaking is difficult on curves of any radius and hence the length of curved road should be minimised. This requires curve radii to be relatively close (but not too close) to the minimum for the design speed to maximise the length of straight sections. This view is the currently accepted best practice for roads except in very flat terrain but care should be exercised to ensure the curves are not too tight.

The alternative view is that very long straight sections should be avoided because they are monotonous and cause headlight dazzle at night. A safer alternative is obtained by a winding alignment with tangents deflecting 5 to 10 degrees alternately from right to left. Straight sections should have lengths (in metres) less than 20 x design speed in km/h. Such 'flowing' curves restrict the view of drivers on the inside carriageway and reduce safe overtaking opportunities, therefore such a winding alignment should only be adopted where the straight sections are very long. In practice this only occurs in very flat terrain. The main aspect is to ensure that there are sufficient opportunities for safe overtaking and therefore, provided the straight sections are long enough, a semi-flowing alignment can be adopted at the same time. If overtaking opportunities are infrequent, maximising the length of the straight sections is the best option.

For small changes of direction it is often desirable to use a large radius of curvature. This improves the appearance and reduces the tendency for drivers to cut corners. In addition, it reduces the length of the road segment and therefore the cost of the road provided that no extra cut or fill is required.

4.5.2 Curve widening

Widening of the carriageway where the horizontal curve is tight is usually necessary to ensure that the rear wheels of the largest vehicles remain on the road when negotiating the curve; and, on two lane roads, to ensure that the front overhang of the vehicle does not encroach on the opposite lane. Widening is therefore also important for safety reasons. Any curve widening that is considered should only be applied on the inside of the curve.

Vehicles need to remain centred in their lane to reduce the likelihood of colliding with an oncoming vehicle or driving on the shoulder. Sight distances should be maintained as discussed above. The levels of widening shown in Table D.4.13 are recommended except for roads carrying the lowest levels of traffic (DC1). Widening should be applied on the inside of the curve and introduced gradually.

Widening on high embankments is often recommended for the higher classes of road. The steep drops from high embankments unnerve some drivers and the widening is primarily for psychological comfort although it also has a positive effect on safety. Such widening is not recommended for LVRs.

Table D.4.13: Widening recommendations (m)

	Single lane roads				Two lane roads			
Curve radius	20	30	40	60	<50	51-150	151-300	301-400
Increase in width	1.5(1)	1.0	0.75	0.5	1.5	1.0	0.75	0.5

Notes:

1. See Section 3.6.4 dealing with hairpin stacks

4.6 Vertical alignment

The two major elements of vertical alignment are the gradient, which is related to vehicle performance and level of service; and the vertical curvature, which is governed by safe sight distances and comfort criteria.

The vertical alignment of a road seems more complicated than the horizontal alignment but this is simply because of difficulties in presentation due to the inclusion of the algebraic difference in gradient (G %) between the uphill and downhill sides. In addition, the equation of the vertical curve is a parabola rather than a circle.

The required sight distance for safety is the basic stopping sight distance.

4.6.1 Crest curves

The minimum length of the curve (L metres) over the crest of the hill between the points of maximum gradient on either side is related to G and to the stopping-sight distance; and therefore to the design speed. Note that although drivers would like to overtake on hills, the required sight distance for safe passing on crests is much too large to be economical on LVRs.

The minimum value of the L/G ratio can be tabulated against the stopping sight distance (Table D.4.6), and therefore the design speed, to provide the designer with a value of L for any specific value of G. The international comparisons give the values shown in Table D.4.14.

Table D.4.14: Minimum values of L/G for crest curves

Design speed (km/h)	30	40	50	60	70	80
Sealed roads	2	4	7	12	21	37
Unsealed roads	3	6	11	19	34	58

4.6.2 Sag curves

Sag curves are the opposite of crest curves in that vehicles first travel downhill and then uphill. In daylight the sight distance is normally adequate for safety and the design criterion is based on minimising the discomforting forces that act upon the driver and passengers when the direction of travel changes from downhill to uphill. On rural roads such considerations are somewhat less important than road safety issues. However, at night time the problem on sag curves is the illumination provided by headlights to see far enough ahead. This depends on the height of the headlights above the road and the angle of divergence of the headlight beams.

To provide road curvature that allows the driver to see sufficiently far ahead using headlights while driving at the design speed at night is usually too expensive for LVRs. In any case, the driving speed should be much lower at night on such roads. As a result of these considerations it is recommended that the minimum length of curve is determined by the driver discomfort criterion. The results are shown in Table D.4.15.

Table D.4.15: Minimum values of L/G for sag curves

Design speed (km/h)	30	40	50	60	70	80
Minimum L/G	0.7	1.3	2.2	3.5	4.8	7.5

In practice a minimum length of curve of 75m will cope with almost all situations on LVRs. For example, on a steep down-hill of 10% followed by an up-hill of the same slope, the required minimum curve length at a speed of 50km/h is 2.2 x (10 + 10) = 44m and 3.5 x (10+10) = 70m at 60km/h.

4.6.3 Gradient

For four-wheel drive vehicles, it is reported that the maximum traversable gradient is about 18%. Two-wheel drive trucks can cope with gradients of 15%, except when heavily laden. Bearing in mind the likelihood of heavily laden small trucks, international rural road standards have a general recommended limit of 12%, but with an increase to 15% for short sections (< 250m) in areas of difficult terrain. Slightly higher standards are recommended for DC4 with a preferred maximum of 10% and an absolute maximum of 12% on escarpments where relief gradients of less than 6% are required for a distance of 250m following a gradient of 12%.

For driving consistency, and hence safety, in terrains other than mountainous terrains and escarpments, limiting values of gradient are also often specified. In flat terrain a maximum gradient of 7% is appropriate for LVRs. In rolling terrain a maximum of 10% is appropriate.

Regional experience indicates that unsealed road sections in excess of 6% gradient are often unsustainable in the medium to long-term. It is expected that the use of alternative surfacings will become more common in Ethiopia to provide a more sustainable solution in critical areas. Therefore criteria need to be developed to identify the critical areas where alternative surfacing are to be recommended. This is dealt with in (Chapter D.7).

4.6.4 Hairpin stacks

Climbing sections on mountain and escarpment roads are often best designed using hairpin stacks. The advantages are that the most favourable site for ascending the escarpment can be selected and a more direct and therefore shorter route will often be possible. However there are several problems.

The limited space to construct cut and fill slopes necessitates either a reduction in geometric standards or more expensive retaining structures. For LVRs the former solution should be adopted.

Lack of suitable sites for disposal of spoil and access difficulties for plant can pose difficulties during construction.

If there are problems of instability they may extend from one loop to another and so the advantage of attempting to choose the most stable section of the escarpment is lost. This is a problem and is dealt with in Chapter D.3.

Storm run-off will, of necessity, become very concentrated so, although the number of drainage structures and erosion controls may be reduced, their capacity will need to be increased. The risk associated with failure of the drainage is therefore correspondingly high and minimising this risk adds to the costs. If the topography allows, some of the problems of stacked hairpins can be reduced by creating several stacks that are offset from each other and staggered across the slope (ie not immediately above or below each other). This will reduce drainage problems and limit the danger of instability to fewer hairpin loops.

The key aspect of their geometric design is that the curves should be as flat as possible and the tangents should be used to achieve the ascent. This is because vehicle traction is much more efficient when the vehicle is travelling in a straight line. The maximum gradient through the hairpin curve itself should be 4% for DC4 and DC3 and up to 6% for DC2 and DC1.

Considerable curve widening will be required where the curve radius is small to ensure that large vehicles can negotiate the bends. Widening is also required for safety reason and, if space allows, to provide a refuge area if a vehicle breaks down.

For LVRs it is recommended that the curves should be designed to allow the passage of the DV4 design vehicle (Table D.4.1). This means that the curve radius at the centre line of the road should be an absolute minimum of 13m and the road should be at least 8m wide.

4.7 Harmonisation of horizontal and vertical alignment

4.7.1 Situations to avoid

When designing the horizontal alignment of a road, the designer must ensure that the other elements of the design are complementary to each other. It is therefore important to note that there are a number of design situations that could produce unsatisfactory combinations of elements despite the fact that the design standards have been followed for the particular class of road in question. These are designs that could provide surprises for drivers by presenting them with unfamiliar conditions. They are therefore comparatively unsafe.

Avoiding such designs is more important for the higher classes of road because design speeds are higher, traffic is much greater and, consequently, any accidents resulting from poor design are likely to be more severe and more frequent. However, in many cases, avoidance of such designs does not necessarily impose a significant cost penalty and therefore the principles outlined below should be applied to roads of all classes.

Multiple curves
In the more hilly and mountainous terrains, horizontal curves are required more frequently and have small radii because the design speeds are low. The tangent sections become shorter and a stage can be reached where successive curves can no longer be dealt with in isolation. There are three situations that should be avoided if possible.

Reverse curves
A curve is followed immediately by a curve in the opposite direction. In this situation it is difficult for the driver to keep the vehicle in its proper lane. It is also difficult for the designer to accommodate the required super-elevation within the space available.

Broken back curves
This is the term used to describe two curves in the same direction connected by a short tangent. Drivers do not usually anticipate that they will encounter two successive curves close to each other in the same direction. There can also be problems fitting in the correct super-elevation in the space available.

Compound curves

Compound curves occur when one curve connects to another of different radius. These can be useful in fitting the road to the terrain but in some circumstances they can be dangerous. Drivers do not usually expect to be confronted by a change in radius, and therefore in design speed, hence if, the change is too great, some drivers are likely to be travelling too fast when entering the tighter part of the compound curve from the larger one. Compound curves should be avoided where curves are sharp and where the difference in radii is large. Thus, in any compound curve the smaller radius should not be less than 67% of the larger one.

Isolated and long curves

An isolated curve close to the minimum radius connected by long straight sections is inherently unsafe. Irrespective of the design speed, actual speeds on long straight sections will be relatively high and therefore a curve of minimum radius will require a significant reduction in speed for most vehicles. It is good practice to avoid the use of minimum standards in such situations. An added bonus is that, provided no extra cutting of filling is required, the use of a larger radius of curvature results in a shorter and less expensive road. Curve widening can help to alleviate this problem if a higher radius curve cannot be used.

The same argument is true, but to a much lesser extent, for any small radius curve that is very long (i.e. the road is turning through a large angle). Drivers can negotiate a short curve relatively safely at speeds in excess of the design speed but they cannot do so if the curve is long hence a large radius should be used in such situations.

4.7.2 Balance

It can be seen that there are several competing factors in providing the optimum horizontal alignment. Small radii curves maximise the length of straight sections and optimise overtaking opportunities. This should be the controlling factor where the terrain is such that overtaking opportunities are infrequent and actual speeds are close to the design speeds. However, in more gentle terrain where overtaking is less of a problem and vehicles generally travel at speeds higher than the design speed, the use of larger radius curves is preferred for the reasons outlined previously.

In summary, engineering choice plays a part in the final design which is essentially a balance between competing requirements.

4.7.3 Phasing

The horizontal and vertical alignment should not be designed independently. Hazards can be concealed by inappropriate combinations of horizontal and vertical curves and therefore such combinations can be very dangerous. Some examples of poor phasing are as follows:

- *A sharp horizontal curve following a pronounced crest curve.* The solutions are to;
  - Separate the curves;
  - Use a more gentle horizontal curve;
  - Begin the horizontal curve well before the summit of the crest curve.
- *Both ends of the vertical curve lie on the horizontal curve.* If both ends of a crest curve lie on a sharp horizontal curve the radius of the horizontal curve may appear to the driver to decrease abruptly over the length of the crest curve. If the vertical curve is a sag curve the radius of the horizontal curve will appear to decrease. The solution is to make both ends of each curve coincide or to separate them completely.
- *A vertical curve overlaps both ends of a sharp horizontal curve.* This creates a hazard because a vehicle has to turn sharply while sight distance is reduced on the vertical curve. The solution is to make both ends of each curve coincide or to separate them completely.

4.7.4 Junctions and Intersections

The result of an accident is likely to be that one or more vehicles will leave the road. Hence, where possible, a safe ‘run-off’ environment should be created and good sight distances provided. Intersections should therefore not be located on high embankments; near to bridges or other high level water crossings; on small radius curves; or on super-elevated curves. To ensure good visibility, vegetation should be permanently cleared from the area surrounding the junction.

It is also advisable to avoid building intersections on gradients of more than 3% or at the bottom of sag curves. This is because:

- Stopping sight distances are greater on downhill descents and drivers of heavy vehicles have more difficulty in judging them; and
- It is advantageous if heavy vehicles are able to accelerate as quickly as possible away from the junction.

The ideal angle that intersecting roads should meet is 90° because this provides maximum visibility in both directions but visibility is not seriously compromised as long as the angle exceeds 70°.

Where two roads have to cross each other, a simple X-cross junction is adequate for LVRs. However, where possible, it is preferable to provide two staggered T-junctions as illustrated in Figure D.4.5 rather than one X-cross junction since there is unlikely to be a cost penalty in doing so. The most heavily trafficked road is retained as a direct through route. The minor road is then split so that traffic has to enter the major road by making a left turn across the traffic stream onto the major road and then a right turn to re-enter the minor road. This method halves the number of possible manoeuvres where the traffic from the minor road has to cross the traffic stream on the major road. The entry points of the two arms of the minor road should be spaced about 100m apart.

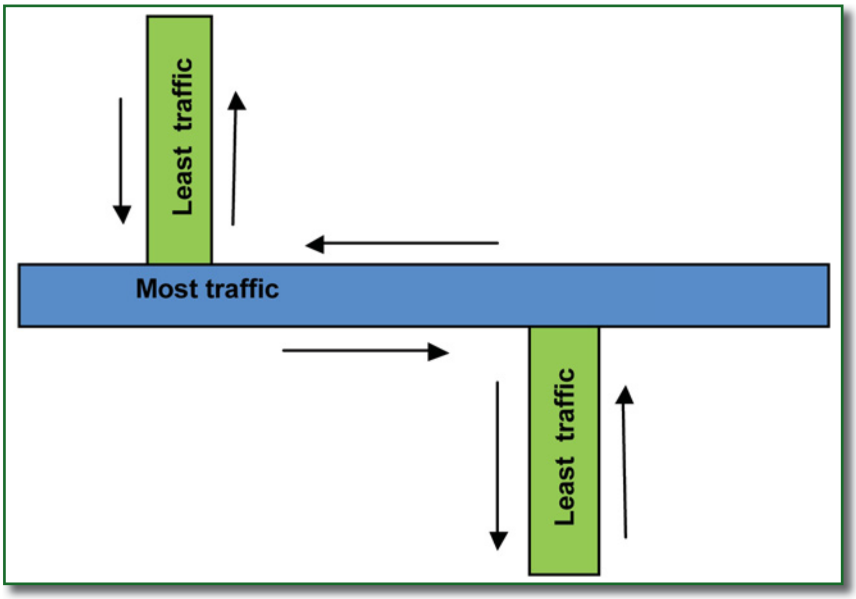


Figure D.4.5: Preferred intersection design

4.8 Safety

The road accident statistics in Ethiopia, in common with many other countries in Africa, show that death rates from road accidents are 30 to 50 times higher than in the countries of Western Europe. The numbers of serious injuries resulting from road accidents are equally alarming. Economic analysis has shown conclusively that this high level of road accidents has economic consequences for the country that is equivalent to a reduction of 2-3% of GDP. This is a very significant drain on the economy. Furthermore, the consequences of the road accidents impose a great deal of grief and anguish on a considerable proportion of the population. Every effort should therefore be made to reduce the number of serious accidents.

The geometric design of the roads has an important part to play in this endeavour and road safety aspects have been highlighted throughout this manual. Road and shoulder widths have been increased to accommodate pedestrians, NMTs, and intermediate forms of transport (IMTs); moderate design speeds have been used for elements of road alignment; parking places and lay-byes for buses have been included in populated areas; account has been taken of reduced friction on unpaved roads; adequate sight distances have been provided; and much more (see Figure D.4.2 for example).

However there are a number of other steps that could be taken to improve safety. These include:

- Traffic calming measures to reduce speeds in populated area;
- Road markings, signage and lighting;
- Segregating pedestrians and motorised vehicles in populated areas;
- Providing crash barriers at dangerous locations;
- Providing a professional safety audit at the design stage.

4.8.1 Traffic calming

The seriousness of road accidents increases dramatically with speed and hence very significant improvements to road safety are possible if traffic can be slowed down. This process is called traffic calming. All such methods have their advantages and disadvantages and the effectiveness of the methods also depends on aspects of driver behaviour that can vary considerably from country to country. Therefore research needs to be carried out in Ethiopia to identify the most cost effective approaches.

The effect of any traffic calming measure on all the road users should be carefully considered before they are installed. Some are unsuitable if large buses are part of the traffic stream; some are very harsh on bicycles, motorcycles and motor cycle taxis; and some are totally unsuitable when there is any animal drawn transport.

The three most common methods are:

- Chicanes;
- Rumble strips; and
- Speed reduction humps.

Chicanes

These are designed to produce artificial congestion by reducing the width of the road to one lane for a very short distance (3-5m) at intervals (typically 300m) along it. They are usually built on alternate sides of the road. They cause drivers to slow down provided that the traffic level is high enough to make it very probable that they will meet an oncoming vehicle. The method is obviously unacceptable if traffic flow is high because the congestion that is causes will be severe. For safety, they must be illuminated at night.

Rumble strips

These are essentially a form of artificial road texture that causes considerable tyre noise and vehicle vibrations if the vehicle is travelling too fast. They are used in two ways. The first is to delineate areas where vehicles should not be. They are effectively a line running parallel to the normal traffic flow so that if a vehicle inadvertently strays onto or across the line the driver will receive adequate warning. Secondly they are used across the road where they are placed in relatively narrow widths of 2 to 4m but at intervals along the road of typically 50 to 200 metres. They are uncomfortable to drive across at speed hence they are usually effective in slowing down the traffic. They do not need to be illuminated at night.

Speed reduction humps and cushions

These are probably the most familiar measures used to slow traffic. They are essentially bumps in the road extending uniformly from one side to the other. Unlike rumble strips, speed reduction humps are quite high and, if they are designed badly, they can cause considerable vehicle damage. They are often used in villages where they are placed at intervals of between 50m and 200m. They are very effective but usually unpopular with drivers.

The shape of the hump is important to reduce the severity of the shock when a vehicle drives over it. Ideally they should cause driver discomfort but not vehicle damage. The height of the bump is usually 50 or 75mm but the width should be at least 1.5m (2.0m is better) and the change in slope from the roadway onto the hump should be gradual. The top of the hump can be rounded or flat. Pipes that are almost buried are completely unsuitable.

Based on a similar principle to the speed hump, speed reducing cushions are more versatile. They are essentially very similar to the speed hump but the hump is not continuous across the road. The width of a two lane road is usually covered by two or three cushions with considerable gaps between them. The idea is that large vehicles will not be able to pass without at least one wheel running over one of the humps but bicycles and motorcycles can pass between them without interference. If suitably designed, the wheels of animal drawn carts could also avoid the humps.

4.8.2 Road markings, signage and lighting

Theft of signs is a problem in some areas and therefore painting signs on permanent features such as buildings, rocks and trees should be considered where necessary.

The extent to which road markings, signs and other road furniture is required depends on the traffic volume, the type of road, and the degree of traffic control required for safe and efficient operation. For low volume roads the primary purpose is to improve road safety hence not all of the features of road furniture and signage described in the ERA's standards (Road Furniture and Markings, Geometric Design Manual-2011) will be used on such roads.

The main elements are:

- Traffic signs provide essential information to drivers for their safe and efficient manoeuvring on the road;
- Road markings to delineate the pavement centre line and edges to clarify the paths that vehicles should follow;
- Marker posts to indicate the alignment of the road ahead and, when equipped with reflectors, provide optical guidance at night;
- Lighting to improve the safety of a road at night time.

4.9 Traffic Signs

Traffic signs are of three general types:

- Regulatory Signs: indicate legal requirements of traffic movement and are essential for all roads;
- Warning Signs: indicate conditions that may be hazardous to highway users;
- Information Signs: convey information of use to the driver.

4.9.1 Warning signs

The physical layout of the road must sometimes be supplemented by effective traffic signing to inform and to warn drivers of any unexpected changes in the driving conditions. Some of the common situations are mentioned below but each situation is unique and the severity of any particular situation can vary considerably. It is therefore recommended that the judgement of an experienced road safety expert is obtained at the road design stage.

For an existing road that is to be upgraded, the hazardous locations should be identified at an early stage and, ideally, should be corrected in the new design. If this is not possible, then suitable road signs should be installed.

The most common situation occurs when the geometric standards for a particular class of road have been changed along a short section of road. This is usually caused by a constraint of some kind that has prevented the standard from being applied continuously and therefore causes an unexpected and potentially dangerous situation. Examples are a sharp bend, a sudden narrowing of the road, or an unexpectedly steep gradient.

A similar situation arises in easy terrain where, despite the fact that the geometric standard of the road has been applied, a hazard such as a bend occurs after a long section of road where drivers are easily able to exceed the design speed of the road by a considerable margin.

As well as changes in the geometric standard of the road, many other relatively unexpected hazards can occur and also need to be signed. For example an unexpected school crossing, a ford or other structure that is not clearly visible from a safe distance – there are many examples too numerous to list. Once again, engineering judgement is required.

A common situation occurs in populated areas where traffic calming measures have been introduced. Speed humps are a particular problem because they are often not sufficiently visible from a reasonable distance, and sometimes they have been badly designed and provide more of a jolt to the vehicle than intended. It is therefore good practice to provide warning signs for these, especially on roads that are likely to be used by traffic unfamiliar to the area. This will include classes DC3 and DC4 and many DC2 roads.

An important consideration on unpaved roads is that the road markings that are used on paved roads to improve safety cannot be used on unpaved roads. This means that if drivers need to be warned of a hazard that is traditionally done by means of road markings, on unpaved roads this will have to be done by means of traffic signs.

4.9.2 Information signs

Information signs are less vital on the lower classes of road frequented primarily by local people. However, for road classes DC3 and DC4 on which a considerable proportion of drivers will not be local, information signs are desirable. They obviate the need for drivers to stop in populated areas to ask questions of pedestrians and hence improve safety, but in most cases this effect is very marginal, especially if the road standards that should be provided in populated areas have been applied. Hence the convenience of some information signs is part of the provision of a particular level of service to the traveller.

4.10 Road Markings

Road markings either supplement traffic signs and marker posts or serve independently to indicate certain regulations or hazardous conditions. There are three general types of road markings in use namely pavement markings, object markings and road studs.

4.10.1 Pavement markings

Pavement markings consist primarily of centre lines, lane lines, no overtaking lines and edge lines. Not all of these are possible, or justified, on low volume roads. However, on a paved, two lane a centre line is desirable. Such a road is not likely to have been built unless the traffic justifies it and hence, for safety reasons, a centre line is recommended.

Other pavement markings such as 'stop', pedestrian crossings and various word and symbol markings may supplement pavement line markings. However, it is obvious that such markings can only be applied to paved roads and then not to all surfacings. In cases where a warning is deemed necessary for safety reasons but road markings cannot be used, road signs must be used instead if applicable.

4.10.2 Object markers

Physical obstructions in or near the carriageway should be removed in order to provide the appropriate clear zone. Where removal is impractical, such objects should be adequately marked by painting or by use of other high-visibility material.

4.10.3 Road studs

Road studs are used on more heavily trafficked roads and in urban areas. They are unlikely to be used on low volume roads but if necessary advice can be found in Road Furniture and Markings, Geometric Design Manual-2011.

4.10.4 Marker Posts

There are two types of marker posts in use namely guideposts and kilometre posts.

Guideposts are intended to make drivers aware of potential hazards such as abrupt changes in shoulder width and alignment, or approaches to structures for example. They are unlikely to be used on a low volume road.

Kilometre posts are a requirement for all trunk and link roads and are therefore only likely to be needed on some roads of class DC4. Details are given in Road Furniture and Markings, Geometric Design Manual - 2011.

4.11 Lighting

Lighting of low volume rural highways is seldom justified except at intersections, railway level crossings, narrow or long bridges, tunnels, sharp curves, and areas where there is activity adjacent to the road (eg markets). Details are provided in Road Furniture and Markings, Geometric Design Manual-2011.

4.12 Safety barriers

Safety barriers are expensive and seldom justified on low volume roads. The geometric design of such roads should be done to eliminate the need for such barriers but sometimes they might be required in highly dangerous situations, for example, on some bends on an escarpment road that cannot be made safe by other means. Expert advice should be sought.

4.12.1 Segregating vulnerable road users

Where possible, non-motorised vehicles and pedestrians should be physically segregated from the motorised vehicles. While this is not specifically part of the geometric design of the road itself, if the terrain and local conditions are suitable for the construction of parallel pathways wide enough for NMTs, then some of the geometric features of the roadway designed to accommodate this traffic will not be necessary and hence considerable savings may be possible. However if traffic does travel on such pathways, sufficient connections need to be made to the roadway itself to enable access in either direction.

4.12.2 Crash barriers

Crash barriers are designed to physically prevent vehicles from crossing them. They are an essential feature of high speed roads to prevent vehicles travelling in opposite directions from colliding with each other head-on, but their primary use on LVRs is simply to prevent vehicles from leaving the road at dangerous places such as when the road comes to its end or where a vehicle could plunge over an edge such as an escarpment. Good geometric design should prevent drivers from experiencing unexpected situations where they might be in danger of losing control but sometimes crash barriers are required, particularly at dangerous points on escarpments. However, they are expensive to install and they must be installed properly otherwise they are not likely to be fit for purpose. They are rarely used on LVRs but could justifiably be used on DC4 standard roads in some circumstances.

4.12.3 Safety audits

The subject of road safety is remarkably complex in that, although many unsafe practices are glaringly obvious, there are many situations where it is difficult to identify what is likely to be unsafe, especially if the project is a new road and one is working from drawings. The history of road safety is full of ideas that were thought to improve road safety but often had no discernable effect or even made things worse. The problem has always been lack of reliable data; there is no substitute for a systematic method of recording the characteristics of road accidents and analysing the data when there is sufficient for reliable conclusions to be drawn.

Professional road safety auditing is the next best thing and is regularly undertaken on every road project in some countries in an attempt to improve the safety design from the very beginning. It is anticipated that this practice will become increasingly common in Ethiopia, especially for road projects located in populated areas.

4.13 Using the standards

There are three design situations namely:

- Upgrading from a lower class of road to a higher class;
- Designing a road to replace an existing track; and
- Designing a completely new road where nothing existed before.

4.13.1 Upgrading an existing road

The basic alignments will already exist but the standards of the existing road should be those applicable to a road of lower class. The new road will require higher standards which may involve a wider cross section, higher design speeds and therefore larger horizontal and vertical radii of curvature. In flat and rolling terrain, larger horizontal radii of curvature are usually achieved by means of minor realignments at the curves themselves. Larger vertical radii of curvature are usually more difficult but, depending on the terrain, can often be achieved by additional fill rather than deeper cutting. In more severe mountainous terrain it may be necessary to make substantial realignments to avoid deep cuts, for example, following a contour more closely to avoid a steep hill with inadequate sight distances over a crest.

The most difficult aspect is likely to occur in mountainous terrain when substantial widening is required. Under these circumstances it may not always be possible to meet the standards of the new road class and therefore adequate warning signs will need to be employed to alert drivers to the lower standards.

In general, however, the main improvements, apart from overall widening, are essentially spot improvements and do not require sophisticated design methods.

4.13.2 Designing a road to replace an existing track

In this case the existing geometric standards will be very much lower than those required hence some substantial re-alignments may be necessary, especially in hilly and mountainous terrain. However, the basic route selection has been carried out by virtue of the fact that there is an existing track and the main control points along the alignment will already be defined. Although re-alignments may be substantial, an experienced engineer could adopt a design-by-eye approach in many cases. However, it is anticipated that, in general, the designs will be done with the help of computer programs based on accurate topographical and other survey data (see Chapter 2 on route selection).

4.13.3 Designing a new road

Designing a geometric alignment for an entirely new road where nothing existed before is a considerably more complex process because of the many different route alignments that are possible and the relative lack of information available at the beginning of the process. In many cases there will need to be a pre-feasibility study to identify possible corridors for the road and to decide whether the project is likely to be viable. This will then need to be followed by a feasibility study to determine the best routes within the best corridors and, finally, a detailed design study based on the route selected. The level of detail in this process depends critically on the class of road being designed and the terrain through which it will pass. Errors at this stage can be costly and, once the road is built, can also impose serious burdens in the future if the road requires excessive maintenance.

The principles of route selection are described in detail in Chapter 2. They are based on surveys of various kinds that provide information about all the likely technical engineering issues related to the new road but also surveys concerned with environmental and social issues as well.
The final design will inevitably be a compromise between many competing factors and there is no formal way of resolving all of them to everyone's satisfaction. Engineering judgement and consensus will be required to arrive at a satisfactory alignment.

5. DRAINAGE

5.1 Introduction

Road drainage design is the general term that is applied to two separate topics namely:

- **Internal road drainage.**
  The process of minimising the quantity of water that remains within a road pavement by maximising the ability of the road to lose water to an external drainage system. Sometimes this definition also includes minimising the quantity of water that gets into a road pavement in the first place.
- **External drainage.**
  This consists of three components:
  a. The process of determining the quantity of water that falls upon the road itself and its associated works that needs to be channelled away from the road by the drainage system. This is water that falls upon the road as rain.
  b. The process of determining the quantity of water that flows in the streams, rivers and natural drains that the road has to cross. This is water that falls as rainfall at locations away from the road.
  c. Design of the individual engineering features of the drainage system to accommodate the flow of water.

This Chapter is concerned with the external drainage system and the drainage standards for roads carrying less than 300 two-axled (and larger) motorised vehicles per day. The Chapter is essentially a guide containing appropriate technical explanations of all the steps in designing the surface water drainage system for LVRs.

Internal drainage is considered in Chapter D.6, Section 6.18 as part of the chapter on pavement design. This Chapter does not deal with route surveying, site investigations, route selection or the actual structural design of bridges and major water crossings; these topics are dealt with in other sections of this manual or in the ERA 2011 manuals. The planning and structural design of river crossings of less than 10m span and drainage structures for roads being considered in this Manual is given in Part E.

Neither rainfall nor rivers distinguish between roads carrying low and high volumes of traffic. Therefore, the basic costs of protecting a road from the effects of water are essentially the same and largely independent of traffic. Hence, for LVRs the cost of the drainage system can comprise a larger proportion of the costs of the road.

There are, of course, different levels of protection associated with the risk of serious damage to the road. For principal trunk roads little risk can be tolerated and so expensive drainage measures must be employed. For LVRs the consequences of failure in the drainage system are correspondingly lower but, within the range covered by LVRs, there are some significant differences depending on the length of the road and the availability of an alternative route.

The challenge for the engineer is to choose a level of protection that is commensurate with the class of road and the consequences of drainage failure. Thus a certain amount of engineering judgement is required and a design manual such as this requires a consensus amongst road professionals.

Unfortunately, although it is possible to define the probability of specific storm events from extensive rainfall records, if such records are available, it is practically impossible to define the overall level of risk inherent in a drainage system design itself. This is because there are so many other factors that influence its performance. First of all, simply calculating the water volumes flowing in the drainage system following a specific storm involves several important assumptions.

Secondly, the drainage system is not a fixed, unchanging system despite every effort by the designer to protect it and to make it so. Changes are always occurring as a result of aspects such as sedimentation, erosion, the transport of debris, growth of vegetation and landslides. For example, sedimentation will always occur in some places within the drainage system. This affects water flow and drainage capacities in complex ways. Partial blockage by debris or landslides, a particularly important problem in mountainous areas, can quickly lead to full blockage and catastrophic failures unless cleared by maintenance activities.

Erosion is also a formidable enemy of the drainage designer. Very erodible soils can be found extensively in many parts of Ethiopia and catastrophic levels of erosion can arise from small perturbations in the smooth flow of water leading to failure of the drainage system.

Naturally the designer attempts to minimise these affects but the effectiveness in doing so is directly related to the cost and to the effectiveness of maintenance (also a function of cost). Hence different levels of risk, and therefore cost, are applied to roads of different standard. This is the designers challenge because such levels of risk are numerically very difficult to define.

5.2 Summary of standards and departures from standards

5.2.1 Design standards and storm return period

Once the drainage design has been completed, and provided maintenance is carried out to remove potential blockages and repair minor damage, a road drainage system should operate successfully for many years. However, drainage systems cannot be designed for the very worst conditions that might occur on extremely rare occasions because it is too expensive to do so. The various standards for the design of drainage are based on different levels of risk that are attached to the likely occurrence of the different storm intensities for which they are designed, assuming that appropriate routine maintenance is carried out.

Storm events are defined by the intensity and duration of rainfall and are extremely variable in nature over periods of many years. Thus a statistical distribution of storm severities shows that very severe storms are quite rare and that less severe storms are more common. The risk of a severe storm occurring is defined by the statistical concept of its likely *return* period which is directly related to the probability of such a storm occurring in any one year. Thus a very severe storm may be expected, say, once every 50 years but a less severe storm may be expected every 10 years.

This does not mean that such storms will occur on such a regular basis. A severe storm expected once every 50 years has, on average, a probability of occurring in any year of 1 in 50 (or 0.02 or 2%). Similarly a storm of lower intensity that is expected to occur, on average, once every 10 years has a probability of occurring in any one year of 1 in 10 (or 0.1 or 10%). The operative words here are "on average" and it is salutary to realise that there is always a finite probability that the worst storm for 200 years may occur tomorrow.

Most drainage structures are likely to be severely damaged if their capacity is exceeded for any length of time hence their capacity is the most important aspect of their design. In general, the more severe the storm for which the structure is designed, the more expensive it is to build; and the cost of designing for the highest possible storm severity (ie zero risk) is prohibitive. Drainage standards are therefore defined by the level of risk. This is done using the concept of return period of the maximum storm for which they are designed.

There are three factors that determine the level of risk that is appropriate for each structure namely;
- The standard of the road (ie the traffic level);
- The cost of the drainage structure itself;
- The severity of the consequences should the road become impassable because of a failure of the drainage system.

If a drainage structure on a road carrying high levels of traffic is damaged or fails completely, the disruption and associated costs to the traffic can be very high and therefore the structures on such a road are designed for low risk (ie for storms of long return periods). They are therefore relatively expensive. On the other hand, if a drainage structure should fail on a road carrying low levels of traffic, the likely disruption to traffic and the associated costs are correspondingly less and hence the higher cost of designing the drainage for low risk cannot be justified. The drainage is therefore designed for shorter storm return periods.

Similarly, the cost of replacement or repair of large and expensive drainage structures is high and therefore they are designed to minimise this risk by designing for very severe storms (ie storms with long return periods). This increases their cost but reduces the risk of damage. Higher risks can be tolerated for smaller and less expensive structures that are usually easier to repair; hence these are designed for less severe storms (ie shorter return periods).

An overriding principle for the designer is to consider the consequences of a drainage failure. In situations where the road is relatively short and an alternative route, albeit a longer one, is available, the social and economic consequences of a drainage failure that makes the road impassable for any length of time are not high. In contrast, there are also many situations in Ethiopia where there is no alternative route at all or, if there is one, it is very long. Under these circumstances additional expenditure to reduce the risk of such an occurrence is justified. This is done by designing for a larger storm (ie a longer storm return period).

It is difficult to calculate the exact trade-off between the cost of designing for low risk and the costs and consequences of failure of a drainage structure. Furthermore, the precision with which design storms can be calculated depends on the availability of detailed rainfall data that are required to have been collected over a period of many years. Even with good rainfall data, there are other uncertain assumptions that need to be made in carrying out the calculations. Thus, in most situations the accuracy of the calculations of the required water flow capacity is not very high despite the apparent sophistication that is apparent in some methods of drainage design and it is therefore prudent to include a factor of safety.

Because of these issues the drainage standards can only be based on a review of practices throughout the world combined with local engineering judgement and consensus. Table D.5.1 indicates the design standards for LVRs in Ethiopia. For strategic routes, routes of very high economic or social importance or if the alternative route in the event of a drainage failure is more than an additional 75km or if there is no alternative route suitable for vehicles, Table D.5.2 should be used instead.

Table D.5.1: Design storm return period (years)

Structure type	Geometric design standard			
	DC4	DC3	DC2	DC1
Gutters and inlets	2	2	2	1
Side ditches	10	5	5	2
Ford	10	5	5	2
Drift	10	5	5	2
Culvert diameter <2m	15	10	10	5
Large culvert diameter >2m	25	15	10	5
Gabion abutment bridge	25	20	15	-
Short span bridge (<10m)	25	25	15	-
Masonry arch bridge	50	25	25	
Medium span bridge (15 – 50m)	50	50	25	-
Long span bridge >50m	100	100	50	-

Table D.5.2: Design storm return period (years) for severe risk situations

Structure type	Geometric design standard			
	DC4	DC3	DC2	DC1
Gutters and inlets	5	5	5	2
Side ditches	15	10	10	5
Ford	15	10	10	5
Drift	15	10	10	5
Culvert diameter <2m	25	20	20	10
Large culvert diameter >2m	50	25	20	10
Gabion abutment bridge	50	25	20	-
Short span bridge (<10m)	50	50	25	-
Masonry arch bridge	50	50	25	
Medium span bridge (10 – 50 m)	100	100	50	-
Long span bridge >50m	100	100	100	-

5.2.2 Methods of design

An international review of LVRs indicated a wide range of practices. The simplest methods are usually described in manuals specifically written for LVRs. These all use relatively short storm return periods to keep costs low. The simplest method of all is essentially a stage construction process whereby simple rules of thumb are used for the initial design with little or no calculation. The road is built and then, in the following year or two, problems that arise where there is inadequate capacity in the drainage system are rectified as quickly as possible. Such an approach is normally used only for very low volume roads such as DC1 and may be applicable if the engineering resources are readily available during the required period following initial construction.

Very few national standards specifically address the problem of designing drainage for LVRs. The implication is that the methods used for all roads should be applied to LVRs but this is impracticable. The methods described in this manual range from the simplest approach appropriate to the lowest standards up to more comprehensive methods that could be used whenever sufficient data are available. The manual does not include the full range of methods suitable for the higher road classes.

5.2.3 Departures from standards

It is fundamental to the concept of setting standards that they should be applied at all times. However, the basic standards for drainage structures and drainage design cannot be precisely defined because sufficient data may not be available to carry out the designs in the ideal way. As a result, the designer must use simpler and apparently less accurate methods. Furthermore, even if data are available to allow more sophisticated methods to be used, there are worrying large differences in the results that the various methods give. Thus whether sophisticated numerical methods or simple methods are used, different answers will arise from the various methods. The question therefore arises as to what is the actual standard; what really is the true design storm for the selected return period? There is no answer to this question. All that can be done is for the designer to use the methods available and to exercise a degree of engineering judgement in selecting the result for the design.

The same arguments do not apply to the detailed engineering design of the components of the drainage system once the maximum water flow has been estimated. For these, standard drawings and specifications are provided. If the designer wishes to depart from these, then written approval will be required from

ERA. The designer must submit all proposals for departures from standards to the appropriate client officer for evaluation as described in the Preface.

5.3 Hydrology: estimating maximum flow for drainage design

Before a drainage structure can be designed, it is necessary to determine the maximum likely flow of water to be accommodated by the structure. Information may be needed on:

- Water catchment area;
- Rainfall characteristics;
- Topography;
- Vegetation and soils;
- Catchment shape;
- Stream and river flows if available;
- Available storage in lakes and swamps;
- Rural and urban development plans;
- Water management plans (eg river basin master plans).

5.3.1 General principles

If the capacity of a water crossing structure is less than the volume of water flowing in the water course, the upstream water level will rise. If this happens the water will flow through the structure at high speed. This can erode the bed and banks of the water course. Furthermore, a high lateral pressure will be exerted on the structure which can cause it to move and collapse. Eventually upstream water levels may rise to the extent that water overtops the structure, damaging the road and blocking access. Water crossing structures must therefore be designed to have a capacity equal to or greater than the maximum water flow that is expected in the water course. This maximum flow depends on the rainfall pattern in the area concerned and the characteristics of the particular catchment area on which the water, which will eventually pass under the structure, falls.

Maximum flow is also important for the design of submersible structures such as drifts and causeways. Although water flows over the surface, damage can still occur if the water rises higher than the level of the approach slabs.

5.3.2 Storm severity and maximum flow

The cost of a drainage structure increases as its capacity increases; hence the first step is to determine the maximum flow for which it is to be designed. To do this it is first necessary to decide on the return period of the design storm for the structure (see Section 5.2.1). Table D.5.1 and D.5.2 show the return periods for the design of typical structures on LVRs in Ethiopia.

Deciding on the return period defines the basic standard of the structure. The next step is to determine how much water the structure must cope with when a storm of the design return period occurs. This depends on:

- The characteristics of the storm itself, namely the intensity, duration and the spatial extent of the rainfall; and
- The characteristic of the ground, or catchment, on which the rainfall falls.

Rainfall and storm characteristics depend on many factors and consequently vary widely between areas of the world and within countries. The details of storms cannot be calculated theoretically and therefore all rainfall information is based on experimental evidence. Thus, in order to determine the characteristics of storms of a particular severity for drainage design purposes, good rainfall data need to be available. Such data must include intensities and durations of all rainfall events over a period of time that is considerably longer than the return period being used. If not, the number of occurrences of the return period storm will be too low for statistically reliable information about its characteristics to be determined. In many countries rainfall records are either not detailed enough or the records do not go back far enough for accurate calculations. This is currently the situation in most of Ethiopia although the situation is improving all the time. However, for LVRs, the situation is usually better because the return periods for design are shorter than for the higher standard roads and there is a greater chance that rainfall data will be adequate.

The characteristics of the storm are only the first part of the problem as these determine how much rainfall falls onto the ground. The characteristics of the catchment area then determine how much of this rainfall actually gets to the drainage structure and how long it takes to arrive. The situation is relatively simple when dealing with the rainfall that runs off the road itself because the surface is uniform; its run off characteristics are well understood; and the catchment area is known accurately. Where the catchment is for a stream or river, the surface can be very variable with a variety of different run off characteristics, thereby making run off estimation less accurate.

Provided rainfall records are available, there are several methods of calculating the maximum flow that the drainage structures must cope with. Only one of them, the 'Rational Method' is really suitable for LVRs and this is described below. However, before presenting details of this, there are several simple observational or empirical methods of estimating the maximum flow in an existing water channel that are suitable for LVRs although they need to be used with considerable care. Preferably as many of these simple methods as possible should be used at the same time to provide the maximum level of reliability.

5.3.3 Simple estimation methods for maximum flow

The maximum water flow can be estimated in a number of simple ways requiring common sense and observational techniques. The methods are based on:

- Direct observation of the size of the watercourse;
- Direct observation of erosion and debris;
- History and local knowledge;
- The Rational Method;
- Correlation Tables;
- The SCS method (US Soils Conservation Service, TR-55)
- Successful practice.

Direct observation of the size of the watercourse

Most watercourses enlarge naturally to a size sufficient for the maximum water flows and no further. Thus a simple way to design the structure is to measure the cross-sectional area of the water course itself. The total cross-sectional area of the apertures of the structure should be equal to that of the water course so that the flow is not constricted. Then upstream levels will not rise, and erosion, overtopping and collapse will not occur.

This method assumes that the watercourse is free to drain and flows at a speed determined by the gradient and nature of the channel. If, however, the water is flowing very slowly, for example, if the water course is backing up from downstream, a reduced cross-sectional area will not cause upstream levels to rise. In this case the structure does not need to have a cross-sectional area equal to that of the water course. Further investigation is required to determine the maximum water level when water is draining freely, but it is possible that a single aperture may be sufficient to ensure that the water is able to gradually drain away as the downstream level falls.

The method also assumes that flood water levels do not rise beyond the watercourse and spread across the surrounding land. If this is the case it is recommended that the total cross-sectional area of the apertures of the structure should remain equal to that of the watercourse and that a series of smaller structures, such as drifts, are constructed along the road on either side of the main structure.

The third assumption of this method is that the channel has not grown to its current size after years of erosion by a much smaller flow of water. If this is the case, the channel size is not a guide to the maximum water flow and is therefore not equal to the required total cross-sectional area of the apertures of the structure. Additional investigation is required to check this.

This method must be supplemented with interviews with local residents.

Direct observation of erosion and debris

Although most water courses enlarge to a size sufficient for maximum water flows, these maximum flows may be very rare with a return period much longer than that for which some of the smaller structures may be designed for LVRs In this case it is more sensible to find indications of typical high water levels. These include lateral erosion on the banks of the water course and debris caught in the branches of trees.

It is unlikely that erosion or debris will remain visible for many years after a rare storm and therefore their presence is a useful indicator of high water levels over a relatively short period of several years, similar to the return period for which some of the smaller structures may be designed. As described above, the total cross-sectional area of the apertures of the structure should be equal to that of the water course up to the level of the erosion or debris.

This method must also be supplemented by interviews with local residents.

History and local knowledge

As suggested by the above two methods, past high water levels can be a reliable way of predicting future levels. Past levels can be indicated either by evidence from the water course itself (as above) or by recorded history, whether in measurements actually taken or from the recollections of local residents.

Measurements may be available but probably only if the site has been of interest to the authorities for some time. This is unlikely to be the case for small water courses in most rural areas. Alternatively, rural people tend to live in the same area for many years and have good memories of significant local events, floods normally being very significant to those reliant upon the land. However, caution is required against both the exaggeration of past levels and the belief that a single high flood many years ago was normal rather than exceptional. Evidence should be sought from as many local residents as possible. It is usually a good idea to make individual enquiries from people living on both banks of a river or stream and from several locations along it. Alternatively a group may be asked to collectively agree on a maximum height of the flood water and to agree on the frequency that such floods occur.

As in methods 1 and 2, it must be confirmed that the high water levels were for water which was flowing rather than backed-up and stagnant. It must also be confirmed that water usage hasn't changed since the measurements were made. New upstream dams and land clearance can reduce or increase wet season flows significantly.

Assuming that one can be confident that they were carefully made and that water usage has not changed, recorded measurements are the most reliable way of predicting future levels. As described above, the total cross-sectional area of the apertures of the structure should be equal to the cross-sectional area of the water course up to the recorded or remembered level.

The Rational method

A number of numerical methods are available for estimating the maximum water flow which can be expected in a water course. All methods require rainfall records and details of the catchment from which rainfall flows into the water course, but some are too complex for use for LVRs. The Rational Method is the most straightforward of these methods. It has a number of advantages and disadvantages shown in Table D.5.3.

Table D.5.3: Advantages and disadvantages of the Rational Method

Advantages	Disadvantages
Simple equation; rapid estimation	Not as accurate as more detailed methods (but within 30%)
Very little data are required	Often over-estimates the flow (depends on catchment size)
	The simplest form should not be used for catchment greater than 3km^2 but adjustments can be made for larger catchments (see below)
	Requires national records of storm duration and intensity

The Rational Method estimates the maximum cross-sectional area of a water course by dividing the maximum expected volume of water, q (m^3/s), by the likely speed of the water, v (m/s).

$$q/v = a \ (m^2) \qquad \textbf{Equation 1}$$

Where:

a = the cross-sectional area (m^2) of the water when the water course is in flood. The total cross-sectional area of the apertures of the structure should be equal to or greater than a

q = the maximum expected volume of water (m^3/s)

q is calculated from the following equation.

$$q = 0.278 \times c \times i \times A \ (m^3/s) \qquad \textbf{Equation 2}$$

Where:

c = the catchment coefficient
i = the intensity of the rainfall (mm/hour)
A = the area of the catchment (km^2)

Details of the method are provided in Part B, Section 7.2.

Flow velocity and size of the drainage channel

Where appropriate, for LVRs, drainage structures that are not sensitive to exact predictions of flow such as fords and drifts should be used, rather than culvert pipes and similar structures that have a fixed size. However, where this is not appropriate, the required size of the drainage channel needs to be calculated. This depends on the maximum flow at the location of the structure namely the value of *q* (m^3/s) calculated in equation 2. The maximum flow must be equal to the area of the channel multiplied by the likely velocity of flow and hydraulic principles may have to be used to ensure that likely velocity is sufficient and the design of the structure is adequate.

For example, for culverts in rolling and mountainous terrain it is often relatively simple to make sure that their size is adequate by ensuring that their slop is 3-5% flow is not restricted on either the upstream of downstream side. However, to ensure that the culvert is not significantly over designed it is usually necessary to use hydraulic principles. The hydraulic design of drainage structures is dealt with in Part E of this manual.

Correlation tables

It is possible to simplify the Rational Method into a correlation Table between catchment area and the required total cross-sectional area of the structure. This is done for the conditions that prevail in a particular region. The Table can then be used to design all structures in the same region in the future.

In order to produce such a correlation table, the Rational Method must be used for a number of typical situations and the following assumptions must be reasonably correct:

- The storm intensity-duration pattern applies in the area in which a structure is to be constructed;
- All catchments in the region are similar, for example, all are undulating with average soil and average vegetation cover;
- All similar structures are to be designed for the same return period.

Additional sophistication can be provided by developing separate correlation Tables for structures that require different return periods. If necessary these can also be repeated for catchments with different surface cover. Table D.5.4 is an example of a typical Table correlating the catchment area with required total cross-sectional area of the structure for a set of average conditions.

Finally, the Table can be extended to show the correlation between catchment area and the required number of standard culvert apertures. For example, assuming that a standard aperture is 600 mm wide, with 600 mm walls onto which a semi-circular arch is constructed, its cross-sectional area is 0.50 m^2 (Figure D.5.1)

Table D.5.4: Catchment area and total cross-sectional area of an example structure for a set of standard conditions based on the Rational method

Catchment area (ha)	Required cross-sectional area (m^2)	
	C=0.2 (rolling terrain, highly permeable cultivated soil)	C=0.7 (rolling terrain, arid area, low permeability soil)
10	0.35	1.0
25	0.75	2.0
50	1.25	3.5
75	1.75	4.7
100	2.2	
125	2.6	
150	3.0	
175	3.5	

Note:

The rainfall intensity is assumed to be 100mm/hr. This table is not applicable for higher intensities

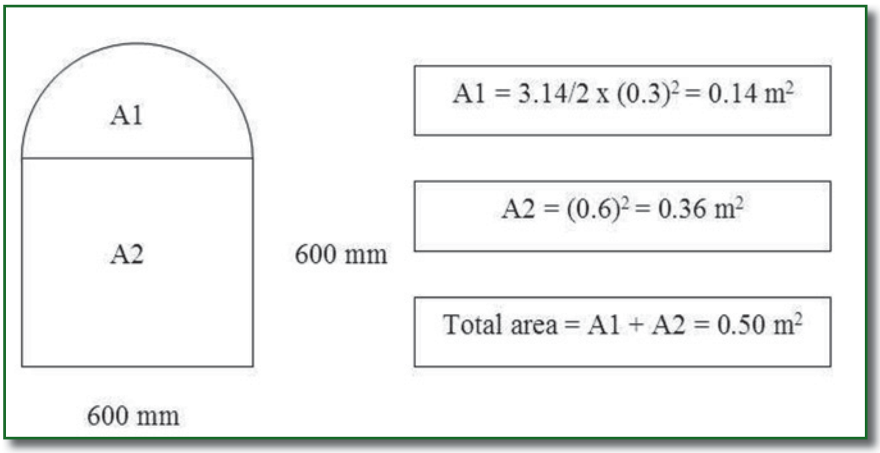


Figure D.5.1: Example of a standard arch culvert

The full correlation Table between catchment area, required cross-sectional area and the required number of standard apertures can now be completed (Table D.5.5). When more than three standard apertures are required, it is recommended that the structure is designed in more detail. Although a series of standard apertures may be the most suitable solution in a wide, shallow channel, if the channel is narrow and deep, a box culvert, larger diameter pipes or even a small bridge may be more appropriate and cost effective. The decision between these alternatives should be made carefully.

Table D.5.5: Catchment area, total cross-sectional area and number of standard arch culverts for a set of standard conditions based on the Rational Method

Catchment area (ha)	Required cross-sectional area (m^2) C=0.2	No. of standard apertures	Required cross-sectional area (m^2) C=0.7	No. of standard apertures
10	0.35	1	1.0	2
25	0.75	2	2.0	4
50	1.25	3	3.5	Carry out more detailed design
75	1.75	4	4.7	
100	2.2	Carry out more detailed design		
125	2.6			
150	3.0			
175	3.5			

The SCS method

The SCS method for calculating rate of run off requires much of the same basic data as the Rational method namely catchment area, a runoff factor, time of concentration and rainfall. However, the SCS method also considers the time distribution of the rainfall, the initial rainfall losses to interruption and storage and a filtration rate that decreases during the course of a storm. It is therefore practically more accurate than the Rational method and is applicable when the catchment area is larger than 50 hectares.

The details of the SCS method are provided in Part B, Section 7.3

Successful practice

This method is similar in many respects to the correlation method above, except that instead of using a theoretical basis for the correlation Table, the success of previous designs is used. Thus if a high proportion of structures along a road or in a region have been in operation for a number of years without overtopping or being damaged during wet season floods, it is reasonable to assume that the relationship between catchment area, catchment characteristics, rainfall intensity and maximum water flow that was used to design the structures, even if extremely simple, is reasonably valid. The only proviso is that care must be exercised to ensure that all the structures have not been over-designed.

Within a region, catchment characteristics and rainfall intensity are normally consistent therefore it can be assumed that a simple relationship between catchment area and maximum water flow also exists. If the area of a number of catchments and the total cross-sectional area of the apertures of the structures are measured, the relationship between them can be established in the form of a Table as shown above. For new designs, it is a simple matter of measuring the area of the catchment and using the Table to establish the required cross-sectional area, although it is necessary to be cautious if the catchment in question is different in topography, soil or vegetation to the catchments used to derive the Table.

5.3.4 Normal flow rates

Whilst information is required on the severity of different storm events in order to design the drainage structures, for some structures it is also necessary to know something about the "normal" or less severe flows that are likely to occur during each year.

For example, drifts and fords are designed for water to flow over the running surface and it is not expected that vehicles can use them for 24 hours a day and for 365 days per year. For such structures, annual rainfall records still need to be examined to determine the rare storms that could engulf the whole structure and put it at risk, but under normal operating conditions water is expected to cover the "carriageway" or running surface at much more frequent intervals. there will be days in the year where the water will be too deep for several hours and it is necessary to design the structure so that it is not impassable for more than, **on average**, a specified number of hours each year. If the volumes of water are too great for a simple ford, then a vented ford of some other structure will need to be considered.

Similarly, the prevention of erosion damage caused by the "regular" flow of water through a structure requires information about the range of flow velocities that occur every year. In other words, slow regular attrition can be as damaging in the long term to some structures as the rare severe storm.

Whilst the hydrological principles described in this chapter apply to these conditions, the hydraulic designs for the various structures are dealt with in Part E.

5.3.5 Direct flow methods

Methods of estimating flow discharges are divided into three categories;

- Direct flow methods;
- Run-off modelling;
- Regionalised flood modelling.

Methods based on simple empirical assessments of stream flows and local recollections, are examples of direct flow methods. They are clearly prone to considerable uncertainty and not suitable for estimating long return period flows.

However, the same statistical techniques that are used to determine rain storms of different return periods can also be applied to direct flow records of streams and rivers. Direct flow data or depth gauge data is often available for regional and local streams and rivers and, together with measurements of the channel geometry, can be used in conjunction with Manning's Equation to determine flow velocity and thus flow volume (discharge, or capacity) (see Part E). Statistical analysis can then be used to determine flows of different return periods. As with rainfall data, the period during which such records have been kept needs to be reasonably long for statistical reliability. If such data are available, direct flow methods are usually the most accurate method of estimating flows for different return periods, especially for large catchments where run-off modelling is not appropriate.

The Rational Method is an example of run-off modelling and, together with rainfall data and the statistical analysis of storms provides a method of estimating longer return period flows for small catchments as described above.

Flood modelling is the least accurate method and is not discussed further.

5.4 Components of external drainage

An effective external drainage system must fulfil several functions:

- Prevent or minimise the entry of surface water into the pavement;
- Prevent or minimise the adverse effects of sub-surface water;
- Remove water from the vicinity of the pavement as quickly as possible;
- Allow water to flow from one side of the road to the other.

This must be achieved without endangering the road or adjacent areas through increased erosion or risk of instability.

Thus an external drainage system consists of several complementary components;

- Surface drainage to remove water from the road surface quickly.
- Side drainage to:
  - Take water from the road;
  - Prevent water from reaching the road.
- Turnouts to take the water in the side drains away from the road.
- Cross drainage to allow the water in the side drains, and from any other sources, to cross the road line by channelling it under or across the road.
- Interceptor drains to collect surface water before it reaches the road.
- Sub-surface drains to cut off sub-surface water and to lower the water table when required.
- Erosion control (often simple scour checks) to slow down the water in the side drains and prevent erosion in the drains themselves and downstream of drainage outlets or crossings.

All these types of drains have to work together in order to protect the road from being damaged by water. Cross-drainage includes structures to allow permanent or seasonal water courses to cross the road line and therefore includes bridges. The appropriate structures for low volume roads are dealt with in Part E of the manual.

5.4.1 General principles

Conservation of the natural drainage system around the road alignment is one of the most important concerns during design and construction. By effectively creating a barrier to natural surface drainage that is only punctuated at intervals by constructed drainage crossings, road construction can lead to significant local increases in catchment areas and increased water flows. Furthermore, in the case of paved roads especially, road drainage reduces the time taken to reach maximum flow by shedding water from impermeable surfaces relatively quickly. Therefore, in addition to constructing a drainage system to convey the design run-off without surcharge, blockage by sediments, or scour, attention must be paid to strengthening those parts of the natural slope drainage system that experience increased run-off, and hence erosion potential, as a result of road construction. The main ways of doing this are to:

- Control road surface drainage;
- Design culverts or drifts that convey water and debris load efficiently;
- Optimise the frequency of drainage crossings to prevent excessive concentration of flow;
- Protect drainage structures and stream channels for as far downstream as is necessary to ensure their safety and prevent erosion of land adjacent to the water course;
- Plant vegetation on all new slopes and poorly-vegetated areas, around the edges of drainage structures and appropriately along stream courses, without impairing their hydraulic efficiency or capacity.

