SECTION B: DESIGN

Contents

Tables

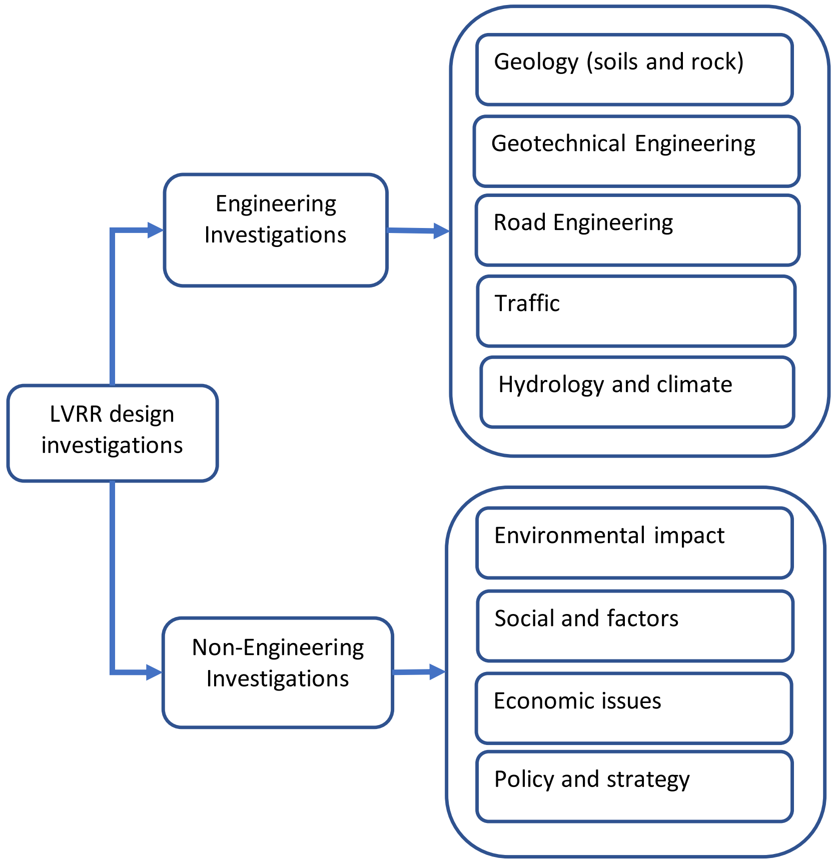
Figures

5 Ground Investigations

5.1 Introduction

Cost effective and sustainable LVRR design depends on the assimilation of relevant information into the design procedures. Consequently, investigation is an essential part of the design process. Commonly, advice on LVRR investigation has largely been focussed on ground investigation ‘engineering’ or ‘geotechnical engineering’ issues. However, given the cross-sectorial nature of LVRR design requirements, other types of data investigation are required including hydrology, traffic and climatic issues. Due importance has also to be given to related factors such as environmental impact and socio-economic issues. In the broad context of a LVRR project, information gathering through investigation should cover both engineering and non-engineering issues (Figure 5.1).

Figure 5.1 Typical Engineering and Non-Engineering Data Sets



The aim of this chapter is to provide guidance on the appropriate types and levels of investigation (data collection) required for best practice design of LVRRs from planning through to construction. The chapter and its associated Annexes also provide users with the necessary guidance on planning, implementation and interpretation of appropriate investigations.

5.2 Engineering Related Investigations

5.2.1 Aims

Engineering design related investigations need to recover a range of data sets as summarised in Table 5.1. Site investigation techniques used to acquire this data encompass a wide range of methods; the amount and type of exploration that is needed for a specific road project will depend on the nature of the proposed project and the environment in which it is to be built.

Table 5.1 Engineering Data Sets

Data Set	Description	Principal Application
Geological	Identification of the geological setting in terms of bedrock, weathering condition and soil types and their relationships.	Definition of a simple geological model including sources of construction materials, natural hazards and foundation conditions.
Geotechnical & Road Engineering	Soil and rock characteristics and geotechnical parameters together with relative locations and geometry. Identification of existing road conditions.	Input to key elements of pavement, earthwork and bridge design.
Traffic	The amounts and types of traffic likely to use the road or roads throughout the design life.	Governs road geometry and key input to detailed pavement design.
Hydrology	Definition of water courses, watersheds and general hydrological conditions.	Essential design input to road drainage, bridge, low water crossing and culverts design
Climate	Definition of current and future climate characteristics including major storm events.	Modification of drainage design to meet climate resilience requirements.

5.2.2 Investigation Stages

Overall site investigations are linked to the principal stages within the project cycle.

- Planning;
- Pre-feasibility (PFS);
- Feasibility Study or Preliminary Engineering Design; (FS/PED)
- Final Engineering Design (FED);
- Construction;
- Maintenance.

In the case of single LVRR projects the PFS may be omitted in some cases. In contrast, larger projects, for example, donor supported programmes involving a number of LVRRs within a single rural road programme, there is likely to be a full suite of stages. Additional special investigations may be required for specific purposes during the Project Cycle; for example, for additional construction materials or to investigate slope stability issues during the construction phase.

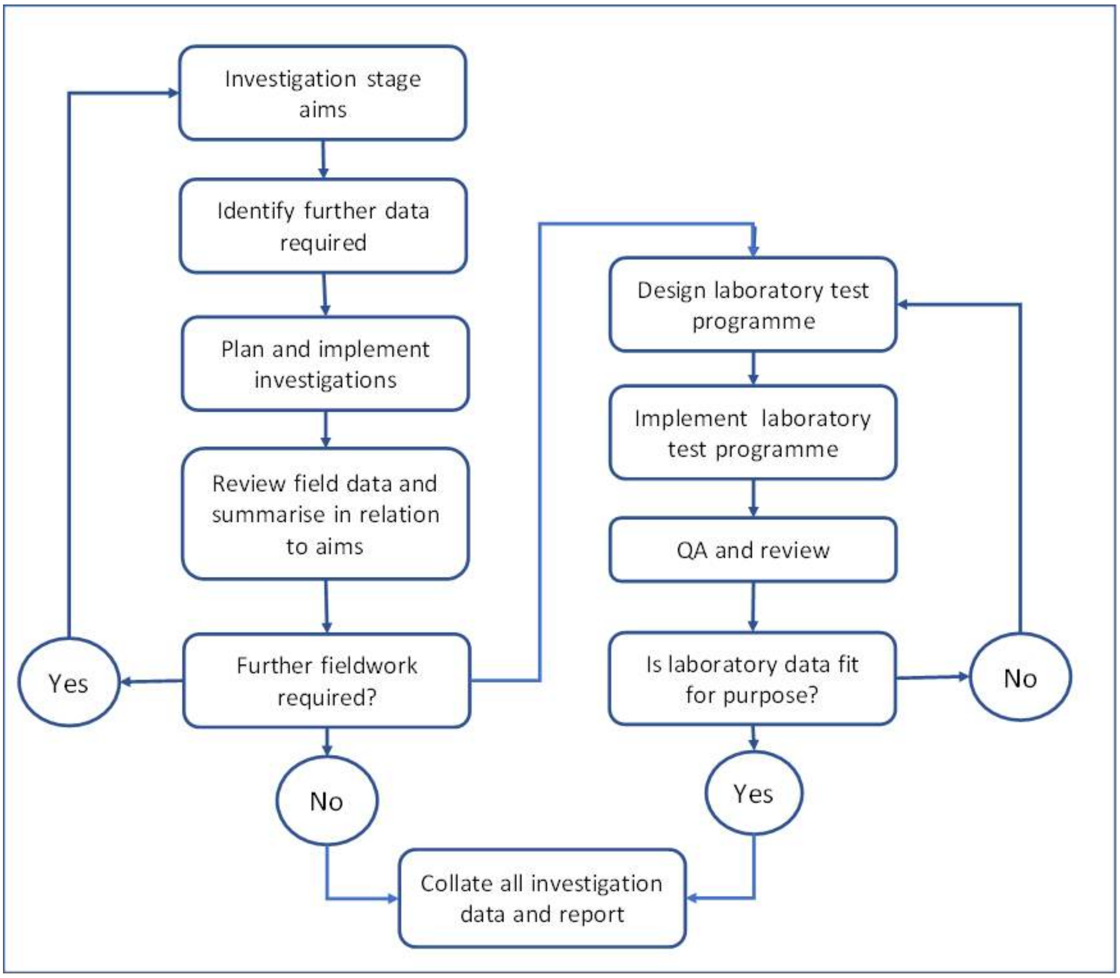
Each phase of a project will have specific objectives and the site investigation requirements for each phase should be focussed on these objectives, Figure 5.2. Investigation budgets are likely to be very constrained for most LVRR projects and hence the procedures and techniques employed must be carefully selected and planned to obtain maximum information for the least cost.

5.2.3 The Myanmar Context

It is unlikely within the Myanmar rural road strategy that many, if any, investigations will be required for completely new alignments, although sections of existing alignment may be modified. Investigations are likely to be focussed on:

- Upgrade or rehabilitation of existing individual roads;
- Upgrade or rehabilitation of groups of roads;
- Modifications to existing alignments;
- Specific road assets (pavement, structures, earthworks).

Figure 5.2 Generic Investigation Framework Adapted For Each Phase



Each of these options will require a different focus. In addition, the Myanmar rural road network provides access and mobility within a range of very different physical and climatic environments, each of which may require different approaches to investigation as well as design.

5.3 Investigation Activities

5.3.1 Desk Studies

Before any on-site investigations are carried out, or even 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 review and analysis of available detailed information (Weltman & Head, 1983; Cook et al, 2001, TRL, 2005).

Studying existing documents, including site investigations from earlier project phases, and examining maps and remote sensing images often eliminates an unfavourable option from further consideration, thus saving a considerable amount of time and money, see Table 5.2.

Table 5.2 Existing Data Sources Relevant to LVRR Design in Myanmar

Information	Functional Use	Application
Existing Myanmar guidelines and manuals.	Main road and highway standards and guidelines from MoC and supporting donors in Myanmar.	Additional design data and information on aspects of geometry, drainage, pavement and materials and structural design of bridges.
Road Design Research	Recent data on geometry, drainage, pavement and materials and structural design of roads and bridges. Note, for example, the DRRD pavement trials in Taunggyi.	Evidence for more cost-effective LVRR designs.
Road and other Engineering Reports	Previous road investigations in Myanmar will provide a range of data and information such as: soil and rock type, strength parameters, hydrogeological issues, construction materials, information on local road performance and sustainability.	Reports may provide geological, hydrological, and geotechnical information for the general area in question that may reduce the scope or better target the nature of the site investigation.
Remote Sensing	Identifies man-made structures, potential hazards, potential borrow source areas. Provides geologic and hydrological information which can be used as a basis for site reconnaissance and tracking site changes over time. Use Google Earth as a low cost approach.	Evaluating photographs or satellite images may save time during construction material or geo-hazard surveys. Google Earth can provide valuable data on terrain, water courses, land-use, alignment geometry and even road condition,
Topographic Maps	Provide a good index or base map. Allows estimation of site topography. Identifies physical features.	Identification of access areas and restrictions, and can estimate potential earthworks requirements possible flood areas before visiting the site.
Geologic Reports and Maps	Provide information on soil and rock types and characteristics. Hydro-geological issues.	A report on regional geology can identify rock types, fracture, orientation and groundwater patterns.
Soil maps	Local soil types. Permeability of local soils.	The local soil survey provides information on near-surface soils to facilitate preliminary borrow pit exploration. Indications of problem soils.
Meteorological and Climatic data	Mean Annual/Monthly; Rainfall and distribution Max and min temperatures; evaporation. Specific reports on climate change in Myanmar.	Indications of current and future climatic risks within a road environment.
Land use /land cover	Distributional and type of soils; drainage and water courses; agriculture and forest cover.	Identify the physical and biological cover over the land, including water, vegetation, bare soil, and artificial structures. Land use changes a key warning indicator of earthwork stability issues.
Local Knowledge	Traffic classification, traffic variation, road user demand, hazards and ground instability, local road performance and maintenance history, accident black spots, water sources, local weather conditions, flood vulnerability and drainage characteristics.	Identification of specific problems and possible hazards along proposed alignment;
Statistics and Future Plans	Population data and demographics; Socio-Economic and household survey information; Development Plans.	Future activities within vicinity of planned road corridor, including changes in land use and use..

5.3.2 Walkover surveys

The initial walkover survey should focus on visually covering the entire length of the proposed alignment before concentrating on detailed locations. Key walkover objectives are summarised in Table 5.3. The walk-over surveys should, in conjunction with the Desk Study, establish the key physical, geotechnical, hydrological, climate impacts, and engineering aspects of the proposed alignment as well as identifying potential problem areas for detailed investigation.

Table 5.3 Road Design Walk-Over Surveys

Survey	Description
1.Initial Site Visit	A broadly-based walkover or "drive and stop" survey to identify general road environment, key features and potential hazards or engineering challenges.
2.Alignment inventory	Walkover survey to systematically log on a strip map all the key features of the alignment or existing road.
3.Road condition	Systematic visual survey of existing road condition. An essential step if a Spot Improvement strategy is being considered.
4.Climate impact	Required to identify areas vulnerable to current and future climate impact. Gathering of local knowledge is key part of this survey. May be combined with surveys 2 and/or 3.
5.Hazard	Detailed additional walkover of hazards identified a part of surveys 2, or 4. May include some simple geotechnical mapping.
6.Hydrological	Walkover survey to log all stream/river crossing and their size, and other potential locations of water crossing. Note issues such as gradients, soil types, existing structure condition, and evidence of flood levels.

Two or more of the above surveys can be combined into a principal Walk-Over survey. Standard forms and procedures are available for use on these surveys (see Annex IV). On some projects, particularly for example those in the mountainous areas of Chin and Shan states, basic engineering geological mapping may be a necessary element of walkover surveys, Figure 5.3 (Geological Society, 1972). Simple walkover geological mapping can be a key element in material investigations (Cook et al 2001, Roughton International 2000).

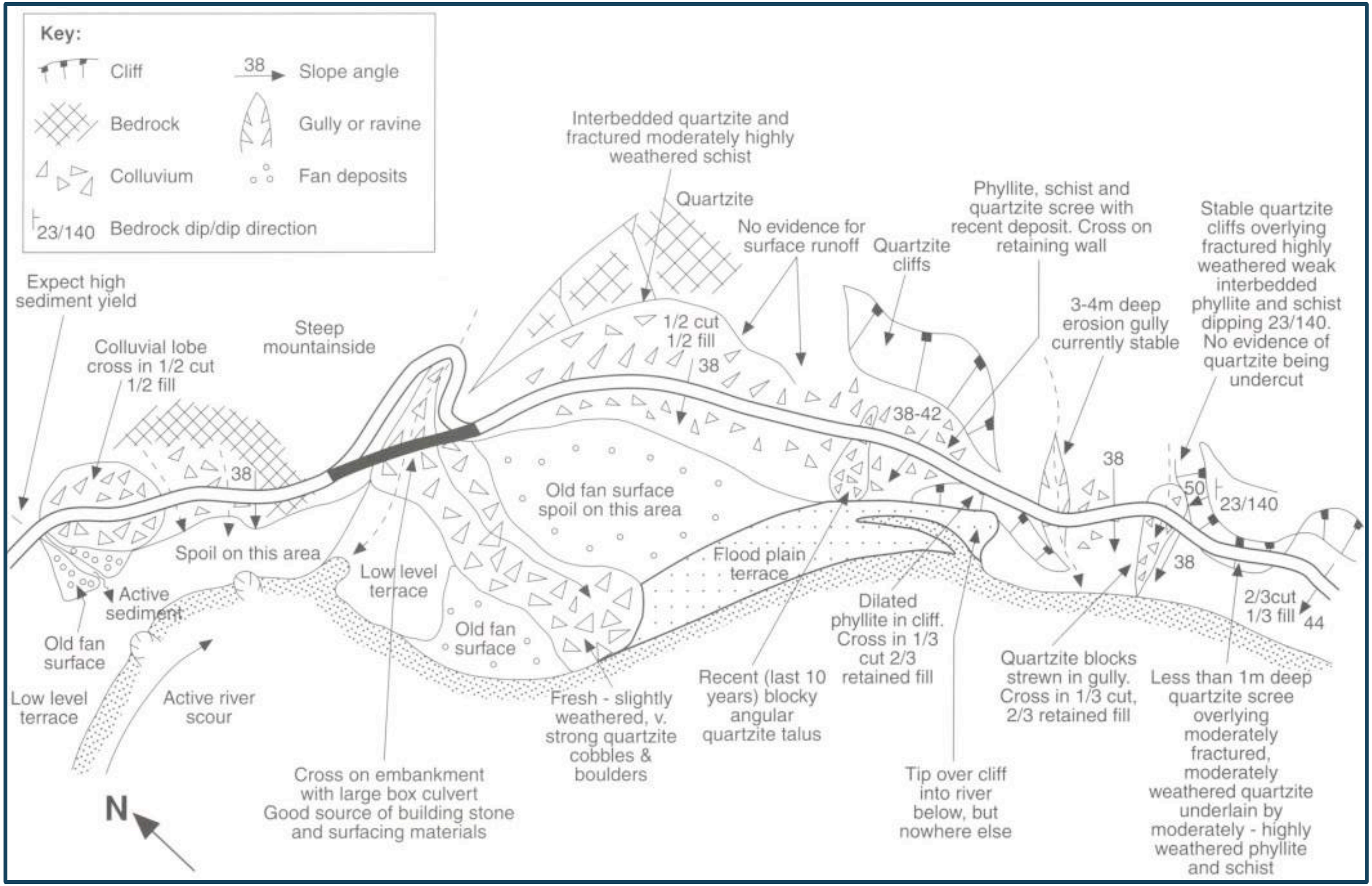
5.3.3 Traffic Survey

Traffic surveys are an essential requirement for estimating the types and amounts of vehicles (motorised and non- motorised) and pedestrians likely to use the road. Details on the methodology of these surveys are included in Chapter 6 and Annex I.

Data from these surveys will be principally analysed in terms of:

- Passenger Carrier Units (PCU); mainly for geometric design and transport service issues;
- Average daily traffic (ADT) adjusted for vehicle type and AADT, mainly for pavement surface and structural design;
- Equivalent Standard Axles (esa); for pavement structural design;
- Axle loads; for pavement and bridge design.

Figure 5.3 Typical Geotechnical map for part of a proposed road alignment in Nepal (TRL, 1997)



5.3.4 Sub-Surface Ground Investigations

Ground investigations should be designed to provide a characterisation of ground conditions or materials relevant to the proposed works (Sabatini at al, 2002). They should establish a basis for the assessment of the geotechnical and road engineering parameters relevant for all stages of the Project Cycle. Investigations may also be required to provide relevant information on groundwater for geotechnical design and construction. Specialist investigations may be required to collect information about identified geohazards and geotechnical engineering problem areas.

Ground investigations may be undertaken using a variety of sampling and testing techniques available in Myanmar, as outlined in Table 5.4. This table presents a list of techniques that may be required for LVRRs in general. Further details of ground investigation procedures likely to be used in Myanmar for LVRR ground investigations are included in Annex IV to this document.

Table 5.4 Standard LVRR Ground Investigation Techniques

Technique	Purpose	Application
DCP survey	In-situ test for strength characteristics. Depth 0.8m or up to 1.5m with extension rods.	Light and portable, gives information on state of near surface ground or existing pavement layers. Testing is quick and simple. Used for pavement design, quality control and light foundation investigations. (See Annex V for further details)
Vane shear test	In-situ shear strength in soft clays.	Especially good for assessing soft clays for embankment foundations. Equipment is easily portable. Can be used in conjunction with boreholes.
Cone Penetration Test or Piezocone	In situ strength and compressibility of soils.	Light portable machines as well as heavier machines are available. Good reliable information in soft to stiff clays and loose to dense sands. Used in areas under moderate to high embankment and for structure foundation investigations.
Test Pits and Trenches	Visual examination of in situ soil profiles; normally 1-2m deep although up to 5m possible with adequate support and safety precautions.	Provides a ground profile and allows good undisturbed sampling as well large bulk samples for testing subgrade and potential fill material. DCP profiles can be undertaken from pit bottoms.
Auguring	Provides in situ information on material present. Wide range of options from hand-held to machine driven.	Can extend to 15-20m depth if machine driven. Hand-held very useful for during walkover surveys. Used for establishing soil profiles and depth to bedrock. Highly disturbed samples. In situ testing possible.
Boring and Drilling	A number of options available from soil wash boring to high quality rock coring to recover samples for examination and testing.	Boring and drilling in most LVRR projects limited to structure sites, deep earthworks and special purposes (e.g. landslide or deep weak soil investigations). Frequently used in conjunction with in situ testing.
Standard Penetration Testing (SPT)	In conjunction with boreholes can provide in situ test strength results in most materials and can be used in weak rocks.	Used in conjunction with auguring or boring holes. Used for gauging ground strength for structure foundation investigation and high earthworks.
Geophysics (Seismic hammer)	Can differentiate between loose unconsolidated sediments and intact bedrock rock.	Light and portable. A sledge hammer and geophones provide a low-cost option. Can use for key areas where rock head is uncertain and critical for design. May require correlation with boreholes.

The use of any technique or combination of techniques for a specific road will be a function of the scale, nature and geotechnical environment of that road. Ground investigations need to be carefully planned and must consider the nature of the ground; the nature and phase of the project; and the project design requirements. Results from the Desk Study and Walk-Over surveys should be used in the planning of cost-effective ground investigations.

5.3.5 Climate Resilience Investigations

Climate resilience studies should be undertaken with a view to assessing the climate risk and defining the appropriate risk reduction options; as outlined in Figure 5.4. A check list for data collection for climate resilience assessment is summarised below (ReCAP 2019, PIARC, 2015):

1. Identify the general climatic environment of the project and its principal characteristics.
2. Define the location of all project roads within the environment
3. Identify specific climate threats
4. Assess the overall condition of the road
5. Identify vulnerabilities of the roads or road assets to the climate threats
6. Identify and prioritise climate impact risks
7. Collect data on climate impact reduction options from similar projects or roads
8. Collect and assess cost data on likely options

The final assessments should be fully integrated with the main phases of LVRR investigations for inclusion in the main project documentation, Figure 5.5.

Figure 5.4 Risk Assessment Sequence (after PIARC, 2017)

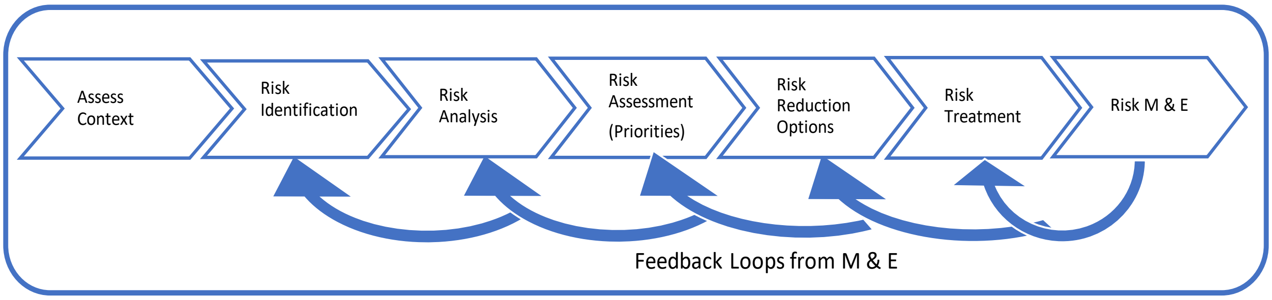
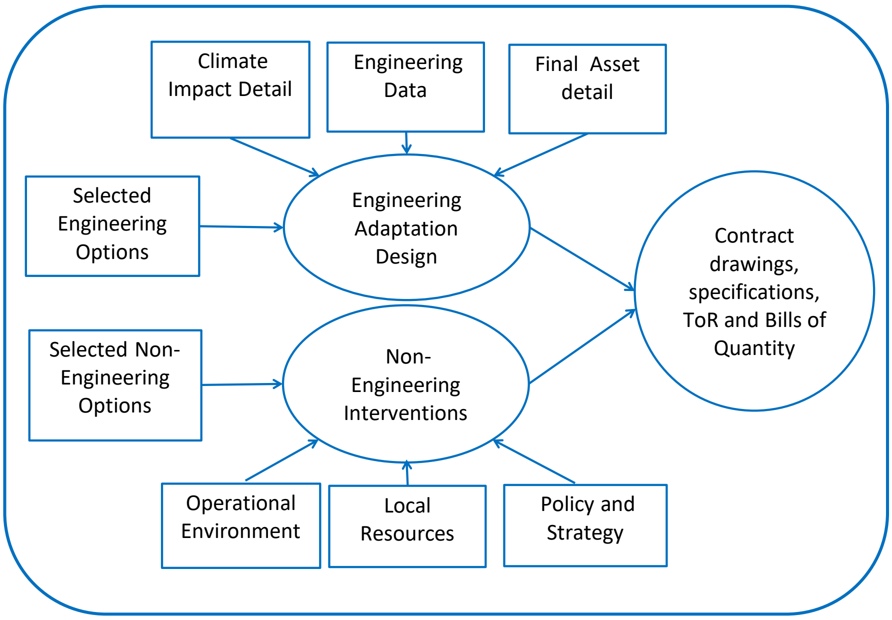


Figure 5.5 Information Flowchart for Climate Resilience



5.3.6 Hydrology and Small Structures

The objective of a structures site investigation is to provide a clear picture of the ground conditions to enable a suitable design to be carried out, as detailed in Chapter 9. The ground underneath a proposed structure should have an adequate bearing capacity to support the load of the structure itself and the vehicles, which pass over it. By identifying and sampling the material excavated from different depths of the trial pits the bearing capacity of the soil can be determined. Bearing capacities are particularly important in the design of structures where large localised loads are expected, (e.g. bridge abutments and piers) and the foundation must be suitable to support these loads as well being able to resist erosion.

The initial assessment should comprise an inventory of all structures (or required structures) on the road network. It is essential that detailed assessments be undertaken at each structure site as these form a large percentage of the overall cost of the road infrastructure (TRL, 1988). Assessments undertaken at sites of proposed structure locations should be sufficiently detailed to ensure:

- Enough time is spent identifying the best location for any new structure;
- The appropriate type of structure is chosen;
- The structure is adequate for the purpose (traffic, water flows and size);
- The design should not need to be significantly changed during construction.

The main issues to be decided during the assessment of new structures are:

- Type of structure;
- Location of the structure;
- Size of structure.

The assessment may be undertaken for either a new structure or the upgrading of an existing structure. In either case the design work will be similar. Table 5.5 summarises key data requirements for small structures design (as detailed in Chapter 9).

Table 5.5 Small Structures Design Data Requirement

Category	Item	Information required
Local Resources	Labour	– Is there an availability of trade skills in the locality e.g. carpentry, stonemasons? – What is the standard of workmanship available? – Options of: – Specialist skills vs. training local labour – Time/cost vs. skills transfer and ongoing maintenance potential – Labour wage rates
	Materials	– What is the availability of local materials (eg masonry stone [rough/dressed], timber, locally manufactured brick and blockwork)? – What is the strength, quality, durability and quantity of local materials? – Steel: what are the imported and delivery costs to site, delays, welding, bending and fixing skills available? – Cement: what are the strengths achievable, delivery/ import delays, types of concrete and experience, quality control and possible testing arrangements? – What are the unit costs of materials?
	Equipment	– What basic specialist equipment is available / would be required for construction AND maintenance (transport, production, loading unloading, mixing, placing, craneage etc.?) – What are the costs of equipment (including transport and servicing costs)?
Design Criteria	General	– What is the reliability of the collected data? – Is a separate structure needed to allow work to commence further along the road? – What will be the cost for construction AND maintenance? – Do pedestrians, animals or IMTs frequently travel along the road?

Category	Item	Information required
	Traffic	– What is the class of road? – Are local standards established for structures on this category of road? – What is the largest type of vehicle that uses the road? – Does vehicle and axle load data exist? – If funds are severely constrained, is a one lane, alternate traffic flow option feasible? – What is the traffic density, does it vary e.g. seasonally or on market days in the local area? – Review standards used elsewhere and recommend appropriate ones. Will the vehicle size or loading increase if the road or structure is improved (new or re-routed traffic)? – Are any exceptional loads transported? - Check for logging, quarries, mining or other industries in the area. What are the possible traffic, economic and safety implications?
Type of Structure	**New Structure**	– Which types of structure would be acceptable?
	Existing Structure	– What is the general condition of the structure? – What was the original design life? – Do as-built records exist? – Are there indications of maximum flood levels on structure? – Are there any signs of post construction settlement? – What are the main problems with the existing structure? – Are there failures in any of the structural elements? – What is the current level of scour around structure? – Indications of excessive loading or abuse? – Dimensions and any possibility of refurbishment or adaptation?
Site Selection	**General**	– Is the depth to firm strata or rock known? – What type of material is available to build on for foundations? – What is the level of the water table? – What is the compressibility or strength of subsoil? – What is the best location of trial pits - to provide the most valuable information? – Is the water / soil chemistry aggressive to building materials? (specialist advice may be required)
Water Parameters	**Water-course Details**	– Is the stream perennial or seasonal? – What is the type of watercourse? (Meandering, straight, bends, presence of weeds) – Is the watercourse and bed stable, e.g. in rock? – What is the low water level? – What are the minimum or normal flow levels? – What are the maximum flood levels (MFL)? (frequency of occurrence and duration) – What are the watercourse cross sections at potential site? – What is the gradient of watercourse upstream and downstream of the crossing point? – Is there evidence of course/bank or level changes, erosion/deposition at the site, upstream or downstream? Consult with old maps and the community – Is there sometimes floating debris in the water? – What is the water velocity during floods? – What is the longitudinal section or profile along the watercourse? Is the watercourse used for private or commercial traffic with headroom requirements? – Size and amount of sediment supplied from catchment area.
	Catchment Details	– Area of catchment? – Are sudden floods encountered? – Shape of catchment? – Gradient of terrain? – Permeability of soil? – Vegetation coverage and type? – Rainfall intensity? – Is the vegetation coverage changing rapidly e.g. Deforestation?

5.3.7 Laboratory Testing

Within the budget constraints of LVRR investigations geotechnical laboratory testing should not be commissioned on an arbitrary or ad hoc basis but should be part of a rationally designed programme. No single test procedure is likely to satisfy design information requirements and a package of test procedures will be needed. An appropriate test programme will include a logical selection and sequence of tests that is function of the geotechnical environment, the nature of the investigation and the road design requirements. Clear objectives should be identified and test procedures and test programmes should be designed with these in mind (Cook et al, 2001, Head 1992). The relationships between in situ conditions and the disturbance or remoulding experienced by the sampled and tested material need to be clearly borne in mind when designing and developing the test regime.

With some specific exceptions, all testing in Myanmar should be performed in accordance with appropriate specification for laboratory procedures in AASHTO and ASTM. Standards such as AASHTO-ASTM lay down procedures of good practice that are in the main based on "normal" experience with temperate zone sedimentary soils. When dealing with tropical residually weathered materials special procedures are often necessary to obtain reliable, relevant and consistent results. This applies particularly to the handling and treatment of samples before testing (Head 1992, Geological Society, 1997).

Annex II outlines the key issues in the design and undertaking of laboratory test programmes. Particular emphasis is placed on the selection of appropriate tests and the need for effective quality management throughout the whole testing and reporting process.

Within an overall aim of assuring that selected materials and designs are fit for their identified purpose, testing is undertaken for a number of reasons.

- Characterisation of soil rock masses and materials along the route;
- Assessment of geotechnical properties influencing earthwork cuts and fills;
- Assessment of geotechnical properties related to natural hazards;
- Identification of potential material resources;
- Construction quality assurance;
- In service post-construction monitoring;

It is useful to divide materials test procedures into a number of general categories that reflect the nature of the test. These are:

- Physical: Tests associated with defining inherent physical properties or conditions;
- Simulation: Tests associated with portraying some form of geotechnical or engineering character either directly or by implication;
- Chemical: Tests aimed at identifying the occurrence of key chemical compounds;
- Petrographic: Those tests or assessments associated with analysing or describing fabric or mineralogy, although this group of tests are unlikely to feature in most LVRR investigations.

An understanding of the properties being measured by the individual tests is important in the selection of appropriate procedures. Simulation testing, in particular, may be based largely on empirical testing procedures rather than modelling expected service behaviours. This issue is particularly important in tropical and sub-tropical environments where assumptions and empirical correlations are derived from testing temperate region soils (Geological Society, 1997).

Tropically weathered material behaviour can be noticeably different from temperate sedimentary soils. The approach to the laboratory investigation of tropical materials in terms of the range of tests employed, their detailed procedures and their interpretation should derive principally from the following:

- Chemically bonded materials (e.g. affects assessment of strength);
- Mineralogical complexity (e.g. influences volume change);
- Fragile relict fabric and texture (e.g. leads to particle break-down);
- Moisture susceptibility.

5.3.8 Special Investigations

There may be a need to include special investigations for specific hazards or geotechnical problems.

Table 5.6 Special Investigations in the Myanmar Road Environments

Issue	Description	Investigation focus
Road side instability	Actual or potential slope failure or erosion impacting on the road alignment.	Geometry of failure and soil rock profiles. Walkover geotechnical surveys and test pitting useful for small failures. Geophysics for bedrock depth. Data required for stabilisation and protection works – see Chapter 10
Soft or organic soils	These soils can cause problems of settlement or failure underneath embankments.	Depths of soft materials by auguring, boreholes or possible geophysics. In situ testing by shear vane or CPT for design of options outlined in Chapter 10.
Marginal construction materials	The requirement to use local materials may require the investigation and use of marginal quality resources.	Sampling and testing to define material properties and possibly the use lime, cement of mechanical modification techniques. (See Chapter 8)
Sensitive soils	Soils that have a fabric vulnerable to sudden collapse or are highly erodible.	Extent and nature of problem soils by pitting and careful sampling. Specialist testing – see Chapter 8 and Annex II
Swell/shrink soils	Soils subject to excessive volume change on wetting drying.	Extent and nature of problem soils by pitting and careful sampling. Specialist testing – see Chapter 8 and Annex *IV*

5.4 Investigation Implementation within the Project Cycle

5.4.1 General

The range of investigation procedures will be applicable throughout the project cycle, although the emphasis may change. Figure 5.6 outlines the stages of investigations linked to a normal project cycle and Table 5.7 indicates the relative importance of investigation procedures with differing stages in the project cycle.

Figure 5.6 Investigation Stages

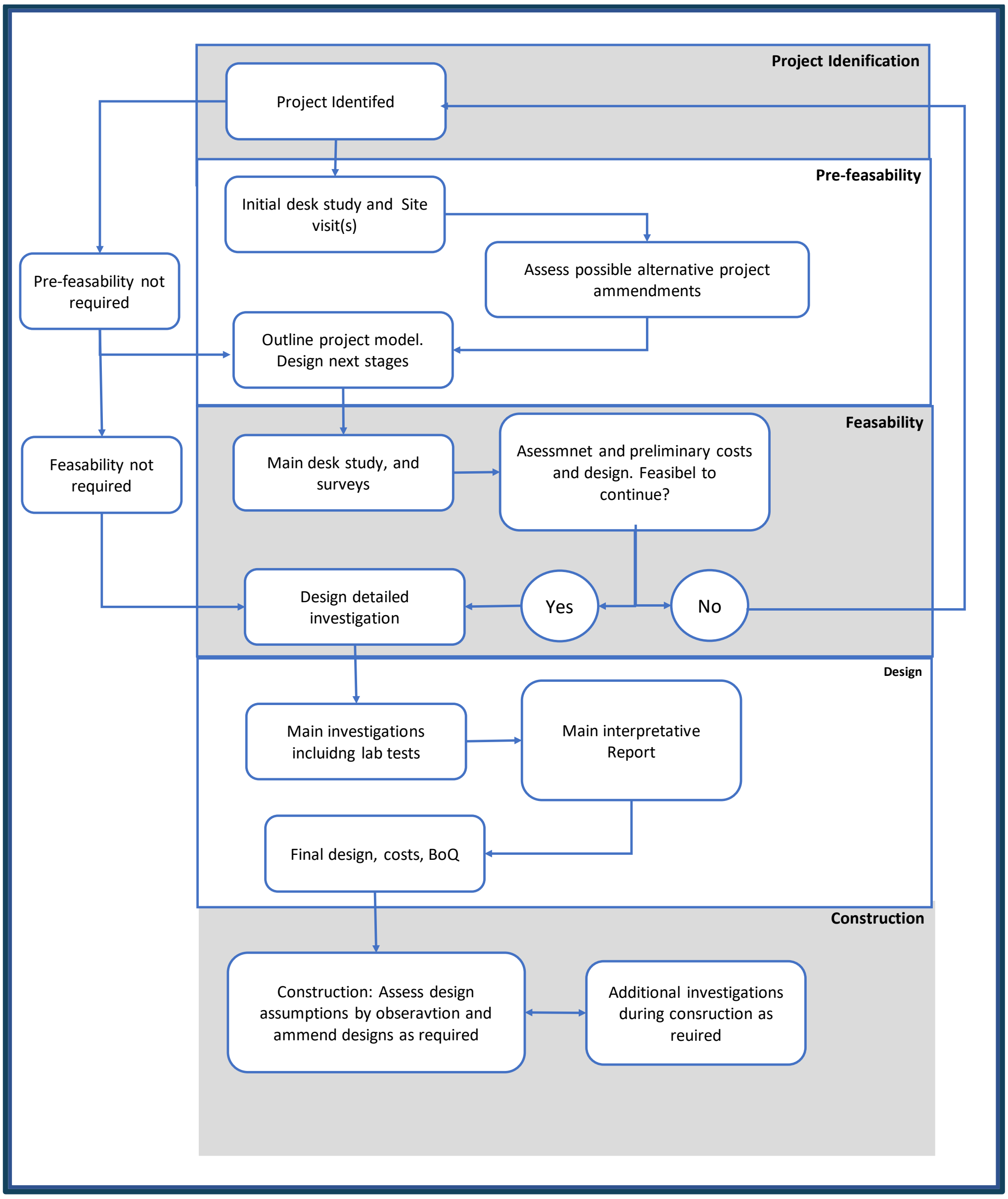


Table 5.7 Relative importance of investigation activities during the project life-cycle

Project Phase	Investigation Activity			
	Desk Study	Walkover - Mapping	Ground Investigation	Laboratory Testing
Planning	A	B	C	C
Feasibility Study	A	A	B	B
Final Engineering Design	B	B	A	A
Construction	C	B	B	A
Maintenance	C	B	C	C
Rehabilitation	B	A	B	B

Note: A: Primary activity; B: Support Activity C: Supplementary Activity

5.4.2 Pre-feasibility Stage (PFS)

The following is a check list for data acquisition and investigation procedures at the PFS stage of a project:

1. Confirmation the road falls within the "LVRR" category for design purposes.
2. A general summary of the likely road environments to be encountered.
3. Whether there are likely to be any major technical challenges.
4. A general assessment of whether or not there are likely to be any major construction materials issues in terms of quality or quantity.
5. General climate and climate change pattern and identification of data sources.
6. Indications of any major problems in terms of flood impact or, alternatively, potential water shortages.
7. Broad classification of project terrain and identification of mapping sources.
8. Sub-grade: identification of reported issues with previous projects.
9. General levels of traffic: for example, within any existing road classification system.
10. Identify the classification levels for the proposed project. If none exists than it may be necessary to outline temporary guidelines.
11. Confirm the relevant standards and specifications that are legally in place and review whether or not they are appropriate.
12. Identification of general legal requirements and any likely major constraints and issues as regards environmental impact.
13. Identification and assessment of the implications of any governing safety policies.
14. Preliminary social impact data.
15. Information on likely Complementary Interventions.
16. General outline costs.

5.4.3 Feasibility

In general terms the Feasibility Stage assesses potential design options and identifies those most likely to provide a sustainable solution within the governing road environment and within the expected budgets. This is generally seen as a critical stage by road authorities and external funders and donors such as the World Bank, ADB or JICA. Relevant Ministry planners and DRRD, Local authorities and Consultants are normally closely involved at this stage.

The following is a check list for data acquisition and investigation procedures at the FS stage of a project:

1. Identify likely sources of construction material in terms of location, quality and quantity and note any particular shortfalls or material sources that have caused problems in the past.
2. Establish current and likely future annual climatic patterns from ongoing desk study. Undertake an assessment of the likely climatic impacts and hence the likely level of Climate Resilience required.
3. Surface and sub-surface hydrology; identify the general hydrological conditions, and variability, prevailing over the proposed alignments by walkover survey, examination of available records and discussion with local groups.
4. Define the overall relative percentage of terrain groups along the alignment. Identify any high risk critical areas.
5. Establish likely minimum strength values for subgrade along alignments. Identify problem areas likely to impact significantly on pavement design.

6. Establish likely requirement for earthworks and outline design.
7. Investigate drainage requirements
8. Investigate of bridge locations and other river crossing sites
9. Investigate the general traffic regime including likely traffic volumes in terms both of PCUs and esa. Identify potential "design vehicles". Define equivalent axle loads for prevailing vehicles either from continuing desk study or preliminary axle load surveys.
10. Identify the level of experience of the potential contractors in terms of the likely pavement and surfacing options. Identify potential training needs for local contractors.
11. Assess the environmental impacts of the proposed pavement and surfacing options within the framework of governing regulations and prepare draft environmental management plans.
12. Identify or draft road safety standards relevant to the geometric design of the proposed pavement options. Link these standards to the identified non-motorised traffic elements that will use the proposed roads.
13. Relevant social impact and safeguards data.
14. Complementary Intervention information.
15. Cost estimation data.

5.4.4 Final Engineering Design (FED)

The FED stage requires sufficient data for preparation of the contract documents including design, technical specifications and Bills of Quantities. Final detailed cost estimation is also likely to be required. The FED stage requires more investigation and considerably more detail than has been required hitherto. The entire process of project design should now be completed with sufficient accuracy to minimise the risk of changes being required after the works contract has been awarded.

The following is a check list for data acquisition and investigation procedures at the FED stage of a project:

1. Sources of material should be defined in terms of location, quality and quantity such that it is clearly established that the road or roads can be built to the required specification with the available materials. Source location, haulage, processing and placement costs need to be investigated and any inflation factors considered.
2. Climatic patterns and the nature of future climate and potential severe climatic events should be confirmed. The levels of Climate Resilience should be defined and adaptation measures selected and designed.
3. Detailed geotechnical data for earthwork design.
4. Detailed geotechnical data on any identified potential hazard areas.
5. Seismic impact data for large earthworks and bridges.
6. Ground water levels should be established and areas and depths of flooding defined.
7. Alignment gradients should be established based on the topographic survey. These may be critical in terms of Spot Improvement strategy.
8. Design sub-grade strengths should be selected based on updated or more detailed site work building on the feasibility data.
9. Detailed bridge abutment and pier foundation conditions should have been defined.

10. Feasibility assumptions on traffic patterns should be cross-checked and, if required, additional surveys undertaken aimed specifically at obtaining data for each vehicle category and axle loading for the pavement layer design.
11. The "Green" Environment. Undertake any further required environmental impacts studies.
12. Social impact and Complementary Intervention data should be collected, finalised and be incorporated into the contract documents.

5.4.5 Construction

Data acquisition and investigation procedures at this stage of a project will largely be centred on the following activities:

1. Quality control through a combination of visual inspection, in situ testing and laboratory check testing (See Chapter 12 for further detail)
2. Investigations into design changes or alterations. This may be due to occurrence of unforeseen ground conditions or a change of plan by the road authority. The standard investigation techniques described above and in Annex IV would still apply.
3. Undertaking a Technical Audit – *see Chapter 14 for further details*.

5.4.6 Level of Investigations

The size and costs investigations vary depending on the type of road and the complexity of the geotechnical and other environments, but the following figures are based on Institution of Civil Engineers (2012) as a percentage of capital cost expenditure

- Bridges 0.2 to 0.5%
- Road 0.2 to 2.0%
- Embankments 0.1 to 0.2%

LVRR investigations in some regions of Myanmar, for example mountain roads in Chin or Shan states or roads on soft clays in the delta area will be at the higher end of the range, or in some cases even higher.

Ideally, sufficient budget should be allocated for a thorough investigation to facilitate cost-effective road design and construction and to reduce the possibility of unexpected conditions being encountered during construction of the works. These can frequently lead to costly delays amounting to much more than would have funded a properly programmed and conducted investigation.

Table 5.8 summarises some guidance on the extent of typical investigations in anticipated normal ground conditions. Additional or deeper investigation would be required in areas of special ground conditions.

Table 5.8 Investigation Requirements and Locations[1]

Asset	Location and Extent
Pavement	Feasibility studies; combination of pits and DCP at 0.5km to 1km spacing and change of soil types. Detailed studies at 200m spacing with DCP, closer in difficult ground conditions. Sampling for soil index tests and CBR/MDD – number depending on soil, type and what DCP design approach is adopted. Boreholes/auger holes in areas of suspect ground.
Deep cut	At least 1 borehole per 200m to Depth (D)= 2m (or D=0.4 maximum cut height) below foundation level. Test pits at 200m. DCP at 50m spacing can be used for interpolation of soil depths.
Embankment	At least 1 borehole per 200m to D= 6m (or D=0.8-1.2 times maximum cut height) below foundation level. Test pits at 200m. DCP at 50m spacing can be used for interpolation of soil depths. Undisturbed sampling for settlement characteristics. Additional Boreholes/auger holes in areas of suspect ground.
Low Water Crossing	No sub surface ground investigation normally required unless poor ground identified during walkover.
Culvert	1 trial pit at outlet. 1.5 m. Possible DCP in weak ground.
Vented ford	2 pit (only 1 required if ford is shorter than 15 meters). At each end of the vented section, preferably one on the upstream and one on the downstream side D=1.5m. Possible DCP in weak ground
Large bore culvert	Pits at each abutment and each pier. 2.5 m (deeper in poor ground conditions). Possible DCP in weak ground.
Bridge	1 borehole (or pit if bedrock close to surface) at each abutment and each pier. To firm strata (minimum of 3m into foundation strata). Geophysics may be used to interpolate bedrock if required.
Borrow pit	An exploration phase of 2-3 pits through depth of usable material to identify possible sources. A more detailed proving phase with pits at 50m spacing is commonly recommended. Large bulk sampling will be required for the whole suite of properties required for earthworks or pavement usage and the development of haulage diagrams (Chapter 8).
Hard rock quarry	Existing quarries can be sampled for quality. Quantity estimate by walkover and terrain mapping. Feasibility or exploration for new resources may require 1 to 2 boreholes to prove depth and quality. Geophysics may be used to determine extent of good rock once it is identified. Suitably representative material for rock and aggregate testing.

[1] Information in this table based on a number of sources (SANRAL, 2013; SAICE,2010; Cook et al 2001; BSI,1997)

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6 Geometric Design

6.1 Introduction

6.1.1 Background

The 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 balance two important objectives namely to provide appropriate levels of safety and comfort for road users and to minimize earthworks to reduce construction costs.

Geometric design covers road width; cross-fall; horizontal and vertical alignments and sightlines; and the transverse profile or cross-section. The cross-sectional profile includes the geometry of the carriageway, shoulders, verges and side drainage ditches. Although embankment heights and side slopes are a vital part of cross-section design they are dealt with in a separate process (Chapter 10).

The cross-section essentially adapts the pavement or carriageway to the road environment and is closely tied-in with drainage. For paved roads, wide, sealed shoulders and high camber or cross-fall can significantly improve the operating environment for the pavement layers by minimizing the ingress of surface water. Most drainage aspects are dealt with in the Pavement (Chapter 7), Drainage and Structures (Chapter 9) and Earthworks (Chapter 10).