5.4.2 Sources of water

The main sources of water ingress to, and egress from, a typical pavement cross-section are listed in Table D.6.20. This chapter is concerned with dealing with the water that flows outside the road prism. Minimising the amount of water that gets into the structure of the road itself and minimising the damage that it can cause are dealt with in Section D.6.18

5.4.3 Road surface drainage

Camber and cross-fall are part of the geometric design of the road. Their values are discussed in the geometric design chapter of this Manual (Chapter D.4) but are repeated here for completeness.

For earth and gravel roads, the design cross-fall or camber should be a minimum of 4% and normally about 6%, except in arid areas. This helps to prevent ponding on slack road gradients and longitudinal scour on long, steep sections of alignment. In service, the cross fall should not be allowed to decrease to less than 3% before maintenance is carried out to restore it. For paved roads cross-fall should be 3% except on super-elevated sections. On structures the cross fall can be relaxed to 2-5% on condition that the surface is durable and positive drainage (eg through scuppers) is provided and regular maintenance is carried out.

In hilly, mountainous and escarpment terrain, an inward-sloping road carriageway (see cross sections in Chapter D.4) is the normal means of shedding water from the road surface. The inward slope incorporates an inherent factor of safety in retaining water that has accidentally escaped from the drainage system. Occasionally, an outward-sloping road surface has been advocated on the grounds that, by allowing water to disperse gently onto the hill slope along the whole length of the road, the potential for erosion is reduced. In practice, the opposite is usually true. The method undoubtedly offers very large financial savings in the reduction of drainage structures, but it is a highly hazardous form of design that cannot be recommended except in areas of very low erosion potential. The design has the following weaknesses:

- In practice it is impossible to design a road geometry for a distributed flow of water (topography is the controlling factor).
- Road settlement and the action of traffic, will, in time, change the design cross-fall
- Road repairs will locally alter the cross-fall.
- Partial blockage of the road by debris results in a change of the flow pattern of drainage water, and instant local surcharge.
- The outer edge of the road is particularly susceptible to erosion which can reach a disastrous level before maintenance crews can be mobilised.
- Slope protection from uncontrolled runoff requires a lengthy period of post construction monitoring and remedial works that is usually not practicable for LVRs.
- Vehicles can slide sideways uncontrollably across a wet road surface and over the edge unless expensive partial safety barriers are provided.

An outward sloping cross section is therefore only suitable for pedestrian, NMT and IMT use and where the longitudinal gradient is low and the surfacing material has low erosion potential.

5.4.4 Side drains

Side drains serve two main functions namely to collect and remove surface water from the immediate vicinity of the road and, where needed, to prevent any sub-surface water from adversely affecting the road pavement structure.

Seepage may occur where the road is in cut and may result in groundwater entering the sub-base or subgrade layers as illustrated in Figures D.5.2 and D.5.3. Inadequate surface or subsurface drainage can therefore adversely affect the pavement by weakening the soil support, and initiating creep or failure of the downhill fill or slope. Localised seepage can be corrected in various ways but seepage along more impervious layers, such as shale or clay, combined with changes in road elevation grades, may require subsurface drains as well as ditches as shown in Figure D.5.4.

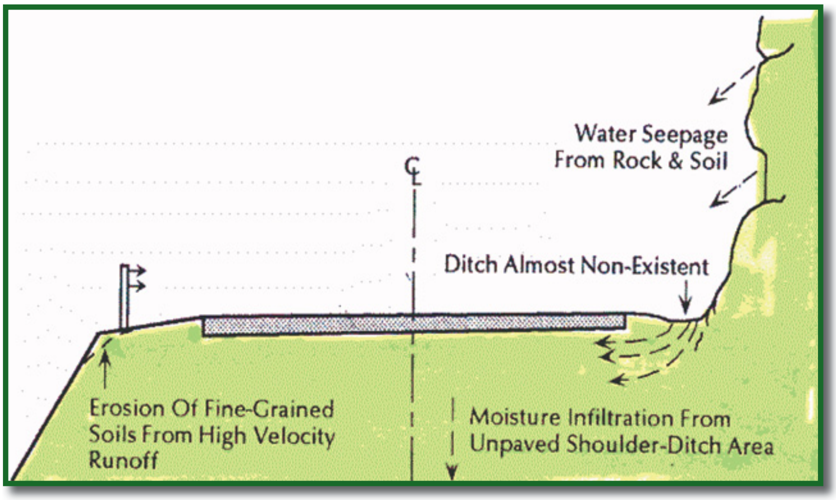


Figure D.5.2: Inadequate side drains

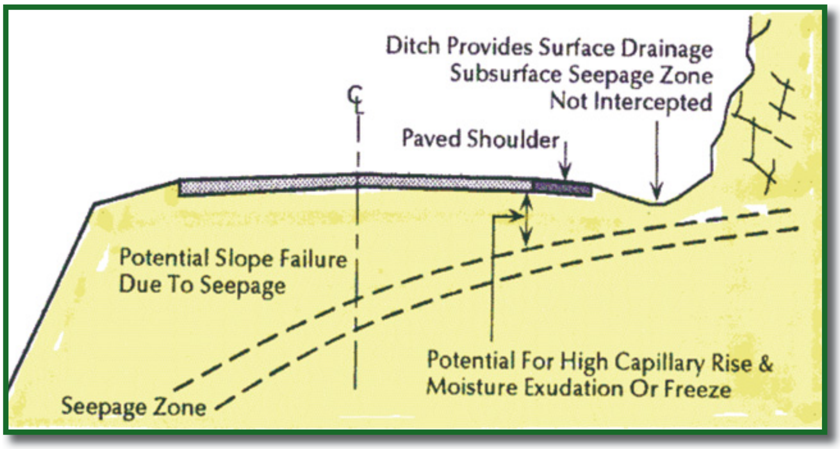


Figure D.5.3: Inadequate side drains and subsurface drainage

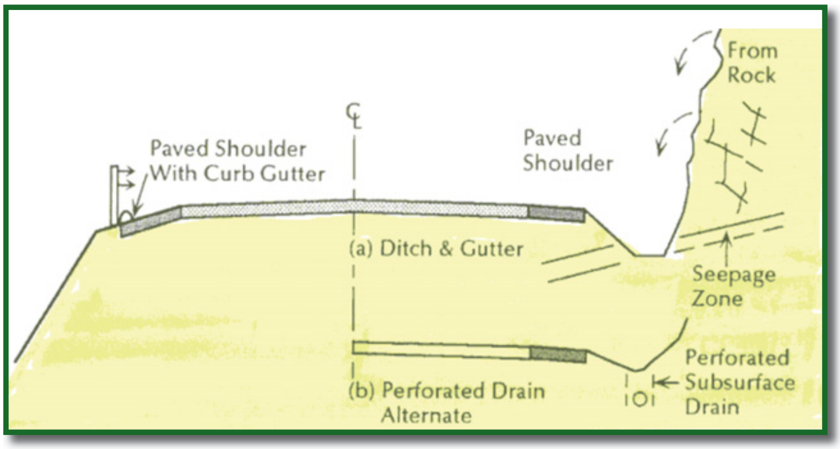


Figure D.5.4: Proper interception of surface runoff and subsurface seepage

If the road has effective side drains and adequate crown height, then the in situ subgrade strength will stay above the design value. If the drainage is poor, the in situ strengths will fall to below the design value. Crown height is discussed in Section D.6.18.

Side drains can be constructed in three forms (Figure D.5.5): V-shaped, rectangular or trapezoidal.

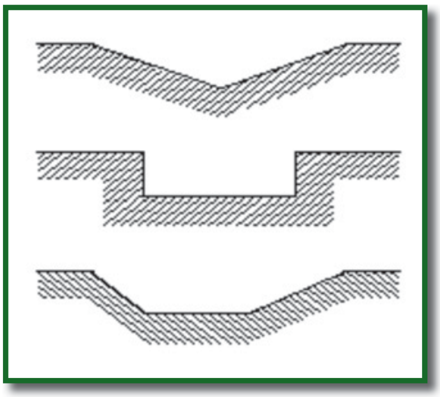


Figure D.5.5: Side drains

The choice of side drain cross-section depends on the required hydraulic capacity, arrangements for maintenance, space restrictions, traffic safety and any requirements relating to the height between the crown of the pavement and the drain invert (as discussed in Section D.618.

Design volumes of run-off are usually estimated using the Rational Method (Section 5.3.4). Flow velocities are calculated from the Manning equation using roughness values shown in Table E.6.5. It should be noted that most published roughness and velocity data are based on clean water flow. Along mountain roads sediment-laden water is more common; hence flow velocities may be lower, but this errs on the side of safety.

Under normal circumstances, the adoption of a trapezoidal cross-section will facilitate maintenance and will be acceptable from the point of view of traffic safety. It is much easier and appropriate to dig and clean a trapezoidal drain with hand tools and the risk of erosion is lower. The minimum recommended width of the side drain is 500mm. This shape carries a high flow capacity and, by carefully selecting the gradients of its side slopes, it will resist erosion.

The V-shape is the standard shape for a drainage ditch constructed by a motor-grader or towed grader. It can be easily maintained by heavy or intermediate equipment but it has relatively low capacity necessitating more frequent structures for emptying it. Furthermore the shape concentrates flow at the invert and encourages erosion.

The rectangular shaped drain requires little space but needs to be lined with rock, brick or stone masonry, or concrete to maintain its shape.

In very flat terrain and reasonable soils it is often best to use wide unlined "meadow drains". These are formed shallow and continuous depressions in the surface that avoid abrupt changes in surface profile. When properly designed, their capacity is high and the flow velocity is low so that erosion should be controlled.

When the subgrade is an expansive soil, changes in moisture content near to the road itself must be minimised. The design of the side drains and side slopes for such conditions are described in Section D.6.19.

As far as traffic safety is concerned, a wide and shallow drain for a given flow capacity is preferable to a deeper one but, particularly on steep sidelong ground, the extra width required to achieve this may be impracticable or too expensive. Side drain covers can be used to provide extra road width in places where space is severely limited but their widespread use is not recommended. They are expensive and the drains that they cover are then difficult to maintain.

Side drains (as well as the road itself) should have a minimum longitudinal gradient of 0.5%, except on crest and sag curves. Slackening of the side drain gradient in the lower reaches of significant lengths of drain should be avoided in order to prevent siltation.

For the construction of LVRs the spoil material from the construction of the side drain is usually used to provide the formation of the road and its camber. When roads are built using labour-based methods this is usually the only source of material (unless the road is to be built on an embankment) hence it is important that the size of the drain is wide and deep enough to provide sufficient material. Failure to do so is often the reason for the resulting low camber and early deterioration of gravel and earth roads. In most circumstances a wide trapezoidal drain is the ideal solution.

Access across side drains for pedestrians, animals and vehicles needs to be considered. Community representatives should be consulted with regard to locations, especially for established routes. The methods that could be used are:

- Widening the drain and taking its alignment slightly away from the road;
- Hardening the invert and sides of the drain;
- Beam/slab covers or small culverts.

The arrangement must be maintainable and not risk blockage of the side drain. Failure to accommodate these needs will usually result in later ad hoc arrangements that compromise the function of the side drain.

Groundwater in the subgrade can be released either by using a drainage layer at sub-base level or by incorporating gravel cross drains (grips) in the shoulder that exit via a weephole in the side drain backed with a piece of filter fabric. The weepholes must be set at the correct level to take the water from the appropriate pavement layer and also the drain must be sufficiently deep so that there is little possibility of the water in the drain being of sufficient depth for it to flow back into the road.

Deeper drains, comprising a filter-wrapped perforated pipe within a graded gravel backfill, can be constructed under very wet slope conditions to a depth of 1-1.5m below the level of the side drain invert, and led to the nearest culvert inlet.

5.4.5 Erosion control in the side drain

When the water flows too fast, it will erode the bottom of the drain. The faster water flows, the more soil it can erode and carry away. There are various methods of reducing erosion, the two most common being to build simple scour checks or to line the drains.

Scour checks (sometimes called check dams) reduce the speed of water and help prevent it from eroding the road structure. Typical designs are shown in Figure D.5.6. The scour check acts as a small dam and, when naturally silted up on the upstream side, effectively reduces the gradient of the drain on that side, and therefore the velocity of the water. The energy of the water flowing over the dam is dissipated by allowing it to fall onto an apron of stones. Scour checks are usually constructed with natural stone, masonry, concrete or with wooden or bamboo stakes. By using natural building materials available along the road side, they can be constructed at low cost and be easily maintained after the road has been completed.

There must be sufficient cross-sectional area above the scour check (ie where the water has been slowed down) to accommodate the maximum design flow. Wide drains are also preferred to reduce the velocity of the water and minimise erosion but space is at a premium in the type of terrain where scour checks are required so wide drains may not always be practicable.

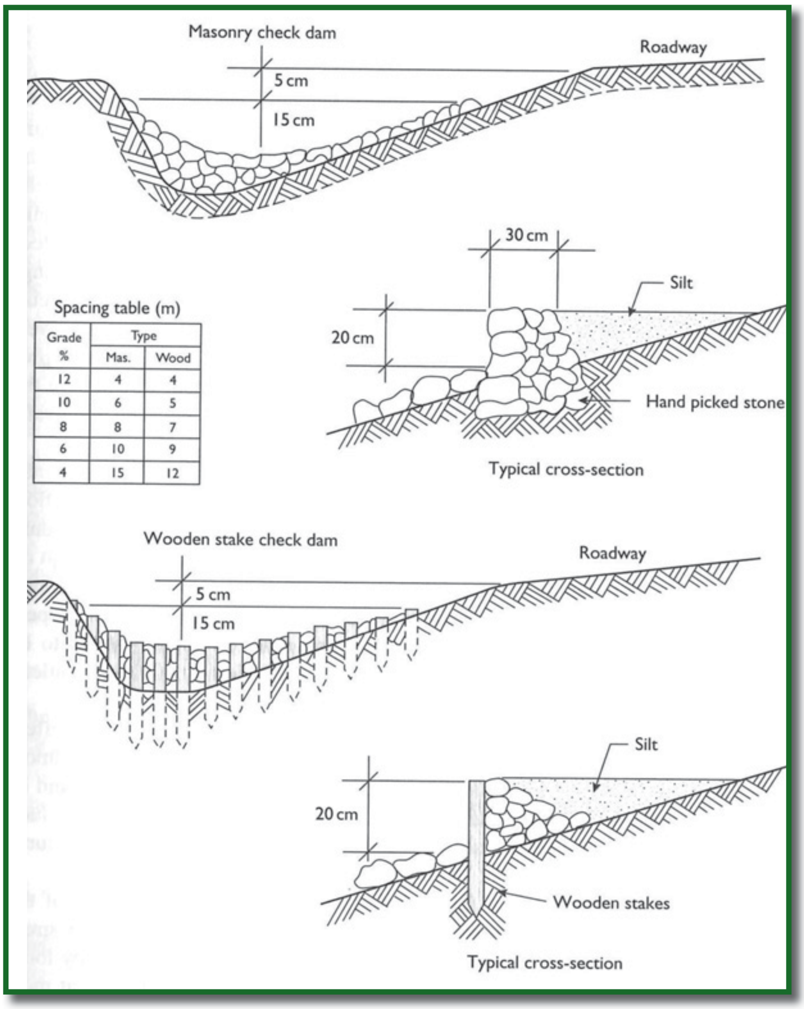


Figure D.5.6: Typical design of scour checks

The distance between scour checks depends on the road gradient and the erosion potential of the soils. Table D.5.6 shows recommended values but these may need to be modified for more erodible soils.

Table D.5.6: Spacing between scour checks

Road gradient (%)	Scour check interval (metres)
3	Not required
4	17
5	13
6	10
7	8
8	7
9	6
10	5
12	4

After the basic scour check has been constructed, an apron should be built immediately downstream using stones. The apron will help resist the forces of the waterfall created by the scour check. Sods of grass should be placed against the upstream face of the scour check wall to prevent water seeping through it and to encourage silting to commence on the upstream side. The long term goal is to establish a complete grass covering over the silted scour checks to stabilise them.

Sections of side drain with scour checks cannot be maintained by motor grader or towed grader and will need to be maintained by hand.

Depending on the strength of the material in which the drains are excavated and the velocity of runoff they are expected to carry, side drains may also need to be lined. The controlling factor is the ease of erosion of the soil. Table D.5.7 indicates the critical velocities for different materials. With velocities greater than those shown, erosion protection measures will be required.

The drains may be lined with heavy duty polythene, or some other impermeable material, before masonry pitching is applied. This will prevent water penetration if the masonry becomes cracked by movement. The lining can also be extended up the banks to prevent lateral erosion. When the cross-sectional area is less than about 0.1m^2 and the gradient is gentle, drains can be lined with unbound masonry. Larger and steeper drains are lined with mortared masonry, although they are considerably more expensive. Any gap between the drain and the hill side must be filled with compacted impermeable material (eg clay) sloping towards the drain to minimise infiltration behind it.

Table D.5.7: Permissible flow velocities (m/sec) in excavated ditch drains

Soil type	Clear water	Water carrying fine silt	Water carrying sand and fine gravel
Fine sand	0.45	0.75	0.45
Sandy loam	0.55	0.75	0.6
Silty loam	0.6	0.9	0.6
‘Good’ loam	0.75	1.05	0.7
Lined with established grass on good soil	1.7	1.7	1.7
Lined with bunched grasses (exposed soil between plants)	1.1	1.1	1.1
Volcanic ash	0.75	1.05	0.6
Fine gravel	0.75	1.5	1.15
Stiff clay	1.15	1.5	0.9
Graded loam to cobbles	1.15	1.5	1.5
Graded silt to cobbles	1.2	1.7	1.5
Alluvial silts (non colloidal)	0.6	1.05	0.6
Alluvial silts (colloidal)	1.15	1.50	0.9
Coarse gravel	1.2	1.85	2.0
Cobbles and shingles	1.5	1.7	2.0
Shales	1.85	1.85	1.5
Rock	Negligible scour at all velocities		

Channels can also be lined with gabion, dry stone pitching, rip-rap or vegetation.

When constructing a channel lining it is important to reproduce, as a minimum, the dimensions of the original channel. A curved rather than rectangular shaped cross-section to the bed lining is preferable. The main disadvantage with channel linings is that a lower channel roughness leads to an increase in flow velocity and hence an increase in scour potential further downstream. In the case of masonry aprons, or gabion mattresses with masonry screeds, some reduction in velocity can be achieved by cementing protruding stones into the surface.

Masonry linings can be constructed to fit the stream bed much more closely than gabion. They are also less easily abraded, but they cannot tolerate significant settlements, loss of support by seepage erosion or high groundwater pressure.

Dry stone pitching is usually only suitable where discharges are lower than 1 m/sec per metre width, and where sediment load is relatively fine-grained.

Grass can provide some resistance to channel erosion and may be used where flow velocities are not expected to be too high. The introduction of grass will also tend to reduce flow velocities, although channel vegetation should not be so widespread as to inhibit or divert flow, which could lead to bank scour. Where immediate effective protection is required, a structural solution is preferable to a vegetative one.

The winning of boulders and cobbles from gully beds for road construction materials can reduce the armouring effect provided by coarse material. If the bed material appears to be weathered and static for much of the time, then its removal could expose more erodible sediments beneath. In such cases, extraction from the channel bed should be discouraged or prohibited. Conversely, where the entire bed deposit is fresh and evidently mobile, the removal of material may not have a significant effect on channel stability, especially if the quantities concerned are small compared to the volume of bed load.

Cascades or steps in the drain long-section can also be a useful means of reducing flow velocity, although both scour checks and cascades can impede the transport of debris, increasing the risk of blockage.

5.4.6 Mitre drains or turnouts

It is normally best practice to discharge the water from the side drains as frequently as possible. If it can be discharged on the same side of the road as the drain, a turnout or mitre drain is used to lead the water away. Mitre drains simply lead the water onto adjacent land therefore care is required to design them to ensure that problems associated with the road are not passed on to the farmer or landowner. It is advisable to consult adjacent land users regarding the discharge of water onto their land to gain their support and agreement and to avoid possible problems in the future.

The principle is to aim for low volumes and low velocities at each discharge point to minimise local erosion and potential downstream problems. The maximum spacing of turnouts depends on the volume of water flowing in the drain and therefore hydrological principles may need to be used to estimate this. However, in many cases it is only water shed from the road itself that flows in the side drain and this is relatively easy to estimate using the Rational Method (Section 5.3.4).

Where soils are very erodible, it may be preferable to increase side drain capacity to convey runoff to the next available safe discharge point rather than to construct side drain turnouts or relief culverts on erodible slopes. With the extra volumes of water that this entails, the design of these less frequent safe discharge points will usually be more expensive.

In mountainous terrain the discharge of water is considerably more difficult and consequently more expensive. This is discussed in more detail in Section D.5.6.

Table D.5.8 gives the maximum spacing. However, spacings of mitre drains should normally be more frequent than this and values as low as one every 20 m may be required to satisfy landowners.

Table D.5.8: Maximum spacing of mitre drains

Road gradient (%)	Maximum mitre drain interval (metres)
12	40
10	80
8	120(1)
6	150(1)
4	200(1)
2	80(2)
<2	50(2

Notes:
1. A maximum of 100m is preferred but not essential
2. At low gradients silting becomes a problem

In order to ensure that water flows out of the side drain into the mitre drain, a block-off is required as shown in Figure D.5.8. It is essential that the mitre drain is able to discharge all the water from the side drain. If the slope of the mitre drain is insufficient, the mitre drain needs to be made wide enough to ensure this.

The desirable slope of the mitre drains is 2%. The gradient should not exceed 5% otherwise there may be erosion in the drain or on the land where the water is discharged. The drain should lead gradually across the land, getting shallower and shallower. Stones may need to be laid at the end of the drain to help prevent erosion.

In mountainous terrain, it may be necessary to accept steeper gradients. In such cases, appropriate soil erosion measures should be considered.

In flat terrain, a small gradient of 1% or even 0.5% may be necessary to discharge water, or to avoid very long drains. These low gradients should only be used when absolutely necessary. The slope should be continuous with no high or low spots. For flat sections of road, mitre drains are required at frequent intervals of 50m to minimise silting.

Angle of mitre drains

The angle between the mitre drain and the side drain should not be greater than 45 degrees. An angle of 30 degrees is ideal (Figure D.5.7).

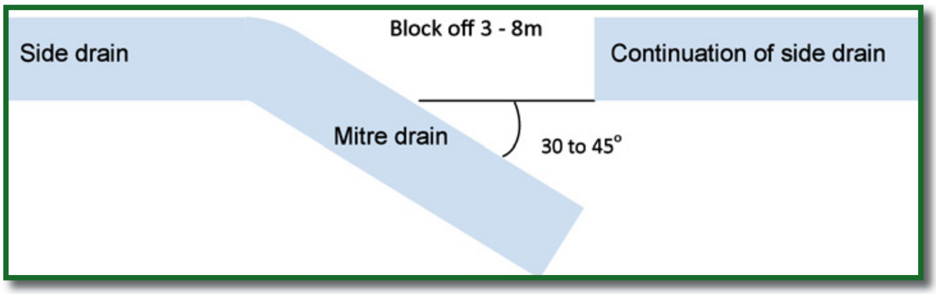


Figure D.5.7: Angle of mitre drain

If it is necessary to take water off at an angle greater than 45 degrees, it should be done in two or more bends so that each bend is not greater than 45 degrees (Figure D.5.8).

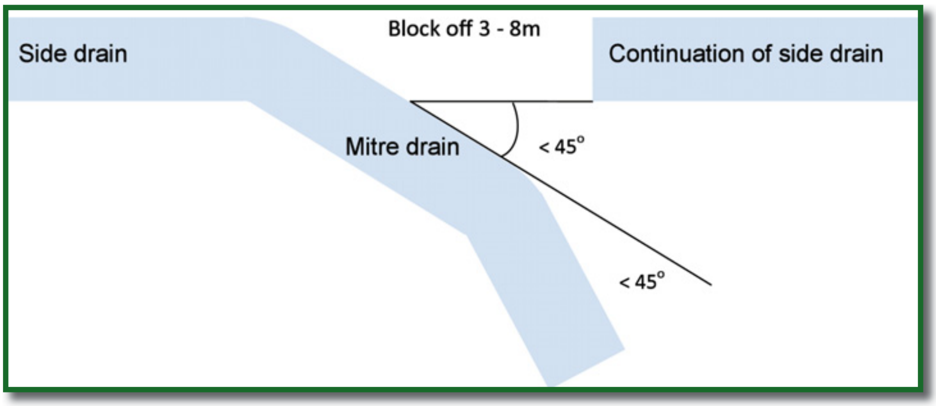


Figure D.5.8: Mitre drain angle greater than 45 degrees

5.4.7 Wet lands

Road crossings in wet areas, including damp meadows, swamps, high groundwater areas, and spring sources, are problematic and undesirable. Wet areas are ecologically valuable and difficult for road building. Soils in these areas are often weak and require considerable subgrade reinforcement. Drainage measures are expensive and may have limited effectiveness. Therefore, if at all possible, such areas should be avoided.

If wet areas must be crossed, special drainage or construction methods should be used to reduce impacts from the crossing which will usually require an embankment. They include multiple drainage pipes or coarse permeable rock fill to keep the flow dispersed, subgrade reinforcement with coarse permeable rock, grade control, and the use of filter layers and geotextiles. The objective is to maintain the natural groundwater level and flow patterns dispersed across the meadow and, at the same time, provide for a stable, dry roadway surface.

Local wet areas can be temporarily crossed, or 'bridged' over, using logs, landing mats, tyres, aggregate, and so on. Ideally, the temporary structure will be separated from the wet area with a layer of geotextile. This helps to facilitate removal of the temporary material and minimizes damage to the site. Also, a layer of geotextile can provide some reinforcement strength as well as provide separation to keep aggregate or other materials from punching into the weak subgrade.

Subsurface drainage, through use of under-drains or aggregate filter blankets, is commonly used along a road in localized wet or spring areas, such as a wet cut bank with seepage, to specifically remove the groundwater and keep the roadway subgrade dry. A typical under-drain design uses an interceptor trench 1-2 meters deep and backfilled with drain rock, as shown in D.5.9.

Subsurface drainage is typically needed in local wet areas and is much more cost-effective than adding a thick structural section to the road or making frequent road repairs. In extensive swamp or wet areas, subsurface drainage will often not be effective. Here, either the roadway platform needs to be raised well above the water table, or the surfacing thickness design may be based upon wet, weak subgrade conditions that will require a relatively thick structural section. A thick aggregate layer is commonly used, with the thickness based upon the strength of the soil and anticipated traffic loads.

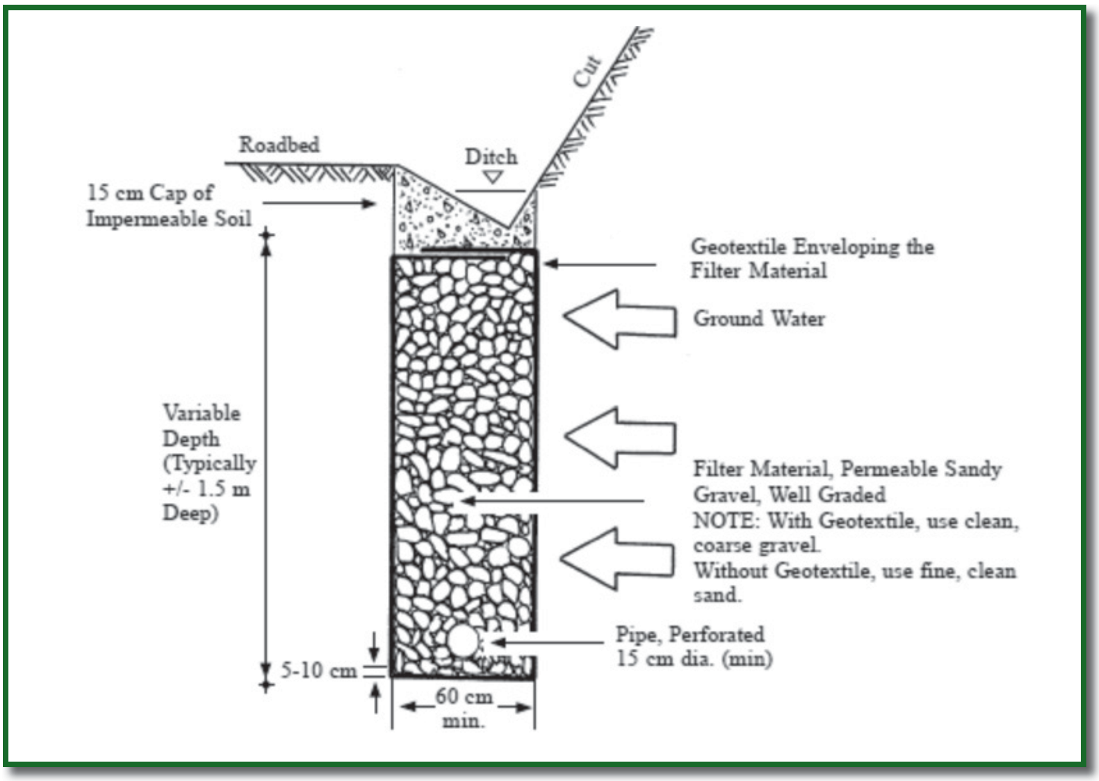


Figure D.5.9: Typical sub-surface drain

5.4.8 Interceptor, cut-off or catch-water drains.

As its names imply, such drains are constructed to prevent water flowing into vulnerable locations by 'intercepting', 'cutting off' or 'catching' the water flow and diverting it to a safe place.

For example, where the road is situated in sidelong ground on a hillside, a significant amount of rainwater may flow down the hill towards the road. This may cause damage to the face of cuttings and even cause landslips. Where this danger exists an interceptor drain should be installed to intercept this surface water and carry it to a safe point of discharge; usually a natural watercourse (Figure D.5.10).

The interceptor drain should be located so that:

- It drains at a satisfactory gradient throughout its length (2%)
- It is not too close to the cut face. It should be at least 3-5m away so that it does not increase the danger of a landslip.

If steep gradients in the drain are unavoidable then scour checks (Section 5.4.5) should be installed.

The material excavated to form the drain is usually placed on the downhill side to form a bund. Vegetation cover should be established as soon as possible in the invert and sloping sides of the interceptor drain and bund to resist erosion. However, where no seepage is tolerable, consideration should be given to lining the drain so that it is truly impermeable thereby minimising the risk that water will weaken the cut slope.

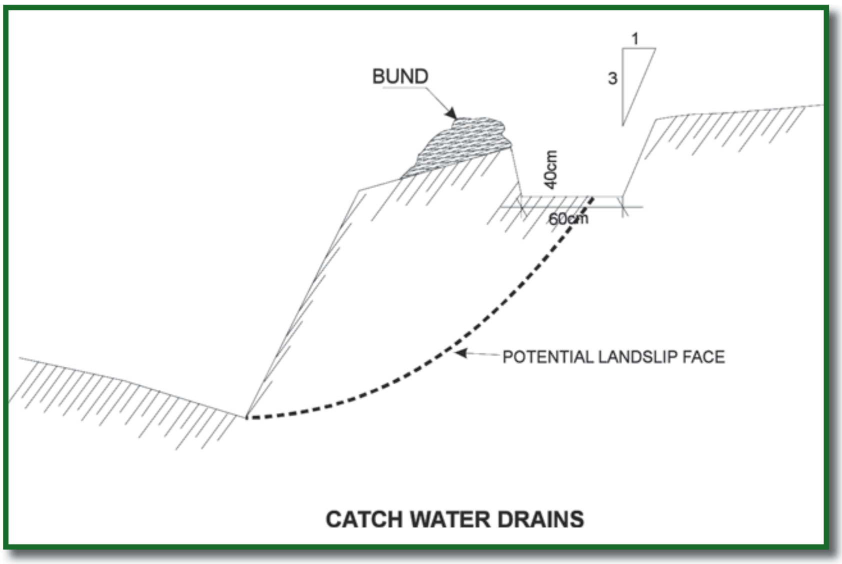


Figure D.5.10: An interceptor, cut-off or catch-water drain

The interceptor drain should normally be 600mm wide, 400mm (minimum) deep with sides back-sloped at 3:1 (vertical: horizontal).

Similar interceptor drains can be used whenever water is flowing towards the road; they are not restricted to protecting cut slopes, but such drains are only useful when surface runoff rates are significant. Surface runoff can be expected only during high intensity rainfall on moderate to steeply-inclined slopes, on slopes of low permeability where vegetation is patchy, or where runoff from agricultural land becomes concentrated onto un-vegetated soil slopes. If surface runoff is substantial, and there is a clear threat of erosion or slope failure further downslope, the use of surface drains is justifiable. However, they are not without problems. They are easily damaged or blocked by debris and are often not seen and therefore not cleaned on a regular basis. In addition, differential settlement or ground movement will dislocate masonry drains, leading to concentrated seepage, if they are constructed without polythene lining. If there is any doubt about their effectiveness, or whether they can be maintained in the long term, it is better not to build them than have them become forgotten and allowed to fall into disrepair, making drainage and instability problems worse.

Factors to be considered in the design of surface drains are:

- Water collected by the drain must be discharged safely in a manner that will not initiate erosion elsewhere.
- Construction of masonry-lined drains should be limited to undisturbed slope materials. Differential settlement, which frequently occurs in made ground and particularly at the interface between natural ground and fill, will lead to rupture.
- Drain gradients should not exceed 15 %.
- For ease of maintenance and to minimize erosion they should be wide and have sloped sides.
- Where people have to cross the drain, easy side slopes should be provided so that the people will not fill the drain to cross it.
- Stepped drain outlets should be provided with a cascade down to the collection point.
- Drains should discharge into a stream channel wherever possible, and preferably into channels that already convey a sizeable flow in comparison to the drain discharge.
- Low points in the drain system should be designed against overtopping by widening or raising the side walls.
- Lengths of drain should be kept short by the construction of frequent outlets in order to reduce erosion potential should drain failure occur.

Where it is not practicable to discharge cut-off drainage into an adjacent stream channel, cascades can be constructed down the cut slope to convey water into the side drain. However, these structures are often vulnerable to the effects of side splash, undermining by seepage erosion and concentrated runoff along their margins. They must be designed to contain the water, and their margins must be protected with vegetation or stone pitching.

In very sensitive locations a simple earth bund can be constructed instead of a cut-off drain. The disadvantage is that material may have to be excavated a little way from the bund and cast or transported. However, the distinct advantage is that the soil surface is not disturbed at the bund and existing vegetation can be encouraged to grow onto the bund to stabilise it. A range of bio-engineering measures can also be used in sensitive areas and specialist advice should be sought on this.

5.4.9 Chutes

Chutes are structures intended to convey a concentration of water down a slope that, without such protection, would be subject to scour. Since flow velocities are very high, stilling basins are required to prevent downstream erosion. The entrance of the chute needs to be designed to ensure that water is deflected from the side drain into the chute, particularly where the road is on the steep grade.

5.5 Erosion control

Erosion forces are one of the most destructive forces an engineer has to contend with in designing and constructing roads but the problem of erosion can be minimised by providing suitable precautions at all stages in the design.

The construction of a road often requires land clearing and levelling in the preparation stage. It involves removal of shrubs and trees that are normally acting as wind-breaks, rainfall 'sponges' and soil stabilisers and therefore, on removal, the soil erosion process is accelerated, especially in sloping areas.

After construction, erosion often appears in road embankments as gullies in the shoulders and embankment slopes; as gouges in the side drains, which endangers the traffic; and in the actual road foundation. It undermines fills and backslopes, initiating landslides, undermines bridge foundations and other road structures, clogging drainage ditches, culverts and other waterways in the watershed.

It is often impossible to make reliable predictions concerning the full extent of erosion protection likely to be required until the road drainage system is fully functioning and the slopes and drainage channels have responded to the new drainage regime. Constructed roads interrupt the natural drainage of an area, and concentrated water discharge through culverts and drains leads to soil erosion if the drainage is not properly designed.

The general design philosophy of stream course protection is to dissipate as much water energy as possible in the vicinity of the road itself, where erosion is likely to be worst, and protect outfall channels down to a point where they are large enough or sufficiently resistant to withstand the increased flow.

Outfall channel protection usually consists of check dams, cascades and channel linings. It is not uncommon to build protection works for 20-60m downstream of culverts, and there are instances where they have been constructed for distances of 500m or more. If investment to this level of protection is considered necessary, it is clearly important to be sure that the measures will be effective.

Protection of erodible channels upstream of culverts is usually accomplished by check dams and cascades constructed over much shorter lengths, and usually within 20m of the inlet.

Good erosion control should preferably start at the top of the rainfall catchment with the objective being the reduction of water run-off towards the road. The road should be designed with sufficient numbers of culverts and mitre drains to avoid large concentrations of water discharging through the structures. Below the road, water should be channelled safely to a disposal point (eg a stream) or dispersed without causing damage to the land.

Often the problem of erosion extends beyond the road environment itself and affects dams, slopes, rivers and streams well away from the road. The steepness of the cut slopes and constructed embankments together with a deficiency in drainage means that landslides may result.

The storage of spoil during road construction may kill local indigenous vegetation which can cause erosion and slope stability problems. In mountainous regions large quantities of spoil can be generated and the balance between cut and fill is difficult to maintain. Storage of spoil or disposal through haulage may be difficult; therefore the process will involve more effective environmental management to avoid erosion problems.

The channelling of run-off through new routes will result in changes within the natural equilibrium. Excessive water flows may be generated when drainage ditches and other water control structures have become blocked or damaged. The excessive flows will find new routes which will result in an enlargement of the erosion problems.

Chain impacts, including soil contamination and damage will affect the road environs. Soil contamination will possibly result in vegetation loss and therefore resistance to erosion. Construction of the road may result in deforestation which, in turn, will lead to erosion of bare slopes, the re-channelling of rivers and streams, possibly minor landslides and changes in the microclimatic conditions. The roadworks themselves will temporarily increase waterborne material because rainfall erodes the surface of temporary or new surfaces before they are stabilised.

5.5.1 Identifying and assessing potential erosion problems

The initial project survey will possibly indicate the range of problems that may be encountered, and the design should include measures to mitigate the problems. The following should be considered:

- Previous or similar construction projects. These can be useful indicators, and evidence of erosion problems can be obtained from the local population.
- Desk study as part of the pre-feasibility study. More detailed information can often be obtained from a desk study, from maps (geological hydrological, and topographic) and aerial photographs if available.
- Historic evidence. Signs of erosion or soil instability and evidence of major floods and local agricultural practices should be sought.
- Drainage design. Consider how water flows will be concentrated by the construction of the road.
- Cleared areas. Review the areas that will no longer be vegetated after the construction.
- Cut or fill slopes. Review the slopes that will be at greater angles than previous natural slopes.

5.5.2 Mitigation measures to control erosion and scour

There are a wide range of methods and techniques that may be employed to prevent erosion and allow for the construction of a road in an environment with little or no erosion impact. The simple technique of replanting cleared areas will be effective generally, while the more difficult cases may be addressed with measures such as retaining walls.

The simplest ways of controlling erosion of soil in road projects is by avoidance. This can be achieved by:

- Reducing the area of ground that is to be cleared;
- Quickly replanting cleared areas, maintaining the planted areas and specific bio-engineering measures;
- Avoiding erosion sensitive alignments;
- Controlling the rate and volume of water flows in the area.

Replanting

An important method for reducing erosion and stability problems is by replanting cleared areas. It is suggested that this procedure should be carried out as early as possible during the construction process, and before the erosion becomes too advanced. It is important to select the correct vegetation that will address the specific engineering function required for stabilisation.

The engineering function of vegetation in erosion protection measures are:

- Retaining material from moving over the soil surface;
- Armour plating the surface against erosion and abrasion;
- Supporting the slope by stabilising it from the base;
- Reinforcing the soil by increasing its effective shear strength;
- Drain the soil profile by taking water into the roots.

Slope protection

Avoiding erosion by stabilising slopes requires good engineering design of the slope form and drainage. This topic is dealt with in more detail in the Chapter 3 on geotechnical issues but is summarised here for convenience.

Slope retaining techniques are necessary when:

- The slopes are too high or steep;
- There is a risk of internal erosion or localised rupture due to drainage problems;
- It is necessary to decrease the amount of earthwork because the road width is limited.

Well-established techniques for slope protection against erosion are:

- Intercepting ditches at the top and bottom of slopes. Gutters and spillways are methods used to control the flow of water down the slope.
- Stepped or terraced slopes to reduce the height of the slope.
- Riprap or rock facing material embedded in a slope face, sometimes with planted vegetation.
- Retaining structures such as gabion cribs.
- Retaining walls.
- Reinforced earth, where the embankment walls build up as the earth fill is placed, within anchors compacted into the fill material.
- Shotcreting and geotextiles; techniques that are usually expensive and should therefore be carefully considered before being used for specific applications.

Table D.5.9 is a summary of the various mitigation measures, their effectiveness and comparative costs.

Table D.5.9: Comparison of various slope erosion mitigation measures

Measures	Effectiveness	Approximate comparative costs
Stepped slopes	High	Raises volume of earthworks depending on distance to borrow and spoil
Riprap	High for embankment protection	Normally high but depends on source of suitable material
Grass seeding or Turfing	Only surface effective, avoids start of erosion	Inexpensive. May require short term watering to establish.
Shrubs	High, even in depth after several years of growth	2 to 3 times cost of grass
Crib walls	Good	One quarter the cost of a retaining wall
Geotextiles	High; good mechanical and chemical resistance	10 to 20 times the cost of vegetation
Gridwork and wooden barricades	Fairly good	5 times the cost of vegetation

Riprap

The size of riprap needed to protect the stream bank and not move is related to the speed of flow as shown in Figure D.5.13. The flow along a long tangent section of stream, or the flow parallel (VP) to the stream, is assumed to be about 2/3, or 67%, of the average velocity (VAVE). The flow in a curved section of stream, with an impinging flow, has an assumed impinging velocity (VI) equal to about 4/3, or 133%, of the average velocity, VAVE. Thus, riprap in an area with relatively fast flow, such as a bend in the channel, will have higher stresses and require larger rock than the size needed in a straight part of the channel. Note that most of the rock should be as large as, or larger than, the size indicated in Figure D.5.13. The Isbash Curve indicates the maximum size rock that might be considered in a critical application. If suitably large rock is not available then the use of cement grouted rock, masonry, or gabions should be considered.

Riprap installation details are shown in the Note in Figure D.5.11. Figures D.5.12 and D.5.13 illustrate the use of rip rap. Ideally riprap should be placed upon a stable foundation and upon a filter layer made either of coarse sand, gravel, or a geotextile. The riprap itself should be graded to have a range of sizes that will minimize the voids and form a dense layer. The riprap should be placed in a layer with a thickness that is at least 1.5 times the size (diameter) of the largest specified stone, with the thickest zone at the base of the rock. In a stream channel, the riprap layer should cover the entire wetted channel sides, with some freeboard, and it should be placed to a depth equal or greater than the depth of expected scour.

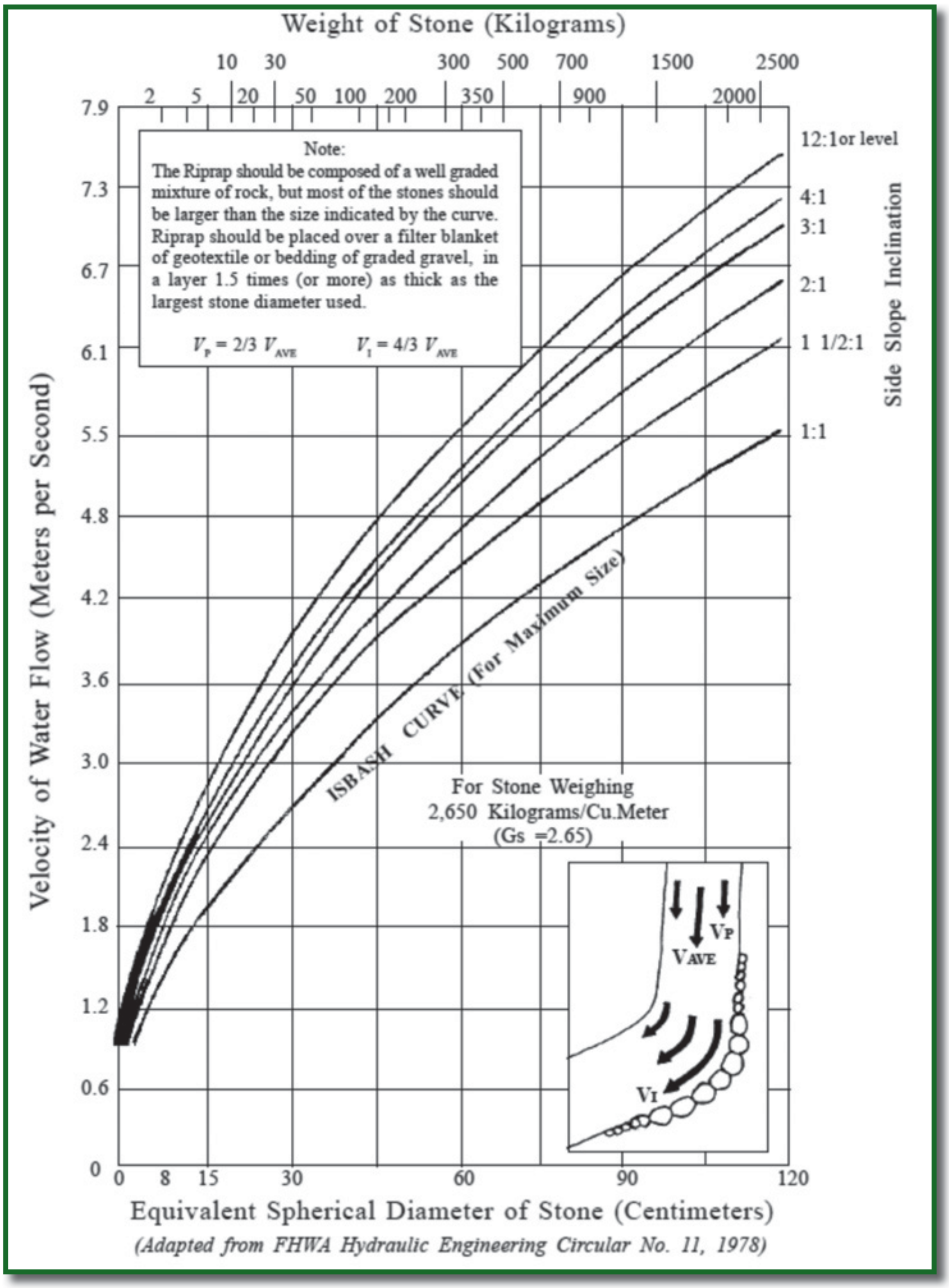


Figure D.5.11: Size of stone that will resist displacement for various velocities of water flow and side slopes

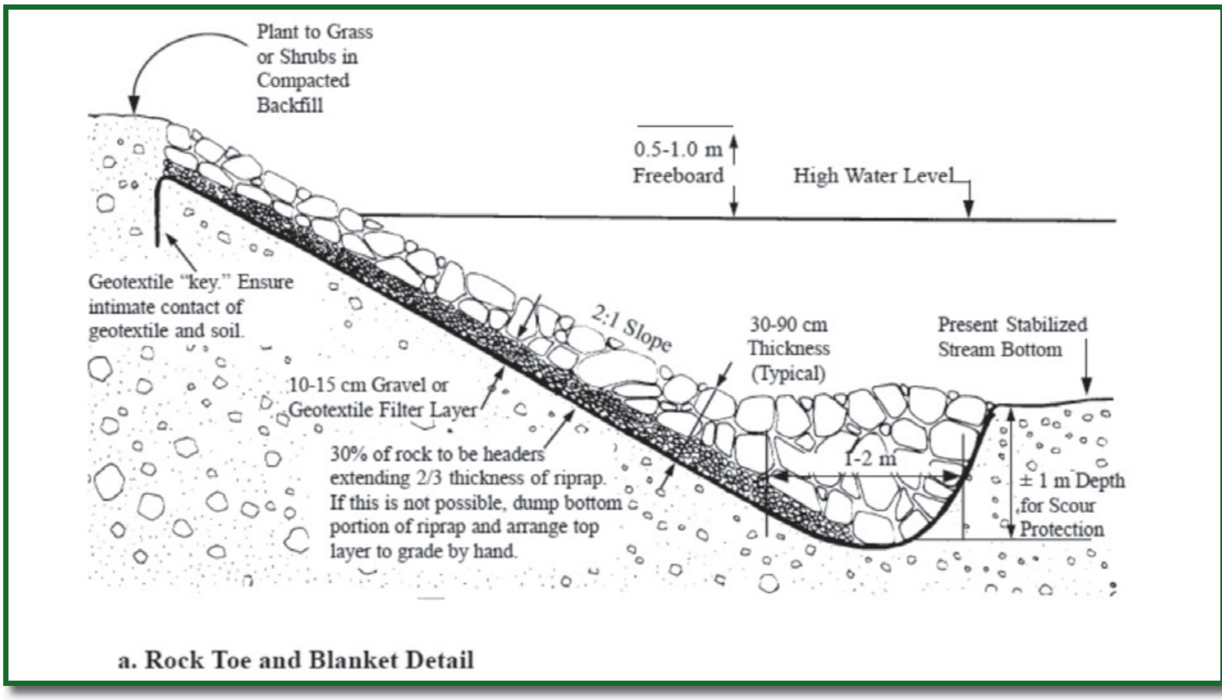


Figure D.5.12: Use of riprap 1

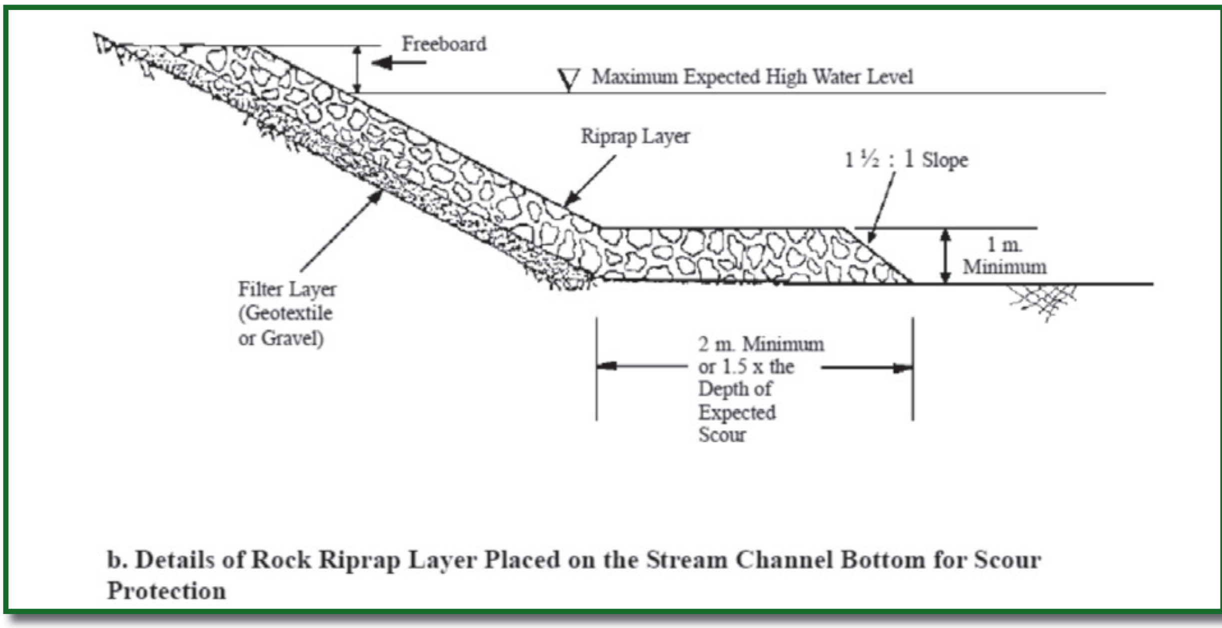


Figure D.5.13: Use of riprap 2

Filters

A filter serves as a transitional layer of small gravel or geotextile placed between a structure, such as riprap, and the underlying soil. Its purpose is to prevent the movement of soil behind riprap, gabions or into under-drains, and allow groundwater to drain from the soil without building up pressure.

Traditionally, coarse sand or well-graded, free draining gravel have been used for filter materials. A sand or gravel filter layer is typically about 150 to 300mm thick. In some applications, two filter layers may be needed between fine soil and very large rock. Filter criteria have been developed to determine the particle size and gradation relationships needed between the fine soil, a filter material, and coarse rock such as riprap and are well documented elsewhere.

Today, geotextiles are commonly used to provide filter zones between materials of different size and gradation because they are economical if manufactured locally, easy to install, and perform well with a wide range of soils. When using geotextiles the fabric should be pulled tight across the soil area to be protected before the rock is placed. The geotextile can be a woven monofilament or a needle punched non-woven geotextile, but it must be permeable. The geotextile needs to have an apparent opening size of 0.25 to 0.5 mm. In the absence of other information, a 200 g/m^2 needle-punched non-woven geotextile is commonly used for many soil filtration and separation applications. Other common geotextile or geosynthetic material applications on roads include subgrade reinforcement to reduce the thickness of needed aggregate over very weak soils; separation of aggregate from soft subgrade soils; reinforcement of soils in structures such as retaining walls and reinforced fills; and entrapment of sediment with silt fences.

If knowledgeable engineers are not available, then geotextile distributors or manufacturers should be consulted regarding the function and appropriate types of geotextile to use in various engineering applications. Alternatively, information is available on the requirements of different geotextiles for filter applications in the references.

Erosion and scour protection of the road itself

Water produces harmful effects on road shoulders, slopes, drainage ditches and all the other road structures featured in the design. Spectacular failures can occur when cuttings collapse or embankments and bridges are carried away by flood-water. High water velocities can result in erosion which, if severe, can lead to the road being destroyed. For each type of structure used in the road, specific erosion problems can occur. Section 5.4.4 describes methods of minimising and controlling erosion in side drains and other forms of drainage ditch. The specific problems associated with fords, drifts and bridge foundations are discussed in Part E of this Manual.

It should also be noted that low flow velocities can result in silt being deposited which will, in turn, block the various drainage structures. The blockage could result in the drainage structure being overtopped at the next flood event. The diverted water then forges a new un-planned route which results in erosion and possible washout in a new area. It is for this reason that the initial step in road drainage design is that of a hydrological assessment. This will provide design discharges from all major drainage structures and for rivers and streams adjacent to the road alignment.

5.6 Particular drainage problems in severe terrain

Much of what has been said already in previous sections is directly applicable in mountainous areas. However, the situations in such terrain are usually much more severe. Drainage structures have to resist scour and transmit debris every year and failure to do so will damage or destroy them more effectively than a hydraulic surcharge. It is apparent from experience in mountainous terrain that culverts and bridges are rarely subjected to surcharge or overtopping before the foundations are undermined by scour or the waterway is blocked by debris, both of which can occur during the course of a single flood.

5.6.1 Drainage of hairpin stacks

The disposal of water from hairpin stacks on escarpments and steep hill sides is unquestionably a major hazard, both for the road and for the surrounding hill slopes. It is imperative to ensure that water is discharged into channels that are fully protected. As much effort should go into the protection of these channels as into the protection of the road itself.

Side drain runoff at hairpin bends is often conveyed via contour drains to discharge into an adjacent catchment, frequently causing severe erosion problems on the slopes and to the channels below. It is preferable, therefore, to contain all water within the stack system itself, thus avoiding the construction of drainage structures remote from the road whose inspection and maintenance might otherwise be overlooked. This approach presents two alternatives for design. One is to construct a large reinforced side drain around the outside of each hairpin bend, the other is to install a relief culvert beneath the carriageway at each bend to take runoff into the inside side drain below. The main problem with the former option is the fact that drain failure will lead to erosion of the hill slope below and eventual undermining of the hairpin bend. Although the latter may be the preferred option, its main drawback is one of awkward geometry. Side drain runoff is forced to make two ninety degree turns, and the reduced gradient between the inlet and outfall restricts the size of culvert that can be utilized. There is also the cost of these extra culverts and the required greater side drain capacity to be considered.

Where there is no choice but to discharge water onto a hill slope, the following sites should be sought in order of preference:

- Gently sloping or terraced ground;
- A slope formed in strong bedrock;
- A concave soil slope to assist in energy dissipation.

A standard detail for side drain turnout flow dissipation and erosion protection is illustrated in Figure D.5.14. Most turnouts curve in plan, which throws high flows to one side, thus concentrating discharge and increasing erosive power. The flow can be more evenly distributed by providing a flared and baffled outlet.

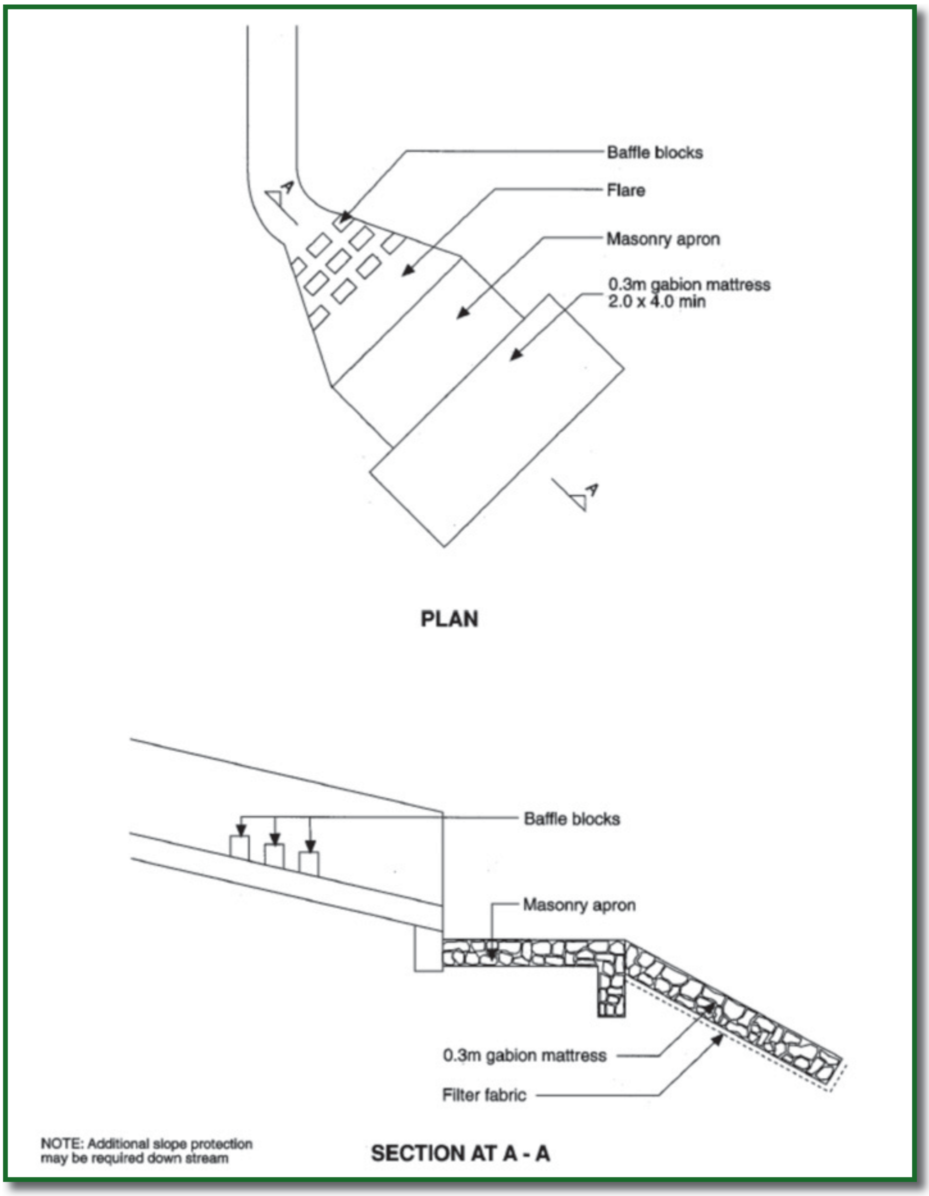


Figure D.5.14: Suggested apron details for side drain turnout

5.6.2 Road construction along valley floors

Where there is a choice it is usually preferable, on hydrological and stability grounds, to adopt a hillside rather than a valley floor alignment. However, the choice of corridor will depend upon the length and practicality of hillside and valley floor options, and the degree of hazard posed by slope failure and flooding along each. Furthermore, some valley floors are significantly more hazardous than others, and it will be necessary to carefully evaluate the risk implications of these hazards before an alignment is chosen.

Valley floor and lower valley side alignments can encounter some or all of the following landforms and hazards:

- Broad rivers that may rise and fall rapidly by several metres on a regular basis.
- Rivers which are actively meandering and changing their plan-form, which could subsequently encroach on the alignment.
- Active river flood plains that are likely to flow full at least once a year. The erosive power against the banks of a river in flood is very great.
- Vigorous tributary streams that are usually highly erosive and capable of transporting large volumes of sediment.
- Fans from tributary valleys that are either eroding rapidly or building up by accumulating debris.
- Flood plain terraces that may be susceptible to river scour on numerous occasions during the wet season, and inundation once every 2-3 years.
- Higher level terraces that may be subject to scour on a regular basis where they protrude onto the active flood plain.
- Rock spurs or promontories that project into the flood plain, forming obstacles to river flow and road alignment.
- Steep, and often eroded, rock slopes on the outside of valley meander bends.
- Slope instability on the lower valley sides in general.

These conditions are most common on youthful valley floors, and especially those with gradients steeper than 1 in 20. The rivers that occupy these valley floors drain steep and frequently unstable catchments. Their flood plains will be either so confined and erosive that the development of terrace sequences has not been possible, or will be subject to cyclic erosion and side slope instability over engineering time-scales to an extent that any preserved terrace surfaces cannot be regarded as safe for road alignment. In such situations, valley floor road alignments should be avoided altogether, otherwise frequent loss of significant sections of road will be inevitable.

Where a valley floor is comparatively mature, and ancient high level terraces are well preserved, then a road alignment located at the back of these terraces, combined with intervening rock cut, may prove satisfactory. If valley side rock mass conditions are not especially adverse to stability, it is usually preferable to construct a road in full cut, or a combination of cut and retained fill through these rocky areas, with a freeboard above the highest anticipated flood level. Where valley side stability conditions are unfavourable, or where river flooding could cause erosion and slope failure to extend far enough upslope to undermine road foundations, it is advisable to examine the practicalities and costs of an alternative alignment altogether.

The cost of constructing roads in major river valleys in mountainous terrain is high and largely independent of traffic levels. The subject is dealt with in detail in ERA's 2011 manual series.

The various design considerations associated with road construction in valley floor locations are discussed below.

5.6.3 Freeboard

It is usual to provide the road surface and associated structures with a freeboard of 2m above design flood level to accommodate surface waves and to provide some leeway in the estimation of flood level. The freeboard can be reduced to 1 - 1.5m in cases where the hydraulic analysis is more reliable. However, the calculation of the design flood is a particularly difficult task when rainfall and flow gauging data are limited or non-existent; and where a catchment run-off regime is subject to short term fluctuations brought about by road construction, land use change, extreme rainstorms and cycles of slope instability, channel incision and sedimentation. Although widely appreciated, it is also important to remember that flood levels can be substantially higher on the outside of meander bends than anywhere else along a given reach.

5.6.4 Flood plain scour and embankment protection.

Flood plain scour, flood plain deposition and valley side instability usually occur at predictable locations. However, external influences, such as tributary fan incursions onto the flood plain, temporary landslide dams and engineering structures, can cause significant short-term modifications to flood plain processes and flow patterns. These should be identified and monitored during the course of construction and maintenance, with appropriate steps taken to protect or locally realign affected sections of road.

It can be assumed that maximum velocities around the concave (outside) banks of river bends and in valley constrictions are between 1.5 - 2 times greater than average or calculated velocities. On highly active flood plains with mobile bed material, predicted and actual scour depths can frequently exceed 5m, and occasionally 10m. Foundation excavations for road retaining walls and other structures are often impracticable at these depths, given the nature of the bed material and the requirements for dewatering the excavation.

Mortared masonry walls are more durable than gabion walls in abrasive riverside locations. and they have the potential to arch over small areas of scour, where gabion walls are more likely to deform. Even when heavy duty selvedge wire is used, gabion boxes are easily broken open by debris-laden water flowing at velocities greater than 4m/s, which is not unusual.

Where there is no choice but to construct a retaining wall within the zone of highly erosive floodwaters, it is worthwhile extending foundation excavations deeper than the depth required for bearing capacity considerations alone, in the expectation that bedrock will be encountered, to obtain a stable foundation for a masonry wall. Alternatively, where the foundation is composed of a significant proportion (usually 50% or greater) of large boulders, the softer materials can be excavated and replaced by concrete to provide a stable foundation for a masonry wall.

However, it is frequently the case that neither of these foundation conditions are achievable within practicable excavations depths, and especially on the outside of river bends where scoured bedrock and boulders have been replaced by finer-grained materials. The potential for foundation scour in these situations will usually dictate that a flexible gabion structure is adopted in preference to a more rigid masonry one, and combined with whatever scour protection works are feasible under the circumstances. Foundation stability can be improved by constructing the retaining wall on a concrete raft, thus reducing differential settlements. Sacrificial walls, double thicknesses of gabion mesh, gabion mattresses and stone rip-rap are likely to prove effective during small and medium-sized flood events only, and will require regular repair or replacement. Reinforced concrete rip-rap can be fabricated in situ if sufficiently large local stone rip-rap is unavailable or cannot be transported to the site, as is often the case. However, the cost of fabricating rip-rap to the required dimension (3m in some cases) is usually prohibitive, and it is usual to adopt a compromise solution under conditions of extreme scour potential.

5.6.5 Cross drainage and tributary fan crossings

Where alignments are located on the lower slopes of steep valley sides, cut slopes can truncate drainage channels with the result that, during heavy rain, sediment and water may overshoot culvert inlets and discharge directly onto the road surface. This is usually remedied by constructing a large catch-wall between the culvert inlet and the road edge, or by providing a dished (concave) concrete causeway. On occasions, concentrated run-off and slope failures will erode new gullies that will require some form of culvert or other drainage provision. A causeway is likely to be the only practicable remedy.

Tributary fans present a range of problems for road alignments. It is useful to differentiate between:

- Mature and stable high level fans;
- Equilibrium fans;
- Immature and unstable fans.

Mature and stable fan surfaces are usually preserved as old, high level landforms that have become incised by rejuvenated stream channels. The principal problem for road construction on high level fans, other than alignment constraints, is the choice of a suitable site to cross the incised channel, bearing in mind that its banks will be composed of unconsolidated and erodible materials.

In the case of equilibrium fan surfaces, all sediment supplied to the fan is transported out of the catchment by one or a number of well-defined channels. Equilibrium fans, and their wide flood plains, will usually comprise a number of distributary flow channels with only one or two of them occupied during normal flow conditions. Other channels may become occupied every 2-3 years or so, in response to floods or landslide (generated debris flows from further upstream). These fans are usually associated with terrace sequences on the adjacent valley floor and, therefore, represent a stage of drainage development between mature high level fans and immature, active fans on flood plains (described below).

A thorough understanding of the flow patterns across equilibrium fan surfaces is required before a road alignment and bridging structures are designed. Artificially increased channelisation and bank protection of the normal flow channel may increase its definition and capacity in order to allow the design of a road crossing that consists of:

- A relatively short span bridge;
- Approach embankments;
- Vented causeways and drifts (multi-culverted embankments to cater for other distributary channels.

The above combination will usually be cheaper to construct and represents a more convenient solution than a multi-span bridge or a lengthy detour into the tributary to find a suitable shorter span bridging site with stable abutment and pier foundations. However, river training and erosion protection of bridge abutments, piers and approach embankments will prove costly and difficult to maintain in the long term. In addition, vented causeways should only be considered where sediment loads are likely to be low.

Immature and unstable fan surfaces are usually characterised by cyclical regimes of erosion and deposition across the whole fan surface during the course of individual storms, and a general process of fan aggradation from one year to the next. The crossing of these fans presents severe problems to valley floor alignments. Sediments will accumulate wherever flow velocity decreases as a result of an abrupt concavity in the channel profile or a sudden increase in channel width. Ideally, valley floor alignments should cross fans immediately upstream of these concavities and, if possible, at the fan apex.

Bridge clearance of at least 5 - 7m should be provided above the existing fan surface level wherever a fan is actively aggrading and its tributary catchment is unstable. However, it is not always possible to conform with this recommendation due to alignment constraints, unstable valley sides adjacent to the fan apex, or the fact that the apex is poorly defined and the fan itself extends upstream into the tributary valley. Under these circumstances, a combination of the following may be the only viable option:

- Gabion check-dams in the stream channel above the fan and erosion control in the catchment above, to control the stream bed level
- River training and scour protection works upstream and downstream of the bridge, to control the stream course
- A commitment to regular maintenance and waterway clearing operations, to keep flow within the channel and to provide room for the accumulation of debris during fan-building episodes.

Design strategies for crossing unstable fan surfaces are summarised below.

1. Where rates of fan deposition are low and where the flow path across the fan is reasonably consistent, a road can be formed on a causeway, preferably constructed in reinforced concrete or gabion. If gabion construction is used, no wire baskets should be left exposed to abrasion by passing rocks. They should be protected with a mortar rendering or equivalent durable surface.

2. Cross the fan via a track that is re-cut after every aggrading or eroding storm flow. This approach will require the following considerations:
   - Vehicular access must be prohibited during and for a few hours after each flood.
   - As the fan surface builds up over time, temporary access will have to be cut deeper into the fan surface and may eventually become waterlogged and impassable during the entire wet season.
   - An alternative to the above is an ever-enlarging detour downstream across the fan, eventually coming to an end when the detour reached the flood plain at the base of the fan.
   - Flood flows will tend to run down either side of the fan and erode the road on the fan approaches.

3. Select a relatively narrow channel across or, preferably to one side of the fan surface (depending on drainage pattern). Use river training gabions and excavation to concentrate flow through this channel. Construct a bridge over the entire width of the fan with at least 7m clearance. This approach will require the following considerations:
   - River training gabions will tend to be scoured and undermined towards the fan apex.
   - Deposition of fine-grained material towards the end of each storm may bury the river training gabions to the extent that during the next storm a new channel will be formed and the existing gabions may be outflanked or destroyed.
   - If the bridge does not extend the full width of the fan, there is a risk that it may be outflanked by changes in flow pattern across the fan surface leading to bridge redundancy and erosion of the approach embankments.

4. Extending the concept of river training further, construct a continuous masonry or concrete spillway from the fan apex to a point downstream of the bridge. The slope and cross-section of this structure must be such that flow velocities are sufficient to transport bedload from the apex of the fan to the flood plain downstream. This approach will require the following considerations:
   - Flow along the external margins of the structure, leading to undermining, is likely to occur unless its inlet is adequately keyed into both banks of the upstream channel.
   - During peak run-off the floor of the spillway may be scoured and eventually destroyed by passing boulders.
   - During the later stages of storm run-off, the channel might still become blocked by fine-grained sediment which will require clearing. The shape of the channel should be made to facilitate this and access to it provided for machinery.

5. Install check dams in the gulley upstream of the fan apex to retain sediment. This approach will require the following considerations:
   - The checkdams may be destroyed during the first few storms in a channel where aggradation of the fan itself is rapid.
   - The volume of sediment that can be trapped behind a check dam system is usually insignificant in comparison with the volume transported and transportable material.
   - Artificially raising the channel bed upstream of the fan apex could easily lead to increased rates of aggradation.

5.7 Potential references and/or bibliography

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6. MATERIALS AND PAVEMENT DESIGN

6.1 Introduction

The main objective of pavement design is to provide the best structural and economical combination of pavement and surfacing materials, types, layer thicknesses and configurations to carry traffic satisfactorily (ie to a pre-determined level of service with minimal maintenance) in a given climatic environment for the design life adopted.

The load carrying capacity of the pavement is a function of both the thickness and stiffness of the materials used in the pavement layers and the support provided by the subgrade. Consequently, a good knowledge of the mechanical properties of the materials comprising the pavement layers and subgrade is important for designing the structure. Climatic conditions as well as both internal and external drainage factors also critically affect the performance of the pavement structure and must be given due consideration in the design process.

The outcome of the design process, in terms of the type and thickness of structure chosen, is influenced by the preceding planning phase and, in turn, determines many aspects of the subsequent construction and maintenance phases of road provision and management. Thus, in order to achieve a successful outcome, there is a need to ensure that the LVR design process is undertaken in a holistic manner with a clear recognition that pavements perform as part of an overall system and must therefore be designed as part of that system. Figure D.6.1 shows the pavement design system used as the basis of this chapter.

6.2 Underlying principles

6.2.1 Approach to low volume road pavement design

The general approach to the pavement design of LVRs differs in a number of respects from that of HVRs. For example, conventional pavement designs are generally directed at relatively high levels of service requiring numerous layers of selected materials. However, significant reductions in the cost of the pavement for LVRs can be achieved by reducing the number of pavement layers and/or layer thicknesses, by using local materials more extensively as well as at lower cost, and more appropriate surfacing options and construction techniques.

Research has also indicated that the road deterioration mechanisms of LVRs are significantly different to those of HVRs. One of the implications of this is that appropriate pavement design options need to be fully responsive to a range of factors that may collectively be referred to as the road environment. This approach pays justifiable attention to the reality of the varying environmental context of the road as summarised in Section D.6.2.4 and discussed in detail in Sections D.6.3 onwards.

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

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

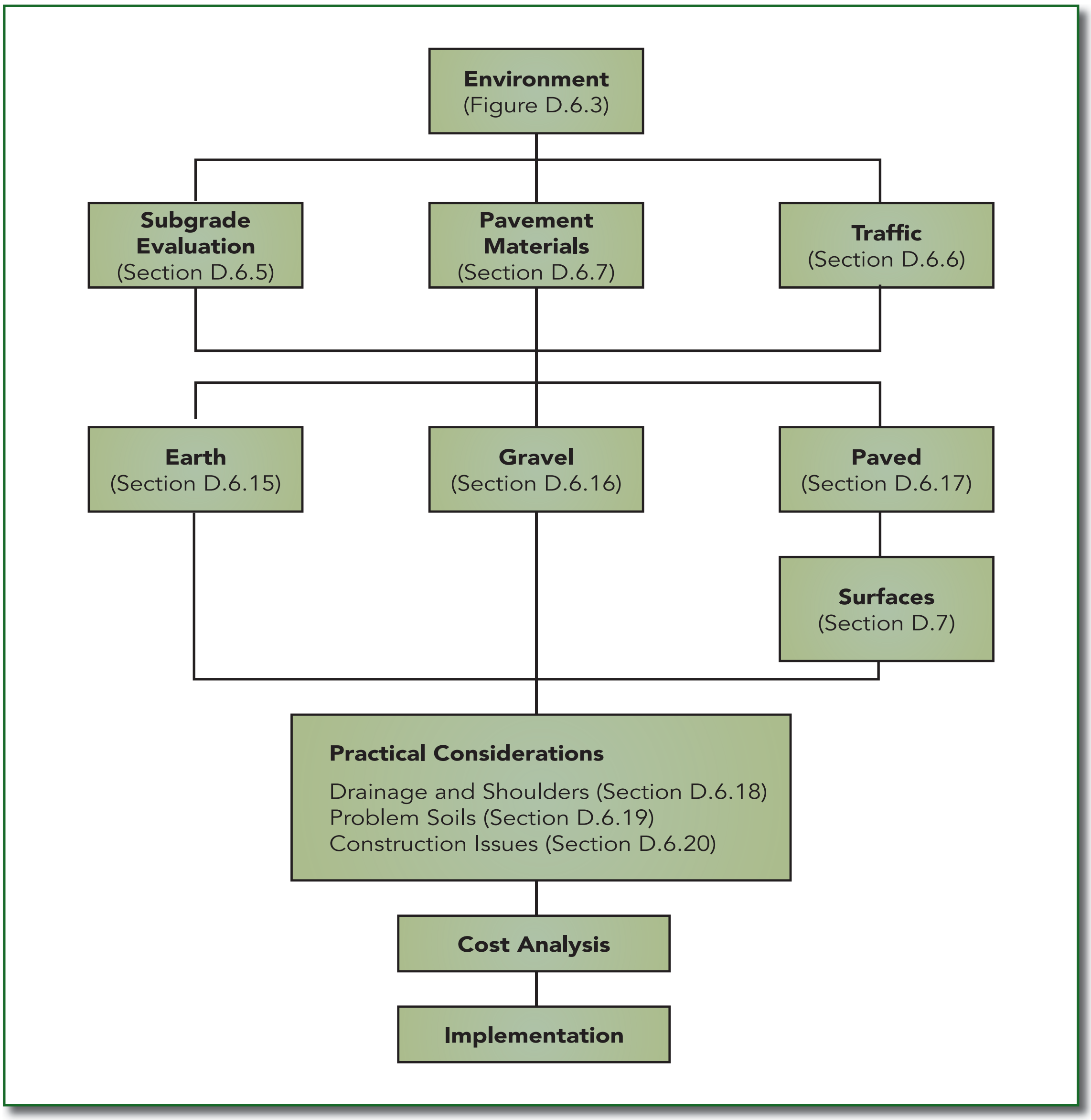

Figure D.6.1: Pavement design system

6.2.2 Pavement structure and function

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

The function of the surfacing is slightly different for gravel and for earth roads where the wearing surface is often permeable and actually wears away under the action of traffic and rainfall. However, the stresses on the subgrade must be reduced to safe levels hence the gravel needs to be replaced regularly and considerably more maintenance is necessary if gravel and earth roads are to perform satisfactorily.

Figure D.6.2 shows a wheel load, W, being transmitted to the pavement surface through the tyre at an approximately uniform vertical pressure, P_0. The pavement then spreads the wheel load to the subgrade so that the maximum pressure on the subgrade is only P_1. By proper selection of pavement materials and with adequate pavement thickness, P_1 will be small enough to be easily supported by the subgrade.

Because of the different functions of the surfacing and pavement structure, these basic components of a road are often independent of one another and a large number of combinations are possible. However, in terms of the design of the overall road, some surfacings (eg bituminous surface treatment such as surface dressing) do not contribute to the overall structural strength of the road, while others (eg penetration macadam) do. In the case of earth or gravel roads, the natural soil or gravel is the main structural component.

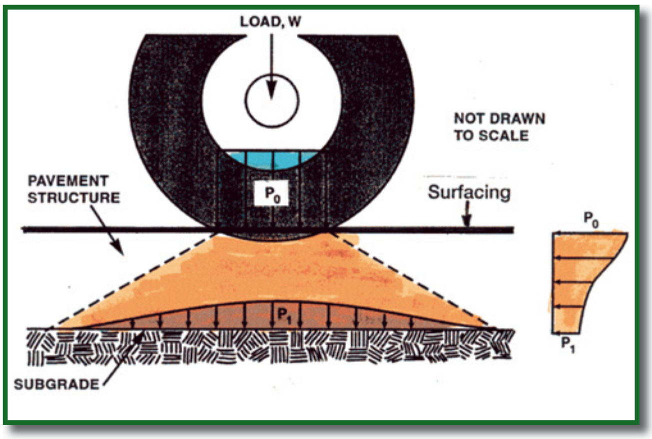

Figure D.6.2: Wheel load transfer through pavement structure

6.2.3 Pavement and surfacing options

There is a wide range of pavement and surfacing options, both bituminous and non-bituminous, that can be used in various combinations and are well suited for incorporation in LVR pavements (see Section D 6.7). These options allow maximum use to be made of locally available materials and least use to be made of more expensive high quality pavement materials, especially where they have to be processed or hauled long distances.

Many of the pavement and surfacing options also allow the use of a high level of local labour, both skilled and unskilled, and a low requirement for imported equipment. This provides the flexibility to use small and medium scale enterprises (SMEs) with the accompanying benefits of higher local employment and savings in costs and foreign exchange.

6.2.4 Road environmental factors

The pavement design process must be fully responsive to the Ethiopian road environment. The various road environment factors that should be considered in the design of LVR pavements are illustrated in Figure D.6.3 and described in the following sections.

Figure D.6.3: Road environment factors

6.3 Climate

The main features of climate that are of importance in the design of LVRs are:

- Temperature and solar radiation;
- Rainfall;
- Water;
- Winds.

Each of the above features can have a profound effect on the design, construction and maintenance aspects of LVR roads and are considered below.

6.3.1 Temperature and solar radiation

Ethiopia's diverse temperature patterns are largely the result of its location in the tropical zone of Africa and its varied topography. In general, the climate comprises dry and hot zones in the eastern lowland regions, a large moderate zone and several wet and cool zones mainly at high altitudes. Temperatures are very varied, ranging from cool to very cold in the highlands to very hot in the lowland areas such as the Dallol Depression. The climate is broadly divided into five zones as illustrated in Table D.6.1 and shown in Figure D.6.5.

Table D.6.1: Ethiopia's climatic zones

Zone	Altitude (m)	Temperature ^{0}C	Rainfall (mm)
Berha	<500	>28	<400
Kola	500-1500	20-28	600-1000
Weina Dega	1500-2500	16-20	About 1200
Dega	2500-3200	10-16	1000-2000
Wurch	>3200	<10	<800

Temperature and solar radiation both have significant implications on aspects of the design of LVR pavements such as the availability of water for compaction purposes or the choice of binder used in bituminous surfacings. These issues are discussed in Sections D.6.3.3, D.6.20.1 and Chapter D.7 respectively.

6.3.2 Rainfall

Rainfall in Ethiopia is variable both in amount and frequency but, as indicated in Figure D.6.4, the general pattern is for rainfall to increase with the increase in distance in a westerly direction. Average annual rainfall is least in the east and south east of the country with a range from about 90 – 500mm and most in the western highlands with a range of 1500 – 2000mm.

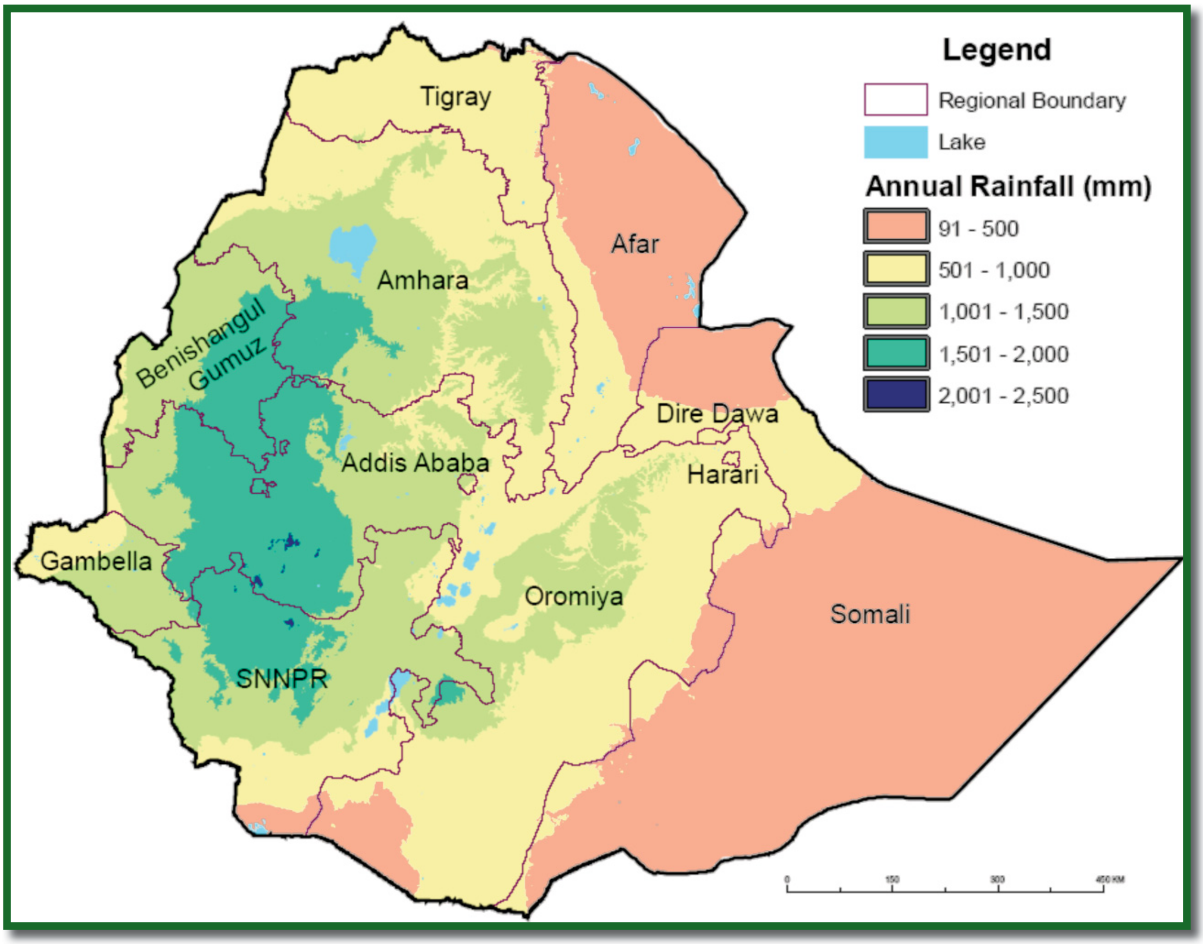

Figure D.6.4: Rainfall pattern in Ethiopia

The direct result of the relatively low precipitation in the eastern and south-eastern areas of the country is scarcity of water. Other than in localised areas where there may be impeded drainage due to impervious, low-lying strata, surface water and shallow water tables are rare. Ground water is usually very deep and soaking of the subgrade does not normally occur other than in localised areas, a factor that needs to be taken into account in correctly assessing the moisture content for assessing the strength of the pavement layers.

In the western, higher rainfall areas of the country, surface run-off may be high leading to erosion of shoulders and side slopes (see Plate D.6.1), increased soil erosion, flash flooding and siltation of waterways from the disturbance of soil. Appropriate design measures must therefore be taken to combat the potential erosive impacts of high/intense rainfall on road performance as discussed in Chapter D.5: Drainage.

Plate D.6.1: Severe erosion of road side slopes in high rainfall area

6.3.3 Moisture in the road pavement

The moisture environment in which a LVR pavement must operate has a particularly significant impact on its performance due to the use of locally occurring, unprocessed materials which tend to be relatively moisture sensitive. Of the sources of moisture entry into a pavement, a number can be controlled through appropriate internal and external design measures which are discussed respectively in Section D.6.18 and Chapter D.5: Drainage.

Evaporation: The relatively low rainfall that occurs in the eastern and south-eastern areas of Ethiopia coincides with long periods of intense sunshine that cause high evaporation rates and a net moisture deficiency. This can affect road construction operations in the summer season in that much more water is required for compaction purposes in a moisture-deficient environment. Various measures should therefore be considered that minimise the evaporation of water during certain construction operations in order to minimise loss by evaporation. Techniques are available for reducing compaction water requirements and are discussed in Section D 6.20.1.

Climatic factor: The climatic descriptor which is used for the pavement design catalogues in this manual is the Weinert 'N' value (Weinert, 1974). This index is calculated as follows:

$$N = 12Ej/Pa \qquad \textbf{Equation 5}$$

where:

Ej = evaporation for the warmest month
Pa = total annual precipitation

N-values less than 4 apply to a climate that is seasonally tropical and wet (the Kolla, Woina Dega, Dega and Wurch regions of Ethiopia), whereas N-values greater than 4 apply to a climate that is arid, semi-arid or dry (the Bereha region of Ethiopia). A map of equivalent N-values for Ethiopia is shown in Figure D.6.5 and provides the means of placing a road in the appropriate climatic zone for design purposes as discussed in Section D 6.17.

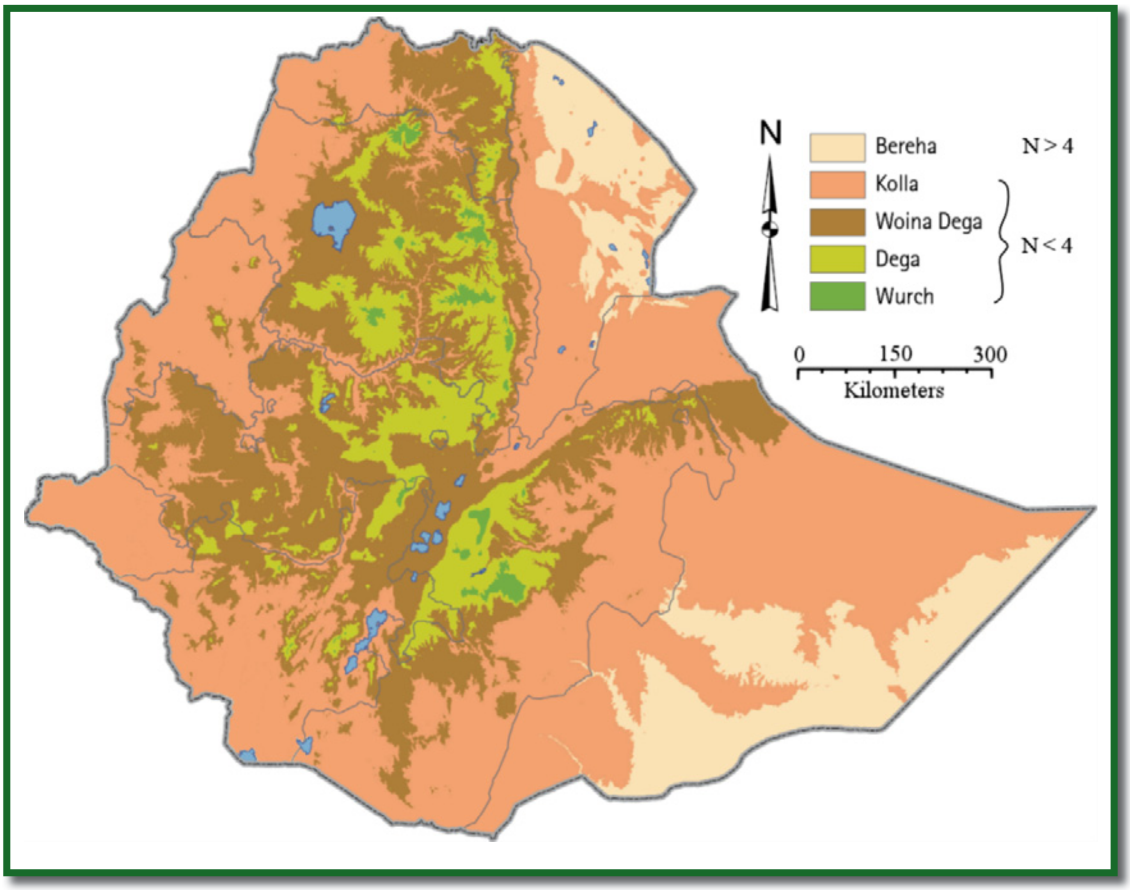

Figure D.6.5: Climatic N-value map for Ethiopia

The climatic zones demarcated by the N-values are macro-climates and it should be kept in mind that different *micro-climates* may occur within these regions. This is particularly important where such local micro-climates can play a significant role in determining the in-situ moisture content of the various pavement layers; a factor which needs to be carefully considered in the choice of N-Value or the subgrade class used for design purposes.

6.3.4 Winds

Wind is a primary erosion and transportation agent in Ethiopia. Strong, hot summer winds are a common occurrence in the eastern and south-eastern areas of the country and have the effect of shifting the finer, generally non-plastic sands, which can cause the build-up of drifts against the edge of the road pavement. Unprotected sand embankments can also migrate under the action of wind for which various design counter-measure are available.

6.4 Surface/Sub-surface hydrology

The surface and sub-surface hydrological features of Ethiopia are characterised by the country's diverse terrain which is responsible for wide regional variations in water flow, whether it be on escarpments, in water courses or in low-lying areas.

In the central and western areas of the country where there can be aggressive water flow within and adjacent to road structures, appropriate internal and external drainage measures are needed to control such water flow as discussed further in Sections D.6.18, D.6.19.4 and Chapter D.5: Drainage.

In the eastern and south-eastern areas of the country there is generally an absence of permanent surface water, although it may accumulate in pans for a short time after rains. Groundwater is found at appreciable depth below the ground surface and is relatively expensive to exploit. In many cases, the water tends to be saline and may be unsuitable for use in road construction – an issue that is discussed further in Section D.6.19.5.

6.5 Subgrade

Subgrades in Ethiopia are inherently variable and reflect the country's diverse geology, topography, soil type, climate and drainage conditions. As the foundation layer for the pavement, the assessment of the subgrade condition in terms of the level of support provided to the pavement structure is one of the most important factors, in addition to traffic loading, in determining pavement thickness design, composition and performance. This level of support, as characterised by subgrade strength or stiffness, is dependent on the soil type, density and moisture conditions at construction and during service. Hence, the selection of a subgrade support value requires careful consideration of the quantity and quality of subgrade data available to the design engineer and the variability of subgrade support within a particular project section. The purpose of subgrade evaluation is therefore to estimate the support that the subgrade will provide to the pavement during its design life.

This section also focuses on the classification of the subgrade in terms of the California Bearing Ratio (CBR) to represent realistic conditions for design. In practice this means determining the CBR strength for *the wettest moisture condition likely to occur during the design life* at the density expected to be achieved in the field.

6.5.1 Subgrade Classification

Subgrades are classified on the basis of the laboratory soaked CBR tests on samples compacted to 97% AASHTO T180 compaction. Samples are soaked for four days or until zero swell is recorded. On this basis, the soaked CBR is used to assign a design subgrade class.

The structural catalogue given in this manual requires that the subgrade strength for design be assigned to one of five strength classes reflecting the sensitivity of thickness design to subgrade strength. The classes are defined in Table D.6.2.

Table D.6.2: Subgrade classes

	Subgrade Class				
Design CBR	S2	S3	S4	S5	S6
Range %	3 - 4	5 - 8	9 - 14	15 - 29	30+

The following points should be noted with the subgrade classes defined in Table D.6.2:

- No allowance for CBRs below 3% has been made because, from both a technical and economic perspective, it would normally be inappropriate to lay a pavement on soils of such poor bearing capacity. Moreover, the measurement of the bearing strength of such soft soils is generally most uncertain and CBRs below 2% are of little significance. For such materials, special treatment is required (see Section 6.19.7).
- The use of Class S2 soils as direct support for the pavement should be avoided as much as possible. Wherever practicable, such relatively poor soils should be excavated and replaced, or covered with an improved subgrade.
- Class S6 covers all subgrade materials having a soaked CBR greater than 30 and which comply with the plasticity requirements for natural sub-base. In such cases, no sub-base is required.

Since the combination of density and moisture content wholly governs the CBR for a given material, it is clear that changes in moisture content will alter the effective CBR in the field, and it is therefore also clear that particular effort must be made to define the design subgrade condition.

The result of incorrect subgrade classification can have significant effects, particularly for poorer subgrade materials with CBR values of 5 per cent and less. If the subgrade strength is seriously overestimated (ie the support is actually weaker than assumed), there is a high likelihood of local premature failures and unsatisfactory performance. Conversely, if the subgrade strength is underestimated (ie the support is stronger than assumed), then the pavement structure selected will be thicker, stronger and more expensive than necessary.

6.5.2 Specifying the design subgrade class

The CBR results obtained from the subgrade soils testing are used to determine which subgrade class should be specified for design purposes in accordance with Table D.6.2. In some cases a variation in results may make selection unclear. In such cases it is recommended that, firstly, the laboratory test process is checked to ensure uniformity (to minimise inherent variation arising from, for example, inconsistent drying out of specimens). Secondly, more samples should be tested to build up a more reliable basis for selection.

Plotting these results as a cumulative distribution curve (S-curve), in which the y-axis is the percentage of samples less than a given CBR value (x-axis), provides a method of determining a design CBR value (Figure D.6.6).

The actual subgrade CBR values used for design depend on the traffic class as shown in Table D.6.3. Thus, as indicated in the Table, for a design traffic class of LV5, the design CBR value should be the lower 10th percentile (ie the value exceeded by 90% of the CBR measurements).

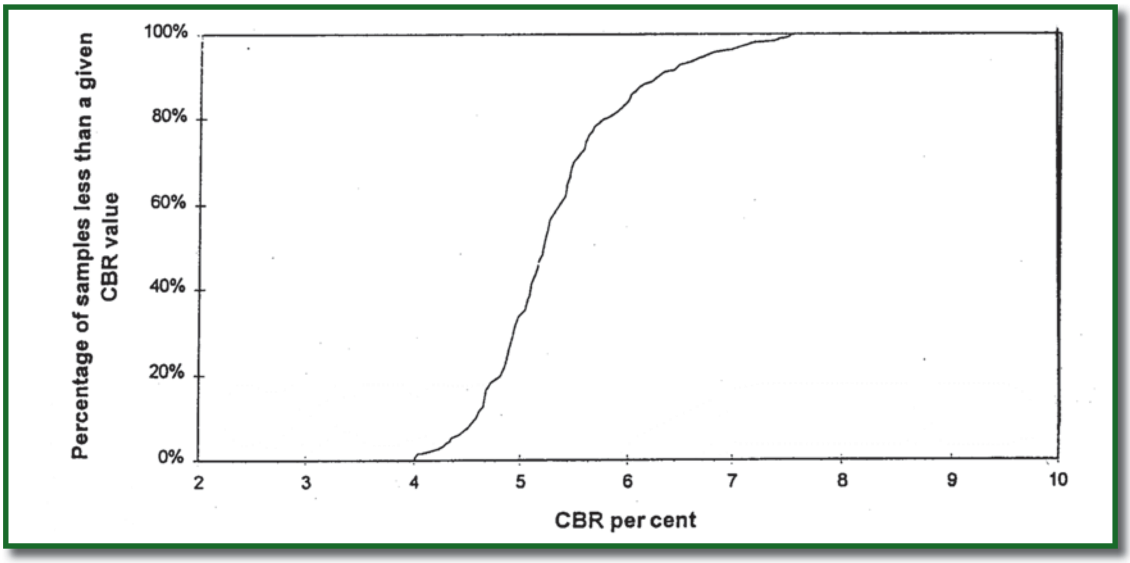

Figure D.6.6: Illustration of CBR strength cumulative distribution

Table D.6.3: Design CBR values related to Traffic Classification

Traffic class	Design CBR
LV5 (0.5-1.0 Mesa)	Lower 10-percentile
LV3 and LV4 (0.1-0.5 Mesa)	Lower 15-percentile
LV1 and LV2 (<0.1 Mesa)	30th percentile

6.5.3 Material depth

It is critical that the nominal subgrade strength is available to a reasonable depth in order that the pavement structure performs satisfactorily. The concept of “material depth” is used to denote the depth below the finished level of the road to which soil characteristics have a significant effect on pavement behaviour. In addition, the moisture regime may need to be controlled by, for example, the provision of adequate subsurface drainage and/or surface drainage. Below this depth the strength and density of the soils are assumed to have a negligible effect on the pavement. The depth approximates the cover required for a soil of less than 3% soaked CBR (ie less than Subgrade Class S2). However, this depth may be insufficient in certain special cases where "problem" soils occur (See Section D.6.19).

Figure D.6.7 shows the material depth in relation to the main structural components of the road pavement, while Table D.6.4 specifies typical material depths used for determining the design CBR of the subgrade for the subgrade classes given in Table D.6.2.

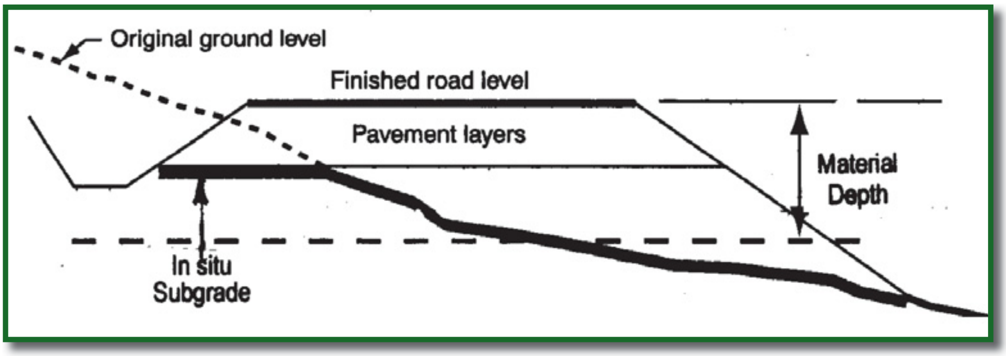

Figure D.6.7: Material depth

Table D.6.4: Typical material depth by road category

Road Category		Material Depth (mm)
DC 7 and DC 8	High volume roads	1,000 – 1,200
DC 5 and DC 6		800 – 1,000
DC 3 and DC 4	Low volume roads	800
DC 1 and DC 2		700

It should be clearly understood that the minimum depths indicated in Table D.6.4 are not depths to which re-compaction and reworking is necessarily required. Rather, they are the depths to which the Engineer should confirm that the nominal subgrade strength is available. In general, unnecessary working of the subgrade should be avoided and limited to rolling prior to constructing overlying layers.

For the stronger subgrades, especially Class S4 and higher (CBR 9-14% and more) the depth check is to ensure that there is no underlying weaker material which could lead to detrimental performance.

It is recommended that the Dynamic Cone Penetrometer (DCP) be used during construction to monitor the uniformity of subgrade support to the recommended minimum depths given in Table D.6.4.

6.5.4 Dealing with poor subgrade soils

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

- The excessive volume changes that occur in some soils as a result of moisture change (ie expansive soils and soils with a collapsible structure);
- The non-uniform support that results from wide variations in soil types over the road length;
- The presence of soluble salts which, under unfavourable conditions, may migrate upwards leading to several problems, including cracking of the surfacing;

- The excessive deflection and rebound of highly resilient soils during and after the passage of a load (eg micaceous soils).

Measures for dealing with these "problem" soils are addressed in Section D.6.19.

6.5.5 Improved subgrade layers

There are many advantages to improving the CBR strength of the in-situ subgrade to a minimum of 15% (Subgrade Class S5) by constructing one or more improved layers where necessary. In principle, where a sufficient thickness of improved subgrade is placed, the overall subgrade bearing strength is increased to that of a higher class and the sub-base thickness may be reduced accordingly. This is often an economic advantage as sub-base quality materials are generally more expensive than fill materials.

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

- Provision of uniform subgrade strength;
- Protection of underlying earthworks;
- Improved compaction of layers above subgrade level;
- Provision of a more balanced pavement structure;
- Provision of a running surface for the traffic during construction;
- Provision of a gravel wearing course in the case of stage construction for future upgrading to a paved road;
- More economical use of pavement materials (thinner layers).

An improved subgrade placed on soils of any particular class must obviously be made of a material of a higher class (up to Class S5, since Class S6 is of sub-base quality). The decision whether or not to consider the use of an improved subgrade layer(s) will generally depend on the respective costs of sub-base and improved subgrade materials.

6.6 Traffic

Determination of the amount and type of traffic is one of the most important factors in the design of LVR pavements and will influence not only the type of surface needed (earth, gravel or bituminous) but also the pavement thickness. The types of traffic using LVRs in Ethiopia vary significantly and include both motorised and non-motorised traffic involving a wide spectrum of road users from pedestrians to 3-wheeled rickshaws, motor cycle taxis to large commercial vehicles. Appropriate traffic surveys are required to provide the information necessary for both geometric and pavement design as discussed in Part B.

The deterioration of paved and unpaved roads caused by traffic results from both the magnitude of the individual wheel loads and the number of times these loads are applied. It is necessary to consider not only the total number of vehicles that will use the road but also the axle loads of these vehicles. Traffic classes are defined for both paved and unpaved roads by ranges of the cumulative number of equivalent standard axles (esas) (see Section D.6.6.2 below) and indicate the pavement structure requirements.

6.6.1 Estimating design traffic loading

The process by which the design traffic loading is estimated is illustrated in Figure D.6.8.

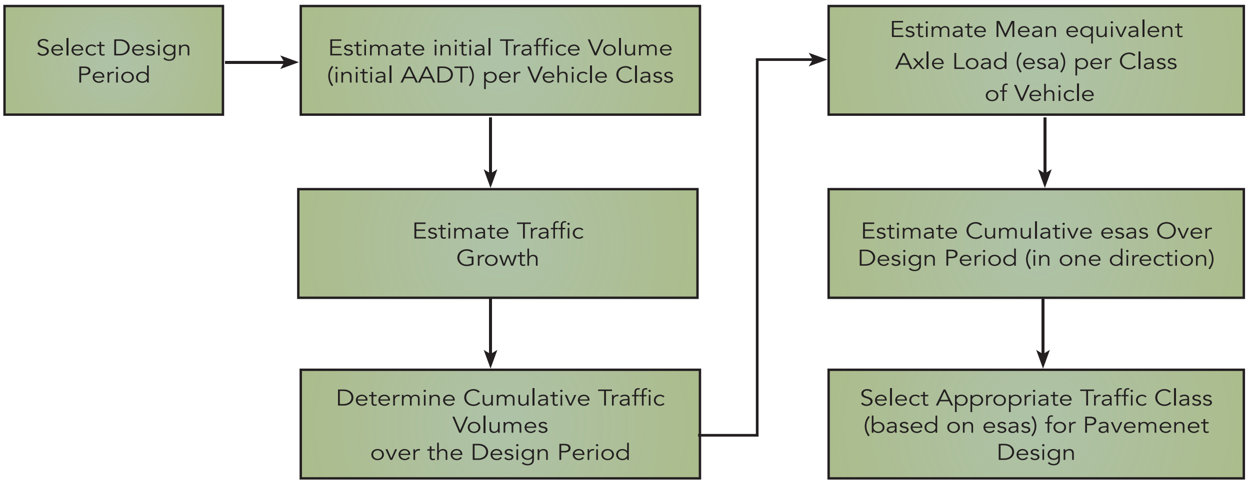

Figure D.6.8: Procedure to determine design traffic loading

In order to determine the cumulative number of vehicles over the structural design period of the road, the following steps are required:

- Determine the initial traffic volume ($AADT_0$) using the results of a traffic survey and any other recent traffic count information that is available. For paved roads, details of the AADT in terms of car, bus, types of truck, and truck-trailers are required (see Table D.6.6).
- Estimate the annual growth rate "i" expressed as a decimal fraction, and the anticipated number of years "x" between the traffic survey and the opening of the road.
- Determine $AADT_1$ the traffic volume in *both directions* in the year of the road opening by:

$$AADT_1 = AADT_0 \, (1+i)^x \qquad \textbf{Equation 6}$$

For paved roads, also determine the corresponding daily one-directional traffic volume for each type of vehicle.

- The cumulative number of vehicles, T over the chosen design period N (in years) is obtained by:

$$T=365.AADT_1 \, [(1+i)N-1]/(i) \qquad \textbf{Equation 7}$$

For paved roads, conduct a similar calculation to determine the cumulative volume in each direction for each type of vehicle.

Construction traffic can also be a significant proportion of total traffic on LVRs (sometimes 20 – 40% of total traffic) and should be taken into account in the design of the pavement.

For very low volume roads (traffic <25 vpd), an elaborate traffic analysis is seldom warranted as environmental rather than traffic loading factors generally determine the performance of roads.

6.6.2 Equivalent standard axles per vehicle class

The damage that vehicles impart on paved or unpaved road is highly dependent on the magnitude of the axle loads of the vehicles and the number of times they are applied. Axle load data for design purposes should preferably 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. Where this is not possible, recourse may be made to historical information.

The damaging power of axles is related to a "standard" axle of 8.16 metric tons using empirical equivalency factors (EFs). The number of standard axles for each vehicle is the sum of the standard axles for each axle of the vehicle.

In order to determine the cumulative axle load damage that a pavement will sustain during its design life, it is necessary to express the total number of heavy vehicles that will use the road over this period in terms of the cumulative number of equivalent standard axles (esas) for all the vehicles.

The relationship between an axle's equivalency factor, EF, and the axle loading is:

$$EF = (P/8160)^n \text{ (for loads in kg)} \qquad \textbf{Equation 8}$$

or

$$EF = (P/80)^n \text{ (for loads in kN)} \qquad \textbf{Equation 9}$$

Where:

EF = load equivalency factor in esas

P = axle load (in kg or kN)

n = relative damage exponent

The value of 4 or 4.5 for the exponent n is often used in line with early findings and the commonly cited "fourth-power damage effects" of heavy axle loads. It is now clear that the value is influenced by various factors, with the most significant being the type of materials used in the pavement structure (eg granular/granular, granular/cemented, bituminous/cemented) and the thickness of the pavement. For LVRs, which will normally comprise granular materials in both the base and subbase, the recommended relative damage exponent 'n' is 4 (Table D.6.5).

General guidance on the likely average total equivalency factors for different vehicle types derived from historical data in Ethiopia is given in Table D.6.6. However, data from any recent axle load surveys on the road in question, or a similar road in the vicinity, is better than using countrywide averages.

The cumulative esas over the design period (N years) are calculated as the products of the cumulative one-directional traffic volume (T) for each class of vehicle and the mean equivalency factor for that class of vehicle. These are added together for all the vehicle classes for each direction. In some cases there will be distinct differences in each direction and separate vehicle damage factors for each direction should be derived. The higher of the two directional values should be used for design.

On narrow roads the traffic tends to be more channelised than on wider two lane roads. In such cases, the effective traffic loading has been shown to be greater than that for a wider road. Table D.6.7 illustrates the effect. The actual design traffic loading (esas) is calculated from Table D.6.7 using the design carriageway widths and type of road to finalise the design values.

Table D.6.5: Load equivalency factors for different axle load groups (esas)

Axle loads measured in kg		Axle loads measured in kN	
Axle load range (kg)	EF	Axle load range (kN)	EF
Less than 1500	-	Less than 15	-
1500 - 2499	-	15 - 24	-
2500 - 3499	0.02	25 - 34	.02
3500 - 4499	0.06	35 - 44	.06
4500 - 5499	0.15	45 - 54	.15
5500 - 6499	0.30	55 - 64	.32
6500 - 7499	0.56	65 - 74	.58
7500 - 8499	0.95	75 - 84	.99
8500 - 9499	1.5	85 - 94	1.6
9500 - 10499	2.3	95 - 104	2.4
10500 - 11499	3.3	105 - 114	3.6
11500 - 12499	4.7	115 - 124	5.0
12500 - 13499	6.5	125 - 134	6.9
13500 - 14499	8.7	135 - 144	9.3
14500 - 15499	11.5	145 - 154	12
15500 - 16499	15	155 - 164	16
16500 - 17499	19	165 - 174	20
17500 - 18499	24	175 - 184	25
18500 - 19499	30	185 - 194	32
19500 - 20499	36	195 - 204	39

Table D.6.6: Average equivalency factors for different vehicle types

Class	Type	No of axles	Average esa per vehicle - all loaded	Average esa per vehicle - half loaded[1]
1	Car	2	-	-
2	4-wheel drive	2	-	-
3	Minibus taxi	2	0.3	0.15
4	Bus/coach	2	2.0	1.0
5	Small truck/small bus	2	1.5	0.7
6	Medium truck	2	5	2.5
7	Large 2-axled truck	2	10	5
8	3-axled truck	3	12	3.5
9	4-axled truck	4	15	7.5
10	5-axled truck	5	17	8.5
11	6-axled truck	6	17	8.5
12	3-axled trailer	3	10	5
13	4-axled trailer	4	12	5

Notes:

1. It is common to find that vehicles have no back load hence half the vehicles are likely to be empty, or nearly so.

Table D.6.7: Factors for design traffic loading

Cross Section	Paved width	Corrected design traffic loading (esa)	Explanatory notes
Single carriageway	< 3.5m	Double the sum of esas in both directions	The driving pattern on this cross-section is very channelized.
	Min. 3.5m but less than 4.5m	The sum of esas in both directions	Traffic in both directions uses the same lane
	Min. 4.5m but less than 6m	80% of the sum of esas in both directions	To allow for overlap in the centre section of the road
	6m 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.

6.6.3 Design traffic classes

All survey data are subject to errors. Traffic data, in particular, can be very inaccurate and predictions about traffic growth are also prone to large errors. Accurate calculations of cumulative traffic are therefore very difficult to make. To minimize these errors there is no substitute for carrying out specific traffic surveys for each project. Methods of carrying out classified traffic counts are described in ERAs 2011 Manual series.

The design method recommended in this manual provides fixed structures for ranges of traffic as shown in Table D.6.8. Therefore, as long as the estimate of cumulative equivalent standard axles is not near one of the traffic class boundaries, traffic errors are unlikely to affect the choice of pavement design. However, if estimates of cumulative traffic are close to the boundaries of a traffic class, then the basic traffic data and forecasts should be re-evaluated and sensitivity analyses carried out to ensure that the choice of traffic class is appropriate. If there is any doubt about the accuracy of the traffic estimates, it is prudent to select the next higher traffic class for design.

Table D.6.8: Traffic classes for pavement design

	Traffic class				
Traffic range (ESA x 10^6)	LV1	LV2	LV3	LV4	LV5/T2(1)
	< 0.01	0.01 – 0.1	0.1 – 0.3	0.3 – 0.5	0.5 – 1.0

Note:

1. LV5/T2 is the transition traffic zone between low-volume/high-volume roads with the former traffic class (LV5) applying to the lower boundary of the traffic range and the latter traffic class (T2) applying to the upper boundary.

6.7 Construction Materials

The maximum use of naturally occurring unprocessed materials is a central pillar of the LVR design philosophy. Current specifications tend to exclude the use of many naturally occurring, unprocessed materials (natural soils, gravel-soil mixtures and gravels) in pavement layers in favour of more expensive crushed rock, because they often do not comply with traditional (HVR-orientated) requirements. However, recent research work has shown quite clearly that so-called "non-standard" materials[2] can often be used successfully and cost-effectively in LVR pavements provided appropriate precautions are observed as discussed in this chapter.

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 properties and behaviour in the prevailing environment.

Ethiopia's diverse geology provides a variety of igneous, sedimentary and metamorphic rocks as well as transported and residual soils, many of which are potentially suitable for use as road construction materials. The selection of these materials for incorporation in a road pavement is generally based on a combination of such factors as availability, structural requirements, environmental considerations, method of construction, economics, previous experience and, of course, quality. These factors need to be evaluated during the pavement design process in order to select the materials that are most suitable for the prevailing conditions. An assessment of the durability of these materials is a key factor in the selection process (See ERA Pavement Design Manual - 2011).

6.7.1 Performance characteristics of the pavement materials

Some understanding of the characteristics of the pavement material is necessary prior to any discussion about the design of LVRs. Table D.6.9 summarises the typical characteristics of unbound and bound materials that critically affect the way in which they can be incorporated into a pavement.

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.

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

Category 2: materials have a moderate dependency on all forms of shear resistance (ie friction, suction forces and cohesion). These materials also have rather limited strength potential and therefore levels of moisture, typically 60-80% of saturation, may be sufficient to reduce the strength contribution from suction or cohesion, leading to premature distress and failure. This occurs at moisture contents lower than those necessary to generate pore pressures.

Category 3: 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.

More than anything else, the management of moisture during the construction and operational phases of a pavement affects its eventual performance, especially when unbound, unprocessed materials are used. Table D.6.10 shows the variation of laboratory CBR with moisture content expressed as the ratio of the field (or in-situ) moisture content (FMC) and the optimum moisture content for compaction (OMC) (Emery, 1985). The Table illustrates the significant effect of drying out of materials of varying quality on their strength (CBR).

As clearly indicated, if such materials can be maintained in a relatively dry state in service, then they can be expected to perform satisfactorily at this "elevated" strength provided appropriate precautions are taken to avoid their wetting up. Such precautions are discussed in Section D.6.18.

Table D.6.9: Pavement material categories and relative characteristics

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

	Of particular significance for LVRs

Table D.6.10: Variation of CBR with moisture content

Material Class	Laboratory Soaked CBR (%)	Laboratory Unsoaked CBR (%) at Varying FMC/OMC Ratios		
		1.0	0.75	0.50
G80	80	105	150	200
G65	65	95	135	185
G55	55	90	125	175
G45	45	80	115	165
G30	30	65	95	140
G25	35	60	90	135
G15	15	45	70	110
G10	10	35	60	100
G7	7	30	50	85

6.7.2 Pavement material types

The material code and outline characteristics of the material types for both paved and unpaved LVRs that are used in the Catalogue of Designs adopted in the manual are described in Table D.6.11.