6.1.2 Myanmar Rural Roads Classification

As outlined in Chapter 2, the Central Committee for the Development of Regional Roads and Bridges has broadly grouped Myanmar rural roads into three administrative classes for the establishment of the CRRN (Core Rural Road Network) in line with the National Strategy for Rural Roads and Access (NSRAA). Section 2.4.2 and Table 2.2 outline this broad three tier classification which forms the basic framework for guiding the Myanmar geometric design standards.

6.1.3 Chapter Scope

This chapter introduces and presents the rationale, considerations and selection procedure for appropriate geometric designs for a particular route and sets out the various Geometric Standards to be used for LVRR in the contrasting road environments of Myanmar.

6.2 Principal factors affecting geometric design standards

The principal factors that affect the appropriate geometric design of a road are:

- Cost and level of service;
- Traffic volume and composition;
- Terrain and topography;
- Safety;
- Pavement type;
- Roadside population (open country or populated areas);
- Administrative or functional classification.

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 design a fixed number of geometric standards to cope with the range of key factors in the relevant network.

6.2.1 Cost

Designs have to be justified economically, and the optimum choice will vary with construction, maintenance and road user costs. As road standards increase, agency costs (road construction and maintenance costs) increase but road user costs decrease. From an economic point of view the standards adopted should be those applicable to the minimum total costs. The position of this minimum is strongly dependant on the traffic level, as would be expected, hence, from an economic point of view; standards should increase as traffic increases.

6.2.2 Traffic Volume and Composition

Whilst economic appraisals leading to the adoption of road geometry are a standard procedure for roads carrying high volumes of traffic, the issue becomes more complex for LVRRs where the traffic mix may include significant numbers of bicycles, motorcycles, motorcycle taxis, trailers drawn by agricultural engines, animal and drawn carts. LVRRs serve social functions that are vital but which are difficult to fit into standard transport economic theory and the connection between traffic and justifiable standard is much more difficult to establish.

The safe and comfortable accommodation of road users is closely related to the width of the roadway and the traveling speed of motorized traffic. For low volume rural roads congestion is seldom an issue and, when it is, it generally arises not specifically from traffic volume but rather from the disparity in speed between the types of traffic which the road serves. In other words the traffic composition is the key factor rather than traffic volume. However, the size of the largest vehicles that use the road 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.

In order to quantify traffic for normal capacity design the concept of equivalent passenger car units (PCUs) is often used. PCU assessment is related to, but different from the ADT values often used in assessing pavement options. PCUs are concerned with the combined space and time that different vehicles occupy Thus a typical 10-tonne truck requires about three times as much road space as a typical car hence it is equivalent to 3 PCUs. A motorcycle requires less than half the space of a car and is therefore equivalent to 0.5 PCUs. Vehicles, that are slow-moving, 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. Hence, for example, a slow moving animal drawn cart although physically smaller than a 10 tonne truck is very slow moving and its total impact in terms of size and road space occupation time is high.

Differing countries have different PCU factors; Table 6.1 is based on the current Nepal standard and is recommended for use in Myanmar.

Table 6.1 PCU values (DoLI, Nepal 2012)

Ref.	Vehicle Type	PCU
1	Car, Light Van, jeeps and Pick Up	1.0
2	Light Truck up to 2.5 tonnes gross	1.5
3	> 2 axle Truck up to 10 tonnes gross	3.0
4	Truck up to 15 tonnes gross	4.0
5	2 axle tractor towed trailers -standard	3.0
6	Single axle tractor towed trailers -standard	1.5
7	Bus up to 40 passengers	3.0
8	Bus over 40 passengers	4.0
9	Motorcycle or scooter	0.5
10	Bicycle	0.5
11	Rickshaw and Tricycle carrying goods	1.0
12	Auto Rickshaw	0.75
13	Hand Cart	2.0
14	Bullock Cart with Tyre	6.0
15	Bullock Cart with Wooden Wheel	8.0
16	Horse-drawn carts	6.0
17	Pedestrian	0.2

Roads should be designed to provide good service throughout their design life and therefore the traffic level to be used in the design process must consider traffic growth. 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 Myanmar might be increase considerably more than on a ‘main access’ road in another area.

6.2.3 Terrain and topography

The terrain has a major influence on geometric standards both because of the much higher costs that would be incurred if the same standards were used in hilly and mountainous terrain and because of the additional safety aspects required for roads in such terrain. The purpose of taking terrain into account is to minimize costs by adjusting the geometric standards to avoid, for example, large earthworks in hill or mountain regions. Table 6.2 outlines a simple terrain classification to be used for assessing the geometric design. The terrain class should reflect the conditions actually along that alignment not the surrounding area. Even in mountainous terrain a road in a wide river valley may not require expensive earthworks and should be designed based on a flat or rolling terrain class.

Table 6.2 Terrain Classification

Classification	Definition
Flat	0 to 10 five-meter contours per km. The natural ground slopes perpendicular to the ground contours are generally below 3%.
Rolling	11 to 25 five-meter contours per km. The natural ground slopes perpendicular to the ground contours are generally between 3 and 25%.
Mountainous	26 to 50 five-meter 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 risks involved. Typical gradients may have to be greater than those typically encountered in mountainous terrain.

An important aspect of geometric design concerns the ability of vehicles to safely ascend or descend steep terrain. 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. Hand or animal drawn vehicles are unable to ascend or descend relatively low gradients safely, and catering for them in hilly and mountainous terrain is rarely possible. In hilly and mountainous terrain geometric standards for LVRRs must take into account of the constraints imposed by the difficulty of the terrain and geometric designs may need to be reduced below standard locally, in order to cope with terrain conditions.

Every effort should be made to design the road so that the maximum gradient does not exceed the standards. Climbing lanes are costly and are, in general, rarely used for LVRRs, but in exceptional cases a climbing lane may be required (Section 6.3.5).

6.2.4 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, large numbers of motorcycles and bicycles, poor driver training and poor enforcement of regulations. In such an environment, traffic safety assumes paramount importance. 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 together with:

- Mixed traffic composition (including NMT and pedestrians);
- Inappropriate public transport pick-up/set-down areas;
- Poor road surface condition (potholes, ruts);
- Dust (poor visibility);
- Slippery unsealed road surfaces.

Traffic safety issues are discussed in detail in Section 6.7.

6.2.5 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 paved road. This is because of the very different traction and friction properties of types of surface. Although not directly considered there is also an issue regarding poor visibility for drivers on unsealed dry dusty roads. Higher geometric standards are generally required for unsealed roads compared to an equivalent sealed road.

A road that is to be paved at later date should still be designed to the different, usually higher, unsealed geometric road standards.

6.2.6 Population

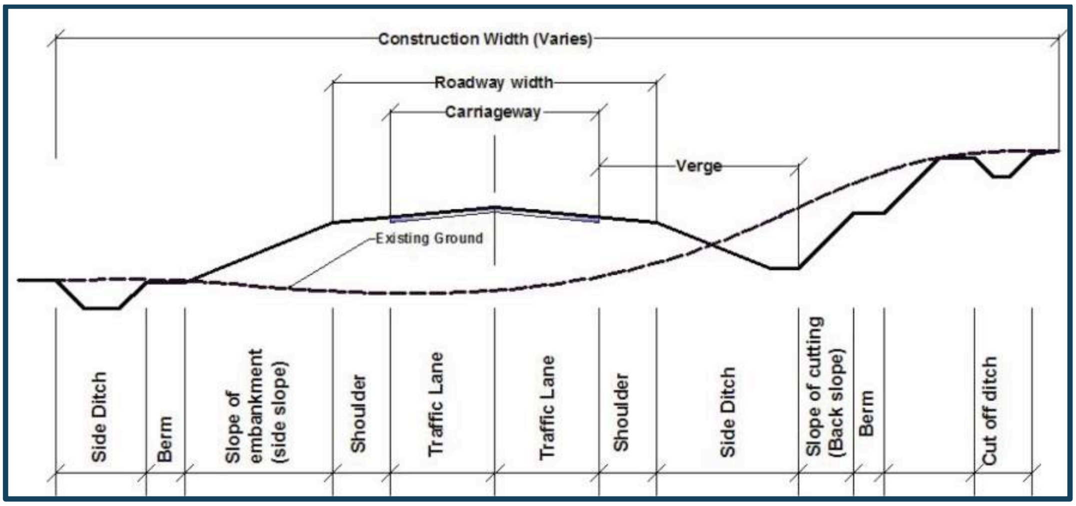
More populated areas in village or peri-urban areas where markets and other business activities take place the geometric design of roads needs to be modified to enhance safety. This may be by using wider shoulders, including specifically designed lay-byes for passenger vehicles to pick up or deposit passengers.

6.3 Cross-section Geometric Design

6.3.1 General Considerations

Geometric cross-sections have to be compatible with the requirements of not only vehicular traffic but also with the needs of pedestrians and non-motorized vehicles. In Myanmar, as in many developing country situations, it is necessary at the earliest stage in the design process.to consider cost effective ways of segregating non-motorized traffic (TRL, 1988).

Figure 6.1 Geometric Elements of a Rural Road Cross Section



6.3.2 Road Width

Road width (carriageway, shoulders and verges) is a key geometric consideration since it can be directly related to cost. A worldwide survey of rural road widths undertaken in Cambodia (SEACAP, 2009) identified a pragmatic relationship between traffic and road width, Figure 6.2.

Figure 6.2 A general Relationship between Traffic and Road Width (CW+Shoulder)

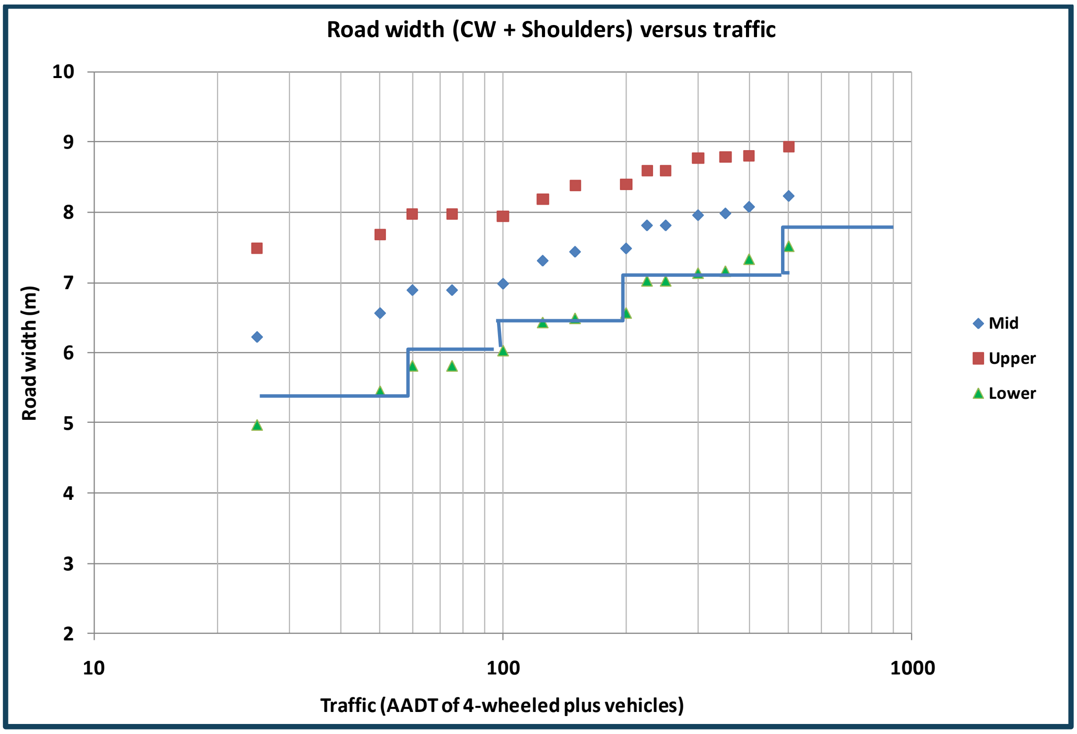


Table 6.2 shows how road width can be assigned to traffic ranges. The central data line is the average and the outer lines are plus and minus one standard deviation. In reality the lines are not straight but comprise a number of steps as shown in the lower line in the figure. For example, for traffic in the range 50-100 PCU a roadway width of 6-8m might be expected, whilst for 300PCU a figure of 7-9m is indicated. The widths of shoulders are key aspect in considering this figure, with wider shoulders being required for higher numbers of NMT. Hence for the same PCU of 50-100 the roadway could be 4m carriageway and 1m shoulders (4m+2x1m=6m) or with high proportion of NMT shoulder widths of 1.5m might be more appropriate (4m+2x1.5m=7m).)Most rural road standards have based the carriageway width on a 'design vehicle' in combination with traffic volume, and adapt the shoulder width to accommodate differences in traffic mix (for road safety reasons).. A truck or a bus of 2.5 or 2.6m width with a length of between 9 and 12m is commonly used as a design vehicle but this may vary with the region and smaller vehicles, such as pickups may be more appropriate design vehicles for very low volume access roads in Myanmar.

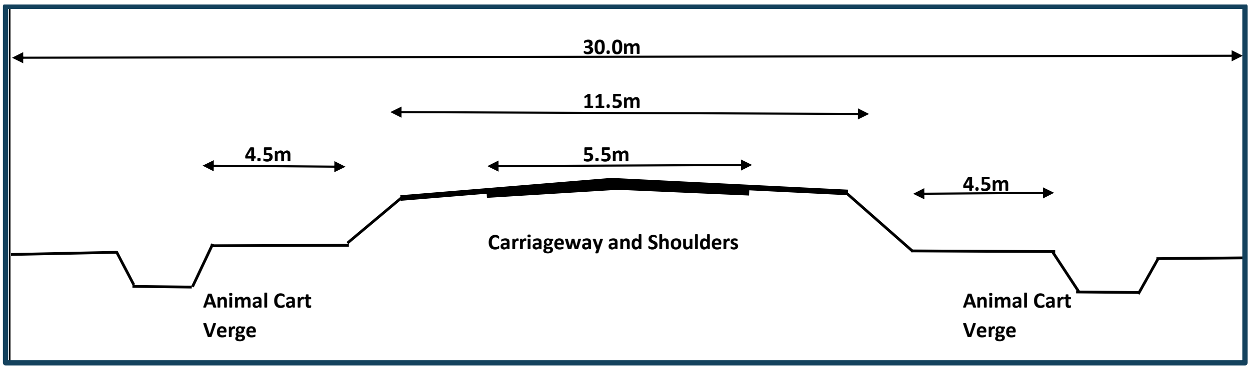
6.3.3 Shoulders and Verges

The shoulders of a road fulfil the following functions:

- Provide safe room for temporarily stopped or broken-down vehicles (clear zone);
- Provide additional manoeuvring space on roads of lower classification and traffic flows;
- Enable non-motorized traffic to travel safely with minimum encroachment on the carriageway;
- 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 by extending the seal over the shoulder.

Geometric design for Myanmar rural roads should recognize that much of the rural traffic may be non-motorized and wide shoulders will be required. In addition, there is a requirement within the overall CRRN classification to include a wide verge for animal drawn traffic, where appropriate and physically possible, Figure 6.3. Such a wide overall construction width would pose virtually insurmountable problems in terms of land-take in the physically constrained mountainous environments and even in most low-lying delta regions.

Figure 6.3 CRRN Class A Road with Animal Cart Verge



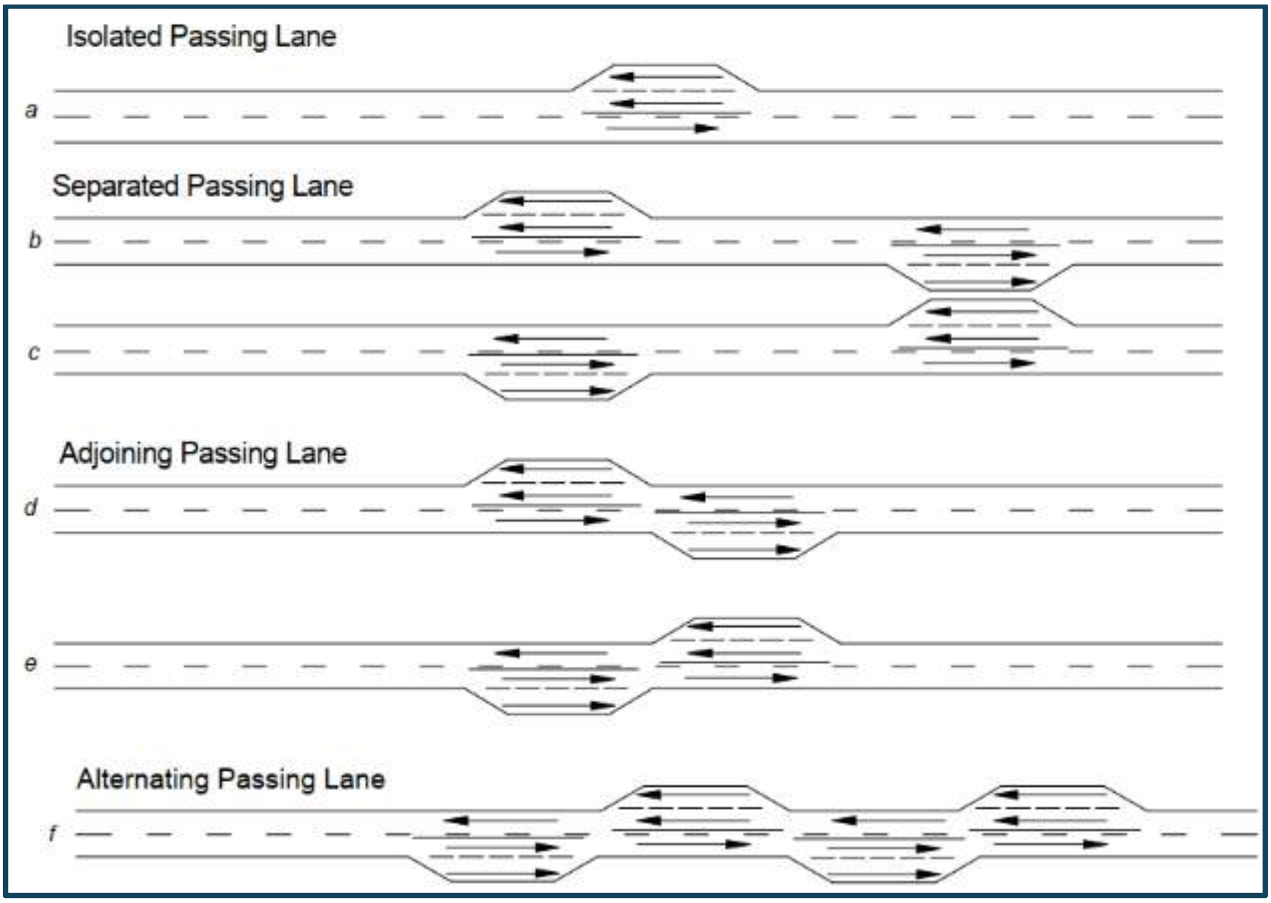
6.3.4 Passing places

Single lane roads may not allow the larger vehicles to pass easily in opposite directions or to overtake slow moving vehicles hence passing places may have to be provided. The use of passing places is a design option for cases where an increase in width over the whole length of an alignment may be unjustified in cost terms or pose significant engineering challenges such over soft soils in delta areas or in steep mountainous terrain.

The increased width at passing places should allow two vehicles to pass at slow speed and hence depends on the design vehicle. For trucks or buses of 2.5m width, the safe minimum is 6.0m. 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 practical places rather than at precise intervals provided that the distance between them does not exceed the recommended maximum.

The length of passing places is dictated by the maximum length of vehicles expected to use the road, again indicating the need to define a design vehicle. In most cases a length of 25m will be sufficient for rural roads. The passing lanes could be configured during the inspection and design stage to select the suitable layout, i.e. the following shapes. Therefore, a total trafficable 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.

Figure 6.4 Passing Place Geometry Options



6.3.5 Climbing lane

A climbing lane is an "extra" lane that is used for short distances in certain parts of the road to improve safety, ease congestion and prevent delays especially in mountainous areas. Climbing lanes help reduce collisions and backups by providing slower moving trucks and vehicles an additional safe lane to travel in. This reduces conflicts between slower moving trucks and passing vehicles.

Climbing lanes are generally applied as a spot improvement, most often on steep sustained grades that cause heavy vehicles, particularly heavy trucks, to travel at slow speeds. Additionally, safety problems may arise when the reduction in speed of heavy trucks exceeds 10-15 mph along the grade). The volume and percentage of heavy trucks are factors in justifying the added cost of the climbing lane.

The economic justification of a climbing lane must be established using a Benefit-Cost analysis.. The benefits of providing climbing lanes may be greater if:

1. There is a high percentage of loaded trucks in the upgrade traffic stream

2. If the geometry of the road, prior to the grade, is very restrictive for passing, thus resulting in high demand for passing.

Table 6.3 may be used as a quick reference to determine if the speed reduction warrant is met on a particular grade. The truck performance curves should be used together with other considerations to determine the exact start and endpoint of the climbing lane.

Table 6.3 Climbing Lane Options

Design Vehicle Mass/Power Rating Metric Imperial	Grade in Percentage						
	2	3	4	5	6	7	8
60 g/W[2] (100 lb/hp)	N/A	N/A	740	410	240	190	180
120 g/W (200 lb/hp)	N/A	N/A	440	280	240	200	160
150 g/W (250 lb/hp)	730	360	280	220	170	140	
180 g/W (300 lb/hp)*	550	340	260	210	160	120	
200 g/W (325 lb/hp)	520	320	260	210	160	120	

(Critical Length of Grade in Metres for a Speed Reduction of 15 km/h)

Notes:

1- A heavy truck is one with a 180g/W ratio

2- Length of specified grade at which the designated design vehicle speed is reduced by 15 km/h from its entry speed (entry speed assumed to be 95 km/h)

3- Conversion factor: 1 g/W = 1.645 lb/hp

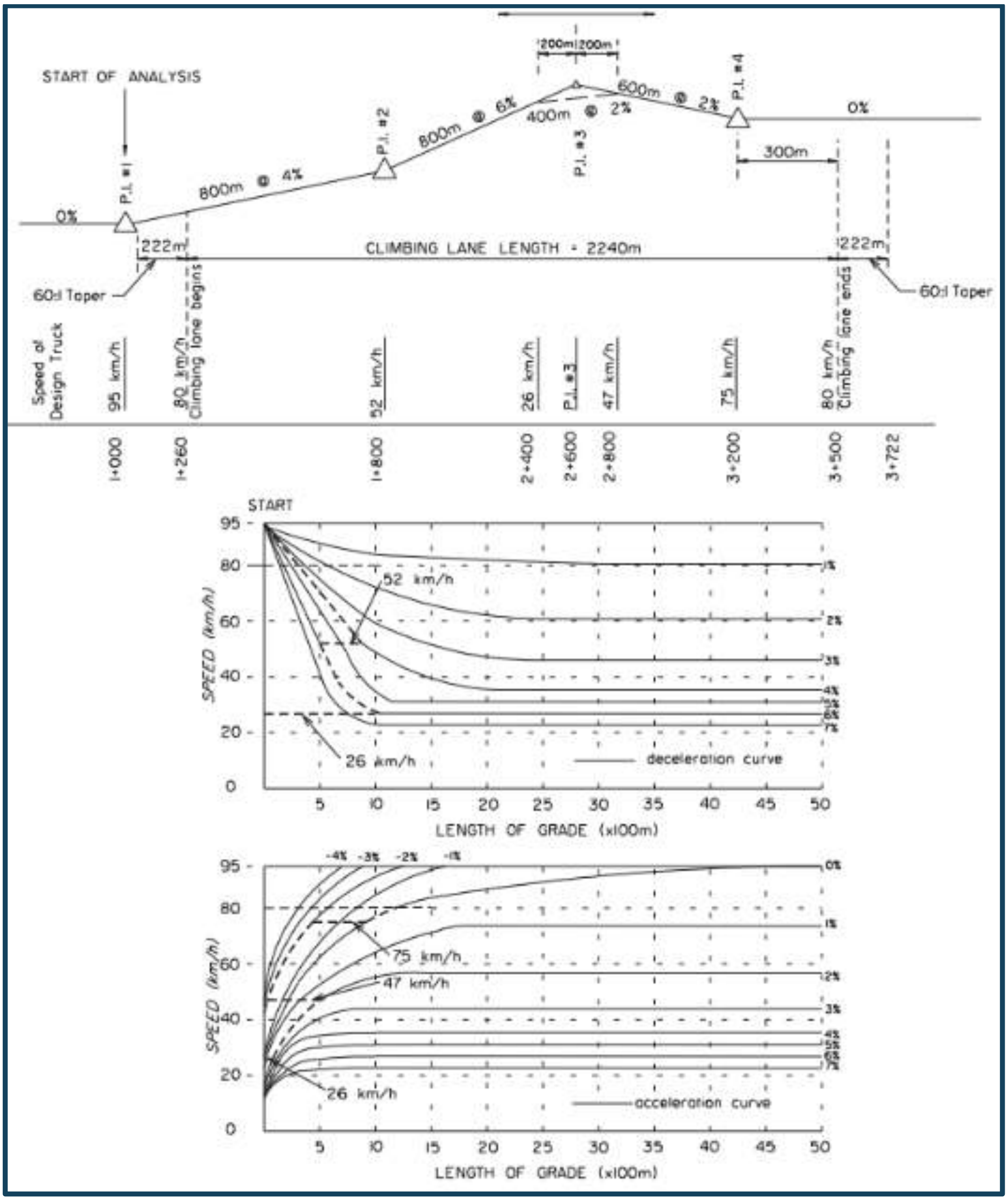
4- Values shown above have been rounded.

5- *180 g/W are normally used for 2 lane roads.

Once the need for a climbing lane has been established by satisfying the speed reduction and traffic volume warrants, the exact start and endpoints and length are determined using the truck performance curves. The following example illustrates the use of truck performance curves.

[2] g/W= gram/Watt (ratio mass/power)

Figure 6.5 Climbing Lane Configurations



The vertical alignment and truck performance curves are shown on the figure above. The design truck is assumed to have a mass/power ratio of 180 g/W, as this is the standard truck. The dashed lines superimposed on the performance curves of the figure show the plot of the design truck speed throughout the alignment section as follows.

1. Entry speed = 95 km/h (assumed) at PI #1 (point of intersection)
2. Truck decelerates to 52 km/h at PI #2 due to 800m upgrade at four percent
3. Truck decelerates to crawl speed (26 km/h) due to 600m upgrade at six percent

The design truck now experiences a grade change whose algebraic difference exceeds four percent; that is, +6% - (-2%) = +8%.

When the algebraic difference exceeds four percent, the vertical curve connecting the grades is approximated through the average grades connecting the quarter points on the semi-tangents of the vertical curve. These quarter points act as new PI's for the purpose of estimating the design vehicle speed. In this example, the length of the vertical curve is 800m. Therefore, the quarter points occur at 200m on either side of the real PI and the grade connecting the quarter points has been estimated at two percent. This approximated grade, 400m in length, reduces the length of the preceding and following grades by 200m each. (The dashed line now enters the acceleration portion of the chart, as the design truck accelerates on the two percent upgrade).

1. Truck accelerates from crawl speed (26 km/h) to 47 km/h on the 400m, two percent upgrade
2. Truck accelerates from 47 km/h to 75 km/h at PI #4 on the 400m, two percent downgrade
3. Truck accelerates from 75 km/h to 80 km/h (the merge speed) on a 300m, zero percent grades.

As per the plot shown on the figure, the climbing lane should begin when the design truck speed reaches 80 km/h (this occurs at 1+260). The 60:1 taper should be introduced before this point. The endpoint of the climbing lane can be placed anywhere after the merge speed has been achieved, that is, after 3+500, provided that the decision sight distance is available. The merge taper is placed after the end of the climbing lane.

6.4 Design Speed and Geometric Design

6.4.1 Principles

Design Speed is the maximum safe speed that can be maintained over a specified section of the road. It should be logically compatible with respect to the alignment topography, the adjacent land use, and the functional classification of the road.

The concept of design speed is useful as it allows the key elements of geometric design to be selected in a consistent and logical way. Design speed is relatively low in mountainous terrain to reflect the necessary reductions in standards required to keep road costs to manageable proportions. It is higher in rolling terrain and, probably, highest of all in flat terrain. The question that must be answered by the designer is simply the selection of design speed for each environment.

The existing design speed guidance in Myanmar is supplied by the MoC, as shown in Table 6.4. Whilst in this table it may be assumed that LVRRs would fit into the “Local Road” category there will be LVRRs where lower designs speeds are necessary and the MoC guidance states that a reduction of 20km/hr can be used if appropriate (MoC, 2015).

Table 6.4 Design Speeds (Extracted from MoC, 2015)

Terrain	Design Speed km/hr		
	Local Road	Collector Road	Sub-Arterial Road
Flat	60	70	80
Rolling	50	60	70
Mountainous	40	50	50

6.4.2 Stopping sight distance

In order to ensure that the design speed is safe, the geometric properties of the road must meet the 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 brow of a hill (vertical alignment) but if there are objects near the edge of the road that restricts 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 the brakes need time to slow the vehicle down hence stopping sight distance is extremely dependant on the speed of the vehicle. Finally, the surface characteristics of the road influence 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.

In order to calculate the stopping sight distance, assumptions have to be made about all of the above factors. Table 6.5 shows the range of values that have been proposed in number of guidelines (ARRB, 2001, TRL, 1988, SEACAP, 2009).

Table 6.5 Stopping Distance Criteria

Parameter	Values used
Drivers reaction time	2.0 – 2.5 seconds
Drivers eye height	1.0 – 1.15m
Object height for stopping	0.1 – 0.2m
Object height for passing	1.0 – 1.3m
Longitudinal friction factor*	0.43 – 0.60

*Depends on speed

As a result of these assumptions, the ranges of stopping sight distances can be derived as shown in Table 6.6. It can be seen that, for each speed, the range is significant. Values towards the higher and the lower end of distance ranges are recommended for unsealed and sealed roads respectively as shown in the Table 6.6.

Table 6.6 Recommended Stopping Distances (m)

Design speed (km/h)	30	40	50	60	70	80
Stopping distance (m)	25-35	35 -55	50 -75	65-100	85-130	115-160
Recommendations: unsealed (m)	35	50	70	93	120	150
Recommendations: sealed (m)	30	40	55	72	95	120

6.4.3 Camber and Cross-fall

Camber, or cross-fall, is essential to promote the removal of water from the road surface. Ponding of water on a road surface quickly leads to deterioration. There is general agreement that camber or cross-fall varies with the road surfacing type from around 2% for concrete roads through 3-4% for sealed roads to 5-6% for unpaved gravel and earth roads.

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 single carriageway, vehicles will generally travel in the middle of the road thus high levels of camber are not such a problem as high levels of cross-fall. The design of unsealed rural roads should make use of this fact so that higher camber is used where appropriate.

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 2% steeper, in other words 5% if the running surface is 3%.

6.4.4 Adverse cross-fall

Adverse cross-fall arises on curves when the cross-fall or camber causes vehicles to lean outwards when negotiating a 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 tires and road surface. For reasons of safety it is recommended that adverse cross-fall is removed where necessary (on all roads regardless of traffic).

Table 6.7 Adverse Cross-fall to be removed if Radii are less than Shown

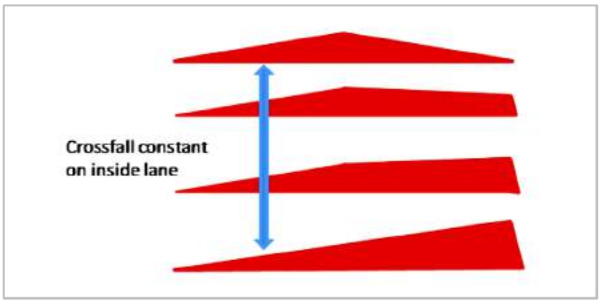
Design speed (km/h)	Minimum radii (m)	
	Paved	Unpaved
<50	500	700
60	700	1000
70	1000	1300
85	1400	*
100	2000	*

*Design speeds not applicable for unpaved roads

Some cross-fall is necessary for drainage as flat sections are not acceptable. Instead, a single value of cross-fall is designed in the proper direction (i.e. all camber is removed) 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 values of 5-6%.

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 section of the curve.

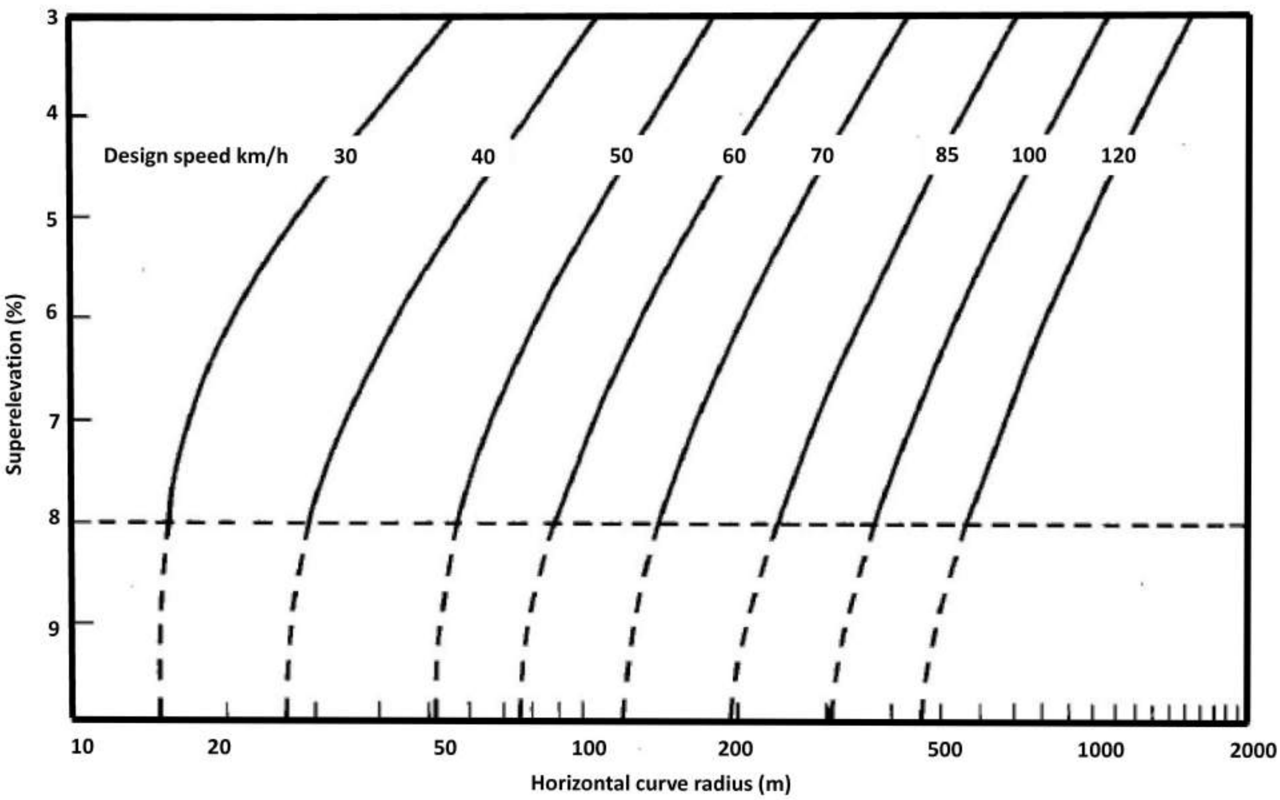
Figure 6.6 Options Sequence of Removal of Adverse Camber



6.4.5 Superelevation

For small radius curves and at the higher speeds of paved sections of road, the removal of adverse cross-fall alone will be insufficient to reduce sideways frictional needs to an acceptable level and cross-fall should be increased by the application of superelevation. A minimum radius is reached when the maximum acceptable frictional and superelevation derived forces have been developed. These minimum radii values are identified in Figure 6.7 for levels of superelevation of up to 10 percent. Although this maximum percentage is rather arbitrary, it is widely considered to be a value above which drivers may find it difficult to remain centred in lane as they negotiate a bend. On paved roads with unsealed shoulders, the outer shoulder should drain away from the paved area to avoid loose material being washed across the road. On unpaved roads, the cross-fall is designed to remove rainwater quickly and effectively, and will be dependent on local conditions and materials. Values of superelevation lower than the minimum value of the cross-fall (4%) would fail to drain the surface, whilst higher values than 6% will be likely to result in lateral erosion. On unpaved roads, the maximum superelevation will therefore be the elimination of adverse cross-fall to 6%. On superelevated sections the whole of the carriageway is drained to the inner shoulder and side drain. This increases the risk of erosion and therefore consideration should be given to paving.

Figure 6.7 Superelevation Design Curves



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 6.8. Between 50% and 75% of the super-elevation should be achieved by the tangent point; a value of 66% is usually used.

Table 6.8 Superelevation Development Lengths

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

6.5 Alignment curvature

6.5.1 Horizontal Curves

Horizontal curves are determined by two main considerations, namely the design speed and the cross-fall or super-elevation. The friction between the road surface and the vehicle wheels also has an effect hence the minimum values of curvature are higher for unpaved roads than for paved.

Horizontal radii values in the following tables are derived from the average values in international practice. As indicated in the Tables below, 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 6.9 Range of Minimum Values of Horizontal Radii of Curvature for Paved roads

Design speed (km/h)	30	40	50	60	70	80
Minimum horizontal radius for SE = 4% (m)	32	60	97	150	210	280
Minimum horizontal radius for SE = 7% (m)	20	40	70	112	170	240
Minimum horizontal radius for SE = 10% (m)	18	35	63	97	145	210

Table 6.10 Range of Minimum Values of Horizontal Radii of Curvature for Unpaved roads

Design speed (km/h)	30	40	50	60	70
Minimum horizontal radius for SE = 4% (m)	35	67	110	165	235
Minimum horizontal radius for SE = 7% (m)	30	60	100	150	215

Note: SE = Super-elevation

6.5.2 Curve widening

Widening of the carriageway where the horizontal curve is tight may be adopted in some cases 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.

Table 6.11 Widening recommendations

Curve radius (m)	Single lane roads			
	20	30	40	60
Increase in width (m)	1.5	1.0	0.75	0.5

6.5.3 Vertical alignment

The gradient is a major aspect of vertical alignment and is related to vehicle performance and level of service. For the low levels of traffic flow, with only a few four-wheel drive vehicles, the maximum traversable gradient is reported as 20% and two-wheel drive trucks are similarly recorded as successfully tackling gradients of 15%, except when heavily laden (TRL, 1988). Bearing in mind the likelihood of heavily laden small trucks and animal-drawn carts, the rural road standards have a proposed general recommended limit of 10%, but with an increase to 15% for short sections in areas of difficult terrain.

6.5.4 Crest curves

The minimum length of the curve L (m) over the crest of the hill between the points of maximum gradient on either side is related to G (see Figure 6.8) and to the stopping sight distance and therefore to the design speed. The minimum value of the L/G ratio can be tabulated against the stopping sight distance, 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 6.12.

Table 6.12 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	34
Unsealed roads	3	6	11	20	34	53

6.5.5 Sag curves

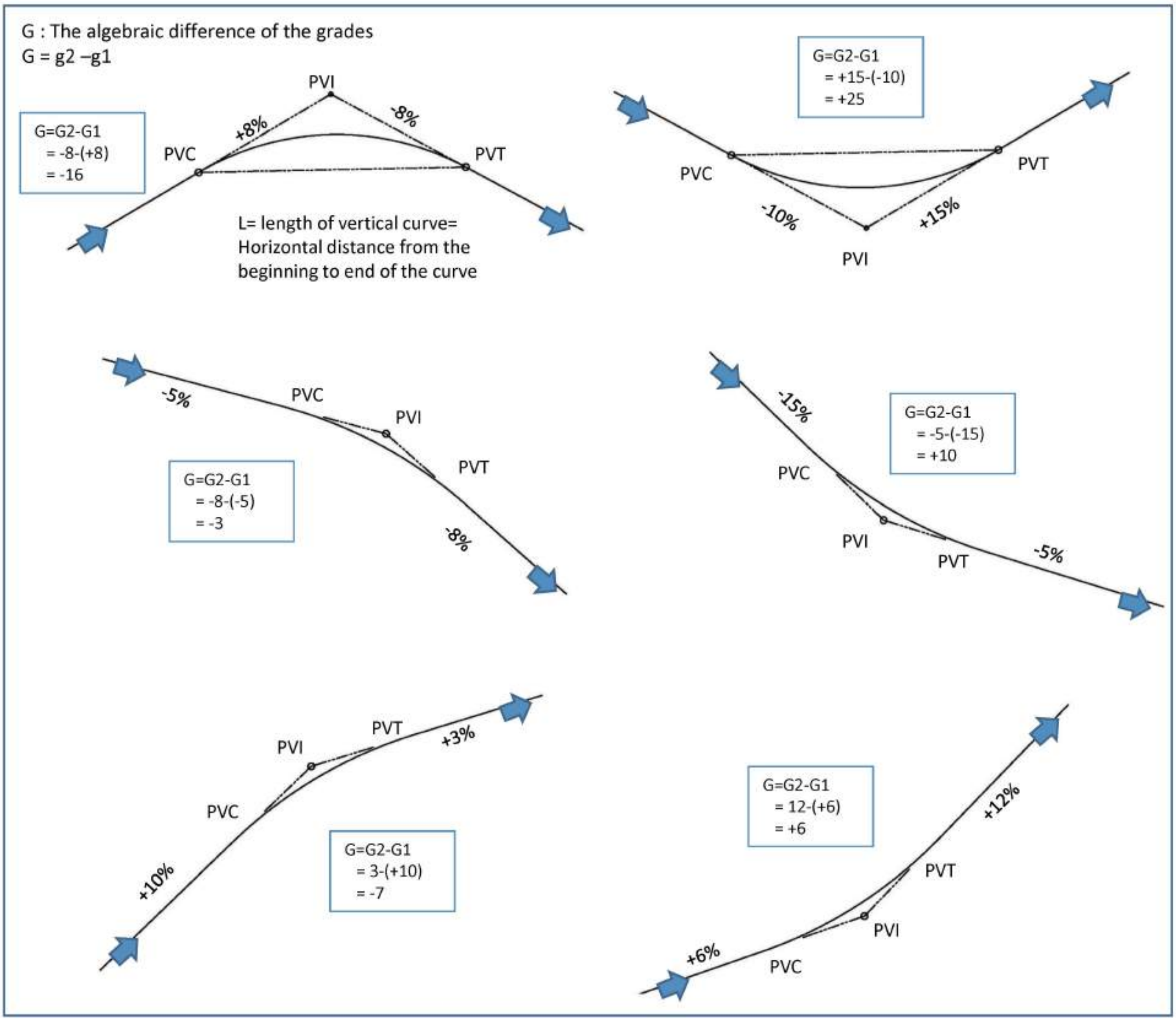
In daylight the sight distance is normally adequate for safety and the design criterion is based on minimising the discomforting forces that act upon the driver and passengers when the direction of travel changes from downhill to uphill. On rural roads such considerations are somewhat less important than road safety issues. However, at night-time the problem on sag curves is the illumination provided by headlights to see far enough ahead. 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 rural roads. 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 6.13.

Table 6.13 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; for example, on a steep down-hill of 10% followed by an up-hill of the same slope, the required minimum curve length is 2.2 x (10 + 10) = 44m at 50km/h and 3.5 x (10+10) = 70m at 60km/h.

Figure 6.8 Sag curves



6.6 Geometric Design Guidance for Myanmar LVRRs

6.6.1 Policy Basis for Recommendations

The guidance in this section is based on a flexible approach to the NSRAA classification of roads for the CRR (Table 6.14) as discussed in Chapter 2.

Table 6.14 Road Classification (NSRAA) and Carriageway Width

Class	Carriageway
A	12 ft (3.66m)
B	12 ft (3.66m)
C	6-12 ft (1.83-3,66m)

6.6.2 General Approach

Based on the design criteria as illustrated above, the following summarized recommendations would be considered for the LVRRs in Myanmar;

1. Use the proposed core CRRN classification system and associated variations as the basis for considering rural road standards.
2. Follow "Desirable" or maximum design standards, and "Acceptable" or minimum design standards as warranted by specific needs and conditions.
3. The decision to use Desirable and Acceptable Rural Road Standards will be dictated by the category of road involved, its operating conditions (traffic volume and type), physical conditions (i.e. terrain), adjacent land use and access needs, and available financial resources.
4. Inclusion of special features such as paving, passing/ climbing lanes, adjacent multi-user paths should be assessed during preliminary design stages and other planning processes and will take into account adjacent development, potential user needs, on-going maintenance requirements and potential drainage issues.
5. Consider the need for facilities to accommodate pedestrians, persons using mobility aids and cyclists on all rural roads, and attempt to provide paved shoulders on rural roads where cycling is prevalent and sidewalks where the roadway leads to a school or community facility.
6. Adoption of the recommendations in the use of local material and labour resources. This will help to relieve constraints due to a shortage of funding and may allow a foreign exchange saving as fewer materials may have to be imported.

6.6.3 Revised Geometric Classification

Based on the "fit for purpose" approach to road design this Manual presents a simple but flexible approach to geometric design based on the particular needs of the Myanmar road environments. Table 6.15 summarises this basic geometric classification.

Table 6.15 Variation on Basic Classification

Class	Basic Carriageway Width (m)	Variations on Carriageway width (m)		Shoulder Width (m)	Default Surfacing	Comment
A	5.5	A1 A2 A3	5.50 4.50 3.60	1.00-3.00	Bitumen sealing or concrete slab	Narrower options for constrained or mountainous locations, with possible passing places Wider shoulders for large % NMT. Possible climbing lanes.
B	3.60	B1 B2 B3	4.50 3.60 3.00	1.00 – 2.00	Bitumen sealing/ concrete slab or unpaved.	Narrower options for constrained or mountainous locations. Wider shoulders for large % NMT. Possible passing places
C	3.60	C1 C2 C3	3.60 3.00 2.50	0.5 - 1.50	Earth or gravel	Narrow options for small vehicle or 2-3 wheel motorised transport.

Table 6.16, Table 6.17 and Table 6.18 provide the supplementary detail for specific classification options.

Table 6.16 Rural Roads Class A

Design Parameter	Comments	Definition		
Terrain		Flat	Rolling	Mountainous
Shoulder width	Depends on traffic mix	1.00-3.00m		1.00m
Carriageway	Traffic and terrain	5.50m	5.50-4.50m	4.50-3.60
Design speed	Defined by terrain	60 km/h	50 km/h	30 km/h
Maximum gradient	Sealed/concrete	6%	10%	12%(1)
Stopping sight distance (m)	Sealed/concrete	55	40	30
Minimum horizontal curve radius (m) SE=4%	Sealed/concrete	150	97	32
Minimum horizontal curve radius (m) SE=7%	Sealed/concrete	112	70	20
Minimum value of L/G for vertical curves	Sealed/concrete	12	7	2
S Minimum value of L/G for sag	Sealed/concrete	3.5	2.2	0.7
Cross-fall	Concrete	2%		
	Sealed	4%		

[1] Gradients up to 15% permitted in cases where lower gradients would incur excessive earthworks and construction cost and where lengths of alignment >10% are kept to <300m. Gradients>12% may require concrete paving.

Table 6.17 Rural Roads Class B

Design Parameter	Comments	Definition		
Terrain		Flat	Rolling	Mountainous
Shoulder width	Depends on traffic mix	1.00-2.50m		1.00m
Carriageway	Traffic, terrain	4.50-3.60m(1)	4.50-3.60m	3.00-2.50m
Design speed	Defined by terrain	50 km/h	40 km/h	30 km/h
Maximum gradient	A limit of 6% for gravel	6%	8%	10%(2)
Stopping sight distance (m)	Gravel	70	50	35
	Sealed/concrete	55	40	30
Minimum horizontal curve radius (m) SE=4%	Gravel(2	110	67	35
	Sealed/concrete	97	60	32
Minimum horizontal curve radius (m) SE=7%	Gravel(3)	100	60	30
	Sealed/concrete	63	35	18
Minimum value of L/G for vertical curves	Gravel	11	6	3
	Sealed/concrete	7	4	2
Minimum value of L/G for Sag	Gravel or sealed	2.2	1.3	0.7
Cross-fall	Gravel	6%(3)		
	Sealed	4%		
	Concrete	2%		

[1] Narrower carriageway in delta areas where river/canal may constrain widths

[2] Gradients up to 15% permitted in cases where lower gradients would incur excessive earthworks and construction cost and where lengths of alignment >10% are kept to <300m.