Table D.6.11: Pavement material types and abbreviated nominal specifications used in the paved and unpaved catalogue of designs

Code	Material	Abbreviated Specifications
G80	Natural gravel	Min. CBR: 80% @ 98/100% AASHTO T180 and 4 days soaking Max. Swell: 0.2% Max. Size and grading: Max size 37.5mm, grading as specified. PI: < 6 or as otherwise specified (material specific).
G65	Natural gravel	Min. CBR: 65% @ 98/100% AASHTO T180 and 4 days soaking Max. Swell: 0.2% Max. Size and grading: Max size 37.5mm, grading as specified PI: < 6 or as otherwise specified (material specific)
G55	Natural gravel	Min. CBR: 55% @ 98/100% AASHTO T180 and 4 days soaking Max. Swell: 0.2% Max. Size and grading: Max size 37.5mm, grading as specified PI: < 6 or as otherwise specified (material specific)
G45	Natural gravel	Min. CBR: 45% @ 98/100% AASHTO T180 and 4 days soaking Max. Swell: 0.2% Max. Size and grading: Max size 37.5mm, grading as specified PI: < 6 or as otherwise specified (material specific)
G30	Natural gravel	Min. CBR: 30% @ 95/97% AASHTO T180 & highest anticipated moisture content Max. Swell: 1.0% 1.5% @ 100% AASHTO T180 Max. Size and grading: Max size 63mm or 2/3 layer thickness PI: < 12 or as otherwise specified (material specific)
G25	Natural gravel	Min. CBR: 30% @ 95/97% AASHTO T180 & highest anticipated moisture content Max. Swell: 1.0% @ 100% AASHTO T180 Max. Size and grading: Max sixe 63mm or 2/3 layer thickness. PI: <12 or as otherwise specified (material specific)
G15	Gravel/soil	Min. CBR: 15% @ 93/95% AASHTO T180 & highest anticipated moisture content Max. Swell: 1.5% @ 100% AASHTO T180 Max. Size: 2/3 of layer thickness PI: < 12 or 3GM + 10 or as otherwise specified (material specific)
G7	Gravel/soil	Min. CBR: 7% @ 93/95% AASHTO T180 & highest anticipated moisture content Max. Swell: 1.5% @ 100% AASHTO T180 Max. Size: 2/3 layer thickness PI: < 12 or 3GM + 10 or as otherwise specified (material specific)
G3	Gravel/soil	Min. CBR: 3% @ 93/95% AASHTO T180 & 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 realistically attained in the field (from field measurements on similar materials or other established information) they should be specified by the Engineer.

6.7.3 Materials requirements for road base

A wide range of materials including lateritic, calcareous and quartzitic gravels, river gravels and other transported and residual gravels, or granular materials resulting from weathering of rocks can be used successfully as road base materials.

Particle size distribution: The grading envelopes to be used for road base are shown in Table D.6.12. Envelope A varies depending whether the nominal maximum particle size is 37.5mm, 20mm or 10mm. A requirement of five to ten per cent retained on successive sieves may be specified at higher traffic (>0.3 Mesa) to prevent excessive loss in stability. Envelope C extends the upper limit of envelope B to allow the use of sandy materials, but its use is not permitted in wet climates.

Envelope D is similar to a gravel wearing course specification and is used for very low traffic volumes. The grading is specified only in terms of the grading modulus (GM) and can be used in both wet and dry climates.

Table D.6.12: Particle size distribution for natural gravel base

Test Sieve size	Per cent by mass of total aggregate passing test sieve				
	Envelope A Nominal maximum particle size			Envelope B	Envelope C
	37.5mm	20mm	10mm		
50mm	100			100	
37.5mm	80-100	100		80-100	
20mm	55-95	80-100	100	55-100	
10mm	40-80	55-85	60-100	40-100	
5mm	30-65	30-65	45-80	30-80	
2.36mm	20-50	20-50	35-75	20-70	20-100
1.18mm	-	-	-	-	-
425µm	8-30	12-30	12-45	8-45	8-80
300µm	-	-	-	-	-
75µm	5-20	5-20	5-20	5-20	5-30
Envelope D 1.65 < GM < 2.65					

Strength and plasticity: The strength requirement varies depending on the traffic level and climate, as outlined in the Catalogue of Structures in Part B (Figures B.5.3, B.5.4 and B.5.5. The soaked CBR test is used to specify the minimum strength of road base material.

The plasticity requirement also varies depending on the traffic level and climate as shown in Tables D.6.13 and D.6.14. A maximum plasticity index of 6 has been retained for higher traffic levels, where the design chart merges to standard design documents, 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 Section D.6.17.2 and Figure D.6.22.

Table D.6.13: Plasticity requirements for natural gravel road base materials

Subgrade class[4]	Property of base	Traffic class (mesas)				
		LV1	LV2	LV3	LV4	LV5
		<0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2	Ip PM Grading	<12 <400 B	<9 <150 B	<6 <120 A[(5)]	<6 <90 A[(5)]	<6 <90 A[(5)]
S3	Ip PM Grading	<15 <550 C[1]	<12 <250 B	<9 <180 B	<6 <90 A[(5)]	<6 <90 A[(5)]
S4	Ip PM Grading	Note[(2)] <800 D[(3)]	<12 <320 B	<12 <300 B	<9 <200 B	<9 <90 A[(5)]
S5	Ip PM Grading	Note[(2)] - D[(3)]	<15 <400 B	<12 <350 B	<12 <250 B	<9 <150 A[(5)]
S6	Ip PM Grading	Note[(2)] - D[(3)]	<15 <550 C[(1)]	<15 <500 B	<12 <300 B	<9 <180 A[(5)]

Notes:

1. Grading 'C' is not permitted in wet environments or climates (N<4); grading 'B' is the minimum requirement
2. Maximum Ip = 8 x GM
3. Grading 'D' is based on the grading modulus 1.65 < GM < 2.65
4. All base materials are natural gravels; Subgrades are non-expansive
5. Envelope A varies depending on whether the nominal maximum particle size is 37.5, 20 or 10mm

Lateritic road base gravels: Lateritic gravels occur in the west and northwestern parts of Ethiopia in the Guraghe, Jimma, Wollega and Gojam highlands. 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.

The behaviour of lateritic materials in pavement structures depends mainly on their 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, or quartz 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 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 slightly different to those given for other natural gravels. These are presented in Table D.6.14. The maximum plasticity index of the lateritic road base is also relaxed in comparison to Table D.6.13. A maximum plasticity index of 9 has been specified for higher traffic levels and weak subgrades. 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 per cent. A minimum field compacted dry density of 2.0 mg/m^3 is required for these materials.

Table D.6.14: Guidelines for the selection of lateritic gravel road base materials

Subgrade class	Property	Traffic class (mesas)				
		LV1	LV2	LV3	LV4	LV5
		<0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2	Ip PM Grading	<15 <400 B	<12 <150 B	<9 <150 A	<9 <120 A	<6 <90 A
S3	Ip PM Grading	<18 <550 C(1)	<15 <250 B	<12 <180 B	<9 <120 A	<6 <90 A
S4	Ip PM Grading	<20[1] <800 GM 1.6-2.6	<15 <320 B	<15 <300 B	<9 <200 B	<9 <90 A
S5	Ip PM Grading	<25(1) - GM 1.6-2.6	<18 <400 B	<15 <350 B	<12 <250 B	<9 <150 B
S6	Ip PM Grading	<25(1) - GM 1.6-2.6	<20 <550 B	<18 <400 B	<15 <300 B	<12 <180 A

Notes:

1. Maximum Ip = 8 x GM
2. Unsealed shoulders are assumed. Further modification to the limits can be made if the shoulders are sealed.
3. The compaction requirement for the soaked CBR test to define the subgrade classes is 100% Mod. AASHTO with a minimum soaking time of 4 days or until zero swell is recorded. This is a relaxation of the soaked CBR requirement for natural gravel base materials given in the catalogues.

Basic igneous rock (including basaltic and doleritic gravels): These materials occur extensively in Ethiopia and their more wide-spread use could result in significant savings provided the characteristics of the material are good enough to serve as a road base material. However, more research work is required before these materials can be used with confidence. Nonetheless, the following indicative limits can contribute to successful use of the material in road bases:

- Maximum secondary mineral content of 20 per cent (determined from petrographic analysis);
- Maximum loss of 12 or 20 per cent after 5 cycles in the sodium or magnesium sulphate soundness tests, respectively;
- Clay index of less than 3 in the dye absorption test;
- Increase in modified glycol-soaked aggregate impact value (AIV) from wet modified AIV should be <4% units;
- Durability mill index of less than 125.

In drier climatic areas (N>4), the materials 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 (these materials are particularly sensitive to moisture);
- The adequacy of the pavement design (the use of Pavement Catalogue 2 with sealed shoulders is suggested);
- The use of double surface treatments or similar.

Engineers need to use considerable judgement, experience and information from other roads in the area to utilise these materials successfully. Risks must be identified and controlled.

Further information on the recommended durability specification limits for different types or road base aggregates are given by the South African Roads Board report PR 88/032:1102 of 1990.

Cinder gravels: Volcanic cinder gravels are pyroclastic materials associated with recent volcanic activity. Such materials occur extensively in Ethiopia, characteristically as straight-sided cone-shaped hills which frequently have large concave depressions in their tops or sides where mixtures of solids and gases were released during the formation of the cone. Cinders vary in colour often within the same cone and may be red, brown, grey or black. The cinder particles also vary in size from large irregularly shaped lumps 50 cm in size, to sand silt sizes. In some cases, however, particles may be more uniform with the largest size not exceeding 3 cm in diameter. Other characteristic features of cinders are their light weight, their rough vesicular surface and their high porosity.

An advantage of cinders as a road construction material is the relative ease with which they can be dug from the quarry; a mechanical shovel or hand tools are usually adequate for their extraction although occasionally a bulldozer may be required. However, despite their extensive occurrence in Ethiopia, cinders have been used for road construction only to a limited extent. Fortunately, however, this trend has been reversed by research that has been carried out into the use of cinders (TRL, 1987). From full-scale trials constructed in the late 1970s using cinders sourced from Modjo and Bekojo, it was concluded that such gravels are capable of carrying in excess of 0.44 mesa when sealed with a surface dressing and designed according to Road Note 31. Thus, with careful selection, cinder gravels typical of those used in the full-scale trials can be used for road bases for LVRs with design traffic loadings up to about 0.5 Mesa.

The results of the full-scale trials also revealed that for gravel surfaced roads, mechanically stabilized cinder gravels perform better than as-dug cinders which lack plastic binder even after the breakdown of the cinder gravels. Recommended gradings for materials which are more resistant to corrugations are presented in the TRL report (Newill et al, 1987).

6.7.4 Material requirements for sub-base

Strength requirements: A minimum CBR of 30% is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95% (preferably 97% where practicable) AASHTO T180 compaction.

Under conditions of good drainage and when the water table is not near the ground surface, the field moisture content under a sealed pavement will be equal to or less than the optimum moisture content in the AASHTO T180 compaction test. In such conditions, the sub-base material should be tested in the laboratory in an unsaturated state.

If the road base allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the road base is pervious, saturation of the sub-base is likely. In these circumstances the bearing capacity should be determined on samples soaked in water for a period of four days. The test should be conducted on samples prepared at the density and moisture content likely to be achieved in the field.

Particle size distribution and plasticity requirements: In order to achieve the required bearing capacity, and for uniform support to be provided to the upper pavement, limits on soil plasticity and particle size distribution may be required. Materials which meet the recommendations of Tables D.6.15 and D.6.16 will usually be found to have adequate bearing capacity.

Table D.6.15: Typical particle size distribution for sub-bases

Sieve Size (mm)	Per cent by mass of total aggregate passing test sieve
50	100
37.5	80 – 100
20	60 – 100
5	30 – 100
1.18	17 – 75
0.3	9 – 50
0.075	5 - 25

Table D.6.16: Plasticity characteristics for granular sub-bases

Climate	Liquid Limit	Plasticity Index	Linear Shrinkage
Moist tropical and wet tropical (N<4)	< 35	< 6	< 3
Seasonally wet tropical (N<4)	< 45	< 12	<6
Arid and semi-arid (N>4)	<55	< 20	<10

6.7.5 Material requirements for gravel wearing course

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

- Good skid resistance;
- Smooth riding characteristics;
- Cohesive properties;
- Resistance to ravelling and scouring;
- Wet and dry stability;
- Low permeability;
- Load spreading ability.

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

Performance-related specifications: Performance related specifications for wearing course materials have been developed for southern Africa based on extensive sampling, testing and monitoring of a large number of test sections (Paige-Green, 1989). These specifications have been successfully implemented in a number of African countries and are considered to be generally applicable to the Ethiopian environment. The specifications identify the most suitable materials in terms of two basic soil parameters – Shrinkage Product and Grading Coefficient – which are determined from particle size distribution and linear shrinkage tests as shown in Figure D.6.9 and defined in Table D.6.18.

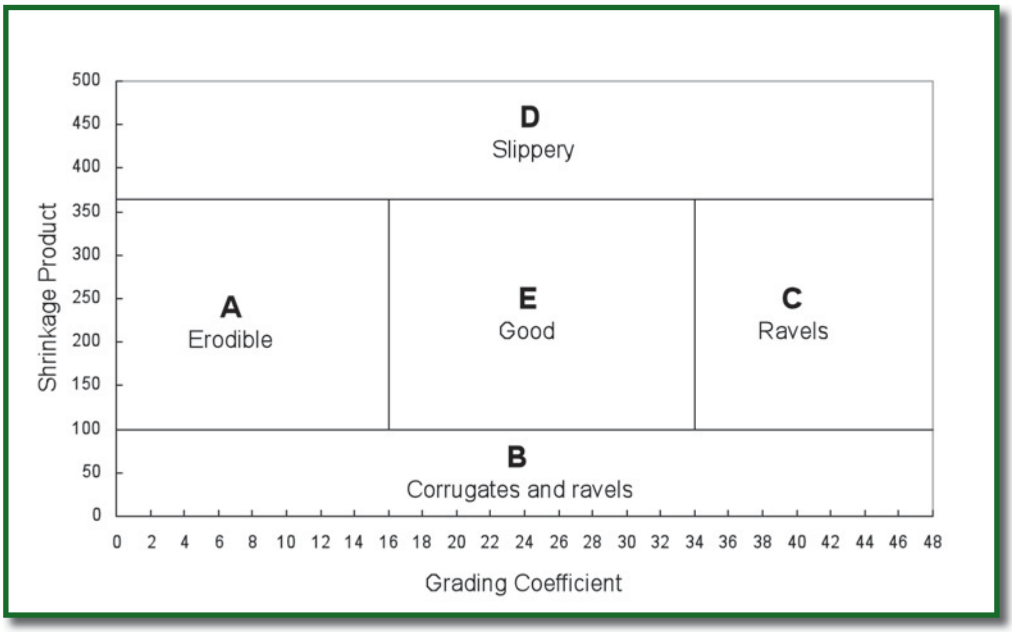

Figure D.6.9: Material quality zones

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

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

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

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

D: Materials with a shrinkage product in excess of 365 tend to be slippery when wet.

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

Gravel loss: Gravel loss is the single most important reason why gravel roads are expensive in whole life cost terms and often unsustainable, especially when traffic levels increase. Reducing gravel loss by selecting better quality gravels or modifying the properties of poorer quality materials is one way of reducing long term costs.

Based on research work carried out in Ethiopia (TRL, 2008), standardised gravel losses (gravel loss in mm/year/100vpd) were determined in relation to the quality of the gravel wearing course (Table D.6.17).

Table D.6.17: Typical standardised gravel loss

Material Quality Zone[1]	Material Quality	Typical gravel loss (mm/yr/100vpd)
Zone A	Satisfactory	20
Zone B	Poor	45
Zone C	Poor	45
Zone D	Marginal	30
Zone E	Good	10

Note:

1. See Figure D.6.9.

The gravel losses shown in Table D.6.17 probably hold only for the first phase of the deterioration cycle lasting possibly two or three years. Beyond that period, as the wearing course is reduced in thickness, other developments, such as the formation of ruts, will also affect the loss of gravel material. However, the rates of gravel loss given above can be used as an aid to the planning for regravelling in the future. A more accurate indication of gravel loss for a particular section of road can be obtained from periodic measurement of the gravel layer thickness.

Material requirements for gravel roads in rural areas: The following specifications for materials for gravel roads in rural areas are recommended in Table D.6.18.

Table D.6.18: Recommended material specifications[1,3] for unsealed rural roads

Maximum size (mm)	37.5
Oversize index (I_o)[a]	≤ 5 %
Shrinkage product (S_p)[b (2)]	100 - 365 (max. of 240 preferable)
Grading coefficient (G_c)[c (2)]	16 - 34
Soaked CBR (at 95 per cent Mod AASHTO)	≥ 15 %
Treton impact value (%) [(4)]	20 – 65

a I_o = Oversize index (percent retained on 37.5 mm sieve
b S_p = Linear shrinkage x percent passing 0.425 mm sieve
c G_c = (Percentage passing 26.5 mm - percentage passing 2.0 mm) x percentage passing 4.75 mm)/100

Notes:

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

Material requirements for gravel roads in 'urban' areas: The specifications in Table D.6.19 are recommended for gravel roads in urban areas where there is a significant number of dwellings and local businesses. In comparison with the limits for gravel roads in rural areas, the limits for the oversize index have been reduced to eliminate stones whilst the shrinkage product has been reduced to a maximum of 240 to reduce the dust as far as practically possible. This lower limit reduces the probability of having unacceptable dust from about 70% to 40%. However, for such areas a sealed road surface is preferred.

Table D.6.19: Recommended material specifications for unsealed 'urban' roads

Maximum size (mm)	37.5
Oversize index (I_o)	0
Shrinkage product (S_p)	100 - 240
Grading coefficient (G_c)	16 - 34
Soaked CBR (at 95 per cent Mod AASHTO)	≥ 15 %
Treton impact value (%)	20 – 65

6.7.6 Material Improvement

Obtaining materials that comply with the necessary grading (particle size distribution-PSD) and plasticity specifications for a gravel wearing course in Ethiopia 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 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 improvement measures are discussed briefly below:

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, 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 (see Plate D.6.2).
- **Rock crusher:** The "Rockbuster" is a patented plant item which is basically a tractor-towed hammermill. The hammermill 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 rill 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 (see Plate D.6.3)

Plate D.6.2: Grid roller

Plate D.6.3: Rock Buster

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

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

Such stabilisation can achieve the following:

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

The following methodology, using a ternary diagram (Figure D.6.10), has been developed for determining the optimal mix ratio for blending two or more materials to meet the required specification:

- Identify potential material sources that can be used to improve the available material;
- 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);
- 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;
- Plot the material properties on the ternary diagram as points a and b respectively (see example in Figure D.6.10);
- 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;
- The mix proportions are then the ratio of the line ac:bc. This can be equated to truck loads and dump spacing;
- Once the mix proportions have been established, the Atterberg Limits of the mixture should be determined to check that the shrinkage product is within the desirable range (100 – 365 (or 240 if necessary)). The 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.

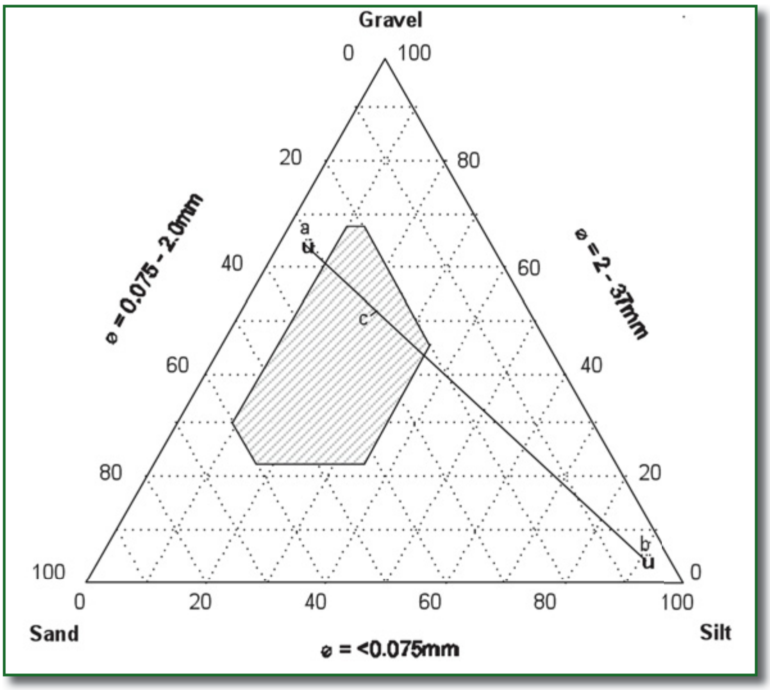

Figure D.6.10: Ternary diagram for blending unsealed road materials

Example

Source material 1 - Grading coefficient of 20 and a shrinkage product of zero. This material plots in Zone B of the specification and is therefore likely to corrugate and ravel.

Source material 2 - Grading coefficient of 4 and shrinkage product of 470. This material plots in Zone D of the specification and would typically be dusty when dry and slippery when wet.

The particle size distributions and other relevant data of each material are provided below:

Parameter	Material	
	A	B
% passing screen size (mm)		
37.5	100	100
26.5	85	100
4.75	42	97
2.0	38	96
0.425	20	94
0.075	7	92
Linear shrinkage	NP	5
Shrinkage product	0	470
Grading coefficient	20	4
% silt/clay (P075)	7	92
% sand (P2 - P075)	31	4
% gravel (100 - P2)	62	4

The relative proportions for each material are plotted onto the ternary diagram as points a and b which are then connected (Figure D.6.10). The midpoint of the line within the shaded area is located at point c. The mix proportions are thus the ratio of the line ac:ab. In this instance, the ratio is approximately 1:4, which indicates that one part of Material B should be mixed with four parts of Material A (ie, one truck load of Material B for every four truck loads of Material A). After blending, the grading coefficient and shrinkage product are 18 and 138 respectively, which fall within Zone E of the specification.

6.8 Terrain

The Ethiopian topography is characterised by a blend of massive highlands, highly rugged terrain, valleys surrounded by low-lands, steppes and semi-desert. The great diversity of terrain reflects the country's geological and geomorphological history which influences the types and occurrences of soils and aggregates throughout the country. There is therefore a wide range of materials and resources available for road construction in terms of location, type, suitability and variability.

Since much of the eastern and south-eastern areas of the country are relatively flat, most roads will tend to be constructed on shallow embankments. This emphasises the importance of appropriate longitudinal and cross drainage measures which are discussed in Chapter D.5: Drainage.

In the central and western regions of the country where the terrain is more mountainous and steep, the physical environment presents a range of challenges that require careful consideration at all stages of LVR projects, but particularly during feasibility stage of the project cycle which are considered further in Chapter D.2: Site Investigations and Route Selection.

6.9 Construction regime

The construction regime governs whether or not the road design is applied in an appropriate manner. Key elements include:

- Appropriate labour, plant and equipment use;
- Selection and placement of materials;
- Quality assurance;

- Compliance with the specification;
- Technical supervision.

A construction strategy should be adopted that is appropriate to the prevailing social, economic, cultural and other needs of Ethiopia. Such a strategy should be aimed at making maximum use of the relatively abundant resource of labour through the use of labour-based technology. This approach involves using a combination of labour and light equipment rather than heavy plant, without compromising the quality of the end product.

6.10 Maintenance regime

All roads, designed and constructed, will require regular maintenance to ensure that the design life is reached. Achieving this will depend on the maintenance strategies adopted, the timeliness of the interventions, the local capacity and available funding to carry out the necessary works. *Unless adequate maintenance is provided, the anticipated pavement design life will not be attained and, indeed, the LVR design philosophy promoted in this manual will be severely compromised.*

Some types of surfacing require considerably less maintenance-than others and should hold sway in the selection of appropriate surfacing for LVRs (See Chapter D.7: Surfacings).

6.11 Road safety regime

Some aspects of pavement structural design are affected by the road environment and can have an impact on the safety aspects of the roads. For example, the generation of dust from some wearing course gravels, the slipperiness of some earth surfaces and the skid resistance of bituminous surfacings . These factors can be addressed through a surface selection procedure as discussed in Chapter D.7: Surfacings and Part C: Complementary Interventions. However, many aspects of road safety are concerned with geometric alignment and vehicle speeds and are addressed in Chapter D.4: Geometric Design.

6.12 The "green" environment

Any road construction and on-going road use and maintenance will have an impact on Ethiopia's natural or bio-physical environment including flora, fauna, hydrology, slope stability, health and safety. These impacts have to be assessed through an Environmental Impact Assessment and mitigated as much as possible by appropriate design and construction procedures.

6.13 Environmentally optimised design (EOD)

Invariably, the road environment factors encountered along a section of road will vary. In such a situation, in order to be as cost-effective as possible, it is necessary to ensure that the use of materials and pavement designs are matched to the road environment at a local level as illustrated in Figure D.6.11 .

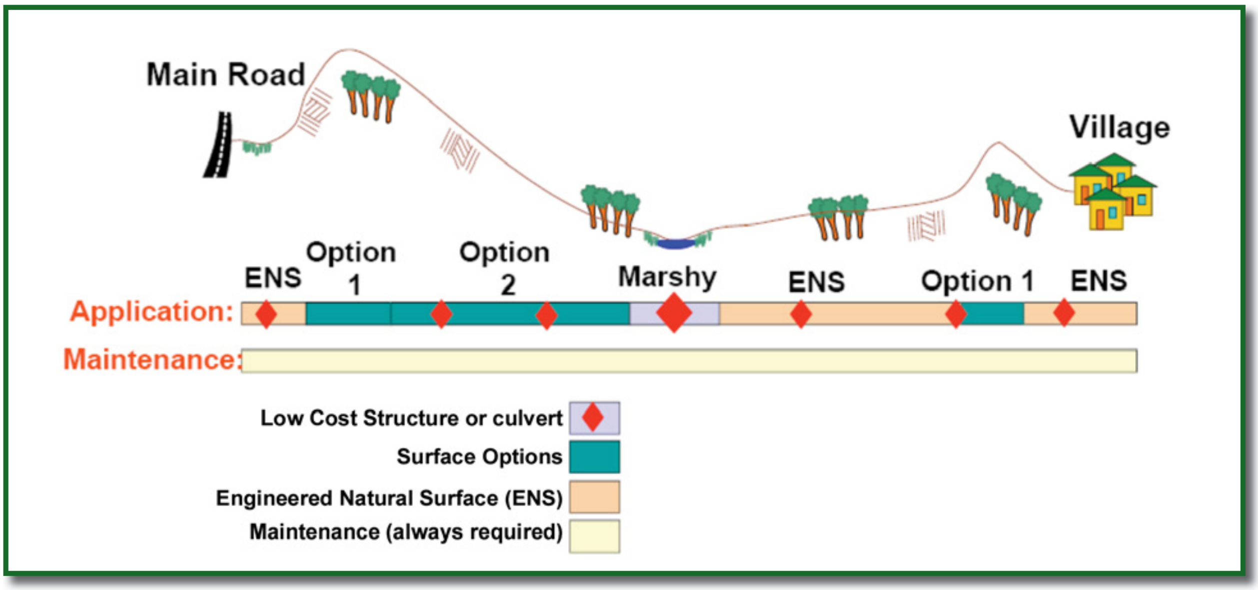

Figure D.6.11: Application of the principle of environmentally optimised design

This can be achieved by selecting different pavement options in response to different impacting factors along the road alignment ie by adopting what is called an *environmentally optimised design* (EOD) approach. This approach economises on deployed resources to achieve an acceptable level of service and results in the use of a spectrum of solutions incorporating spot improvement through to whole road link, in the process using appropriate surfaces ranging from engineered natural surfaces (ENS), gravel and, where appropriate, durable paved surfaces.

6.14 Structural design

6.14.1 Reliability and terminal condition

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

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

On the other hand roads carrying low levels of traffic can be allowed to deteriorate a little more before they are deemed to need repair. They are therefore designed with a lower safety factor. The level of reliability is set lower, typically 80% (or 50% for the lowest traffic levels). This is equivalent to designing them with a lower level of planned terminal serviceability as shown in Figure D.6.12.

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

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

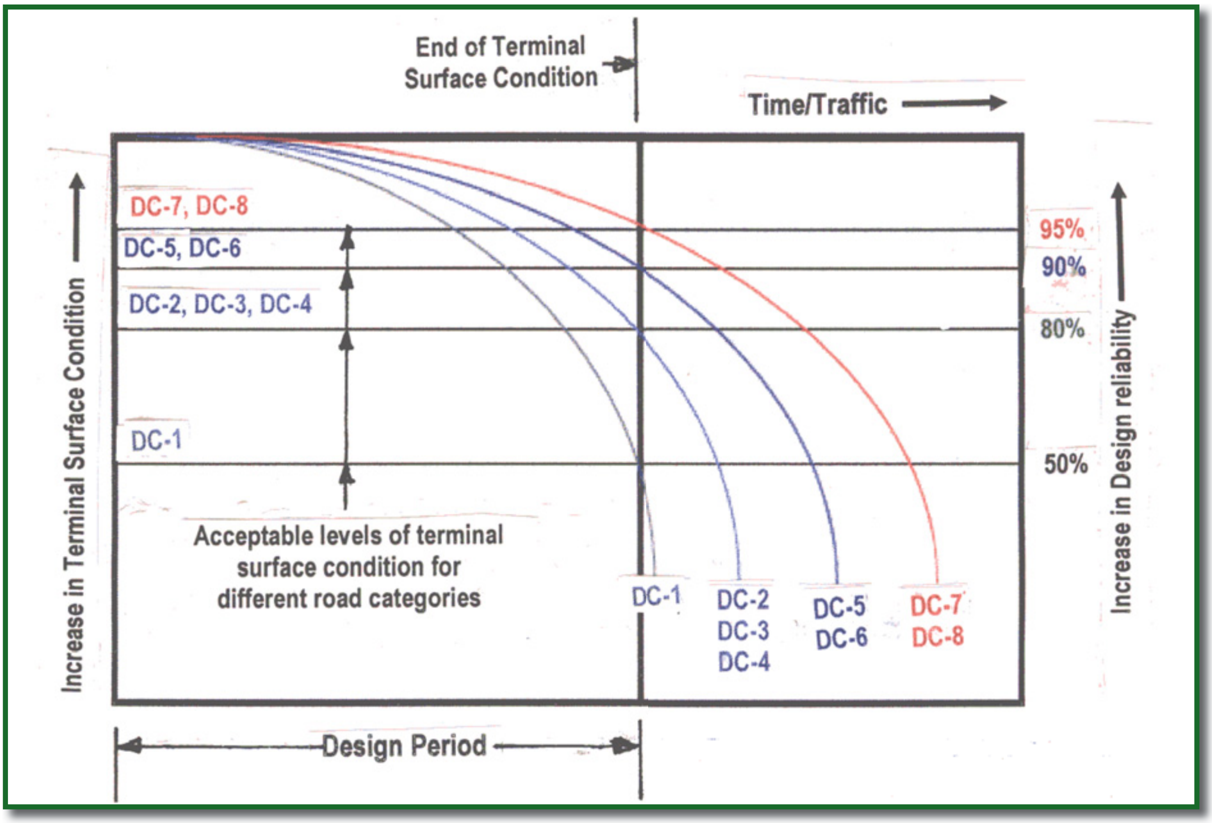

Figure D.6.12: Design reliability in relation to road category and terminal surface condition

6.14.2 Design and analysis period

The structural design period, or design life, is the period during which the road is expected to carry traffic at a satisfactory level of service, linked to a specified level of risk or design reliability (see Section D.6.14.1), without requiring major rehabilitation or repair work. It is implicit, however, that certain maintenance work will be carried out throughout this period in order to achieve the expected design life.

In a country such as Ethiopia with a low density of roads and an expanding economy, many of today's LVRs will be tomorrow's secondary or principal roads. Furthermore, LVRs are highly dependent on an adequate maintenance regime if they are to perform as desired. For these reasons it is anticipated that changes will be inevitable in a relatively short time hence the period of time for which LVRs are designed is relatively short. In this manual a period of 10 years is used for earth and gravel roads and a period of 15 years for paved roads.

6.14.3 Upgrading Strategy

The decision as to when a road should be upgraded to a higher, more expensive structural standard is often not a simple choice between a paved and an unpaved road. In practice, a spot improvement strategy should be adopted as described in Section D.6.13. Thus, over a period of time, a road will undergo a number of such improvements and will eventually consist of a mixture of structural designs. Figure D.6.13 illustrates the various types of structure that are likely to be found on such a road. When the traffic level is high enough a decision may be made to upgrade the entire road, or at least substantial sections of it, to a specific structural standard. Such a decision will depend on a consideration of whole life (or life-cycle) costs.

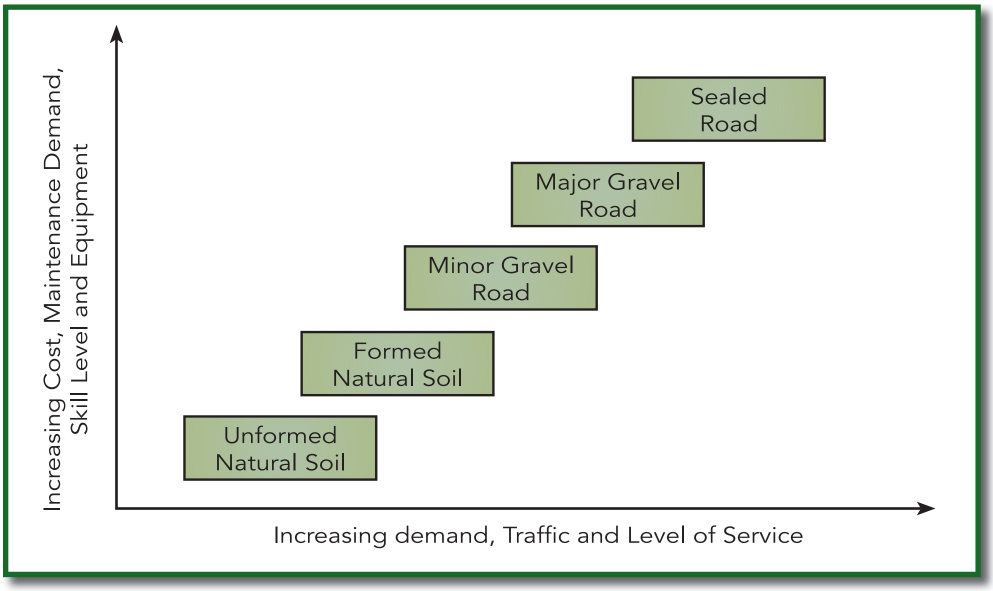

Figure D.6.13: Components of a typical EOD designed LVR

6.15 Design of earth roads

Earth roads provide the cheapest, most basic form of access to rural communities for both non-motorised and motorised traffic. Such roads are normally the first stage in the construction of a more durable road and may be either "unformed" or "formed" as generally defined below:

Unformed roads: These are "non-engineered" roads that typically consist of a track that is cleared of vegetation (see Plate D.6.4) but no significant earthworks are carried out. They are often not all weather roads and can carry only very light traffic and then only in the dry weather or where the in-situ soils are good (eg sand-clay or sand-silt-clay). Usually, minimum drainage is provided. Where poor soils are used, the roads will generally be impassable in wet weather.

Formed roads: These are "engineered" roads that typically consist of the excavated in-situ material (subgrade) in the vicinity of the alignment which is shaped to form a camber that is generally raised above existing ground level and includes side drainage (see Plate D.6.5). When constructed with adequate quality materials, provided with a proper camber (5-8%), adequately drained and properly maintained, the performance is enhanced and they will normally carry higher volumes of traffic than unformed roads. The term "engineered natural surface" (ENS) as used in this manual can also be used to describe such roads.

6.15.1 Rationale for ENS

The traffic-bearing ability of earth roads depends heavily on the type of soil forming the running surface, and on the prevailing moisture conditions. In all but arid areas, the aim at every stage of development should be to keep the road and its environs as dry as possible. When saturated, most soils are too weak to carry any significant volume of traffic. However, attention to basic construction and maintenance practice can greatly assist in extending the periods of the year during which these roads can carry traffic satisfactorily. In all situations except arid conditions, ENSs, with the additional investment in camber and drainage features, will usually have far superior performance to unformed roads.

Plate D.6.4: Typical unformed earth road

Plate D.6.5: Typical formed earth road

Experience shows that a well cambered and drained ENS will usually quickly dry out after rain so that bearing capacity is rapidly restored. This suggests that Wereda and Kebele engineered earth LVRs can be viable if communities and road users are aware of the implications and try to avoid significant trafficking when wet. They also need to be aware of the importance of a well-maintained camber and drainage.

6.15.2 Design criteria

Although ENS roads are relatively simple structures, their analysis is quite complex because they are subjected to rather severe environmental conditions and because the variability in their characteristics is also very high. Their performance depends on the same factors as for all roads but the sensitivity of performance is much greater:

- Soil properties;
- Rainfall (amount and intensity);

- Traffic (type, volume and tyre pressure);
- Longitudinal gradient, width;
- Quality of drainage;
- Level of water table.

As a consequence, there are no formal methods of design per se. Rather; the main focus of attention is on the manner of their construction and maintenance which is intrinsically linked to their performance.

6.15.3 Estimating traffic capacity

Estimating the likely performance of earth roads requires an assessment of the traffic carrying capacity of the soils under varying environmental conditions. Research undertaken on the trafficability of soils in the United States of America (Alvin and Hammitt, 1975) provide some guidance on the traffic carrying capacity of ENS roads from a knowledge of the bearing capacity (CBR) of the soil, the equivalent single wheel load of the vehicles and the tyre pressures (Figure D.6.14). Thus, if the strength of the earth road is known (in terms of its in-situ CBR), the nomograph permits predictions to be made of the load carrying ability of the road. [The definition of the terminal condition in this study was when the rut depth in the soil exceeded 75 mm].

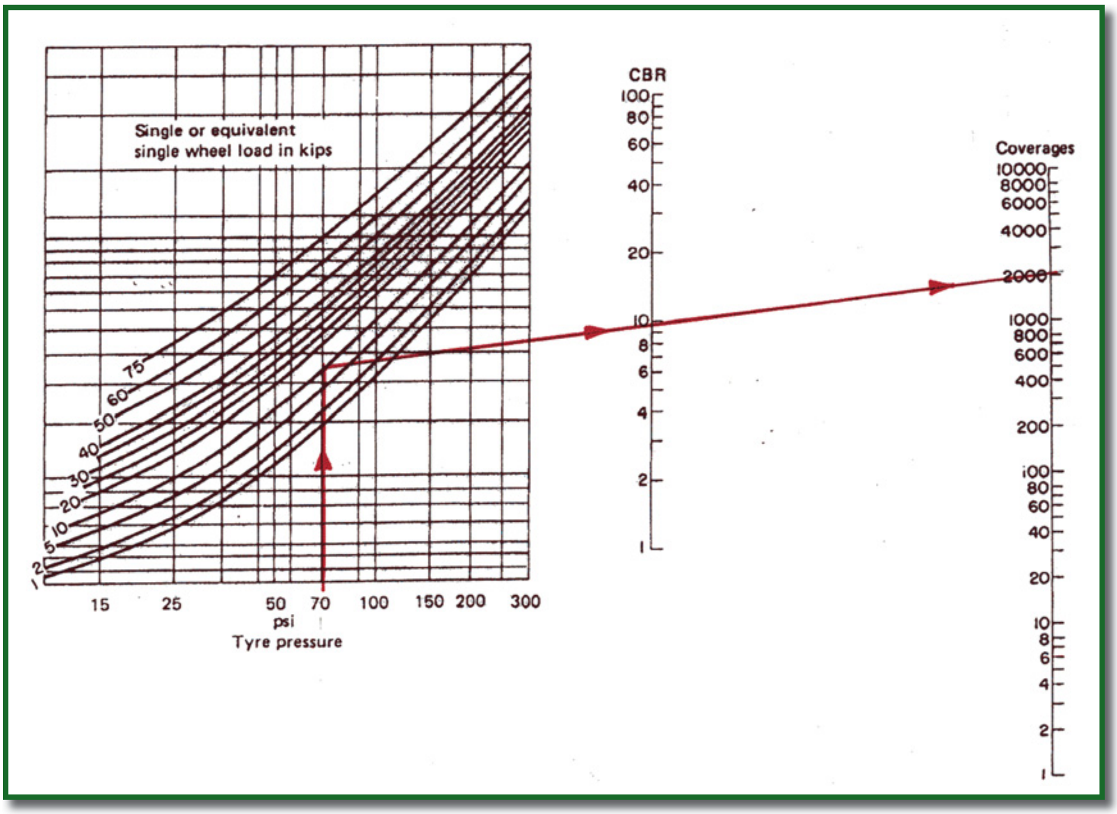

Figure D.6.14: Carrying capacity of soils

As illustrated in the nomograph (Figure D.6.14), an ENS road with an in-situ CBR of 10% can be expected to provide approximately 2,000 coverages of vehicles with a single wheel load of 10 kips (4.54 tonnes) and a tyre pressure of 70 psi (482 kPa) before serious deformation is likely to occur. Since the wheel loads will not be concentrated on exactly the same path, but will wander slightly across the width of a road, one complete coverage is equivalent to the passage of 2.7 vehicles. Thus, 2,000 coverages is equivalent to 5,400 vehicles with the characteristics indicated above.

For a single lane road, the wheel loads will be restricted to narrower channels, as described in Table D.6.7 and therefore the coverages will be different. For example, for a narrow single lane road and using Table D.6.7, the number of vehicles that the earth road can accommodate before failure decreases to approximately 1350 vehicles (5400/4). For a route carrying 50 vpd and assuming 15% of them are relatively heavy (4.54 tonne wheels), this translates into a need to maintain, re-grade or reshape the surface about every 4 to 6 months. For soils with higher CBR this will be longer. It is important for both designers and road managers to appreciate that ENS are low initial cost but that they require an ongoing commitment to regularly reshape/regrade the surface to keep it in a serviceable condition.

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

6.15.4 Construction

ENS roads are normally constructed using the in-situ materials excavated from areas adjacent to the road or from the side drains after any topsoil or vegetation matter has been removed from the ground surface. The most commonly used technique is to excavate the material to form the side drains and use this to build up the road cross section into a camber to allow rainwater to be drained off the surface into side drains, or down the embankment slopes, and away from the road without causing erosion. This can best be achieved using labour, motor or tractor towed graders. Side drains, turnout drains, cross drainage and erosion control should be provided as discussed in Chapter D.5 as for all road surface types. The ditch-to-camber earthworks technique usually provides sufficient material for an adequate cross section without the need for expensive longitudinal haul or double handling of material. For LVR the soil can be cast in one operation by labour from the ditch to the camber (see Plate D.6.6).

Plate D.6.6: Manual excavation of side ditch material to form ENS camber (prior to spreading and compaction)

Guidance on the preferred method of construction is provided in Figures D.6.15, D.6.16 and D.6.17 (Howe and Hathway, 1996). In each case the road surface is 30 cm above the level of any shallow water which may be standing alongside the road (ie on the ground in Figure D.6.15 and in the side drains in Figure D.6.16). This 30 cm "freeboard: is necessary to enable the road to dry out a few centimetres below the surface.

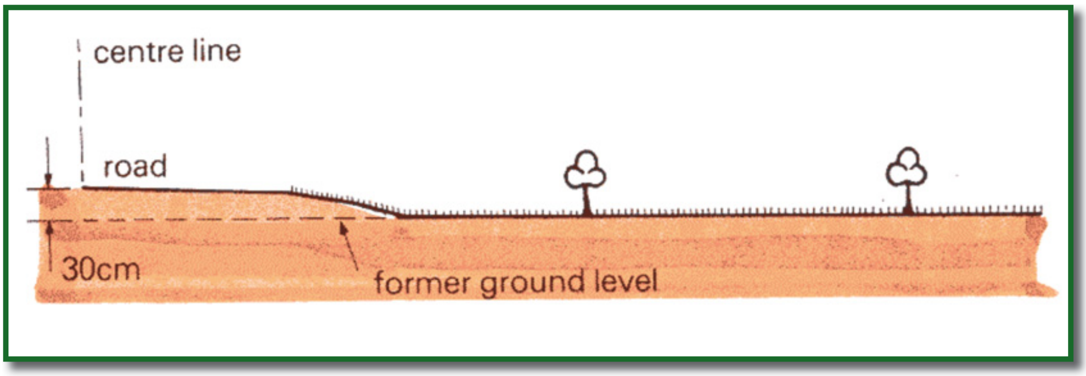

Figure D.6.15: Road built above ground level – incorrect

Figure D.6.15 shows an ENS road built completely above ground level by importing soil from a borrow site. This is expensive because of the cost of transporting the soil.

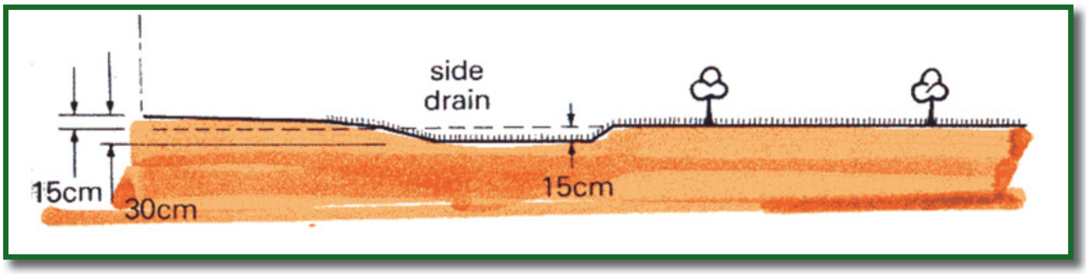

Figure D.6.16: The "high level method" - correct

Figure D.6.16 shows the "high-level" method of construction. Soil for the road is dug and thrown from the side drain until the camber is high enough. A drain 15cm deep and a compacted road 15cm high can provide the required 30cm free-board. In order to provide enough material, the width of the side drain should be increased as the width of the road itself increases. Care is required to make sure enough material is provided. Failure to do so will inevitably mean that the either the camber or crown height (freeboard) or both will be inadequate Since the drain and camber are formed in one operation, the cost of construction is minimised.

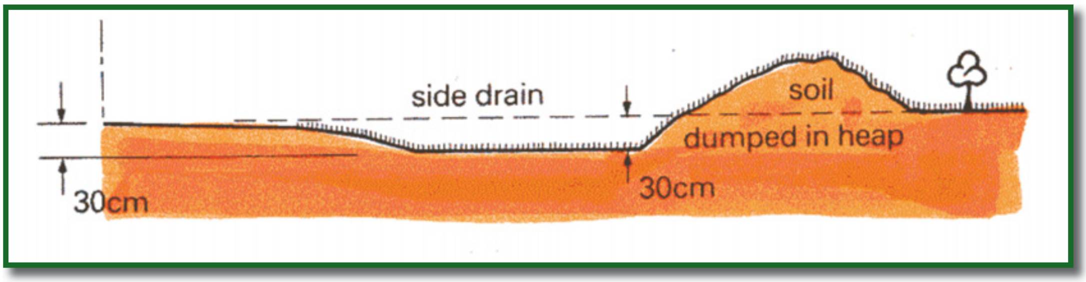

Figure D.6.17: Road built at ground level - incorrect

Figure D.6.17 shows what would happen if the road were built at ground level. Soil has to be excavated to form the camber and side drains and dumped uselessly in the bush. Because the side drains are well below ground level, it is difficult or impossible to run water from them to the surrounding ground.

Standard cross-sections for earth roads are shown in Chapter D.4: Geometric Design.

6.15.5 Maintenance

Unformed roads: Particular attention must be paid to the problem of erosion in the wheel tracks and side drains (see Figure D.6.18(a) and D.6.18(b)) for which efficient control and disposal of run-off water is critical. Where the topography allows, wide, shallow longitudinal drains are preferred. They minimise erosion, and will not block as easily as narrow ditches.

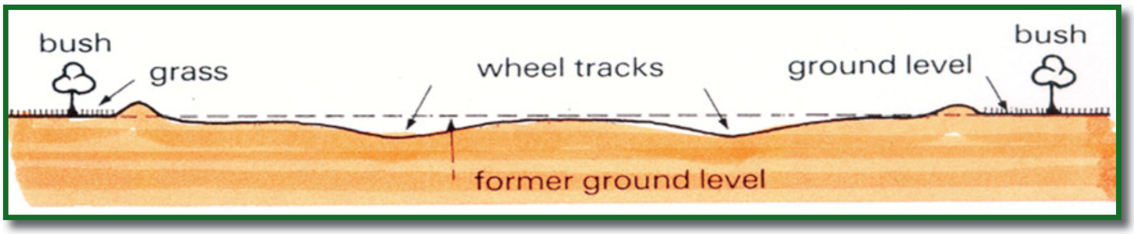

Figure D.6.18: (a) – Erosion in wheel tracks

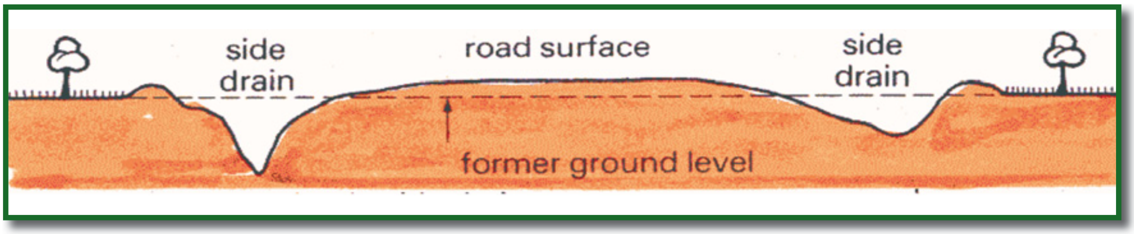

Figure D.6.18: (b) – Erosion in side drains

Any unformed road will fail, probably sooner rather than later, if avoidable erosion due to water flow is allowed to continue unabated. Each rainy season the level of the road will subside due to removal of soil. The more it subsides, the more difficult it will be to drain the water off the road. Thus, erosion damage, though imperceptible at first, is liable to increase rapidly within a few years. However, such avoidable erosion may be easily prevented by adopting a variety of soil conservation and construction techniques that are fully addressed in *Earth Roads – Their Construction and Maintenance* (Howe and Hathway, 1996).

ENS roads: Under traffic, ENS roads will become rutted and reshaping will be required. This should consist of blading soil inwards from the outside of the road edges and serves to raise the road bed, provide a cambered surface and initiate a drainage system (Figure D.6.19b). The simpler expedient of digging out the rutted soil and throwing it to the edges of the road to expose a fresh soil surface will result in a sunken road prone to water logging and being impassable to traffic in the rainy season. The establishment of grass up to the road edges assists in preventing erosion. Simple turnout drains should also be opened to discharge water collected from the road way.

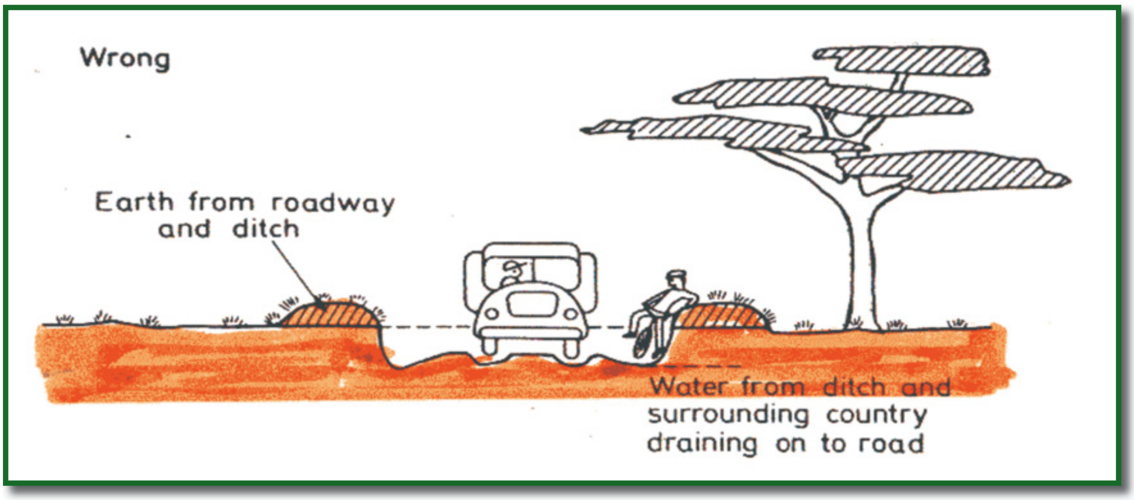

Figure D.6.19: (a) - Incorrectly maintained ENS road (Odier et al, 1971)

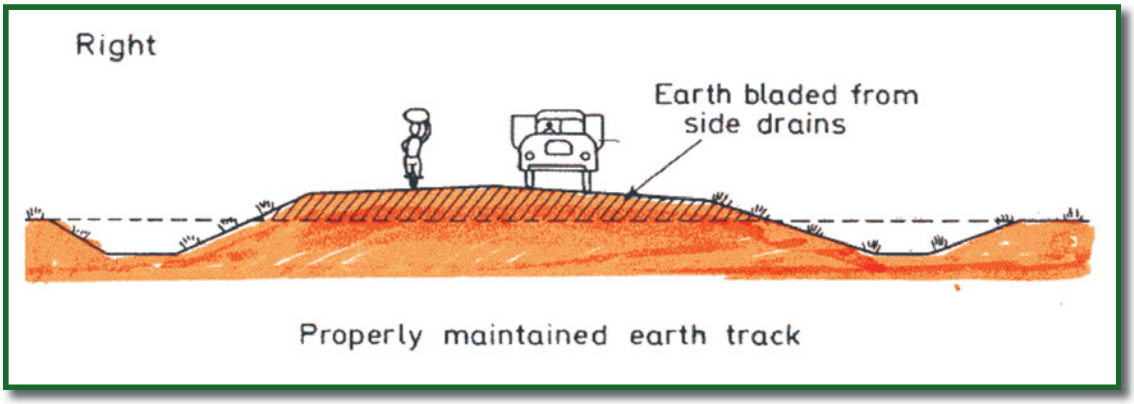

Figure D.6.19: (b) - Correctly maintained ENS road (Odier et al, 1971)

6.16 Design of gravel roads

A gravel road generally serves as the first stage in the making of an all-weather road designed to particular standards of alignment and traffic carrying capacity. The gravel surface provides an improved and more durable surface than earth.

The performance of a gravel surfaced road depends on the quality of the materials, the location of the road (terrain and rainfall), and the traffic volume using the road. Where good quality in-situ road building materials occur, they can provide a strong enough pavement structure to carry the expected traffic for many years with no additional structural layers being required, but suitable drainage must be provided and they must be properly shaped and compacted.

Generally, a gravel road consists of a wearing course and a structural layer (base) which covers the in-situ material (Figure D.6.20).

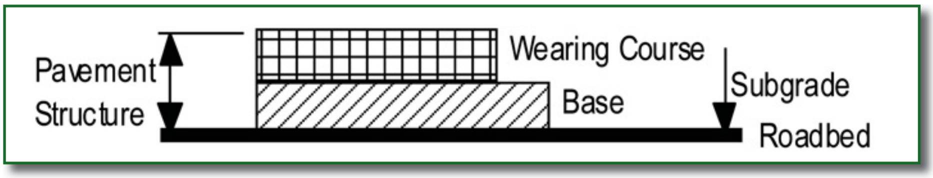

Figure D.6.20: Gravel road - Typical pavement layers

The purpose of the structural layer is to protect the sub-grade from excessive stresses imposed by traffic and thereby avoid unacceptable deformation within a specific design period. However, because the stresses on the sub-grade increase as gravel is worn away, it is necessary to ensure that a minimum thickness of the structural layer is maintained in service by providing a protective layer – the wearing course – throughout the design life of the road.

To reduce the adverse impact of rainfall and water, the road must be constructed with an appropriate camber (typically 4-6%) to effectively shed surface water. To achieve adequate external drainage, the road must also be raised above the level of existing ground such that the crown of the road is maintained at a minimum height (hmin) above the table drain inverts as shown in Figure D.6.21.

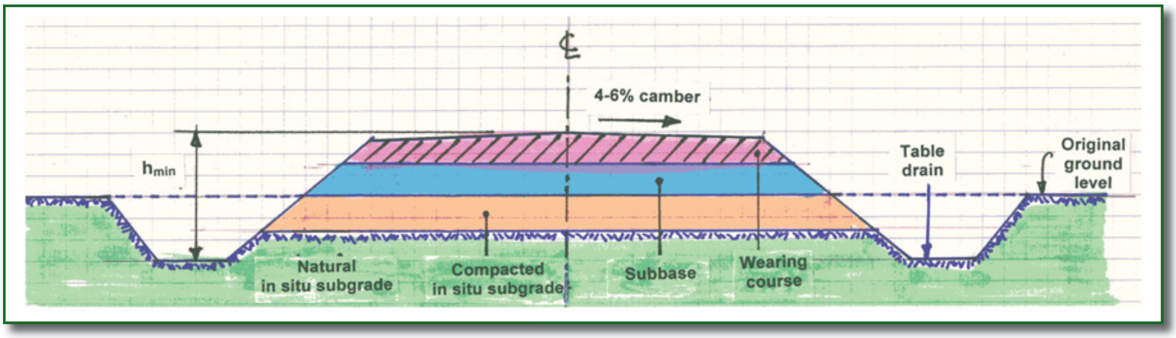

Figure D.6.21: Typical gravel road cross section in flat terrain.

The minimum height is dependent on the climate and road design class as shown in Table D.6.20. Unless the existing road is well below existing ground level, this can usually be achieved by proper "forming", ie shaping of the road bed to ensure adequate road levels, coupled with the cutting of table drains to an appropriate depth below existing ground level. Where necessary, additional fill will have to be imported or obtained from shallow cuttings to achieve the required hmin.

Table D.6.20: Required minimum height (hmin) between road crown and invert level of drain in relation to climate

Road Class	Climate	
	Wet (N < 4)	Dry (N > 4)
	h_{min} (mm)	hmin (mm)
DC-1	350	250
DC-2	400	300
DC-3	450	350
DC-4	500	400

6.16.1 Design method

Several methods have been developed internationally for designing gravel roads but most are based solely on traffic volume and do not take into consideration the load characteristics of traffic. In Ethiopia the heavy vehicle component on the higher categories of gravel roads can often be in excess of 20 per cent. For this reason the design method adopted in this manual is based on the South African TRH20 manual "Unsealed roads: Design, construction and maintenance". Ver. 1.4 (November 2008)' which takes both traffic volume and loading into account in the design process.

The design catalogue in the TRH20 manual was developed from the DCP design method (Kleyn and van Zyl, 1988) and was calibrated with Heavy Vehicle Simulator (HVS) testing carried out in South Africa. It assumes a failure criterion of 25mm deformation of the subgrade.

Gravel roads are divided into two broad categories for design purposes namely 'major' and 'minor' gravel roads:

6.16.2 Major and minor gravel roads.

A major gravel road is one that is very likely to be upgraded to a higher standard in the foreseeable future with a surfacing that does not wear away as quickly as gravel, for example, a bituminous surfacing. For major gravel roads, specific engineering features should be included at the time the gravel road is first constructed. A major gravel road will normally have a design AADT between 75 and 300 and will therefore fall into road class DC-3 or DC-4.

Ideally, the same engineered design approach can also be applied to gravel roads of classes DC-1 and DC-2 but, for these classes, referred to as "minor" gravel roads, a simplified approach is presented in Section 6.16.8.

The approach to the design of class DC-3 and DC-4 roads is as follows:

- The sub-grade should be prepared in the same way as for a low volume sealed road to comply with the requirements described in Section D.6.5.
- It is assumed that the wearing course will be replaced at intervals related to the expected annual gravel loss and before the structural layer is exposed to traffic and itself begins to wear away;
- The geometry and drainage are upgraded to acceptable minimum levels during construction. This may require the introduction of a fill layer between the compacted in-situ sub-grade and the wearing course.
- It should be noted that gravel roads in classes DC3 and DC4 are likely to incur extremely high maintenance costs in some circumstances namely;
- When the quality of the gravel is relatively poor (Section D.6.5).
- Where no sources of gravel are available within a reasonable haul distance.
- On road gradients greater than about 6%.
- In areas of high and intense rainfall.

In these circumstances spot improvements will almost certainly be justified, as outlined in Section D.6.16 and, in some cases, it may prove to be more economical to build a fully paved road at the outset.

6.16.3 The structural design procedure

1. The first step is to determine the traffic volume and traffic loading. This step is similar for all roads and is described in detail in Section D.6.6.
2. The second step is to determine the strength of the sub-grade at the appropriate moisture condition. This is slightly different for an unsurfaced road and is described below.
3. Step 3 requires measurements of the quality of the gravel that is to be used. If only very poor gravel is available, blending with another gravel or soil to improve its properties may be an option (Section D.6.7.6).
4. Step 4 requires the thickness of gravel base that is necessary to avoid excessive compressive stresses in the sub-grade to be determined. This depends on the information obtained in steps 1 to 3 but the necessary calculations have already been carried out based on research in South Africa and the results are presented in Tables D.6.21 (a), (b) and (c).
5. Finally the thickness of the wearing course needs to be calculated based on the expected rate of gravel loss and a realistic choice of the frequency of re-gravelling. Estimating the annual gravel loss is discussed in Section 6.16.7. If the annual gravel loss is expected to be GL and the road is likely to be re-gravelled every R years, the gravel loss that occurs between re-gravelling operations will be R x GL and therefore this depth of gravel needs to be provided for the wearing course.

6.16.4 Moisture regime and determination of material strength

The determination of the material strength in the in-situ sub-grade and sub-base is related to the climate, local moisture conditions and the elevation of the road above natural ground level.

In an arid, semi-arid or dry climate (the Bereha and Kolla regions of Ethiopia where the Weinert N-Value is greater than 4) and under conditions of good drainage (Table D.6.20) with the water table not near to the ground surface, both the in-situ sub-grade and sub-base layer strengths should assessed at the optimum moisture content (OMC) for compaction at the appropriate compaction level.

In contrast, in a seasonally tropical or wet climate (eg the Weina Dega, Dega and Wurch regions in Ethiopia, where the Weinert N-Value is less than 4) the in-situ sub-grade strength should be assessed in the soaked condition. The design CBR should be based on the 50th percentile (See Table D.6.3).

6.16.5 Material classification

Test pits should be excavated as a part of a conventional centre-line survey to a depth of at least 500 mm to determine the existing pavement profile. Sampling and indicator testing (Atterberg limits, grading and CBR) of selected or combined samples should be undertaken in each soil horizon. The upper 150 – 200 mm layer should be classified for the in-situ sub-grade in accordance with the sub-grade classes presented in Table D.6.2. The gravel materials should be classified in accordance with the materials classes presented in Table D.6.11.

6.16.6 Selection of appropriate pavement structure

Based on the information obtained in Steps 1 to 4, the road base thickness is selected from Tables D.6.21a, D.6.2.1b or D.6.2.1c.

Table D.6.21a: Catalogue for major gravel roads – strong gravel (G45)

Subgrade Strength Class CBR (%)	Traffic Classes (esa x 10^6)				
	<0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4)	175	225	250	300	350
S3 (5-7)	150	200	225	250	300
S4 (8-14)	100	150	200	200	250
S5 (15-29)	100	125	150	175	200

Table D.6.21b: Catalogue for major gravel roads – medium gravel (G30)

Subgrade Strength Class CBR (%)	Traffic Classes (esa x 10^6)				
	<0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4)	175	250	290	325	370
S3 (5-7)	150	200	250	275	325
S4 (8-14)	125	175	200	220	275
S5 (15-29)	100	100	150	175	200

Table D.6.21c: Catalogue for major gravel roads – weak gravel (G15)

Subgrade Strength Class CBR (%)	Traffic Classes (esa x 10^6)				
	<0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4)	225	325	375	NA	NA
S3 (5-7)	200	250	325	350	NA
S3 (5-7)	200	250	325	350	NA
S5 (15-29)	150(1)	150(1)	200(1)	200(1)	NA

Note:

1. This is the additional depth of compacted sub-grade material

The following points should be noted in connection with the selection of gravel for the structural base layer:

- The thicknesses required increases considerably if the gravel is weak hence stronger gravels should generally be used if they are available at reasonable cost.
- On relatively weak subgrades (S2 and S3), the use of strong gravels (G45) should be avoided because of the poor "balance" of such pavements. Instead, the use of an improved subgrade layer should be considered for the advantages provided (Section D.6.5.5).
- Where the available gravel is not homogeneous, it will be necessary to substitute a particular class of gravel with one or more different classes of gravel of appropriate thickness. The following conversion factors may be used for this purpose (Emery, 1985).

  G45 = 1.5 x G15
  G30 = 1.2 x G15

Thus, a 200mm layer of G45 material could be substituted with a 300mm layer of G 15 material.

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

6.16.7 Determination of wearing course thickness

The desired wearing course thickness depends on the annual gravel loss and the number of years between re-gravelling operations.

Predicted gravel loss (GL).
The interaction between traffic and rainfall contributes significantly to the loss of material from a gravel-surfaced road. Based on research work carried out in Ethiopia (TRL, 2008), standardised gravel losses (gravel loss in mm/year/100vpd) were determined in relation to the quality of the gravel wearing course as shown in Table D.6.22:

Table D.6.22: Typical standardised gravel loss

Material Quality Zone[1]	Description of Material Quality	Typical gravel loss (mm/yr/100vpd)
Zone A	Satisfactory	20
Zone B	Poor	45
Zone C	Poor	45
Zone D	Marginal	30
Zone E	Good	10

Note:

1. See Figure D.6.9.

These rates of gravel loss increase quite significantly on gradients greater than about 6% and also in areas of high and intense rainfall. On gradients, the increase could be greater than 50% depending on the steepness of the gradient and material quality and therefore spot improvements should be considered.

The rates of gravel loss given here can be used as an aid to the planning for re-gravelling in the future. A more accurate indication of gravel loss for a particular section of road can be obtained from periodic measurement of the gravel layer thickness.

Re-gravelling frequency

Re-gravelling should take place before the sub-base is exposed in order to avoid significant deformation which will necessitate reconstruction and loss of the strength that has been built up in the sub-grade by traffic moulding over time. Where the initial gravel road has been properly designed and constructed with appropriate quality materials, and also has adequate drainage, the re-gravelling frequency, R, will be approximately as assumed (see Section 6.16.3) which is typically in the range 5 – 8 years. This can decrease considerably if poor quality gravels have to be used. For example, if the gravel loss rate is 45mm per year per 100vpd, a class DC4 gravel road carrying 200vpd will lose 90mm per year and require re-gravelling every two years at the most. It would be surprising if an economic analysis did not show that such a road should be fully paved.

Wearing course thickness

This is determined from the product of the annual gravel loss, GL, and the re-gravelling frequency, R, as discussed above.

6.16.8 Minor gravel roads

A minor gravel road is one which is unlikely, in the foreseeable future, to be upgraded to a bituminous standard. This applies to roads which have a design AADT typically less than 50 and will fall into classes DC-1 or DC-2. Where budgetary constraints or other reasons do not allow the construction of these roads to an engineered gravel standard as described above, lesser gravel sub-base and wearing course thicknesses may be used. However, a relatively lower level of service should be expected, coupled with a greater risk of shear failures in the sub-grade.

The approach to the design of these categories of roads is as follows:

- The catalogue is based on the AADT of the road and assumes approximately 30% commercial vehicles;
- The subgrade materials should not necessarily comply with the requirements of a low volume sealed road;
- A nominal wearing course thickness of 150mm of G15 is assumed for all road classes and sub-grade conditions with the sub-base thickness being influenced by the sub-grade class;
- Drainage, but not necessarily geometry, is upgraded to acceptable minimum levels during construction. As for Class DC-3 and DC-4 roads, this can be achieved by building up the formation to an appropriate height to achieve the hmin requirements given in Table D.6.20.

Based on the above approach, the recommended sub-base thicknesses and wearing course material strengths for different sub-grade and traffic conditions are presented in the design chart in Table D.6.23.

Table D.6.23: Typical standardised gravel loss

Subgrade Strength Class CBR (%)	Traffic Classes (AADT) DC1/DC2[1] (< 75)
S2 (3-4)	150 WC 200 G15[2]
S3 & S4 (5-14)	150 WC
S5 (15-29)	Earth Road

Notes:

1. If more than 10 heavy vehicles per day, design as a major gravel road
2. If a G30 material is available the thickness can be reduced to 150 mm

6.17 Structural design of paved roads

The structure of a paved road consists typically of one or more layers of material with different strength characteristics (Figure D.6.22), each layer serving the purpose of distributing the load it receives at the top over a wider area at the bottom. The layers in the upper part of the structure are subjected to higher stress levels than those lower down and therefore need to be constructed from stronger material. The surfacing may be either structural or non-structural in terms of its contribution to the overall strength of the road pavement.

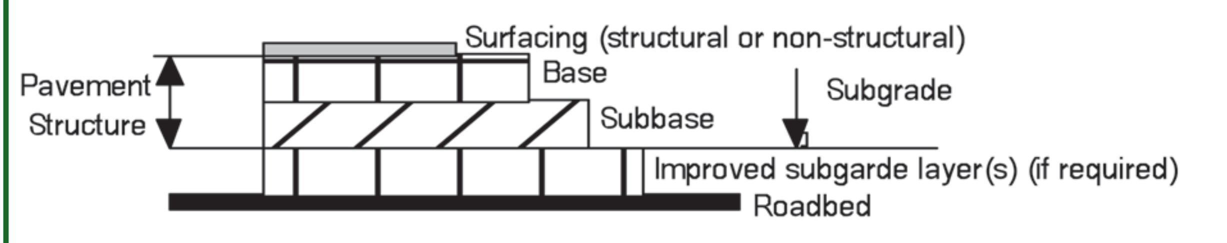

Figure D.6.22: Paved road - Typical pavement layers

6.17.1 Design methods

There are a number of methods that have been developed for the design of flexible paved roads ranging from the simple to the complex and based on both mechanistic/analytical and empirical methods. The purely empirical design methods are limited in their application to conditions similar to those for which they were developed whilst the mechanistic/analytical methods require a considerable amount of material testing and computational effort and their application to highly variable, naturally occurring materials which make up the bulk of LVR pavements is questionable.

The pavement design method used here is an empirically-based design method.

The DCP method is useful where a basic or more developed pavement structure is already in place and needs to be enhanced or upgraded.

6.17.2 Design method for Bituminous surfaced roads

Design charts or catalogue methods are the easiest to use because all the practical and theoretical works have been carried out and different structures are presented in chart form for various combinations of traffic, environmental effects, pavement materials and design options.

The design method presented in this manual is based on research undertaken in a number of countries in southern Africa (TRL, 1999). It differs from the traditionally accepted design criteria applied to the design of heavily trafficked roads in that it recognises the controlling influence of the road environment on the deterioration of lighter pavement structures. By incorporating a recognised climatic variable, the N-value (Section D.6.3.3), the geographical transferability of the research findings can be undertaken with confidence in Ethiopia.

The LVR design process for bituminous surfaced roads is outlined in the flow chart presented in Figure D.6.23. This process indicates the sequence of steps that are required to produce a pavement design that is appropriate and adequate for an individual road.

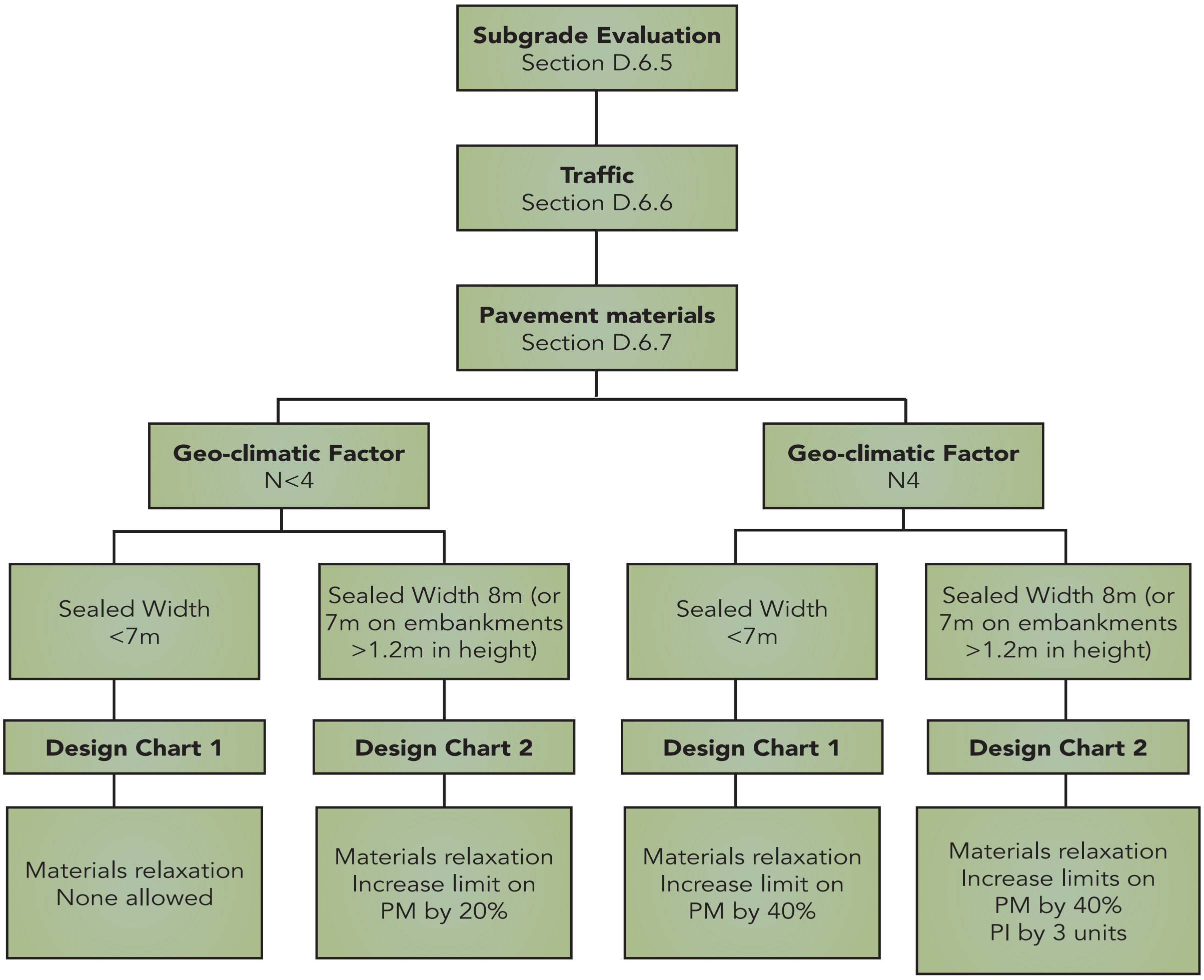

Figure D.6.23: Flow chart for bituminous surfaced road pavement design process

Climate: The design method utilises two design charts each applicable to a different climatic zone characterised by the Weinert N-values (Section D.6.3.3) and the shoulder and drainage design adopted.

An N-value map for Ethiopia is shown in Section 6.3.3 and Figure D.6.5 and provides the means of placing the road in the appropriate climatic zone for design purposes. The two design charts are presented in Part B: Design Standards, and offer a total of sixty different pavement structures depending on traffic and subgrade class.

N-values less than 4 imply a climate that is seasonally tropical and wet (the Weina Dega, Dega and Wurch regions in Ethiopia), whereas N-values of greater than 4 imply a climate that is arid, semi-arid or dry (the Bereha and Kolla regions of Ethiopia).

Traffic and environmental effects: For a correctly constructed pavement carrying low levels of traffic, there is a low risk of a pavement failure being induced by traffic, and deterioration is controlled mainly by environmental factors. However, as the traffic levels increase, the specification for road bases should approach those of traffic design charts for high volume roads presented in ORN 31 (TRL, 1997). Experience suggests that the transition from low-volume to high-volume roads is typically in the 1.0 Mesa range (see Figure D.6.24).

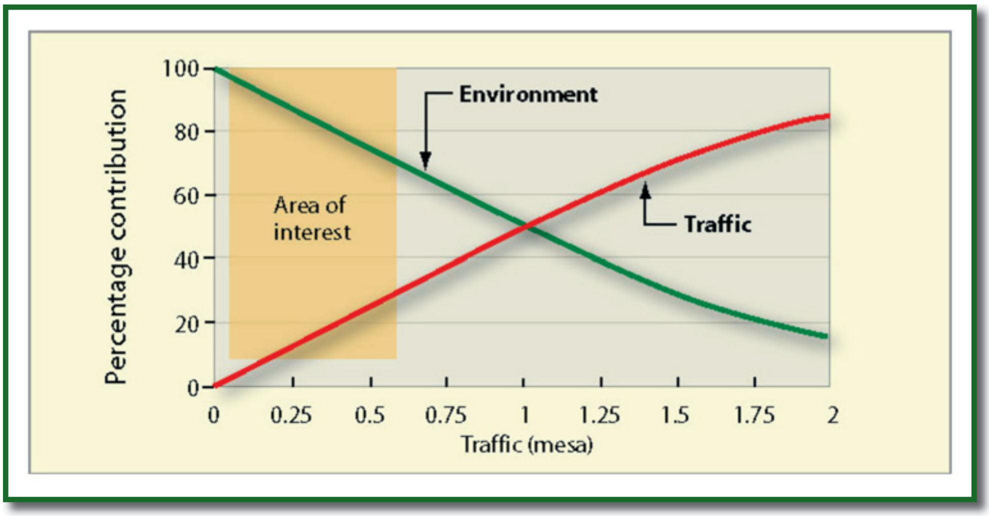

Figure D.6.24: Traffic loading versus dominant mechanism of pavement distress (Schematic only)

Sealed width: When the total sealed width is 7 metres or less, the outer wheel-track is within one metre of the edge of the seal. This affects pavement performance adversely because of seasonal moisture ingress. Therefore, relatively stronger pavements are necessary in these situations. If the road width is sufficient for the outer wheel to be more than 1.5 metres from the pavement edge, and good drainage is ensured by maintaining the crown height at least 750mm above the ditch invert, an improvement in performance occurs.

This is reflected in the catalogues where different sealed surface widths are treated separately. Thus a wider sealed cross-section in climatic zones where N<4 (a relatively wet environment) allows a shift from Catalogue 1 (N<4) to Catalogue 2 (N>4). This allows the use of thinner pavement layers and a relaxation of the quality requirements for the base.

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

6.17.3 Use of the design charts

For guidance, the following design options are used in the catalogues related to the design traffic class shown in Table D.6.24:

Table D.6.24: Design traffic classes

Design traffic classes					
Traffic Classes (ESA x 10^6)	<0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.

Climatic zones N < 4

(a) Where the total sealed surface is 8 m or less, use Pavement Design Chart 1 (Table B.6.6). No road base materials adjustments are allowed.

(b) Where the total sealed surface is 8 m or more, use Pavement Design Chart 2 (Table B.6.7). The limit on the plasticity modulus of the road base may be increased by 20 per cent.
(c) Where the total sealed surface is less than 8m but the pavement is on an embankment in excess of 1.2 m in height, use Pavement Design Chart 2 (Table B.6.7). The limit on the plasticity modulus of the road base may be increased by 20 per cent.
(d) If the engineer deems that other risk factors (eg poor maintenance and/or construction quality) are too high, then Pavement Design Chart 1 should be used.

Climatic zones N > 4
Use Pavement Design Chart 2 (Table B.6.7).
(a) Where the total sealed surface is less than 8 metres, the limit on the plasticity modulus of the road base may be increased by 40%.
(b) Where the total sealed surface is over 8 metres and when the pavement is on an embankment in excess of 1.2 metres in height, the plasticity modulus of the road base may be increased by up to 40% and the plasticity index by 3 units.

The design flow chart in Figure D.6.23 should be used iteratively depending on conditions on the individual project as in the following example:

Once the quality of the available materials and haul distances are known, the flow chart and the design charts can be used to review the most economical cross-section and pavement; this involves assessment of design traffic class, design period, cross-section and other environmental and design considerations. It may be more economical to use a wider cross-section in the seasonal tropical and wet climate zone, and then shift to Design Chart 2 than to design a narrow cross-section and a pavement using Design Chart 1, however the minimum width of carriageway and shoulders is controlled by the geometric standards adopted and this depends on traffic volume and composition.

Reducing Risks in Special Cases: When the project is located close to the border between the two climatic zones, the lower N-value should be used to reduce risks. When close to the borderline between two traffic design classes, and in the absence of more eliable data, the next highest design class should be used.

6.17.4 Design method for non-bituminous surfaced road

The design charts for non bituminous surfaced roads are given in Part B, Section 6.4.2, and as with the design charts for bituminous surfaced roads, are empirically-based.

6.18 Drainage and Shoulders

Moisture is the single most important factor affecting pavement performance and long-term maintenance costs. Thus one of the significant challenges faced by the designer is to provide a pavement structure in which the detrimental effects of moisture are contained to acceptable limits in relation to the traffic loading, nature of the materials being used, construction/maintenance provisions and degree of acceptable risk. This challenge is accentuated by the fact that most low volume roads will be constructed from natural, often unprocessed, materials which tend to be moisture sensitive. This places extra emphasis on drainage and moisture control for achieving satisfactory pavement life.

Two inter-related aspects of drainage need to be considered during road design, namely *internal and external* drainage. This section focuses on internal drainage only which is concerned with water that enters the road structure directly from above the road pavement or directly from below and the measures that can be adopted to avoid trapping water within the pavement structure. External drainage which seeks to control water before it enters the pavement structure is discussed in Chapter D.5: Drainage.

6.18.1 Sources of Moisture Entry into a Pavement

The various causes of water ingress to, and egress from, a pavement are listed in Table D.6.25 and discussed in this Section.

Table D.6.25: Typical causes of water ingress to, and egress from a road pavement

Means of Water Ingress	Causes
Through the pavement surface	through cracks due to pavement failure
	penetration through intact layers
From the subgrade	artesian head in the subgrade
	pumping action at formation level
	capillary action in the subbase
From the road margins	seepage from higher ground, particularly in cuttings
	reverse falls at formation level
	lateral/median drain surcharging
	capillary action in the subbase
	through an unsealed shoulder collecting pavement and ground run-off
Through hydrogenesis (aerial well effect)	condensation and collection of water from vapour phase onto underside of an impermeable surface
Means of Water Egress	**Causes**
Through the pavement surface	through cracks under pumping action through the intact surfacing
Into the subgrade	soakaway action
	subgrade suction
To the road margins	into lateral/median drains under gravitational flow in the subbase
	into positive drains through cross-drains acting as collectors

6.18.2 Permeability

Permeability is a measure of the ease with which water passes through a material and is one of the key material parameters affecting drainage. Moisture ingress to, or egress from, a pavement will be influenced by the permeability of the pavement, subgrade and surrounding materials. The relative permeability of adjacent materials may also govern moisture conditions. A significant decrease in permeability with depth or across boundaries between materials (ie permeability inversion) can lead to saturation of the materials in the vicinity of the inversion. Typical permeability values for saturated soils are presented in Table D.6.26.

Table D.6.26: Typical material permeabilities (Lay, 1998)

Material	Permeability	Description
Gap-graded crushed rock	> 30 mm/s	Free draining
Gravel	> 10 mm/s	
Coarse sand	> 1 mm/s	
Medium sand	1 mm/s	Permeable
Fine sand	10 μm/s	
Sandy loam	1 μm/s	Practically impermeable
Silt	100 nm/s	
Clay	10 nm/s	Impermeable
Bituminous surfacing[(1)]	1 nm/s	

Note:

(1) Applies to well-maintained double chip seal. Thicker asphalt layers can exhibit significant permeability as a result of a linking of air voids. Permeability increases as the void content of the mix increases, with typical values ranging from 300 μm/s at 2% air voids to 30 μm/s at 12% air voids. Typically, a 1% increase in air voids content will result in a three-fold increase in permeability (Waters, 1982).

6.18.3 Achieving effective internal drainage

The following guidance is provided for achieving effective internal drainage of the road structure.

Side drainage and crown height above drain invert. Side drainage is one of the most significant factors affecting pavement performance. The "drainage factor" is the product of the height of the crown of the road above the bottom of the ditch (h) and the horizontal distance from the centreline of the road to the bottom of the ditch (d) and can be used to classify the type of drainage prevailing at the road site. This classification of road drainage is shown in Table D.6.27.

Implied in the Table is the critical nature of the crown height which correlates well with the actual service life of pavements constructed from natural gravels. A minimum value, h, of 0.75m is recommended as illustrated in Plate D.6.7.

Table D.6.27: Classification of road drainage

Drainage Factor DF = d x h	Classification
<2.5	Very poor
2.6 – 5.0	Poor
5.1 – 7.5	Moderate
> 7.5 or free draining	Good

Note:

1. Classification can move up one class if longitudinal gradient >1%

Irrespective of climatic region, if the site has effective side drains and adequate crown height, then the in-situ subgrade strength stays above the design value. If the drainage is poor, the in-situ strengths will fall to below the design value.

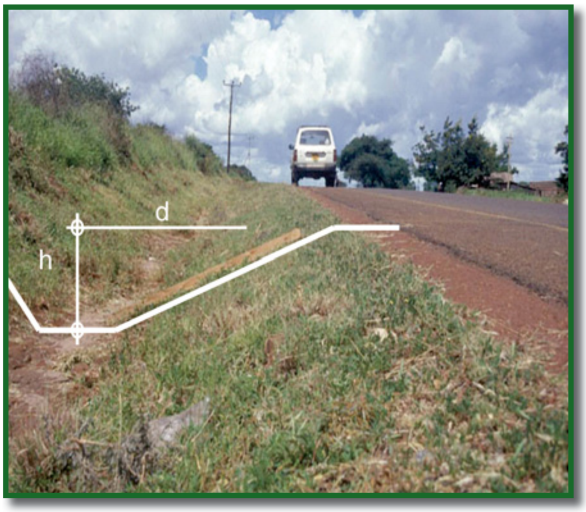

Plate D.6.7: Example of a well-drained pavement where the drainage is classified as "good", ie the drainage factor DF (d x h) >7.5

Drainage within pavement layers: Drainage within the pavement layers themselves is an essential element of structural design because the strength of the subgrade in service depends critically on the moisture content during the most likely adverse conditions. Since it is impossible to guarantee that road surfaces will remain waterproof throughout their lives, it is critical to ensure that water is able to drain away quickly from within the pavement. This can be achieved by a number of measures as follows:

Avoiding permeability inversion: A permeability inversion exists when the permeability of the pavement and subgrade layers decreases with depth. Under infiltration of rainwater, there is potential for moisture accumulation at the interface of the layers. The creation of a perched water table could lead to shoulder saturation and rapid lateral wetting under the seal may occur. This may lead to base or sub-base saturation in the outer wheeltrack and result in catastrophic failure of the base layer when trafficked. A permeability inversion often occurs at the interface between sub-base and subgrade since many subgrades are cohesive fine-grained materials. Under these circumstances, a more conservative design approach is required that specifically caters for these conditions.

In view of the above, it is desirable for good internal drainage that permeability inversion does not occur. This is achieved by ensuring that the permeability of the pavement and subgrade layers are at least equal or are increasing with depth. For example, the permeability of the base must be less than or equal to the permeability of the sub-base in a three layered system.

Where permeability inversion is unavoidable, the road shoulder should be sealed to an appropriate width to ensure that the lateral wetting front does not extend under the outer wheeltrack of the pavement.

Ensuring proper shoulder design: When permeable roadbase materials are used, particular attention must be given to the drainage of this layer. Ideally, the roadbase and subbase should extend right across the shoulders to the drainage ditches. In addition, proper crossfall is needed to assist the shedding of water into the side drains. A suitable value for paved roads is about 2.5 to 3% for the carriageway, with a slope of about 4-6% for the shoulders. Increased crossfalls, typically about 2-3% more, are required for unsurfaced roads.

Lateral drainage can also be encouraged by constructing the pavement layers with an exaggerated crossfall, especially where a permeability inversion occurs. This can be achieved by constructing the top of the sub-base with a crossfall of 3-4% and the top of the subgrade with a crossfall of 4-5%. Although this is not an efficient way to drain the pavement it is inexpensive and therefore worthwhile, particularly as full under pavement drainage is rarely likely to be economically justified for LVRs. Figure D.6.25 illustrates the recommended drainage arrangements for a LVR.

If it is too costly to extend the roadbase and subbase material across the shoulder, drainage channels at 3m to 5m intervals should be cut through the shoulder to a depth of 50mm below subbase level. These channels should be back-filled with material of roadbase quality but which is more permeable than the roadbase itself, and should be given a fall of 1 in 10 to the side ditch. Alternatively, a preferable option would be to provide a continuous layer of pervious material of 75mm to 100mm thickness laid under the shoulder such that the bottom of the drainage layer is at the level of the top of the subbase.

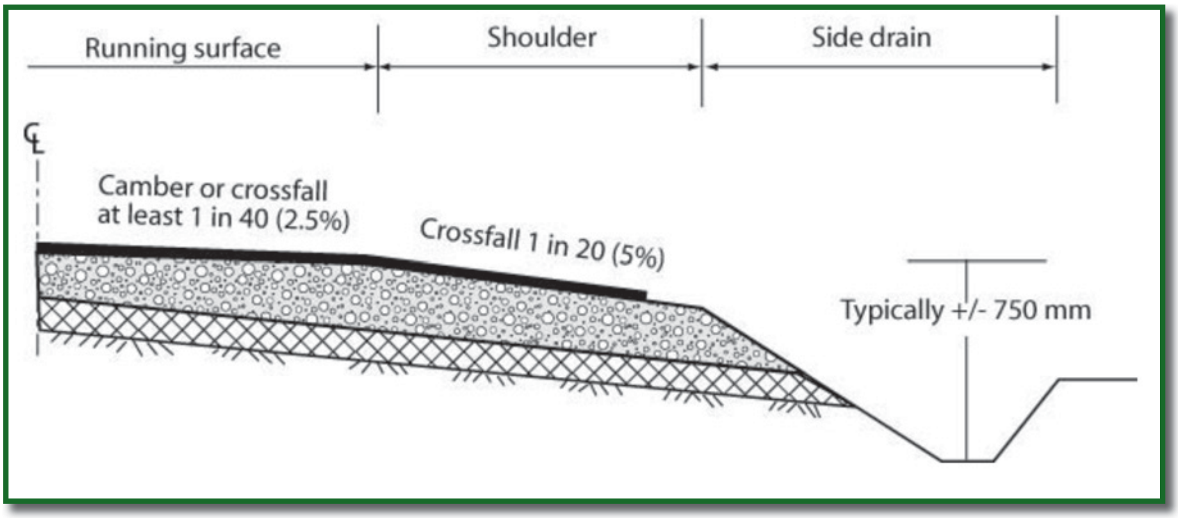

Figure D.6.25: Recommended drainage arrangements

Sealing of shoulders: It is now generally recommended that, wherever possible, shoulders should be sealed, for the following reasons:

- They provide better support and moisture protection for the pavement layers and also reduces erosion of the shoulders (especially on steep gradients);
- They improve pavement performance by ensuring that the zone of seasonal moisture variation does not penetrate to under the outer wheel track (see Figure D.6.26);
- They reduce maintenance costs by avoiding the need for regravelling at regular intervals;
- They reduce the risk of road accidents, especially where the edge drop between the shoulder and the pavement is significant or the shoulders are relatively soft.

For the above reasons, it is generally the case that if it is economically justifiable to pave a road then it is very likely that it will also be economically justifiable to provide paved rather than unpaved shoulders. This should be undertaken as part of the design of the pavement cross-section.

Unsealed shoulders: A common problem associated with the use of unsealed shoulders is water infiltration into the base and subbase for a number of reasons, which are illustrated in Figure D.6.26 and include:

- Rutting adjacent to the sealed surface;
- Build-up of deposits of grass and debris;
- Poor joint between the base and shoulder (more common when a paved shoulder has been added after initial construction).

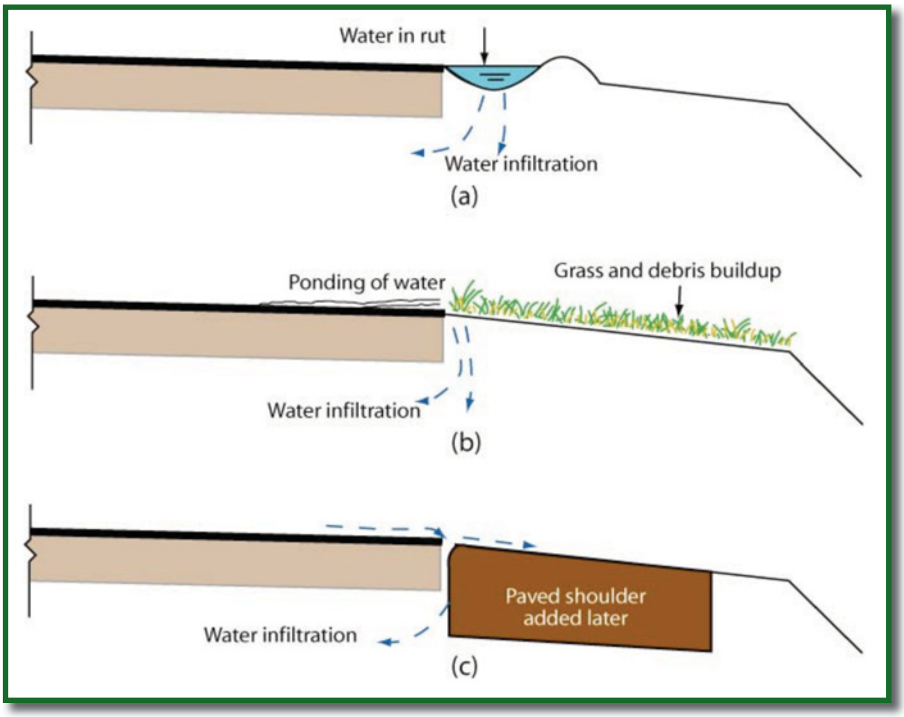

Figure D.6.26: Typical drainage deficiencies associated with pavement shoulder construction (adapted from Birgisson and Ruth, 2003)

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

Plate D.6.8: Infiltration of water through a permeable surfacing and subsequent outflow to an impermeable shoulder

This "boxed" construction is a common cause of road failure due to the reduction in strength and stiffness of the pavement material and the subgrade below that required to sustain the traffic loading.

Adopting an appropriate pavement cross-section: In terms of pavement cross-section, the two moisture zones in the pavement which are of critical significance are the equilibrium zone and the zone of seasonal moisture variation (see Figure D.6.27: Right with a sealed shoulder; left with an unsealed shoulder).

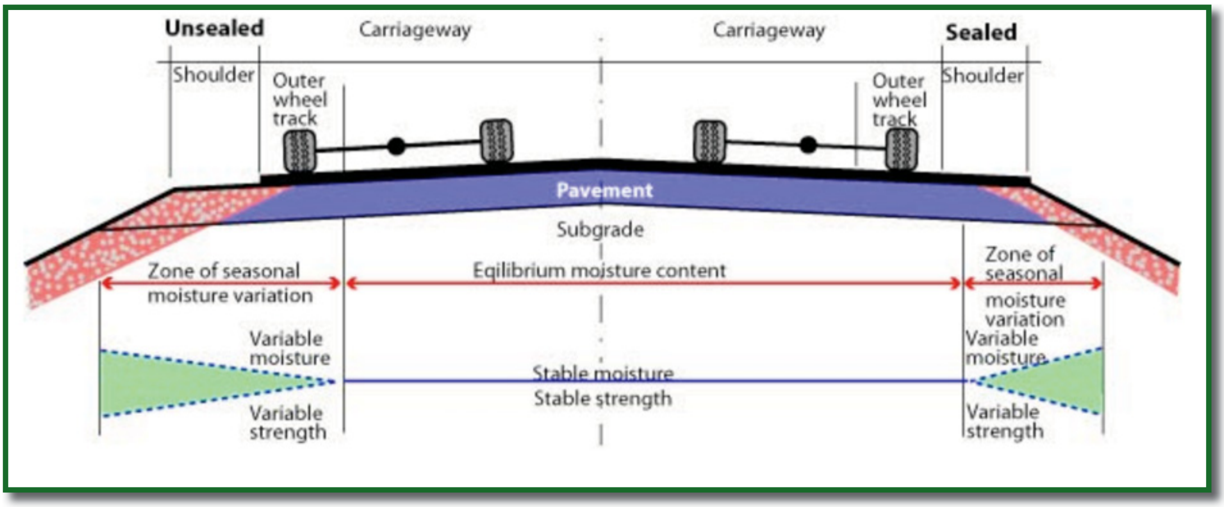

Figure D.6.27: Moisture zones in a typical LVR

From extensive research work carried out in a number of tropical regions of the world (eg O'Reilly, 1968; Morris and Gray, 1976; Gourley and Greening, 1999), it has been found that:

- In sealed pavements over a deep water table, moisture contents in the equilibrium zone normally reach an equilibrium value after about two years from construction and remain sensibly constant thereafter.
- In the zone of seasonal variation, the pavement moisture does not reach an equilibrium and fluctuates with variation in rainfall. Generally, this zone is wetter than the equilibrium zone in the rainy season and it is drier in the dry season. Thus, the edge of the pavement is of extreme importance to ultimate pavement performance, with or without paved shoulders, and is the most failure-prone region of a pavement when moisture conditions are relatively severe.

In order to ensure that the moisture and strength conditions under the outer wheel track will remain fairly stable and largely independent of seasonal variations, the shoulders should be sealed to a width of between about 1.0 and 1.2 m from the edge of the sealed area (Figure D.6.28).

Adopting a holistic and integrated approach: The drainage measures highlighted above are all aimed at:

- Preventing water from entering the pavement in the first place;
- Facilitating its outflow as quickly as is reasonable, given the cost implications;
- Ensuring that the presence of water in the road for an extended period of time does not cause failures.

It should be appreciated, however, that the adoption of any single measure on its own is unlikely to be as effective as the adoption of a judicious mixture of a number of complementary measures applied simultaneously. Such an approach forms part of the philosophy of minimising the risks associated with using locally occurring natural materials in the pavements of LVRs.

6.19 Problem Soils

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

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

This section focuses on typical measures that may be considered when dealing with problem soils that occur in subgrades along the alignments of LVRs. The investigation and testing of such soils to determine their engineering properties are not dealt with in this section but, rather, in the Site Investigation and Route Selection chapter in Section D.2.6.7 and in the Site Investigation Manual - 2011.

6.19.1 Performance risk

In assessing the appropriateness of the measures available for dealing with problem soils, a careful balance has to be struck between the cost of the measures and the benefits to be derived. This would require that a life-cycle analysis be carried out to determine whether the costs of the measures would be at least off-set by the benefits. Bearing in mind the relatively small user benefits generated by LVRs when compared with higher trafficked roads, it is unlikely that the more extensive and costly measures would be justified. This is particularly the case for unpaved roads where the consequences of rectifying the problem as and when it arises are likely to be relatively small in comparison to the initial cost of implementing the measures.

6.19.2 Expansive soils

Expansive soils are those which exhibit particularly large volumetric changes (swell and shrinkage) following variations in moisture contents. The mechanism of expansion illustrated in Figure D.6.28 is that of seasonal wetting and drying, with consequent movement of the water table. Soils at the edge of the road wet up and dry out at a different rate than do those under a paved surface, thus bringing about differential movement. It is this movement rather than the low soil strength (most expansive soils are often relatively strong at their equilibrium moisture content) which brings about failure. Such failure typically takes the form of associated longitudinal crack development, occurring first in the shoulder area and developing subsequently in the carriageway, as well as general unevenness of the pavement surface, arcuate cracking and settlement near trees and transverse humps and cracks at culvert sites (Plates D6.9 and D.6.10).

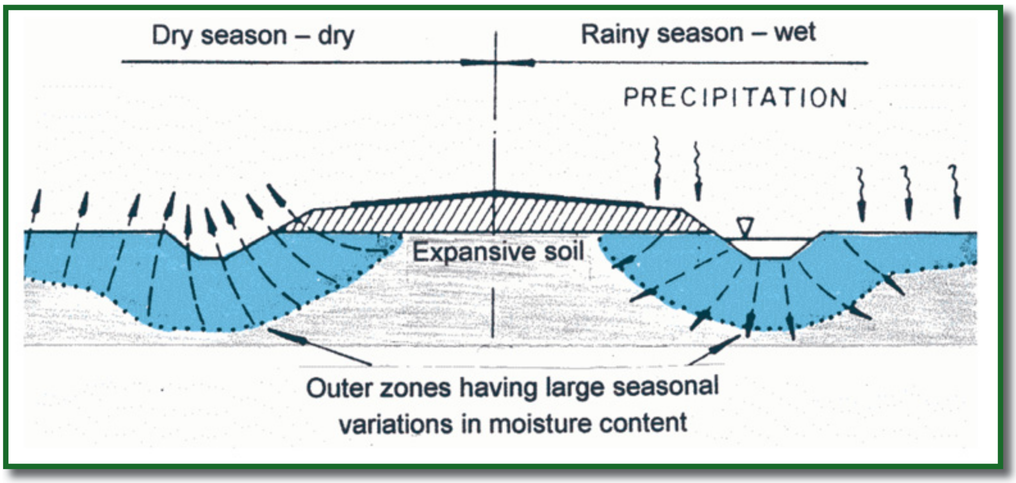

Figure D.6.28: Moisture movements in expansive soils under a paved road

Generally, all of the following conditions must be satisfied before significant movement can take place:

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

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

Plate D.6.9: Expansive "black cotton" soil exhibiting wide-spaced shrinkage cracks

Plate D.6.10: Typical longitudinal cracking and pavement deformation caused by an expansive soil subgrade

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

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

Many of the above countermeasures would be prohibitively expensive and difficult to economically justify for application to LVRs. Nonetheless, there are a number of relatively low-cost measures that can be adopted based on practical experience in a number of African countries. The LVR countermeasures shown in Table D.6.28 provide a guide for the selection of appropriate countermeasures that could be considered for LVRs based on the degree of expansiveness, as defined in Chapter D.2 – Site Investigations and Route Selection. The final choice should be based on a life-cycle cost analysis of the options presented.

Table D.6.28: Countermeasures for dealing with expansive soils

Expansiveness of soil	Alternative design and construction measures over expansive soils	
	Design Traffic < 100,000 esa	Design Traffic > 100,000 esa
Low	Countermeasure A	Countermeasure A
Medium	Countermeasure A	Countermeasure A
High	Countermeasure A	Countermeasure B
Very high	Countermeasure B	Countermeasure C1 or C2

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

- Remove vegetation during the dry season as long as possible in advance of construction.
- Construct any cuttings necessary, however shallow.
- Undertake construction when the in-situ material is at equilibrium moisture content (ie at the end of the rainy season). If construction takes place in the dry season, the roadbed should be watered to saturation immediately prior to the placing of the backfill material.
- Extend side slopes of the embankment to 1:4 for Design Traffic 1 and 1: 6 for Design Traffic 2 (there is no design traffic 1 or 2 in the table above). Utilise excavated material to flatten the side slopes of the embankment.
- No side drains unless necessary due to site conditions in which case locate them from the toe of the embankment for a distance of 4m for design traffic < 100,000 esa and 6m for design traffic 2 > 100,000 esa (see Figure D.6.29)
- Seal shoulders.
- Remove/do not plant trees within a distance of 1.5 times their mature height from the edge of the seal.

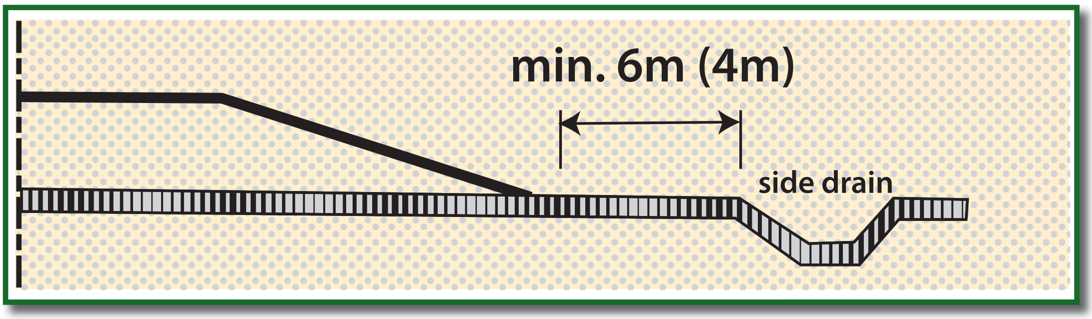

Figure D.6.29: Location of side drains in expansive soils

Countermeasure B: Use of a 'pioneer" layer (Figure D.6.30) as follows:
- Adopt countermeasures listed for Measure A.
- Place a loose layer "pioneer" layer (about 100-200mm in thickness) of permeable sand, gravel or rock fill over the clay to cover the full width of construction. It is essential that this layer should remain loose and permeable and must therefore not be compacted or trafficked.
- Allow the "pioneer" layer to stand through one full rainy season in order to pre-wet the roadbed as much as possible by the elimination of evapotranspiration, and the collection of rainwater. Prevent localised ponding of water.
- Compact the "pioneer" layer in advance of construction during the following dry season and utilize it as the first layer of fill.
- Ensure minimum earthworks cover of 0.6m for Design Traffic 1 and 1m for Design Traffic 2.
- Do not use active clay as fill.
- Replace clay under culverts to a depth equivalent to the reduction of surcharge caused by the culvert.
- Waterproof culvert joints.
- Prevent ponding of water at culvert inlets and outfalls and adjacent to road.

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

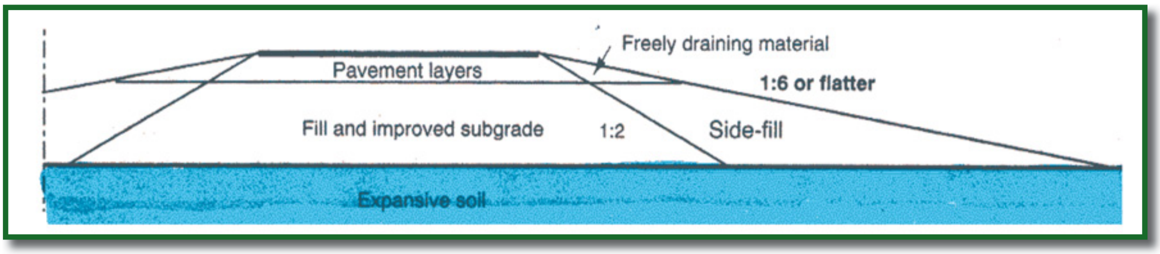

Figure D.6.30: Construction on expansive soils (use of pioneer layer)

Countermeasure C 1: Partial excavation (for embankments < 2m in height)
- Adopt countermeasures listed for Measure A;
- Excavate expansive soil over width to toe 1:2 side slope and to depth of 0.6m;
- Stockpile excavated at sides for eventual grading on to shoulder slopes;
- Backfill excavation with non-expansive fill. Ensure minimum earthworks cover of 0.6m for Design Traffic 1 and 1m for Design Traffic 2;
- Fill above ground level to be constructed with 1:2 side slopes;
- Grade and spread excavated expansive soil on fill side slopes to lengthen their slope to 1:6 or flatter, thereby extending the distance of the road over which transpiration will be reduced;
- Expansive material must not be used for the shoulder slope to the pavement – these must be constructed as wedges of permeable material as shown in Figure D.6.31.

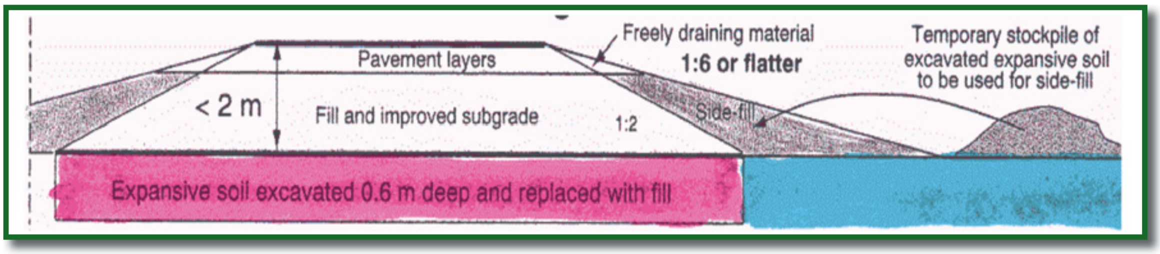

Figure D.6.31: Construction on expansive soils (embankment height <2m)

Countermeasure C 2: Partial excavation (for embankments ≥2m in height)

- Adopt same countermeasures as for Measure (C.1) except for the following:
- Excavate expansive soil under the width of the 1:2 side slopes (see Figure D.6.32).

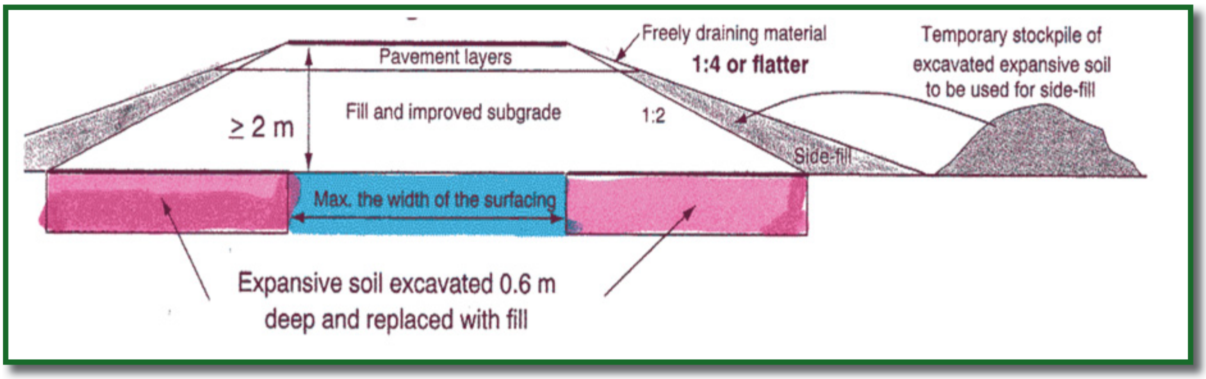

Figure D.6.32: Construction on expansive soils (embankment height ≥2m)

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

6.19.3 Collapsible soils

Collapsible soils occur mostly in the arid and semi-arid regions of eastern and south eastern Ethiopia. They exhibit a weakly cemented soil fabric which, under certain circumstances, may be induced to rapid settlement. A characteristic of these soils is that they are all unsaturated; generally have a low dry density; and a low clay content. At the in-situ moisture content they can withstand relatively large imposed stresses, well in excess of the overburden pressure, with little or no settlement. However, without any change in the applied stress, but an increase in moisture content, additional settlement will occur as shown in Figure D.6.33 and illustrated in Plate D.6.11. The rate of settlement will depend of the permeability of soil.

Countermeasures for dealing with collapsible soils: Methods for dealing with collapsible soils depend on the degree of collapse potential as discussed Chapter D.2: Site Investigations and Route Selection. The countermeasures include:

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

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

Plate D.6.11: Collapse settlement in excess of 150mm following impact compaction

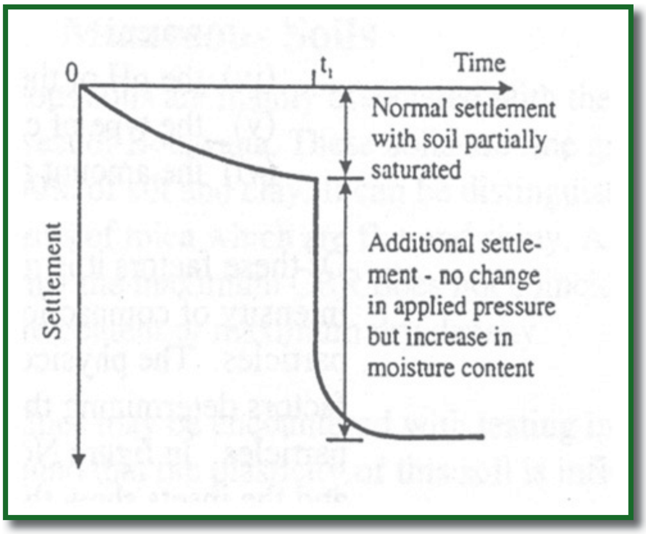

Figure D.6.33: Manner of additional settlement due to collapse of soil fabric

6.19.4 Dispersive/erodible soils

Dispersive and erodible soils are prevalent over many areas of Ethiopia (Plates D.6.12 and D.6.13). Although these soil types are similar in their field appearance (highly eroded, gullied and channelled exposures), they differ significantly in the mechanisms of their actions and are differentiated as follows:

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

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


Plates D.6.12 and D.6.13: Examples of severe erosion in erodible/dispersive soils in Ethiopia

Countermeasures for dealing with erodible/dispersive soils: Methods for dealing with dispersive soils include (Paige-Green, 2008):

- Avoiding the use of such soils in fills as far as possible and removing and replacing it in the subgrade;
- Managing water flows and drainage in the area well;
- Treating the soil with lime or gypsum to allow the calcium ions to replace the exchangeable sodium cations and reduce the problem;
- Compacting the soil at 2 to 3% above optimum moisture content to as high a density as possible (Elges, 1985) .

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

- Ensuring that the drainage in the area is well controlled;
- Covering the soils with non-erodible materials and vegetation;
- Once erosion has occurred, back-filling the channels and gullies with less erodible material and redirecting the water flows.

6.19.5 Saline soils

Saline soils occur mostly in the arid or semi-arid regions of Ethiopia where the dry climate combined with the presence of saline materials and/or saline ground or surface water, create conditions that are conducive to the occurrence of salt damage. The presence of soluble salts in the subgrade or pavement materials can cause damage to the bituminous surfacings of roads. Such damage occurs when the dissolved salts migrate to the road surface, mainly due to evaporation, become supersaturated and then crystallize with associated volume change. This creates pressures which can lift and physically degrade the bituminous surfacing and break the adhesion with the underlying pavement layer as illustrated in Plates D.6.14 and D.6.15. Generally, the thinner the surfacing layer is, the more likely the damage, primes being the most susceptible and thick, impermeable seals the least susceptible.

Plate D.6.14: An example of severe distress to a road surfacing due to salt attack resulting in damage within two years of its construction (Botswana)

Plate D.6.15: Salt damage may appear in the form of "blistering", "heaving" and "fluffing" of the prime surfacing.

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

Guidelines for the prevention and repair of salt damage to roads and runways have been developed based on research work carried out in the southern African region (Botswana Roads Department, 2003). These guidelines provide guidance on methods of testing and measurement of salts as well as repair methods where damage has already occurred.

6.19.6 Micaceous soils

Micaceous soils are those soils which contain large quantities of mica (muscovite) and occur in such materials as weathered granite, gneiss, mica schist and phylite materials that occur in various areas of Ethiopia. These soils often cause problems with compaction because of the "spring action" of the muscovite materials which may prevent achievement of the intended density or, even if it is achieved initially, can cause rutting in the compacted layer at a later stage.

Countermeasures for dealing with micaceous soils: Methods for dealing with erodible soils include:

- Removing the micaceous soil layer to below the material depth in the subgrade;
- Stabilizing the micaceous soil with lime or cement.

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

6.19.7 Low-strength soils

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

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

Further details on the respective methods of treatment for low-strength soils need to be established in the design stage at project level and the appropriate measure will depend on soil properties, site conditions, available equipment, available materials, experience from other sites with similar conditions and construction economy. The subgrade class will need to be re-defined according to the new subgrade strength after treatment.

6.20 Construction Issues

One of the challenges of utilising natural gravels in LVR pavements is to maximise their strength, increase their stiffness and bearing capacity, increase their resistance to permanent (plastic) deformation and reduce their permeability (and, hence, susceptibility to moisture ingress). These attributes can be achieved through effective compaction, as discussed below.

6.20.1 Subgrade compaction

Effective subgrade compaction is one of the most cost-effective means of improving the structural capacity of pavements. A well compacted subgrade possesses enhanced strength, stiffness and bearing capacity; is more resistant to moisture penetration; and less susceptible to differential settlement. The higher the density, the stronger the subgrade support, the lesser the thickness of the overlying pavement layers and the more economical the pavement structure. Thus, there is every benefit to achieving as high a density and related strength as economically possible in the subgrade.