[3] Gravel cross-fall must be maintained at between 4 and 6%.

Table 6.18 Rural Roads Class C

Design Parameter	Comments	Definition		
Terrain		Flat	Rolling	Mountainous
Shoulder width	Depends on traffic mix	1.00-1.50m		0.50m
Carriageway	Traffic, terrain	3.60-2.50[(1)]	3.60-3.00m	3.00-2.50m
Design speed	Defined by terrain	50 km/h	40 km/h	30 km/h
Maximum gradient	A limit of 6% for gravel	6%	8%	10%[(2)]
Stopping sight distance (m)	Earth/gravel	70	50	35
	Concrete[3]	55	40	30
Minimum horizontal curve radius (m) SE=4%	Earth/gravel	110	67	35
	Concrete	97	60	32
Minimum horizontal curve radius (m) SE=7%	Earth/gravel	100	60	30
	Concrete	63	35	18
Minimum value of L/G for vertical curves	Earth/gravel	11	6	3
	Concrete	7	4	2
Minimum value of L/G for Sag	Gravel or concrete	2.2	1.3	0.7
Cross-fall	Gravel	6%[3]		
	Concrete	2%		

[1] Narrower carriageway in delta areas where river/canal may constrain widths

[2] Gradients up to 15% permitted in cases where lower gradients would incur excessive earthworks and construction cost and where lengths of alignment >10% are kept to <300m.

[3] Concrete option retained for high flood risk zones in delta/coastal areas.

[4] Gravel cross-fall must be maintained at between 4 and 6%.

6.6.4 Relaxation of standards

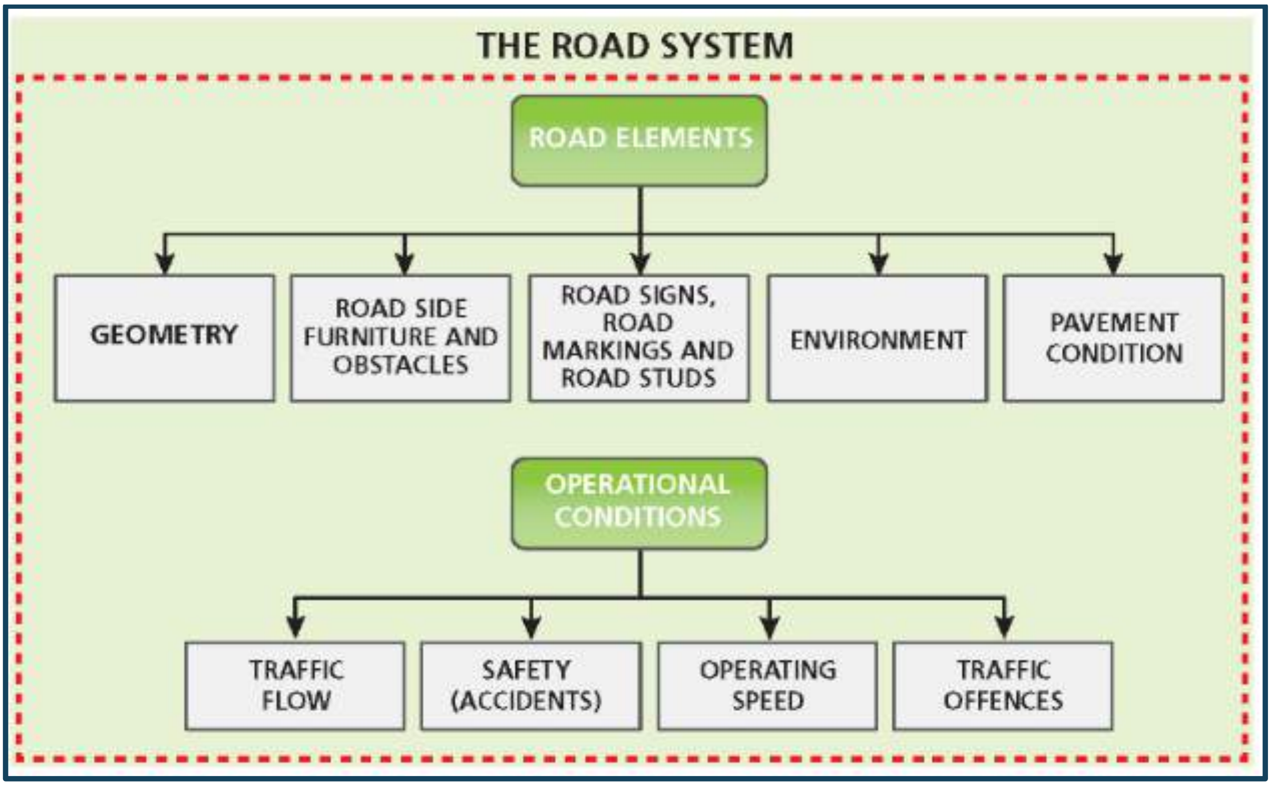
The standards summarised in Tables 6.16 to 6.18 are intended to provide guidance for designers rather than to be considered as rigid requirements. The justification for construction of a particular road usually based, at least in part, on a detailed economic appraisal, and relaxations of standards may be essential in order to achieve an acceptable economic compromise. In other circumstances, an economic evaluation may be improved substantially by the inclusion of a short section with a relaxed standard road where achievement of the normal design standard would be expensive, although the safety implications of this would need serious consideration (TRL 1988).

6.7 Traffic Safety

6.7.1 Importance of Road Safety

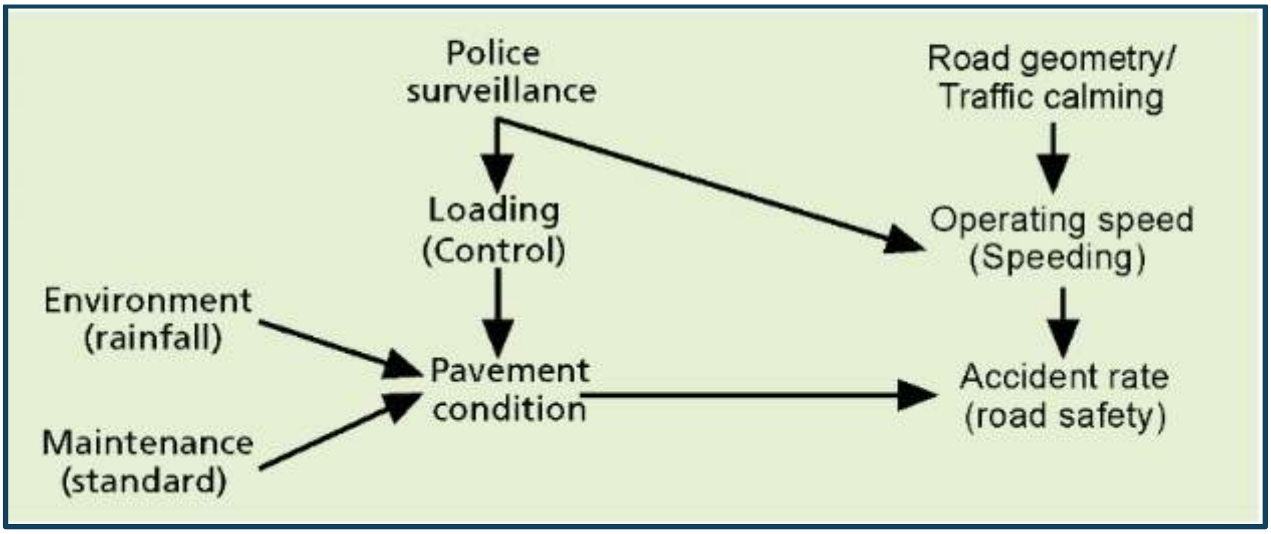
Because of its multi-dimensional nature, road safety cannot be discussed in isolation from a number of related factors. Road safety is linked to and influenced by, various elements of the road system, including road design, environmental conditions, road maintenance and related pavement condition.

Figure 6.9 Safety and the Road System



Many of the road elements and operational conditions are also interrelated. For example, overloading has an important influence on pavement condition and is influenced by police surveillance, while speeding is also influenced by the road geometry and police surveillance. Both overloading and speeding, in turn, have an influence on safety whilst the environment, coupled with the maintenance standard applied, affects pavement condition and, in turn, road safety.

Figure 6.10 Interrelationship between Road Elements and Operational Conditions



Thus, although the importance of designing for safety on LVRRs is now widely recognized, the actual process of identifying key design features and resolving the conflict of safety and other considerations is complex (interrelated) and requires tackling in a holistic manner.

6.7.2 Typical Causes of Road Crashes

The following are typical causes of crashes that occur on LVRRs where many vulnerable road users are put in a high risk situation:

1. Inadequate planning and designing for road safety due in part to the non- inclusion of pedestrian and non-motorised traffic (NMTs) in traffic surveys, and consequent failure to take proper account of the operational environment.
2. The provision of relatively steep cambers, typically 5-7% on gravel roads, in order to shed water off the road. This camber may provide little difficulty to motorized vehicles which tend to travel along the centre of the road. However, it can be very dangerous for cyclists and motorcyclists who often carry very large/heavy loads and, as a result, are unable to easily manoeuvre out of the way of fast-approaching traffic. This often results in them falling off their bicycles, damaging both themselves and their goods.
3. The road alignment outside the longitudinal drainage ditch is seldom cleared by more than a few metres, and, in addition, people often tend to build houses quite close to the road. In these situations, there is potentially a considerably increased risk to pedestrians, particularly young children, as there is little warning to motorized traffic when pedestrians or animals decide to cross the road.
4. A combination of poor motorcycle driver behaviour, such as use of inappropriate speed, and poor road condition, such as a potholed or slippery surface, causing the motorcyclist to lose control.
5. Relatively fast-moving motorized traffic competing for limited road space with much slower-moving non-motorized modes of traffic and pedestrians.

6.7.3 Planning for Road Safety

It is often possible to improve road safety characteristics at markedly little or no extra cost, provided the road safety implications of design are considered at the planning stage. This requires adherence to a number of key principles that include:

- Catering for all road users
  - Includes non-motorized vehicles, pedestrians, cyclists, motorcyclists, disabled persons, etc.
  - Have implications for almost all aspects of road design, including carriageway width, shoulder design, side slopes and side drains.
- Providing a clear and consistent message to the driver
  - Roads should be easily “read” and understood by drivers and should not present them with any sudden surprises which should be addressed by appropriate signage or other measures.
- Encouraging appropriate speeds and behaviour
  - Traffic speed can be influenced by altering the “look” of the road, for example by providing clear visual clues such as changing the shoulder treatment or installing prominent signing.
- Reducing conflicts
  - It cannot be avoided entirely but can be reduced by design, including staggering junctions or using guard rails to channel pedestrians to safer crossing points.
- Creating a forgiving road environment
  - Forgives a driver’s mistakes or vehicle failure to the extent that this is possible without significantly increasing costs.
- Ensures that demands are not placed upon the driver, which is beyond his or her ability to manage.

- Undertaking appropriate traffic counts
  - Ensuring that traffic counts also include pedestrians, bicycles, motorcycles, such information will be influential in planning aspects of the geometric layout of the road, such as shoulder widths.
- Undertaking road safety audits

Because of the paramount importance of road safety on LVRRs, a road safety audit should be undertaken at the planning stage of a new project or before upgrading an existing project. Such an audit should systematically identify hazardous features, including crash “black spots” and the crash potential related to the improvement/upgrading of the road, and should propose treatments that will reduce crash risk to road users. Site specific remedial treatments should be identified and prioritized for early implementation, based on the risks identified at the audit stage.

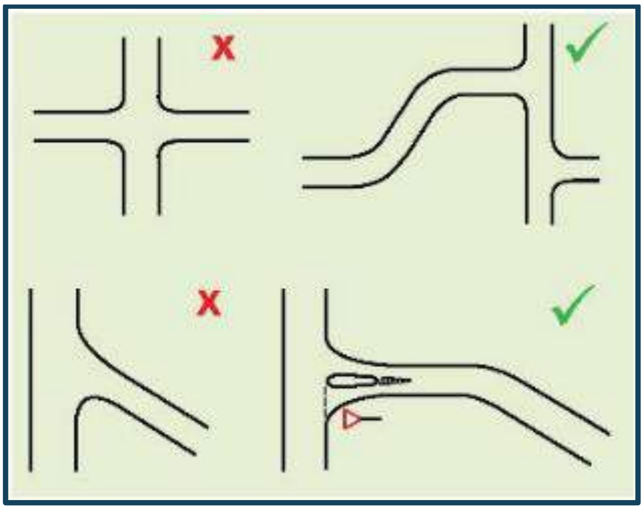
6.7.4 Designing for Road Safety

Appropriate design implies designing for all road users and has implications for almost all aspects of LVRR design, including carriageway width, shoulder design, side slopes and side drains. Specific measures or combinations of measures include:

Junctions

Conflicts may be reduced by staggering junctions or using guard rails to channel pedestrians to safer crossing points. The following figures illustrate some good and poor practices.

Figure 6.11 Good and Bad Junction Designs

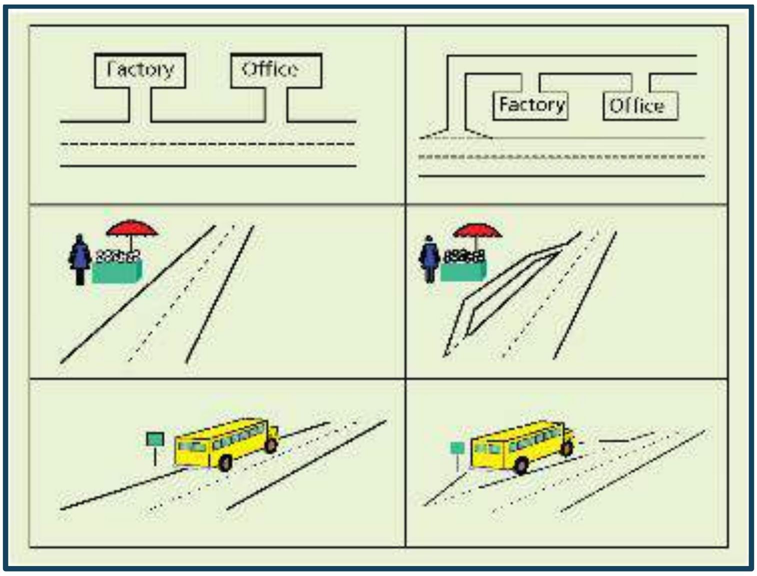


Roadside access

- Road safety in areas that provide roadside access to a variety of facilities may be improved by;
- Prohibiting direct frontal access to major routes and instead use of service roads;
- Using lay-bys or widened shoulders to allow villagers to sell produce;
- Using lay-bys for buses or taxis to avoid restriction and improve visibility.

Examples of the above measures are illustrated in the following Figure 6.12.

Figure 6.12 Examples of improving roadside access to reduce crashes



Clear Zone

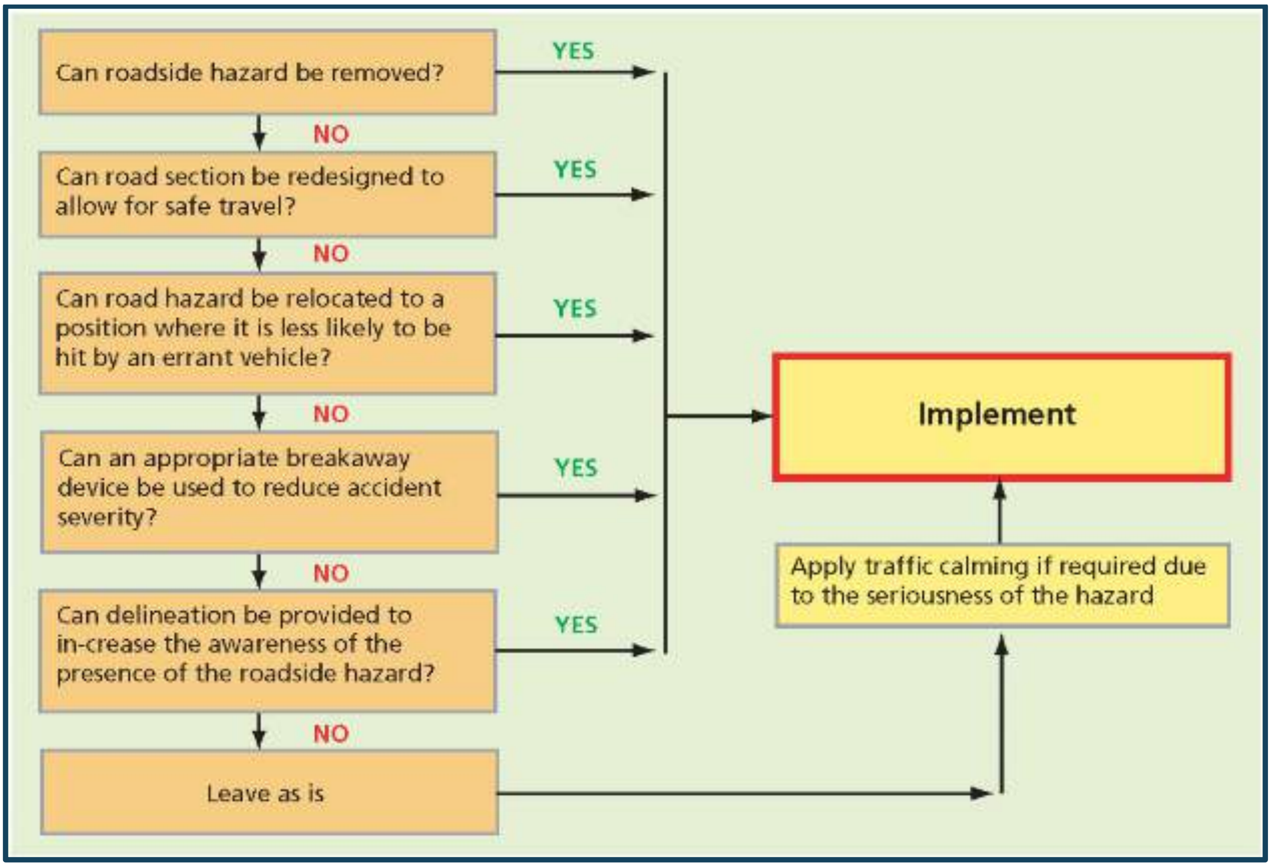
Many crashes are made more severe because of obstacles that an out-of-control vehicle may collide with. The concept of clear zones identifies these obstacles and attempts to eliminate such hazards. The aim is to provide a roadside that is forgiving so that. It enables the driver to avoid colliding with anything, and recover control. This means that there must be an obstacle-free strip – the Clear Zone (Safety Zone or Recovery Zone) – on both sides of the road because vehicles run off the road to both the left and to the right. Shoulders are usually classed as part of the Clear Zone. Features that are regarded as obstacles are:

- Embankment side slopes steeper than 1V:4H. Slopes as steep as 1V:3H may be acceptable provided that there is a clear run-out area at the bottom of the embankment);
- Back slopes steeper than 1V:2H;
- Non-deformable rigid obstacles such as concrete guard posts, bridge piers and abutments, retaining walls, rock cuttings, walls, culvert headwalls, and rigid supports for sign gantries and large signs. Obstacles such as trees, lighting posts, and supports for signs;
- Ditches and open drains (unless designed to be traversable);
- Fences.

Where obstacles in the Clear Zone cannot be avoided, it will be necessary to consider whether they should be shielded by a safety barrier or marked with guard posts.

The following figure provides guidance on dealing with roadside hazards in order to provide a “forgiving roadside”.

Figure 6.13 Procedure for dealing with roadside hazards



6.7.5 Traffic Calming

There is a number of relatively low-cost traffic calming measures that can be introduced, particularly within villages, to reduce vehicle speed and thus improve the safety of road users.

Specific measures include:

- Encouraging police to enforce local speed limits;
- Providing regulatory traffic signs of local speed limits;.
- Calming traffic with speed humps, rumble strips, road narrowing, pedestrian crossings and specially demarcated low speed zones. Traffic calming measures in villages require special attention, for example, in terms of a comprehensive “village treatment” which will induce a driver to reduce speed significantly as he or she passes through a village.

Village treatment – paved roads

The objective of the “village treatment” approach to traffic calming is to develop a perception that the village is a low-speed environment and to encourage the driver to reduce speed as a result of this perception. To this end, the road through the village is divided in three zones, namely:

- The approach zone;
- The transition zone;
- The core zone.

Approach zone: This is the section of road prior to entry into the village, where the driver needs to be made aware that the open road speed is no longer appropriate. This is the section of road where speed should be reduced to about 40 km/h, before entering the village. The village entry should be marked.

Transition zone: This is the section of road between the village entrance, and the core zone of the village. The target speed, and posted speed limit in this zone would be typically 40 km/h. The first road hump or humps in a series of humps will be sited in this zone. In this context, with adequate advance warning provided by the approach zone, road humps are quite safe.

Core zone: This is the section identified as being in the centre of the village, where most of vehicle/ pedestrian conflicts would be expected to take place. This would normally be where the majority of shops, bus-bays or other pedestrian generating activities are located. This is the section where pedestrian crossing facilities are most likely to be established and where the target speed, and posted speed limit, should typically be reduced further to 30 km/h. Road humps would normally be provided within this zone

Road humps

These are the main self-enforcing means of producing a speed reduction. There are two types of humps as follows:

Circular profile hump which has been designed to provide the required reduction in speed while at the same time providing a reasonably comfortable ride for passengers and the least damaging effect on vehicles when traveling at the advisory speed. The specific purpose of the Circular profile hump is to lower traffic speeds so that drivers have little option but to slow down before reaching the core zone. For this reason, the first hump in a series of humps should always be a Circular profile hump, and should always be sited in the transition zone.

Flat-top hump: of which the top portion of this hump is flat with a ramp on either side. The flat-top hump will generally be used at locations within the core zone of the village where there is a need for zebra crossings on popular pedestrian routes (usually near schools, bus stops and markets). In this situation, the hump may be combined with a pedestrian crossing, which would be sited on the flat part of the hump.

Figure 6.14 Road humps

Circular profile hump

Appropriate signage for pedestrian crossing on flat top hump

Unpaved roads

Although traffic levels on unpaved roads in villages generally tend to be lower than on paved roads, traffic speeding, combined with dust emissions, is a major problem for which appropriate traffic calming measures are also required. Such measures are, in principle, similar to those for paved roads in terms of signage. However, special measures need to be taken to embed road humps in the gravel substrate so as to anchor them and minimize their horizontal movement under the action of traffic. Various materials may be used for constructing gravel road humps including cement-mortared brick or stone masonry and stabilized gravel. In addition, where required for cross drainage purposes, the use of raised culverts and drifts, by virtue of their natural profile, can also act as traffic calming devices. In all cases, their location should be well signed which is very important to avoid serious accidents. It should also be noted that speed humps very often lead to accumulation of water and measures need to be in place to avoid this.

6.7.6 Traffic Control and Safety for LWCs

Traffic safety is a principal concern on low-water crossings (LWCs). These crossings present particular safety issues, especially when driving through water and where there are dips in the carriageway vertical alignment.

Because LWCs involve water periodically flowing over the road, they are inherently dangerous during those periods of inundation. Water depth of 0.25m to 0.7m feet has enough lateral force to push a vehicle off the crossing.

To provide for safety where LWCs are used, traffic engineers and resource managers must use prudent design and safety measures (such as traffic warning devices) along with aggressive driver education programs. When common sense indicates that a crossing may be especially hazardous—such as where the road platform is high above water, alignment is poor, speeds are relatively high, and flows are swift and deep. The design should be carefully evaluated and a risk assessment made for the site.

Conventional guardrails and borders, typically 0.75 to 1.0m high, cannot be placed along most low-water-crossing structures because they will act as trash racks during overtopping, and are likely to be damaged during high flows. This Manual recommends low curbs, borders, or delineators for defining the carriageway, identifying the edge of the structure, and keeping traffic on the structure, particularly where the structure is raised. For safety and to minimize flow and debris obstruction timber curbs may be used,. Use object markers to define each corner of the structure, but place them out of the active flow channel to avoid snagging debris.

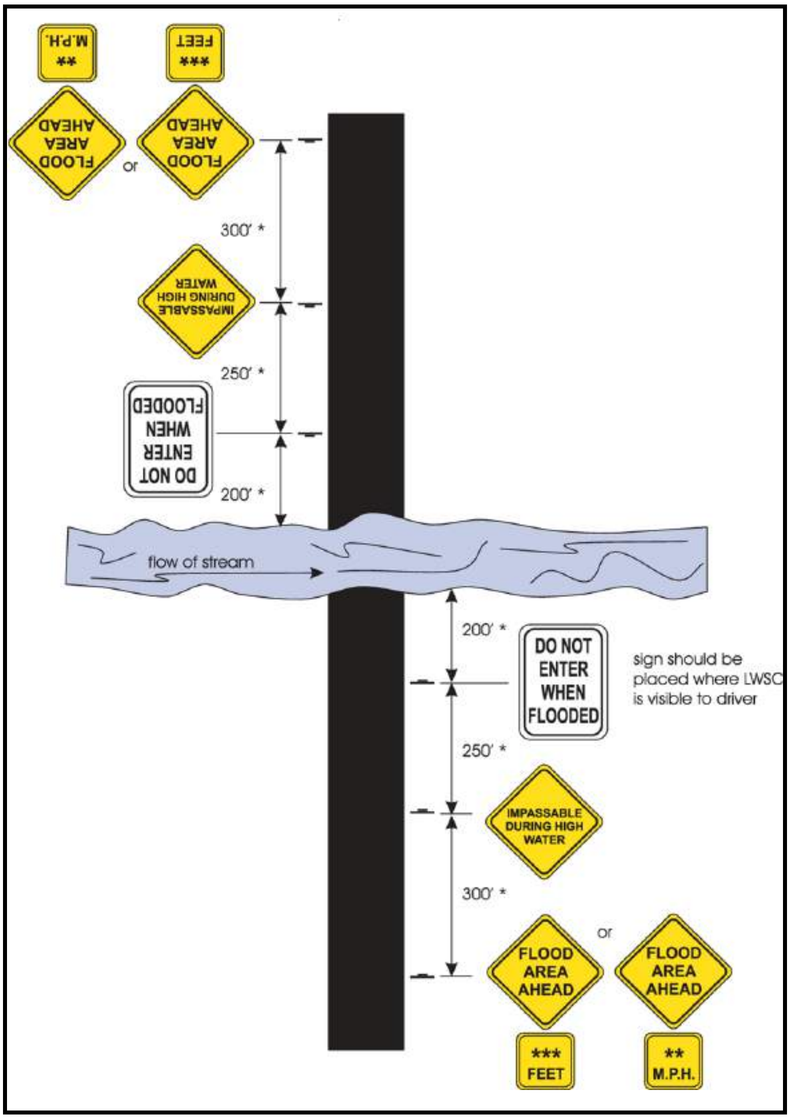
The need for safety measures increases with the height of the structure, particularly on vented fords and low-water bridges where the roadway platform is elevated more than a couple of feet. When conventional bridge railings are not used, the site needs to be evaluated for safety based upon traffic speed, traffic volume, alignment, structure dimensions, other local hazards, and curb design. If the ford cannot be made safe, then a conventional bridge with safety railings or another type of structure should be built.

Ideally, LWCs should be located where the road is straight and sight distance is good. Adequate warning signs are critical for identifying the approaching LWC and warning drivers that the crossing may be flooded and have periodic traffic delays. The safe but conservative design criterion for vented fords that limits water depth over the structure to 0.2m is during the high-design flow. This limit greatly reduces the likelihood of a vehicle being swept away if it enters the water. This criterion, however, may require large vents and is impractical or costly to implement on many unvented fords on rural or forest roads. Therefore, warning devices are a more practical solution in most applications.

Use traffic warning signs along the road, notifying traffic that it is approaching a low-water crossing and that there is the possibility of flooding. Suggested warning signs can include “FLOOD AREA AHEAD,” “IMPOSSIBLE DURING HIGH WATER,” and “DO NOT ENTER WHEN FLOODED”.

Where practical, use depth markers to indicate the depth of flow over the structure. Depth markers may be impractical or require periodic maintenance in channels carrying a lot of debris. Alternatively, a system of coloured posts could be used where flow level green suggests safe passage, flow level yellow suggests marginally safe conditions, and flow level in the red zone indicates an unsafe condition.

Figure 6.15 Signing of Low Water Stream Crossing (LWSC)



6.7.7 Regular Maintenance

It is necessary to ensure that safety hazards are minimized by carrying out regular maintenance of road safety infrastructure. Such measures include:

- Vegetation growth which can obscure visibility, for example, at sharp curves;
- Potholes in the road surface;
- Flooding resulting from blocked culverts;
- Dirty, damaged or missing traffic signs;
- Faint road markings;
- Damaged bridges and guardrails;
- Scoured road shoulders.

Failure to carry out adequate road maintenance can impact adversely on road safety in that it prevents motorcycles and non-motorized users from using the shoulders when required to move off the carriageways due to on-coming motorized traffic.

6.7.8 Road Safety Education

Road safety education is an important tool to raise awareness of problems and behaviour related to traffic and road safety. It involves teaching children, who are often the most vulnerable group of affected road users, and adults to be safer road users. It does so by developing:

- Knowledge and understanding of road traffic;
- Behavioural skills necessary to survive in the presence of road traffic;
- An understanding of their own responsibilities for keeping themselves safe;
- Knowledge of the causes and consequences of road crashes;
- A responsible attitude to their own safety and to the safety of others.

6.7.9 Law enforcement

Traffic law enforcement is meant to achieve the safe and efficient movement of all road users including non-motorized traffic and pedestrians. In this regard, enforcement of traffic rules (such as speed limits, stop signs and rules at pedestrian crossing facilities) can be used to significantly improve road user behaviour and safety. This situation highlights the need to promote traffic law enforcement more vigorously, including the use of well-mounted campaigns which, ideally, should be accompanied by education and publicity.

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7 Surfacing and Pavement Design

7.1 Introduction

7.1.1 Chapter Aims

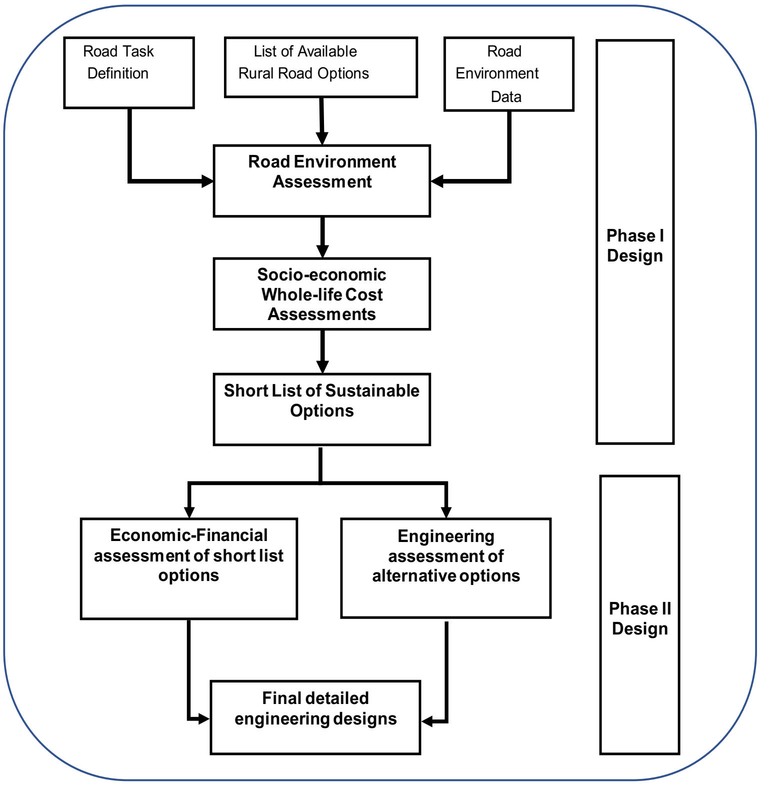
The emphasis in this chapter is on the logical phased selection and design of appropriate pavements. It provides information on the constituents and performance characteristics of the pavement options, an overview of the various types of paving and surfacing available and appropriate for use in Myanmar. The subsequent sections of the Chapter then provide guidance on the design process for the various options.

The Chapter is based on the good practice recommendations and valuable lessons learnt from DRRD projects (DRRD, 2019) and from other Asian countries with similar environments (World Bank, 2012, SEACAP, 2006,). Lessons may also be learnt from other LVRR Manuals, such as those for Laos (SEACAP 2009) Tanzania (AfCAP, 2016), South Sudan (AfCAP, 2012) and Malaysia (MPW, 2012). However, particularly in the case of experience outside South or South East Asia, care has to be taken in modifying recommendations to suit specific road environments of Myanmar.

7.1.2 Design Strategy

The overall pavement design strategy is in line with the LVRR principles laid out in Chapter 3 (Figure 3.6) and, in particular, employs a staged approach within the general project cycle. Figure 7.1 summarises the two phase design process; option selection and option detailed design.

Figure 7.1 Two-Phase Pavement Design Process



7.1.3 Pavement Options

There are currently a limited number of pavement options in common use for LVRRs in Myanmar: these are:

- Earth or Natural Surface;
- Gravel wearing course;
- Unsealed macadam;
- Penetration Macadam;
- Concrete (reinforced and unreinforced)

There are, however, now a large number of appropriate unsealed, bituminous and non-bituminous paving and surfacing options available for use on LVRRs (Henning et al, 2005, Intech-TRL, 2006a; Cook, 2012). A wider selection of LVRR design options can, therefore, now be considered suitable for application in the various road environment of Myanmar (Table 7.1).

Table 7.1 LVRR Pavement Design Options

Bituminous Surfacings	Structural Bases and Sub-Bases
Chip Seal, (Single or double)	Water-Bound Macadam (WBM)
Sand Seal, (Single or Double)	Dry-Bound Macadam (DBM)
Otta seal, (Single or Double)	Graded Crushed Stone
Slurry Seal	Natural Sands/Gravel
Cape Seal	Laterite (Ferricrete)
Penetration Macadam	Chemically Stabilised Natural Soil
Pre-Mix	Emulsion Stabilised Sand
Unsealed Unpaved	Mechanically Stabilised Natural Soil
Engineered Natural Surface	Armoured Gravel
Gravel Wearing Course	
Non-Bituminous Paved	Concrete
Hand-Packed Stone	Steel Reinforced slabs
Water-Bound Macadam (WBM)	Non-Reinforced slabs
Stone Setts/Cobble Stones	Cast in Situ Blocks (Hysen Cells)
Fired Clay Brick	
Concrete Brick	

7.2 Unsealed-Unpaved Options

7.2.1 Engineered Natural Surface (ENS)

An Engineered Natural Surface uses the compacted in situ soil at the road location to form a basic surface for traffic. Essential provisions are a compacted camber (4-6%), side drains and an effective drainage system. Typically soils with an in service CBR of a minimum of about 15% can provide a year-round running surface for light motor traffic (Rolt, 2007). 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 Environmentally Optimised Design (EOD) strategy to improve their traffic carrying capacity (see Annex VI).

7.2.2 Natural Gravel

One or more layers of natural gravel placed directly on the existing shaped earth formation and compacted with an appropriate (5-6%) surface camber (ARRB, 2000; Intech-TRL, 2006b). The layers could be mechanically stabilised or blended with other material to improve the material properties. Alignment with steep gradients may need to be improved in situ by upgrading to higher standard surface under a Spot Improvement or EOD strategy.

A Natural Gravel, or Gravel Wearing Course (GWC) surface is often considered as the usual upgrade option for ENS roads where improvement is justified. However, particular care should be taken in considering this option. Local environment factors may restrict the satisfactory use of natural gravel surface for sections of route that are affected by:

- Longitudinal gradient >6-10% (depending on rainfall intensity);
- Annual rainfall >2,000mm;
- Excessive haul distances for initial and maintenance (re-)gravelling;
- Available gravel material does not meet specifications;
- Dust emissions in settlements or adjacent to high value crops;
- Seasonal flooding.

As annual gravel loss rates or costs may be excessive in these cases, other surface options should be considered. (Cook and Petts, 2005)

7.3 Non-Bituminous Paved Options

7.3.1 Waterbound/Drybound Macadam

A Macadam layer 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. (Intech-TRL, 2006b)

In Water-bound Macadam (WBM) the aggregate fines are washed or slushed into the coarse skeleton with water. Dry-bound macadam is a similar technique; however instead of water and deadweight compaction being used a vibrating roller is used. The development of small vibrating rollers has made the use of this technique attractive for labour based 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. WBM may be overlain by a Gravel Wearing Course as an unsealed option, as used on the Shan Plateau region.

7.3.2 Hand-packed stone

Hand Packed Stone is a proven labour-based technique (IFG, 2008a). The surfacing consists of a layer (typically 150 – 300 mm thick) of large broken stones pieces, laid by hand and tightly packed together and wedged in place with smaller stone chips rammed by hand into the joints using hammers and steel rods The remaining voids are filled with sand. The Hand Packed Stone is normally bedded on a thin layer of sand or 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.

7.3.3 Stone setts or cobble stones

Stone sett and cobble stone surfacing are historically well-established labour-based techniques that have been adapted successfully as a robust option on low volume rural roads where there is a good local supply of suitable stone (Intech-TRL, 2006b). These options are suited to homogeneous rock types that have inherent orthogonal stress patterns (such as granite) that allow for easy break of the fresh rock into the required shapes by labour based means 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 (kerbs). 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.

The Cobble Stone technique is similar to Stone Setts, 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. Suitably graded sand is compacted into the inter-block joints

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.

7.3.4 Fired clay brick - 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 engineering quality bricks must achieve certain strength, shape and durability requirements (Intech-TRL, 2006b). The fired bricks are generally laid on edge to form a layer of typical 100mm thickness on sand or sand-cement bedding layer. Sand or sand cement filling is used for the inter-brick joints.. Kerbs or edge restraints are necessary and can be provided by sand-cement bedded and 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.

7.3.5 Concrete brick paving

Concrete block paving may be used for LVRR roads and design and construction requirements in a similar manner to Fired Clay brick incremental paving. (Intech-TRL, 2006b; Cement & Concrete Assoc. of Australia, 1997; Jones & Promprasith, 1991).

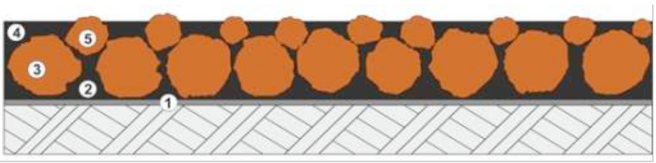
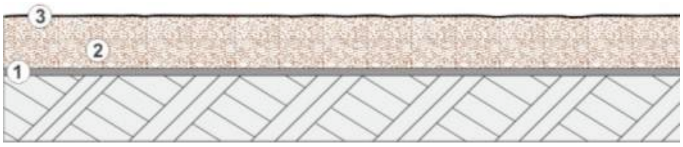
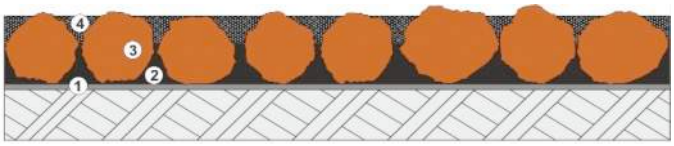
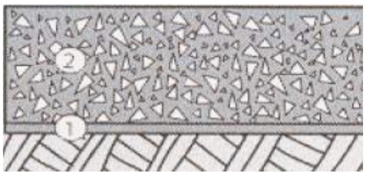
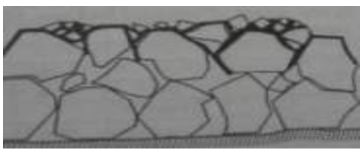
7.4 Bituminous Surfacing

7.4.1 General

Bituminous surfacing or surface treatments generally comprise an admixture of different proportions of stone or sand, and bitumen (AfCAP, 2012a). The bitumen may be a penetration grade, cutback or emulsion option.. The bituminous surfacing usually require good quality, screened or crushed stone or sand, but lower quality aggregate may be used for some types of seals (e.g. Otta Seal).

An effective bond between the surface treatment and the surface of the road base is essential for good performance. This can be achieved through the use of an appropriate grade of bitumen (the prime coat) before the start of construction of the surface layer treatment. Some typical types of bituminous surface treatment are shown in Figure 7.2 and discussed in Sections 7.4.2 to 7.4.8.

Figure 7.2 Examples of Typical Thin Bituminous Surface Treatments

		**Single Chip Seal** 1.Prime coat 2.Bitumen binder 3.Aggregate
		**Double Chip Seal** 1.Prime 2. Bitumen binder (2 layers) 3. Aggregate: 1st coarser spread layer 4. Aggregate: 2nd finer spread layer
		**Sand Seal** 1.Prime Coat 2.Binder squeezed into sand 3.Sand
		**Otta Seal** 1.Bitumen binder 2.Graded aggregate 3.Sand seal
		**CapeSeal/Slurry Seal** 1.Prime coat 2.Bitumen binder 3.Aggregate 4.Slurry seal
		**Pre-Mix (Asphaltic Concrete)** 1. Prime 2. Pre-Mix
		**Penetration Macadam** 1.Coarse macadam 2. Finer macadam 3. Bitumen penetrating into macdam

7.4.2 Sand seal

This seal consists of a spray of bituminous 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 or diversions. For new construction, two layers are usually specified as single layers tend not to be durable. Single sand seals are generally not a recommended option. 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 minimise the risk of bleeding.

7.4.3 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 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. The following are typical uses:

- 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 of low traffic roads;
- A component of a Cape Seal.

7.4.4 Chip seal

This seal (single or double) consists of a spray(s) of bituminous binder (either hot bitumen or bitumen emulsion) followed by the application of a layer(s) of aggregate (stone chippings). These are commonly referred to Single Bituminous Surface Treatment (SBST) or Double Bituminous Surface Treatment (DBST). 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 DBST);
- Temporary by-passes or diversions (normally SBST);
- Maintenance resealing (normally SBST);
- First layer of a Cape Seal.

7.4.5 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.

7.4.6 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 relatively hard (high viscosity) binders used in Chip seals. Otta seals can be single or double (two layers).

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 reseals (normally single Otta Seal).

7.4.7 Penetration Macadam

Penetration macadam is a well understood sealing option in Myanmar and consists of layers of successively finer broken, or crushed rock, interspersed with applications of heated bitumen to infill voids and seal the surface. An initial layer of 40-60 mm aggregate is keyed-in and rolled onto the underlying base. A first penetration of bitumen (commonly at 5-6kg/m2) is sprayed into the initial 40mm aggregate layer and immediately afterwards a second stone application is made by hand onto the grouted aggregate, using 10–

20mm chippings. A second penetration of bitumen is then (commonly 3-5kg/m2) is then sprayed onto the cover layer of chippings The effect is to achieve a matrix of keyed stones grouted and sealed with bitumen to a depth of about 60 – 80mm. It is laid as a surfacing on a previously prepared (typically WBM or DBM) road base (MoC, 1983,IFG, 2008b).

7.4.8 Cold Pre-Mix Asphalt

Cold pre-mix asphalt consists of crushed stone aggregate bound together with Bitumen Emulsion (60 to 65 % slow setting grade, laid in layers, and compacted on a prepared base in 20-30mm layers. Cold mix technology requires no heating and hence low-cost machinery like concrete mixers can be utilised to provide considerably high progress rate (Amino E, & Hongve J, 2013).

7.5 Concrete Surfacing Options

7.5.1 Concrete Slab

Un-reinforced or reinforced cement concrete slab pavements can be used to provide a high strength, durable road surface with very low maintenance requirements. Joints are required to accommodate thermal expansion and contraction and particular care is required to construct them properly. 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 (ARRB, 1993). Cost-wise It will be difficult to justify normal steel reinforced concrete paving for LVRRs, so that design of such pavements is not included in this Manual (Intech-TRL 2006a, b).

Geo-Cell paving comprises plastic geocells being stretched out over a prepared road base or sub-base and pegged in place. The geocells act as an in situ formwork to create an incremental block paving surface by placing and compacting pavement quality concrete into the geocells. The geocells remain as a sacrificial formwork which effectively creates the incremental block paving. The concrete requires to be cured as normal pavement quality concrete (Roughton International 2008).

Geo-Cells is a patented sacrificial formwork system used to cast 3D interlocking articulated block paving and dam liners in situ. The Geo-Cells product is a large mat comprising square, hollow cells fabricated from thin plastic film. The mat is equipped with integral laced rigging supplied ex factory.

Geo-Cell paving has good durability, load bearing and load spreading characteristics and low cost maintenance procedures, and does not require expensive equipment to construct or maintain. Maintenance resources for Geo-Cell pavements are similar to those concrete slab pavements

7.6 Selection of Pavement Layer Options

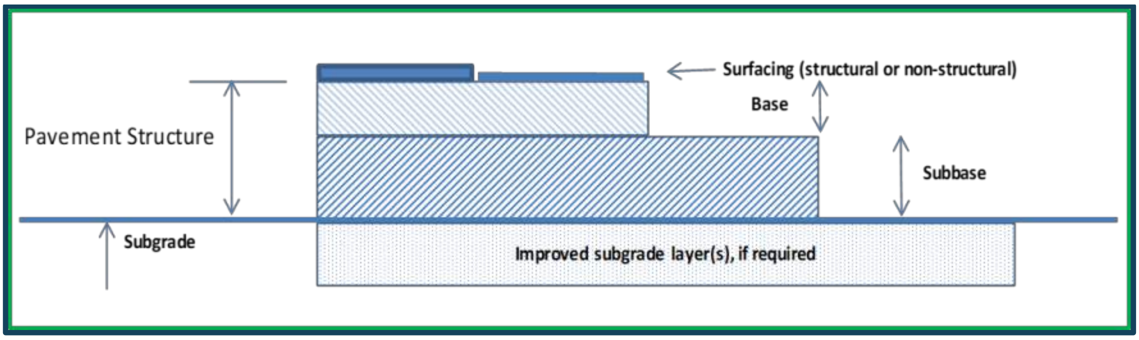
7.6.1 Requirements

The surfacing options will normally require structural support from a combination of the following:

- Natural subgrade;
- Imported subgrade;
- Subbase;
- Base.

Most commonly pavements comprise a both sub-base and base overlying the subgrade, although in some options only a sub-base may be required, Figure 7.3.