Maximising the strength potential of a subgrade soil can be achieved, not necessarily by compacting to a pre-determined relative compaction level, as is traditionally done but, rather, *by compacting to the highest uniform level of density possible without significant strength degradation of the particles ("compaction to near refusal").* In so doing, there is a significant, beneficial, gain in density, strength and stiffness and reduction in permeability, the benefits of which generally outweigh the costs of the additional passes of the roller.

Compaction to near refusal ensures that the soil has been compacted at an appropriate moisture content to its near elastic state as shown in Figure D.6.34 at which point the air voids in the material are relatively low (< 5%) with the significant benefit of reduced pavement deflection and increase in pavement life as illustrated in Figure D.6.35. If, however, the volume of voids is high after construction, the pavement will densify under traffic loading and rutting will appear in the wheel tracks. Further, if both the moisture content is high in service and the air voids are also high, the pavement is potentially unstable and serious deformation is likely to occur, particularly with heavy traffic using the road. These potentially adverse situations emphasise the importance of ensuring that the subgrade compaction is carried out properly by controlling both the air voids and moisture content at which the specified density is attained.

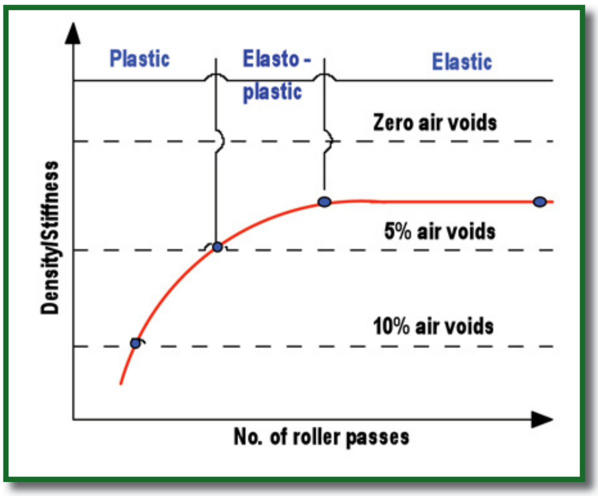

Figure D.6.34: Illustration of concept "compaction to refusal"

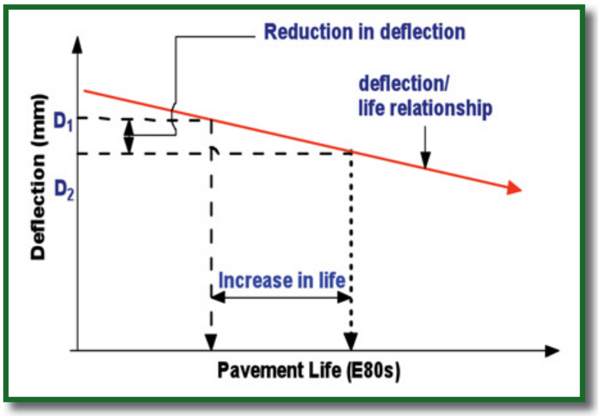

Figure D.6.35: Deflection-life relationship and benefits of "compaction to refusal"

Compaction at low moisture content: In the arid or semi-arid, north-eastern and south-eastern regions of Ethiopia where rainfall is less than 500mm per annum, water is often scarce and problems arise when large quantities (up to 2,000 m3/km are needed for road construction. In these regions qualified consideration can be given to "dry compaction" techniques for the compaction of the subgrade and pavement layers. As illustrated in Figure D.6.36, high densities can be achieved at low moisture contents using conventional compaction plant. However, as shown in Figure D.6.37, soils compacted at low moisture contents will have high air voids. As indicated above, should the degree of saturation increase in service, this may allow an ingress of water into permeable pavements even if the road surface and shoulders are sealed, resulting in a loss in soil strength and resulting deformation of the pavement structure. A life-cycle analysis will allow a determination to be made of the preferable option.

Impact compaction provides an alternative to conventional compaction plant for undertaking compaction at low moisture contents. Impact Compactors are non-circular, relatively high-energy 'rollers', typically three-(see Plate D.6.16), four- or five- sided. Large-wheeled tractors are used for pulling the compactors at operational speeds of 12 – 15 km/hr producing a series of high amplitude/high impact blows delivered to the soil at a relatively low frequency (90 – 130 blows per minute) with the energy per blow varying between 10 and 25 kilojoules, depending on the mass and amplitude of the compactor.

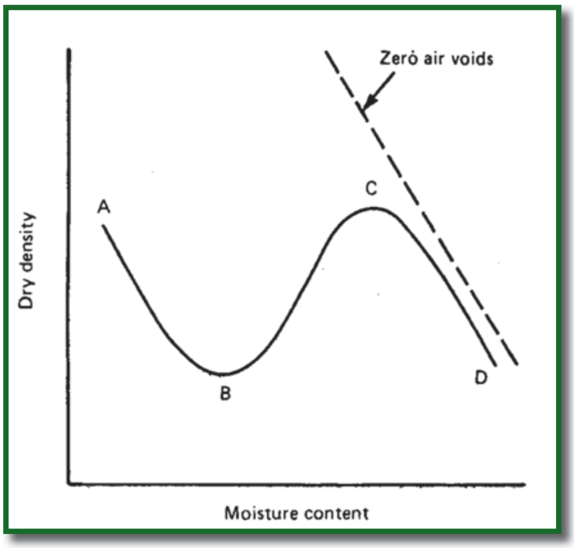

Figure D.6.36: Extended compaction curve for low moisture contents (Parsons, 1992)

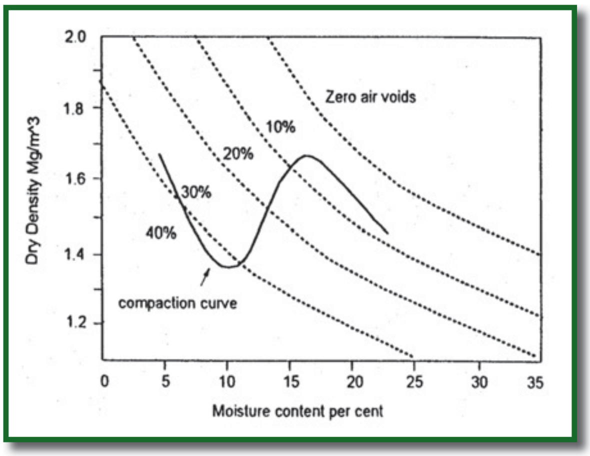

Figure D.6.37: Air voids in dry and well compacted soil (Parsons, 1992)

Plate D.6.16: Three-sided impact compactor

Due to their very high energy density per blow, their main advantage over conventional compaction plant is their depth effectiveness, typically of the order of one metre of fill or in-situ layers, thereby producing deep, well-balanced, relatively stiff pavement layers. These rollers are well suited for densifying collapsible soils. They have been successfully used in low-cost road systems and, when appropriately specified, offer a cost-effective option for LVR construction.

Minimum Compaction Requirements: Table D.6.29 gives the minimum compaction requirements for the various layers in the pavement. For the reasons stated in Section D.6.20.1, where the higher densities can be realistically attained in the field (compaction to refusal) from field measurements on similar materials or other established information, they should be specified by the Engineer.

Table D.6.29: Minimum compaction requirements

Pavement Layer	Material Class	Target Density (Relative Compaction)
Roadbase	G80 G65 G55 G45	98% – 100% T180
Subbase	G30	95% -97% T180
Subgrade/Fill Wearing Course	G15 G7	93% -95% T180
Roadbed	Sand Gravel	100% T180 93% -95% T180

6.20.2 Quality Attainment

LVR design procedures assume that both the material properties and levels of density specified are achieved in the field. However, in order to attain the specified densities, it is essential to ensure, as far as practicable, the uniform application of water, the uniformity of mixing and uniformity of compaction at or near OMC.

It is also important to note that layers below the one being compacted should be of sufficient density and strength to facilitate effective compaction of the upper layer(s). Adherence to the compaction recommendations given in Table D.6.24 should ensure this.

Granular materials which are well graded are easier to compact than poorly graded ones. It may therefore be more economical to get the gradation right (eg by mechanical stabilisation) before wasting time and energy with excessive rolling. Improved grading is also likely to improve the material strength (CBR) to an extent where a subbase quality material could become eminently suitable for road base.

Whilst it is necessary for natural gravels to be brought to OMC for efficient compaction, it is necessary to ensure that premature sealing does not lock in construction moisture. This can be achieved by allowing a significant amount of drying out to occur before sealing takes place, particularly for materials that rely on soil suction forces for strength gain and improved stability.

The variability of natural gravels is a significant factor in the reliability of performance of the pavement. However, various measures can be taken during construction to reduce such variability. These include:

- Careful selection during the winning process. Physical properties of natural gravels in most deposits tend to change with depth and location. Careful selection of the material during the winning process, coupled with appropriate testing on a grid pattern (eg use of the linear shrinkage test) will often facilitate uniform stockpiling of the material.
- Processing of stockpiled material: Power screens have been proved effective in screening out and blending in to overcome deficiencies and can be particularly useful in attaining the requirements for gravel wearing curse materials.
- Quality control and assurance: Quality attainment and control are paramount when using unprocessed materials for LVR construction. Quality assurance procedures and the use of statistical control methods are recommended. Such measures will eliminate the costly ramifications flowing from arbitrary decisions to include or exclude the use of certain readily available materials.

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7. SURFACING

7.1 Introduction

There are a large number of bituminous and non-bituminous surfacing options available for use on LVR pavements. These surfacings fulfil a variety of functions which, collectively, preserve the integrity of the underlying pavement layers and improve the functionality of the road in service. The basic local materials of natural soils/gravels, stone, fired clay brick can be used with or without a range of binders/sealers to offer a range of attributes which need to be matched to such factors as expected traffic levels and loading, locally available materials and skills, construction and maintenance regimes and the environment. Careful consideration should therefore be given to all these factors in order to make a judicious, cost-effective choice of surfacing to provide satisfactory performance and minimise life cycle costs.

This chapter provides an overview of the various types of surfacings available and appropriate for use in Ethiopia in relation to a range of local factors. The chapter also provides information on the constituents and performance characteristics of the surfacings, the factors affecting their choice and the general approach to their design.

7.2 Types of Surfacings

Road surfacings may be grouped according to their main constituents as follows:

Basic
S-01: Engineered Natural Surface (ENS)
S-02: Natural gravel

Stone Paving
S-03: Waterbound/Drybound Macadam (WBM - DBM)
S-04: Hand Packed Stone (HPS)
S-05: Stone Setts or Pavé (SSP and MSSP)
S-06: Mortared Stone (MS)
S-07: Dressed stone/cobble stone (DS, CS, MDS, MCS)

Fired Clay Brick
S-08: Unmortared/mortared joints (CB, MCB)

Bituminous
S-09: Sand Seal
S-10: Slurry Seal
S-11: Chip Seal
S-12: Cape Seal
S-13: Otta Seal

Concrete
S-14: Non-reinforced concrete (NRC)
S-15: Ultra-thin reinforced concrete pavement (UTRCP)

An outline description of the above surfacing types is presented below while their relative advantages and disadvantages are summarised in Annex D.3.

7.2.1 Basic Surfacings

S-01: Engineered Natural Surface (ENS)

An Engineered Natural Surface (Plate D.7.1)using the compacted soil at the road location to form a basic surface for traffic. Essential provisions are a compacted camber (3-6%), side drains and an effective drainage system. Typically soils with an in service CBR of a minimum of about 15 or more can provide a year round running surface for light motor traffic. Route sections with steep gradients, or weak or problematic soils can be improved in situ by upgrading to higher standard surface under a spot improvement or EOD strategy to improve their traffic carrying capacity throughout the year.

Plate D.7.1: An example of an Engineered Natural Surface (ENS)

S-02: Natural Gravel

One or more layers of natural gravel (Plate D.7.2) placed directly on the existing shaped earth formation and compacted with an appropriate surface camber (typically 3-6%). The layers could be mechanically stabilized or blended with other material to improve the properties.

Plate D.7.2: An example of a Natural Gravel Surfacing (Cinder gravel)

7.2.2 Stone Paving

S-03: Waterbound/Drybound Macadam

A Macadam layer essentially consists of a stone skeleton of single sized coarse aggregate in which the voids are filled with finer material. The stone skeleton, because of its single size large material will contain considerable voids, but will have the potential for high shear strength, if confined properly. The stone skeleton forms the "backbone" of the macadam and is largely responsible for the strength of the constructed layer. The material used to fill the voids provides lateral stability to the stone skeleton but adds little bearing capacity.

In Waterbound Macadam (WBM) the aggregate fines are washed or slushed into the coarse skeleton with water. Dry-bound macadam is a similar technique to the original WBM, however instead of water and deadweight compaction being used in the consolidation of fine material, a vibrating roller is used. The development of small vibrating rollers has made the use of this technique attractive for rural road works in some locations.

WBM or DBM are commonly used as layers within a sealed flexible pavement, but in the appropriate circumstances may be used as an unsealed option with a suitably cohesive material being used as the fines component. The WBM or DBM may be constructed as a low cost, initial surface to be later sealed and upgraded in a 'stage construction' strategy.

Plate D.7.3: An example of a Waterbound/Drybound Macadam

S-04: Handpacked Stone

Hand Packed Stone surfacing consists of a layer (typically 150 – 300 mm) of large broken stones pieces, tightly packed together and wedged in place with smaller stone chips rammed by hand into the joints using hammers and steel rods (Plate D.7.4). The remaining voids are filled with sand. The Hand Packed Stone is normally bedded on a thin layer of sand gravel. For use by heavy traffic, the layer should be compacted with a vibrating or heavy non-vibrating roller. An edge restraint or kerb constructed of large or mortar jointed stones improves durability and lateral stability.

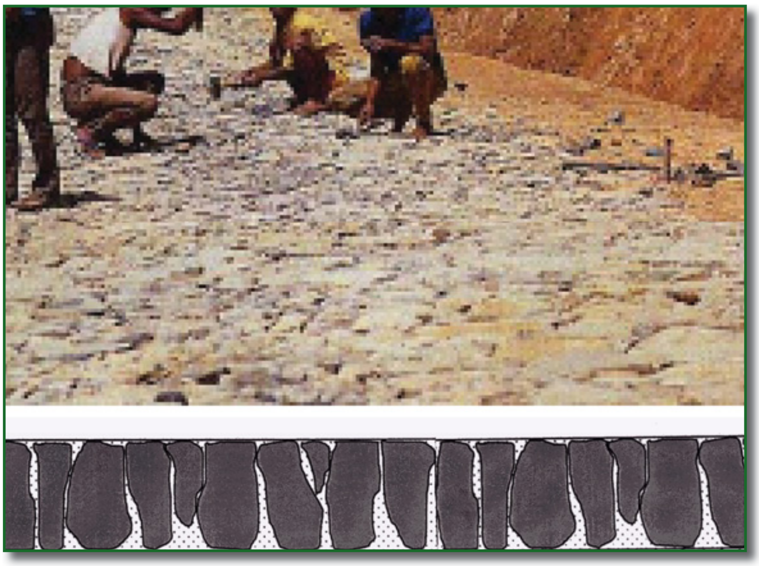

Plate D.7.4: Hand-packed Stone

S-05: Stone Setts or Pavé

Stone sett surfacing or Pavé (See Plates D.7.5 and D.7.6) is an historically well-established technique that has been adapted successfully as a robust option on low volume rural roads where there is a good local supply of suitable stone. It consists of a layer of roughly cubic (100mm) stone setts laid on a bed of sand or fine aggregate within mortared stone or concrete edge restraints. The individual stones should have at least one face that is fairly smooth, to be the upper or surface face when placed. Each stone sett is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. Sand or fine aggregates is brushed into the spaces between the stones and the layer then compacted with a roller.

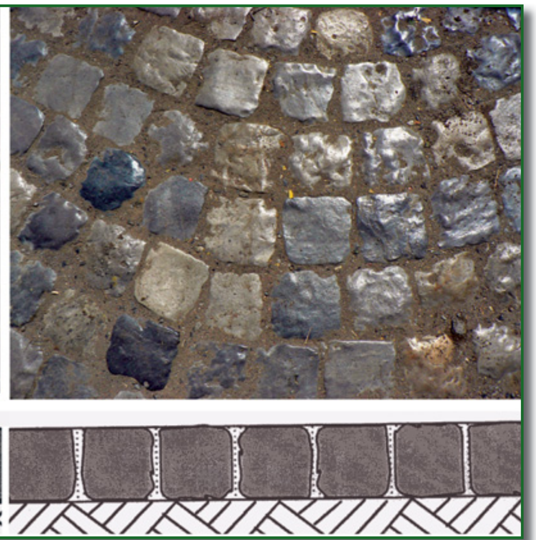

Plate D.7.5: Stone Setts (Linear pattern)

Plate D.7.6: Stone Setts (Radial pattern)

S-06: Mortared Stone

Mortared Stone Paving (Plate D.7.7) consists of a layer of natural selected stones, laid on a bed of loose sand or fine aggregate with the joints filled with sand–cement mortar. The stones do not need to be dressed to a regular shape. The individual stones should have at least one face that is fairly smooth and even, to be the upper or surface face when placed. Stone size is typically from 100 – 300mm. The bedding sand around each stone is adjusted with a small hammer and the stone is then tapped into position and to the final level of the surrounding stones. Sand–cement mortar and small stones are used to fill the joints between the individual stones. When the mortar has set the layer should be covered in sand or other moisture retaining material and kept wet for a few days to aid curing. Mortared Stone paving should not be trafficked until 7 days after laying.

Plate D.7.7: Mortared Stone

S-07: Dressed Stone/Cobble Stone Paving

Dressed or Cobble Stone Paving (Plate D.7.8) has been used for centuries as a strong, durable road surface. The technique is similar to Stone Setts or Pavé, however the individual stones are larger, normally of size 100 – 300mm. They are cut from suitable hard rock and 'dressed' manually to a cubic shape with a smooth, flat finish on at least one face using hammers and chisels. The dressed stones are laid on a bedding sand layer (20 – 70mm) and tapped into final position with a hammer. Sand is brushed into the joints between the stones. Covering with loose sand and compacting with a heavy roller can improve durability. An edge restraint or kerb constructed (for example) of large or mortared stones is required for durability. Sand-cement mortar joints and bedding can be used to improve durability and prevent water penetrating to moisture susceptible foundation layers and weakening them.

Plate D.7.8: Dressed Stone/Cobble Stone

7.2.3 Fired Clay Brick

S-08: Unmortared or mortared joints

Bricks suitable for road surfacing can be produced by firing clay in large or small scale kilns using coal, wood or some agricultural wastes as a fuel. The bricks must achieve certain strength, shape and durability requirements. The fired bricks are generally laid on edge to form a layer of typical 100mm thickness on sand or sand-cement bedding layer and jointed similarly (Plate D.7.9). Kerbs or edge restraints are necessary and can be provided by sand-cement mortared fired bricks. The fired bricks are normally laid in a herring bone or other approved pattern to enhance load spreading characteristics. Un-mortared brick paving is compacted with a plate compactor and the jointing sand is topped up if necessary. For mortar bedded and jointed fired clay brick paving, no compaction is required. When the mortar has set the layer should be covered in sand or other moisture retaining material and kept wet for a few days to aid curing. If mortared bedding and jointing are used the surface should not be trafficked until 7 days after laying.

Plate D.7.9: Fired clay brick

7.2.4 Bituminous surfacings

Bituminous surfacings or surface treatments generally comprise an admixture of different proportions of stone or sand and bitumen. The bitumen may be a penetration grade, cutback or emulsion (the last being particularly suitable for labour based methods of construction as heating is avoided; or small scale works). The bituminous surfacings usually require good quality, screened or crushed stone or sand, but lower quality aggregate may be used for some types of seals (eg Otta Seal).

An effective bond between the surface treatment and the surface of the roadbase is essential for good performance. This can be achieved through the use of an appropriate grade of bitumen (the prime or prime coat) before the start of construction of the surface treatment.

Types of Surface Treatments

Some typical types of surface treatment are shown in Figure D.7.1.

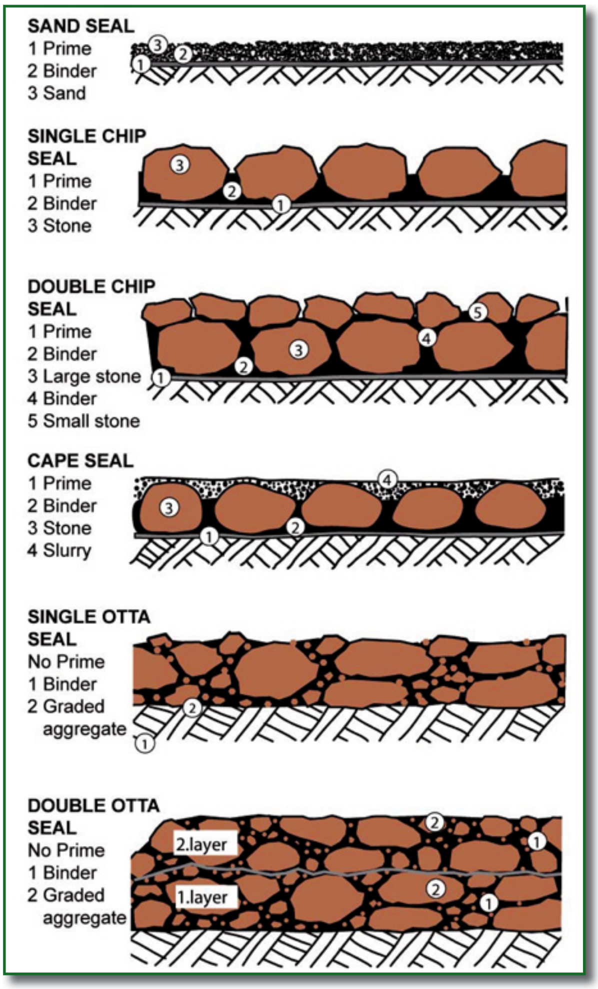

Figure D.7.1: Examples of typical surface treatments

S-09: Sand Seal

This seal consists of a spray of binder followed by the application of a coarse, clean sand or crusher dust as aggregate. This surfacing is used on low-volume roads, especially in drier regions, but can also be used for maintenance resealing, or for temporary by-passes. For new construction two layers are usually specified as single layers tend to be not durable. There is an extended curing period (typically 8 – 12 weeks) between the first and second seal applications to ensure complete loss of volatiles from the first seal and thus prevent bleeding.

S-10: Slurry Seal

A Slurry Seal consists of a homogeneous mixture of pre-mixed materials comprising fine aggregate, stable-mix grade emulsion (anionic or cationic) or a modified emulsion, water and filler (cement or lime). The production of a slurry can be undertaken in simple concrete mixers and laid by hand, or more sophisticated purpose-designed machines which mix and spread the slurry.

Slurry Seals can be used for treating various defects on an existing road surface carrying relatively low traffic for which the following are typical applications:

- Arrest loss of chippings;
- Restore surface texture;
- Reduce unevenness because of bumps, slacks and/or ruts;
- Rectify low activity surface cracking;
- New construction as a grout seal following a single Chip Seal or in multiple layers directly on the base course of low traffic roads;
- A component of a Cape Seal.

S-11: Chip Seal
This seal (single or double) consists of a spray(s) of bituminous binder followed by the application of a layer(s) of aggregate (stone chippings). The binder acts as a waterproofing seal preventing entry of surface water into the road structure while the chippings protect this film from damage by vehicle tyres.

Chip Seals can be used for a number of purposes, including:
- New construction (normally double surface dressings only);
- Temporary by-passes (normally single surface dressings);
- Maintenance resealing (normally single surface dressing);
- First layer of a Cape Seal.

S-12: Cape Seal
A Cape Seal consists of a single 13mm or 19mm aggregate, penetrated with a binder and covered with a slurry seal. If 19mm aggregate is used, the slurry is applied in two layers. The function of the slurry is to provide a dense void filler to enhance the stability of the single-sized coarse aggregate layer. The coarse aggregate is left proud to provide the macro texture for skid resistance.

S-13: Otta Seal
An Otta Seal is a sprayed bituminous surfacing comprising a mixture of graded aggregates ranging from natural gravel to crushed rock with relatively soft (low viscosity) binder, with or without a sand cover seal. This type of seal contrasts with the single sized crushed aggregate and relatively hard (high viscosity) binders used in Chip seals. The following are the main types of Otta Seals:
- Single / Double Otta Seal:
  - Pen / medium / dense graded;
  - Sand Seal / no Sand Seal cover.

Otta Seals can be used for a variety of purposes, including:
- New construction (single or double Otta Seals with/without sand seal;
- Temporary seal (normally single Otta Seal - diversions, haul roads, temporary accesses, etc);
- Maintenance reseal (normally single Otta Seal).

Performance characteristics
The mechanism of performance of surface treatments varies in relation to the composition of their constituents as illustrated in Figure D.7.2 and described below.

Type A: (eg Sand seal and Otta Seal):
These seal types, like hot-mix asphalt, rely to varying extents on a combination of mechanical particle interlock and the binding effect of bitumen for their strength. Early trafficking and/or heavy rolling is necessary to develop the relatively thick bitumen film coating around the particles. Under trafficking, the seal acts as a stress-dispersing mat comprised of a bitumen/aggregate admixture.

Type B: (eg Chip seal, Cape Seal):
These seal types rely on the binder to “glue” the aggregate particles to the base. Where shoulder-to-shoulder contact between the stones occurs, some mechanical interlock is mobilized. Under trafficking, the aggregate is in direct contact with the tyre and requires relatively high resistance to crushing and abrasion to disperse the stresses without distress.

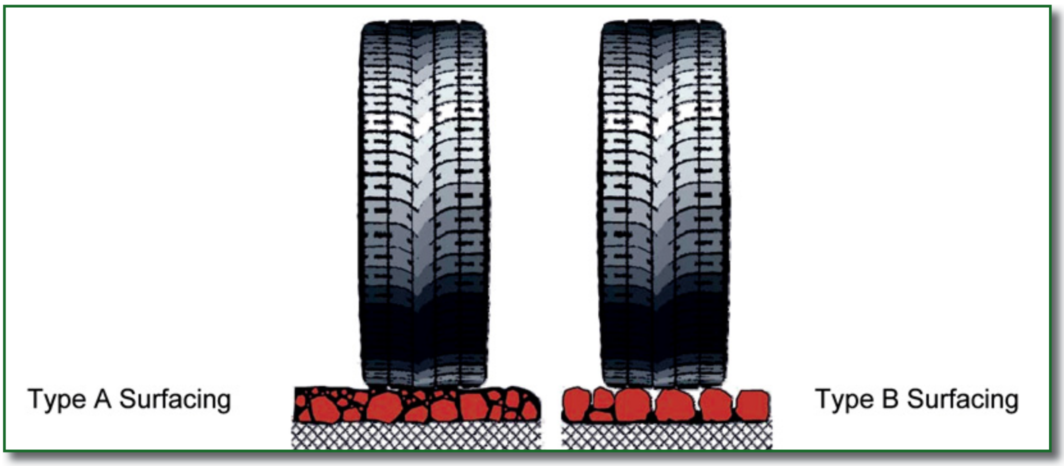

Figure D.7.2: Differing mechanisms of performance of surface treatments

Typical service lives

The life of a bituminous surfacing treatments can vary widely in relation to a number of factors as indicated below:

- **Climate:** Very high temperatures cause rapid binder hardening through accelerated loss of volatiles, while low temperatures can lead to brittleness of the binder leading to cracking or aggregate loss resulting in reduced surfacing life.
- **Pavement strength:** Lack of underlying pavement stiffness will lead to fatigue cracking and reduced surfacing life.
- **Base materials:** Unsatisfactory base performance and absorption of binder into certain base materials (eg pedogenic materials) will lead to reduced surfacing life.
- **Binder durability:** The lower the durability of the binder, the higher the rate of its hardening, and the shorter the surfacing life.
- **Design and construction of surfacing:** Improper design and poor construction techniques (eg inadequate prime, uneven rate of binder application or 'dirty' aggregates) will lead to reduced surfacing life.
- **Traffic:** The higher the volume of heavy traffic the shorter the surfacing life.
- **Stone polishing:** The faster the polishing of the stone, the earlier the requirement for resurfacing.
- **Aggregate size:** The larger the aggregate size, the shorter the service life, all other factors being equal.

Typical service lives of bituminous surface treatments are given in Table D.7.1.

Table D.7.1: Typical bituminous surfacing service lives[1]

Type of surfacing	Typical service life (years)
Single Sand seal	2 – 3
Double sand seal	3 - 6
Slurry seal	2 - 4
Single chip seal	3 - 5
Double chip seal	7 - 10
Single Otta seal	6 - 10
Single Otta seal plus sand seal	8 - 12
Cape Seal (13mm + single slurry)	6 - 10
Cape seal (19mm + double slurry)	8 - 14
Double Otta seal	12 – 16

Note:

1. Assumes that timeous routine and periodic maintenance is carried out.

Factors affecting choice of bituminous surface treatments

The various factors affecting the choice of surface treatments in relation to the operational requirements is indicated in Table D.7.2.

Table D.7.2: Factors affecting choice of bituminous surface treatments

Parameter	Degree	Type of Surfacing						
		SS	SIS	SCS	DCS	CS	SOS+ SS	DOS
Service life Required[1]	Short	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred
	Medium	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred
	Long	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Suitable/Preferred
Traffic level	Light	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred
	Medium	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred
	Heavy	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Suitable/Preferred
Impact of traffic turning action	Low	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred
	Medium	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred
	High	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Suitable/Preferred
Gradient	Mild	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred
	Moderate	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred
	Steep	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred
Material quality	Poor	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred	Suitable/Preferred
	Moderate	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Suitable/Preferred	Suitable/Preferred
	Good	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred	Less suitable/not preferred
Pavement and base quality	Poor	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Suitable/Preferred
	Moderate	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred
	Good	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred
Suitability for labour-based methods		Suitable/Preferred	Suitable/Preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Suitable/Preferred
Contractor experience/ capability	Low	Less suitable/not preferred	Less suitable/not preferred	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred	Suitable/Preferred
	Moderate	Suitable/Preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Less suitable/not preferred	Suitable/Preferred	Suitable/Preferred
	High	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred
Maintenance capability	Low	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred	Suitable/Preferred
	Moderate	Not suitable/not applicable	Not suitable/not applicable	Not suitable/not applicable	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred
	High	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred	Suitable/Preferred

Key:

SS = sand seal, SIS = Slurry Seal, SCS = single chip seal, DCS = double chip seal, CS = Cape seal, SOS+SS = Single Otta seal + sand seal, DOS = double Otta seal

Suitable/Preferred	Less suitable/not preferred	Not suitable/not applicable

Note:

1. Short < 5 years; Medium 5 – 10 years, Long > 10 years

The final choice of a surface treatment should be based on the Surfacing Decision Management System (SDMS) described in Section D.7.3.3 of this Chapter and a life-cycle cost analysis in which the various factors discussed above, as well as the service life of the treatment, should all be taken into account.

Design of bituminous surfacings

The design of a particular type of surface treatment is usually project specific and related to such factors as traffic volume, climatic conditions, available type and quality materials. Various methods of design have been developed by various authorities for the design of surface treatments. The approach to the design of surface treatments given in this section is generic, with the objective of presenting typical binder and aggregate application rates for planning or tendering purposes only. Where applicable, reference has been made to the source document for the design of the particular surface treatment which should be consulted for detailed design purposes.

Prime Coat

An effective bond between the surface treatment and the existing road surface is essential for good performance of a bituminous surfacing. This generally requires that the non-bituminous road surface must be primed with an appropriate grade of bitumen before the start of construction of the surface treatment. Typical primes are:

- **Bitumen primes:** Low viscosity, medium curing cutback bitumens such as MC-30, MC-70, or in rare circumstances, MC-250, can be used for prime coats.
- **Emulsion primes:** Bitumen emulsion primes are not suitable for priming stabilized bases as they tend to form a skin on the road surface and to not penetrate this surface.
- **Tar primes:** Low-viscosity tar primes such as 3/12 EVT are suitable for priming road surfaces but are no longer in common use because of their carcinogenic properties which are potentially harmful to humans and the environment.

The choice of prime depends primarily on the texture and density of the surface being primed. Low viscosity primes are necessary for dense cement or lime stabilized surfaces while higher viscosity primes are used for untreated, coarse-textured surfaces. Emulsion primes are not recommended for saline base courses.

The grade of prime and the nominal rates of application to be used on the various types of pavements are given in Table D.7.3.

Table D.7.3: Typical prime application rates in relation to pavement surface type

Pavement surface	Prime	
	Grade	Rate of application (l/m2)
Tightly bonded (light primer)	MC-70	0.6 – 0.7
Medium porosity (medium primer)	MC-30/MC-70	0.7 – 0.8
Porous (heavy primer)	MC-30	0.85 – 1.1

Sand Seal (S-09)

Design: There are no formal methods for the design of sand seals with the binder and aggregate application rates being based on local experience.

Materials: Typical constituents for sand seals are:

- **Binder:** The following grades of binder are typically used:
    - MC-800 cut-back bitumen;
    - MC-3000 cut-back bitumen;
    - Spray-grade emulsion (65% or 70% of net bitumen);
    - 150/200 penetration grade bitumen.

- **Aggregate:** The grading of the sand may vary to a fair degree, but the conditions of Table D.7.4 must be met:

Table D.7.4: Grading of sand for use in sand seal

Sieve size (mm)	Percentage by mass passing through sieve
6.7 0.300 0.150	100 0-15 0-2
Sand equivalent (%): 35 Min	

Application rates: For planning or tender purposes, typical binder and aggregate application rates for sand seals are given in Table D.7.5.

Table D.7.5: Binder and aggregate application rates for sand seals

Application	Hot Spray Rates of MC3000 cut-back bitumen (l/m²)	Aggregate Application Rate (m³/m²)
Double sand seal used as a permanent seal	1.2 – 1.4 per layer	0.010 – 0.012 per layer
Single sand seal used as a cover seal over an Otta Seal or Surface Dressing	0.8 – 1.0	0.010 – 0.012
Single seal used as a maintenance remedy on an existing surfaced road	1.0 – 1.2	0.010 – 0.012

Slurry Seal (S-10)

Design: The design of a Slurry Seal surfacing is based on semi-empirical methods or experience with the exact proportions of the mix being determined by trial mixes for which the following guidelines may be used:

Materials: The typical composition of a slurry is as follows:

- Filler should be between 1% and 2% of the mass of fine aggregate.
- Undiluted Bitumen Emulsion should be approximately 20% by weight of fine aggregate.

Application rates: For planning or tender purposes, the typical composition of the slurry may be based on the mass proportions indicated in Table D.7.6.

Table D.7.6: Gives a nominal slurry seal mix.

Material	Proportion (Parts)
Fine aggregate (dry)	100
Cement (or lime)	1.0 - 1.5
60% Stable grade emulsion	20
Water	+/- 15

Chip Seal (S-11)

Design: The design methods for both single and double chip seals are presented in Overseas Road Note 3 (2nd edition): *A guide to surface dressing in tropical and sub-tropical countries*. In essence, the design is based the concept of partially filling the voids in the covering aggregate and that the volume of these voids is controlled by the Average Least Dimension (ALD) of the sealing chips. Corrections to the spray rate need to be subsequently carried out to take account of site conditions as described in the guide.

Materials: Typical constituents for chip seals are:

- **Binder:** The bituminous binder can consist of any of the following:
  - 80/100 or 150/200 penetration grade bitumen;
  - MC 3000 grade cutback bitumen;
  - spray grade anionic (60) or cationic (65 or 70);
  - Modified binders (polymer modified and bitumen rubber);
  - Foamed bitumen.

- **Aggregate:** The aggregate for a Chip Seal shall be durable and free from organic matter or any other contamination. Typical grading requirements for Chip Seals are given in Table D.7.7.

Table D7.7: Aggregate requirements for Chip Seals

Sieve Size (mm)	Nominal Aggregate Size (mm)			
	19.0	13.2	9.5	6.7
	Grading (% passing)			
26.5	100			
19.0	85-100	100		
13.2	0-30	85-100	100	
9.5	0-5	0-30	85-100	100
6.7	-	-	0-5	0-40
4.75	-	-	0-5	0-40
2.36	-	-	-	0-5
0.425 (fines)	<0.5	<0.5	<0.5	<2.0
0.075 (dust)	<0.5	<0.5	<0.5	<1.0
Materials Properties				
Flakiness Index	Max 20	Max 25	Max 25	Max 30
10% FACT (dry)	AADT > 1000 vpd: Min 160 kN; AADT < 1000 vpd: 120 kN			
10% (wet)	Min 75% of corresponding 10% FACT dry			

Application rates: For planning purposes, typical binder and aggregate application rates for single Chip Seals are given in Table D.7.8.

Table D.7.8: Binder and application rates for Chip Seals

Item	Double Chip Seal		Single Chip Seal (Reseal)	
	2nd 9.5 mm 1st 19.0 mm	2nd 6.7mm 1st 13.2 mm	13.2 mm	9.5 mm
Aggregate spread rates (m³/m²)				
2nd layer	0.09	0.007		
1st layer	0.015	0.011	0.012	0.010
Hot spray rates of 80/100 pen grade bitumen (l/m²)				
Traffic AADT < 200	3.0 (total)	2.3 (total)	1.6	1.3
Traffic AADT 200-1000	2.5 (total)	1.9 (total)	1.3	1.0

Conversions from hot spray rates in volume (litres) to tonnes for payment purposes must be made for the bitumen density at a spraying temperature of 180°C. For planning purposes, a hot density of 0.90 kg/l should be used until reliable data for the particular bitumen is available.

Adhesion agents: The success of a bituminous seal depends not only upon the strength of the two main constituents – the binder and the aggregate – but also upon the attainment of adhesion between these materials - a condition that is sometimes not achieved in practice. In such a case a proprietary adhesion agent could be used to facilitate the attainment of a strong and continuing bond between the binder and the aggregate. The agent can be used in the aggregate pre-coating material (see below), in the binder or in both.

Precoating agents: Surfacing aggregates are often contaminated with dust on construction sites and, in that condition, the dust tends to prevent actual contact between the aggregate and the binder. This prevents or retards the setting action of the binder which results in poor adhesion between the constituents. This problem can be overcome by sprinkling the aggregate with water or, alternatively, by using an appropriate pre-coating material which increases the ability of the binder to wet the aggregate and improve adhesion between binder and aggregate.

A number of materials may be used for pre-coating aggregates including diesel fuel oil, cutback bitumen, bitumen pre-coating emulsion and proprietary products.

Cape Seal (S-12)

Design: As a combination single seal + slurry seal, the design of a Cape Seal is similar to that for a Chip Seal and Slurry Seal as described above.

Materials: Typical constituents for Cape Seals are:

- **Binder:** As is the case with Chip Seals, a variety of binder types may be used for constructing a Cape Seal.
- **Aggregate:** The same requirements are required as for Chip Seals and Slurry Seals.

Application Rates: For planning purposes, typical binder and aggregate application rates for single Chip seals are given in Table D.7.9.

Table D.7.9: Nominal Application rates for single Chip Seals

Nominal size of aggregate (mm)	Nominal ratesof application For planning/tender purposes	
	Binder (litres of net bitumen cold per m2)	Aggreagte (m^3/m^2)
13.2 19.0	0.6 1.1	110 75

Otta Seal (S-13)

General Design Principles: The design of the Otta Seal relies on an empirical approach in terms of the selection of both an appropriate type of binder and an aggregate application rate. Full details of the design methods are given in the *Botswana Guideline No. 1: The Design, Construction and maintenance of Otta Seals (1999).*

As a general guide, the choice of binder in relation traffic and aggregate grading is given in Table D.7.10.

Table D.7.10: Choice of binder in relation to traffic and grading

AADT (vpd) at time of construction	Type of Bitumen		
	Open Grading	Medium Grading	Dense Grading
> 1000	N/A	150/200 pen. grade	MC 3000 MC 800 in cold weather
100 - 1000	150/200 pen. grade	150/200 pen. Grade in cold weather	MC 3000 MC 800 in cold weather
< 100	150/200 pen. grade	MC 3000	MC 800

For design purposes, preferred grading in relation to traffic

Application Rates: The following Application rates for binder and aggregates are recommended:

- **Binder:** As a general guide, Table D.7.11 gives the hot spray rates for primed base courses.

Table D.7.11: Nominal binder application rates for Otta Seal

Type of Otta seal \ Grading	Open	Medium	Dense	
			AADT < 100	AADT>100
Double 1st Layer 2nd Layer	1.7 1.6	1.8 1.4	1.8 2.0	1.7 1.9
Single with Sand Cover Seal 1st Layer Fine sand Crusher Dust/Coarse River Sand	1.7 0.8 0.9	1.8 0.7 0.8	2.0 - -	1.9 0.9 0.7
Single	1.8	1.9	2.1	2.0
Maintenance Reseal (Single)	1.7	1.8	2.0	1.8

The following points should be noted with regard to the binder application rates:

- Hot spray rates lower than 1.6 l/m^2 should not be allowed.
- Binder for the sand seal cover seal shall be MC 3000 for crusher dust or coarse river sand and MC 800 for fine sand.
- Where the aggregate has a water absorbency of more than 2%, the hot spray rate should be increased by 0.3 l/m^2.

- **Aggregate:** As a general guide, Table D.7.12 gives the aggregate application rates for Otta Seals.

Table D.7.12: Nominal aggregate application rates

Type of Seal	Aggregate Application Rates (m^2/ m^3)		
	Open Grading	Medium Grading	Dense Grading
Otta Seals	63 - 77	63 - 77	50 - 63
Sand Cover Seals	83 - 100		

The following points should be noted with regard to the aggregate application rates:

- Sufficient amounts of aggregate should be applied to ensure that there is some surplus material during rolling (to prevent aggregate pick-up) and through the initial curing period of the seal.
- Aggregate embedment will normally take about 3 – 6 weeks to be achieved where crushed rock is used, after which any excess aggregate can be swept off. Where natural gravel is used the initial curing period will be considerably longer (typically 6 – 10 weeks).

7.2.5 Concrete Surfacings

Non-reinforced Concrete slab surfacing

Non-reinforced or reinforced cement concrete slab pavements can be used to provide a high strength, durable road surface with very low maintenance requirements (Plate D.7.10). Concrete of minimum 20Mpa quality is required to be used. Joints are required to accommodate thermal expansion and contraction. Particular attention is required for the design and construction of these joints. When the concrete has set the layer should be covered in sand or other moisture retaining material and kept wet for a few days to aid curing. Concrete surfaces should normally not be trafficked until 7 days after casting. It will be difficult to justify normal reinforced concrete paving for LVRs, however Ultra Thin Reinforced Concrete Paving may be an affordable option see 7.2.5(b).

Plate D.7.10: Concrete slab surfacing

Ultra Thin Reinforced Concrete Paving

An Ultra Thin Reinforced Concrete Pavement (UTRCP) option has been developed in South Africa as a low maintenance surfacing suitable for LVRs not subjected to heavy axle loading. A thin (50-60mm) layer of reinforced concrete is used in essence as a rigid "structural surfacing" over a good sub-base layer comprising well compacted good quality material, the top 150mm of which should have an effective CBR of 80%. In contrast to a more conventional NRCP the pavement layers below a UTRCP must contribute significantly to the strength of the pavement as a whole.

It should be emphasised that the formal design approach to this option is still under development and that its use within an Ethiopian LVR road environment should be undertaken with caution.

Areas where the use of UTRCP can be considered include:

- Surfacing of a new road or the rehabilitation/upgrading of an existing road;
- All traffic and road classes from low-volume urban streets to inlays, to "provincial" roads where typical traffic volumes are below 2 000 vehicles per day with less than 5% heavy vehicles (at this stage);
- Areas of steep grades and stop/start heavy traffic;
- Areas where maintenance is unlikely.

The concrete is only 50-60mm thick and therefore tolerances and quality control are critical and the success of the UTRCP process is therefore dependant on attention to detail. This applies not only to the concrete layer (concrete strength, thickness, placing, curing) but also to the placing, supporting and joining of the steel mesh panels as well as the tolerances of the layer supporting the UTRCP. The need for meticulous monitoring and control during construction cannot be over-emphasised. Competent site staff must be intensively involved in all the processes associated with and control of all the construction activities.

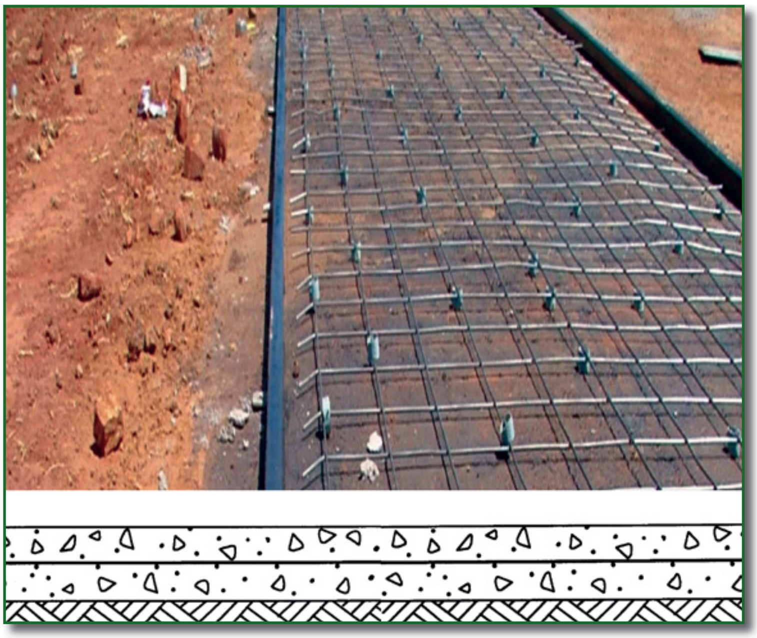

Plate D.7.11: Ultra Thin Reinforced Concrete Paving

7.3 Choice of pavement and surfacing

The various factors that typically affect the choice of a surfacing can be grouped under the following headings:

- Available materials;
- Operational environment;
- Road task;
- Natural environment.

These factors are illustrated in Figure D.7.3.

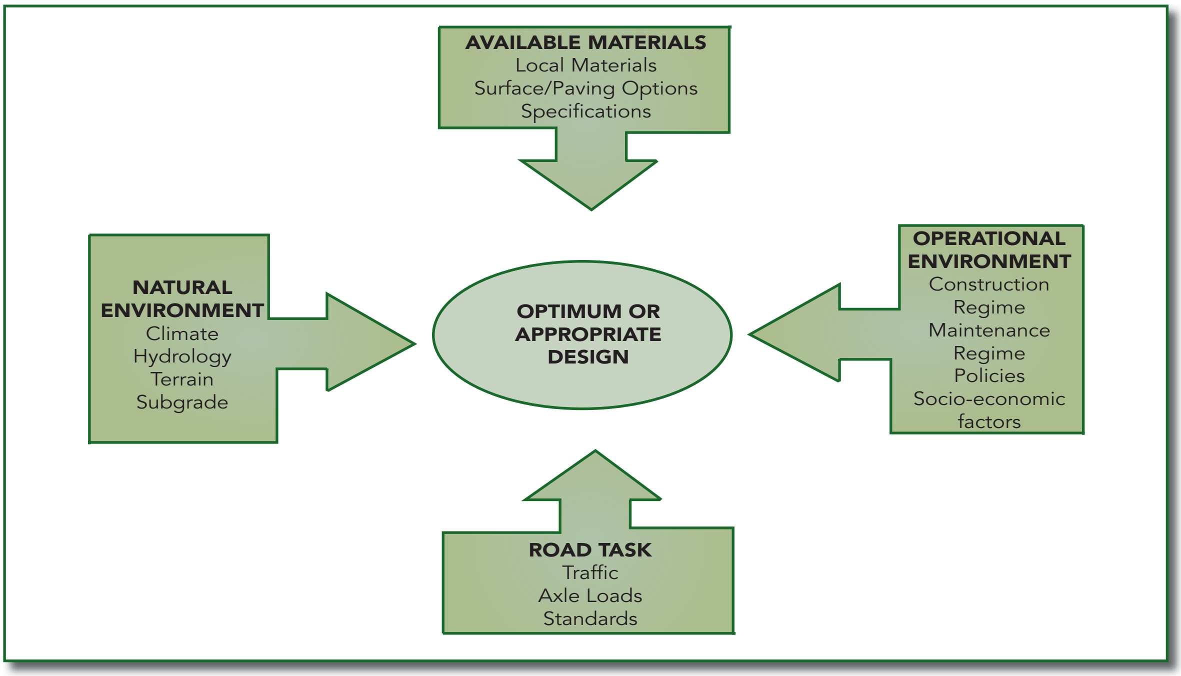

Figure D.7.3: General road surface selection factors

More specifically, the following factors should also be considered in short-listing surfacing types for more detailed consideration:

- Existing base/surface conditions;
- Design life;
- Materials (type and quality);
- Safety (skid resistance - surface texture, etc.);
- Riding quality required;
- Maintenance (capacity and reliability).

The final selection of surfacing should then be made on the basis of life-cycle costing.

7.3.1 Evaluation framework

A rational method is required for the selection of the most appropriate surface or paving structure for a particular section of low volume rural or urban road. The Surfacing Decision Management System (SDMS) provides such a procedure for assessing the various factors that influence the suitability of surface-paving options for a specific section of rural road.

When ENS or natural gravel are considered to be unsuitable options, the separate Matrices of Surfacing and Paving Options (Tables D.7.15 to D.7.18) will further guide the user to identify the most appropriate options. The key objective is the elimination of unsuitable or high risk options using a series of road environment related "screens" before proceeding to Final Engineering Design (FED) for the surfacing/paving and their Whole Life Costing. Figure D.7.4 shows the basic steps in the SDMS procedure.

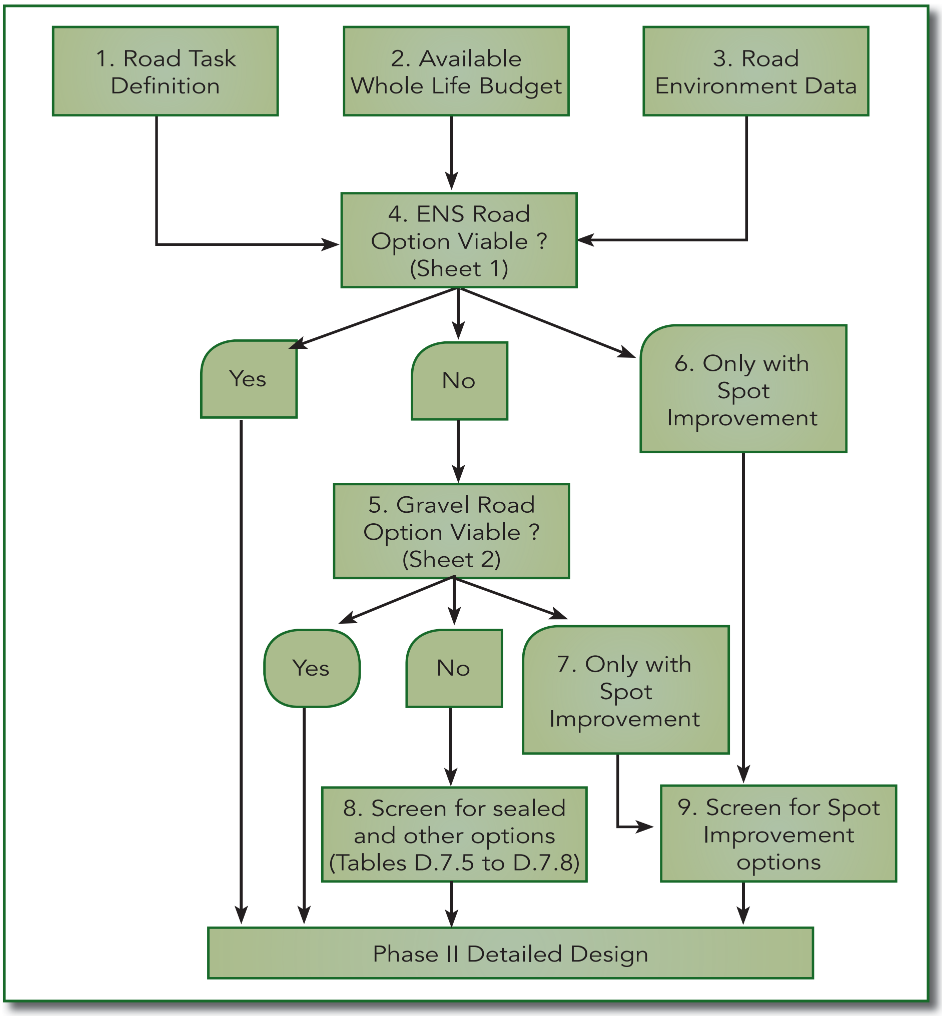

Figure D.7.4: Overview of the SDMS procedure

7.3.2 SDMS Procedure

Step 1 of the three-step SDMS procedure is illustrated in Figure D.7.5 while each of the explanatory sheets (Sheets 1-3) supporting the sequential activities are presented in Figures D.7.6 to D.7.8.