Figure 7.3 Typical Pavement Layers

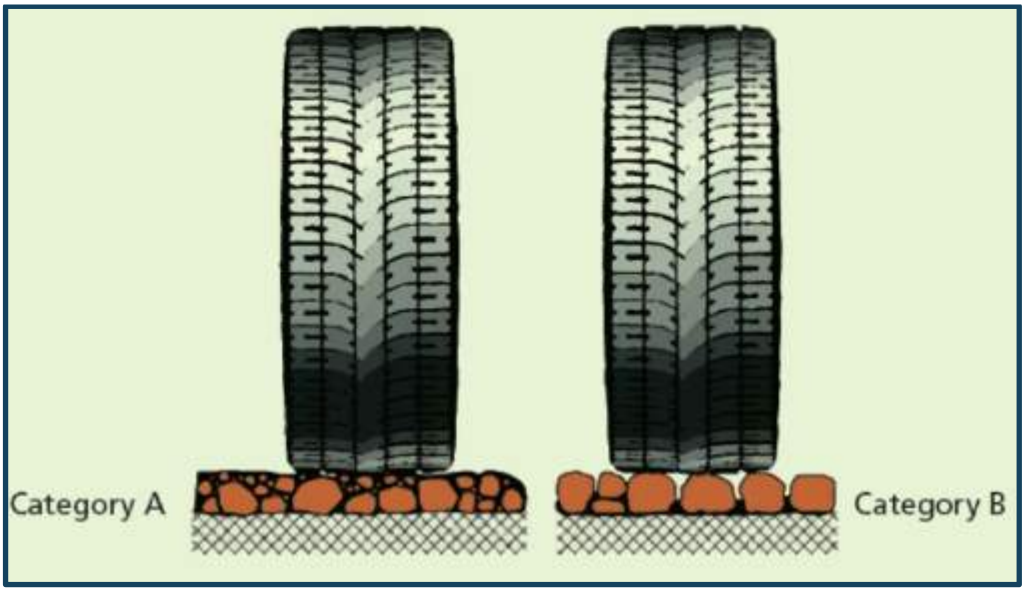


Note: Thin bituminous seals are considered as non-structural, other surfacings such as Asphalt Concrete contribute structural strength.

7.6.2 Performance characteristics of Bituminous Seals

The mechanism of performance of surface treatments varies in relation to the composition of their constituents as illustrated in Figure 7.4 and described below.

Figure 7.4 Differing Mechanisms of Performance of Bituminous Surfacings



Category A: (e.g. pre-mix, sand seal and Otta seal):

These seal types, like cold-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 are necessary to develop the relatively thick bitumen film coating around the particles. Under trafficking, the seal acts as a stress-dispersing mat comprising the bitumen/aggregate mixture.

Catergory B: (e.g. 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 mobilised. 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.

The life of a bituminous surfacing treatment 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. Future climatic increases in temperature need to be considered in bitumen type selection.

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 more porous base 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 (e.g. 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. The impact of heavier traffic may be particularly evident on tight curves especially those on gradients more than about -6%.

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

7.6.3 Factors affecting choice of bituminous surface treatments

The various factors affecting the choice of surface treatments in relation to the operational requirements are indicated in Table 7.2 (Intech-TRL, 2006a).

Table 7.2 Appropriate Surfacing Selection

Parameter	Sub-Division	Type of Surfacing						
		SS	SIS	SCS	DCS	CS	SOS+ SS	DOS
Service life required (years)	Short (<5)	✓	✓	✓	?	?	?	?
	Medium (5-10)	X	X	X	✓	✓	✓	?
	Long (>10)	X	X	X	?	?	?	✓
Traffic level (AADT)	Light (<100)	✓	✓	✓	?	?	?	?
	Medium (100-300)	X	X	X	✓	✓	✓	?
	Heavy (>300)	X	X	X	✓	✓	✓	✓
Impact of traffic turning action	Low (no trucks)	✓	✓	✓	?	?	?	?
	Medium (trucks)	X	X	X	✓	✓	✓	?
	High (3-axle trucks)	X	X	X	?	?	?	✓
Gradient	Mild (<5%)	✓	✓	✓	✓	✓	✓	?
	Moderate (5-10%)	X	X	X	✓	✓	✓	?
	Steep (>10%)	X	X	X	✓	✓	✓	✓
Material Quality	Poor	X	X	X	X	X	✓	✓
	Moderate	?	?	?	?	?	✓	✓
	Good	✓	✓	✓	✓	✓	?	?
Existing pavement and base quality	Poor	X	X	X	?	?	?	✓
	Moderate	?	?	?	✓	✓	✓	✓
	Good	✓	✓	✓	✓	✓	✓	✓
Suitable for labour-based method[3]s		✓	✓	✓	✓	✓	X	X
Contractor experience/ capability	Low	?	?	X	X	X	✓	✓
	Moderate	✓	?	?	?	?	✓	✓
	High	✓	✓	✓	✓	✓	✓	✓
Maintenance capability	Low	X	X	X	X	X	✓	✓
	Moderate	X	X	X	✓	✓	✓	✓
	High	✓	✓	✓	✓	✓	✓	✓

SS	Sand seal	**SIS**	Slurry seal	**SCS**	Single chip seal
DCS	Double chip seal	**CS**	Cape seal	**SOS**	Single Otta seal
DOS	Double Otta seal				

Where;

✓	**= Suitable/preferred**	?	= OK, but maybe not optimal	X	= Not suitable/applicable

[3] Otta seal requires significant amounts of tyred rolling to bring up the bitumen

Pavement Option Selection

The selection of pavement options can be considered as a filtering process as demonstrated in the following Tables 7.3 to 7.6 (Intech-TRL, 2006a).

Table 7.3 Preliminary Engineering Filter – Paving Materials

Required Materials	PAVING CATEGORY														
	Engineered Natural Surface	Gravel Surface	Waterbround/Drybound Macadam	Hand Packed Stone	Stone Setts, Cobble Stone or Pavé	Mortared Stone	Dressed Stone/Cobble Stone	Fired Clay Bricks	Concrete Blocks	Bituminous Sand Seal	Bituminous Slurry Seal	Bituminous Chip Seal	Cape Seal	Otta seal	Non-Reinforced Concrete
Crushed stone aggregate			✓	✓					✓		✓	✓	✓		✓
Stone pieces/blocks				✓	✓	✓	✓								
Natural gravel		✓												✓	
Colluvial/alluvial gravel		✓												✓	
Weathered rock		✓													
Clay soil								✓							
Sand					✓	✓	✓	✓	✓	✓	✓				✓
Cement						✓		✓	✓						✓
Bitumen										✓		✓	✓	✓	
Bitumen Emulsion										✓	✓	✓	✓		

Table 7.4 Preliminary Engineering Filter - Pavement Layers / Shoulders

PAVING CATEGORY	BASES							SUB-BASES						SHOULDERS			
Required Materials	Waterbound macadam	Drybound macadam	Gravel	Armoured gravel	Cement stabilised soil	Lime stabilisede soil	Emulsion s tabilised soil	Waterbound macadam	Drybound macadam	Gravel	Cement stabilised soil	Lime stabilisede soil	Emulsion stabilised soil	Stone macadam	Gravel	Cement stabilised soil	Lime stabilisede soil
Crushed stone aggregate	✓	✓		✓				✓	✓					✓			
Stone pieces/ blocks																	
Natural gravel			✓	✓						✓					✓		
Colluvial/alluvial gravel			✓	✓						✓					✓		
Weathered rock			✓	✓						✓					✓		
Fired clay bricks																	
Clay soil						✓						✓					✓
Sand					✓		✓				✓		✓			✓	
Cement					✓						✓					✓	
Lime						✓						✓					✓
Bitumen																	
Bitumen Emulsion							✓						✓				

Table 7.5 Engineering Environment Filters Surfacing

	Engineered Natural Surface	Gravel Surface	Waterbround/Drybound Macadam	Hand Packed Stone	Stone or Pavé	Mortared Stone	Dressed Stone/Cobble Stone	Fired Clay Brick, concrete Block	Bituminous Sand Seal	Bituminous Slurry Seal	Bituminous Chip Seal (single)	Bituminous Chip Seal (double)	Cape Seal	Ottaseal (single)	Ottaseal (double)	Non-Reinforced Concrete
Traffic Regime: See Table 7.7																
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 7.8)																
A: low erosion regime	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
B: Moderate erosion regime				✓	✓	✓	✓	✓				✓	✓	✓	✓	✓
C: High erosion regime					✓	✓	✓							✓	✓	
D: Very high erosion regime					✓	✓	✓							✓	✓	

Table 7.6 Secondary Engineering Filters - Pavement Layers / Shoulders

	BASES							SUB-BASES						SHOULDERS				
	Waterbound macadam	Drybound macadam	Gravel	Armoured gravel	Cement stabilised soil	Lime stabilisede soil	Emulsion s tabilised soil	Waterbound macadam	Drybound macadam	Gravel	Cement stabilised soil	Lime stabilisede soil	Emulsion stabilised soil	Stone macadam	Gravel	Cement stabilised soil	Lime stabilisede soil	Sealed
Traffic Regime: See Table 7.7																		
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 7.8)																		
A Low erosion regime														✓	✓	✓	✓	✓
B Moderate erosion regime														✓				✓
C High erosion regime																		✓
D Very high erosion regime																		✓

Table 7.7 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 equivalent vehicles per day including up to 20 medium goods vehicles, with no significant overloading. Typical of a Rural Road with individual axle loads up to 6 tonne.
High	Between 100 and 300 equivalent motor vehicles per day. Accessible by all vehicle types including heavy and multi-axle (3 axle +) trucks, Construction & timber materials haulage routes. Specific design methodology to be applied.

Table 7.8 Definition of Erosion Potential

Road alignment longitudinal gradient	Annual Rainfall (mm)			
	< 1000	1000 - 2500	2500 - 4000	>4000
Flat (< 1%)	A	A	B	C
Moderate (2-6%)	A	B	B	C
High (6-8%)	B	C	C	D
Very High (>8%)	C	C	D	D
A = Low; B = Moderate; C High; D = Very High				

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

7.7 Unsealed LVRR Design

7.7.1 Design of Engineered Natural Surfaces (ENS)

Figure 7.5 Typical Engineered Natural Surface Road Cross Section in Flat Terrain

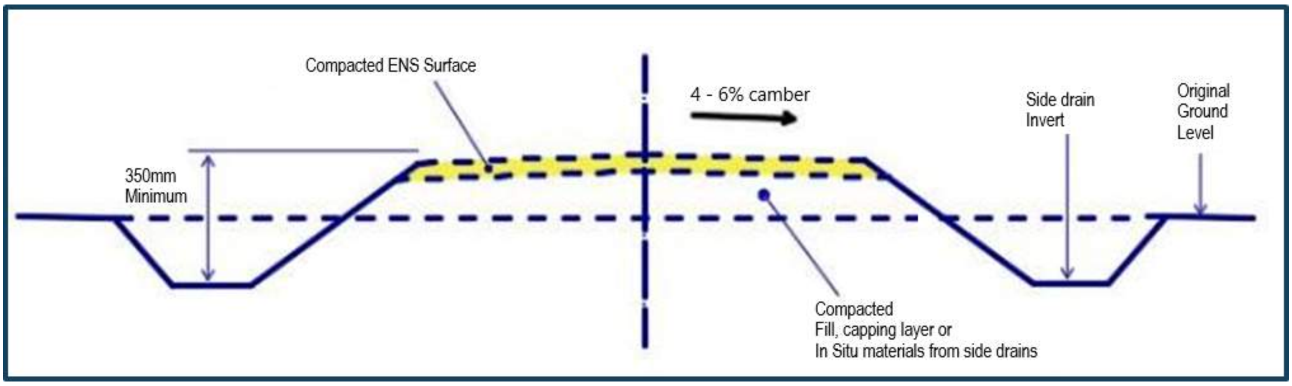
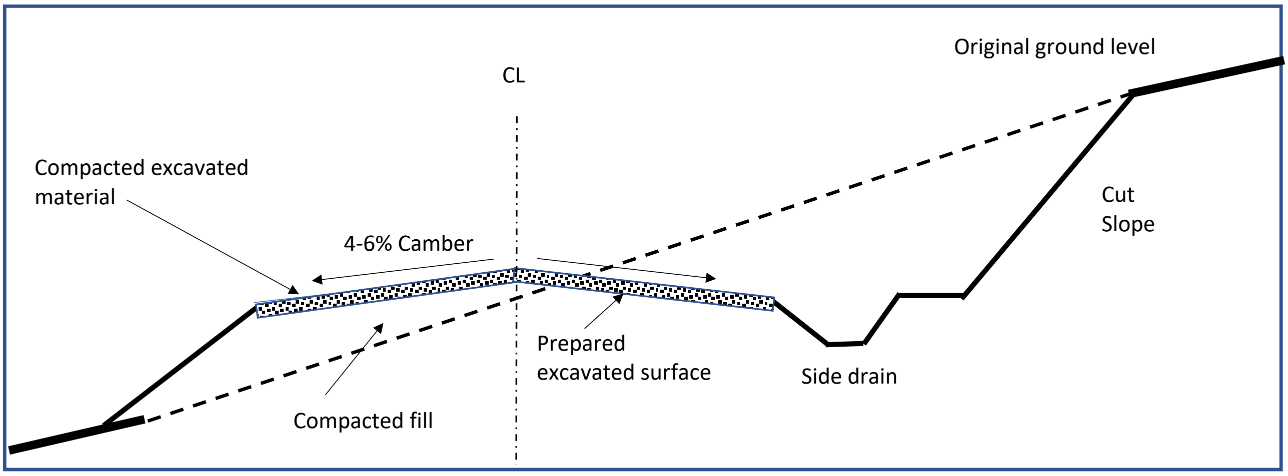


Figure 7.6 Typical Engineered Natural Surface Road Cross Section in Hill Terrain



- The side drain may be trapezoidal, V or rectangular (lined) in shape. The edge of road/shoulder should be at least 30cm above the bed or invert of the side drain, whatever the side drain cross section shape, or the surrounding ground level. These requirements do not apply in sandy free-draining soil or expansive soil subgrades.
- The crown height of the earth road should be at least 50 cm above the bed or invert of the side drain, whatever the side drain cross section shape, or the ground level.
- Where the topography allows, wide, shallow trapezoidal side drains for earth roads are preferred. They minimise erosion risk and will not block as easily as narrow ditches. The ditches usually grass over in time, binding the soil surface and further slowing down the speed of water, both of which act to prevent or reduce erosion.
- The surface of earth roads should be mechanically graded or manually shaped and compacted to provide a suitably robust and running surface for traffic and the road surface should have a minimum camber of 4% to ensure water runs off the surface and into the side drains.
- Areas where there are specific problems (usually due to water or to the poor condition of the subgrade) may be treated in isolation by localised replacement of subgrade, gravelling or other surface upgrade, installation of culverts, raising the roadway or by installing other drainage measures. This is the basis of a "spot improvement" approach
- Water should be drained away from the carriageway side drains by installing lead off (mitre or turn out) drains, to divert the flow into open space away from the road.
- These requirements need to be maintained to keep the ENS in a satisfactory serviceable condition.

7.7.2 Design of Natural Gravel Roads

7.7.2.1 General

A gravel road can be considered to consist of a wearing course and a structural layer (base) which covers the in situ material and provides adequate structural protection for the road foundation. The wearing course will suffer material losses due to traffic and weather and should be regularly reshaped and replenished under the maintenance regime to ensure that the structural gravel layer retains at the minimum design thickness. In practical terms the wearing and structural layers will usually be of the same material and source and be laid in accordance with the requirements discussed hereafter.

Wearing course gravel material losses can be of the order of 25-50mm/100vpd on flat sections of route. Higher losses occur due to factors of rainfall, gradient, poor quality material, or poor (or lack of) maintenance practices.

For the Gravel Road cross section, the side drain may be trapezoidal, V or rectangular (lined) in shape. The edge of road/shoulder should be at least 30cm above the bed or invert of the side drain whatever the side drain cross section shape or the surrounding ground level. These requirements do not apply in sandy free-draining soil or expansive clay subgrades.

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 always maintained at a minimum height (hmin) above the side drain inverts or adjacent ground level, allowing for the variation in wearing course thickness. Cross sections are shown here schematically for convenience (Figure 7.7).

Figure 7.7 Typical Gravel Road Cross Section in Flat Terrain

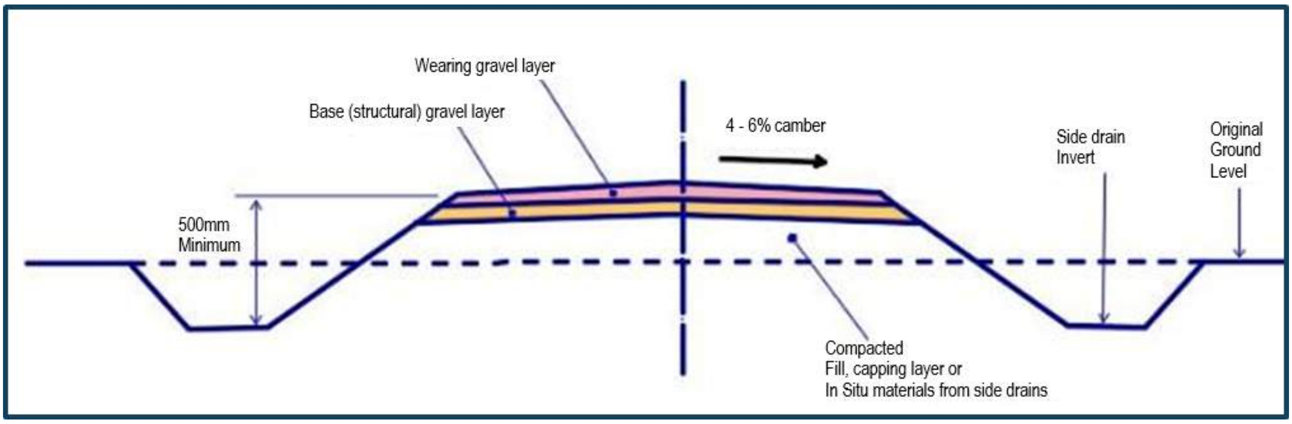
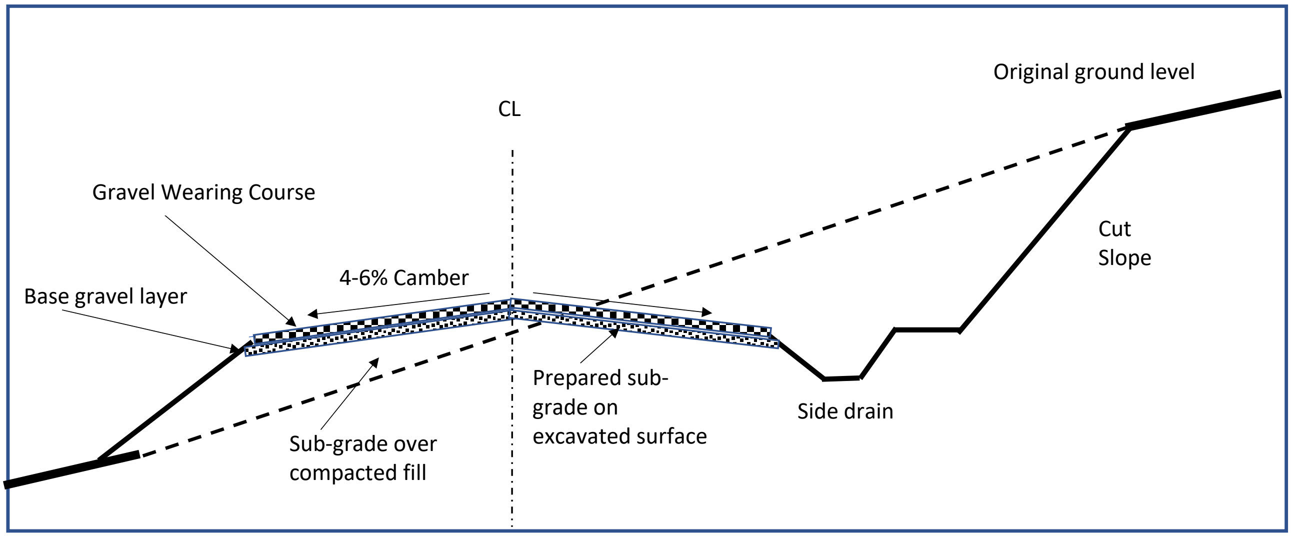


Figure 7.8 Typical Gravel Road Cross Section in Hill Terrain



The design procedure consists of the following steps:

1. Assess likely Maintenance regime.
2. Traffic (Baseline flow and forecast).
3. Material and geotechnical information (Field survey and material properties).
4. Subgrade (Classification, foundation for expansive soils and material strength).
5. Thickness design (Gravel wearing coarse thickness).
6. Materials design.

7.7.2.2 Design method

The required gravel thickness shall be determined as follows:

1. Determine the minimum thickness necessary to avoid excessive compressive strain in the subgrade (Base Gravel layer thickness and any necessary subgrade improvement: D1).

2. Determine the extra thickness needed to compensate for the gravel loss under traffic during the period between re-gravelling maintenance operations (Gravel Wearing Course GWC layer thickness: D2).

3. Determine the total gravel thickness required by adding the above two thicknesses (D1+ D2).

7.7.2.3 Minimum thickness required

It is necessary to limit the compressive strain in the subgrade to prevent excessive permanent deformation at the surface of the road. Figure 7.9 gives the minimum gravel thickness required for each traffic category with the required thickness of the Gravel Wearing Course (GWC) and base layer depending on subgrade strength category. The layer thickness takes account of a maximum placement and compacted thickness of 200mm.

The thicknesses are also based on a suitable gravel with a soaked CBR of >15% (G15). Stronger G25 to G45 gravels may be available for use and in this case some reductions in thickness may be permitted using the following equivalences:

G45 = 1.5 x G15

G25 = 1.3 x G15

A minimum thickness of 150mm should be preserved, thus a 100AADT road gravel with a CBR of 6% could be constructed with 200 and 150mm layers of G15 or with 150mm of G45 material over 130mm of G25 material or 100mm of G45.

Figure 7.9 Minimum Thickness LVRR Gravel Roads

Sub-grade	AADT		
	<20	20-49	50-200
S5-6. CBR>15%	mm 150	mm 175	mm 200
S4 CBR 8-14%	mm 150	mm 150 100	mm 200 100
S3 CBR 5-7%	mm 200	mm 150 150	mm 200 150
S2 CBR 3-4%	mm 200 100	mm 200 150	mm 200 200

7.7.2.4 Gravel loss

The gravel wearing course (GWC) material losses will depend on a range of factors such as:

- Rainfall
- Gradient
- Traffic
- Gravel type
- Maintenance

It is important to assess the likely annual rate of loss to determine maintenance liabilities and ensure that adequate arrangements are in place for the relatively expensive periodic maintenance re-gravelling. The following provides guidance on likely surface gravel loss rates for gravel surface material that is within specification and adequately maintained.

According to TRL Laboratory Report LR 1111, an estimate of the annual average gravel loss can be estimated from the following equation (TRL, 1984):

$$GL = f.(4.2 + 0.092\ T + 3.50\ R^2 + 1.88V).T^2 / (T^2 + 50)$$

Where

GL	=	the annual gravel loss measured in mm
T	=	the total traffic volume in the first year in both directions, measured in thousands of vehicles
R	=	the average annual rainfall measured in m
V	=	the total (rise + fall) as a percentage of the length of the road
f	=	0.94 to 1.29 for lateritic gravels
	=	1.1 to 1.51 for quartizitic gravels
	=	0.7 to 0.96 for volcanic gravels (weathered lava or tuff)
	=	1.5 for coral gravels
	=	1.38 for sandstone gravels

These gravel loss estimates are only indicative, and there is no substitute for analysis of local gravel road performance/maintenance records to develop realistic estimates of actual gravel losses in the local environment. Loss rates will be significantly higher on steep grades. Research on comprehensive sample of road environments in Vietnam supported the above figures on the basis of measured gravel losses indicating gravel loss of greater than the 20mm/y could be taken as the limit of sustainability. (Cook and Petts 2005)

7.7.2.5 Total thickness required

The wearing course of a new gravel road should have a total thickness D calculated from:

D = D1 + N.GL (normally with a minimum of 150mm)

Where

D1 is the minimum thickness from Figure 7.9,

N is the period between re-gravelling operations in years,

GL is the annual gravel loss calculated above and

Maintenance re-gravelling operations should be programmed to ensure that the actual gravel thickness never falls below the minimum thickness D1.

7.8 Design of Bituminous Surfacing

7.8.1 General

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 of materials. Various methods of design have been developed by 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 (TRL, 2000; TRL 2003; AfCAP 2012b). 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.

7.8.2 Prime coat

This is used to provide an effective bond between the surface treatment and the existing road surface or underlying pavement layer and is essential for good performance of a bituminous surfacing. This generally requires that the non-bituminous base layer 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 bitumen such as MC-30, MC-70, or in rare circumstances, MC-250, can be used for prime coats.
- **Emulsion primes**: Bitumen emulsion primes may not be suitable for priming stabilised 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 principally on the texture and density of the surface being primed. Low viscosity primes are necessary for dense cement or lime stabilised 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 7.9.

Table 7.9 Typical Prime Application Rates in Relation to Pavement Surface Type

Pavement surface	Prime	
	Grade	Rate of application (l/m^2)
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

7.8.3 Chip Seal

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*. Additional guidance may be found in AfCAP 2012 and TRL, 2003. 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 references.

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) emulsion;
- Modified binders (polymer modified and bitumen rubber).

Aggregate: The aggregate for a Chip Seal should be durable and free from organic matter or any other contamination. Typical grading requirements for Chip Seals are given in Table 7.10.

Table 7.10 Aggregate Requirements for Bituminous 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 < 1000vpd: 120kN			
10% (wet)	Min 75% of corresponding 10% FACT dry			

Application rates: For planning purposes, typical binder and aggregate application rates for single bituminous Chip Seals are given in Table 7.11.

Table 7.11 Binder and Application Rates for Chip Seals

Item	Double Chip Seal		Single Chip Seal (Reseal)	
	2^{nd} 9.5 mm 1^{st} 19.0 mm	2^{nd} 6.7 mm 1^{st} 13.2 mm	13.2 mm	9.5 mm
Aggregate spread rates (m^3/m^2)				
2^{nd} layer	0.09	0.007		
1^{st} layer	0.015	0.011	0.012	0.010
Hot spray rates of 80/100 pen grade bitumen (l/m^2)				
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

Note: *See also* Table 7.18

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.

7.8.4 Sand seal

There are no formal methods for the design of sand seals with the binder and aggregate application rates being based on local experience. 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 7.12 must be met.

Table 7.12 Grading of Sand for 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 7.13.

Table 7.13 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

7.8.5 Otta seal

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 to traffic and aggregate grading is given in Table 7.14.

Table 7.14 Choice of Otta Seal 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 7.15 gives the hot spray rates for primed base courses.

Table 7.15 Nominal Binder Application Rates for Otta Seals (l/m²)

Type of Otta seal	Grading			
	Open	Medium	Dense	
			AADT < 100	AADT > 100
Double Layer 1st Layer 2nd Layer	 1.7 1.6	 1.8 1.4	 1.8 2.0	 1.7 1.9
Single Layer 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/m2 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 7.16 gives the aggregate application rates for Otta Seals.

Table 7.16 Nominal Otta Seal Aggregate Application Rates

Type of Seal	Aggregate Application Rates (m^3/m^2)		
	Open Grading	Medium Grading	Dense Grading
Otta Seals	0.013 – 0.016	0.013 – 0.016	0.016 – 0.020
Sand Cover Seals	0.010 – 0.012		

The following points should be noted with regard to the aggregate application rates:

1. 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.
2. 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).

7.8.6 Slurry seal

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 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 7.17.

Table 7.17 Nominal Slurry Seal Mix Components

Material	Proportion (Parts)
Fine aggregate (dry)	100
Cement (or lime)	1.0 – 1.5
60% Stable grade emulsion	20
Water	+/- 15

7.8.7 Cape seal

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 7.18.

Table 7.18 Nominal Bituminous Chip Seal Application Rates

Nominal size of aggregate (mm)	Nominal rates of application For planning / tender purposes	
	Binder (litres of net Bitumen cold per m^2)	Aggregate (m3/m2)
13.2	1.4	0.009
19.0	1.6	0.014

7.8.8 Use of Bitumen Emulsion

Bitumen emulsion based applications eliminate many of the hazards of working with hot bitumen; application can be planned more easily to progress at the same pace as the completion of the base and thus eliminate the danger of damages to the base before it is sealed. They are much more labour-friendly than hot bitumen applications and will contribute to a higher local and gender-balanced labour content of the project.

Anionic emulsions can give poor adhesion with acidic rocks (such as granite and quartzite) but good adhesion with basic rocks (such as basalt) and with limestone. Cationic emulsions are suitable for use with both rock types. There are two main grades of anionic and cationic emulsions; rapid, or slow setting and normally contain between 40 and 70 per cent bitumen, with around 65 per cent being the best for most labour-based LVRR work.

Rapid setting (RS) emulsions are best for most surface dressing including penetration macadam; whilst slow setting (SS) is considered best for SS Otta seal, slurry seal and pre-mix.

7.8.9 Cold Pre- Mix Asphalt

Cold Pre-mix Asphalt consists of aggregate and bituminous emulsion material mixed and constructed in accordance with the specifications. Aggregate consists of coarse aggregate of crushed rock, composed of hard, durable particles and a filler of finely crushed stone, sand, or other washed natural gravel. The portion of the material retained on a 5-mm sieve is known as coarse aggregate, and that portion passing a 5 mm sieve as fine aggregate. The total mineral aggregate shall be graded within one of the gradations shown in the table below.

The batching and laying of cold mix asphalts can be tightly controlled and thus ensure a uniform and high quality of the end product. Bituminous materials shall be Bitumen Emulsion (60 to 65 % slow setting grade) satisfying AASHTO M 208. The percentage of bituminous material, by weight, to be added to the aggregate will usually be between 3.5 and 7 percent of the weight of the dry aggregate. The exact percentage to be used shall be based on preliminary laboratory tests and field sieve analyses of the aggregates furnished or in place.

7.9 Structural Design of Paved Roads

7.9.1 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 not only a calibration against empirical data but also a considerable amount of material testing and computational effort and their application to highly variable, naturally occurring materials which make up the bulk of LVRR pavements is questionable. The pavement design method used in this manual is an empirically-based design method.

7.9.2 Design method for paves 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 See section 7.10.5 to 7.12.2.

7.9.3 Traffic and environmental effects:

For a correctly constructed pavement carrying low levels of traffic and low axle loads, 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 bases and sub-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 (300 AADT) range. Annex VII to this Manual discusses the impact of increasing the traffic limit for LVRR design from the current 300 AADT to around 750AADT.

7.9.4 Sealed width

When the total sealed width is 7 metres or less, the outer wheel-track of normal motor traffic 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 650-750mm above the ditch invert, an improvement in pavement performance occurs.

7.9.5 Bituminous Sealed LVRR Design Charts

In some countries with a relatively dry climate a relaxation in specification items is possible; the definition of “dry” in this case being defined having a Weinert “N” value >4. As indicated in section 4.2.2, all Myanmar regions have N values < 4 and can be considered “wet”, Table 7.19, no additional “dry” option charts have been included.

Once the quality of the available materials and haul distances are known, the design chart 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.

The design charts do not cater for weak subgrades (CBR < 3%) and other problems soils. In these cases additional measures should be taken to improve or overlay the weak subgrade (ORN 31; TRL, 1993).

Table 7.19 Thin Bituminous Pavement Design Chart for Structural Layers (mm)

Traffic range (mesas)	Layer	LV1	LV2	LV3	LV4	LV5
Subgrade class (CBR)		< 0.01	0.01 – 0.1	0.1 – 0.3	0.3 – 0.5	0.5 – 1.0
S2 (3-4%)	Base	150 G65	150 G65	150 G65	175 G80	200 G80
	Sub-Base	150 G15	125 G30	150 G30	175 G30	175 G30
	Subgrade		130 G15	175 G15	175 G15	200 G15
S3 (5-7%)	Base	125 G65	150 G65	150 G65	175 G65	200 G80
	Sub-Base	150 G15	100 G30	150 G30	150 G30	150 G30
	Subgrade		100 G15	150 G15	150 G15	150 G15
S4 (8-14%)	Base	175 G45	150 G65	150 G65	175 G65	200 G80
	Sub-Base		120 G30	200 G30	200 G30	200 G30
S5 (15-29%)	Base	175 G45	125 G65	175 G65	175 G65	175 G80
	Sub-Base		125 G30	150 G30	150 G30	150 G30
S6 (>30%)	Base	150 G45	150 G65	175 G65	175 G65	200 G80

Note; G80, G65 etc refer to granular materials with a soaked CBR of 80%, 65% etc as defined in Table 8.2; Subgrade= in situ or imported capping material.

Recent re-analysis of existing data has clearly indicated that savings in terms of material quality can be made, for example by allowing a reduction in base material strength below 80% CBR. The implications of this recent research are discussed in Section 7.11.2.

7.9.6 Penetration Macadam

Penetration Macadam provides structural contribution as well providing a seal. It is normally used only with Dry-Bound or Water-Bound macadam structural layers. The current Ministry of Construction specification document (MoC, 1983) provides guidance on the design and construction of Penetration Macadam using water bound macadam as the underlying structural layers. Table 7.20 below may also be used with a reduction in the thickness of the base layer equal to the 40-60mm contribution of the Penetration Macadam. This design option is not recommended above 300,000 esa.

7.10 Non bituminous surfaced roads

7.10.1 Introduction

Table 7.20 lists the non-bituminous pavement (NBP) options with their respective design charts.

Table 7.20 Non-Bituminous Pavement Surfacing Options

NBP Option	Code Ref.	Table
Water-bound and Dry-bound Macadams	WBM and DBM	7.21
Hand-Packed Stone	HPS	7.22
Stone Setts or Pavé	SSP and MSSP	7.23
Cobblestone / Dressed Stone	CS, DS & MCS, MDS	7.23
Fired Clay Brick	CB, MCB	7.23
Non-reinforced Concrete	NRC	7.24
Concrete Cells		

In many cases the specifications for the strength of these materials is flexible and, depending on the materials available, substitutions can be made.

7.10.2 Water-bound and Dry-bound Macadam (WBM and DBM)

The structural designs for WBM and DBM are as shown in Table 7.21 with the WBM or DBM itself acting as the wearing course. A capping layers and a sub-base are required as indicated but thicknesses can be reduced if stronger material is available.

Table 7.21 Thickness Designs for WBM and DBM Pavements (mm)

Traffic range (mesas)	Layer	LV1	LV2	LV3	LV4	LV5
Subgrade class (CBR)		< 0.01	0.01 – 0.1	0.1 – 0.3	0.3 – 0.5	0.5 – 1.0
S2 (3-4%)	Base	150 WBM	150 WBM	150 WBM	NA	NA
	Sub-Base	150 G30	150 G30	175 G30		
	Subgrade		150 G15	200 G15		
S3 (5-7%)	Base	150 WBM	150 WBM	150 WBM	NA	NA
	Sub-Base	125 G30	125 G30	150 G30		
	Subgrade		100 G15	150 G15		
S4 (8-14%)	Base	150 WBM	150 WBM	150 WBM	NA	NA
	Sub-Base	100 G30	150 G30	200 G30		
S5 (15-29%)	Base	150 WBM	150 WBM	150 WBM	NA	NA
	Sub-Base	NOTE	NOTE	NOTE		
S6 (>30%)	Base	150 WBM	150 WBM	150 WBM	NA	NA
		NOTE	NOTE	NOTE		

Notes:

1. The capping layer of G15 material and the sub-base layer of G30 material can be reduced in thickness if stronger material is available.
2. On subgrade > 15%, the material should be scarified and re-compacted to ensure the depth of material of in situ CBR >15% is in agreement with the recommendations in Table 7.21.

7.10.3 Hand-packed stone (HPS)

The structural designs for Hand Packed Stone with a Sand Bedding Layer (SBL) are as shown in Table 7.22. Hand Packed Stone is a proven labour-based technique. The Hand Packed Stone is normally bedded on the thin layer of sand or 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. A degree of interlock is achieved and has been assumed in the designs shown in Table 7.22.

Table 7.22 Thicknesses Designs for Hand Packed Stone (HPS) Pavement (mm)

Traffic range (mesas)	LV1	LV2	LV3	LV4	LV5
Subgrade class (CBR)	**< 0.01**	**0.01 – 0.1**	**0.1 – 0.3**	**0.3 – 0.5**	**0.5 – 1.0**
S2 (3-4%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	175 G30	125 G30	150 G30	150 G30	
		150 G15	200 G15	200 G15	
S3 (5-7%)	150 HPS	200 HPS	200HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	125 G30	200 G30	150 G30	150 G30	
			150 G15	150 G15	
S4 (8-14%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	30 SBL	
	100 G30	150 G30	200 G30	200 G30	
S5 (15-29%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	NOTE	NOTE	NOTE	NOTE	
S6 (>30%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	NOTE	NOTE	NOTE	NOTE	

Notes: The capping layer of G15 material and the sub-base layer of G30 material can be reduced in thickness if stronger material is available

7.10.4 Stone sett or Cobble Pavements

Each stone sett is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. Sand or fine aggregate is brushed into the spaces between the stones and the layer is then compacted with a roller or plate compactor. Suitable structural designs are shown in Table 7.23.

Table 7.23 Thicknesses Designs for Various Discrete Stone, Brick or Block Surfacings (mm)

Traffic range (mesas)		LV1	LV2	LV3	LV4	LV5
Subgrade class (CBR)		**< 0.01**	**0.01 – 0.1**	**0.1 – 0.3**	**0.3 – 0.5**	**0.5 – 1.0**
S2 (3-4%)	Stone/block/brick	DE	DE	DE	DE	DE
	Bedding	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	100 G65	150 G80	150 G80	150 G80	150 G80
	Sub-Base	100 G30	150 G30	150 G30	175 G30	200 G30
	Subgrade	100 G15	175 G15	175 G15	200 G15	200 G15
S3 (5-7%)	Stone/block/brick	DE	DE	DE	DE	DE
	Bedding	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	125 G65	125 G80	125 G80	150 G80	150 G80
	Sub-Base	100 G30	125 G30	125 G30	150 G30	175 G30
	Subgrade		150 G15	150 G15	150 G15	175 G15
S4 (8-14%)	Stone/block/brick	DE	DE	DE	DE	DE
	Bedding	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	150 G65	150 G80	150 G80	150 G80	175 G80
	Sub-Base		150 G30	200 G30	200 G30	225 G30
S5 (15-29%)	Stone/block/brick	DE	DE	DE	DE	DE
	Bedding	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	125 G65	125 G80	150 G80	150 G80	150 G80
	Sub-Base		125 G30	125 G30	125 G30	150 G30
S6 (>30%)	Stone/block/brick	DE	DE	DE	DE	DE
	Bedding	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	Base	125 G65	150 G80	150 G80	150 G80	175 G80
	Sub-Base	NOTE	NOTE	NOTE	NOTE	NOTE

Notes:

1. DE = Discrete Element layer; block stone (SSP, MSSP, CS, DS, MCS, MDS); clay brick or concrete block
2. The capping layer of G15 material and the sub-base layer of G30 material can be reduced in thickness if stronger material is available
2. The capping layer can be G10 provided it is laid 7% thicker
3. The base layers (G65 and G80) must not be weaker
4. The sub-base layers can be material stronger than G30 and laid to reduced thickness;
5. On subgrades > 15%, the material should be scarified and re-compacted to ensure the depth of material of in situ CBR >15% is in agreement with the recommendations.

7.10.5 Fired clay brick pavement

The surfacing consists of a layer of edge-on engineering quality bricks within mortar bedded and jointed edge restraints, or kerbs, on each side of the pavement. The thickness designs are as shown in Table 7.23 for LV1 and LV2. Fired clay brick surfacings are not suitable for traffic classes above LV2.

7.10.6 Concrete block pavement

This option comprises rectangular concrete bricks (usually around 70mm thick) being laid in a herringbone or other pattern to camber within confining edge-kerbs (cast either before or after brick placement). They are compacted into place, with sand brushed-in at the joints. A sand cement mortar joint or bituminous seal may be specified to be used to waterproof the finished surface as a separate operation, although this is usually unnecessary on a well-constructed sub-base. As a refinement, the concrete bricks may be cast with a top edge chamfer to assist surface drainage.

The thickness designs are as shown in Table 7.23 for LV1 and LV2. Fired clay brick surfacings are not suitable for traffic classes above LV2. Engineering quality concrete blocks are recommended, typically 200x100x70mm thick with minimum 28 day cube strength of 25MPa.

7.10.7 Mortared options

In some circumstances (e.g. on slopes in high rainfall areas and volume susceptible subgrade) it may be advantageous to use mortared options. This can be done with Hand-packed Stone, Stone Setts (or Pavé), Cobblestone (or Dressed Stone), and Fired Clay Brick pavements. The construction procedure is largely the same as for the un-mortared options except that cement mortar is used instead of sand for bedding and joint filling. The behaviour of mortared pavements is different to that of sand-bedded pavements and is more analogous to a rigid pavement than a flexible one. There is, however, little formal guidance on mortared option, although empirical evidence indicates that inter-block cracking may occur. For this reason the option is currently only recommended for the lightest traffic divisions up to LV2 until further locally relevant evidence is available.

7.10.8 Non reinforced concrete (NRC)

The non-reinforced cement concrete option for LVRRs involves casting slabs of maximum 4.0 to 5.0 metres in length or width between formwork with load transfer dowels between them. In some cases, where continuity of traffic demands it, these slabs may be half carriageway width. The thickness designs are given in Table 7.24. The end slab panels of a section of NRC paving should be increased in thickness by 50mm or lightly reinforced with a steel grid to counteract the wheel impact loading as traffic moves onto the end slab from adjacent surfacing.

Contraction joints of 10mm width are required between slabs, to relieve tensile stresses. Expansion joints are required at 200m intervals, with all joints are to be filled and sealed with a mixture of sand and bitumen, with a reservoir of bitumen provided at the top of each joint. All joints are to be provided with load transfer steel dowels.

14 mm diameter mild steel reinforcing bars of 500mm length should be placed at 250mm centres at all expansion and contraction joints. At expansion joints the dowel bar should be anchored into the concrete at one end and the other end coated with bitumen and fitted into a PVC sleeve. The PVC tube is omitted at contraction joints. Concrete slabs should normally be constructed at full carriageway width. In some circumstances (e.g. to allow traffic flow in constrained alignments)) construction of half-width concrete slabs can be used. Longitudinal joints should have a load transferring dowel and sealing arrangement similar to that of transverse contraction joints.

Table 7.24 Thicknesses Designs (mm) - Non-Reinforced Concrete Pavement (NRC)

Traffic range (mesas)	LV1	LV2	LV3	LV4	LV5
Subgrade class (CBR)	**< 0.01**	**0.01 – 0.1**	**0.1 – 0.3**	**0.3 – 0.5**	**0.5 – 1.0**
S2 (3-4%)	160 NRC	170 NRC	175 NRC	180 NRC	190 NRC
	150 G30	150 G30	150 G30	150 G30	150 G30
S3 (5-7%)	150 NRC	160 NRC	165 NRC	170 NRC	180 NRC
	125 G30	125 G30	125 G30	125 G30	125 G30
S4 (8-14%)	150 NRC	150 NRC	160 NRC	170 NRC	180 NRC
	100 G30	100 G30	100 G30	100 G30	100 G30
S5 (15-29%)	150 NRC	150 NRC	160 NRC	170 NRC	180 NRC
	100 G30	100 G30	100 G30	100 G30	100 G30
S6 (>30%)	150 NRC	150 NRC	160 NRC	170 NRC	180 NRC

Notes:

1. Cube strength = 30 MPa at 28 days.
2. On subgrades > 30%, the material should be scarified and re-compacted to ensure the depth of material of in situ CBR >30% is in agreement with the recommendations in the table.

7.10.9 Geo- Cells

Rather than being considered for design purposes as a concrete or rigid pavement this construction should be considered more like a flexible block paving surface where the blocks are cast in-situ. Hence, Table 7.24 provides appropriate guidance. Although Geo-Cells have internationally been constructed with 75mm, 100mm and 150mm thicknesses, the current guidance for Myanmar is to use 150mm thick cells with 30MPa concrete (DRRD, 2019).

Contraction joints of 10mm width are required at 50m intervals in the pavement, to relieve tensile stresses. Expansion joints are required at 250m intervals, with all joints are to be filled and sealed with a mixture of sand and bitumen, with a reservoir of bitumen provided at the top of each joint. All joints are to be provided with load transfer steel dowels.

14 mm diameter mild steel reinforcing bars of 500mm length should be placed at 250mm centres at all expansion and contraction joints. At expansion joints the dowel bar should be anchored into the concrete at one end and the other end coated with bitumen and fitted into a PVC sleeve. The PVC tube is omitted at contraction joints. Concrete slabs should normally be constructed at full carriageway width. In some circumstances (e.g. to allow traffic flow in constrained alignments)) construction of half-width concrete slabs can be used. Longitudinal joints should have a load transferring dowel and sealing arrangement similar to that of transverse contraction joints.

7.11 Design Chart Amendments with Constrained Traffic and Axle-loads

7.11.1 Research

Recent research being undertaken by ReCAP (TRL, 2019) has re-evaluated, or back-analysed, the performance a wide range of existing sealed and unsealed LVRRs, This has indicated that many, well-constructed, sealed LVRRs are performing better than their design anticipated. This follows on from work previously undertaken on sealed roads in Vietnam (Intech-TRL, 2006) and on very low traffic-low axle load designs for Laos (SEACAP, 2008)).

The consequence from this work is that some reductions in layer thickness and materials specifications may be possible in the appropriate circumstances.

7.11.2 Amended Design Charts

Table 7.25 presents proposed amendments to thin sealed LVRR designs as proposed in TRL (2019) and Table 7.26 presents proposed designs for very low traffic and low axle loads in Laos. This latter approach has also emphasised the use of imported capping layer materials as means of reducing base/sub-base thickness. This would prove a possible approach for Class C roads in areas where there is lack of higher quality base or sub-base materials, for examples the Delta Region.

Table 7.25 Revised Design Table for Thin Sealed LVRRs with axles < 8 tonnes

Subgrade Class	Layer	Traffic (mesa): Axle loads < 8 tonnes			
		<0.1	0.1-0.3	0.3-0.5	0.5-1.0
S1 and S2 ≤4%	Base	150 G45	150 G45	150 G60	150 G60
	Sub-Base	125 G25	125 G25	150 G25	150 G25
	Subgrade	125 G15	125 G15	150 G15	150 G15
S3 and S4 5-14%	Base	125 G45	125 G45	150 G60	150 G60
	Sub-Base	125 G25	125 G25	125 G25	125 G25
	Subgrade	125 G15	125 G15	100 G15	100 G15
S5 and S6 15->30%	Base	175 G45	175 G45	175 G45	175 G45

Table 7.26 Revised Design Table for Roads with Very Low Traffic and Axles < 4.5 Tonnes

Subgrade Class	Traffic (mesa): Axle loads < 4.5 tonnes		
	Layer	<0.01	0.01-0.1
2-4%	Base	100 G45	100 G45
	Sub-Base	100 G25	125 G25
	Subgrade	200 G10	250 G10
5-7%	Base	100 G45	100 G45
	Sub-Base	100 G25	150 G25
	Subgrade	100 G10	175 G10
7-11%	Base	100 G45	100 G45
	Sub-Base	100 G25	150 G25
	Subgrade		100 G10
>11%	Base	100 G45	100 G45
	Sub-Base	100 G25	150 G25

7.12 Pavement Drainage

7.12.1 General

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 and 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 9.

7.12.2 Sources of moisture entry into a pavement

The various causes of water ingress to, and egress from, a pavement are listed in Table 7.27.

Table 7.27 Typical causes of water ingress to, and egress from a road pavement

<table>
<tr><th>Means of Water Ingress</th><th>Causes</th></tr>
<tr><td rowspan="2">Through the pavement surface</td><td>through cracks due to pavement failure</td></tr>
<tr><td>penetration through intact layers</td></tr>
<tr><td rowspan="3">From the subgrade</td><td>artesian head in the subgrade</td></tr>
<tr><td>pumping action at formation level</td></tr>
<tr><td>capillary action in the sub-base</td></tr>
<tr><td rowspan="5">From the road margins</td><td>seepage from higher ground, particularly in cuttings</td></tr>
<tr><td>reverse falls at formation level</td></tr>
<tr><td>lateral/median drain surcharging</td></tr>
<tr><td>capillary action in the sub-base</td></tr>
<tr><td>through an unsealed shoulder collecting pavement and ground run-off</td></tr>
<tr><td>Through hydrogenesis (aerial well effect)</td><td>condensation and collection of water from vapour phase onto underside of an impermeable surface</td></tr>
<tr><th>Means of Water Egress</th><th>Causes</th></tr>
<tr><td>Through the pavement surface</td><td>through cracks under pumping action through the intact surfacing</td></tr>
<tr><td rowspan="2">Into the subgrade</td><td>soakaway action</td></tr>
<tr><td>subgrade suction</td></tr>
<tr><td rowspan="2">To the road margins</td><td>into lateral/median drains under gravitational flow in the sub-base</td></tr>
<tr><td>into positive drains through cross-drains acting as collectors</td></tr>
</table>

7.12.3 Permeability

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 (i.e. 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 7.28.

Table 7.28 Typical Material Permeability

<table>
<tr><th>Material</th><th>Permeability</th><th>Description</th></tr>
<tr><td>Gap-graded crushed rock</td><td>> 30 mm/s</td><td rowspan="3">Free draining</td></tr>
<tr><td>Gravel</td><td>> 10 mm/s</td></tr>
<tr><td>Coarse sand</td><td>> 1 mm/s</td></tr>
<tr><td>Medium sand</td><td>1 mm/s</td><td rowspan="2">Permeable</td></tr>
<tr><td>Fine sand</td><td>10 μm/s</td></tr>
<tr><td>Sandy loam</td><td>1 μm/s</td><td rowspan="2">Practically impermeable</td></tr>
<tr><td>Silt</td><td>100 nm/s</td></tr>
<tr><td>Clay</td><td>10 nm/s</td><td rowspan="2">Impermeable</td></tr>
<tr><td>Bituminous surfacing (1)</td><td>1 nm/s</td></tr>
</table>

Source: AfCAP, 2012

Note: 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).

7.12.4 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 and, in particular, the nature of the crown height, A minimum value, h, of 0.65 to 0.75m is recommended for LVRRs; where h is the height of the crown of the road above the bottom of the ditch.

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.

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 discussed below.

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 wheel track and result in catastrophic failure of the base layer when trafficked. A permeability inversion often occurs at the interface between sub-base and subgrade since many subgrades are cohesive fine-grained materials. Under these circumstances, a more conservative design approach is required that specifically caters for these conditions.

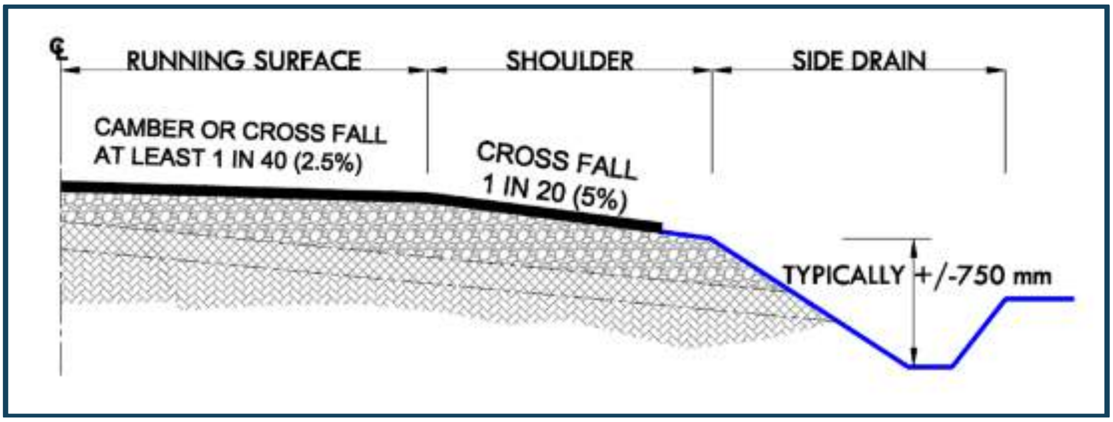
In view of the foregoing, it is desirable for good internal drainage that permeability inversion does not occur. This is achieved by ensuring that the permeability of the pavement and subgrade layers are at least equal or are increasing with depth. For example, the permeability of the base must be less 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 a lateral wetting front does not extend under the outer wheel track of the pavement.

Ensuring proper shoulder design: When permeable base materials are used, particular attention must be given to the drainage of this layer. Ideally, the base and sub-base should extend right across the shoulders to the drainage ditches. In addition, proper crossfall is needed to assist the shedding of water into the side drains. (Chapter 6)

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 relatively inexpensive and therefore worthwhile of consideration, particularly as full under pavement drainage is rarely likely to be economically justified for LVRs. Figure 7.10 illustrates the recommended drainage arrangements for a paved LVR.

Figure 7.10 Recommended pavement drainage arrangements



If it is too costly to extend the base and sub-base material across the shoulder, drainage channels or 'grips' at 3m to 5m intervals should be cut through the shoulder to a depth of 50mm below sub-base level. These channels should be back-filled with material of sub-base quality but which is more permeable than the sub-base/base 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 sub-base. The purpose of such measures should be clearly stated on construction drawings.

Sealing of shoulders: Advantages of sealed shoulders:

- 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 7.10);
- They reduce maintenance costs by avoiding the need for reshaping and re-gravelling 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 may be economically justifiable to provide paved rather than unpaved shoulders. This should be undertaken as part of the design consideration of the pavement cross-section.

Unsealed shoulders: A common problem associated with the use of unsealed shoulders is water infiltration into the base and sub-base for a number of reasons, which include:

- Rutting adjacent to the sealed surface;
- Build-up of deposits of grass and debris;
- Poor joint between the base and shoulder (common when a paved shoulder has been added after initial construction).

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 pavement and shoulders under even light trafficking. This ancient type of road construction is totally unsuited to modern traffic loading. "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 a holistic and integrated approach: The foregoing highlighted pavement drainage measures 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.

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8 Construction Materials

8.1 Introduction

As indicated in Chapter 3, construction materials are a key element of the LVRR road environment and their identification and characterisation are vital factors in the development of cost-effective LVRR designs.

A key objective in sustainable rural road design and construction is to best match the available construction material to the road task and the local environment. When reserves are limited or of marginal quality, their cost-effective usage is a priority and it is important to use materials to ensure that they are neither sub-standard nor wastefully above the standards demanded by their engineering task. Hence the necessity of applying locally relevant specifications and either adapting designs or improving materials to suit. (Cook et al 2001). A common reason for construction costs to escalate once construction has started is 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 materials actually available.

The types of natural road construction materials may be summarised as follows:

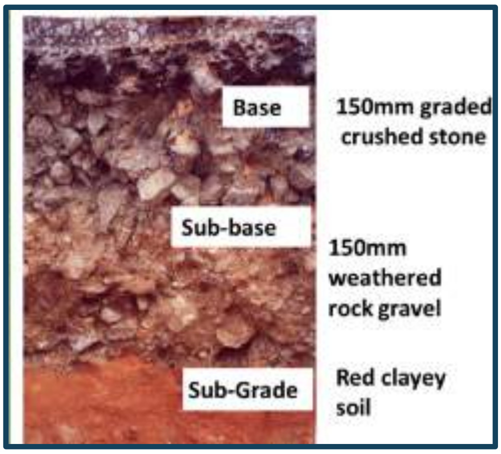
- Common fill
- Imported (selected) subgrade
- Subbase and base aggregate
- Road surfacing aggregate
- Surfacing Block or Paving stone
- Aggregates for structural concrete
- Filter/drainage material
- Gabion fill
- Rock fill (embankments)
- Rip-rap
- Masonry stone

This Chapter gives guidance on the identification and selection of sources of appropriate road-building materials with respect to key characteristics that govern their performance. The associated Annexes II and III provide additional detail on geotechnical testing of construction materials and on the use of marginal materials.

8.2 LVRR Natural Material Requirements

8.2.1 Specification of Suitable Materials

Figure 8.1 Natural Materials in a Typical Sealed Pavement



Construction materials requirements outlined in this chapter have been developed from Myanmar Ministry of Construction specifications reviewed in conjunction with experience from recent and ongoing LVRR projects in Myanmar and elsewhere in the region (MoC 1983).

Table 8.1 summarises key characteristic requirements of LVRR construction materials and Table 8.2 presents definitions for a range of granular materials utilised in LVRR design and construction. To some extent these vary in response to project specific location and road task, as detailed in Chapter 7 for pavements, Chapter 9 for structures and Chapter 10 for Earthworks.

Table 8.1 Key Material Characteristics

Road Use	Ley Specification Criteria
Common Fill	Plasticity, swell potential, compacted strength, max particle size
Imported sub-grade	Plasticity, swell potential, compacted strength, max particle size
Sub-Base/Base	Variable for, crushed stone, natural gravel and laterite Plasticity, swell potential, compacted strength, grading
Gravel Wearing Course	Plasticity, swell potential, durability, compacted strength, grading
Surfacing Aggregate	Fines, grading, shape, particle strength and durability, bitumen adhesion, polishing, abrasion,
Stone Surfacing	Cobbles, hand-packed stone, strength, durability, shape
Filter Material	Fines, grading
Concrete Aggregate	Fine; fines, grading, deleterious inclusions, Coarse: Strength, shape, durability, deleterious inclusions, silica reaction (mineralogy)
Gabion Fill	Strength, durability, size, shape.
Rock fill/	Strength, durability, size, shape
Rip Rap	Strength, durability, size, shape
Mortar Stone	Strength, durability, size, shape

Table 8.2 Material Types and Abbreviated Nominal Specifications

Code	Material	Abbreviated Specifications
G80	Natural gravel or graded crushed stone	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/G60	Natural gravel or graded crushed stone	Min. CBR: 60 or 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/G50	Natural gravel or graded crushed stone	Min. CBR: 50 or 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 or graded crushed stone	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 or graded crushed stone	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/soil	Min. CBR: 30% @ 95/97% AASHTO T180 & highest anticipated moisture content Max. Swell: 1.0% @ 100% AASHTO T180 Max. Size and grading: Max size 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	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	Soil	Min. CBR: 3% @ 93/95% AASHTO T180 & highest anticipated moisture content Max. Swell: N/A Max. Size: 2/3 layer thickness

8.2.2 Common Fill

In general, location and selection of fill material for low volume roads does not pose significant problems. Exceptions include:

- Organic and peaty soils;
- Clays with high liquid limit and plasticity (swelling clays);
- Fabric sensitive soils;
- Highly micaceous soil.

Problems may exist in Myanmar in flood plain or deltaic deposits where very fine compressible 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 for fills should be limited as much as possible both on environmental impact ground, especially in agriculturally productive areas.

8.2.3 Imported Subgrade

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

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 should be avoided due to the additional haulage costs. Import of strong (CBR>10-15%) subgrade materials can also be considered in the context of providing cost saving resulting from a consequent reduction of pavement thickness. Where improvement is necessary or unavoidable, mechanical and chemical stabilisation methods on existing soils can be considered.

As discussed in Chapter 7, subgrades (either in situ or imported) are classified on the basis of the laboratory soaked CBR tests, or the equivalent DCP values. This subgrade strength is normally assigned to one of six strength classes, Table 8.3.

Table 8.3 Subgrade Classes

Design CBR class	S2	S3	S4	S5	S6
CBR range (%)	3 - 4	5 - 7	8 - 14	15 - 29	30+

8.2.4 Base and Sub-base

A wide range of materials including lateritic, calcareous and quartzitic gravels, river gravels and other transported and residual granular materials resulting from weathering of rocks can be used successfully as base and sub-base material. Sub-base and base materials are expected to meet requirements related to maximum particle size, grading, plasticity, and CBR.

The CBR strength required of both the subbase and base will be a function the traffic, the environment (in situ moisture conditions predominantly) and the underlying support (Chapter 7).

The recommended grading envelopes to be used for base are shown in Table 8.4. Envelope C modifies the upper limit of envelope B to allow the use of sandy materials, but its use is not recommended in wet climates. Envelope D is a simple grading definition that can be used for very low volume, low axle load, traffic in both wet and dry climates.

Table 8.4 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					

The plasticity requirement also varies depending on the traffic level as shown in Table 8.5 and Table 8.6.

Table 8.5 Plasticity Requirements for Natural Gravel Base/Sub-base Materials

Subgrade class4	Property of base	Traffic class (mesas)				
		<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. PM: Plasticity modulus
2. Grading 'C' is not normally permitted in the wet Myanmar environments; grading 'B' is the minimum requirement
3. Maximum Ip = 8 x GM
4. Grading 'D' is based on the grading modulus 1.65 < GM < 2.65
5. All base materials are natural gravels; Subgrades are non-expansive
6. Envelope A varies depending on whether the nominal maximum particle size is 37.5, 20 or 10mm

Laterite soils are widely distributed in Lower Myanmar and have been used in the past for road construction and building purposes (Tin Tin Ohn, 1993).

Figure 8.2 Nodular Laterite

Photo J Cook

The requirements for selection and use of lateritic gravels for bases are slightly different to those given for other natural gravels (Table 8.6, CIRIA 1988). The most important requirements for a laterite to show good road performance are that the material is well graded with a high content of hard nodules, or quartz particles with adequate fines content (Figure 8.2). However, when judging the gradation of 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 predominantly kaolinite. Thus, there is a significant difference in the specific gravities of the coarse and fine fractions, and the grading should be calculated taking this into account.

For design traffic levels greater than 0.3 Mesa, a requirement is 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.

Table 8.6 Guidelines for the Selection of Lateritic Gravel Base/Sub-base Materials

Subgrade class	Property	Traffic class (mesas)				
		<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	<15 <250 B	<12 <180 B	<9 <120 A	<6 <90 A
S4	Ip PM Grading	<20 <800 GM 1.6-2.6	<15 <320 B	<15 <300 B	<9 <200 B	<9 <90 A
S5	Ip PM Grading	<25 - GM 1.6-2.6	<18 <400 B	<15 <350 B	<12 <250 B	<9 <150 B
S6	Ip PM Grading	<25 - 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. If sealed shoulders >1m are specified in the design, the PM may be increased by 40 %

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..

For granular sub-bases a minimum CBR of 25-30% is commonly required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95% AASHTO T180 compaction. Materials which meet the recommendations of Table 8.7 and Table 8.8 should perform adequately as sub-bases when used within the recommendations of the design catalogues in Chapter 7.

Table 8.7 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 8.8 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) (not normally occurring in Myanmar)	< 55	< 20	< 10

Under certain circumstances, mechanical or chemical treatments may be required to improve the quality to the required standard. This often requires the use of special equipment or processing plant (Section 8.4).

8.2.5 Gravel Wearing Course

Ideally, an unsealed Gravel Wearing Course (GWC) 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 (suitable amount of plasticity);
- 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 characteristics of the coarse particles. There is also a requirement for there is sufficient plasticity to hold the GWC together. Typical required properties are summarized in Table 8.9.

Table 8.9 Material Specifications for Unsealed Rural Roads

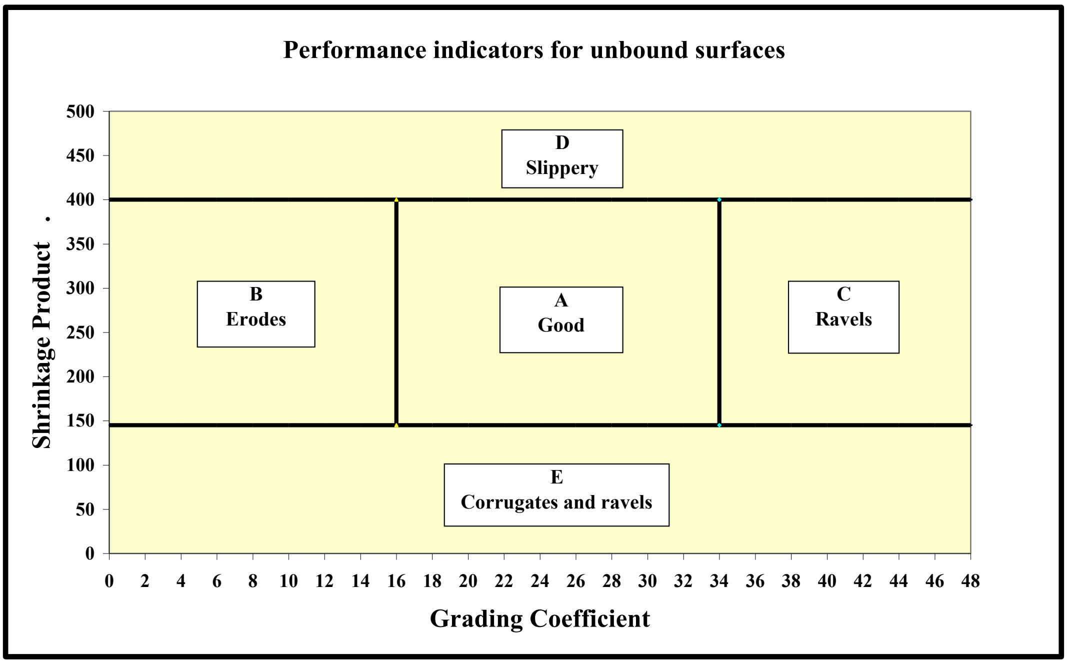
Characteristic	Requirement
Maximum size (mm)	37.5
Oversize index (I_o)	≤5%
Shrinkage product (S_p)	100 to 365 (max.240 preferable)
Grading coefficient (G_c)	16 to 34
Soaked CBR (at 95% Mod. AASHTO)	≥15%
a I_o = % retained on 37.5mm sieve)	
b S_p = Linear shrinkage x passing 0.425 sieve	
c G_c = (% passing 26.5mm – % passing 2.00mm) x (% passing4.75mm)/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.

The likely influence of material properties on GWC performance are summarised in Figure 8.3.

Figure 8.3 Gravel Wearing Coarse 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.

8.2.6 Road Surfacing Aggregate

The requirements for aggregate to be used in a bituminous surfacing layer are that they must be durable, strong and should also show good adhesion with bituminous binders. It should also be resistant to the polishing and abrasion action of traffic. Adhesion failure implies a breakdown of the bonding forces between a stone aggregate and its coating of bituminous binder, leading to physical separation. Mechanical failure by fretting and subsequent ravelling of the surface is one consequence of adhesion failure.

Basic rocks (e.g. basalt) are considered to have better adhesion properties than acidic rocks (e.g. granite). Experience has indicated, for example, that coarse granite with large feldspar crystals is likely to experience bitumen adhesion difficulties.

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

Table 8.10 Typical Requirements for Bitumen Chip Seal Aggregate

Sieve Size (mm)	Nominal Aggregate Size (mm)			
	19.0	14	10	6
	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
Flakiness Index	Max 20	Max 25	Max 25	Max 30
10% FACT (dry)	AADT < 1000 vpd: 120 kN			
10% FACT (wet)	Min 75% of corresponding 10% FACT dry			

Pre-mix coarse aggregate will be the material component fully retained on an 4.75mm sieve and consist of clean crushed rock or crushed gravel or blended combinations of both, free from decomposed stone, organic matter, shale, clay and any other substances and have following characteristics

- Aggregate Crushing Value or AIV of not greater than 30 or LAA value of 40%.
- Bulk specific gravity not less than 2.50

- Flakiness index not greater than 35%
- Sodium sulphate soundness test Weight loss not more than 12%
- Not less than 75% by weight of the particles shall have at least two fractured faces.

Fine aggregate should be composed of clean, hard durable particles, rough surfaced and angular, free from vegetable matter, soft particles, clay balls or other objectionable material.

The mix of the coarse and fine aggregates combined shall comply with the grading given in Table 8.11.

Table 8.11 Typical Grading for Bituminous Pre-mix

Sieve Size (mm)	Passing by Weight %
25	100
20	75-100
12.5	60-80
4.75	35-55
2.4	20-33
0.600	6-18
0.075	2-8

8.2.7 Block or Paving Stone

The block or paving stones should be a strong, homogenous, isotropic rock, free from significant discontinuities such as cavities, joints, faults and bedding planes. Rocks such as fresh granite, basalt and crystalline limestone have proven to be suitable materials. Quartzite rock is generally not suitable, nor is any rock that polishes or develops a slippery surface, or erodes under traffic. Rock for paving stone should be tested to ensure it meets the following requirements;

- Uniaxial compressive strength >75MPa;
- Los Angeles Abrasion value: <25%;
- Sodium Sulphate Soundness <10% loss.

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

8.2.8 Manufactured blocks or bricks

The fired clay bricks should be of engineering standard with the following characteristics:-

- Dimensions: 200x100x70mm;
- Water absorption: <16% of their weight of water after 1 hour soaking;
- Unit weight: >1200 kg/m^3;
- Crushing strength: >25MPa.

Concrete blocks should have dimensions: 200x100x70mm thick composed of concrete with minimum 28 day cube strength of 25MPa and maximum aggregate size of 6mm.

8.2.9 Aggregates for Structural Concrete

Concrete aggregate is divided into two parts: coarse aggregate and fine aggregate. The fine aggregate is normally naturally occurring sand, although fine crushed stone can also be used, with particles up to about 2mm in size. The coarse aggregate is normally stone with a range of sizes from about 5mm to 25mm (or sometimes larger); it may be naturally occurred gravel, or more commonly crushed or hand-broken quarry stone. In areas without hard stone resources and with an established fired clay brick industry, burnt bricks can be machine or hand crushed to be used in concrete (Howard Humphreys, 1998)

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

Both the coarse and fine aggregates need to contain a range of particle sizes; a ratio by volume of one-part fine aggregate to two parts coarse aggregate is generally used. Aggregates can be crushed and screened by hand or by machine. Both reinforced and mass concrete has a range of potential uses within LVRR projects including; pavement slabs, bridges, culverts, retaining walls and drainage linings. Each of these may have particular aggregate specifications. In general, the concrete aggregates should conform to the current Myanmar Standards for concrete aggregate with no uncrushed rounded coarse natural aggregate being acceptable.

Table 8.12 Typical Concrete Aggregate Grading Requirements

AASHTO Sieve (mm)	Coarse Aggregate Nominal Grading: Size Down (mm), % Passing by Weight				Fine Aggregate Grading % Passing by Weight
	37.5 mm	25.0 mm	19.0 mm	12.5 mm	
50	100				
37.5	95-100	100			
25		95-100	100		
19	35-70		90-100	100	
12.5		25-60		90-100	
9.5	0-5		20-55	40-70	100
4.75		0-10	0-10	0-15	95-100
2.36		0-5	0-5	0-5	80-100
1.18					50-85
0.600					25-60
0.300					10-30
0.150					1-10
0.075					0-3

8.2.10 Filter/Drainage Material

Filter materials have crucial roles in assisting in controlling the ingress and flow of water and in the reduction of pore water pressures within earthworks, retaining structures and the pavement. Filter materials can account for a significant proportion of the construction material costs, particularly in wetter regions where road designs need to cater for the dispersion of large volumes of water, both as external drains and as internal layers within wet-fill embankments. The general requirements for filter material are summarised in Table 8.13.

Table 8.13 Basic Requirements for Filter/Drainage Materials

Key Engineering Factor	Material Requirement
Permeability	The fundamental filter property is primarily a function of material grading. It is generally desirable for filter aggregates to be equidimensional as this aids flow distribution and facilitates packing. It is also considered better to use material with rounded to sub-rounded rather than angular particles.
Strength	Aggregate particles need to be load resistant to abrasion and any loads imposed by the road design.
Resistance to Degradation	Aggregate particles need to be resistant to breakdown due to wetting and drying and weathering during construction and for the life of the project.
Resistance to Erosion	The as-placed material must be resistant to internal and external erosion.
Chemical Stability	Aggregate should generally be inert and resistant to alteration by groundwater. Weak surface coatings such as clay, iron oxide, calcium carbonate, gypsum are undesirable.

8.2.11 Miscellaneous Rock Materials

As listed previously, there are a number of additional applications for moderately strong to strong rock in LVRRs. Table 8.14 summarises these uses and the required properties:

Table 8.14 Miscellaneous Rock Use

Rock Use	Key requirements
Gabion Fill	Rock fragments ranging between 100 mm and 250 mm in size. with a variation of 5% oversize and/or 5% undersize rock, provided it is not placed at the exposed surface. In all cases, oversize rock shall not be larger than 300 mm or smaller than 100 mm. Rocks should be strong, angular to round, and durable so that they will not disintegrate under wetting-drying cycles during the life of the structure.(Smith, 1999)
Rock Embankment	Strong angular rock preferably cubic and not elongated or platy in shape. It is suggested that, in general, fragments should not exceed a maximum dimension of two-thirds of the layer thickness. Fragments should be intact and preferably homogeneous without planes of weakness (bedding). Fragments should be durable so that they will not disintegrate under wetting-drying cycles during the life of the structure.(Smith, 1999; USDI, 1998)
Riprap	Riprap should comprise strong, durable fragments. They should be angular and not subject to breaking down when exposed to water or weathering. It is recommended the specific gravity should be at least 2.5. The sizes of stones used for riprap protection are determined by purpose and specific site conditions.(Smith, 1999)
Rock masonry	Durable strong angular rock durable so that it will not disintegrate under wetting-drying cycles during the life of the structure.

8.3 Material Types in Myanmar

8.3.1 Geological Framework

The inherent nature of natural road building materials used in road construction has a profound effect on their engineering performance; hence it is important to understand of the geological nature rocks and their weathered products if they are to be used successfully. Knowledge of geology also provides a framework for identifying material sources and characterising their likely behaviour.

Table 8.15 Broad Material Types

Resource Group	Description	General Material Uses
Hard-Rock	Strong to very strong igneous, sedimentary and metamorphic rock types normally requiring drill-and blast quarrying techniques for excavation.	Materials require crushing and classifying before being utilised as road aggregates. Relatively high cost quarry development and material processing.
Weak-Rock	Weak to very weak igneous, sedimentary and metamorphic rocks that may be excavated by mechanical means, including ripping where necessary. This group includes rocks that have been weakened by weathering processes.	Materials may require some processing before being utilised for road pavements.
Residual Soils and Duricrusts	Soil-like materials that have been formed largely in situ by tropical and sub-tropical weathering processes. Materials generally excavated by borrow-pit techniques. Occasionally indurated duricrust may require ripping.	These materials may be utilised as-dug or classified for fill, capping layer, sub-base or base. Would generally not be considered for surfacing aggregates.
Transported Soils (Alluvium or Colluvium)	Soil-like materials such as sand and gravel that have undergone processes of erosion, transportation and deposition in addition to weathering. Materials generally excavated by borrow-pit techniques	Sound gravel and cobble materials can be processed to produce high quality aggregates. The sorting action of erosion and transportation may result in materials lacking in some particle sizes.

8.3.2 General Materials Groups

Myanmar contains a wide range bedrock types as well as their weathering products, transported soils and residual soils (Barber et al, 2017) many of which may be utilised as materials for constructing LVRR. Tables 8.16 to 8.18 summarise the natural materials most commonly found in Myanmar that may be used as road construction materials and give a broad indication of their uses and possible problems.

Table 8.16 Uses of Hard Rocks Found in Myanmar

Rock Type	General Description	General Material Uses
Granite-diorite	Medium-coarse grained, light coloured igneous rock. Contains quartz, feldspars and possible micas Susceptible to deep and variable weathering.	Strong durable rock used for block stone, concrete and surfacing aggregate, good shape, poor bitumen adhesion when rock contains large feldspar crystals.
Dolerite	Medium grained tightly crystalline dark basic minor intrusive composed largely of plagioclase feldspars and augite.	Very strong durable rock used for block stone, concrete and surfacing aggregate, good shape, good bitumen adhesion. Potential problems with in-service deterioration if weathered.
Andesite	Fine grained intermediate lava composed essentially of plagioclase feldspar and mafic minerals (hornblende, biotite, augite)	Strong durable rock used for block stone, concrete and surfacing aggregate. Possible poor aggregate shape.
Basalt	Fine-grained dark basic lava. Composed largely of plagioclase feldspars and augite and sometimes olivine. Varieties rich in olivine/chlorite susceptible to rapid deterioration and disintegration. Aggregates can be susceptible to disintegration problems in service.	Strong durable rock used for block stone, concrete and surfacing aggregate. Possible anisotropic character and poor aggregate shape. Aggregates can be susceptible to disintegration problems in service.
Quarzitic Sandstone	Medium grained detrital sedimentary rock with clasts composed of quartz particles, fabric may be cemented by silica, iron oxides or carbonates. Great variability, a function of fabric and matrix. May be interbedded with weaker materials.	Variable strength and durability, Great variability, a function of fabric and matrix. May be interbedded with weaker materials. Possible use as an aggregate; and some types of rock fill.
Siltstone	Similar to sandstone but with predominantly silt-sized particles. Tends to be interbedded with other sedimentary materials, including mudstone.	Variable strength, shape and durability. Great variability, a function of fabric and matrix. May be interbedded with weaker materials. Unlikely to be of use as an aggregate. Poor quality rock fill.
Limestone and Dolomite	Consist essentially of crystalline calcium carbonate. If magnesium carbonate then the term Dolomite is appropriate.	Very strong to strong and durable. Used as block stone and aggregate. May contain minor amounts of non-carbonate detritus.
Shale/Slate	A very low-grade metamorphic rock in which cleavage planes are pervasively developed throughout the rock. Poor durability and shape. Marked tendency to split along cleavage planes (fissile).	Weak to moderately strong rock. Low durability. Unlikely to be used as an aggregate. Possible use as poor quality rock fill.
Schist	A medium to high-grade metamorphic rock characterised by the parallel alignment of moderately coarse grains usually visible to the naked eye. The preferred orientation described as schistosity.	Strong to very strong. Moderate durability Poor particle shape. Possibility of free mica being produced during processing. Use as rock fill.
Gneiss	Medium to coarse mineral grains with a variably developed layered or banded structure, minerals tended to be segregated, e.g. quartz/feldspar/ mafic mineral banding.	Strong to very strong. Moderate durability Poor particle shape. Use as rock fill. Possible use as marginal aggregate if well processed. Potential shape problems.
Quartzite	A contact metamorphic rock formed from a quartz rich sandstone or siltstone. Contains more than 80% quartz.	Strong to very strong. Good durability, possible use as aggregate. . Abrasive to construction plant

Table 8.17 Possible use of Weak Rocks in Road Construction

Rock Type	Potential Uses	Potential Problems
Mudstone	As embankment fills and possible selected fill/capping layer material.	Very low particle strength. Potential for slaking and swell/shrink in wet climates.
Shale	As embankment fill and selected fill/capping layer material. Possible use as sub-base material in dry climates.	Potential for slaking and swell/shrink in wet climates. Requires care in compaction for embankment fill as breakdown of material in a voided rock fill could lead to in service settlement.
Weak limestones	As embankment fill and selected fill/capping layer material. Possible selected use as sub-base or base material for low volume roads.	Possible poor as-dug gradings. Low particle strength and in-service deterioration.
Weak Sandstones	As embankment fill and selected fill/capping layer material. Possible use as sub-base or base material in dry climates.	Possible poor as dug grading. Low particle strength and potential for in service deterioration.
Weathered Hard Rocks	As-dug: As embankment fill and selected fill/capping layer material. Sub-base/base material for low volume roads.	Any problems highlighted 8.2 will be accentuated by weathering. Particular problems associated with the rapid deterioration of weathered basic igneous materials.

Table 8.18 Broad Soil Types in Road Construction

Rock Type	Description	Potential Use
Soil from Highly Weathered Rocks (Saprolite)	Soil-like material within the weathering profile that has retained the relict structure of the parent rock.	Generally used for common fill. Problems resulting from over-compaction and break-down of material fabric. High mica content in some weathered rocks
Residual Soil	True residual soil has developed a new-formed fabric to replace the remains of rock fabric material.	Used for common fill. Generally less problems than with saprolitic soil.
Residual Gravel	Concentrations of weathering resistant quartz within residual soil profiles.	Usability as-dug is a function of the ratio of fines to gravel. Commonly used as sub-base and, if processed or stabilised, as base/sub-base material
Transported soils	Colluvium, Alluvium, Coastal deposits ranging from clay to coarse cobble and boulder	Commonly used as common fill and pavement materials depending on their grading, particle strength, plasticity and shape.
Laterite	"As dug" materials highly variable in strength, size and durability	Commonly used as sub-base, base and GWC. Can be modified / stabilized with lime. Higher plasticity materials will be subject to significant loss of strength on saturation.

8.4 Materials Management

8.4.1 Information requirements

Guidance on the site investigation, including sampling and laboratory testing of construction materials is included in Chapter 5 and Annexes II and III of this Manual. The principal objective in acquiring information about road construction materials is to identify geotechnical materials that are capable of meeting the engineering, economic and environmental requirements of the project. Information relevant to this objective may be summarised in Table 8.19.

Table 8.19 Key Data Sets in Materials Management

Data Set	Comment
Source locations and their history	Potential borrow and quarry locations should be clearly identified on relevant maps (electronic or hard copy) together with co-ordinates and means of access. Information on existing locations should be acquired in terms of materials performance on existing roads and/or any relevant laboratory testing. Local knowledge on each location can be extremely useful.
Geological environment	The geological setting should be identified in terms of rock and soil types together with any relevant hydrological information (flood risk etc.).
Geotechnical character	The geotechnical properties of the materials likely to be produced by the location must be defined and assessed in comparison with proposed specifications. Samples from existing stockpiles of materials should be sampled where possible, otherwise representative samples from test pits or bored core (from quarry investigations) must be taken.
Volumes of material	Estimate the volume in cubic metres of useable material at the locations not including unusable overburden and taking account of bulking and compaction factors.
Project specifications	Identify at an early stage the actual project materials specifications that are proposed and review these in the light of available materials. Propose any adjustments that might be possible within concept of local material usage.
Costs	Obtain costs of material extraction and processing as well all costs associated with haulage to the project site. Use this information to establish haulage diagrams for actual construction costing (See Chapter 12)

8.4.2 Extraction and Processing of Materials

The extraction and processing of road construction materials can be a large component of cost in the overall project budget. For LVRRs there should be an emphasis on eliminating processing or reducing it as much as possible.

There are five main types of material extractive operation (Smith & Collis, 2020):

- Quarrying: extraction in drilled and blasted material, e.g. hard rock;
- Borrow pitting: extraction of unconsolidated material, e.g. gravels and weak rocks;
- Cut to fill operations along a road alignment;
- Mining: underground material extraction, either by shaft or audit;
- Dredging: extraction of unconsolidated material from under water.

In borrow areas the minimum thickness of a deposit normally considered workable for excavation for materials for subgrade, sub-base and base is of the order of one 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, (Figure 8.4).

Processing (crushing and screening) of as-excavated resource is undertaken to produce construction materials that meet required specifications by means of either mechanical alteration or physical selection. In general terms, materials utilised for common fill, would normally require no processing in contrast to high quality hard-rock aggregates which can be subjected to several phases of crushing and sorting. The amount of processing required is a function of the relationship between the as-extracted character and the required mechanical, chemical and physical properties.

Hard rock quarry development and extraction will almost inevitably involve a drill and blast operation followed by mechanical processing, although some operations may be undertaken using labour-based methods (Figure 8.4).

Figure 8.4 Material Extraction

Variably thick gravel layer extracted for GWC
Photo J Cook

Labour based aggregate production
Photo J Cook

Processing plants can be fixed or mobile. Fixed plant is more common in large, established quarries, while semi-mobile plant is more appropriate for major construction projects where the life of the quarry is directly related to the duration of the project. For many LVRR projects light mobile crushing and screening plant is utilised in small-scale operations where minimal amounts of processing are required.

Processing is used to:

- Reduce the excavated material to suitable sizes of aggregate;
- Group the sizes together where required into appropriate gradings;
- Remove unwanted fines;
- Reduce oversize.

Table 8.20 Crushing Options and Limitations (Smith & Collis, 2020; McNally. 1998)

Crusher Type	General Application	Description
Jaw Crusher	Usually as primary crushers, small versions may be used as secondary crushers.	Rock is broken by slow compression-release cycles between plates, on fixed and one moving on opposite sides of a wedge-shaped chamber. This narrows downwards so that after blocks are split on the compression stroke the resultant pieces slip further down on the release stroke until on the next cycle they are released. In most quarrying operations the single toggle machines operate satisfactorily – well suited to small and medium-sized operations, including mobile plant.
Gyratory Crusher	Usually as primary crushers; cut-down versions may be used as secondary crushers.	A gyratory crusher resembles a pestle in a narrow open-base mortar, or two cones one inverted within the other. The inner solid cone moves eccentrically around the fixed outer bowl alternately opening and closing gaps around the lower rim.
Rolls Crusher	Primary crusher	Machines can be fitted with single double or multiple rollers, although double rolls are most common – one fixed and the other spring-loaded. Use of the rolls crusher is limited to weaker rock (UCS<100Mpa) and non-abrasive rock such as limestone and shale. Cheap in relation to high capacity and easily transportable – good choice for weak rock for select fill or for demolition rubble.
Cone Crusher	Usually used as secondary or tertiary crushers	Similar in operation to small gyratory crushers in having an oscillating inner and a static outer one the inner cone is pivoted from below rather than suspended from above. They have large capacities in relation to small size. Compared to impact crushers they produce a narrower range of sizes, less fines and more flaky particles.
Impact Crusher	Usually used as secondary or tertiary crushers	Rocks broken by the action of rapidly rotating or beaters attached to a central shaft that - may be horizontally or vertically mounted. The feed particles cascade into the crushing chamber and shatter on impact with the beaters or are deflected by them to strike hardened breaker plates lining the chamber. Relatively light and cheap for their capacity and do not require elaborate foundations. Their main disadvantage is the cost of frictional and chipping wear on breakers and plates and the consequent downtime for replacement. Generally limited to rocks with UCS <150Mpa and free quartz content <5-7%. Advantageous with wet and sticky clay-rich gravels. Product shape is good.

Figure 8.5 Material Extraction Methods

Jaw crusher
Photo J Cook

Small mobile crusher combined with screen
Photo J Cook

Medium sized crushing and screening plant used for a LVRR network rehabilitation
Photo J Cook

8.4.3 Resource Restoration

The environmental damage caused by uncontrolled extraction and rehabilitation practices can extend over a wide area and may only become apparent after project completion. Examples include soil erosion causing siltation of natural water courses and health and safety issues from flooded borrow pits. Environmental damage caused by borrow pits, and their access roads, is often most severe in areas important for subsistence farming. However, historically, restoration of borrow pits has been the exception rather than the rule.

Ideally borrow pits should be reinstated as closely as possible to their original ground level (Roughton International, 2000). This avoids permanently changing the landscape and altering local drainage patterns. Excavated soil and gravel can be replaced with spoil materials derived from road construction or improvement works. Spoil materials may comprise cut materials not required for embankment construction or unsuitable subgrade materials, provided that they do not contain large quantities of plant matter and are sufficiently dry to allow placement.

Backfill materials should not contain materials that might migrate and pollute ground water. If no spoil material is available for back-filling then provision of drainage structures may be necessary to prevent the erosive action of surface water and/or ponding in the abandoned pit.

In the case of large borrow pits exploiting thin near surface gravels, areas that are worked out should be progressively back-filled, top-soiled and planted.

Re-establishment of top-soil and vegetation is a key part of any borrows area restoration. Replacing topsoil and planting of vegetation on shallow slopes (less than 20°) presents no difficulty, but needs to be carried out with care in an appropriate season of the year. Slopes >20^{O} may have to restored in line with anti-erosion and bioengineering measures described in Chapter 10 (Earthworks).

8.5 Using Locally Available Materials

8.5.1 Benefits

The maximum use of naturally occurring unprocessed materials is a central pillar of the LVRR design philosophy. Current international 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 requirements. However, recent research work has shown quite clearly that so-called “non-standard” materials can often be used successfully and cost-effectively in LVRR pavements provided appropriate precautions are observed (Cook et al 2002; AUSTROADS, 2019).

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.

8.5.2 Marginal Materials

PIARC (1989) 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.*"

Local material usage may involve either the innovative use of an established local material or the adaptation of a non-standard or out-of-specification material within a revised design.

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

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

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

Summary of the use of marginal materials

1. Appropriate use of materials is a key factor in appropriate design of LVRRs.
2. There is an increasing body of knowledge that supports the use of locally available construction materials even though they may be non-standard in character.
3. There is scope for increased use of non-standard local materials and innovative designs in Myanmar LVRRs
4. A knowledge-based decision process is vital for the selection of appropriate LVRR materials.

8.6 Material Improvement

8.6.1 General

Many natural materials can be modified to make them suitable for road pavements, but this process is only economical when the cost of overcoming a deficiency in one material is less than the cost of importing another which is naturally acceptable.

The common practice is to refer to materials whose properties have been artificially improved as having been "stabilised" rather than "modified". Many guidelines will adhere to more scientifically correct definitions where stabilisation refers to materials whose character, strength and durability have been significantly altered whilst modification refers to a process where smaller percentages of additive are added to "modify" properties rather than radically alter them. To avoid any confusion this Manual will use the term "stabilisation" for materials improvement by addition and mixing.

8.6.2 Mechanical Stabilisation

The simplest method of increasing the strength is to stabilise it mechanically. In areas where good quality materials are not readily available it may be possible to blend two materials to produce an acceptable product (Figure 8.6). For example, it may be an option in the deltaic area of Myanmar to mix limited amounts of imported crushed rock with local clayey materials to produce an acceptable sub base or even base.

Blending of materials is carried out for two main reasons namely to

1. Improve the stability of cohesive soils of low strength by adding coarse material or
2. Improve the stability of otherwise unstable granular materials by adding a fine material

The well-established processes of constructing Water-Bound Macadam and Dry-Bound Macadam are actually mechanical stabilisation methods of achieving a satisfactorily strong and interlocking pavement layer by using aggregates of 2 (or possibly 3) sizes that may be produced by simple processing methods.

Mechanical stabilisation of fine grain soils, by the addition of non-cohesive granular material, requires sufficient granular material to be added to ensure that the granular fragments are in contact forming a matrix with particle-particle contact throughout.

Figure 8.6 Lateritic soil mechanically stabilised with crushed stone aggregate as base material.

Photo J Cook

Care must be taken to ensure that the plasticity of the fines fraction is controlled. The strength of a blended material must always be determined by testing samples that are representative of the field-mixed product and not on artificially well-mixed laboratory samples.

Mechanical stabilisation is usually found to be the most cost-effective process for improving poorly graded materials; however, this cannot always be achieved. It is important to consider the practical limits of this type of processing. For example, production of a uniform mixture by the addition of granular material to a clay-rich one may produce a uniformly graded material, but one in which the clay may still play the dominant role in determining the properties of the material.

8.6.3 Chemical Modification

Chemical stabilisation normally involves the incorporation of relatively small percentages of lime or cement (Sherwood, 1993). These stabilisers are called hydraulic binders which 'set' in the presence of water. They can dramatically increase the strength of unbound materials making them suitable for use in the main load bearing layer of a road pavement, or they can be mixed with soils in small amounts which merely alter the physical characteristics of the soil, such as plasticity or moisture or moisture condition, rather than to significantly strengthen it.

The common features are that the natural materials are bound together using a low percentage of the chosen stabiliser. The choice of stabiliser is largely dependent on the properties of the destabilised material. Materials with a low plasticity, and therefore low clay content, are more suitable for cement stabilisation. Materials with higher plasticity and a more cohesive nature are better stabilised with lime.

It is not unusual for designers to achieve stabilisation by using both cement and lime for a particular project. Everything depends upon achieving the desired strengths and other engineering properties at the lowest cost. Table 8.21 summarises the envelopes of suitability for the different medication methods.

Table 8.21 Guide to Selecting a Method of Stabilisation (after Austroads 1998)

	Plasticity Index (Ip)					
	Ip<10	10<Ip<20	Ip>20	Ip<6 PP<60[1]	Ip<10	Ip>10
Stabilisation Type	**Applicability[2]**					
Cement	A	A	B	A	A	A
Lime	B	A	A	C	B	A
Bitumen	B	B	C	A	A	C
Bitumen-Cement	A	B	C	A	A	B
Mechanical	A	C	C	A	A	B

(1): PP; Plasticity Product = Ip x % passing 75 micron

(2) A: Usually suitable. B: Doubtful. C: Usually not suitable

8.6.3.1 Cement Stabilisation

The amount of cement added is usually less than 5%. The initial chemical reactions occur quite quickly hence the processing of the materials has to be completed in a fairly short time; construction must be completed within two hours. The cement is then allowed to cure for a period of, usually, 7 days. Although not essential, use of a batching plant to blend the cement with the host material and water rather than mixing on the road gives a more consistent mix and a better result.

Cement can be used to stabilise most soils. The exceptions are those with a high organic content, which retards the hydration process, and those with clay content outside the normal specification range and where it is difficult to mix the soil/cement mixture evenly. Addition of cement to base materials results in a reduction in plasticity and swell, and an increase in strength and bearing capacity. CBR values well in excess of the minimum requirement for destabilised normally result.

8.6.3.2 Lime Stabilisation

Lime may be in one of the following forms;

(a) quicklime: calcium oxide (CaO),

(b) slaked or hydrated lime: calcium hydroxide (Ca(OH)2)

(c) calcium carbonate (CaCO3).

Calcium carbonate has no cementing properties and only quicklime and hydrated lime are used as stabilisers in road construction. For hydrated lime the majority of the free lime (i.e. Ca(OH)2 that is not combined with other constituents) should be present as calcium hydroxide. Lime for building purposes is required to be 95% pure calcium hydroxide while agricultural lime may be only 65% pure. The quantity is often also referred to as the 'available lime'. For a particular, application it is important to carry out laboratory tests with the same lime as will ultimately be used for the project.

The ICL test (Sherwood, 1993) can be used to give a rapid indication of the minimum amount of lime that needs to be added to a material to achieve a significant change in its properties. Samples of the material are mixed with water and different proportions of the lime being used. The minimum amount of lime needed to give a pH of 12.40 is expressed as the ICL of the material.

The test can be completed in one hour and is thus a rapid means of establishing the minimum amount of lime required for stabilisation. However, it does not dispense with the need to carry out strength determinations because it does not establish whether the soil will react with lime to produce a substantial strength increase. Research has also suggested that the lime percentage obtained from the test does not necessarily produce the maximum cured compressive strengths for tropical and sub-tropical soils (TRB 1987).

Quicklime has a much higher bulk density and is less dusty than hydrated but is generally not used in LVRR projects due to its caustic nature and consequent health and safety issues.

Typically, 3 to 5 per cent of stabiliser is necessary to gain a significant increase in the compressive and tensile strengths. The gain in strength with lime stabilisation is slower than that for cement and a much longer time is therefore available for mixing and compaction. Lime has a much lower specific gravity than cement so, for a given percentage mass, a higher volume is available and it is therefore easier to achieve uniform mixing.

The production of cementitious compounds can continue for ten years or more but the strength developed will be influenced by the materials and the environment. Between one month and two to three years after compaction there can be a four-fold increase in the elastic modulus.

8.6.4 Bitumen Stabilisation

It is also possible to stabilise marginal materials with the addition of small percentages of slow setting bitumen emulsion. This process is only effective with sand materials with little or no fines and is likely to be more costly than the other stabilisation options and should only be considered in, for example, coast areas where only sand is available for construction purposes within reasonable haul distances. Recent trials research, however, has shown it to be an effective procedure in the appropriate circumstances. (Intech-TRL, 2006)

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9 Cross Drainage and Small Structures

9.1 Introduction

This Chapter is concerned with the external and cross-drainage system and the drainage standards for LVRRs. It is essentially a guide containing appropriate technical explanations of all the steps in designing their surface water drainage systems. Internal pavement drainage is considered in Chapter 7 and earthwork drainage is dealt with in the following Chapter 10. Due to the vulnerability of drainage structure to climate impact, the threats from current and future climate are given appropriate attention, particularly with respect to data collection and analysis. This Chapter does not deal with route surveying, site investigations, and route selection or the actual structural design of major bridges. The planning and structural design of river crossings of greater than 10m span and major drainage structures is dealt with in the current MoC Bridge Design Manuals.