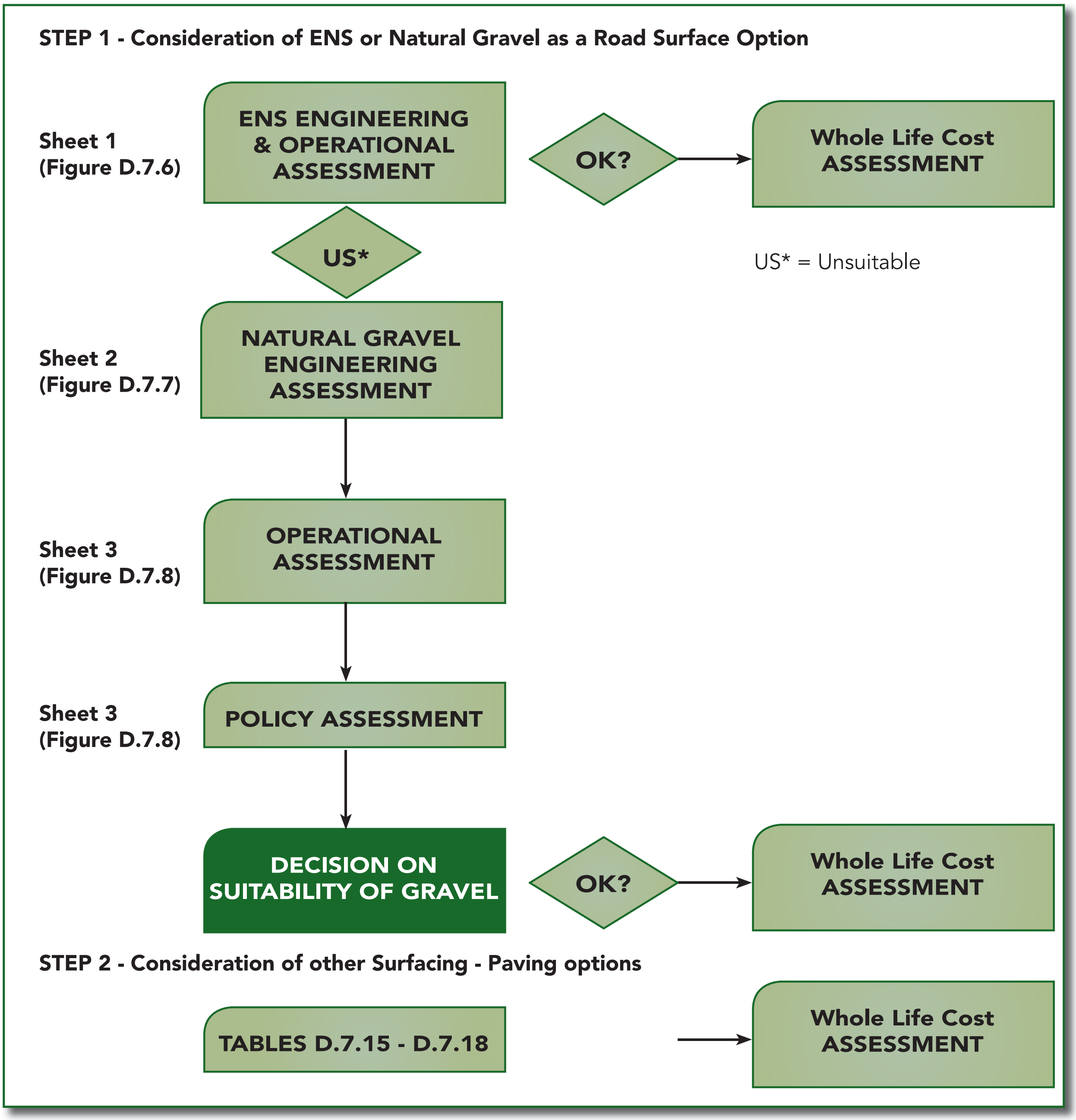

Figure D.7.5: Overview of SDMS procedure

SHEET 1 - Assessment of suitability and Engineered Natural Surface - ENS (Engineered Earth Road) as a feasible option

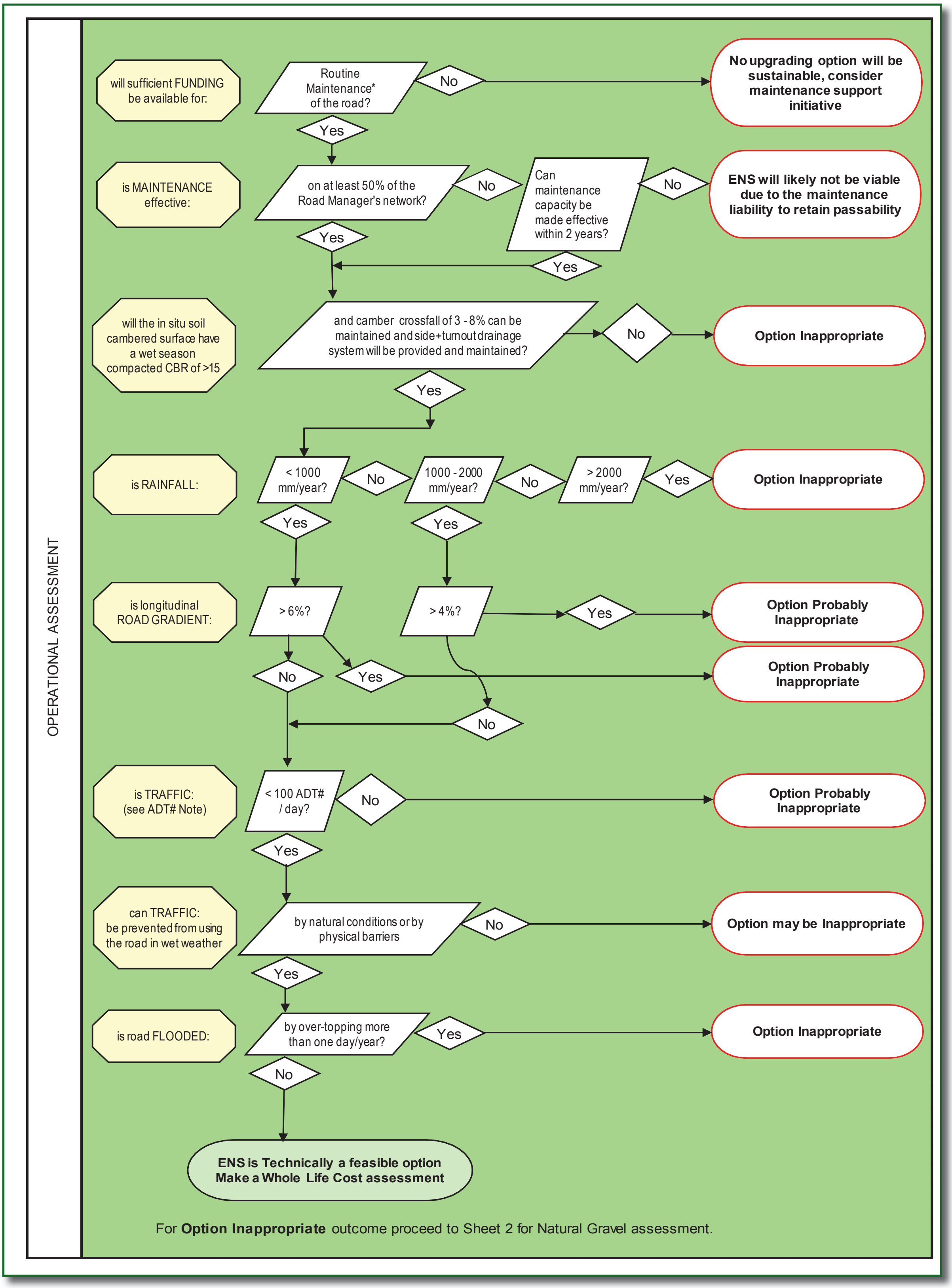

Figure D.7.6: Decision Flow Chart for the Preliminary Consideration of LVR Surface Options for a road section – STEP 1

SHEET 2 - Engineering Assessment of Natural Gravel Surface Option

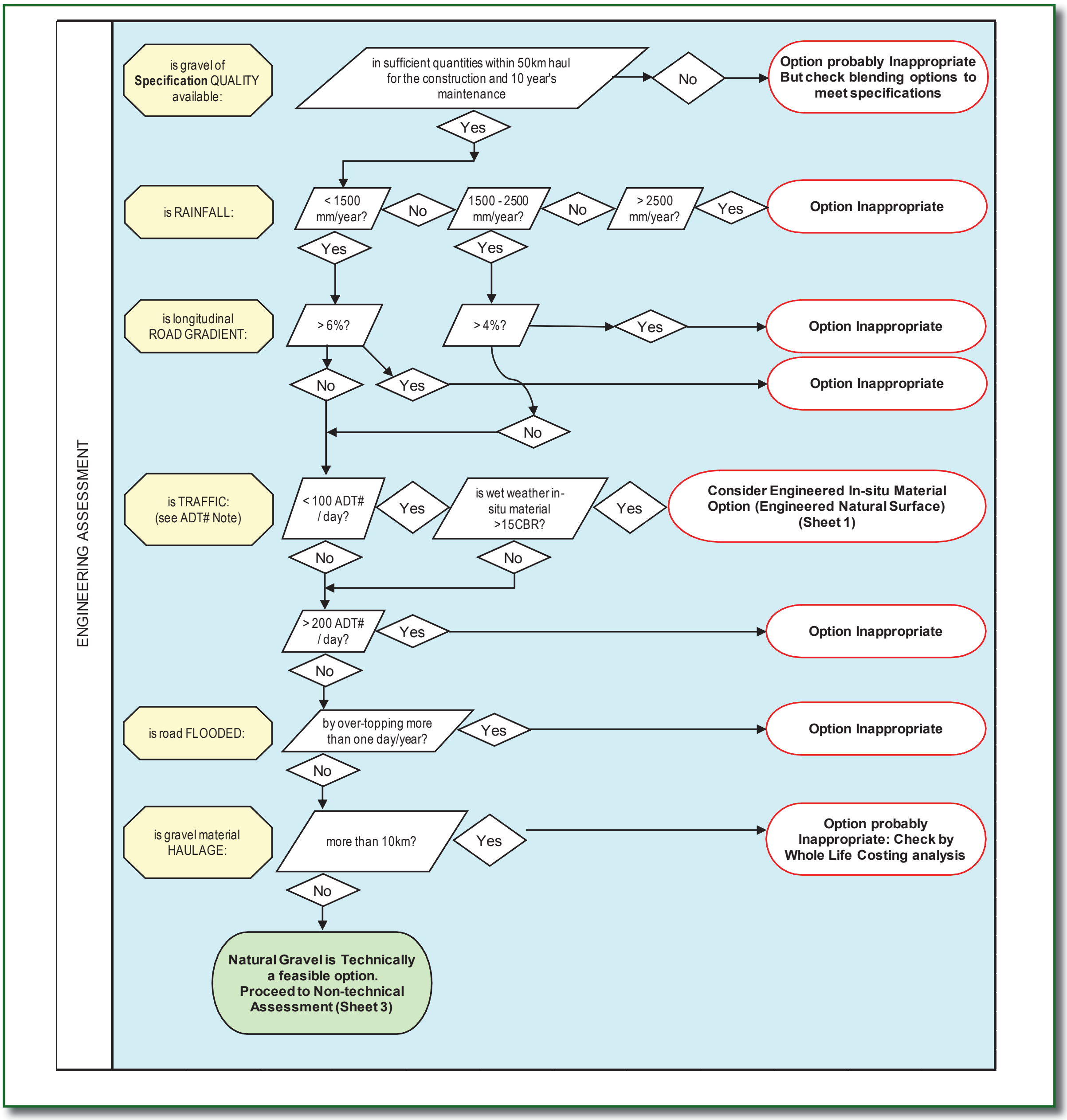

Figure D.7.7: Decision Flow Chart for the Preliminary Consideration of LVR Surface Options for a road section – Step 1 continued

SHEET 3 - Operational, Socio-economic and Economic Assessment of Natural Gravel as a surface option.

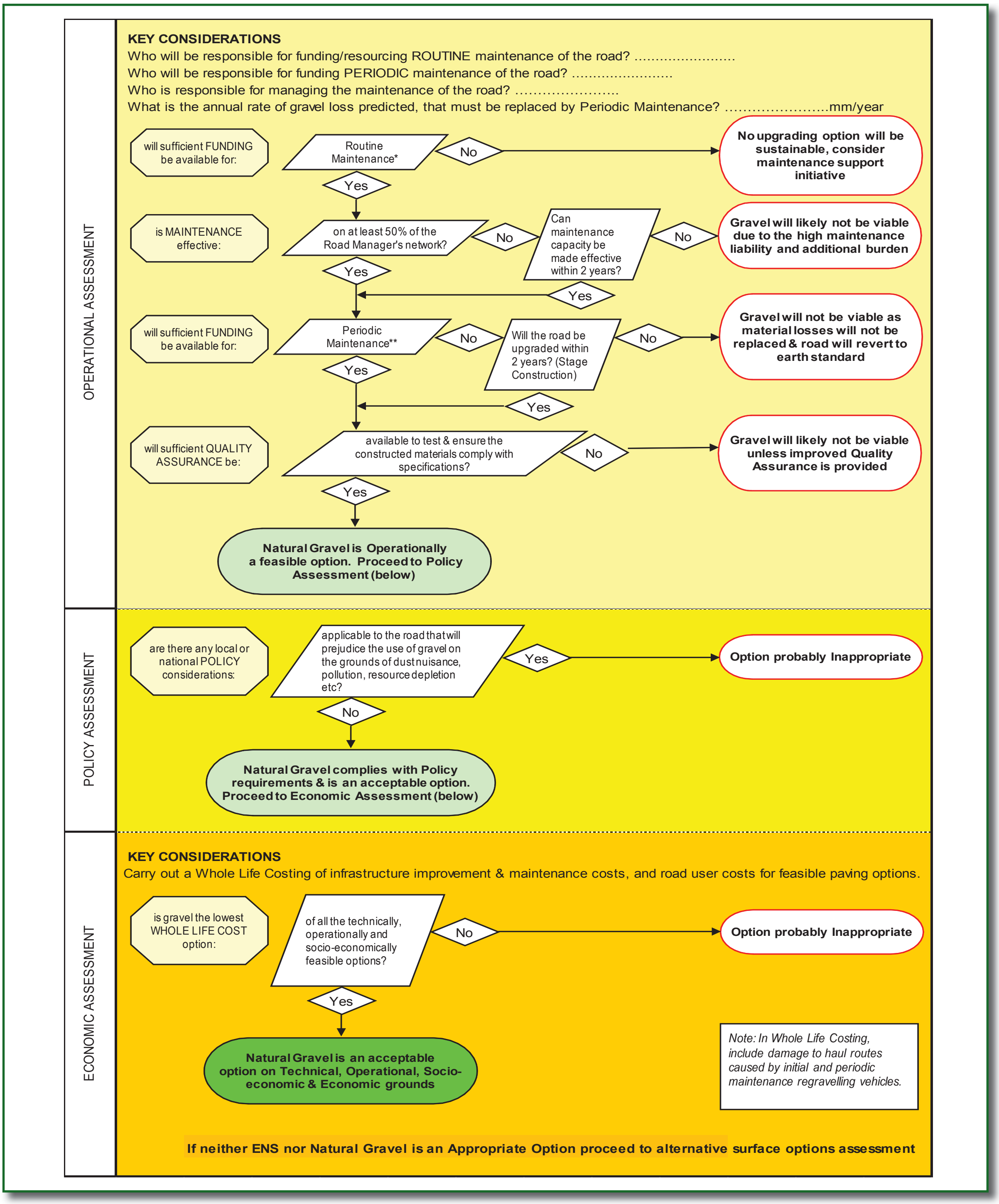

Figure D.7.8: Decision Flow Chart for the Preliminary Consideration of LVR Surface – Step 1 continued

Step 2 involves the consideration of surfacing/paving options (S-01 to S-15) as listed in Section D.7.2.

If the Step 1 assessment indicates that neither ENS nor Natural Gravel are viable options for a particular road section, then the assessment should proceed to the 'screening' process (see Tables D.7.15 to D.7.18) to select a shortlist of appropriate and viable surface and/or paving options based on the evaluation criteria included in these tables.

In the screening process the Tables D.7.13 and D.7.14 set out the evaluation criteria in terms of indicative traffic regime and erosion potential.

Table D.7.13: Definition of Indicative Traffic Regime

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

Table D.7.14: Definition of Erosion Potential

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

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

In the following Tables

√	Indicates suitable for evaluation	╱	Mortared

Note:
Cost ratings are indicative only and will depend on local factors.

Table D.7.15: Preliminary engineering filter - surfacing

PAVING CATEGORY	BASIC		STONE					BR	BITUMEN					CONC	
	Engineered Natural Surface	Gravel Surface	Waterbround/Drybound Macadam	Hand Packed Stone	Stone Setts or Pavé	Mortared Stone	Dressed Stone/Cobble Stone	Fired Clay Brick Pavement: Un-/mortared Joints	Bituminous Sand Seal	Bituminous Slurry Seal	Bituminous Chip Seal	Cape Seal	Ottaseal	Non-Reinforced Concrete	Ultra-thin Reinforced Concrete
Economically available Materials	S01	S02	S03	S04	S05	S06	S07	S08	S09	S10	S11	S12	S13	S14	S15
Crushed stone aggregate			√	√						√	√	√		√	√
Stone pieces/blocks				√	√	√	√								
Natural gravel		√											√		
Colluvial/alluvial gravel		√											√		
Weathered rock		√													
Fired clay bricks								√							
Clay soil								√							
Sand					√	√	√	√	√	√				√	√
Cement						√								√	√
Lime															
Bitumen									√		√	√	√		
Bitumen Emulsion									√	√	√	√			
Reinforcement steel															√

Table D.7.16: Primary engineering filter - pavement layers / shoulders

PAVING CATEGORY	BASES							SUB-BASES						SHOULDERS			
	Waterbound macadam	Drybound macadam	Natural gravel	Armoured gravel	Cement stabilised soil	Lime stabilisede soil	Emulsion s tabilised soil	Waterbound macadam	Drybound macadam	Natural gravel	Cement stabilised soil	Lime stabilisede soil	Emulsion stabilised soil	Stone macadam	Natural gravel	Cement stabilised soil	Lime stabilisede soil
Economically available Materials																	
Crushed stone aggregate	√	√		√				√	√					√			
Stone pieces/ blocks																	
Natural gravel			√	√						√					√		
Colluvial/alluvial gravel			√	√						√					√		
Weathered rock			√	√						√					√		
Fired clay bricks																	
Clay soil						√						√					√
Sand					√		√				√		√			√	
Cement					√						√					√	
Lime						√						√					√
Bitumen																	
Bitumen Emulsion							√						√				
Reinforcement steel																	

Table D.7.17: Primary engineering filters (continued) - surfacing

PAVING CATEGORY	BASIC		STONE					BR	BITUMEN							CONC	
	Engineered Natural Surface	Gravel Surface	Waterbround/Drybound Macadam	Hand Packed Stone	Stone Setts or Pavé	Mortared Stone	Dressed Stone/Cobble Stone	Fired Clay Brick Pavement: Un-/mortared Joints	Bituminous Sand Seal	Bituminous Slurry Seal	Bituminous Chip Seal (Single)	Bituminous Chip Seal (double)	Cape Seal	Ottaseal (Single)	Ottaseal (Double)	Non-Reinforced Concrete	Ultra-thin Reinforced Concrete
Traffic Regime: See Table D.7.3	S01	S02	S03	S04	S05	S06	S07	S08	S09	S10	S11	S11	S12	S13	S13	S14	S15
Light traffic	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Moderate traffic		√	√	√	√	√	√	√				√	√	√	√	√	√
Heavy traffic (overload risk)					√		√					√			√	√	
Construction Regime																	
High labour content	√			√	√	√	√	√	√	√	√	√	√			√	√
Intermediate machinery	√	√	√	√					√	√	√	√	√			√	√
Low cost	√	√		√					√	√	√						
Moderate cost			√		√	√		√				√	√	√			
High cost							√								√	√	√
Maintenance Requirement																	
Low					√	√	√	√								√	√
Moderate				√					√	√	√	√	√	√	√		
High	√	√	√														
Erosion Regime (See Table D.7.4)																	
A low erosion regime	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
B Moderate erosion regime				√	√	√	√	√				√	√	√	√	√	√
C High erosion regime					√	√	√								√	√	√
D Very high erosion regime					√	√	√								√	√	√

Table D.7.18: Secondary engineering filters - pavement layers / shoulders

	BASES							SUB-BASES						SHOULDERS				
	Waterbound macadam	Drybound macadam	Natural gravel	Armoured gravel	Cement stabilised soil	Lime stabilisede soil	Emulsion s tabilised soil	Waterbound macadam	Drybound macadam	Natural gravel	Cement stabilised soil	Lime stabilisede soil	Emulsion stabilised soil	Stone macadam	Natural gravel	Cement stabilised soil	Lime stabilisede soil	Sealed
Traffic Regime: See Table D.7.3																		
Light traffic	√	√	√	√	√	√	√	√	√	√	√	√	√	/	/	/	/	√
Moderate traffic	√	√	√	√	√		√	√	√	√	√	√	√	/	/	/	/	
Heavy traffic (overload risk)	√	√						√	√	√	√			/	/	/	/	
Construction Regime																		
High labour content																		
Intermediate machinery	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Low cost			√	√						√					√			
Moderate cost	√	√			√	√		√	√		√	√		√		√	√	√
High cost							√						√					
Maintenance Requirement																		
Low	√	√	√	√	√	√		√	√	√	√	√	√					√
Moderate							√							√		√	√	
High															√			
Erosion Regime (See Table D.7.4)																		
A Low erosion regime														√	√	√	√	√
B Moderate erosion regime	/	/	/	/	/	/	/	/	/	/	/	/	/	√				√
C High erosion regime	/	/	/	/	/	/	/	/	/	/	/	/	/					√
D Very high erosion regime	/	/	/	/	/	/	/	/	/	/	/	/	/					√

APPENDIX D.1

ASIST Information Service
Technical Brief No 9

Material selection and quality assurance

for labour-based unsealed road projects

International Labour Organisation
ADVISORY SUPPORT, INFORMATION SERVICES, AND TRAINING (ASIST)
Nairobi, Kenya

The Employment-Intensive Programme (EIP) is a sub-programme within the Development Policies Department (POLDEV) of the ILO. Its objective is to promote the use of local resource based technologies in infrastructure works in developing countries and to strengthen their capacity to apply such technologies.

ASIST is a sub-regional programme under the EIP, one of whose objectives is to achieve an improved effectiveness of road construction, rehabilitation and maintenance in Sub-Saharan Africa and thereby promote employment and income generation in the rural and urban areas.

The aim of ASIST Technical Briefs is to spread knowledge about labour-based technology and management amongst policy makers, planners, designers, implementers and trainers.

Material selection and quality assurance

for labour-based unsealed road projects

First edition

This publication was developed by the ASIST technical team in Harare, Zimbabwe, and Nairobi, Kenya.

Written by Dr P Paige-Green of the CSIR Division of Roads and Transport Technology, Pretoria, South Africa (Contract Report CR-97/047).
Editing and layout by David Mason.

International Labour Organisation
Advisory Support, Information Services, and Training (ASIST)
Nairobi, Kenya

Technical Brief No. 9: Material selection and quality assurance for labour-based unsealed road projects

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This document is part of an occasional series of technical briefs produced by ILO/ASIST to synthesise and summarise technical information on important aspects of labour-based technology.

The original work for this brief was undertaken under contract by Dr P Paige-Green of the Division of Roads and Transport Technology (Transportek) of the CSIR in Pretoria. He produced a report, which was subsequently edited to produce this brief.

Transportek have also put together a Gravel Road Test Kit (see Annex B) for use with this brief. This kit was demonstrated by them at the Sixth Regional Seminar for Labour-based Practitioners, held in Jinja, Uganda, in 1997. ASIST plans to evaluate the kit during 1999.

List of abbreviations and definitions

AASHTO American Association of State Highway and Transportation Officials. This association adopted a test proposed by the US War Department in 1943, to cater for larger earth moving and compaction equipment. It is now referred to as the AASHTO test.

ASIST Advisory Support, Information Services and Training for labour-based technology.

Atterberg limits Atterberg limits are measured for soil materials passing the No. 40 sieve: the shrinkage limit (SL) is the maximum water content at which a reduction in water content will not cause a decrease in the volume of the soil mass. This defines the arbitrary limit between the solid and semisolid states. The plastic limit (PL) is the water content corresponding to an arbitrary limit between the plastic and semisolid states of consistency of a soil. The liquid limit (LL) is the water content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil.

BLS bar linear shrinkage.

BS British Standard. BS 1377 defines the British Standard compaction test, introduced by R. R. Proctor in 1933. It used a compactive effort which roughly corresponded to that available in the field at the time.

CBR California Bearing Ratio. A measure of soil strength, determined from the load required to penetrate the surface of the compacted soil, expressed as a percentage of a standard value.

Clegg Hammer A simple device utilising a decelerometer, installed in a modified Proctor compaction hammer, to evaluate the stiffness of a material by measuring the deceleration encountered when the falling hammer meets the material.

DCP Dynamic Cone Penetrometer. Apparatus for estimating the *in situ* shear strength of a material by dynamically driving a standard cone through the material.

Grading Coefficient (G_c) A measure of the potential for particle interlock defined by the product of the gravel component of the material (the percentage retained between the 26.5 and 2 mm sieves) and the percentage passing the 4.75 mm sieve.

Maximum dry density (MDD) The maximum dry density which can be achieved under a specified compaction effort at the optimum moisture content.

Optimum moisture content (OMC) The moisture content at which the maximum dry density for any combination of material and compaction effort is obtained. The importance of this is particularly relevant to labour-based projects as the compaction effort using small pedestrian rollers can seldom be equated to the traditional AASHTO and BS compaction efforts. Higher OMCs will often be necessary to achieve maximum density for these efforts.

Oversize index (I_o) The stoniness as defined by the percentage of material larger than 37.5 mm.

Proctor Mr R. R. Proctor was the author of the original BS compaction standard. The compactive effort is supplied by a 2.5 kg hammer with a 50 mm diameter head falling freely from 300 mm above the top of the soil sample.

Rapid Compaction Control Device (RCCD) A simple impact penetrometer which injects a small cone into the material to estimate the shear strength of the material.

Ravelling A process where the surface material of a road is broken down by traffic to form loose material (*e.g.* gravel). The process is likely to occur where there is a deficiency of fine material, low cohesion between particles, poor particle size distribution, and inadequate compaction.

Shrinkage Product (S_p) A measure of the plasticity of the soil defined by the product of the bar linear shrinkage and the percentage passing the 0.425 mm sieve.

vpd Vehicles per day. That is, a count of the number of vehicles passing along a road in one day.

1 Background

The implementation of labour-based construction techniques for unsealed roads is beneficial in that it creates employment opportunities and assists with the development of small contractors whilst upgrading the transportation network in developing countries. Improvement in the techniques utilised during this type of construction project will result in greater cost-effectiveness and better performance of the completed product. The Advisory Support, Information Services and Training (ASIST) programme of the International Labour Organisation (ILO) has taken the lead in this. With funding from the Swedish International Development Cooperation Agency (Sida), ASIST is currently involved in the implementation of innovative technologies on various labour-based road projects in Zimbabwe and in other countries of Sub-Saharan Africa.

During an earlier visit to labour-based projects in progress in Zimbabwe, the Division of Roads and Transport Technology (Transportek) of the CSIR in Pretoria was contracted by ASIST to evaluate the procedures used regarding material selection, testing and control. In the second phase of the contract, the brief was to prepare a short guideline document on the selection and control of borrow materials, and on control of the construction process during labour-based unsealed road projects. Recommendations on the thickness design of the road are also provided.

The guidelines themselves are incorporated as an Annex to this document. The background to the decisions as to what is incorporated in the guidelines, with justification for these decisions, makes up the main text of this Technical Brief.

2 Material specifications

2.1 Recommended specifications

The performance of any unsealed road is primarily a function of the materials from which the road is constructed. It is therefore essential that the best available materials which comply with, or are as close as possible to, the appropriate material requirements be used for construction. These material requirements need, of necessity, to be simply and rapidly determined at low cost to allow sufficient samples to be tested prior to use on the road.

Numerous material specifications have been developed and utilised over time in various countries, which take into account the local material and environmental conditions. Most specifications, however, have been derived from the original AASHTO requirements which are primarily based on theoretical considerations for maximum particle packing of low plasticity materials (the dominant material derived from glacial tills in the northern United States). Experience has shown that materials with low plasticity lack adequate cohesion to resist ravelling, or the formation of corrugations, under traffic.

Regional specifications were subsequently adapted to allow for slightly higher plasticities, but in very few cases was the lower limit for plasticity specified. For this reason, variable success was obtained using the available specifications. Various projects to determine performance-related specifications for unsealed road materials were therefore carried out in South Africa and Namibia during the 1980s and early 1990s (see References 1, 2, 3, and 4). These specifications have recently been evaluated in a number of regions and countries (including Zimbabwe) and have generally been found to be more appropriate than those previously used (and in many cases more appropriate than even those currently used).

The traditional properties used in existing material specifications for unsealed roads are particle size distribution, Atterberg limits, remoulded strength, and aggregate hardness. These are similar to those found by local research to be necessary. All these parameters are critical to the performance of materials in unsealed roads, but the traditional methods of defining and evaluating them are considered to be inappropriate for labour-based projects. This is discussed further in this brief.

The material specifications recommended for the selection of borrow materials for wearing courses for unsealed roads using labour-based construction methods are given in Table 1. These should be the desired specifications for a project. Testing of all potential borrow materials for compliance with these should optimally be carried out during the borrow pit or initial materials evaluation. This should be done by a central laboratory for a Public Authority, or by a commercial laboratory, using traditional test methods, *e.g.* those contained in TMH 1 (see References 6 and 7).

If no central laboratory is available, or if the results from the laboratory are delayed, all the above testing except the soaked CBR can be determined in a simple field 'laboratory' using minimal equipment. If full testing facilities do not exist and if the road is likely to carry less than 50 vehicles per day with less than 10 per cent heavy vehicles, the Shrinkage Product and Grading Coefficient alone can be taken as the preliminary acceptance criteria. The material strength (CBR) can be evaluated during proof rolling trials as discussed in Chapter 3. Full test methods are provided in Chapter 3.

Table 2.1: Material specifications for labour-based road projects

Maximum size (mm)	37.5
Oversize Index (I_o)	• 5 %
Shrinkage product (S_p)	100 – 365
Grading coefficient (G_c)	16 – 34
Soaked CBR (%)	• 15 at 95 % Modified AASHTO density
Treton Impact value (%)	20 – 65

I_o = Percentage retained on 37.5 mm sieve
S_p = Bar Linear shrinkage × per cent passing 0.425 mm sieve
G_c = (Per cent passing 26.5 mm – per cent passing 2.0 mm) × per cent passing 4.75 mm/100
Treton Impact Value (see Section 3.4.3)

The relationship between the Shrinkage Product and the Grading Coefficient is directly related to the performance as shown in Figure 1, with zones E1 and E2 being the recommended areas for best performance. This Figure shows the predicted performance and the implications (potential problems) of using material not falling within the specified limits.

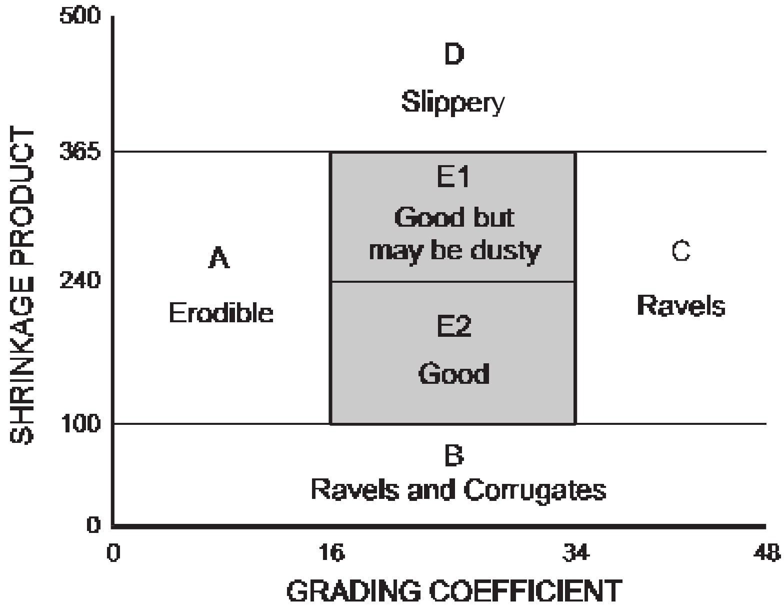

Figure 2.1: Relationship between Grading Coefficient, Shrinkage Product, and performance

3 Borrow material testing

3.1 General

It is essential that, during the initial proposal stage for any project, suitable borrow pits are located, the materials are adequately tested for compliance with the specifications given in Chapter 2, and the suitable borrow areas are carefully delineated in the field. In most cases, this should be carried out by the regional soils laboratory, as far as possible using traditional test methods and equipment as discussed in Chapter 2. Problems have, however, been encountered in the past, with the test results often only becoming available after construction has commenced. The following methods are proposed for control testing of materials during construction, but could also be used to replace or complement the initial borrow investigations where problems with obtaining results in time are encountered.

3.2 Test requirements

Traditional test techniques have been developed, based on the assumption that various basic services and facilities are available. On many labour-based projects, certain simple assumptions, such as that electricity and running water will be available, are invalid. The test techniques and methods summarised in this chapter and in Annex A allow for these. As far as possible, solar energy, local water (preferably potable), and unsophisticated equipment are utilised. It is assumed that everyday objects such as batteries are available.

The specifications discussed in the previous chapter are mostly based on simple tests which can be carried out rapidly on site using minimal equipment. The following parameters should be evaluated:

- Grading
- Shrinkage
- Aggregate hardness
- Material strength.

These tests are carried out as follows, with the complete methods of non-standard tests being presented in Section 3.4.

3.2.1 Grading

The grading requirements for the characterisation of material for unpaved roads are based on only five sieve sizes, that is 37.5 mm, 26.5 mm, 4.75 mm, 2 mm, and 0.425 mm. For the testing, the material needs first to be dried[1], the mass determined, and then the material sieved (manual shaking) through the recommended sieves above with a soft brush

[1] Air drying in direct sunlight is adequate for most materials which are potentially suitable, although the use of a solar oven is recommended.

being used where necessary. The mass of each portion is determined. The oversize index, grading coefficient and percentage passing the 0.425 mm sieve can then be determined. It should be noted that the influence of the hygroscopic moisture content on the parameters determined is negligible. The fraction passing the 0.425 mm sieve should be retained for shrinkage testing.

3.2.2 Shrinkage

The bar linear shrinkage test is carried out on the fraction passing the 0.425 mm sieve. The material should be moistened until it is at or very near the liquid limit (this can be checked with a simple fall-cone device (see Section 3.4)), placed in the mould, and oven-dried at 105°C until all shrinkage has stopped. The length of the sample is then measured and the percentage shrinkage calculated. It is recommended that the sample is dried for at least 12 hours (overnight if not done in a solar oven), but experience has shown that this can take as little as four or five hours, depending on the soil. The length of time necessary can be checked by drying to constant mass. However, preliminary research has shown that air-drying of samples is not effective for repeatable results.

3.2.3 Aggregate hardness

Aggregate hardness measurements are necessary to identify those materials which will disintegrate under rolling or traffic, as well as those which are excessively hard and will result in a rough road if too much of this type of material is included. The Treton test is used to determine this. The Treton impact value is determined by means of a simple impact hammer action on a single sized sample (obtained during the sieve analysis). This test is unnecessary if the road is unlikely to carry many buses or heavy vehicles (more than two per day) or if the material lacks a significant proportion of medium to coarse gravel (< 15 per cent retained on a 16 mm sieve).

3.2.4 Material strength

Material strength is an indication of the capacity of the material to support the wheel loads of the traffic using the road. The traditional method for determining this property is the soaked California Bearing Ratio (CBR) test. This test is routinely carried out in a central or typical site laboratory but is expensive to set up, requires a large amount of equipment, and is relatively time consuming.

As an alternative, it is considered more practical to first carefully compact a sample of the material, at the estimated optimum moisture content, to the required thickness on a subgrade prepared to the same standard as that which will be used in construction. Then to measure the resistance to penetration with a Dynamic Cone Penetrometer (DCP) (see Figure 2), a Rapid Compaction Control Device (RCCD) (see Figure 3) or a Clegg Hammer. The moisture content at the time of testing (assumed to be at or about OMC) should be taken into account (See Section 4.2.1). Acceptable values of penetration for the DCP and RCCD are given in Table 3.1.

Table 3.1: Penetration rates of DCP and RCCD for equivalent soaked CBR values of 15 % (tested at OMC)

Apparatus	Penetration rate	Penetration (3 blows)	Penetration (20 blows)
	(mm/blow)	*(mm)*	*(mm)*
DCP	• 5	• 15	• 100
RCCD	• 9	• 27	—

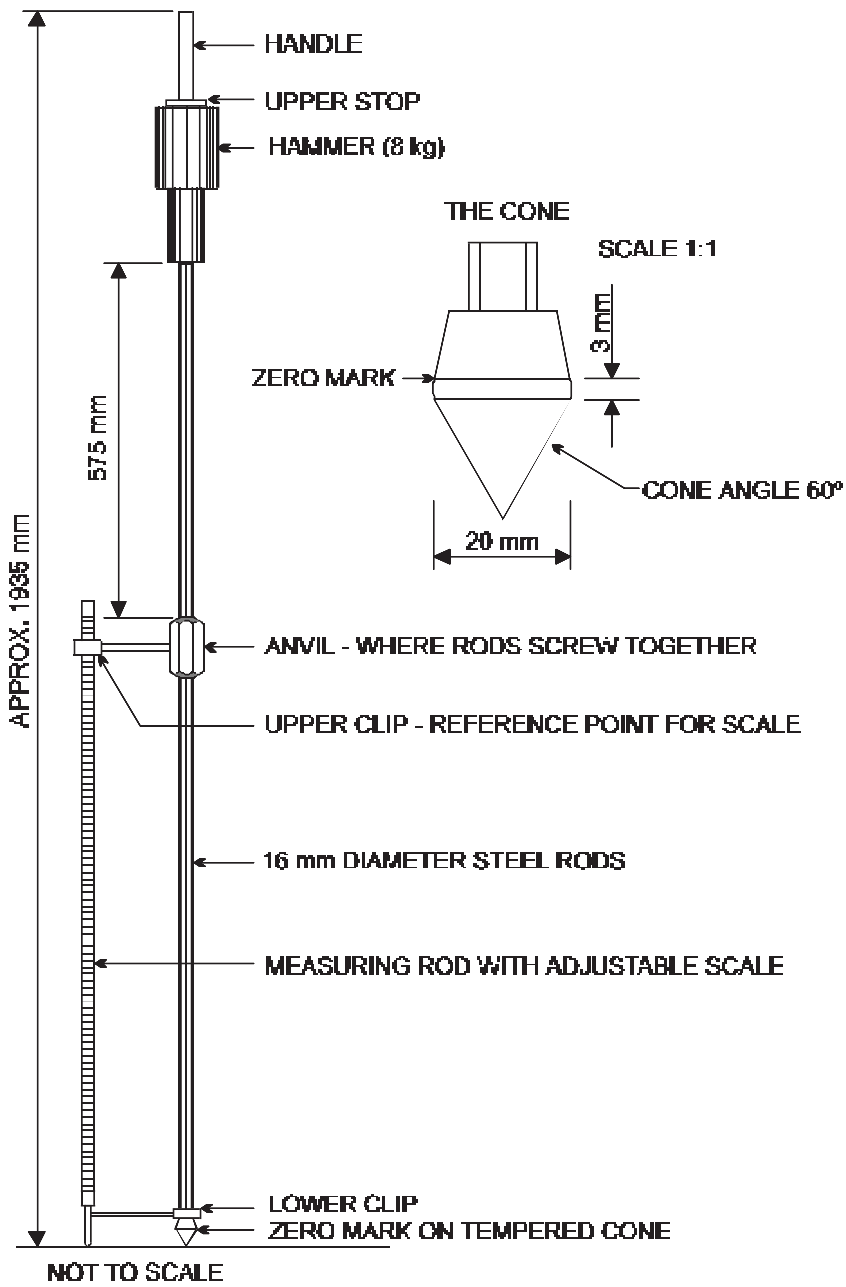

Figure 3.1: DCP test apparatus

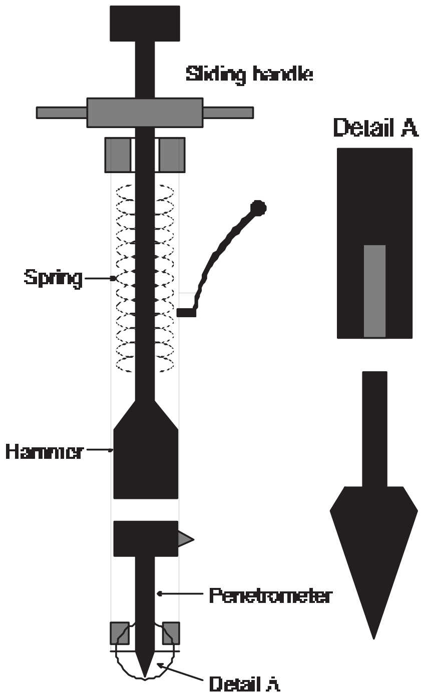

Figure 3.2: Diagram of RCCD device

The RCCD is recommended for use since the test is simpler and quicker. The apparatus is more robust (only periodic calibration of the spring is necessary), but less bulky than that required for the other two methods of control, and it has less operator variability. More tests per job lot (day's production) can be carried out more economically with the RCCD than with the other methods. However, the DCP penetration rates given in Table 3.1 can also be used for material characterisation and control purposes.

Selection of materials based on the specified G_c and S_p will in most cases exclude those which are likely to have insufficient CBR strength.

3.3 Frequency of laboratory testing

The frequency of testing of borrow pits needs to strike a balance between cost and time and statistical validity of the results. It is proposed that, even for labour-based projects, the location of borrow materials and borrow-pit testing should preferably be done according to traditional methods. If full laboratory facilities are not available, the methods described in this report can be substituted.

The frequency of testing will depend on the variability of the material: the more homogeneous the material the less the amount of testing necessary for statistical validity of the results. Unless proper testing of the borrow materials is carried out prior to commencement of the project, it is usually not possible to quantify the variability in advance of construction. For projects of this nature it is thus necessary to test samples from at least five locations per borrow pit (covering the full depth of the layer to be used) in order to quantify the variability. The sample locations should be randomly selected within the pit. This variability is used as an indication of the variation to be expected within the borrow pit, and for a simple process control technique during the construction operation.

It is recommended that at least ten RCCD or DCP tests (at least two per square metre) be carried out at points selected in a stratified random pattern when compaction is tested during proof rolling.

3.4 FULL TEST METHODS

3.4.1 Sieve analysis for grading coefficient

SCOPE

In this method, a soil, sand or gravel sample is separated by dry sieving for determination of the grading coefficient and to prepare fine material for the bar linear shrinkage test.

APPARATUS

- Sheet of canvas 1 metre by 1 metre for coning and quartering of the material
- The following test sieves: 37.5 mm, 26.5 mm, 4.75 mm, 2 mm and 0.425 mm with pan and cover
- Balance with pan, accurate to 1 g, to weigh up to 5 kg
- Various pans of 250 to 300 mm diameter and 20 mm deep
- Drying oven (Solar) to maintain a temperature between 105 and 110°C
- Various stiff brushes
- Thermometer (0 to 120°C)

METHOD

Size of sample The size of the test sample should be such that at least 100 g of material passes the 0.425 mm sieve, but not less than 2 kg in all. This should be prepared from a bulk sample of at least 5 kg by coning and quartering on the canvas sheet.

Preparation of the sample Air-dry the sample until it is friable and particles separate with ease. If the sample is still too wet, it should be dried in an oven at a temperature not exceeding 50°C.

Dry sieving Dry sieve the material as follows: shake the material through each sieve in turn, starting at the 37.5 mm sieve, until further shaking results in minimal additional material passing each sieve. The larger particles (> 4.75 mm) should be brushed with a stiff bristle brush to remove all fines adhering to them. Determine the mass of the soil fines (< 0.425 mm) and transfer these to a marked paper bag. It is recommended that they are dried in a solar oven prior to being weighed but, if this will delay the testing, this step may be omitted since the use of air-dried weights will not affect the results unduly.

Determination of the particle size distribution The masses of the individual fractions retained on each sieve should be determined (preferably after being oven-dried but after air-drying if necessary). The masses of these fractions should be determined to the nearest 1 g. Record the masses retained on each sieve and that of the material passing the 0.425 mm sieve.

CALCULATIONS

1 Calculate the total mass of material as the sum of the masses retained on the individual sieves as well as of that passing the 0.425 mm sieve.

2 Calculate the cumulative percentages passing each sieve (by mass of the total dry sample) accurately to the nearest 1 per cent. All results should be normalised to 100 per cent passing the 37.5 mm sieve by multiplying the percentage passing each sieve by the percentage passing the 37.5 mm sieve (P37) divided by 100. If 100 per cent passes the 37.5 mm sieve, this step is not necessary.

3 Calculate the grading coefficient. This is the percentage material passing the 26.5 mm sieve and retained on the 2 mm sieve, multiplied by the percentage passing the 4.75 mm sieve, as follows:

$$GC = (P26 - P2) \times P475/100$$

where *P26 = cumulative percentage passing the 26.5 mm sieve*
P2 = cumulative percentage passing the 2 mm sieve
P475 = cumulative percentage passing the 4.75 mm sieve

3.4.2 Determination of the linear shrinkage of soils

SCOPE

This method covers the determination of the linear shrinkage of soil when it is dried from a moisture content equivalent to the liquid limit to the oven-dry state.

Definition
The linear shrinkage of a soil, for the moisture content equivalent to the liquid limit, is the decrease in one dimension, expressed as a percentage of the original dimension of the soil mass, when the moisture content is reduced from the liquid limit to an oven-dry state.

APPARATUS

- A shrinkage mould made from 10 mm stainless steel bar with internal dimensions of 150 mm ± 0.25 mm long × 10 mm ± 0.25 mm wide × 10 mm ± 0.25 mm deep, and open on two sides (see Figure 3.3)
- A stainless steel plate to fit under the shrinkage mould

- A small thick-bristle paint brush, about 5 mm wide
- Silicone lubricant spray (*e.g.* Q20 or WD 40)
- A spatula with a slightly flexible blade about 100 mm long and 20 mm wide
- A solar drying oven
- A pair of dividers and a millimetre scale
- A standard cup, drop cone and guide-tube for estimating the liquid limit
- A thermometer (0 to 120 °C).

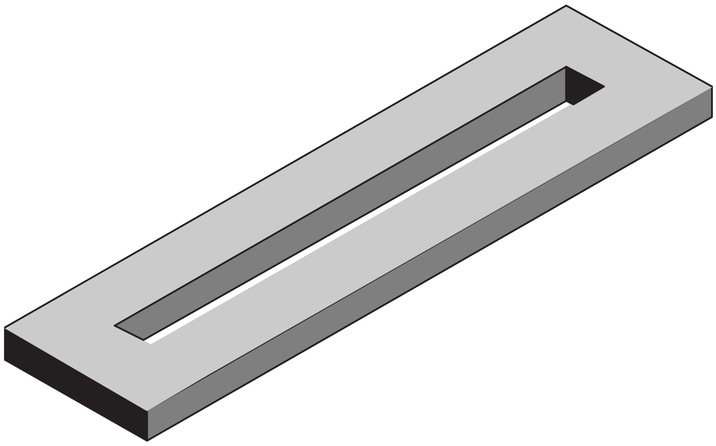

Figure 3.3: Mould for bar linear shrinkage test

METHOD

Waxing the mould The interior of a clean, dry shrinkage mould is sprayed evenly with the silicone lubricant

Filling the mould The moisture content at which the test is carried out must be as close to the liquid limit as practically possible. A simplified drop-cone device based on the British Standard liquid limit method is used to ensure that the moisture content is correct. Sufficient material to fill the cup provided should be mixed up and placed evenly in the cup to a level between 2 and 5 mm below the rim of the cup. The cone should be placed in the guide tube on the surface of the soil in the cup and allowed to penetrate for five seconds. The cone should penetrate to a depth of 20 mm, equivalent to the calibration mark on the cone. If the penetration is below this mark, the material is too dry and additional water is required. The material would then need thorough re-mixing before the penetration test is repeated. If the penetration is too high (*i.e.* the cone sinks into the material to a depth above the calibration mark), the material is too wet and needs to be dried out by mixing in sunlight until repetition of the penetration test gives a result within the defined limits.

The lubricated mould should be placed on the plate provided, and one half should be filled with the moist soil by taking small pieces of soil on the spatula and pressing the soil down against one end of the mould. Then work along the mould until the whole side is filled and the soil forms a diagonal surface from the top of one side to the bottom of the opposite side (see Figure 3.4(a)).

The mould is now turned round and the other portion is filled in the same manner (see Figure 3.4(b)). The hollow along the top of the soil in the mould is now filled so that the soil is raised slightly above the sides of the mould (see Figure 3.4(c)). The excess material is removed by drawing the blade of the spatula once only from one end of the mould to the other. The index finger is pressed down on the blade so that the blade moves along the sides of the mould (see Figure 3.4(d)). During this process the wet soil may pull away from the end of the mould, in which case it should be pushed back gently with the spatula.

NB The soil surface should on no account be smoothed or finished off with a wet spatula.

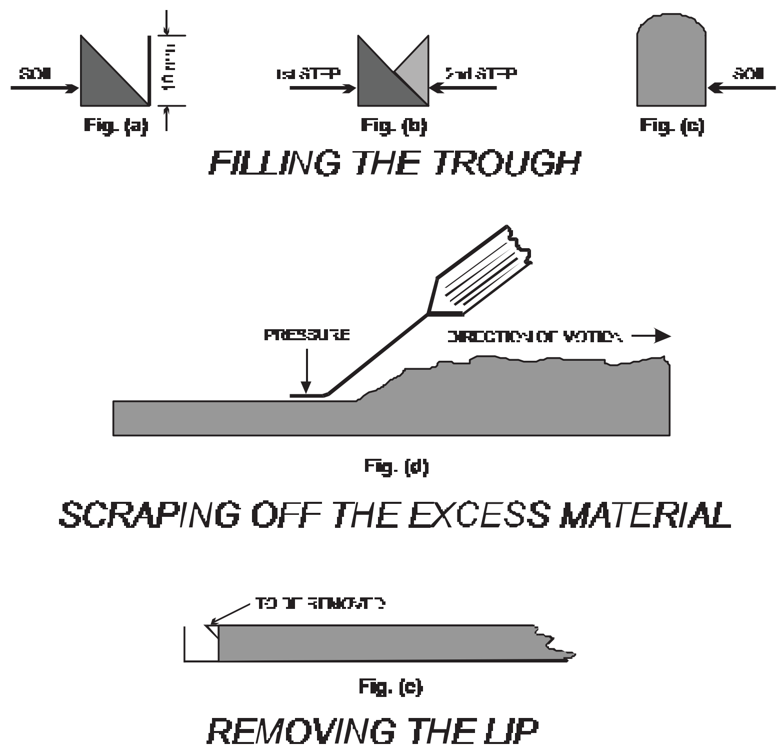

Figure 3.4: Preparation of material for shrinkage test

Drying the wet material The mould with wet material is now placed in the solar oven and dried at a temperature of between 105 and 110°C (the lid may need to be partially opened to maintain a reasonably constant temperature) until no further shrinkage can be detected. As a rule, the material is dried out for 12 hours, although three hours should be sufficient time in the oven. The mould (with the material) is taken out of the oven and allowed to cool in the air.

Measuring the shrinkage It may be found that the ends of the dry soil bar have a slight lip or projecting piece at the top. These lips must be removed by abrading with a sharp, narrow spatula, so that the end of the soil bar is parallel to the end of the mould (see Figure 3.4 (e)). If the soil bar is curved, it should be pressed back into the mould with the fingertips so as to make the top surface as level as possible.

The loose dust and sand removed from the ends, as well as any loose material between cracks, should be emptied out of the mould by carefully inverting it whilst the material is held in position with the fingers. The soil bar is then pressed tightly against one end of the mould. It will be noticed that the soil bar fits better at one end than at the other end. The bar should be pressed tightly against the end at which there is a better fit. The distance between the other end of the soil bar and the respective end of the mould, is measured by means of a good pair of dividers, measuring on a millimetre scale, to the nearest 0.5 mm, and recorded.

CALCULATIONS

The bar linear shrinkage (BLS) is calculated as follows:

$$BLS = LS \times 0.67\ (\%)$$

where LS = linear shrinkage in mm.

NOTES

After being tested, the soil bar should be examined to ensure that the corners of the mould were filled properly and that no air pockets were contained in the soil bar. If there are air pockets, the test should be repeated.

3.4.3 The determination of the Treton impact value of aggregate

SCOPE

The Treton value is an indication of the resistance of aggregate to impact. The aggregate is subjected to ten blows of a falling hammer and the resulting disintegration is measured in terms of the quantity passing the 2 mm sieve, which is then expressed as a percentage of the mass of the test sample. This is called the Treton value.

APPARATUS

- A Treton apparatus consisting of a base plate, anvil, cylinder, and a hammer weighing 15 kg ± 50 g (see Figure 3.5). The base plate should be placed on a firm concrete block.
- The following test sieves, all 200 mm in diameter: 19.0 mm, 16.0 mm and 2 mm. The bigger sieves should be made of perforated plate and the 2 mm sieve of wire mesh.
- A balance to weigh up to 200 g, accurate to 1 g.

METHOD

From the field sample, screen out a sufficient quantity (at least 200 g) of the fraction between 19 mm and 16 mm (see Note (i)). Select a sample of 15 to 20 of the most cubical pieces, so that their total mass (in grams) will be as close as possible to 50 times the relative density of the aggregate in grams (it is not necessary to determine the relative density. An estimate will be satisfactory (2.65 for granitic and sedimentary materials and 2.9 for dark basaltic and metamorphic materials)). Weigh the sample accurately to 1 g, and place the particles as evenly spaced as possible on the anvil in such a manner that their tops are approximately in the same horizontal plane.

Place the cylinder over the anvil and tighten the clamp screws. Place the hammer in the cylinder so that the top of the hammer is level with the top of the cylinder and let it drop ten times from this position.

Remove the cylinder, and sieve all the aggregate on the anvil and base plate thoroughly through a 2 mm sieve. Weigh the aggregate retained on the sieve to the nearest 1 g, and record the mass. The test should be carried out in triplicate (see Note (ii)).

CALCULATIONS

Calculate the Treton value to the first decimal place as follows and report to the nearest whole number:

Treton value = $(A - B)/A \times 100$

where A = *the mass of the stone particles before tamping (g)*
B = *the mass of the stone particles retained on the 2 mm sieve after tamping (g).*

NOTES

(i) If the aggregate is noticeably variable as regards type or hardness, each type should be tested and reported separately. In this case an estimate should be made of the percentage of each type and a weighted average determined.

(ii) The Treton value, as reported, should be the average of three determinations. If any individual result differs from the others by more than five units, further tests should be carried out.

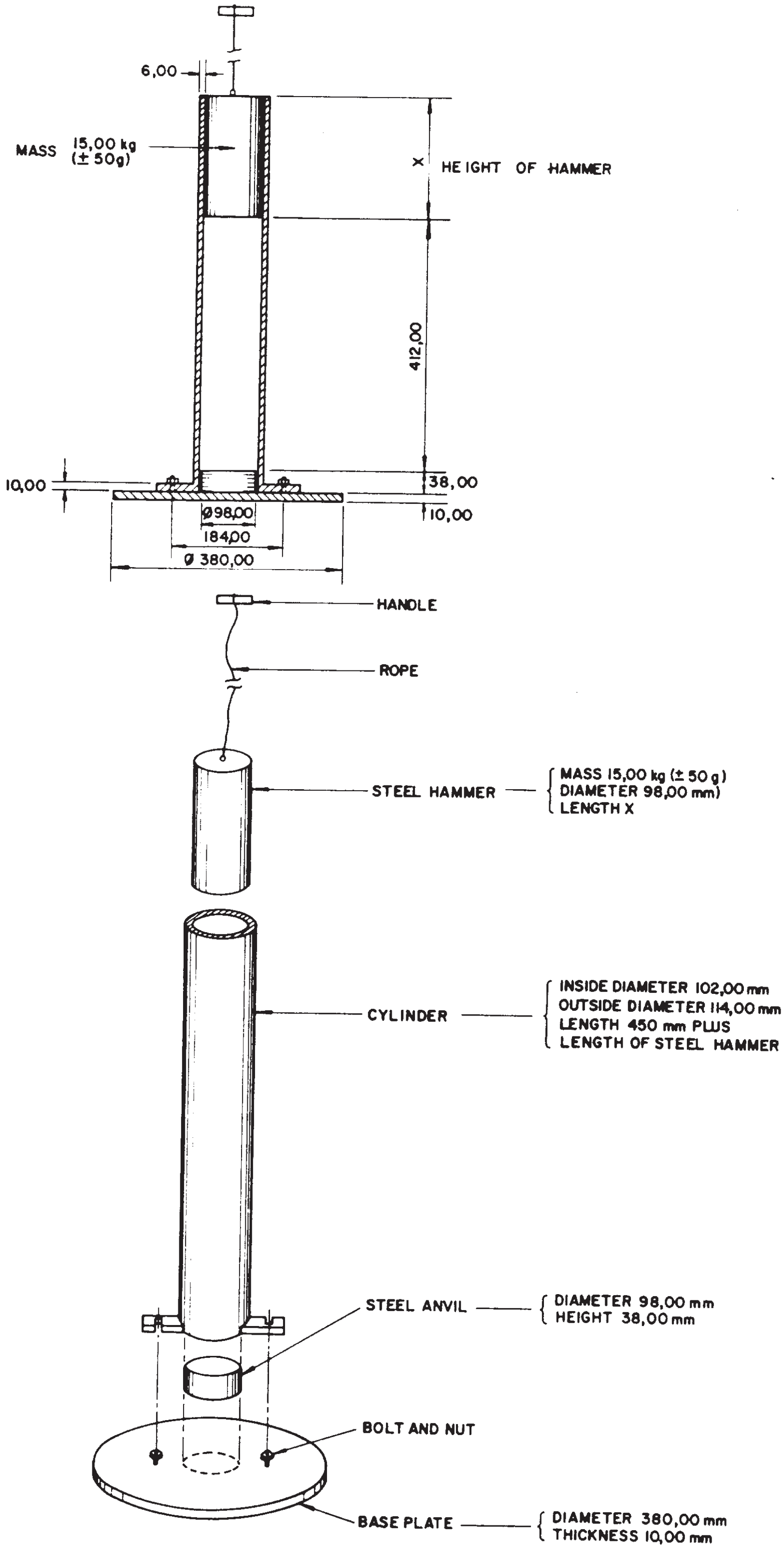

Figure 3.5: Treton apparatus

4 Construction quality assurance testing

4.1 Material testing and control

The properties of the material should be tested on a regular basis during construction to ensure that they do not differ from the accepted specification. For control testing purposes, only the grading coefficient and shrinkage product need be tested.

This testing shall be done daily to ensure that the material to be used for the following job lot complies with the specifications. It is recommended that samples of the material to be used the next day (or for the next job lot if a weekend follows) be taken during the morning and tested so that the material can be approved first thing in the morning before use. The test techniques are such that this is possible. The individual results of the borrow pit testing should be plotted on Figure 2.1 with the mean and standard deviations of the two parameters which can be used to define a rectangle. At least 90 per cent of the routine daily test results should plot in this rectangle as work in the borrow pit progresses. The test results should be plotted on this figure on an ongoing basis. If there is a trend to move out of the rectangle towards the limits of the E1/E2 block (in Figure 2.1), this would be indicative of a change in the material properties. Additional testing should then be carried out to determine the cause of this and to identify remedial action, *e.g.* blending of different materials, redefinition of the boundaries of the borrow pit, or adjustment of the depth of excavation, *etc.*

4.2 Construction quality assurance

A number of factors should be controlled during construction. These include:

- Moisture content
- Thickness
- Compaction
- General finish.

4.2.1 Moisture content

One of the principal factors in the construction process, and which affects the final compaction, is the moisture content. In most soil materials the natural variation in optimum moisture content (OMC) is wider than the limits around OMC permitted for successful compaction. In addition, the actual process of adding and mixing water to soil materials, particularly in labour-based projects, often leads to significant variation of the moisture content within the material. In addition, most moisture content determinations are slow (except for nuclear methods, but these are often unreliable for moisture contents of natural gravels) and the results are frequently only available after compaction is completed. For this reason, the manual control of the moisture after laboratory calibration of the "feel" of the material at various moisture contents at and around optimum is considered the most practical and effective solution.

In most cases, the test techniques for moisture content render the results practically meaningless in the context of labour-based construction. The process of moistening the material, (whether this is done in the borrow-pit or on the road) is not discussed in this report.

The control of moisture during construction should be carried out visually by squeezing a sample of the material as tightly as possible in the hand. The material should be moist enough to stick together when squeezed without any visible sign of free water on the surface. If the material disintegrates, it is too dry for compaction. If free water is ejected or if the soil sticks to the hand, it is too wet. If the "sausage" formed by squeezing in the hand is squeezed diametrically between the thumb and forefinger, it should break with some crumbling. It should not break by deformation under the finger pressure, nor should there be excessive crumbling. It should be noted that non-cohesive soils behave differently, but that all materials for wearing courses should have some cohesion. The above technique is considered most practical and suitable for the purpose. If possible, this method should be practised in the laboratory with material at various known moisture contents, and correlated with the laboratory determined optimum moisture content to "get the feel" prior to commencement of compaction.

It is currently difficult to correct the field strength for any deviation from the expected moisture content at the time of testing. The following approximate model is based on the combination of various parameters (soaked CBR (CBR_s) and optimum moisture content (OMC)) and models. It has been developed to assist with evaluating whether the results are in the right range for the DCP and RCCD penetration rates (DN_c and $RCCD_c$) immediately after compaction at compaction moisture content (CMC):

$$DN_c = 0 \cdot 144 \left(e^{-1 \cdot 33 \frac{CMC}{OMC}} \right) \left(CBR_s^{0 \cdot 46} \right)^{-0 \cdot 787}$$

$$RCCD_c = 0 \cdot 0735 \left(e^{-1 \cdot 33 \frac{CMC}{OMC}} \right) \left(CBR_s^{0 \cdot 46} \right)^{-0 \cdot 775}$$

4.2.2 Thickness

It is important that the thickness of the material be closely controlled. This should be controlled prior to compaction, with allowance for the bulking factor. Spreading of the material should be as consistent as possible to ensure that all material is placed at a similar loose density. The bulking factor is usually between 25 and 35 per cent, depending on the gradation of the material and on the compaction effort, but this should be determined accurately during the initial proof rolling of the material.