Roads form a barrier to the natural drainage of surface water from the surrounding land into streams, lakes and rivers. In the absence of any control arrangements the water would find its own way across the road, resulting in gullies and washouts along the road. Without provision for dealing with cross drainage, roads can also cause flooding in adjacent areas causing damage to property, crops and livestock. Flooding will also disrupt communications causing economic and social harm and even a threat to life, if traffic is washed off an inundated road surface. In road design it is essential that cross drainage structures are located along the road alignment where water courses would otherwise be cut. Several types of cross-structures are used to allow water to be safely removed downstream while protecting the road structure, adjacent property and maintaining communications along the road during high flow periods (Larcher et al, 2010)

Neither rainfall nor rivers distinguish between roads carrying low and high volumes of traffic. Therefore, the basic approaches to protecting a road from the effects of water are essentially the same and largely independent of traffic. Hence, for LVRRs 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 LVRRs the consequences of failure in the drainage system are correspondingly lower but, within the range covered by LVRRs, there are some significant differences depending on the length of the road as well as the availability of an alternative route.

Water crossing structures are used as cross-drains for ditch relief and to pass water under a road at natural drainage and stream crossings. The objective is to provide all-season access to as many as possible road users in line with the National Strategy for Rural Roads and Access (GoM, 2017). The design and choice of an acceptable type of crossing are based on the technical and economic feasibility of the structure and the structure's ability to meet other environmental objectives such as prevention of erosion and sedimentation. A final choice is based on an evaluation of costs and the ability to meet the hydrological and hydraulic design criteria for the site.

The challenge for the engineer is to choose a level of protection that is appropriate for the class of road and the consequences of drainage failure. This challenge is compounded by the need to assess future climate impacts. The choice of structure is based on good engineering judgement taking into account:

- Human safety
- Economic feasibility/cost benefit
- Length of structure required
- Stream crossing geometry
- Sediment and debris loading
- Terrain stability
- The magnitude of the water discharge and flood history
- Future climate-based changes to the flow regime.

The Chapter covers the commonly used drainage solutions used on LVRRs from drifts to small bridges as set out in the following sections.

9.2 Typical crossing and drain types

9.2.1 Best Management Practices for Erosion Control and Water Quality Protection

All types of crossing structure should be built according to best management practices (BMP):

1. Use BMPs and incorporate erosion-control measures into the design, construction, and maintenance of crossings to protect water quality.
2. Incorporate construction dewatering into the project. Avoid working in the water!
3. Develop a project "erosion-control plan," including appropriate physical, vegetative, or biotechnical measures, types of materials, and timing.
4. Choose appropriate project BMPs and include them in project budgets, design, and project implementation. Monitor them for implementation and effectiveness.
5. Periodically inspect and maintain the structure to ensure that it is functioning properly.
6. "Disconnect" the road from the stream crossing by diverting road surface water before reaching the crossing, armouring ditches and stabilising the roadway surface (often steep) approaching the crossings.

9.2.2 Low Water Crossing Structures

Low water crossings (LWC)s, fords, or drifts, as they are also often known, can offer a desirable alternative to culverts and bridges for stream crossings on low-volume roads where road use and stream flow conditions are appropriate. Like other hydraulic structures for stream crossings, they require specific site considerations and specific hydrologic, hydraulic, and biotic analyses. Ideally, they should be constructed at a relatively narrow, shallow stream location and should be in an area of bedrock or coarse soil for good foundation conditions. A ford can be narrow or broad, but should not be used in deeply incised drainages that require a high fill or excessively steep road approaches.

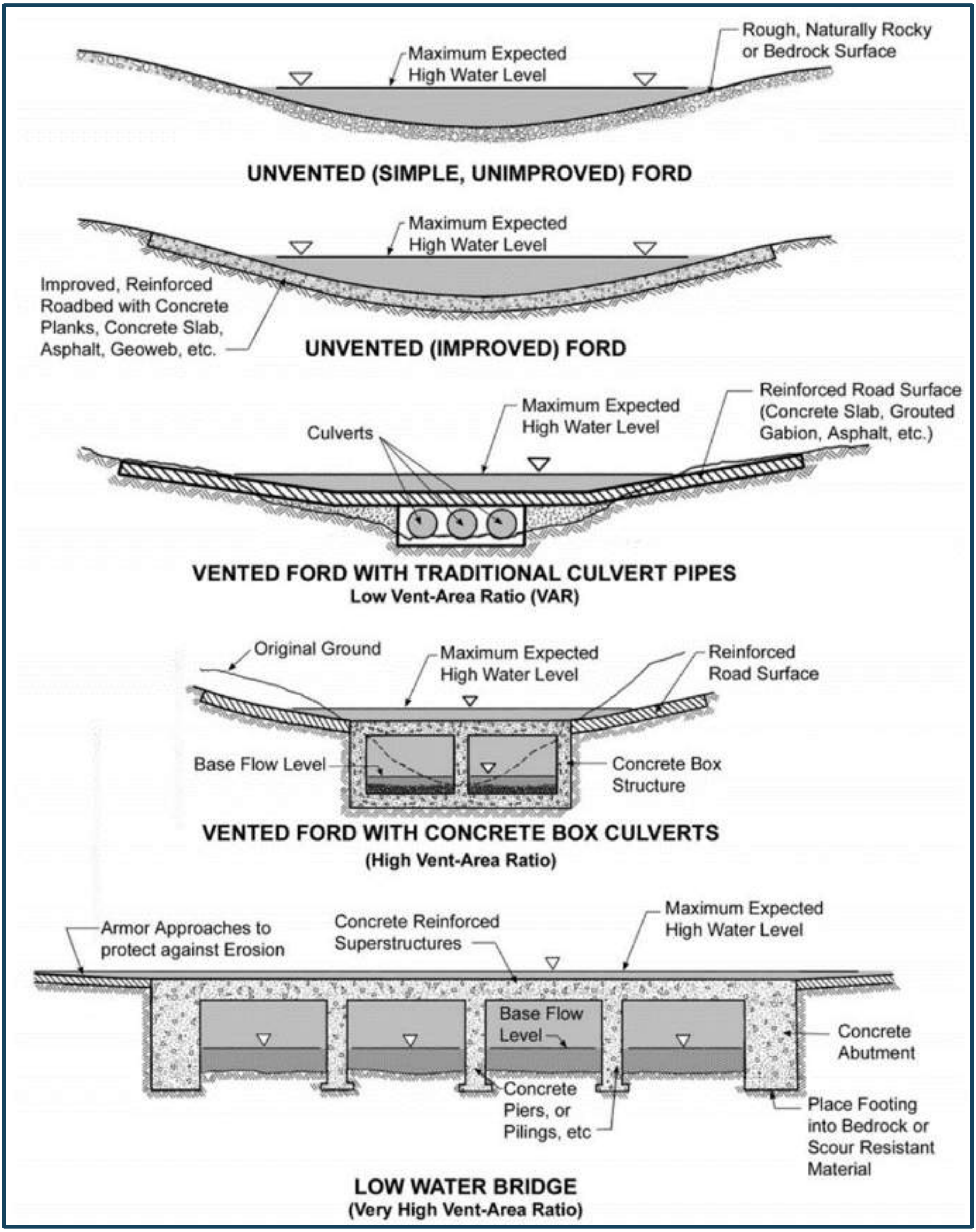
Figure 9.1 A typical LWC

Figure 9.2 Typical crossing suitable for LWC construction

LWCs are road-stream crossing structures designed to be overtopped by high flows or by debris. They can be desirable alternatives to culverts and bridges and they can offer substantial environmental advantages in some stream environments. They are useful, for example, where streamflow is highly variable and large amounts of woody debris pose a risk to crossing structures.

Figure 9.3 shows the basic low-water crossings types, from simple ford though to low water bridge.

Figure 9.3 Low Water Crossing Types



Source: Keller G & Sherar J, 2003

LWCs are commonly used in areas with highly variable flows, such as dry streams subject to flash floods and thunderstorm-prone areas. High, short-duration peaks followed by long intervals of very low or no flow is most conducive to low-water crossings as long as traffic interruptions during floods are tolerable. Because standard crossings need to be very large to convey such high flows together with their debris loads, they may not be economically feasible for many low-volume roads. Streams with highly variable flows may also be less stable than streams in which steady base flows support vigorous riparian vegetation. Putting a large expensive structure on a channel that may shift within the structure's lifetime is even less desirable

The cost of the typical structure like a bridge or culvert is usually significant in the overall cost of roads, particularly a low volume roads. Hence, the causeways, drifts and the low water crossing, in general, maybe a cost-effective method for crossing wide rivers which are dry for the majority of the year or have low permanent flows and can be closed for short periods of time without serious consequences.

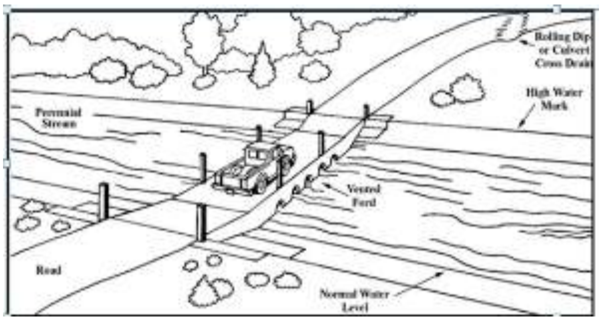
Myanmar is one of the countries most affected by extreme weather events according to the Global Climate Risk Index (Kreft et al., 2016). Further to the normal monsoon rains season, several parts of Myanmar have been affected by extreme floods. Hence, LWCs like submersible bridges, vented or unvented drifts are a suitable and cost-effective choice in many rural and flooded areas across the country.

Three main types of crossing structures are designed to be submerged at some flows: (1) unvented (simple) fords, (2) vented fords, and (3) low-water bridges. Because basic designs require tailoring to individual site requirements and locally available materials, many variations of each of these basic types of low-water crossing structures have been developed over time.

Improved fords or drifts

Improved fords have a stable driving surface of rock, concrete, asphalt, concrete blocks, concrete planks, gabions, geocells, or a combination of materials. Figure 9.5 shows a typical example. Sometimes a small channel or slot is included at the structure's low point to pass very low flows and aquatic animals. The downstream roadway edge may be stabilized and defined with logs, riprap, gabions, or Jersey barriers.

Figure 9.4 Improved Ford

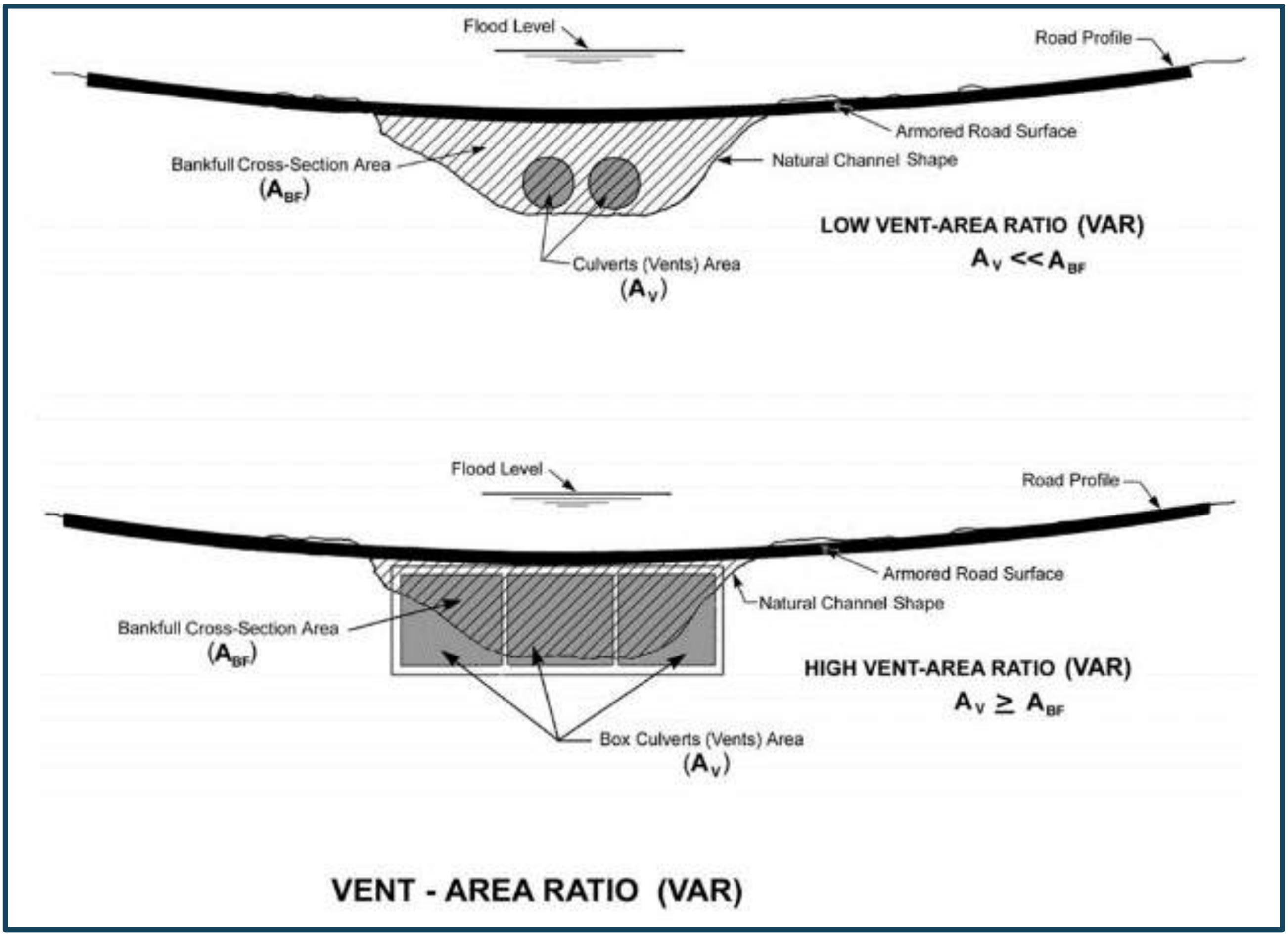


Source: Keller G & Sherar J, 2003

Vented fords

Vented fords fall into two categories—low vent-area ratio (VAR) and high VAR—each of which affects stream channels differently (Figure 9.5). Vented fords with culverts that are small relative to the bankfull channel area have a low VAR. A vent opening that approximates or exceeds the size of the bankfull channel has a high VAR.

Figure 9.5 VAR-ratio definition sketch (Keller G & Sherar J, 2003)



Low water level bridges

Low-water bridges could be described as structures with elevated decks with a natural stream bed bottom. This distinguishes them from wider box culverts which have an engineered bottom section as part of the culvert structure. They may be designed with one or several piers. Low-water bridges generally have greater capacity and are able to pass higher flows underneath the driving surface than most vented and unvented fords. As with fords, however, low-water bridges are designed and installed with the expectation they will be under water at higher flows.

In general, low water bridges are not a very suitable option for Myanmar where flooding is rather seasonal following the monsoon rather than of brief duration flash flood type. Where a bridge is required it is preferable to have the deck elevated above the deign flood for the crossing.

9.2.3 Potential Benefits of Low-Water Crossings

LWCs are generally less expensive to construct. More often than not, designs are less complicated, construction is quicker, and fewer materials are involved. Although the initial cost of more complex low-water crossings may exceed those of simple culvert installations, the lower long-term maintenance and repair costs may still make selecting a low-water crossing more economical.

Simple LWCs like unvented fords are useful in naturally unstable channels such as alluvial fans and braided streams, or in channels with extreme flow variations. Because they obstruct flows less than most culverts, they are less likely to cause flow diversions or accelerations both of which can exacerbate a channel’s inherent tendency toward instability. They can also be inexpensive to reconstruct in a new location if the channel does move.

LWCs are very useful in catchments that have experienced severe disturbances and where substantial mobilization of rock and woody debris is expected. They are most suited for rural roads with low-to-moderate traffic speeds. Unimproved fords may only be driven over at low speeds of less than 15 to 30 km per hour. Vented fords with a broad, smooth dip and gentle transitions may be suitable for speeds up to 50 to 70 km per hour. If high-speed traffic is anticipated, then low-water crossings are likely unsuitable for that road.

The following section lists the both the advantages and disadvantages of low water crossing structures to assist with decision making when deciding whether a low water crossing is the most appropriate structure for the crossing:

Advantages

1. Structures designed for overtopping.
2. Less likely than culverts to be damaged by debris or vegetation plugging.
3. Typically, less expensive structures than large culverts or bridges.
4. Less susceptible than other structures to failing during flows higher than the design flow.
5. Good for "storm-proofing" roads where large amounts of sediment and debris are expected, for example, after a large storm event.

Disadvantages

1. Can be dangerous to traffic, particularly NMT, during high-flow periods
2. Have periodic or occasional traffic delays during high-flow periods.
3. Are not well-suited to deeply incised drainages.
4. Are typically not desirable for high use or high-speed roads.
5. Can be difficult to design for aquatic organism passage.

9.2.4 Key engineering design elements for low-water crossings

Structure-Site Compatibility

Select and design structures to maintain the function and bedload movement of the natural stream channel. Conform to the natural channel shape and elevation where possible.

1. Avoid "damming" the natural channel or adjacent flood plains. Keep the channel open.
2. Do not cause significant aggradation in the channel upstream of the structure, or degradation or down cutting downstream of the structure.
3. Do not confine or narrow the normal (bank full) flows.
4. Do not increase the natural stream channel velocity.
5. Accommodate major flood flows without significant drops in the water surface profile.
6. Align structures perpendicular to the stream channel.

Roadway and Site Geometry

Build a structure that fits the site, with a vertical and horizontal alignment that will be safe and will allow the design vehicle to pass over the crossing.

1. Select a site with a relatively straight road alignment.
2. Locate a crossing at a straight reach of the stream.
3. Conform to the natural dip of the channel as much as possible.
4. Limit grades into the ford to 10 percent or less if possible.
5. Use a vertical curve dip through the ford, sufficiently gentle enough not to catch the bumper or undercarriage of vehicles passing through the ford.
6. Provide enough space for backing up and turnaround when needed.

Site Hydrology

Ideally use either a flow-duration or flood-frequency (peak discharge) design approach to specifically size the low-water crossing structure. Nonetheless, when site hydrological conditions are unknown or difficult to determine, LWCs make a good structure choice. They can easily be designed to overtop a large volume of water and/or debris, and they are not sensitive to the exact flow quantity. Determining the hydrological properties of a site should be an interdisciplinary process, involving hydrologists and engineers. In summary the following steps for low water crossing design are:

1. Determine the peak design flows (Q50 or Q100 events) to select the maximum size of the structure and identify maximum high-water level.
2. Determine low-flow information (baseflow to Q2, or bank full flow) to size the vents in a structure, and estimate the frequency of probable delays.
3. Quantify flows suitable for fish passage through structure or vents.
4. Estimate traffic-delay times using either flow-duration data or field knowledge of the site.

Hydraulic Design

1. Determine the site hydraulic factors needed for prudent structure design.
2. Determine flow capacity through vents and over the structure, up to the high water elevation.
3. Use computer models such as Hydroculv and Hydrochan, Manning's Equation, pipe capacity nomograms, or broad-crested weir formulas to determine flow through and over respective components of the ford.
4. Determine stream velocities (through the structure) that will require riprap or other scour protection measures.
5. Limit velocities to those suitable for required fish passage using FishXing software.

Scour, Bank Protection, and Preventing Channel Changes

There are basically three types of scour or erosion. The first two are caused by the existence of the drainage structure itself in concentrating the flow of water and/or increasing its velocity. There are two aspects:

- Erosion/scour around the structure itself that threatens its integrity and its continued existence.;
- Erosion/scour that occurs because of the structure but upstream and especially downstream away from it.

The third type is essentially natural scour or erosion that occurs within all natural water channels irrespective of the existence of man-made drainage structures. This will alter the hydraulic environment over time and needs to be considered in the design of the road.

The amount of scour is dependent on the speed of the water flow and the erodibility of the material that the water comes into contact with. If the flow is not parallel to the constriction more scour will occur on one side than the other. Water is accelerated around abutments, piers and other obstructions, creating vortices with high velocities at abrupt edges on the obstruction, increasing the scour depth, often dramatically.

1. Protect the channel, the structure, and its foundation against scour and erosion and prevent accelerated stream flows that can damage structures, wash out the approaches, or provide a source of sediment into the watercourse.
2. Prevent a "waterfall" and other scour-critical areas by keeping structures low to the channel and by avoiding channel constriction and mid-channel structures or obstructions.
3. Install scour protection or energy dissipation measures, including rock riprap, concrete aprons and cut-off walls, gabion basket aprons, or plunge pools.
4. Protect streambanks with vegetation, bioengineering measures, erosion control or reinforcing mats, gabions, concrete blocks, rock riprap.
5. When riprap is used, size and place the rock to prevent rock movement resulting from the velocity and force of water.
6. The amount of scour experienced at a structure is proportional to the restriction in the normal water flow. Hence, as a general principle, wherever possible, any constrictions to water flow should be minimised.
7. Cut-off walls, also called curtain walls, should be provided at the edge of a structure to prevent water eroding the material adjacent to the structure. The location and depth of cut-off walls for the various structures is shown in Table 9.1 and Table 9.2.

Table 9.1 Cut-off Wall Locations

Structure	Locations
Drift	Upstream and downstream of drift slabs.
Culvert	Edges of inlet and outlet apron.
Vented drift/ford	Upstream and downstream sides of main structure and approach ramps.
Large diameter culvert	Upstream and downstream sides of approach ramps. These foundations of the main structure should be built at a greater depth than the standard cut-off walls below the possible scour depth.

Table 9.2 Cut-off Wall Depths

Structure	Cut off wall depth(m)	comments
Drift	1.5	
Relief culvert	1.0	
Water course culvert	1.5	Head walls and wing walls
Vented Drift	2.0	

9.2.4.1 Structural Design of Driving Surface:

Design low-water crossings to support the design vehicle for the onsite soil conditions.

1. Provide at least 300 mm compacted soil cover over culverts or a concrete slab (typically at least 150 to 200 mm thick) over box culverts, based upon manufacturers' requirements or structural analysis.
2. Construct the roadway driving surface with material durable enough or heavy enough to resist the shear stresses or lateral forces of the water flow.
3. Protect the entire "wetted perimeter" of the ford (the area of the entire high flow), plus freeboard (typically 600 to 1,100 mm of additional height).
4. Remove soft or organic subgrade soils and replace the soil with select, structurally sound material in a layer thick enough that will support the traffic without deformation.

9.2.4.2 Traffic Control and Safety

Consider all traffic safety issues to produce a safe crossing site.

1. Ideally locate low-water crossings at sites where the road is straight and where good sight distance exists.
2. Build 150 by 250 mm wood or 400 mm high concrete curbs to define the roadway and keep traffic on the structure.
3. Place object markers along the road at each corner of the structure to define each entrance of the structure.
4. Install warning signs to identify the approaching ford and warn drivers of flooding and possible traffic delays.
5. Use marker posts that indicate the depth of flow.
6. Consider making the ford extra wide for traffic safety, and wherever possible, using 4:1 or flatter fore slopes on embankments.
7. If site evaluation determines that a ford would be unsafe, choose a conventional structure such as a culvert or standard bridge.

9.2.4.3 Materials Selection

Choose strong, durable, cost-effective materials for construction of low-water crossings. The driving surface may be made of local rock, aggregate confined in geocells, gabions, concrete planks, asphalt, masonry, or a massive concrete slab. Most vented box fords are made of structural steel-reinforced concrete, because of its strength and durability.

1. Use local riprap where appropriate, cost effective, and available in the necessary size (Smith, 1999). Riprap is unsuitable if it is undersized and if the forces of water can move it.
2. Where suitably large rock is not available for scour protection, use alternative materials such as gabions, grouted riprap, root wads with boulders, concrete blocks, or massive concrete.
3. In relatively low-velocity, low-energy areas use vegetative or biotechnical streambank stabilisation measures, erosion control mats, or turf reinforcing mats.
4. Maintain materials quality control in the structure in accordance with appropriate standard specifications.

9.2.5 Culverts

There are two main types of culverts namely, box culverts and pipe culverts. For low volume roads in Myanmar these are generally of constructed of reinforced concrete. Box culverts can either be constructed *in situ* or for smaller culverts these may be prefabricated. Pipe culverts are generally prefabricated off-site and relocated during road construction. Both box and pipe culverts a can be single or multiple cells (box) or barrels (pipe). Multiple units allow a larger flow across a wider span without increasing the height of the crossing allowing road elevation to remain more constant at the crossing. Box culverts also reduce the super-elevation required for the road surface compared with pipe culverts, which is a consideration especially in road sections where numerous crossings are required such as when traversing paddy field areas.

The size and type of culverts to be chosen depends on the design flood flow of the channel to be culverted.

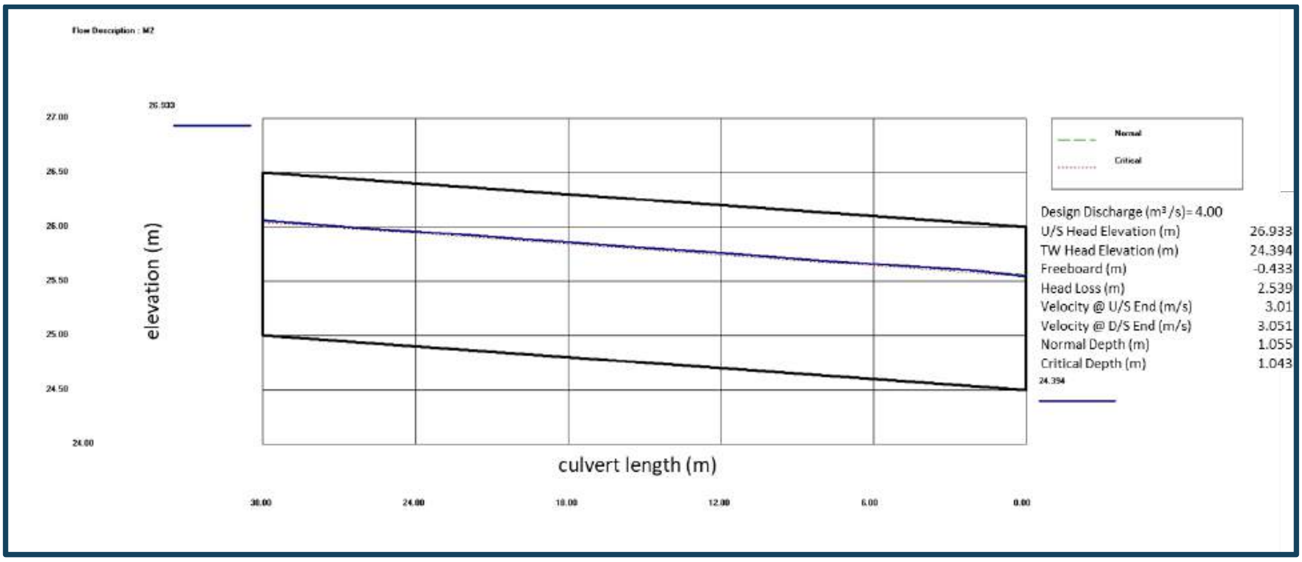
Table 9.3 Design discharge capacity of Culverts use for road cross-drainage

Type	Size	Design Qmax (m^3/s)	Type	Size	Design Qmax (m^3/s)
PC	1 x 0.8 m diam.	1.00	BC	1.0 x 0.6 m	0.8
	1 x 1.0 m diam.	1.50		0.8 x 0.8 m	1.0
	2 x 1.0 m diam.	3.00		1.0 x 1.0 m	1.6
	3 x 1.0 m diam.	4.50		1.5 x 1.0 m	2.3
BC	1 cell x 2 x 1.5 m	4.50	BC	1 cell x 3 x 2.5 m	17.50
	2 cell x 2 x 1.5 m	8.10		2 cell x 3 x 2.5 m	25.00
	3 cell x 2 x 1.5 m	11.25		3 cell x 3 x 2.5m	32.50
BC	1 cell x 3 x 1.7 m	8.00	BC	1 cell x 3 x 3.0 m	14.00
	2 cell x 3 x 1.7 m	15.00		2 cell x 3 x 3.0 m	27.00
	3 cell x 3 x 1.7 m	22.50		3 cell x 3 x 3.0 m	39.00
BC	1 cell x 3 x 2.0 m	9.50			
	2 cell x 3 x 2.0 m	17.50			
	3 cell x 3 x 2.0 m	25.50			

Note: BC – box culvert PC – pipe culvert, Qmax maximum discharge through the culverts

Table 9.3 shows typical maximum design flows for both pipe and box culverts single and in parallel given typical road crossing conditions. If required where inlet or outlet condition to a culvert may be steep or restricted, calculations for culvert diameter can by calculated from hydraulic principles using inlet and outlet conditions and varying slope length, cross-sectional area, roughness and profile of the proposed culvert to find an optimal size. While this could be done manually with spread sheet normal it is more practical to use a software application such as HydroCulv which is licence free software or using part of a larger proprietary hydraulic design package. Figure 9.6 shows a typical design output from a culvert design developed with HydroCulv software.

Figure 9.6 Typical output from culvert design package



Estimating the design discharge for choosing the appropriate size of culverts is discussed further in Section 9.6.5.

In general box culverts are easier to construct using a local labour and construction methods and are more easily checked for quality. Pipe culverts may often not be available locally and local construction of pipes may not be of adequate standards e.g. pipes manufactured locally for lining wells are unlikely to be suitably reinforced for load bearing road applications.

Culverts are required at smaller river or creek crossings where they convey river water directly under the road. However, where roads traverse low lying valley bottom, especially those which have been developed for paddy, regular relief culverts will be required at intervals. Generally, these culverts are dry for most of the time. Relief culverts are especially important when the road is embanked above surface level to prevent the road acting as a dam to flows across the low lying areas.

Without regular relief culverts the flow regime to the paddy field areas may also be disrupted by the road embankment potentially isolating area of paddy from both water supply and drainage and thus damaging crops. In general, across extended areas of embanked road culverts must be provided at no more than 250m spacing. Relief culverts for side drainage will also be required in steep terrain in mountainous regions (section 9.2.7).

As well as the considerations above concerning relief culverts, paddy field areas require special consideration concerning culvert crossings in Myanmar, where large areas are developed as paddy fields. As well as natural drainage channels the location of culverts should take into account the specific needs of paddy field cultivation to minimise disruption to agriculture in the vicinity of the road. Stakeholder consulting of the location of crossings is always recommended as crossing requirements may not always be obvious from purely engineering considerations

Where irrigation delivery and drainage channels cross the road alignment these must always have a culvert provided. Even though these channels may transmit very small flows, it is always recommended that a minimum allowable size culvert is installed (0.8 m) while this maybe oversized for the likely flow it is important that a smaller size culvert is not installed. Smaller culverts may become easily blocked with debris and may prove impossible to clear, unlike larger culverts which can be cleared manually.

Figure 9.7 Typical Single Cell Box Culvert

Figure 9.8 Box Culvert with Two Cells

Figure 9.9 Typical Single Barrel Pipe Culvert

Figure 9.10 Pipe Culvert with Two Barrels

In many parts of Myanmar farmers pump water from adjacent canals and rivers into their fields using river-bank diesel pumps, lifting typically from one to four metres. This pumping often requires delivery pipes to be run across roads running parallel to the channels. Delivery pipes are often of flexible construction and to avoid damage, pipes are sometimes run through dug channels across the road, damaging the road surface, or are laid in small culverts across the road. The best solution for this agricultural requirement is, in consultation with stakeholders, to build the road with small culverts incorporated to accommodate irrigation delivery pipes. There should be no restriction of the minimum size of culvert in this case as no natural open flow will be passing through the culvert. It is important that the location of the delivery pipe culverts is chosen in consultation with local farmers.

Table 9.4 Typical Design Capacities for Culverts

Type	Size	Design Qmax (m^3/s)	Type	Size	Design Qmax (m^3/s)
PC	1 x 0.8 m diam.	1.00	BC	1.0 x 0.6 m	0.8
	1 x 1.0 m diam.	1.50		0.8 x 0.8 m	1.0
	2 x 1.0 m diam.	3.00		1.0 x 1.0 m	1.6
	3 x 1.0 m diam.	4.50		1.5 x 1.0 m	2.3
BC	1 cell x 2 x 1.5 m	4.50	BC	1 cell x 3 x 2.5 m	17.50
	2 cell x 2 x 1.5 m	8.10		2 cell x 3 x 2.5 m	25.00
	3 cell x 2 x 1.5 m	11.25		3 cell x 3 x 2.5m	32.50
BC	1 cell x 3 x 1.7 m	8.00	BC	1 cell x 3 x 3.0 m	14.00
	2 cell x 3 x 1.7 m	15.00		2 cell x 3 x 3.0 m	27.00
	3 cell x 3 x 1.7 m	22.50		3 cell x 3 x 3.0 m	39.00
BC	1 cell x 3 x 2.0 m	9.50			
	2 cell x 3 x 2.0 m	17.50			
	3 cell x 3 x 2.0 m	25.50			

Note: BC – box culvert PC – pipe culvert, Qmax maximum discharge through the culverts

9.2.6 Small bridges

Where conditions are too wide to install culverts and there is a risk of large debris such as trees being washed down the water course and lodging in the culvert, a small bridge is the preferred option. This would usually consist of a single span and will span the natural river-bed, unlike a culvert which has a concrete base.

The size of the bridge span will generally be dependant of the width of the channel to be crossed and as small bridges do not largely reduce capacity of flow, the bridge invert should be adjusted to the maximum flow level design discharge. This will often be chosen based on known or estimated flood levels in the channel at the crossing point, taking future climate predictions into account.

Small bridge capacity may also be estimated using design software to model the flow regime of a design discharge capacity that has been developed using procedures as set out in Section 9.6.5. Using modelling software such as HEC-RAS or similar will require detailed survey of upstream and downstream of the bridge crossing including at least eight cross sections and a long profile of the river reach upstream and downstream of the proposed bridge crossing. Modelling may therefore may often not be considered a cost effective method for design of low volume rural roads especially where many crossings occur. Modelling may also be complicated by backwater conditions at the crossing. This often occurs where a rural road is running along the bank of a larger river bank and the crossing is required to allow a tributary to enter the main water course under the road alignment. This situation is very commonly found in rural roads. Here modelling at a level suitable for the design of low volume road is unlikely to be successful and previous wet season water levels are likely to a more reliable indication of bridge requirements.

9.2.7 Side Drains

To avoid build-up of water at road-sides both culverts and small bridges may need to have road side drains leading to the crossing structure. A summary drain types and their criteria are set out in Table 9.5.

Side drains are required to ensure water is drained quickly away from the road after a storm event with the aim of having no residual water sitting within 0.3m of the road sub-grade. In general, the side drain grade will normally follow the roadway grade with the minimum grade for an unpaved drain being 1%.

Mortared masonry lined side drains are required on grades where soils are easily eroded. For these soils, lined channels are recommended for all grades of 3% or more (Type 2 protected earth drain). However, he soils in some areas are much firmer than in others and therefore the final selection of which sections of road need to be lined and which ones do not, will have to be assessed after roadside foundations are excavated.

For the purposes of design, all road sections with grades exceeding 3% should be considered as requiring lining of the side drains with adjustments to be made at the time of construction.

In residential area where there are many houses and especially schools or other utility buildings, covered U-drains are recommended (Type 3, U-drain covered). Where the road is cut into a hill side and in order to save uphill slope excavation V-drains with reduced top width as compared with trapezoidal drains are recommended (Type 4, V-drain uncovered). Where the road width is restricted by hard geology road width can be minimised by using covered V-drains over which vehicles can pass when two-way traffic is required (Type 5, V-drain covered with stone or masonry). Typical drain sections are set out in Figure 9.11 with typical side drain cross sections shown in Figure 9.12.

To ensure the capacities of the side drains are not exceeded, the maximum spacing between culverts for different grades for the standard designs are also recommended. For example, at a 2% grade, the maximum spacing between cross culverts is 250 to 500 m decreasing to 160 m at 6% grade and 80m at 10% grade.

Table 9.5 Standard Drains

No.	Type of drain	Criteria for drain type
1	earth drain	side slope<3%
2	protected earth drain	masonry >3% slope
3	u-drain (covered)	RC/masonry near village
4	v-drain uncovered	steep slopes/limited space area
5	v-drain covered with rc stone or masonry	steep slopes, very limited space

Figure 9.11 Drain Cross Sections

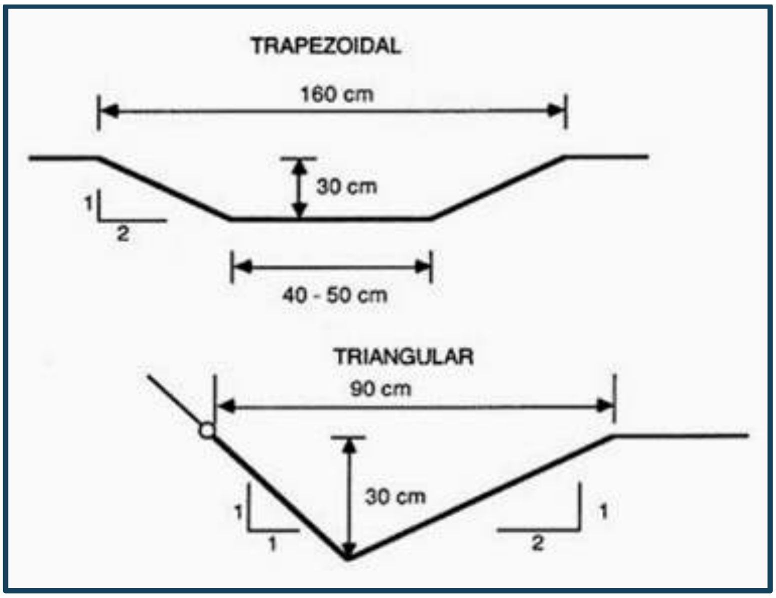
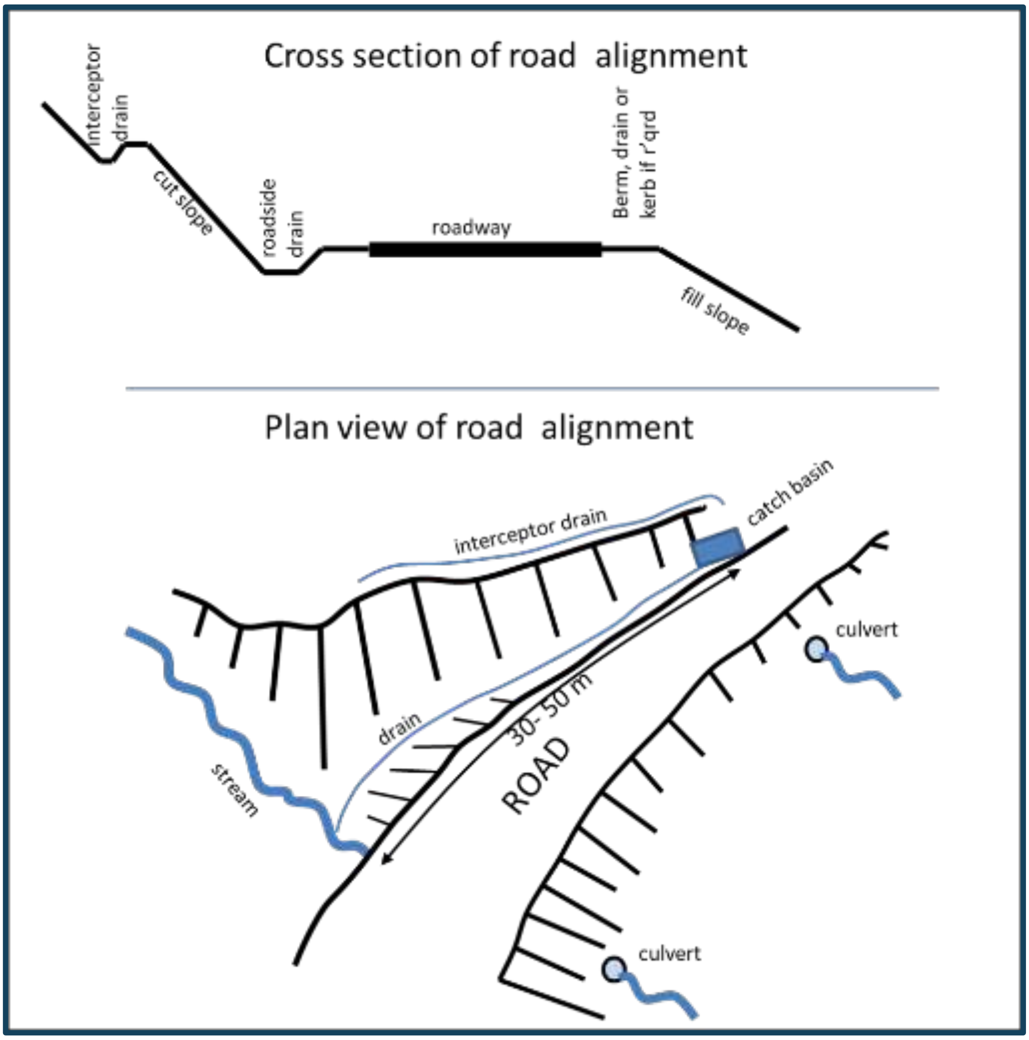


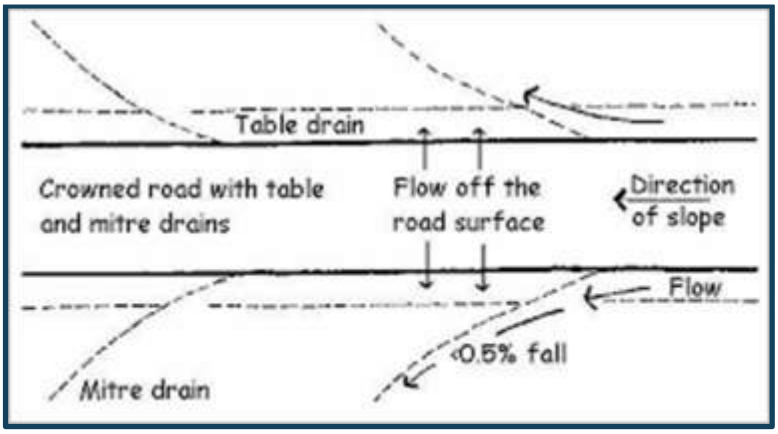
Figure 9.12 Typical Side Drain Arrangement (Keller G & Sherar J, 2003)



Source : Low Volume Road Engineering Best Practices; USDA Forest Service

Mitre drains (side drain relief outlets / culverts) are required on the sections of roads where water can be safely evacuated away from the road. This reduces the flow rates that need to be carried in the side drains. The relief drains are angled at about 30^{o} from the side drain. Figure 9.13 shows a sketch of a typical mitre drain arrangement.

Figure 9.13 Typical Mitre Drain Schematic (Keller G & Sherar J, 2003)



Source : Low Volume Road Engineering Best Practices; USDA Forest Service

9.3 Selecting the Best Structure for the Crossing Site

To select the best structure for the crossing it is important to the site, costs, stream flow patterns, channel characteristics, and aquatic organism passage needs. The following section set out criteria for indicating the type of crossing that may be preferable at any location.

Table 9.6 Some Criteria for selection of suitable crossing type

No	Criteria at crossing	Preferred crossing type		
		Low-water crossing	Culvert	Small bridge
1	These is an existing crossing structure other than a low –water crossing	not suitable	may be suitable	may be suitable
2	The road is a critical route or does not have alternative access to the area.	not suitable	may be suitable	maybe suitable
3	The channel is entrenched to moderately entrenched (broad and shallow).	suitable	not suitable	not suitable
4	Occasional traffic delays are acceptable.	suitable	not suitable	not suitable
5	The channel is ephemeral or has relatively low baseflow.	suitable	may be suitable	may be suitable
6	The catchment has large flow fluctuations or a “flashy” response.	suitable	may be suitable	may be suitable
7	The channel carries a large amount of debris.	suitable	not suitable	may be suitable

9.3.1 Low water crossings

Based on the criteria shown in Table 9.6 the suitability for construction of a low water crossing can be assessed.

In general the most suitable area for a low water crossing will be where criteria three is satisfied and criteria four is not a critical restriction. For criteria five to seven other alternatives such as a culvert or small bridge would be more suitable and more cost effective than a low water crossing.

In general, the criteria for making a low water crossing a preferable solution are set out in Table 9.7.

Table 9.7 Criteria for selection of a low-water crossing

Criteria	Most Applicable
Access Priority	Low
Alternative route	Available
Traffic speed	Low
Average daily traffic	Low
Flow variability	High
High-flow duration	Short(hours)
High-flow frequency	Seldom (rare closure)
Debris loading	High
Channel Entrenchment	Broad and shallow

Table 9.8 lists some questions and possible solutions that need to be considered to correctly design a low water crossing.

Table 9.8 Choosing the Type of Low Water Crossing

Considerations	Decisions
Is road use low and is the stream ephemeral, or does it have a low base flow and high peak "flashy" flows?	If YES, first consider a simple (at-grade), unimproved ford.
Are the channel bottom and streambank materials soft or erodible?	If YES, consider an improved ford with a hardened driving surface.
Is stream continuity required because of environmental issues at this crossing?	If YES, consider (1) an unimproved ford with a natural bottom; (2) an improved at-grade ford with a roughened driving surface, (3) a low-water bridge, or (4) a high-VAR ford.
Is driving through water frequently prohibited or are long traffic delays unacceptable?	If YES, consider only the vented structures and low-water bridges with an elevated driving surface
Is the channel incised or entrenched?	If YES, consider a vented structure with boxes that match the channel's shape.
Is the channel very broad or does it carry a considerable base flow with high peak flows?	If YES, consider a relatively long span low-water bridge
Does the channel carry a lot of large woody debris?	If YES, consider an unimproved or improved unvented ford.
Does the drainage pass periodic debris torrents through an incised channel?	If YES, consider rock-fill fords. Alternatively, massive concrete vented fords have been used with trash racks to pass the debris over the structure.
Is a barrier needed to exclude exotic species?	If YES, consider an improved, unvented ford with a raised platform or a raised vented ford with a perched outlet (consider, however, potential adverse channel effects).
Is a grade control structure needed?	If YES to promote aggradation, first consider an improved unvented ford with a raised platform (a low dam). A vented ford with perched vents may also work. To stop headcutting, consider using a structure with a solid, stable bottom and downstream cut-off wall

9.3.2 Culverts

Generally, culverts are placed in relatively small crossings with maximum design discharge of around 40 m^3/sec. Flows larger than this will entail many multiples of culvert pipe or boxes and a small bridge may become a more suitable choice.

Culverts are unlikely to be suitable where the channel is quite entrenched and where much debris is expected to be entrained. In these cases a small bridge may be a more suitable choice. Table 9.9 shows the criteria for selecting a culvert.

Table 9.9 Criteria for selection of a culvert

Criteria	Most Applicable
Access Priority	Moderate to high
Alternative route	May not be available
Traffic speed	Medium to normal road speed
Average daily traffic	Any
Flow variability	Low to medium
High-flow duration	Any
High-flow frequency	Any
Debris loading	Low
Channel Entrenchment	Shallow

The choice of whether to use box or pipe culverts depends on various factors. In general box culverts are easier to construct either *in situ* or in a local yard. It may not be possible for pipe culverts of suitable quality and reinforcement to be manufactured locally and this may limit use of this type of culvert in some areas. Box culverts do also have the advantage that by they have a flat top slab which allows easier road construction of over the culvert and it is easier to construct the crossing with a flush surface to normal road elevation to allow maintained of traffic speed. Pipe culverts have to be set lower to allow a flush road surface and need to be covered by a layer of compacted soil to spread traffic load.

9.3.3 Small Bridges

Generally small bridges are more expensive to construct than culverts so they are often placed where culverts are not suitable due to such factors as: topography e.g. across deeply incised channels; where there is a high debris load expected which is likely to block the restricted openings of culverts or; where design flood discharge would entail installation of too many culvert barrels. Table 9.10 shows the criteria for selecting a small bridge.