Control of the thickness during construction is carried out by inserting a calibrated probe (Figure 4.1) through the uncompacted material to confirm that the thickness prior to compaction is equivalent to the required final layer thickness plus a correction for bulking. The bulking factor should be determined during the proof rolling by measurement of the thickness before and after rolling. It can also be estimated by comparison of the mass of a known volume (best done in a large measuring cylinder) with the maximum dry density of the material determined in the laboratory. In most cases a bulking value of 30 to 35 per cent may be assumed. One advantage of knowing the bulking factor accurately is that this can be used to ensure that adequate compaction has been achieved by monitoring the initial and final thickness of the layer.

Thickness before compaction = design thickness × (1 + BF (%) / 100)

where BF is the percentage bulking factor for the material.

4.2.3 Compaction

The compaction achieved in the field is arguably the most important aspect of the construction process. It is neither economically nor practically possible to determine sufficient densities on labour-based projects for construction quality control, to take into account the natural variability of the material. It is thus recommended that a simple device such as the Rapid Compaction Control Device (RCCD) or DCP be used for this purpose. Both tests are quick and repeatable and many tests can be done at little cost.

Should the test results show up areas which are unacceptable, the reasons for this should be investigated. Poor results can be attributed to the use of material that is too wet, material that has not received adequate compaction effort, or to the presence of a pocket of poor quality material. The actual cause should be identified and corrective action taken. This may involve scarifying and drying out prior to recompaction if the material is too wet, additional compaction if necessary, or replacement of poor material where appropriate.

A simple method of sand replacement density determination can be carried out, but it is difficult to relate this to a standard for the evaluation of relative compaction in highly variable natural materials. To determine relative compaction, the density in the road should be compared with the actual laboratory maximum dry density (MDD) for that material. This would require an MDD test to be done on the identical material tested *in situ*. This is neither practical nor economical.

4.2.4 Visual inspection

It is imperative that supervisors are trained to carry out a comprehensive visual inspection of the completed layer prior to excessive drying out of the material. This inspection should be carried out during the latter part of the shift prior to demobilisation of staff and plant for the day. It should be extremely thorough and should cover the total job lot completed in the shift. During this inspection, large stones, excessively moist areas, poorly compacted areas, bumps and depressions, areas of thin material, material segregation, *etc*. should be located and the appropriate remedial action taken.

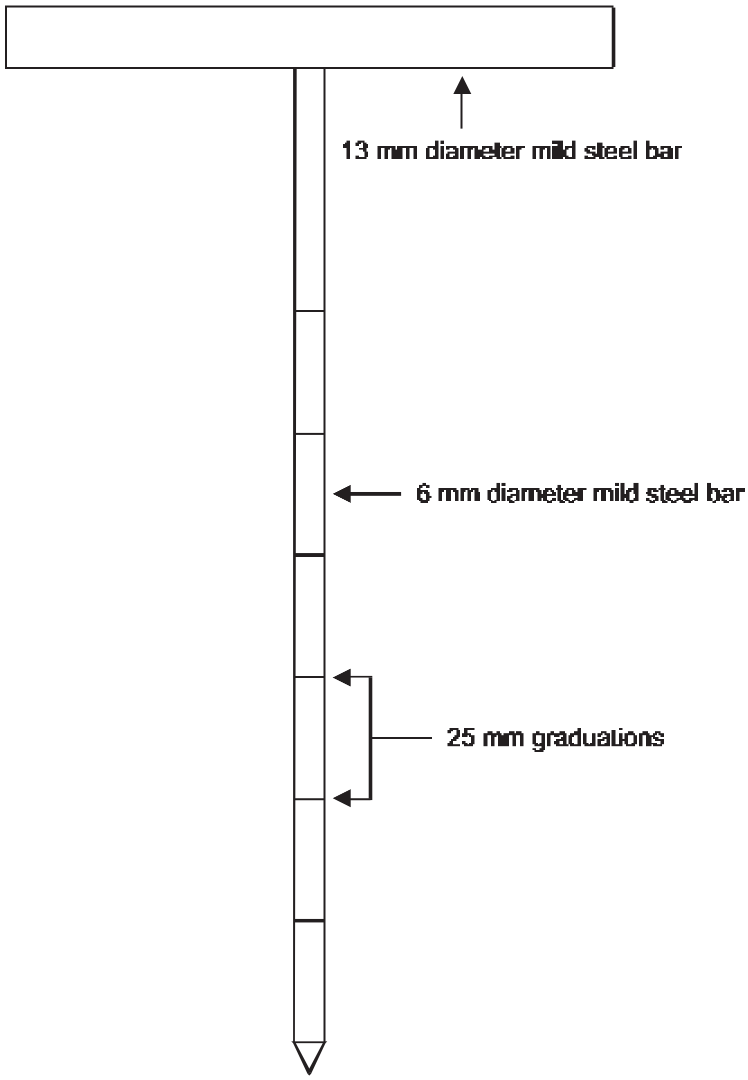

Figure 4.1: Thickness probe

5 Thickness design

Considerable debate has been held and continues to be held regarding the need for a full structural design for unsealed roads. The fact is that wearing courses seldom fail as a result of punching into the subgrade (*i.e.* shear failure of either the wearing course or subgrade material). Failure is typically either the result of continued slippage of the vehicle tyre against the soil when a high moisture content prevails (lack of frictional resistance leading to plastic failure) resulting in settlement of the tyre into the material; or shear failure of the upper portion of the wearing course with lateral displacement. The former situation is confirmed by the fitting of chains to a vehicle wheel to increase this friction which restores passability to the vehicle without excessive additional deformation of the wearing course. The latter situation is manifested by ruts in the wearing course with lateral displacement of the wearing course material. Only when the wearing course becomes excessively thin does shearing of the underlying subgrade material become possible when this is soaked.

The thickness design should take into account the fact that most deformation which may occur in unpaved roads is rectified during routine maintenance, and that with time, all roads lose material through environmental factors and traffic wear. It is thus necessary in terms of thickness design to allow for these two aspects and for a minimum remaining thickness to prevent the subgrade being exposed at the surface.

The rate of gravel loss under traffic is typically greater than that resulting from environmental influences and is mostly related to traffic volume. Other factors such as climate, material properties and, to a lesser extent, geometrics affect the rate of gravel loss. As for most roads, the primary design criterion should be the life before major maintenance (regravelling) is required.

For roads carrying less than 100 vehicles per day (vpd) on a subgrade material with a minimum soaked CBR of 3 per cent, a wearing course 150 mm thick of material with a minimum soaked CBR value of 15 will provide an adequate structure. In areas where the subgrade (or wearing course) is likely to become soaked (*i.e.* to be under standing water for more than 24 hours) resulting in the CBR of the subgrade and base decreasing to less than 3 or 15 per cent respectively, the application of two layers of wearing course quality material (each 150 mm thick) is recommended. The standard thickness of 150 mm of material as proposed in the specification has been shown to be adequate to resist excessive deformation under most conditions, even persistent rainfall, provided the shape of the road is such that excessive ponding on the road structure cannot occur. The necessity for proper maintenance of unsealed roads cannot be overemphasised. Extended periods of poor or no maintenance will always result in significant deterioration of the road, loss of gravel, and in extreme difficulty in restoring the condition of the road.

In addition to the subgrade being compacted to a high density to provide a platform for compaction of the wearing course, it is equally important that the subgrade density is such that loss of wearing course by being punched into the subgrade or by taking up significant rutting which may occur in the subgrade, is minimised. This type of "gravel loss" is typically made up during routine maintenance but results in a premature need for regravelling.

Thinner layers (not less than 100 mm) can be placed to reduce the cost of material and construction, but in the long term this is generally not cost-effective. Once about 50 to 75 mm of material has been lost, the wearing course will need to be replaced, resulting in a premature regravelling operation. Layers thicker than 150 mm are not recommended as it then becomes difficult to obtain adequate compaction, particularly through the full depth, but also at the surface. It should be remembered that, as the layer of uncompacted material becomes thicker, the distance from the firm platform required to compact against becomes greater and more energy is absorbed by the loose material. This is particularly relevant when light rollers are utilised.

If the traffic is likely to be higher than 100 vehicles per day, including more than ten heavies, it may be necessary to increase the thickness of the wearing course over those areas with low subgrade strengths (soaked CBR of less than 5 per cent); or else to include a 150 mm thick selected layer with a minimum soaked CBR of 10 per cent. Recommended thicknesses and wearing course material strengths are summarised in Table 5.1.

Table 5.1: Recommended thicknesses and material strengths for different subgrade and traffic conditions

Subgrade CBR	Traffic (vpd)	Layers/thickness	Min. soaked CBR
> 3	< 100	150 wearing course	15
< 3	< 100	150 wearing course 150 selected layer	15 5
< 5	> 100 (> 10% heavy)	150 wearing course 150 sub-base	15 10

It is possible to scientifically and mechanistically design the layer thickness in which aspects, such as subgrade strength and stiffness, wearing course material strength and stiffness, annual gravel loss through traffic and environmental factors, traffic compaction, *etc*., are taken into account. However, these all require additional testing and environmental data, much of which are not available in remote locations. In general, the design proposed above will, with routine maintenance, provide a road which will last between 5 to 10 years without requiring major maintenance other than periodic grader blading.

6 Conclusions and recommendations

6.1 Conclusions

The construction of labour-based unsealed roads, if done with the correct materials and appropriate quality assurance, will result in roads which will perform as well as those built conventionally using plant-based methods. Particular attention should be paid to material selection and control, as well as to construction control.

6.2 Recommendations

It is recommended that these guidelines be implemented and augmented where necessary. Although every project will have certain unique characteristics and problems, these guidelines are seen as a generally applicable solution for routine labour-based construction.

As a consequence of this assignment, Transportek is currently producing a labour-based construction test kit which will include all the necessary testing and analysis equipment to carry out the testing recommended in this Brief. Copies of the test methods to meet the requirements of these guidelines will also be included (a summary of the contents of the kit is provided in Annex B). It is recommended that this be presented at appropriate venues and seminars.

As these techniques are applied, any problems which may be encountered in practice, and improvements in the methods identified with experience, can be incorporated into the Guidelines.

7 References

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Annex A: Guideline document

WHERE THERE IS NO SOILS LAB!

Material selection and tests for labour-based gravel roads

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1 Introduction

These guidelines describe a step-by-step method of evaluating borrow materials for use as the wearing course in labour-based unsealed roads, and for ensuring that the quality of the construction is appropriate.

The guidelines assume that all the required testing apparatus has been provided for the project and that potable water and normal day-to-day requirements such as batteries are available. They also assume that supervisors have been given adequate training in the use of these guidelines.

It will also be noted that the test methods given in this section are a simplification of those described in the main document, for ease of use in the field. The effect of this simplification is considered to be insignificant in terms of the repeatability and reproducibility of the tests and their implications on the use of the materials.

2 Material properties

2.1 MATERIAL GRADING

In order to achieve good particle interlock from the wearing course material, a good particle size distribution is necessary. This is defined by the Grading Coefficient. In addition, it is important to limit the quantity of large stones in the material by specifying the Oversize Index. These parameters are simply determined as described below.

2.1.1 Determination of the grading coefficient and oversize index

SCOPE

In this method, a soil, sand or gravel sample is separated by dry sieving for determination of the grading coefficient and to prepare fine material for the bar linear shrinkage test.

APPARATUS

- Sheet of canvas 1 metre by 1 metre for coning and quartering material
- The following test sieves: 37.5 mm, 26.5 mm, 4.75 mm, 2 mm and 0.425 mm with pan and cover
- Balance with pan, accurate to 1 g, to weigh up to 5 kg
- Various pans of 250 to 300 mm diameter and 20 mm deep
- Drying oven (Solar) to maintain a temperature between 105 and 110°C
- Various stiff brushes
- Thermometer (0 to 120 °C)

METHOD

STEP 1 A test sample such that at least 100 g of material passes the 0.425 mm sieve, but not less than 2 kg in total, should be prepared from a bulk sample of at least 5 kg by coning and quartering using a canvas sheet.

STEP 2 Air dry the sample until it is friable and the particles can be separated with ease. If the sample is still too wet, it should be dried in a solar oven at a temperature not exceeding 50°C.

STEP 3 Dry sieve the material as follows: shake the material through each sieve (37.5, 26.5, 4.75, 2.0 and 0.425 mm) in turn, starting at the 37.5 mm sieve, until further shaking results in minimal additional material passing the sieve. Brush the larger particles (> 4.75 mm) with a stiff bristle brush to remove all fines adhering to these. Determine the mass of the soil fines (< 0.425 mm) and transfer them to a marked paper bag for subsequent testing. (It is recommended that they be dried in a solar oven prior to being weighed, but if this will delay testing, this step may be omitted, since the use of air-dried weights will not affect the results unduly.)

STEP 4 The masses of the individual fractions retained on each sieve should be determined (preferably after being oven-dried but after air-drying if necessary). The masses should be determined to the nearest 1 g. Record the masses retained on each sieve and that passing the 0.425 mm sieve. A form for recording and calculating the results is included at the end of this document.

RESULTS

Calculate the total mass of material as the sum of the fractions retained on the individual sieves as well as that passing the 0.425 mm sieve. Determine the percentage of each of these as a percentage of the total mass of (dry) material tested.

Calculate the cumulative percentages passing each sieve to the nearest 1 per cent by summing the percentage passing each sieve, starting from the finest sieve. All results should first be corrected for the percentage retained on the 37.5 mm sieve by multiplying the percentage passing each sieve by the percentage passing the 37.5 mm sieve (P37) divided by 100. If all the material passes the 37.5 mm sieve, this step is not necessary.

Calculate the **grading coefficient** as the percentage material passing the 26.5 mm sieve and retained on the 2 mm sieve (P26 and P2 respectively) multiplied by the percentage passing the 4.75 mm sieve (P475) using the following formula:

$$GC = (P26 - P2) \times P475/100$$

The **oversize index** is defined as the percentage of the total material retained on the 37.5 mm sieve.

INTERPRETATION

The limits for the grading coefficient are shown in Figure A.3 and should be between 16 and 34. A maximum oversize index of 5 per cent is permitted to retain a good riding quality over time.

2.2 Material cohesion

In order to minimise the loosening of the surfacing material and the formation of corrugations, it is necessary that the materials should have some plasticity. This is determined using the bar linear shrinkage test described below by measuring the linear shrinkage of a soil dried from a moisture content equivalent to the liquid limit to the oven-dry state.

2.2.1 Determination of the linear shrinkage of soils

SCOPE

This method covers the determination of the linear shrinkage of soil when it is dried from a moisture content equivalent to the liquid limit to the oven-dry state.

Definition *The linear shrinkage of a soil for the moisture content equivalent to the liquid limit, is the decrease in one dimension, expressed as a percentage of the original dimension of the soil mass, when the moisture content is reduced from the liquid limit to an oven-dry state.*

APPARATUS

- A shrinkage mould made from 10 mm stainless steel bar with internal dimensions of 150 mm ± 0.25 mm long × 10 mm ± 0.25 mm wide × 10 mm ± 0.25 mm deep, and open on two sides (see Figure 3.3)
- A stainless steel plate to fit under the shrinkage mould
- A small thick-bristle paint brush, about 5 mm wide
- Silicone lubricant spray (*e.g.* Q20 or WD 40)
- A spatula with a slightly flexible blade about 100 mm long and 20 mm wide
- A solar drying oven capable of maintaining a temperature of 105 to 110°C
- A pair of dividers and a millimetre scale
- A standard cup, drop cone and guide-tube for estimating the liquid limit
- A thermometer (0 to 120 °C).

METHOD

STEP 1 The interior of a clean, dry shrinkage mould is sprayed evenly with the silicone lubricant.

STEP 2 The fines (*i.e.* the fraction passing the 0.425 mm sieve) saved during the grading analysis are used for this test. Add water to the fines and mix thoroughly until the consistency is at the liquid limit. The drop-cone device is used to ensure that this initial moisture content is correct. Sufficient material to fill the cup provided should be mixed up and placed evenly in the cup to a level between 2 and 5 mm below the rim of the cup. The cone should be placed in the guide tube on the surface of the soil in the cup and allowed to penetrate for 5 seconds. The cone should penetrate to a depth of 20 mm, equivalent to the calibration mark on the cone. If the penetration is below this mark, the material is too dry and additional water is required. The material would then need thorough re-mixing before the penetration test is repeated. If the penetration is too high (*i.e.* the cone sinks into the material to a depth above the calibration mark), the material is too wet and needs to be dried out by additional mixing until repetition of the penetration test gives a result within the defined limits.

STEP 3 The lubricated mould should be placed on the plate provided and one half should be filled with the moist soil by taking small pieces of soil on the spatula and pressing the soil down against one end of the mould and working along the mould until the whole side is filled and the soil forms a diagonal surface from the top of one side to the bottom of the opposite side (see Figure A1(a)).

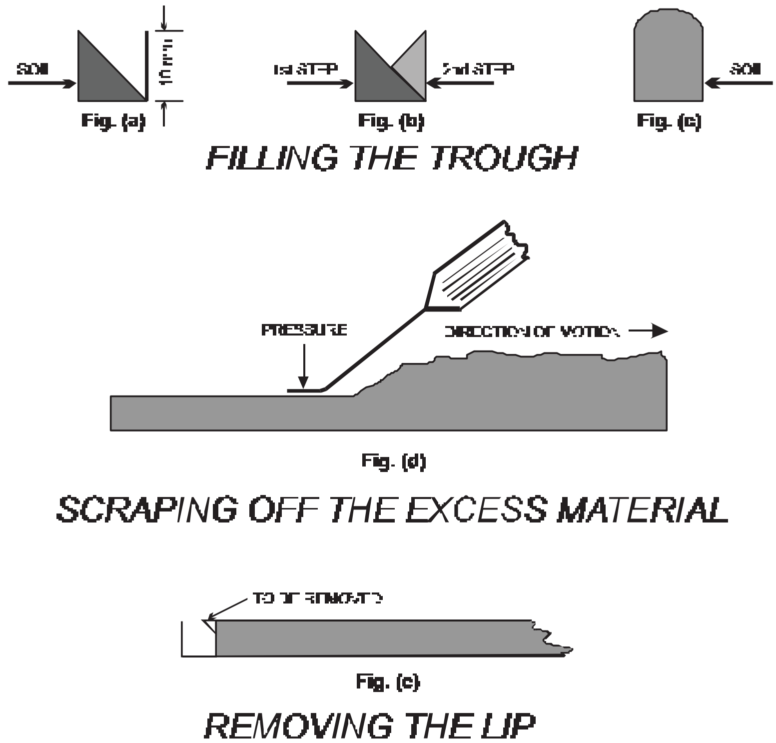


Figure A1: Preparation of material for the linear shrinkage test

The mould is now turned round and the other portion is filled in the same manner (see Figure A1 (b)). The hollow along the top of the soil in the mould is now filled so that the soil is raised slightly above the sides of the mould (see Figure A1 (c)). The excess material is removed by drawing the blade of the spatula once only from one end of the mould to the other. The index finger is pressed down on the blade so that the blade moves along the sides of the mould (see Figure A1 (d)). During this process the wet soil may pull away from the end of the mould, in which case it should be pushed back gently with the spatula. *On no account should the surface of the soil be smoothed or finished off with a wet spatula.*

STEP 4 The filled mould is now placed in the drying oven and dried at a temperature of between 105 and 110°C until no further shrinkage can be detected. As a rule, the material is dried out overnight, although three hours in the oven should be sufficient. The mould with the material is taken out of the oven and allowed to cool.

STEP 5 It may be found that the ends of the dry soil bar have a slight lip or projecting piece at the top. These lips should be removed by abrading with a sharp, narrow spatula, so that the end of the soil bar is parallel to the end of the mould (see Figure A1 (e)). If the soil bar is curved, it should be pressed back into the mould with the fingertips so as to make the top surface as level as possible.

The loose dust and sand, removed from the ends, as well as any loose material between cracks, should be emptied out of the mould by carefully inverting the mould whilst the material is held in position with the fingers. The soil bar is then pressed tightly against one end of the mould. It may be noticed that the soil bar fits better at one end than at the other end. The bar should be pressed tightly against the end at which there is a better fit. The gap between the soil bar and the end of the mould is measured by means of a good pair of dividers, measuring on a millimetre scale, to the nearest 0.5 mm and recorded on the form included with this document.

RESULTS

The bar linear shrinkage (BLS) is calculated from the measured shrinkage LS (in mm) as follows:

$$BLS = LS \times 0.67\ (\%)$$

NOTE

After the test, the soil bar should be examined to ensure that the corners of the mould were filled properly and that no air pockets were contained in the soil bar. If air pockets were contained, the material should be tested again.

INTERPRETATION

A value for the shrinkage product in excess of 100 is required but it should not exceed 365, otherwise slipperiness will result when the material is wet.

2.3 MATERIAL STRENGTH

In order to support the loads applied by vehicles, the material should have an adequate strength at the density at which it will perform in the field. The soaked California Bearing Ratio is typically specified for this parameter but its measurement requires bulky, expensive equipment. For labour-based projects, the following simple but equivalent method is, however, recommended. For this, a small section of road equivalent to that of the full scale construction is processed. This should be five metres long and one metre wide (or the width of the roller if wider than one metre), with one metre of material surrounding the central section to be tested.

2.3.1 The determination of the compacted strength of material

SCOPE

The objective of this procedure is to ensure that the compacted strength of the borrowed material as placed in the field complies with the acceptance testing carried out on the borrow material. Secondary objectives of this procedure are to identify the limit to be used for control testing after compaction and to identify the number of roller passes for optimum compaction.

APPARATUS

- Compactor equivalent to that proposed for use
- DCP or RCCD apparatus.

METHOD

STEP 1 A sufficient quantity of the proposed material (between 4.0 and 4.5 cubic metres) should be dumped on a section of the proposed subgrade prepared to the same standard as that of the proposed road. Water should be added to bring this material to its estimated optimum moisture content.

STEP 2 The moist material should be spread to a thickness which will provide a compacted thickness equivalent to the design thickness of the layer.

STEP 3 Using the compaction method and plant which will be used during full construction, a complete roller pass should be given to the layer. The strength of the layer is then determined using a DCP or RCCD and the results recorded on the field test data form provided.

STEP 4 Repeat Step 3, plotting the penetration rate obtained from testing against the number of roller passes until no further strengthening of the material occurs (*i.e.* until the measured penetration rate reaches a minimum). This identifies both the number of passes above which no additional benefit from rolling is obtained, and the final maximum strength of the material at compaction moisture content.

INTERPRETATION

The maximum strength of the material should comply with the requirements given in Table A1 below:

Table A1: Penetration rates of DCP and RCCD and equivalent soaked CBR (tested at OMC)

Apparatus	Penetration rate	Penetration (3 blows)	Penetration (20 blows)	Equivalent soaked CBR
	(mm/blow)	*(mm)*	*(mm)*	*(%)*
DCP	• 5	• 15	• 100	15

RCCD	• 9	• 27	—	15

STEP 6

If the above requirements are not met, either the material has inadequate strength to resist deformation or to avoid becoming slippery when wet, or additional compaction energy is required in order to achieve a higher density and increase the strength of the material.

A minimum soaked equivalent CBR strength of 15 per cent is required for acceptable passability and trafficability. This is related to the DCP and RCCD results as indicated in the table above.

2.4 AGGREGATE STRENGTH

In order to ensure that the aggregate particle strength is sufficient to avoid this component of the material breaking down excessively under rolling and traffic, a simple strength test (Treton Impact Value test (Figure A2)) is recommended, as described below. This test also identifies those materials which are too hard to break down and which could result in excessive stoniness of the road. A sample of the aggregate is subjected to ten blows of a falling hammer and the resulting disintegration is measured in terms of the quantity passing the 2 mm sieve.

2.4.1 Determination of the Treton impact value of aggregate

SCOPE

The Treton value is an indication of the resistance of aggregate to impact. The aggregate is subjected to ten blows of a falling hammer and the resulting disintegration is measured in terms of the quantity passing the 2 mm sieve, which is then expressed as a percentage of the mass of the test sample. This is called the Treton value.

APPARATUS

- A Treton apparatus consisting of a base plate, anvil, cylinder and a hammer weighing 15 kg ± 50 g (Figure A2). The base plate should be placed on a firm concrete block.
- The following test sieves, 200 mm in diameter: 19.0 mm, 16.0 mm and 2 mm. The bigger sieves should be made of perforated plate and the 2 mm sieve of wire mesh.
- A balance to weigh up to 200 g, accurate to 1 g.

METHOD

STEP 1 From the field sample, screen out a sufficient quantity (at least 200 g) of the – 19.0 + 16.0 mm fraction. If the aggregate is noticeably variable as regards type or hardness, each type should be tested and reported separately. In this case an estimate should be made of the percentage of each type.

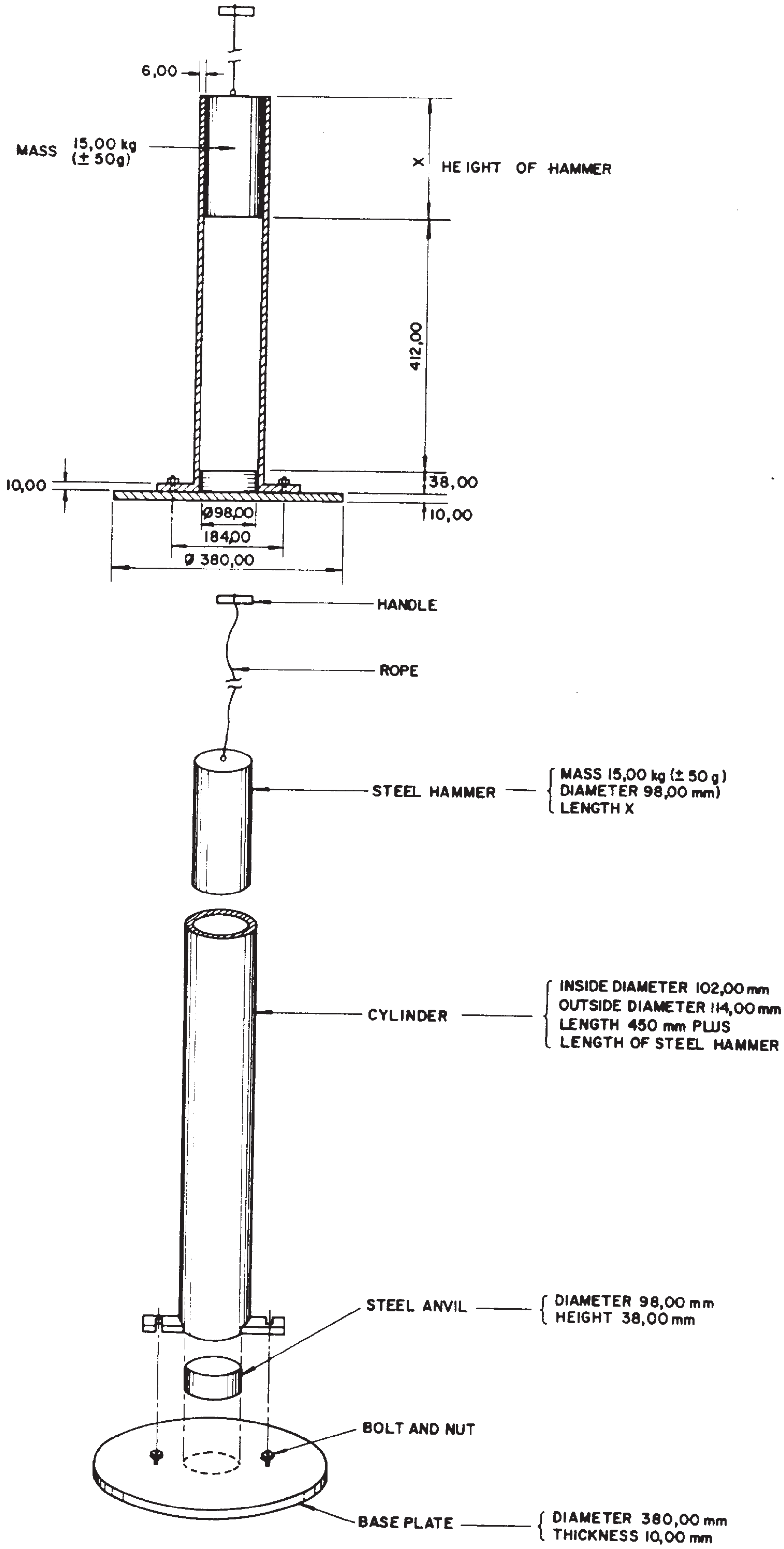


Figure A2: Treton apparatus

STEP 2 Select 15 to 20 of the most cubical pieces. Weigh the aggregate pieces to an accuracy of 1 g, and place them as evenly spaced as possible on the anvil in such a manner that their tops are approximately in the same horizontal plane.

STEP 3 Place the cylinder over the anvil and tighten the clamp screws. Place the hammer in the cylinder so that the top of the hammer is level with the top of the cylinder and let it drop ten times from this position.

STEP 4 Remove the cylinder and sieve all the aggregate on the anvil and base plate thoroughly through a 2 mm sieve. Weigh the aggregate retained on the sieve to the nearest 1 g, and record the mass. The test should be carried out in triplicate. (If any individual result differs from the others by more than five units, further tests should be carried out.)

RESULTS

Calculate the Treton value to the first decimal place as follows:

Treton value = $(A - B)/A \times 100$

where A = *the total mass of the stone particles before tamping (g)*
B = *the total mass of the stone particles retained on the 2 mm sieve after tamping (g).*

Report the value to the nearest whole number.

INTERPRETATION

Recommended Treton impact values should lie between 20 and 65. Materials with values less than 20 will be too hard and cause excessive roughness whilst those with values higher than 65 will be too soft and break down under traffic.

2.5 APPLICATION

The results obtained from the grading and linear shrinkage testing are evaluated using Figure A-3

For the best performance, the results should plot in zone E1 or E2 of the diagram. The potential problems associated with materials plotting in the other zones are identified in the zones. To reduce dust, materials should preferably plot in the E2 zone.

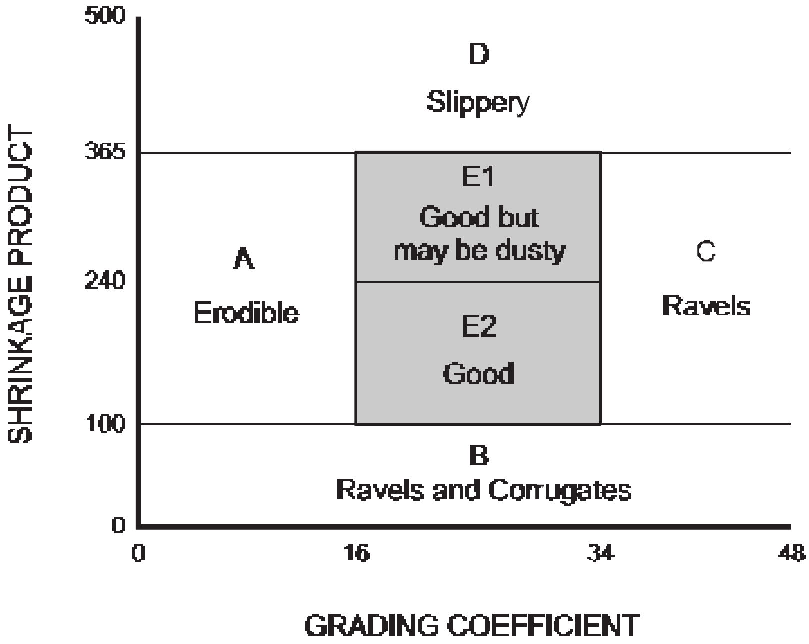


Figure A3: Relationship between Grading Coefficient, Shrinkage Product and performance

3 Quality assurance during construction

3.1 Material testing and control

The quality of the borrow material should be controlled on a regular basis during construction to ensure that its properties do not differ from the accepted specification. For this purpose, only the grading coefficient and shrinkage product need be tested.

This testing is required on a routine basis to ensure that the material to be used for the following job lots comply with the specifications. It is recommended that samples of the material to be used should be taken two or three days prior to that material being used and tested so that the material can be approved at least the day before it is processed in the borrow pit, loaded and hauled. The test techniques described in the section on Material Properties above are such that this is possible.

The individual results of the borrow pit testing should be plotted on Figure A3. The test results should be plotted on this figure on an ongoing basis and if there is a trend to move out of Zone E, it is indicative of a change in the material properties. Additional testing should be carried out to determine the cause of this and to identify remedial action, *e.g.* blending of material, redefining of boundaries of the borrow pit, adjustment of depth of excavation, *etc*.

3.2 Construction quality assurance

A number of factors should be controlled during construction. These are:

- Moisture content
- Thickness
- Compaction
- General finish.

3.2.1 Moisture content

It is very difficult to get a high degree of compaction if the moisture content is not close to the Optimum Moisture Content (OMC) for that material. As it is difficult to get an accurate, usable determination of the moisture content quickly, the visual determination of this in the field is recommended.

The control of moisture during construction must be carried out by squeezing a sample of the material as tightly as possible in the hand.

The material should be moist enough to stick together when squeezed, with no visible sign of free water on the surface.

If the material disintegrates, it is too dry for compaction. If free water is ejected or if the soil sticks to the hand, it is too wet.

If the “sausage” formed by squeezing in the hand is squeezed diametrically between the thumb and forefinger, it should break with some crumbling. It should not break by deformation under the finger pressure, nor should there be excessive crumbling.

3.2.2 Thickness

The thickness of the layer should be closely controlled prior to compaction, with allowance for the bulking factor. Spreading of the material should be as consistent as possible to ensure that all material is placed at a similar loose density.

To produce a 150 mm thick compacted layer, 190 to 200 mm of loose material is typically required.

Control of the thickness during construction is carried out by inserting a calibrated probe (Figure A4) through the uncompacted material to confirm that the thickness prior to compaction is within the required limits.

This should be checked in at least 25 locations and, where the thickness is deficient, more material should be added.

3.2.3 Compaction

The compaction achieved in the field is the most important aspect of the construction process. It is recommended that a simple device such as the Rapid Compaction Control Device (RCCD) or DCP is used for this purpose. Both tests are quick and repeatable, and many tests can be done rapidly and at little cost.

Not less than six tests should be done on any job lot and in no case should the penetration rate exceed the specified maximum permissible penetration rate.

Should the tests show up areas which are unacceptable, the reasons for this should be investigated. Poor results can be attributed to the material being too wet, material that has not received adequate compaction effort, or to a pocket of poor quality material.

The actual cause needs to be identified and corrective action taken. This may involve scarifying and drying out prior to recompaction if the material is too wet, additional compaction if necessary, or replacement of poor material where appropriate.

3.2.4 Visual inspection

Supervisors should be trained to carry out a comprehensive visual inspection of the completed layer prior to excessive drying out of the material. This inspection should be carried out during the latter part of the shift prior to demobilisation of staff and plant for the day, should be extremely thorough and should cover the total job lot completed in the shift. During this inspection the following should be evaluated:

- the presence of large stones
- excessively moist areas
- poorly compacted areas
- bumps and depressions
- areas of thin material
- material segregation.

Appropriate action to rectify the problems should be taken before the material has dried out.

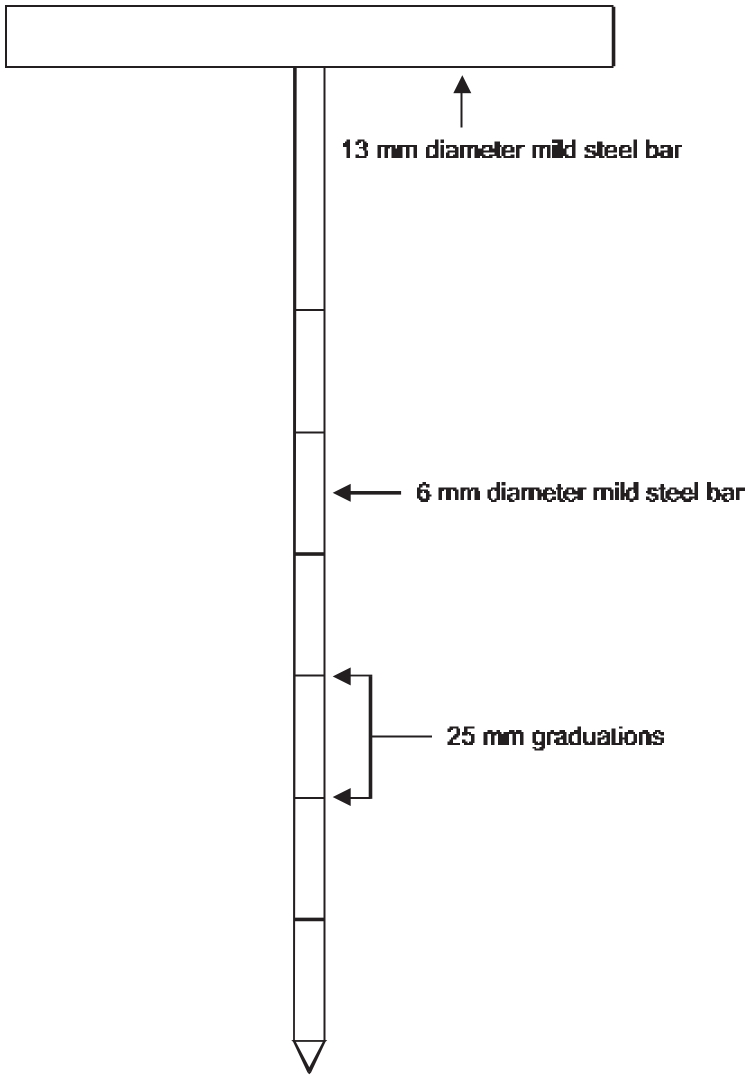


Figure A4: Thickness probe

LABORATORY TEST RESULTS

PROJECT: ______________________________ DATE: __________

SAMPLE NUMBER: ____________________ OVEN DRIED: YES ☐ NO ☐

SAMPLE LOCATION: ________________________________

GRADING ANALYSIS

Sieve size mm	Mass retained (g)	% of total retained	Cumulative % passing		Normalised cumulative % passing	
37.5				D		
26.5				A		A
4.75				C		C
2.0				B		B
0.425						
< 0.425						

G_C = (A – B) x C/100 = (........ – ....) x (......./100) =.................

I_O = 100 – D = (100 – ....) = ...

LINEAR SHRINKAGE

Mould Number	Linear shrinkage (LS) mm	Bar linear shrinkage (BLS)* mm	Shrinkage product (SP)**

* BLS = LS x 0.67
** SP = Mean BLS x Percent passing 0.425 mm

TRETON IMPACT VALUE

Test Number	Total Mass (A) g	Mass Retained on 2 mm Sieve (B) g	Treton value*

* Treton value = ((A – B)/A) x 100

Operator: ____________________

FIELD TEST RESULTS

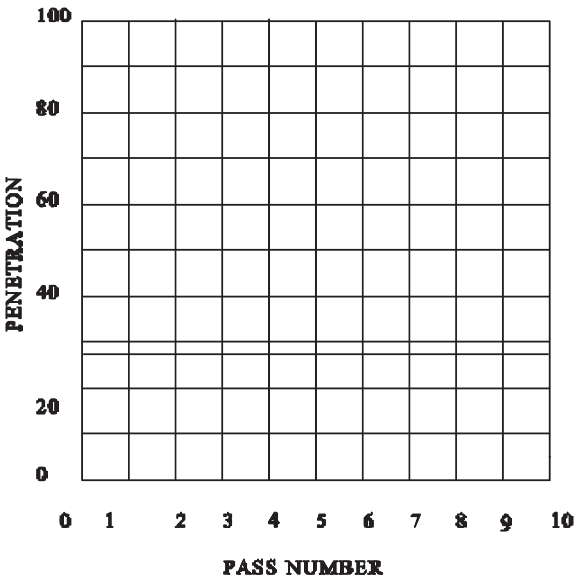
PROJECT: ____ DATE: ______________

TEST NUMBER: ____

TEST LOCATION: ____

COMPACTED STRENGTH

Pass number	1	2	3	4	5	6	7	8	9	10
RCCD reading (3 blows)										
DCP penetration (mm/blow)										



THICKNESS

(25 readings in mm from the probe per job lot)

Operator: ____

Annex B: Contents of CSIR testing kit

The following are the basic contents of the CSIR field testing kit. Certain items (*e.g.* balance and oven), which are necessary in one test, are not repeated in the requirements for subsequent tests.

Grading

- Sheets of canvas 1 metre by 1 metre for coning and quartering material.
- Test sieves: 37.5 mm, 26.5 mm, 4.75 mm, 2 mm and 0.425 mm with pan and cover.
- Balance with pan, accurate to 1 g, to weigh up to 5 kg.
- Various pans of 250 to 300 mm diameter and 20 mm deep.
- Drying oven (Solar) to maintain a temperature between 105 and 110°C.
- Various stiff brushes.
- Electronic calculator (Solar powered).
- Wind shield for balance.
- Levelling platform for balance.
- Thermometer (0 to 120°C).

Linear shrinkage

- Shrinkage moulds with internal dimensions of 150 ± 0.25 mm long × 10 ± 0.25 mm wide × 10 ± 0.25 mm deep and made of 10 mm thick stainless steel bar, open on two sides.
- A steel plate to fit underneath the shrinkage moulds.
- Silicone lubricant spray (*e.g.* Q20 or WD 40).
- A spatula with a slightly flexible blade about 100 mm long and 20 mm wide.
- A pair of dividers and a millimetre scale.
- A standard drop cone and calibrated tube for estimating the liquid limit.

Material strength

- RCCD or DCP test apparatus.

Aggregate strength (Treton)

- A Treton apparatus consisting of a base plate, anvil, cylinder and a hammer weighing 15 kg ± 50 g (Figure A2). The baseplate should be placed on a firm concrete block.
- The following test sieves, 200 mm in diameter: 19.0 mm, 16.0 mm and 2 mm. The larger sieves should be made of perforated plate and the 2 mm sieve of wire mesh.

Thickness

- Thickness probe.

APPENDIX D.2

1. Rainfall analysis for hydrological design

The high variability associated with rainfall events requires a statistical approach to rainfall analysis. For hydrological design it is the worst storm events in a reasonably long period of time (typically 10, 20, or 50 years) that the engineer is concerned with. Because these storms are relatively rare events the statistical information about them lies near the edges of any rainfall-frequency distribution curve and it is at these extremes that the shape of any distribution curve is least well defined. The technique used to analyse the data is therefore called 'extreme value statistical analysis'. In practice the rainfall-frequency distribution is not a normal distribution (in the statistical meaning of the term) but usually a non-symmetrical distribution with a long tail. The most common is the 'gumbel' distribution but a Log-normal distribution is sometimes applicable. Such distributions can be applied to rainfall data as follows:

For a selected duration of rainfall, for example, the 24-hour, 2-hour, 1-hour, 30-minute rainfall, the highest value is selected for each calendar year. This series is termed an 'annual maximum series'. The annual data are then listed in order of descending values and a rank assigned to each, rank 1 being the highest rainfall level. Thus if there is 20 years of data the lowest rainfall will be rank 20. The cumulative frequency distribution (or non-exceedence probability) and the recurrence interval i.e. the return period, are then calculated.

20 year historical data for the 1-hour storm

Rainfall (mm)	Rank	Recurrence interval T years	Non exceedence Probability F	Probability of storm in any year
90	1	21.0	0.95	95% chance of a storm not exceeding this size in any year
70	2	10.5	0.90	or 10% chance of a storm exceeding this size
63	3	7.0	0.86	etc
55	4	5.3	0.81	
48	5	4.2	0.76	
45	6	3.5	0.71	
41	7	3.0	0.67	
39	8	2.6	0.62	
38	9	2.3	0.57	
35	10	2.1	0.52	
33	11	1.9	0.48	
30	12	1.8	0.43	
28	13	1.6	0.38	
27	14	1.5	0.33	
25	15	1.4	0.29	
23	16	1.3	0.24	
22	17	1.2	0.19	
20	18	1.2	0.14	
18	19	1.1	0.10	
15	20	1.1	0.05	

The recurrence interval $T = (N+1)/n$ where N is the number of years of data and n is the rank

The non-exceedence probability $F = 1 – 1/T$

These data should, ideally, fit a Gumbel or Log normal distribution. The equations are:
Gumbel $F = \exp[-\exp((-r-B)/A)]$ and
Log-normal $F = [(\ln r-B)/A]$ where is the standard normal distribution function

In these equations A and B are the parameters that need to be fitted to the data and r is the depth of rainfall from the data above.

It should be noted that the same method can be applied to other data, for example a set of data showing maximum rainfall depths for storms of other durations during each year or a set of data showing maximum rainfall intensities in each year. The actual data can be fitted to these distributions using the traditional least squared deviation approach to obtain the parameters A and B.

Once the relationships have been established the rainfall depth for any recurrence interval can be determined. Usually the values for longer recurrence intervals are needed because rainfall records rarely go back far enough in time. Therefore it should be noted that however well the Gumbel or Log-normal distribution appears to fit the data for the lower and more frequent storm intensities, this is an extrapolation of very variable data and should be borne in mind when designing the drainage system; the apparent accuracy of many of the hydraulic calculations is simply not justified or required.
It is unusual for adequate rainfall data to be available to develop a full set of rainfall intensity curves for the location of interest. Normally only rainfall data for a fixed duration, most commonly for 24-hours, is available and no information on the time duration within the 24-hour period exists. In such cases it may be appropriate to use a generalized relationship between the rainfall falling within a time t hours and that falling in 24 hours. Such a relationship might be of the following form:

$$Rt/R24 = (t/24)[(b+24)/(b+t)]n$$

Where	Rt	=	rainfall in a duration t hours
	R24	=	rainfall in 24 hours
	B, n	=	constants.

The value of b ranges from 0.2 to 0.5 and n ranges from 0.5 to 1.1 and they will need to be computed by analyzing rainfall data from as many rain gauges as possible and including data about shorter time duration storms.

Annex B

Culvert capacity (alternative method)

Maybe we don't need this and the nomographs in the chapter. I will assess which is best

(This Annex provides an alternative method of estimating culvert sizes. The use of equations is preferred by some readers to the use of the nomographs.

If the culvert is relatively short and the streambed slope is sufficient to avoid accumulation of water on the downstream side of the culvert, inlet conditions are likely to prevail. In this case, the culvert will act as an orifice and the capacity can be determined in a relatively simple manner on the basis of headwater height and inlet geometry (barrel shape, cross-sectional area and the inlet edge). Barrel slope affects the inlet control performance to a small degree but may be neglected.

These conditions, known as inlet control, are those that normally occur in most parts of Ethiopia. Estimates of culvert capacity in cases where a high degree of accuracy is not required may be approximated by the expressions in Table B1. The expressions are based on the following assumptions:

- Inlet Control;
- Wingwall Angle = 450;
- Vertical Headwall.

Table B1: Simplified Formulae for Calculation of Discharge Capacity

Type	Discharge Capacity Q (m^3/s) (with inlet control)		
	Hw/D=1.00	Hw/D=1.25	Hw/D=1.50
Concrete Pipe	$1.3 \times D^{2.5}$	$1.9 \times D^{2.5}$	$2.2 \times D^{2.5}$
Corrugated Metal Pipe	$1.1 \times D^{2.5}$	$1.6 \times D^{2.5}$	$1.8 \times D^{2.5}$
Arch Culvert (semi-circular)	$2.3 \times H^{2.5}$	$3.4 \times H^{2.5}$	$4.0 \times H^{2.5}$
Box Culvert	1.5 x B x H1.5	2.1 x B x H1.5	2.5 x B x H1.5
D : diameter of a pipe culvert (m) Hw : headwater height (m)		B : width of a box culvert (m) H : height of a box/arch culvert (m)	

For major culverts, where an under-design could have serious consequences in terms of road failure or damage caused by flooding of upstream areas, detailed calculation of expected culvert performance should be carried out taking into account the geometry of the culvert and the characteristics of the surrounding area.

The calculations should be carried out using recognised computer programs or nomographs, as presented, for example, by the U.S. Department of Transportation, Federal Highway Administration (1985). Tables for the hydraulic design of pipes sewers and channels Volumes I & II, 7th edition, published by HR Wallingford (UK), may also be used where different conditions exist, or greater accuracy is needed.

Table B2: Simplified Formulae for Calculation of Discharge Capacity

Type	Discharge Capacity Q (m^3/s) (with inlet control)		
	Hw/D=1.00	Hw/D=1.25	Hw/D=1.50
Concrete Pipe	$1.3 \times D^{2.5}$	$1.9 \times D^{2.5}$	$2.2 \times D^{2.5}$
Corrugated Metal Pipe	$1.1 \times D^{2.5}$	$1.6 \times D^{2.5}$	$1.8 \times D^{2.5}$
Arch Culvert (semi-circular)	$2.3 \times H^{2.5}$	$3.4 \times H^{2.5}$	$4.0 \times H^{2.5}$
Box Culvert	1.5 x B x H1.5	2.1 x B x H1.5	2.5 x B x H1.5
D : diameter of a pipe culvert (m) Hw : headwater height (m)		B : width of a box culvert (m) H : height of a box/arch culvert (m)	

APPENDIX D.3

1. Advantages and disadvantages of Surfacing/Paving Options

1.1 Basic

S-01: Engineered Natural Surface (ENS)

- **Advantages:**
  - Lowest cost option for basic access (no imported materials required);
  - Easy to construct and maintain using labour-based methods or simple, low-cost grading equipment.
- **Disadvantages:**
  - Suitable only for relatively light traffic (normally up to AADT 50) in gentle terrain (gradient < 6%) with relatively low rainfall (< 2000mm per annum) with moderately strong soils (soaked CBR>15%);
  - May be impassable in wet weather, particularly with plastic materials;
  - Relatively high maintenance requirements;
  - Susceptible to water erosion on gradients;
  - Dust pollution in dry weather.

S-02: Natural Gravel

- **Advantages:**
  - Good performance when properly constructed with adequate quality material
  - Suitable for light to medium traffic (normally AADT < 200)
  - Usually lower initial cost than most other surfacing options
  - Can be used as an intermediate surface in a planned and resourced "stage construction" strategy (however surfacing gravel and road base gravel specification requirements differ)
- **Disadvantages:**
  - A diminishing, finite resource of often variable quality;
  - Need for sustained maintenance programme;
  - High maintenance costs, particularly re-gravelling;
  - Dust pollution in dry weather with health and environmental concerns;
  - Traffic, climatic and longitudinal gradient (typically < 6%) constraints on use relating to rate of gavel loss.

1.2 Stone Paving

S-03: Waterbound/Drybound Macadam

- **Advantages:**
  - Proven performance in all climates
  - Suitable for light and medium traffic
  - Does not require expensive equipment
  - Stone may be broken manually and/or mechanically crushed
  - Can be later upgraded/overlaid with another surface type
  - Can be used in water-scarce and water sensitive locations
- **Disadvantages:**
  - Hand crushed aggregate is usually single sized which creates poor interlock and strength. Improving grading can be achieved by adding different size material but this requires careful supervision and additional labour resources.
  - Stone type and shape is important
  - If non-vibrating equipment used for WBM it should be heavy
  - WBM not suitable for use on weak subgrade which will be weakened by excessive use of water.
  - If used as road surface, will probably require medium levels of maintenance.
  - WBM requires water to be available.

- Smooth to medium roughness.

S-04: Handpacked Stone

- **Advantages:**
  - Proven performance in all climates
  - Suitable for light and medium traffic
  - Does not require expensive equipment to construct or maintain
  - Suitable for construction by small contractors or communities in remote areas with access problems for crushing equipment or heavy plant.
  - Can be later upgraded/overlaid with another surface type
  - Low maintenance, easily repairable
  - Can be later upgraded by sealing in a stage construction strategy
- **Disadvantages:**
  - Requires hard stone to be available locally
  - Stones must not be rounded in shape
  - Smooth to high surface roughness, depending on skill of laying. Rough surface can be uncomfortable for traffic, especially bicycles, motorcycles or carts.
  - Surface is porous, so foundations should not be liable to severe weakening when wet
  - Medium to high surface roughness

S-05: Stone Setts or Pavé

- **Advantages:**
  - Proven performance in all climates
  - Suitable for heavy traffic
  - Does not require expensive equipment to construct or maintain
  - Suitable for construction by small contractors or communities in remote areas with access problems for crushing equipment or heavy plant.
  - Can be constructed at any gradient.
  - Low maintenance, easily repairable.
  - Can be later upgraded by sealing in a stage construction strategy
- **Disadvantages:**
  - Requires hard stone to be available locally
  - Cobble stones must not be roughly cubical in shape
  - Requires skill in laying to achieve a smooth finished surface
  - If non-vibrating equipment used it should be heavy
  - Surface is porous, so foundations should not be liable to severe weakening when wet
  - Smooth to medium surface roughness
  - Stones that polish by traffic, or are slippery when wet, must not be used.

S-06: Mortared Stone

- **Advantages:**
  - Proven performance in all climates.
  - Suitable for light to heavy traffic.
  - Does not require expensive equipment to construct or maintain.
  - Built with unshaped stones and laid by hand. It is therefore suitable for construction by small contractors or communities themselves, or in remote areas with access problems for crushing equipment or heavy plant.
  - Can be constructed at any gradient.
  - Low maintenance, easily repairable.
  - Light compaction equipment is only required for the foundation layers.
- **Disadvantages:**
  - Requires hard stone to be available locally.
  - Stone requires to have at least one smooth, even face.
  - Requires skill in laying to achieve a good bedding and smooth, even finished surface.
  - Smooth to medium surface roughness.
  - Stones that 'polish' by traffic, or are slippery when wet, must not be used.

- Cannot be used until the mortar joints have set and hardened sufficiently (usually about 5-7 days in hot/warm climate).

S-07: Dressed Stone/Cobble Stone Paving

- **Advantages:**
  - Proven performance in all climates.
  - Suitable for light to heavy traffic.
  - Does not require heavy compaction equipment or any other expensive equipment to construct or maintain.
  - It is suitable for construction by small contractors or communities themselves, or in remote areas with access problems for crushing equipment or heavy plant.
  - Erosion resistant, durable, not damaged by diesel/lubricant spillage.
  - Can be constructed at any gradient.
  - Minimal maintenance required, easily repairable.
  - Surface easy to clean, suitable also for urban use.
  - High residual value; the materials can be recycled into other types of paving, or be overlaid with another surface.

- **Disadvantages:**
  - Requires hard stone to be available locally.
  - Stone must be suitable for dressing by hand into a cubic shape.
  - Requires skill in laying to achieve a smooth finished surface.
  - Surface is porous (unless mortar jointed), so foundations should not be liable to severe weakening when wet.
  - Smooth to high surface roughness, depending on skill of dressing/laying. Rough surface can be uncomfortable for traffic, especially bicycles, motorcycles or carts.
  - Stones that 'polish' by traffic, or are slippery when wet, must not be used.

S-08: Fired Clay Brick – Unmortared or mortared joints

- **Advantages:**
  - Proven performance in all climates
  - Options to produce the bricks in a sustainable or low-carbon-footprint way
  - Bricks can be produced in small scale kiln close to the road site, thus reducing transport costs and energy
  - Suitable for heavy traffic
  - Does not require expensive equipment to construct or maintain
  - Suitable for construction by small contractors or communities in remote areas with clay and energy sources but no/few hard stone resources
  - Mortared option can be constructed at any gradient.
  - Surface easy to clean, suitable also for urban use.
  - Low maintenance, easily repairable.
- **Disadvantages:**
  - Requires suitable clay and energy sources to be available locally
  - High quality control required on production and burning process
  - Requires skill in laying to achieve a smooth finished surface
  - Surface of un-mortared option is porous, so foundations should not be liable to severe weakening when wet
  - Smooth to medium surface roughness

1.3 Bituminous Surfacings

- S-09: Surface Dressing (Chip Seal)
- S-10: Sand Seal
- S-11: Slurry Seal
- S-12: Graded Aggregate (Otta) Seal
- S-13: Cape Seal

- **Advantages:**
  - Seals and protects the base and provides strength at the road surface so that the latter can resist the abrasive and disruptive forces of traffic.
  - Protects the pavement from moisture ingress and, in so doing, prevents loss of pavement strength thereby permitting the use of many materials that would otherwise not be appropriate.
  - Improves safety by providing a superior skid-resistant surface, free from corrugations, dust and mud, often increasing light-reflecting characteristics and allowing the application of pavement markings.
  - Reduces vehicle operating and maintenance costs and extends vehicle life.
  - Most bituminous seals (except Ottaseal) are suitable for labour based methods if emulsions are used.

- **Disadvantages:**
  - Relatively costly to construct where bitumen is a high cost item and/or has to be hauled long distances to the project site.
  - Some types (eg Chip Seal, Cape Seal) require relatively high quality crushed aggregates
  - Labour based operations require a high level of quality control to ensure, inter alia, correct and consistent bitumen application rates

1.4 Concrete

Unreinforced:

- **Advantages:**
  - Proven experience in all climates
  - Long expected life span
  - Suitable for all traffic, including heavily loaded trucks
  - Good load spreading properties and suitable for weak subgrades
  - Does not require expensive equipment to construct or maintain
  - Suitable for labour-based, small scale contractor or community construction
  - Erosion resistant, durable, not damaged by diesel/lubricant spillage
  - Can be constructed at any gradient
  - Minimal maintenance required, easily repairable
  - Suitable for roads that suffer flooding (providing foundation remains intact)
  - Surface easy to clean, suitable for urban use

- **Disadvantages:**
  - Expensive to construct
  - Requires to be cured and to gain strength before opening to traffic
  - Cement and steel reinforcement are high components of the total cost, particularly if they have to be imported and/or hauled for a long distance
  - If frost conditions are expected, a higher quality concrete is required.

Ultra-thin reinforced concrete

- **Advantages:**
  - Proven experience for LVR
  - Long expected life span
  - Suitable for light and medium traffic
  - Good load spreading properties and suitable for weak subgrades
    - Does not require expensive equipment to construct or maintain
    - Suitable for labour-based, small scale contractor or community construction
    - Erosion resistant, durable, not damaged by diesel/lubricant spillage
    - Can be constructed at any gradient
    - Minimal maintenance required, easily repairable
    - Surface easy to clean, suitable for urban use

- **Disadvantages:**
  - Relatively expensive to construct
  - Requires high quality concrete and good quality control processes from sub-grade preparation through to curing
  - Requires to be cured and to gain strength before opening to traffic
  - Cement and steel reinforcement are high components of the total cost, particularly if they have to be imported and/or hauled for a long distance