Table 9.10 Criteria for selection of a small bridge

Criteria	Most Appropriate
Access Priority	High
Alternative route	May not be available
Traffic speed	Normal road speed
Average daily traffic	Any
Flow variability	High
High-flow duration	Any
High-flow frequency	High
Debris loading	High
Channel Entrenchment	Any

9.4 Climate change impacts

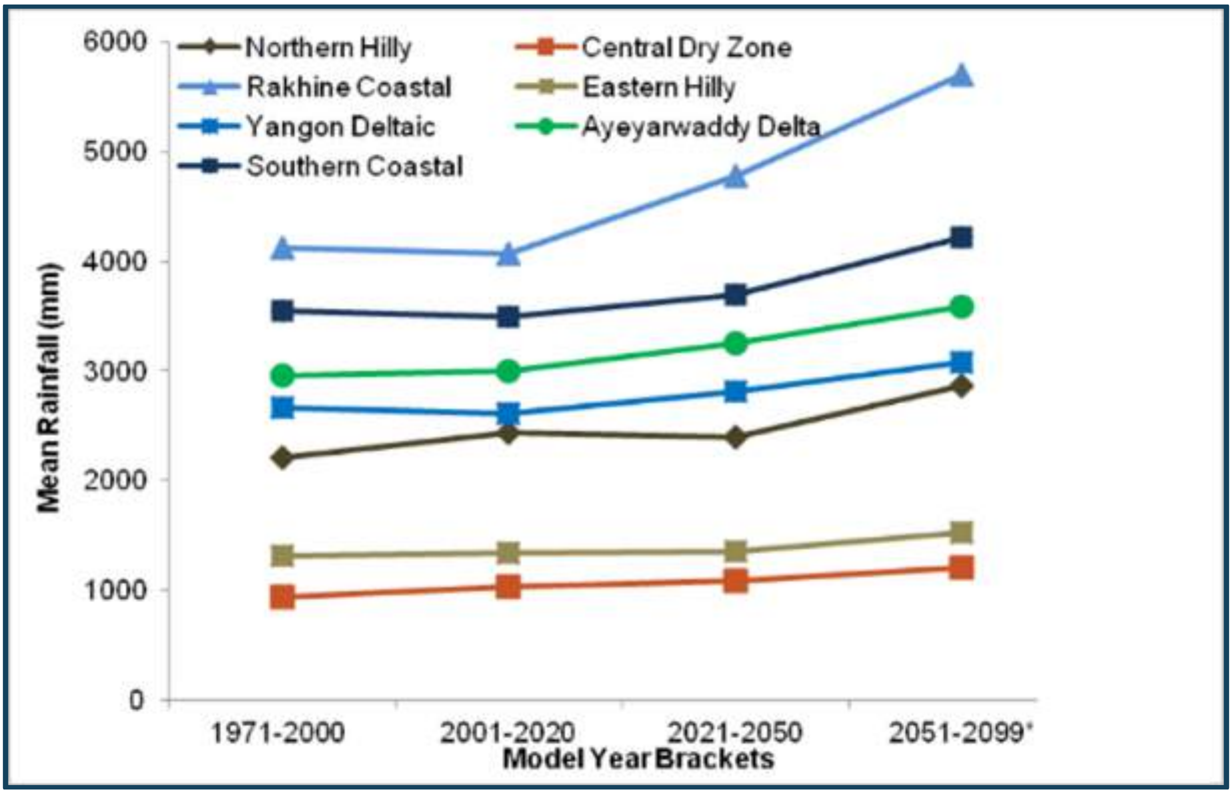
Run-off (for drainage/culvert design) determination is based on estimates of the amount of rainfall that will drain from a given catchment within a given period. Usually, historical records of observed rainfall are used to determine the average return interval (ARI) in years for maximum rainfall. This rainfall is then translated into run-off using catchment-specific models to determine maximum flows and design criteria for the road drainage system.

Climate change is expected to affect these design calculations in a number of ways, through increasing the intensity and frequency of heavy rainfall events, and through changing the antecedent moisture loading of soils and the average water contained in storage ponds. The most significant change is expected to come from a general increase in the maximum rainfall associated with heavy rainfall events. The reason for this change is that a warmer atmosphere can generally hold more water, so that more water is available during any particular rainfall event. As a first approximation, the increased amount of rainfall over a typical 24-hour heavy rain event is scaled in proportion to the increase in temperature. The expected change of the absolute rainfall depth in design storms can be incorporated into standard design calculations and subsequent run-off modelling.

More complex rainfall modelling is required where catchments are large with a complex topography, or have specific characteristics that warrant a more detailed investigation of changes in rainfall characteristics.

Other climate changes that may be of relevance to design are changes in temperature (temperature influences evaporation rates from organic surfaces and hence initial water retention capacity), and changes in mean rainfall (which also affects the initial water retention capacity of pervious surfaces). Both aspects could therefore lead to a shift between the fast and slow run-off components of a run-off model. It is important that this manual, which may be in use for many years, takes due consideration of climate change to allow climate resilient design (Chapter 4).

Figure 9.14 Predicted rainfall trends for the seven physiographic regions in Myanmar



Source: Myanmar's National Adaptation Programme of Action (NAPA) to Climate Change

As can be seen in Figure 9.14 for the design life time horizon of LVRR roads, there would appear to be small change in predicted annual rainfall. However, a conservative approach is recommended as predictive models have, historically, often underestimated change. The annual graph, importantly, does **not** take into account increased seasonality and intensity of rainfall.

The likely increased frequency of storms, the increased spatial variability and increased intensity of the rainfall are aspects having the most impact on road drainage and must be taken into consideration in preparing climate resilient designs. The impacts of the likely increased short duration storms of high intensity rainfalls may be significant. These will result in increased rapid run-off often carrying debris (vegetation). Traditionally, for rural roads, it is often recommended that drainage design considers an ARI of about 10 years but, taking account of the forecast climate change impacts, the historically determined 20 year ARI rainfall storm data should now adopted for the drainage designs. Adjustments recommended include:

1. Increased design capacity of culverts i.e. the minimum culvert size of 0.8 m x 0.8 m is recommended instead of 0.6 or less width or pipe diameters used traditionally,
2. Closer spacing of cross-drainage structures than traditionally used (especially for the roads with more erodible soils)
3. Improved road side drainage with lining installed at lesser slopes than traditionally used
4. Increased design capacity of bridges to allow for increased flows as well as passage of debris (trees)

Many townships in southern Myanmar are with within the Ayeyarwady and Sittoung delta area thus, water levels adjacent to the road may often be controlled by regional water levels associated with the Ayeyarwady River rather than locally produce runoff. It should also be noted that this area in in the Cyclone Zone as set out below.

According to according to the National Adaption Programme of Action Myanmar the following changing cyclone regime is found in southern part of Myanmar.

> *In the past (before 2000), cyclones made landfall (i.e. centre of the storm moved across the coast) along Myanmar's coast once every three years. Since the turn of the century, cyclones have made landfall along Myanmar's coastline every year. From 1887 to 2005, 1,248 tropical storms formed in the Bay of Bengal. Eighty of these storms (6.4% of the total) reached Myanmar's coastline. Recent cyclones of note include Cyclone Mala (2006), Nargis (2008) and Giri (2010). Cyclone Nargis hit the coast in May 2008 and was the most devastating cyclone on record that Myanmar has ever experienced. The Ayeyarwady Delta and the eastern part of Yangon were most affected experiencing wind speeds >258 km/h. The main impacts included: i) extensive damage to mangroves, agricultural land, houses and utility infrastructures; ii) salt-water intrusion into agricultural lands and freshwater sources causing economic, social and environmental damage; iii) loss of livelihoods and homes (3.2 million people affected), including 138,373 deaths; and iv) damages of ~US $4.1 billion. Cyclone Giri hit the coast in October 2010 resulting in a maximum storm surge of approximately 3.7 m and wind speeds in excess of 120 km/h. The Cyclone caused damage and loss of government buildings, households, schools and farm assets. The death toll was significantly less than that of Cyclone Nargis (45 people). However, the cyclone resulted in 70,000 people left without homes[4].*

[4] Myanmar NAPA report page 27 ibid.

It should be noted that it is unlikely that, based on cost, any LVRR in the southern townships can be protected from surges of several metres. A strategy of "living with floods" is perhaps a better approach and is used elsewhere in areas suffering from regular inundation such as in the Mekong Delta. In this approach roads are designed to be resilient to inundation and can be brought rapidly back into use after inundation subsides without any lasting damage. This approach depends on good construction of embankments and adequate relief culverts to allow excess water to drain away quickly.

9.5 Environmental Considerations for Choosing a Crossing Location in the Catchment

In addition to the large-scale road or road system evaluation, the site should be evaluated with respect to its watershed context. The entire watershed is linked via its drainage network and upstream and downstream changes can seriously affect a site. Although Table 9.10 is not an exhaustive list the questions that should be considered, it will help identify essential ecological processes that should be factored into decisions about crossing locations and design.

Table 9.11 Considerations for Choosing a Crossing Location

No	Design Queries	Issues	Action required for road design
1	Is the crossing on or just downstream from unstable landforms (e.g., alluvial fans, landslides)? Is it located in a depositional area?	Landslides and earthflows can intermittently produce large amounts of sediment that may cause downstream culvert structures to plug and fail. Alluvial fans and other depositional areas are located where valley gradient flattens or where a confined stream enters a wider valley. Crossing structures in these locations are subject to plugging. In addition, when deposition happens rapidly, such as during a large flood, the channel may shift to another location, leaving the structure isolated.	Topographic and soils and land cover survey of the catchment area to investigate soil stability and sediment transport in the river.
2	Is the channel stable at the watershed scale? Is there a headcut working upstream that could affect the site in the future? What changes from planned watershed development could affect channel stability upstream or downstream of the site and, therefore the site itself?	A stream is a dynamic continuum. Changes in watershed and channel conditions occurring upstream or downstream can affect any point on the stream. For example, streams continuously adjust in response to floods, changes in sediment loads, or changes in riparian conditions that control bank stability. Channel incision initiated by, for example, gravel mining or channel straightening can migrate upstream, affecting the entire system's bank and bed stability. An existing or planned dam in the river system will change the channel sediment load and can affect streambed elevations.	Topographic and soils survey of the catchment area to investigate soil stability and sediment transport in the river. Survey of sand and gravel mining activities in the river channels and adjacent areas
3	What types of flows are expected from the watershed? Are base flows steady, or is the watershed "flashy" with brief, high peak flows? Is most flow clear water or do high flows carrying a lot of sediment and debris?	As well as the rainfall regime in the catchment, parameters such as catchment shape and slope and vegetation cover great affect the flow regime of rivers. Soil erosion and area of human disturbance of the catchment such as forest clearance and agricultural activities on steep slopes significantly increase erosion and sediment transport and deposition in the river channels.	Survey of rainfall runoff relationships (RRR) in the catchment should be carried out as part of river crossing designs. The RRR in conjunction with the results of the topographic, soil and land cover surveys should be used to better understand the design requirements for river crossings
4	What hydrologic changes are likely to occur due to any planned watershed development? How might the current streamflow regime (i.e., flow quantity, timing, and duration) change over the structure lifetime?	Changes in land cover, such as road and housing development, fires, or timber harvests, can change the proportion of precipitation that runs off quickly in floods. Because the road network connects directly to the stream system through ditches and crossings, runoff is delivered to the stream system more quickly, increasing peak discharge and stream power. The increase in the erosive capabilities of the stream can lead to the undermining and outflanking of a structure. If major development is foreseen, consider selecting structures with larger capacities, and upgrading or re-armoring existing structures. The same changes that increase peak flows may also decrease baseflows because a greater proportion of precipitation runs off rather than infiltrating into the soil mantle for storage and slow release later in the season. If such decreases are foreseen, consider changing the crossing structure design to ensure low-flow passage for aquatic organisms.	A socio-economic survey of activities in the catchment is required to estimate possible future changes in the catchment flow and sedimentation regime, thus affecting cross-river structures in the catchment area.

No	Design Queries	Issues	Action required for road design
5	What aquatic biota is present? What are their passage needs?	Vulnerable species can be greatly affected by flow regime changes such as migratory passage blocking cause by cross river structures such as fords.	Ecological surveys should be carried out to identify vulnerable aquatic species and mitigation measures they may require to ensure sustainability of the ecology.
6	What are the constraints on crossing location, (e.g., nearby archaeological sites, private land, location of threatened, endangered, or sensitive plants.)	The impact on historical and other sites of cultural significance needs to be understood in order to maintain cultural objects such as sites of religious significance near water courses.	A cultural survey of the road alignment needs to be carried out including where the river crosses water courses.
7	How is the current crossing affecting the stream? Is it causing sediment deposition (aggradation) upstream and/or incision (degradation) downstream? Is it causing bank erosion?	Entrenched channels require a quite different approach to river crossing design to those which are on active flood plains which may need more structures such as embankment and relief culverts rather than a single river crossing structure. Entrenched channel crossings usual have their design capacity estimated by the upstream catchment conditions whereas flood plain areas may be affected by downstream conditions. This is especially so in the southern delta areas of Myanmar where flooding in the large deltas may be regional and prolonged.	In entrenched channels design capacity of the structure can be estimated using catchment parameters such as rain runoff relationships. In flood plain areas design may rely more on historical information of previous flood levels.

9.6 Hydrological design for crossings and drains

9.6.1 General Approach

Drainage structure design should be based on some reasonable design flow, as well as site characteristics and environmental considerations such as irrigation use. Determining a reasonable design flow for any engineered drainage structure is critically important, both for the structure to perform properly and to prevent failures of structures. A reasonable design flow is commonly based upon a storm event that will have an ARI of 10 to 100 years, depending on type and value of the structure and local regulations. In general, in Myanmar LVRRs are designed to a 10 year ARI standard with bridges over 12 metres having a 50 to 100 year ARI.

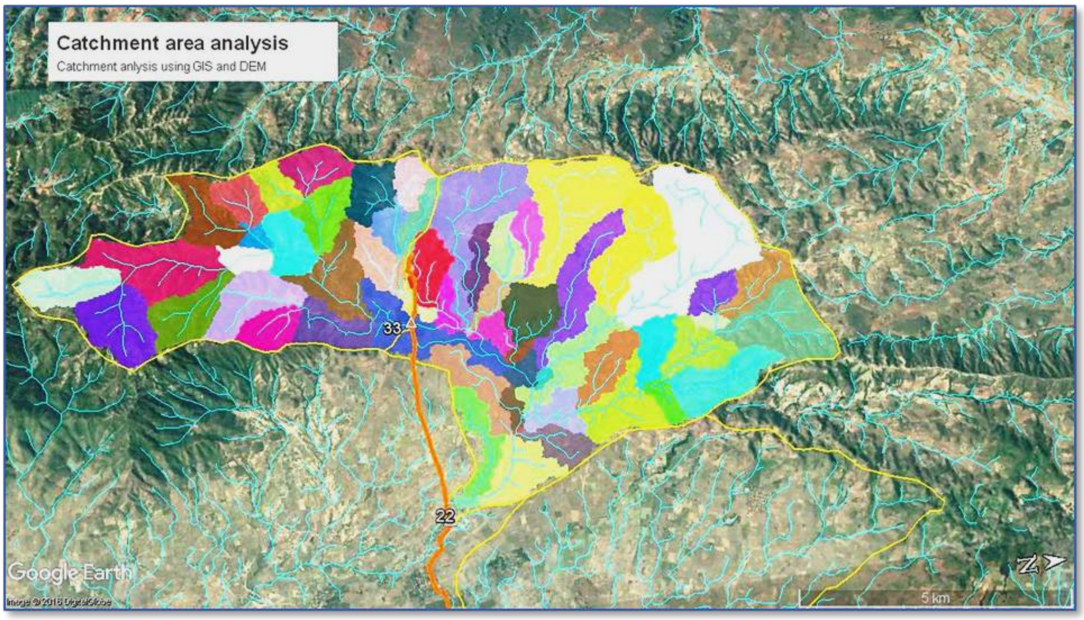
Any culvert of other crossing structure has a finite flow capacity that should not be exceeded. Bridges also have a specific capacity for the given cross- sectional area, but typically it is large. Drifts and vented drifts design is based upon estimates of both low flows and peak flows for that specific drainage, but are less sensitive to flow estimates.

For a known catchment area several methods can be employed to estimate a design flow for a given return interval. For catchments with bridges of less than ten metres span, that are being considered for this manual, it is very unlikely that a gauging station with a record of actual flows will be available in the catchment, or in adjacent similar catchments, so that directly measured flow data will not be available. Therefore, indirect methods to estimate design discharges with be required at almost all crossing locations. These are based on: the catchment (watershed) area draining to the structure and the rainfall intensity duration frequency (IDF) relationship.

9.6.2 Catchment area

To determine a design flow most design methods require that the drainage catchment area is defined. Traditionally this has usually been carried out using topographic maps delineating the catchment (water shed) boundary by following the ridge contours on the map to enclose the drained area. This area can then be measured using a plan meter or counting grid squares enclosed. Disadvantages with this method are that it is labour intensive and is prone to map misinterpretation leading to error. Topographic maps are also not necessarily immediately available in Myanmar.

Figure 9.15 Catchment area analysis using a DEM and GIS



The preferred method is to use a digital elevation model (DEM) and catchment analysis using a geographic information system (GIS). The DEM data can be downloaded for the area of interest in Myanmar anywhere there is an internet connection.

Several DEMs are available for public download at no cost[5]. As well as proprietary GIS, public domain GISs such a QGIS are also available to use at no cost and can carry out the required catchment analysis to provide both catchment area and the alignment of the main water courses.

Figure 9.15 shows a typical catchment area reporting to a crossing structure that has been analysed using a DEM analysis by GIS and overlaid on a Google Earth Image.

9.6.3 Rainfall Intensity duration frequency (IDF)

IDF curves are preferably developed from instantaneous rainfall record of at least 10 years in length from rainfall, stations within a representative distance of the crossing point. Instantaneous record is produced by modern data loggers using tipping bucket or electronic weighing rain gauges.

In entrenched channels design capacity of the structure can be estimated using catchment parameters such as rain runoff relationships. In flood plain areas design may rely more on historical information of previous flood levels.

Nevertheless, daily and monthly rainfall records of more than 10 years are often available as each township generally has an operational climate station. Many of these climate stations were upgraded in the 1990s and records are available for a good length of time in many townships. Using monthly rainfall data and IDF curve approximation can be developed based on a relationship between the annual maximum monthly rainfall and the intensity and duration of rainfall likely to occur. As an example, using 20 years of monthly rainfall data collected at Magway station in the Central Dry Zone an IDF relationship for a range of ARIs has been developed. This is shown in Table 9.12 and charted graphically in Figure 9.16.

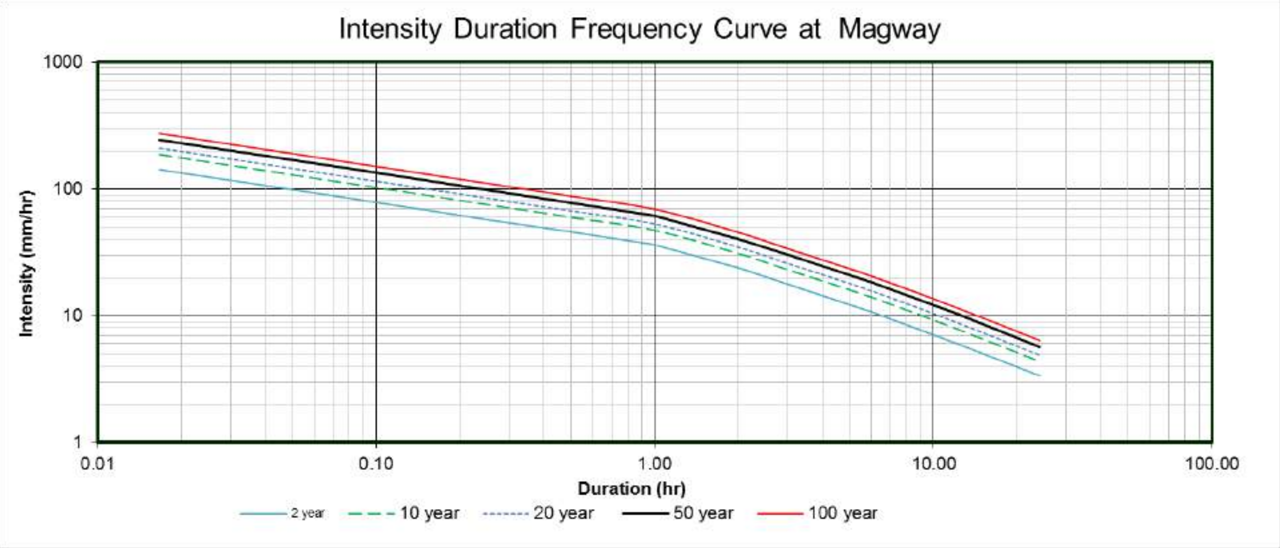
Table 9.12 Intensity Duration Frequency (IDF) table at Magway (Central Dry Zone)

Time		Intensity (mm/hr.)						
(minutes)	(hours)	1 year	2 year	5 year	10 year	20 year	50 year	100 year
1	0.02	127	143	166	187	209	244	274
5	0.08	74	83	97	109	122	142	160
10	0.17	59	66	77	86	97	113	127
15	0.25	51	58	67	75	85	98	111
30	0.5	41	46	53	60	67	78	88
60	1	32.1	36	42	47	53	62	69
120	2	21.0	23.6	27.5	30.9	34.7	40	45
180	3	15.7	17.6	20.5	23.0	25.8	30.1	33.8
360	6	9.5	10.7	12.4	14.0	15.7	18.3	20.5
720	12	5.4	6.1	7.1	7.9	8.9	10.4	11.7

[5] The following web site may be used to down load several type DEM covering Myanmar including the SRTM and Aster DEMs : https://earthexplorer.usgs.gov/. DEMs are available worldwide at a resolution of 1 arc second (approximately 30m). This is a free site and anyone can open an account to download data at no cost.

1440	24	3.0	3.3	3.9	4.4	4.9	5.7	6.4

Figure 9.16 Intensity-Duration-Frequency-Curve at Magway



9.6.4 Rainfall Runoff Coefficients

Table 9.13 shows typical runoff factors from various land types which can be used in the development of storm design runoff using the Rational Method.

Table 9.13 Rational Method typical values of runoff coefficient "C"

Land cover	Runoff Coefficient
Bare Soil	0.20-0.60
Cultivated Fields (sandy soil)	0.20-0.40
Cultivated Fields (clay soil)	0.30-0.50
Turf, Meadows	0.10-0.40
Steep Grassed Areas	0.50-0.70
Wooded Areas with Level Ground	0.05-0.25
Forested Areas with Steep Slopes	0.15-0.40
Bare Areas, Steep and Rocky	0.50-0.90
Asphalt Pavement	0.80-0.90
Cobblestone or Concrete Pavement	0.60-0.85
Gravel Surface	0.40-0.80
Native Soil Surface	0.30-0.80
Residential, Flat	0.40-0.55
Residential, Moderately Steep	0.50-0.65
Commercial or Downtown	0.70-0.95

9.6.5 Design Discharge

Table 9.14 shows the types of methods that can be used for determining design discharges discharges for drain and river crossing sizing.

Table 9.14 Design flow analysis methods for various catchment sizes

Catchment type	Catchment area	Typical type of analysis
Small	<2500 ha	Rational method, local experience
Medium	4,000 ha to 10,000 ha	Regression analysis, historical flood marks, manning's, local experience
Large	>4,000	Gauging data, high water marks, statistical methods or regression analysis

Typically for small catchment areas the Rational Method is often used.

This method allows calculation of runoff of a catchment area by using the Rational Formula as follows:

Q= 0.278 C I A

Where:

Q = flow in cubic metres per second (m^3/sec)

C = run off coefficient, expressing fraction of the rainfall that is assumed to become direct runoff (typical ground cover values of C are shown in Table 9.13)

I = intensity of rainfall in mm/hour for the durations of the corresponding time of concentration for the catchment. (Table 9.12 and Figure 9.16 shows typical values for a range of T_cs as calculated by the method shown below)

A = the drainage area in km^2.

T_c = the time of concentration. This is the time period (duration) required for rain water to reach the outlet from the most remote point in the drainage area

The formula used to calculate T_c is:

$T_c = (0.87\ L^3 / H)^{0.385}$

Where:

L = the length of the catchment area in kilometres

H = the difference in level for the highest point of the catchment to the outlet level (at the road crossing).

The Rational Formula is based on the theory that the runoff rate is linearly related to rainfall intensity. This means that the runoff rate would become constant if a uniform rain of a constant intensity falls on an impervious specific area. The actual runoff, which varies over the area, is however far more complex than the formula indicates.

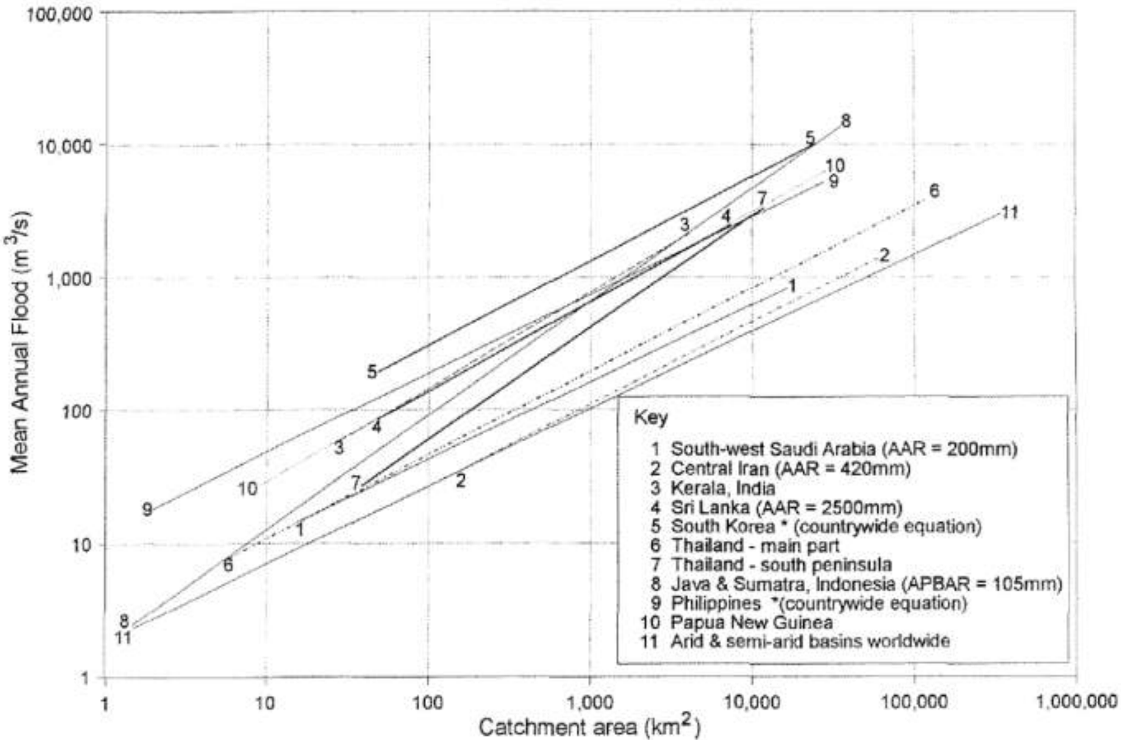
Since the error of runoff estimate increases with increasing size of the drainage area , the Rational Formula is normally limited to an area size of about 25 km^2 (2500 ha).

For larger areas the formula should be used with care and the catchment split into small areas with uniform runoff coefficient rates. Other empirical, graphical or statistical formulas should be considered where catchment areas exceed 25 km2.

Often for larger catchment the best indication of flood flow in the catchment is local knowledge of previous floods or existing flood marks.

Regional flood analysis is a commonly used procedure to develop flood estimates for catchments where little or no flood data exists. It is also a useful procedure providing an independent assessment of design floods that are computed by other methods. The regional flood frequency curves have their most useful applications in estimating the flood potential of an ungauged catchment Regional flood frequency curves show the ratio of floods for a given return period relative to the mean annual flood (MAF). It is therefore necessary to make an estimate of the mean annual flood for the ungauged catchment. The mean annual flood is dependent upon many variables, the most important and commonly available being the drainage area. The mean annual flood for a particular catchment is determined graphically by plotting mean annual floods against respective drainage areas of all gauged stations in the region. The flood of any given frequency for the ungauged area is then obtained by determining the corresponding flood ratio from the regional frequency curve for the region of which the ungauged basin is a part and multiplying it by the estimated mean annual flood of the ungauged basin. Some typical regional mean annual floods are shown plotted in Figure 9.17[6] (J. R. Meigh & F. A. K. Farquharson et al). It can be seen that the man annual floods r vary widely with region. Unfortunately, due to lack of available historical data Myanmar has not been included in the chart shown in Figure 9.17. However improved data sets for Myanmar are becoming available as the hydrometric system of Myanmar is now being currently upgraded. Inclusion of re-evaluation and historic data previously unavailable and may allow regional curves to be developed. It should be noted that using the regional flood method is likely to be most useful for larger crossings.

Figure 9.17 Comparison of regional mean annual floods



Source: A Worldwide Comparison of Regional Flood Estimation Methods and Climate

[6] Reference: *A worldwide comparison of regional flood estimation methods and climate*, J. R. Meigh & F. A. K. Farquharson ,Hydrological Sciences-Journal-dés Sciences Hydrologiques, 42(2) April 1997

9.7 Hydrological data sets

Table 9.15 shows the data sets are required to carry out analysis for crossing designs allowing design storms of the selected average return interval (ARI) to be for each crossing.

Table 9.15 Data Sets and Sources Required to Develop Design Storms

Data type	Source
Rainfall data	Department of Meteorology and Hydrology, Ministry of Transport and Communications
Climate data	
Hydrological and Hydrographic data	
Land cover	Google Earth, Remote sensing data e.g. landsat https://earthexplorer.usgs.gov/
Digital elevation models (DEM)	Aster, STRM, other https://earthexplorer.usgs.gov/

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10 Earthworks

10.1 Introduction

In order to comply with horizontal or vertical geometric guidelines and thus permit reasonable access for users, Low Volume Rural Road (LVRR) alignments in Myanmar within hilly or mountainous areas may require the construction of cut or embankment earthworks. Embankments may also be required in low lying areas to raise alignments above flood levels, or as approaches to bridges. These earthworks should be designed to minimise subsequent slope failure by implementing designs and construction procedures that are compatible with the engineering properties of the excavated soil-rock or the placed fill, whilst at the same time considering the impact of these earthworks on existing slopes or foundations.

Cut and fill earthworks are likely to be of particular significance in the mountainous or steep hilly regions such as parts of Shan and Chin states, whilst the lower lying area of the coastal and the delta and coastal states, may require alignments to be raised on embankment; frequently on soft and compressible soils.

The interaction of LVRR route alignment and the geometry or instability of the natural slopes may be such that construction to recognised safe angles is not an economical or engineering feasibility. If temporary road closures and debris clearance can be tolerated and allowed for in maintenance, then steeper slopes may be an economic option, otherwise engineered or bio-engineered stabilisation or protection measures may have to be considered. This is will be the case particularly in areas of identified natural hazard or where significant potential climate vulnerabilities have been identified.

This Chapter covers aspects of earthwork design including embankments, cut-slopes and imported subgrade. It contains sections on low cost erosion and slope protection including the use of cost-effective bio-engineering options. This Chapter also outlines the potential vulnerability of earthworks to current and future climate impacts and the consequent importance of adequate earthwork drainage.

10.2 General Principles of Earthwork Design and Construction

10.2.1 Alignment and Earthwork Balance

In an ideal situation there should be a focus on achieving an “Earthwork Balance”, whereby the amount of excavated materials is equal to the volume of fill required; thus removing the requirement either for spoil disposal or for additional borrow areas. In many cases this target of balanced earthworks is not possible, particularly in low lying flat coastal or steep mountainous areas. It should, however, be a design target to reduce excess spoil or excess borrow to minimum, on both cost and environmental impact grounds. In this context the innovative use of marginal earthwork materials and approaches to cut-slope stabilisation should be a priority for Myanmar LVRR designers.

10.2.2 Excavation

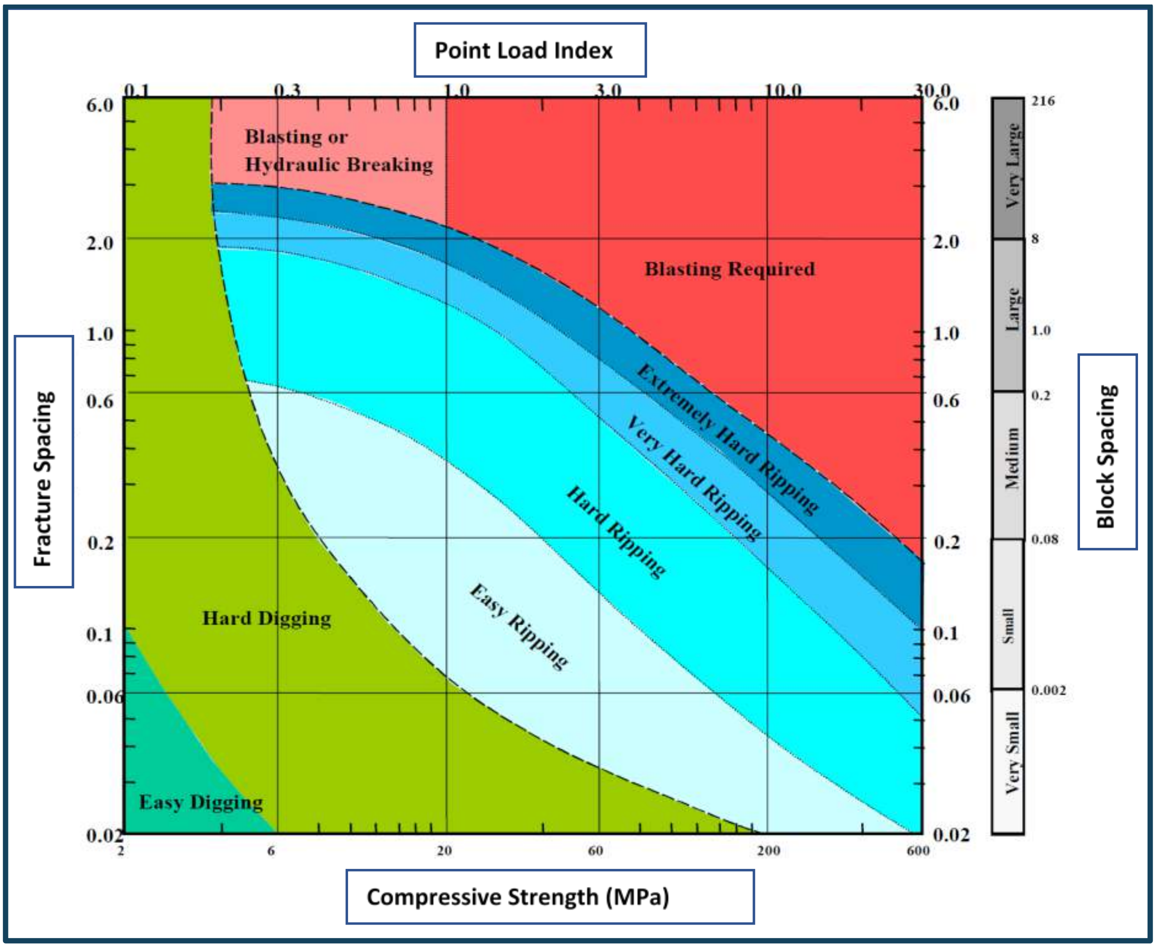
Methods of excavation are determined by a combination of intact soil-rock strength and rock mass structure, Figure 10.1 (Pettifer & Fookes, 1994).

Excavation of rock slopes should be undertaken in such a way that disturbance, for example, due to blasting, is minimised (Harber et al, 2011). 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 relevant specifications.

Structural excavation includes the excavation of material to permit the construction of culverts, foundations for bridges, or retaining walls. Suitable materials taken from structural excavations should be used either in backfilling around the completed structure or in other parts of the construction site.

When insufficient material for the formation of embankments is available from excavations performed within the limits of the right-of-way, additional suitable material is taken from borrow pits (Roughton International, 2000). In addition to geotechnical or material requirements, there are requirements relating to environmental impact compliance concerning the condition in which borrow pits should be left when they are abandoned (See Chapter 3, Section 5).

Figure 10.1 Assessment of Excavatability



Source: Pettifer & Fookes, 1994

10.2.3 Fill Placement and Compaction

Compaction is the simplest way of improving the engineering properties of soil or rock fill materials. Essentially, air is expelled, and particles are forced into closer contact with water volume remaining the same (Parsons, 1992). Compaction is aimed at:

- Reducing permeability;
- Reducing in service compressibility;
- Increasing shear strength;

Earthwork compaction is usually defined in terms of a percentage (usually around 85-90%) of a defined Maximum Dry Density (MDD) under a defined compactive effort (AASHTO Modified). Earth fills are constructed using relatively thin layers of soil. The maximum thickness of loose soil is usually 250–300mm. The soil is thoroughly compacted before the next layer is placed

Except as otherwise specified, rock embankments are normally constructed in successive layers of 0.75m or less in depth. When, in the opinion of the Engineer, the rock sizes necessitate a greater depth of layer and the height of fill will permit, the layer depth may be increased as required, but depth of layer should not exceed one metre.

Table 10.1 lists the more common options for compaction of fill (and also pavement layers; see chapter 7). For LVRRs there are likely to be a number of constraints on the selection of optimum machine options on site, for example:

- Small contractors may have limited capacity to employ to multiple machine options;
- Narrow carriageway widths combined with the need for maintaining cross-fall will the limit width of compactor drum;
- Small LVRR contractors may have limited experience in the application of some options.

In the common LVRR construction package in Myanmar of a single road, or section of a road, the aim should be to select the most flexible option. In this case the use of small to medium sized (3-5 tonne) smooth wheel rollers offers the option of both vibratory compaction and deadweight compaction. Special cases, for example extensive rock fill or high cohesive fills would call for additional specialist plant.

Table 10.1 Typical Compactor Options

Machine		Description and Use
1	Vibrating-Plate compactor	Small hand compactors used for compacting fill behind, for example, retaining walls or close to bridge or culvert structures.
2	Pedestrian Roller	Usually vibrating. Hand operating rollers for narrow access tracks or trails and possibly shoulders.
3	Smooth wheel deadweight roller	Traditional 8t three-wheel compactor commonly used in Myanmar. May also be smaller tandem option. Best suited to surface finishing.
4	Smooth wheel vibrating roller	Single or tandem drum – low frequency high amplitudes suited for thicker cohesive fills with higher frequency-lower amplitudes for thinner more granular material layers.
5	Deadweight sheepsfoot roller usually a vibrating option	Tamping machine with rod-like feet. Most effective in softer or wetter cohesive fills. Leaves an indented surface which may require finishing. Unsuitable for near surface subgrade or pavement layers. Not suitable for rock fill.
6	Pneumatic tyred roller	The kneading effect of the deadweight and tyre pressure is very effective fine granular materials. Specific requirements for some sealing options – particularly Otta Seal
7	Grid rollers	Heavy steel mesh drums used for breaking up oversize but weak rock-fill.
8	Impact rollers	“Square-wheel” specialist rollers for either thick lifts or for compacting in situ voided or collapsible soils.

Figure 10.2 Typical LVRR Compacting Machines

a. Vibrating Plate Compactor

b. Pedestrian Roller

C. Three Wheel Deadweight Roller

D. Small 2t Smooth Wheel Vibrating Roller

E. Small Sheepsfoot Roller

F. Impact Roller

10.3 Cut-Slope Design

10.3.1 General Approach

Cut slope design for LVRRs is likely to be initially guided by existing standards or on past experience with similar soil and rock materials in similar environments, rather than individually designed. However critical cuts particularly greater than 10m in height may require a more detailed engineering geological or geotechnical assessment, depending on the complexity of the ground conditions. This would include an assessment of the strength of the soil-rock materials and the mass structure (See Chapter 5).

The slope angles indicated in Table 10.2 (derived from regional experience based on Ingles, 1985) have been provided as a general guide for LVRRs. It is emphasised that these angles cannot be applied without due consideration of the actual ground conditions.

Table 10.2 General Guidance on Cut-Slope Angles (V:H)

Rock Materials (1)		Slopes (V:H) for Various Cut Heights		
		< 5 m	5-10 m	10-15 m
Strong to moderately strong rock. granite, basalt, limestone, sandstone. (with no adverse structure)		1:0.2	1.03 - 1.08	
Weak rock. Shale, bedded sandstone, moderately weathered granite, basalt, limestone, sandstone		1:0.4 – 1.05	1.06-1.08	
Very weak rock. Mudstone, highly weathered granite, basalt, limestone, sandstone		1:1	1:1.5	1.1.5-1.75
Soil Materials (2)				
Coarse colluvium	Poorly graded	1:1.25-1:1.5		
Clayey sand soil	Dense or well graded	1:0.8 – 1:1.0	1:1.0-1:1.5	(3)
	Loose	1:1.0 – 1:1.5	1:1.2-1:	(3)
Sandy soil, mixed with gravel or rock	Dense, well graded	1:0.8 – 1:1.2		1:1.0 – 1:1.5
	Loose, poorly graded	1:1.0 – 1:1.5		1:1.5 – 1:2
Cohesive soil		1:1.0 – 1:1.2		(3)
Cohesive soil, Mixed with rock or cobbles		1:1.0 – 1:1.2	1:1.2 – 1:1.5	(3)

Notes (1): Rock slope angles may be governed to a large extent by interacting discontinuities (joints, bedding, faults). (2) Residual soils may contain relict structure that influences stability (3) Slopes will require special investigation or comparison with existing earthwork stability

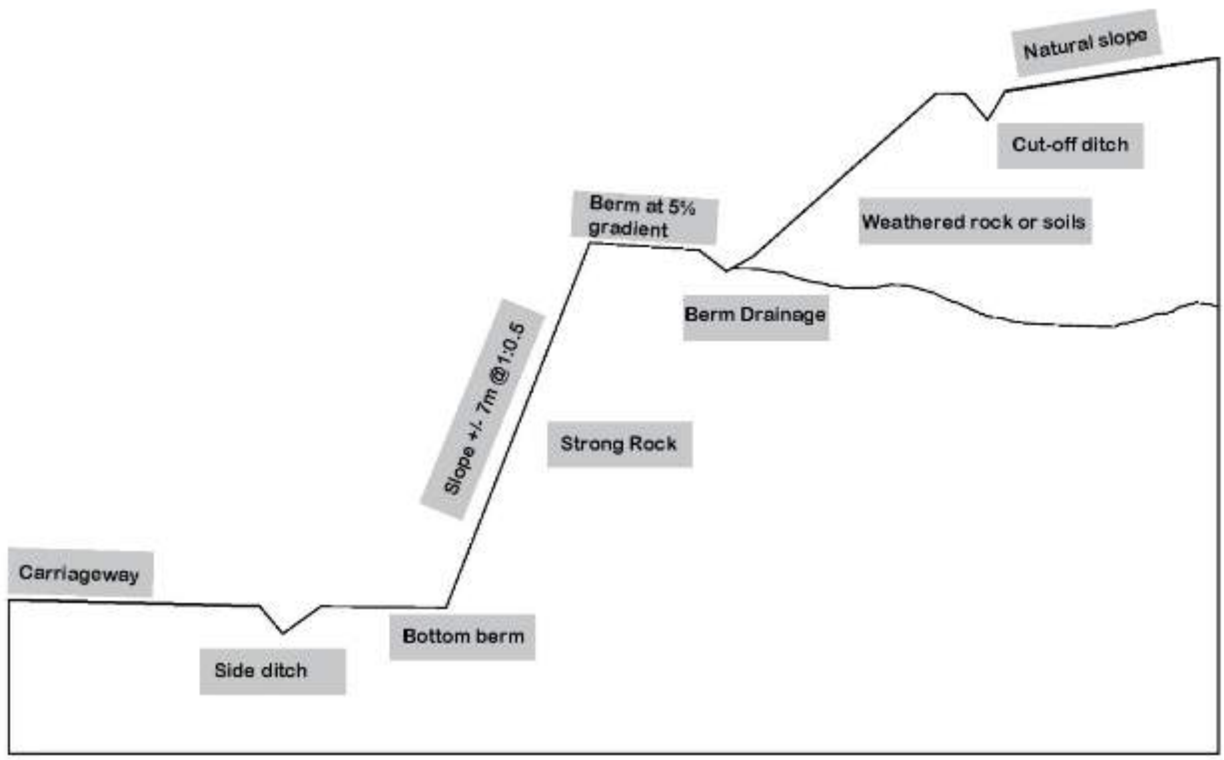
As Table 10.2 indicates, cuttings in strong homogenous rock masses can often be very steep where adverse structure is not present although, persistent joint, bedding or foliation surfaces may determine the final cut slope profile. In heterogeneous slopes, where both weak and hard rock occur, the appropriate cut-slope angle can be determined on the basis of the location, nature and structure of the different materials and the variations in permeability between the different horizons (Harber et al, 2011).

One of the most effective ways to decide upon a suitable cut slope is to survey existing cuttings in similar materials along other roads or natural exposures in the surrounding areas. Generally, new cuttings can be formed at the same slope as stable existing cuttings if they are in the same material with the same overall structure (Cook et al, 1992).

Benched slopes are generally used in deeper cuts (>10m) or where layered soil rock profiles are encountered. The construction of benches should be considered as a means to intercept falling debris and control the flow of water. Berms are commonly provided at every 5-10m height of a slope or, if possible, at material boundaries and changes of slope angle. If land-take is available, then a bottom berm is advantageous beneath erodible slopes to prevent debris continually falling on the carriageway or side ditch (ICIMOD, 1992).

There is no hard rule regarding the dimension of benches, but bench widths of 1.5 to 2.0m are common with a bench drain and cross-sectional slope of up to 5% (Figure 10.3). 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.

Figure 10.3 Schematic Cut Profile



10.4 Embankment Design

10.4.1 General Approach

Embankment design has to take into account two key elements; the character of the available fill materials and geotechnical nature of the foundations. Embankment slopes and cross sections should be designed also taking account the geometric alignment requirements and terrain.

Embankments impose a load on the underlying foundation soil. On some soils, this may result in settlements and, if the foundation soil is extremely weak, an embankment slip failure may occur. The settlement characteristics of soil profiles vary considerably from minimal-problem well-drained granular soils to geotechnical difficult soft clay or organic soils.

Most types of soil and broken rock can be used for construction of embankments, but materials of the AASHTO classification A-1, A-2-4, A-2-5 and A-3 are preferred (USDI,1998). More plastic materials may create problems in wet weather. Highly expansive clays and organic soils should not be used as fill. Typical angles for embankment fill on sound foundations are presented in Table 10.3.

Table 10.3 Suggested Fill Slope Gradients

Fill materials	Embankment Side-slope (V:H) for Various Heights			
	5m	5-10m	10-15m	15-20m
Well graded sand, gravels, sandy or silty gravels	1:1.5 – 1:1.8		1:1.8 – 1:2.0	
Poorly graded sand	1:1.8 – 1:2.0			
Weathered rock spoil	1:1.5 – 1:1.8			1:1.8 – 1:2.0
Sandy soils, hard clayey soil and hard clay	1:1.5 – 1:1.8		1:1.8 – 1:2.0	--
Soft clayey soils	1:1.8 – 1:2.0		-	--

Note: Derived from regional experience based on Ingles, 1985.

For embankments > 5-10m in height or on steep sidelong ground, detailed geotechnical investigation and analysis may be required. The overall stability of a fill slope on a hillside may be difficult to assess. Before constructing a fill slope on side-long ground, it is necessary to terrace or step the formation in order to prevent a possible slip surface from developing at the interface between the fill and the natural ground. Problems can occur when strata or foliations in the rock masses beneath the fill are dipping parallel to the ground slope, or where the groundwater table is at or very close to the surface. Effective sub-surface drainage is a key requirement in such cases.

10.4.2 Soft Soils Foundations

The aim of the embankment designer should be to ensure that any settlement should be complete prior to the placement of the road pavement. In the case of problem soils this may require the use of special construction techniques. LVRR designs in the coastal regions of Myanmar can benefit from recent research in Bangladesh (Mott Macdonald, 2017) which, building on work in Indonesia (WSP, 2001), identified a range of techniques from the literature review, and discussions with local stakeholders:

- Excavate and Replace / Displacement
- Sand Compaction Pile
- Sand Drain (with surcharge)
- Prefabricated Vertical Drains (with surcharge)
- Geotextile basal reinforcement
- Cement Columns.

The constrained budgets of most LVRR programmes in Myanmar demands that the most geotechnical attractive solutions listed above may have to be rejected on cost grounds in favour of more pragmatic options. For embankments founded on soft soils the most usual design option in low-cost road engineering is excavation down to satisfactory strength materials where possible. Where this is not feasible then detailed geotechnical analysis will be required. Low-cost options for designing and constructing embankments on weak or problem spoils for LVRRs are outlined in Table 10.4.

Table 10.4 Low Cost Options for Embankment on Soft Ground

Technique	Application
Replacement	The weak or problem soil is removed, either partly or completely, and replaced by more suitable material. The embankment will then be founded on firmer ground and settlement will be greatly reduced. Granular free draining material (sand, gravel or a mixture of sand and gravel) should be used as fill material when filling is to be done below water. Acceptable cohesive soil can be used when the excavation is dry and the fill material can be compacted in lifts as normally specified. In partial excavation a layer of free draining material may be required as a drainage blanket at the base of the fill to speed up consolidation of the remaining soft layer during construction. The economic limits to full removal would be around 3-4 m.
Counterweight berms	The principle of counterweight berms is to add weight to the toe of the embankment to increase the resistance against slip or lateral spreading. When used in front of an approach fill to a bridge this method will increase stability thus reducing lateral pressure on the substructure. This option is very effective in solving stability problems with soft soils but will not solve the long-term settlement problem that may be particularly associated with organic materials.
Surcharging	Surcharging involves placing temporary additional load onto the proposed embankment to increase primary settlement. The load applied should be sufficient that the settlement during the construction period is equal to the total expected settlement from the embankment less the allowable post construction settlement. When the desired settlement has been achieved the surcharge is removed. The effectiveness of this method depends on the following factors: • Thickness of the soft soil; • Permeability of the soft soil; • The presence of drainage layers; • Available construction time; • Shear strength of the soft soil.
Staged construction:	As consolidation progresses in the soft soil under the embankment load, the void ratio in the subsoil decreases and. hence, density increases and the undrained strength increase and increase in shear strength of the subsoil is a function of the degree of consolidation. Therefore, the rate of filling can be controlled to allow sufficient consolidation to provide the required strength increase. This method should be considered when the design height of the embankment exceeds the critical height that can be safely supported by the subsoil.
Use of light material	The stability and amount of settlement of road embankments constructed on soft soil depend on the weight of the embankment; therefore, reducing the weight of the embankment will reduce stress in the subsoil and reduce excessive settlement and instability. By using lighter fill material than ordinary fill the weight of embankment will be reduced.

10.4.3 Expansive Soils

Expansive soils are those which exhibit particularly large volumetric changes (swell and shrinkage) following variations in moisture contents. Expansive soils can be thick and laterally widespread which makes the implementation of countermeasures costly, particularly for LVRRs. 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 (AfCAP, 2012) include the following:

1. Placing an uncompacted layer(s) of sand, gravel or rock fill over the clay and wetting up, either naturally by precipitation or by irrigation.
2. Pre-wetting (2-3 months) to induce attainment of the equilibrium moisture content before constructing the pavement.
3. Partially or completely removing the expansive soil and replacement with inert material.
4. Modifying or stabilizing the expansive soil with lime to change its properties.

5. Increasing the height of the fill (surcharge) to suppress heave.

6. Minimizing or preventing moisture change using waterproofing membranes

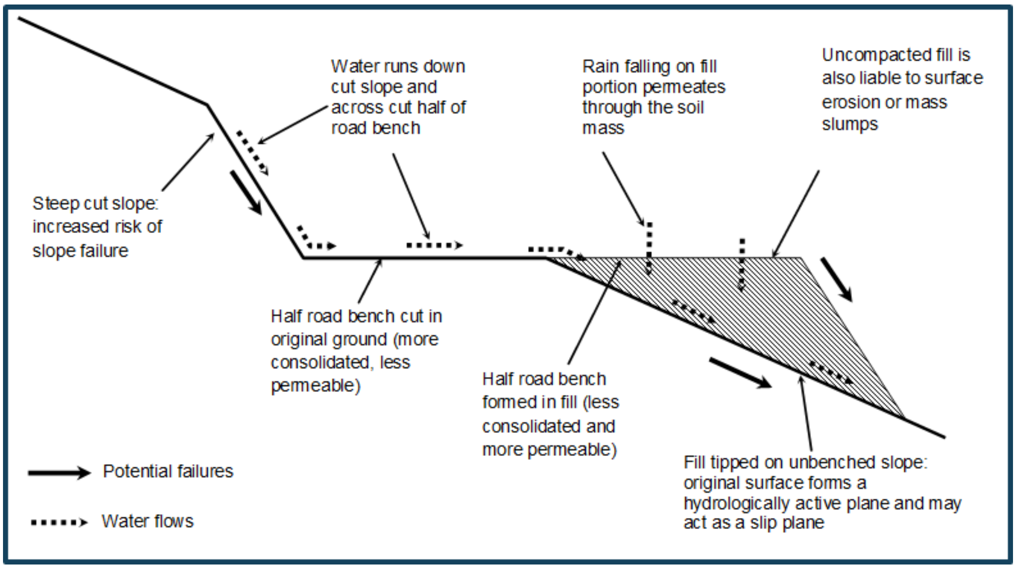
10.5 Cut to Fill

Cut-fill cross sections are a combination of excavation into hillside above the alignment and placement of the excavated fill on the "down" side. Although the cut-fill option is attractive in terms of earthwork balance it also a frequent cause of failure unless adequate design and construction precautions are adopted, Figure 10.4.

Vital requirements for an adequate cut to fill design are:

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

Figure 10.4 The Cut-Fill Situation



10.6 Slope Protection and Stabilisation

10.6.1 Earthwork and Natural Roadside Slope Instability

Roadside slope failures typically occur where a slope is too steep, a cut slope in soil and/or weathered rock contains weak erodible materials or adverse joints, or fill material is not properly compacted. In all three cases, the change in groundwater regime following rainfall can lead to an overall reduction in stability and possibly failure. Once slope failures are initiated they can develop rapidly. Erosion can also take place on unprotected earthworks as well as slopes adjacent to river channels, especially downstream of culverts, bridges and roadside turnouts. In addition, uncontrolled runoff can erode the roadside drains, road pavement and the edge of the road. Simple erosion occurrences can lead to larger slope failures if not dealt with.

Key references for the recommendations in the following sections are: Fookes et al, 1985; ICIMOD, 1992; DoR, Nepal, 2007; Geological Society, 2011.

10.6.2 Instability Location and Types

Roadside instability may involve one or more of the following situations

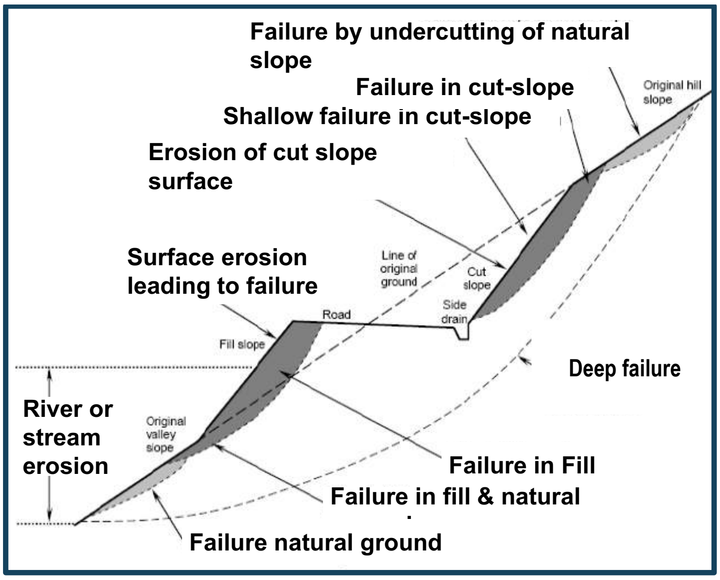
- Above alignment erosion or failure of cut-slopes;
- Below alignment erosion or failure of embankment slopes;
- Failure in natural ground;
- Combinations of the above.

Table 10.5 and Figure 10.5 summarise common instability scenarios.

Table 10.5 Common Types of Erosion and Slope Failure

Mechanism	Description	Depth
Erosion on the surface	Rills and gullies form in weak, unprotected surfaces. Most common on fill and tipped spoil. Erosion should be expected on all bare or freshly prepared slopes.	Usually in the top 0.1 metre, but can become deeper if not controlled.
Gully erosion	Erosion that is established on the slope continues to develop and grow bigger. Large gullies often have small landslides along the sides.	Usually in the top 0.5 metre, but can become deeper if not controlled.
Shear failure (translational landslide, planar or debris slide)	Mass slope failure on a shallow slip plane parallel to the surface. This is the most common type of landslide, slip or debris fall. The plane of failure is usually visible but may not be straight, depending on site conditions. It may occur on any scale, and large areas of subsidence may also be due to these.	Frequently 0.5 metre or less below surface (or along a local discontinuity) but may be several metres deep.
Shear failure (rotational landslide)	Mass slope failure on a deep, curved slip plane, with the toe of the debris rising slightly. Some small, deep landslides in residual soil are the result of this process, but generally they are not common.	Often more than 1.5 metres deep, even in a small failure.
Slumping or flow of material when very wet	Slumping or flow where material is poorly drained or has low cohesion between particles and liquefaction is reached. These sometimes appear afterwards like planar slides, but are due to flow rather than sliding. The resulting debris normally has a rounded profile. They may occur on steeply cultivated land following intense rainfall.	Frequently 0.5 metre or less below surface.
Rock or debris fall or collapse	Collapse due to failure of the supporting material. This normally takes the form of a rock fall where there is a weakness or fracture in a rock mass, or where a weaker band of material has eroded to undermine a harder band above.	0.5 to 2 metres in road cuts; deeper in natural cliffs.
Debris flow	In gullies and small, steep river channels (bed gradient usually more than 15°), debris flows can occur following intensive rain storms. This takes the form of a rapid but viscous flow of liquefied mud and debris. The depositional area may cover a broad area below the outlet of the channel.	The flow depth is usually 1 to 2 metres deep.

Figure 10.5 Typical Roadside Slope Failure Scenarios (After SEACAP, 2008)



10.6.3 General approaches

Slope stabilisation and erosion control can employ a range of methods to mitigate the causes of failure together with measures to improve stability. In the case of LVRRs in particular, 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 modifying earthworks (cuts and fills), retaining structures and revetments, surface and sub-soil drainage and bioengineering. Stabilisation methods that involve more substantial engineering works include anchoring, piling and deep subsurface drainage, but these are rarely used on low volume roads. For LVRRs, combinations of bioengineering, low cost retaining walls such as gabions, dry-stone and mortared masonry walls, and surface drainage structures are a cost-effective method to stabilise slopes, Figure 10.6, Table 10.6 and Table 10.7.

Figure 10.6 General Stabilisation Options (Dept of Roads Nepal, 2007)

A. Problem avoidance	B. Reduction of driving force	C. Internal strength increase	D. External restraint	E. Slope protection	F. Debris control	G. Bank protection	General Option	Earthfall	Rockfall-Topple	Rock Slide	Debris Slide	Soil Slide (R)	Soil Slide (T)	Debris Flow	Earth Flow (Soil Creep)	River Erosion	Slope erosion
√							**Removal**	1	1	2	2	2	2	2			
√						√	**Realignment**	1	1	2	2	2	2	2	2	1	
	√	√					**Earthwork**	1	1	2	2	2	2				
	√	√		√			**Drainage**			2	1	1	1	1	2		1
			√		√	√	**Retaining wall**			2	2	2	1	2	1	1	
				√	√		**Revetment Wall**			2	2	1	1	2	2		1
		√		√	√	√	**Bioengineering**				2	2	2	2	1	2	1
					√		**Check Dams**				1			1			2
			√			√	**Tied-back wall**			1	1	1	1	2	2	1	
			√			√	**Pile wall**			2	1	1	1	2	2	1	
			√				**Buttress**	1	1	2	2						
						√	**River Training**			2	2	2	2			1	
			√				**Anchors-Bolts**	1	1	2		2	2				
√					√		**Catch works**	2	2	2	2						
				√	√		**Surface Protection**	2	2	2							1

B,C D:	Primarily Slope Stabilisation	1	Principal option to be considered for problem solution
A, E,F,G	Slope protection, problem control	2	Secondary option

Table 10.6 Stabilisation Options for Above the Road Problems

Instability	Stabilisation options	Drainage options	Protection options
Erosion of the cut slope surface	Bioengineering and drainage (Section 10.7)	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, where gullies are long or slopes are very steep, small check dams may be required; Sometimes a revetment wall at the toe helps to protect the side drain.
Failures in cut slope	Reduce the slope grade and if this is feasible, then add erosion protection; A retaining wall to retain the sliding mass; For small sites where the failure is not expected to continue, a revetment might be adequate.	A subsoil drain may be required behind a wall if there is evidence of water seepage; Herringbone surface drains may be required if the slope drainage is impeded.	Bio-engineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation
Failures in cut slope and hill slope	Reduce the slope grade, and if this is feasible, then add protection. A retaining wall may be required to retain the sliding mass. This may need to be quite large, depending on the depth of the slip plane.	A subsoil drain may be required behind a wall if there is evidence of water seepage; Herringbone surface drains may be required if the slope drainage is impeded.	Bio-engineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation
Failures in hill slope but not cut slope	Reduce the slope grade, and if this is feasible, then add protection. A retaining wall to support the sliding mass, as long as foundations can be found that do not surcharge or threaten the cut slope.	A subsoil drain may be required behind a wall if there is evidence of water seepage; Herringbone surface drains may be required if the slope drainage is impeded.	Bio-engineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation
Deep failure in the original ground beneath the road	Consider re-alignment of road away from instability. If slow moving, short term option may be to repave or gravel the road.	Ensure road-side drainage is controlled.	Bio-engineering will not be effective.

Table 10.7 Stabilisation Options for Below the Road Problems

Instability	Stabilisation options	Drainage options	Protection options
Erosion of the fill slope surface	Bioengineering and drainage (Section 10.7)	Ensure road-side drainage is controlled	Bio-engineering a key option
Failures in fill slope	Re-grade or remove, replace and compact fill; Before replacing fill, cut steps in original ground to act as key between fill and original ground; A new road retaining. wall may be the only option	Ensure road-side drainage is controlled	Bio-engineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation.
Failure in fill slope and original valley slope	Re-grade or remove, replace and compact fill; Before replacing fill, cut steps in original ground to act as key between fill and original ground; A new road retaining wall may be the only option.	Ensure road-side drainage is controlled	Bio-engineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation
Failure in original slope	Re-grade if sufficient space between road and valley side; A new road retaining wall may be the only option.	Ensure road-side drainage is controlled	Bio-engineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation
Removal of support from below by river erosion	May need extensive river training works to prevent further erosion.	None	Slope protection (walls and rip-rap etc.) may be necessary – possible with additional bioengineering options

10.6.4 Earthwork drainage

Slope stability is greatly influenced by the movement of water; either by the erosive impacts of surface water or the changes in pore pressure resulting from rainfall infiltration and concentration within the slope mass. Water may decrease pore suction in the underlying soil and increase pore water pressure, thereby reducing the effective stress and the stability of the slope. Hence, the construction of surface and sub-surface drainage structures is vital to ensure that excess water can be intercepted and conveyed to a safe location where it will not create further instability problems. (TRL, 1997, GEO, Hong Kong 2000; DoR Nepal, 2007, see also Chapter 9)

Principal earthwork drainage options are summarised below,

Cut-off drains are used to reduce surface runoff at the crest of a cut slope or slope failure. In order to reduce the likelihood of continuing slope movements breaching the drain, they are sometimes located many tens of metres above the failure crest. The potential problem with cut-off drains is that unless they are regularly maintained, they can create their own instability problem. It is not recommended that cut-off drains be constructed unless regular maintenance can be assured.

Herringbone (or chevron) drains are constructed herringbone fashion on slope faces to collect surface seepages and surface runoff. They are often quite shallow (about 1m deep), but can be much deeper. Care needs to be taken to ensure that the construction of the drain does not lead to further instability, and to ensure that the drain can still function in the event of minor downslope movements.

Counterfort drains are used to depress a high water-table. These drains are constructed at right angles to the toe of the slope and are often dug to a depth of 3 metres or more at intervals of 3-10 metres depending on the permeability of the subsoil.

Horizontal drains are used to intercept groundwater and seepage at depth. They require the use of plastic pipes and specialist drilling equipment that may not always be available, and they are not easy to install.

Lined channels or cascades are likely to be necessary if a watercourse or gully is a direct cause of the instability in the first place. A lined channel may be necessary to divert an existing watercourse from the failed area, or to train the watercourse within defined limits. The lining itself may be impermeable (mortared masonry and/or concrete) or permeable (gabion). The structure may comprise cascades, chutes and check dams. As a general rule, gabion structures are preferred since they are flexible and allow water ingress provided they are located below the wet season groundwater table.

Check dams are necessary where undue scour would otherwise occur from the stream flow of water.

10.6.5 Retaining and Revetment walls

Retaining walls must be designed to withstand the pressure exerted by the retained material attempting to move forward down the slope due to gravity. The lateral earth pressure behind the wall depends on the angle of internal friction and the cohesive strength of the retained material. Lateral earth pressures are smallest at the top of the wall and increase towards the bottom. The total pressure may be assumed to be acting through the centroid of a triangular load distribution pattern, one-third above the base of the wall. The wall must also withstand pressure due to material 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 considered in the design. Dissipation of ground water is normally achieved by constructing horizontal drains behind the wall with weepholes.

Figure 10.7 Typical LVRR Retaining Structures (DoR, Nepal, 207)

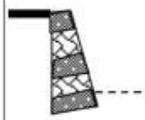
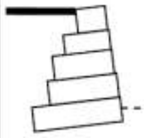
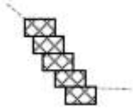
Types	RCC Crib	Dry Masonry	Banded Masonry	Cement Masonry	Gabion Masonry	Reinforced Earth
Schematic						
Top width (m)	1.2	0.6–1.0	0.6–1.0	0.5–1.0	1.0	4.0 or 0.7–0.8H
Base width	0.4–0.6H	0.5-0.7H	0.6–0.65H	0.5–0.65H	0.6–0.75H	4.0 or 0.7–0.8H
Front batter (V:H)	4:1	3:1	varies	10:1	6:1–4:1	3:1
Back batter (V:H)	4:1	vertical	vertical	varies	varies	3:1
Foundation dip (V:H)	1:4	1:3	1:3	1:10–1:6	1:6–1:4	horizontal
Foundation depth (m)	0.5–1.0	0.5	0.5–1.0	0.5–1.0	0.5	0.5
Height range (m)	4.0–12.0	1.0–4.0	4.0–8.0	1.0–10.0	1.0–6.0	3.0–12.0
Fill slope angle (°)	< 30°	< 30°	< 20°	35°-60°	35°-60°	< 35°

Figure 10.8 Typical LVRR Revetment Structures (2) (DoR, Nepal, 207)

Types	Dry Masonry	Banded Masonry	Cement Masonry	Gabion	Drum Wall	Concrete
Schematic						
Top width (m)	0.5	0.5	0.5	2.0	1.0	0.5
Base width	0.3-0.35H	0.23H	0.25H	2.0	1.0	0.25H
Front batter (V:H)	varies	varies	varies	varies	varies	varies
Back batter (V:H)	3:1-5:1	3:1	3:1	3:1-5:1	3:1	3:1
Foundation dip (V:H)	1:3-1:5	1:3	1:3	1:5	1:3	1:3
Foundation depth (m)	0.5	0.5	0.5	0.5–1.0	0.25	0.5
Height range (m)	3.0–6.0	3.0–6.0	1.0–8.0	1.0–6.0	1.0–2.2	1.0–12.0
Hill slope angle (°)	35°-60°	35°-60°	35°-70°	35°-60°	15°-35°	35°-70°

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 construction work. They are useful as revetments for erosion protection and as a means of supporting soil against very shallow movement. Dry stone walls should not exceed 5-6m in height.

As with 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.

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

Gabion walls are cost effective because they employ mainly locally available rock and local labour. Gabion structures are commonly used for walls of up to 6m high, but are used in appropriate locations up to 10m. 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 may not be preferred as retaining walls immediately below and adjacent to rigid pavement roads due to the possibility of flexible movement of 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.

10.7 Bio-engineering

10.7.1 General

Bio-engineering can be broadly defined as the use of vegetation, either alone or in conjunction with engineering structures, and non-living plant material, to reduce erosion and shallow-seated instability on slopes. Table 10.8 contains key primary references on which this section is based and are which are accessible for detailed guidance derived from recent research and practical application.

Table 10.8 Primary Bioengineering References

Reference	Description
TRL 1997. 'Principles of low-cost road engineering in mountainous regions, with special reference to the Nepal Himalaya', Overseas Road Note 16, Crowthorne: Transport.	Contain basic outline designs on bioengineering and associated "hard" engineering options.
Howell, J. 1999. Roadside Bio-engineering: Site Handbook and Reference Manual. Department of Roads, Kathmandu	A comprehensive study of bio-engineering options in mountainous terrain Nepal
SEACAP, 2008. Scott-Wilson for DFID and Ministry of Public Works, Laos	A collection of manuals, documents and training materials based on a DFID-funded South East Asian Community Access Programme (SEACAP)
ADB, 2017. A series of documents prepared by ICEM ADB and for the Ministry of Agriculture and Rural Development (Vietnam)	These include: Detailed design of bio-engineering options; guidance on their use; relative costs outline BoQ and a Final Report

In bio-engineering applications there is an element of slope stabilisation as well as slope protection in which the principal advantages are:

- Vegetation cover protects the soil against rain splash and erosion, and prevents the movement of soil particles down slope under the action of gravity;
- Vegetation increases the soil infiltration capacity, helping to reduce the volume of runoff;.
- Plant roots bind the soil and can increase resistance to failure, especially in the case of loose, disturbed soils and fills;
- Plants transpire considerable quantities of water, reducing soil moisture and increasing soil suction;
- The root cylinder of trees holds up the slope above through buttressing and arching;
- Tap roots or near vertical roots penetrate into the firmer stratum below and pin down the overlying materials;
- Surface run-off is slowed by stems and grass leaves.

In summary, vegetation is important in the control of erosion and shallow forms of instability (1-3m depth at most). It is also important to appreciate that the beneficial effects may be insignificant under extreme conditions of rainfall or drought.

10.7.2 Key factors

The main factors to be addressed when selecting the particular species for use in bio-engineering works are as summarized as:

1. 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.

2. The plant must be capable of growing in the location of the site.

3. There is no single species or technique that can resolve all slope protection problems.

4. 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.

5. 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.

6. 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.

7. Shrubs (i.e. woody plants with multiple stems) can often grow from cuttings taken from their branches; plants propagated by this method tend to produce a mass of fine, strong roots. These are often better for soil reinforcement than the natural rooting systems developed from a seedling of the same plant.

8. 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.

9. Plant roots cannot be expected to contribute to soil reinforcement below a depth of 500mm.

10. Plants cannot be expected to reduce soil moisture significantly at critical periods of intense and prolonged rainfall.

11. 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).

10.7.3 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.

10.7.4 Recommended techniques

Table 10.9 (ADB, 2017) provides the different types of bio-engineering techniques recommended for various kinds of slopes and soil materials for both cut and fill situations. Particular plants species must be selected that are compatible with the specific Myanmar environment.

The combination of "hard" low-cost engineering solutions and "soft" bioengineering options has been recognised as an effective and sustainable means of slope protection and shallow failure stabilisation, (TRL 1997), Figure 10.9.

Figure 10.9 Typical Combinations of Geotechnical and Bio-engineering Options (ADB, 2017)

a. Combined shrub line, fascine and grass protection combined with stone and gabion drainage.

b. Combined rip-rap and brush vegetation protection of roadside river embankment.

Table 10.9 Recommended general bio-engineering procedures

Slope Angle	Slope Length	Material Drainage	Moisture Condition	Existing/Potential Problems	Mitigation Action	Primary Techniques - Protection	Secondary Techniques - Resilience
$>40^0$	>15m	Good	Damp	Erosion, slumping	Armour, reinforce, drain	Diagonal lines of grass planting	Diagonal lines of grass planting
			Dry	Erosion	Armour, reinforce	Contour lines of grass planting	Contour lines of grass planting
		Poor	Damp	Surface slumping, erosion	Drain, armour, reinforce	1. Downslope grasslines & vegetated stone pitch rills or, 2. Chevron grass lines and vegetated stone pitched rills.	None available – consider engineering options with good drainage.
			Dry	Erosion, surface slumping	Armour, reinforce, drain	Diagonal lines of grass planting	Diagonal palisades at wide spacing
	<15m	Good	Any	Erosion	Armour, reinforce	1.Diagonal lines of grass planting, or 2.Bamboo mesh and planted grass	Diagonal palisades at wide spacing
		Poor	Damp	Surface slumping, erosion	Drain, armour, reinforce	1.Downslope lines of large grass planting, or 2. Diagonal lines of large or short grass planting	None available – consider hard engineering options with good drainage.
			Dry	Erosion and surface slumping	Armour, reinforce, drain	1.Bamboo mesh and short grass, or 2.Contour lines of large grass planting or 3.Diagonal lines of large grass planting	Contour lines
30^0-45^0	>15m	Good	Any	Erosion	Armour, reinforce, catch	1.Downslope grass lines & vegetated stone pitch rills, or 2.Site planted grass, mulch and jute or bamboo mesh	Live poles or truncheons
		Poor	Any	Surface slumping, erosion	Drain, armour, reinforce	Site-specific drainage and shrub/tree planting	Nothing further required
	<15m	Good	Any	Erosion	Armour, reinforce, catch	1.Brush layers of wood cuttings, or 2.Contour lines of grass planting, or 3.Contour facines, or 4.Palisades of wood cuttings, or 5.Large grass planting and jut or bamboo mesh	Short grass planting between brush layers and facines. Live poles between grass lines.
		Poor	Any	Surface slumping, erosion	Drain, armour, reinforce	1.Diagonal large grass lines, or 2.Diagonal brush layers, or 3.Site-specific drainage and shrub/tree planting	Shrub or tree planting
$<30^0$	Any	Good	Any	Erosion	Armour, catch	Contour lines of large grass planting	Shrub or tree planting Live poles or truncheons
		Poor	Any	Surface slumping, erosion	Drain, armour, catch	Diagonal lines of large grass planting	Shrub or tree planting Live poles or truncheons
Any Loose sand		Good	Any	Erosion	Armour	Bamboo mesh and planted grass	Live poles, truncheons
Any Laterite		Poor	Any	Erosion, surface slumping	Armour, drain	Diagonal lines of grass and shrub/tree planting	Nothing else
Any gully $<45^0$				Erosion	Armour, reinforce, drain	1. Live check dams or 2. vegetated stone pitching	Nothing else

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11 Design Process

11.1 Introduction

Previous chapters in this manual have presented guidance on the detail of design processes and procedures; this chapter aims to place these actions in the context of the Project Cycle. These actions are described in a series of diagrams and check lists that provide a clear pathway for road engineers at central, state, region and township to follow. The flowcharts and check lists are linked to specific sections in the previous chapters.

11.2 The Project Cycle for LVRRs

Most of the features of a LVRR project can be related to the need to take a series of important decisions and consequent actions during the life of the project. These decisions must be taken in a particular sequence. The result of each decision has a major influence on the form of the next part of the project preparation process.

The increasing cost of each successive stage of the process means that it is very important to review the process at each stage and to make a clear decision whether the next stage of the process is justified. Major decisions on whether, or not, to continue with project preparation is made at the end of each stage. As the cost of each successive stage is many times greater than that of the previous stage, the importance of the decision and the amount of information needed to make it, increases at each stage.

11.3 The Pre-feasibility or Planning Stage

In general terms, this is the stage at which the overall project and its strategic objects are determined, potential budgets are defined, and strategic financial and broad engineering risks are identified. This process takes into account government policies and programmes that impact on road development which is, therefore, examined in a very wide socio-economic and policy-orientated context. There will normally be an initial assessment of the project against previously defined criteria. Projects that do not meet selection criteria are screened out or modified.

This is the stage when the general road tasks to be met are defined. The identification of whether, or not, the likely road falls within the LVRR envelope is crucial decision at this stage.

Design, construction and maintenance decisions will impact cross-sectorially and on local communities and other sector activities, e.g. agriculture, water, health, and education, as well as commercial activities such as local transporters, suppliers and traders. Consideration of these impacts and consultation with other ministries and stakeholders at this early stage will help mobilise support and maximise the beneficial impacts of the road works.

Figure 11.1 presents a general flow chart for this project phase, Figure 11.2 provides a check list of key actions and Table 11.1 provides cross referencing to the other sections within this Manual.

Figure 11.1 General Pre-Feasibility Flow Chart (Cook et al, 2013)

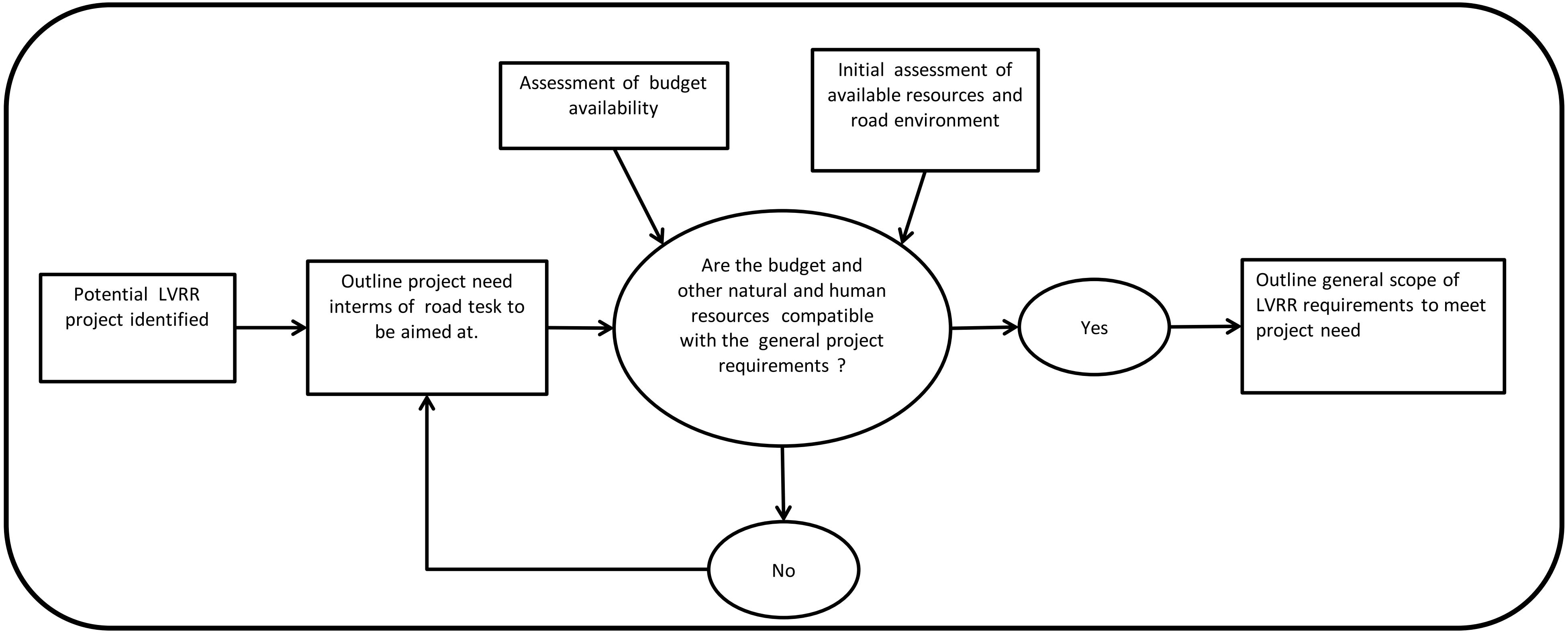

Figure 11.2 Check List of Pre-Feasibility Actions and Outputs

Input to Prefeasibility Stage

Project Location defined with possible alternative alignments
Project aim defined. New alignment/Upgrade/rehabilitation/spot improvement
Indicative outline **budget**

Prefeasibility Stage Actions

Initial desk study for key documents and road history
Road task defined along with **classification and general standards**
Initial visits to alignment
Climate, general character and probable level of **resilience**
Identify requirement for major **bridges**

Prefeasibility Stage Outputs

Pre-feasibility Report, highlighting key issues and any potential problems
Location defined with possible alternative alignments
Road task defined along with **classification and general standards**
Project aim defined, upgrade/rehabilitation/spot improvement
Identify potential large scale natural **hazards or geotechnical/materials issues**
Initial desk study for key **documents** and **road history**
Climate: general character and probable level of **resilience**
Probable **pavement styles-** unsealed/sealed/spot improvement
Identify major **bridge sites**
Initial **economic assessment**
Establish liaison with **local communities**
Initial listing of **Complementary Interventions**

Prefeasibility Stage Decisions

Project should proceed to Feasibility Study
Project should proceed Feasibility Study with strategic modifications
Project should not proceed

Table 11.1 Key References in Manual

Pre-feasibility Action	Reference
Initial desk study	Section 5.3.1, Table 5.2
Road task, classification and general standards	Identify the relevant standards and specifications Section 2.4.2
Initial visits to alignment	Drive over; Section 5.3.2
Climate character and level of resilience to be designed for	Regional climate pattern and identification of data sources Section 4.2
Identify requirement for major bridges	General requirements, Section 9.2

11.4 The Feasibility Stage

In the context of the overall project this is the stage where a more detailed economic and engineering assessment is made and the main engineering problems and any other issues affecting the route are identified.

In the context of this manual the feasibility stage of a rural road project assumes that a need has been identified and that the solution to the need falls within the LVRR envelope. In general terms, the Feasibility Stage assesses paving, earthwork, drainage and bridge options and identifies those most likely to provide a sustainable solution within the governing road environment and within the expected budgets. This is generally seen as a critical stage by road authorities and external funders and donors such as the World Bank, ADB or JICA. Relevant Ministry or DRRD planners and Consultants are normally closely involved at this stage.

As part of the feasibility study it is important to identify and investigate the major technical, environmental, financial, economic and social constraints in order to obtain a broad appreciation of the viability of the competing options. For low volume roads, one of the most important aspects of the feasibility study is communication with the people who will be affected by the road.

An assessment of available resources is generally required both to confirm the feasibility of a proposed LVRR and to identify sustainable and appropriate strategic design options within a sustainable framework.

As noted in Chapter 8, the appropriate use of locally available materials is a key issue in cost-effective LVRR design and construction. Information on the performance of materials from sources which have been used for existing roads would be very useful at this stage. Particularly important would be knowledge of any potential problems with existing sources. If there are no existing materials sources then more detailed materials exploration investigations need to be initiated

At feasibility stage, sufficient data is required to identify the most suitable options appropriate to the particular road requirements. Data are generally required that are sufficient to obtain likely costs to an accuracy (of at least) ±25%.

Figure 11.3 presents general flow chart for this project phase, Figure 11.4 summaries check list of key actions and Table 11.2 provides comment and cross referencing to the other sections within this Manual.

Figure 11.3 General Feasibility Flow Chart (Cooke et al, 2013)

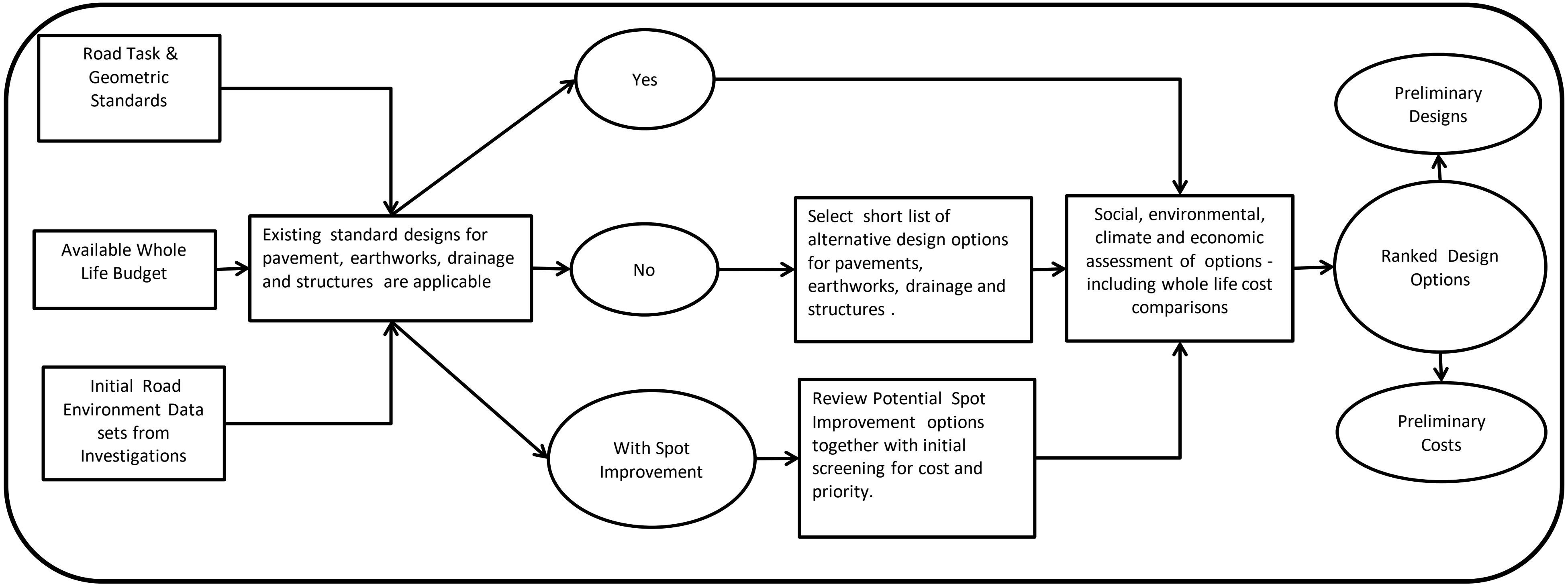

Figure 11.4 Check List of Feasibility Actions and Outputs

Input to Feasibility Stage

A **decision to proceed** with a Feasibility Study for a road project that has been defined in terms of corridor location and task (or purpose).

Feasibility Stage Actions

Desk study to review available information relevant to preliminary design
Preliminary ground a and materials investigations
Walkover survey of alignment to identify general character, areas of potential hazard and climate vulnerability
Define the key aspects of the **Road Environment**
Identify locations for **stream and river crossing**
Undertake an assessment of **traffic** in terms of vehicle numbers and type
Detailed **economic assessment**

Feasibility Stage Outputs

Feasibility Report, preliminary **design and costs and ecomomic assessment**
Information sources identified and listed
Geotechnical and climate resilience hazards identified
Potential **materials resources** identified
Environmental issues identified
Potential **alignment** issues defined
Standards to be used defined
Pavement options identified
Bridge sites defined
Liaison with **local communities on safeguard issues**
Complimentary Interventions defined and costed

Feasibility Stage Decisions

Project should proceed to Final Design Stage
Project should proceed Final Design Stage only after key amendments
Project should not proceed
Funding sources defined

Table 11.2 Key References in Manual

Feasibility Action	Reference
Desk study to review available information relevant to preliminary design	Section 5.3.1, Table 5.2
Preliminary ground a and materials investigations	General Table 5.4; Materials, Section 8.3; details Annex IV
Walkover survey of alignment to identify general character, areas of potential hazard and climate vulnerability	Walkover, Section, Table 5.3 and associated forms Annex IV; climate Section 5.3.5,
Define the key aspects of the Road Environment Identify locations for stream and river crossing	Road environment impacts in general; Chapter 4. Structures 5.3.6, Section 9.2, Table 9.5
Undertake an assessment of traffic in terms of vehicle numbers and type	Initial survey, Section 5.3.3 and detailed procedures and forms in Annex I

11.5 The Final Engineering Design (FED) Stage

The FED stage requires sufficient data for preparation of the contract documents including technical specifications and Bills of Quantities. Final detailed cost estimation is also likely to be required. The FED stage requires more investigation and considerably more data (than the previous stages, as explained above). The entire process of project design should now be completed with sufficient accuracy to minimise the risk of changes being required after the works contract has been awarded.

For the particular case of the pavement and surfacing elements, the FED stage incorporates the Phase I (Options Selection) outcomes into the Phase II detailed designed procedures. This will include the design and specification of the pavement structural layers and any overlying surfacings together with associated shoulders and pavement drainage.

Feasibility assumptions on traffic patterns should be cross-checked and, if required, additional surveys undertaken aimed specifically at obtaining data for each vehicle category and axle loading for the pavement layer design. Risks of likely axle-overloading should be pragmatically assessed.

Sources of material should now be defined in terms of location, quality and quantity such that it is clearly established that the road or roads can be built to the required specification with the available materials. Source, haulage, processing and placement costs need to be investigated and any inflation factors considered.

Climatic patterns and the incidence of severe climatic events should be confirmed. The levels of Climate Resilience that may be required should be defined

Data are generally required that are sufficient to obtain likely costs to an accuracy of better than about ±10%.

Figure 11.5 presents a typical FED flow chart, in this case using pavement design as an example, Figure 11.6 summaries check list of general key actions and Table 11.3 provides comment and cross referencing to the other sections within this Manual.

Figure 11.5 Pavement FED General Flow Chart

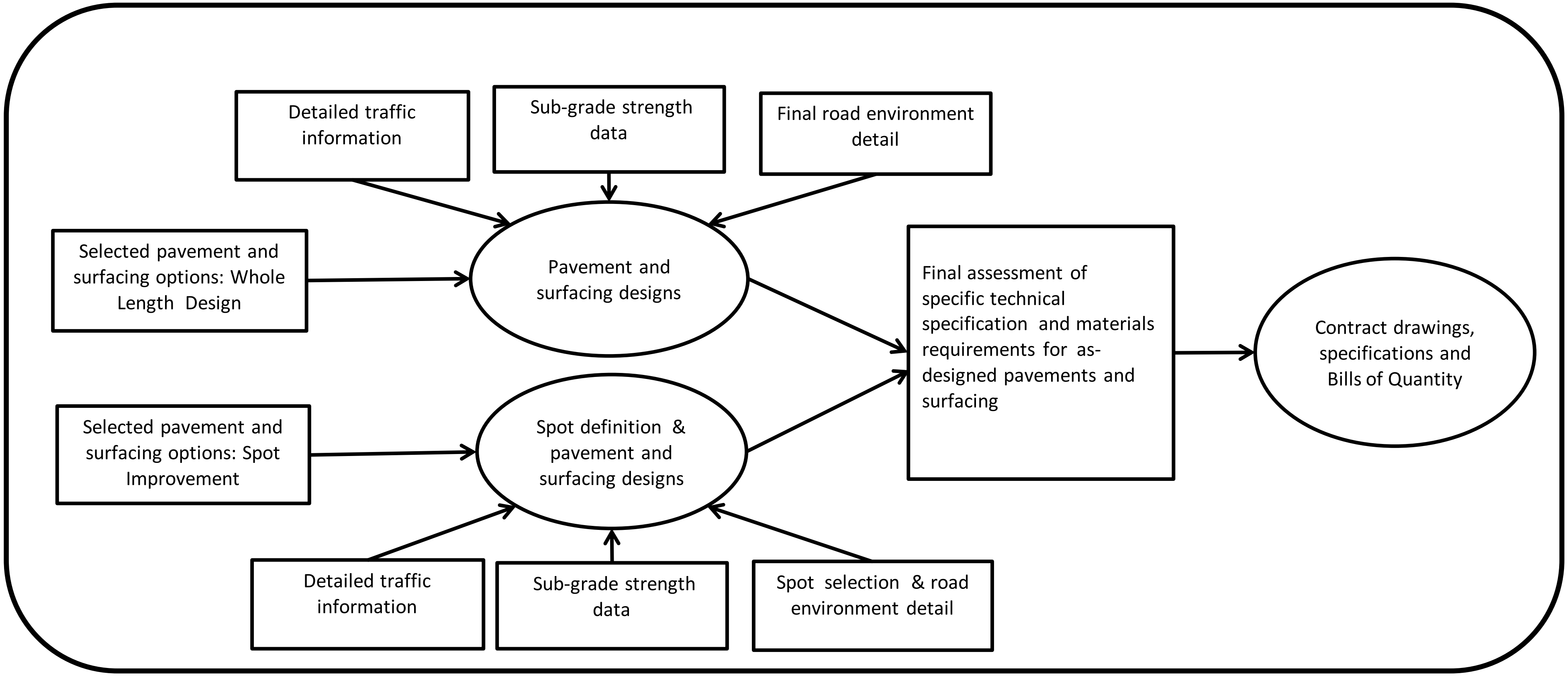

Figure 11.6 Check List of FED Actions and Outputs

Input to Final Engineering Design

Alignment defined
Funding sources agreed
Preliminary **design and BoQ**

Final Design Stage Actions

Undertake detailed **ground investigations**
Undertake detailed **materials investigations**
Final Engineering Designs **(FED) Pavement**
Final Engineering Designs **(FED) Earthworks**
Final Engineering Designs **(FED) Drainage**
Final Engineering Designs **(FED) Structures**
Final **climate resilience** applications
Agree **procurement arrangements** and draft documents
Finalise **quantities and costs**

Final Design Stage Outputs

FED and Report, including the following:
Technical **specifications**
Detailed **road designs** and drawings including cross and longitudinal sections
Materials usage plan (haul diagrams)
Drawingsl for **earthworks protection** or stabilisation
Drawings for **climate resilience** works
BoQ and **Engineers Estimate cost**
Environmental safeguard plans
Technical Audit procedure agreed
Supervision guidelines
Complimentray Interventions.

FED Stage Decisions

Project should proceed to procurement
Assessment of procurement

Table 11.3 Key References in Manual

Pre-feasibility Action	Reference
Undertake detailed ground investigations	Chapter 5 with detail in Annex IV. Tables
Undertake detailed materials investigations	Chapter 8, laboratory testing Annex II; marginal materials Annex III
Final Engineering Designs (FED) Pavement	Unpaved Section 7.7, sealed Section 7.8 and 7.9, non-bituminous paved Section 7.10:
Final Engineering Designs (FED) Earthworks	Sections 10.2, 10.3, and 10.4, Tables 10.1 and 10.2
Final Engineering Designs (FED) Drainage	Earthwork; section 10.5.4;
Final Engineering Designs (FED) Structures	Sections 9.2,5, 9.2.6, 9.2.7 and 9.3 in general.
Final climate resilience applications	Sections 9.4; 10.6
Agree procurement arrangements and draft documents	Section 12.3. Contractors capable of undertaking the works should be identified and any required training programmes for local contractors or labour-based organisations must be defined

References

Cook J R, Petts R C, Rolt J. 2013. Low Volume Rural Road Surfacing and Pavements: A Guide to Good Practice. Research Report for AfCAP and DFID