REPUBLIC OF LIBERIA

MINISTRY OF PUBLIC WORKS

MANUAL FOR LOW VOLUME ROADS

PART C - HYDROLOGY, DRAINAGE DESIGN AND ROADSIDE SLOPE STABILISATION

2019

PART C

HYDROLOGY, DRAINAGE AND ROADSIDE STABILIZATION

Manual for Low Volume Roads

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

PREFACE

The affordable provision of low volume roads depends on appropriate design of drainage structures and ensuring the stability of the road. Access for communities in rural areas can be lost if a drainage structure fails during the rains or a land slide breaches the road. Liberia experiences high rainfall, particularly in the coastal areas, and rainfall is expected to increase in intensity due to climate change. Drainage structures will therefore become more vulnerable to damage during the rains and will require more careful design and higher standards of construction.

The Manual for Low Volume Roads provides detailed guidance on the design, construction and maintenance of rural roads. It caters for roads carrying less than about 300 vehicles per day or one million equivalent standard axles over the design life. Part C of the manual addresses drainage-related aspects of the design of LVRs. These include the calculation of flows in watercourses, discharge rates, the location and design of appropriate drainage structures, associated roadside stabilization measures, and provision for the anticipated effects of climate change. Tailored specifically for conditions in Liberia, it includes worked examples to help the reader understand the detailed application of the methods and practices described.

Manual Updates

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

All suggestions to improve the manual should be made in writing to the Ministry of Public Works, which is responsible for issuing periodic updated versions of the Manual.

Honorable Minister Mobutu Vlah Nyenpan

Ministry of Public Works

Monrovia

ABBREVIATIONS, ACRONYMS AND INITIALISMS

>	:	Greater than
<	:	Less than
%	:	Percentage
AADT	:	Annual Average Daily Traffic
AASHTO	:	American Association of State Highway and Transportation Officials
AC	:	Asphaltic Concrete
AfCAP	:	Africa Community Access Partnership
ALD	:	Average Least Dimension
ARRB	:	ARRB Group, formerly the Australian Road Research Board
ASTER	:	Advanced Spaceborne Thermal Emission and Reflection Radiometer
ASTM	:	American Society for Testing and Materials
BC	:	Binder Course
CBR	:	California Bearing Ratio
CERGIS	:	Centre for Remote Sensing & Geographic Information Services
CS	:	Cobblestone
CSIR	:	Council for Scientific and Industrial Research
Cu	:	Shear Strength (undrained)
DBM	:	Dry Bound Macadam (Dense Bitumen Macadam)
DCP	:	Dynamic Cone Penetrometer
DF	:	Drainage Factor
DFID	:	UK Government's Department for International Development
DN	:	DCP penetration rate in mm/blow
DS	:	Dressed Stone
DV	:	Design Vehicles
EF	:	Equivalence Factor
e.g.	:	For example (abbreviation for the Latin phrase exempli gratia)
EIA	:	Environmental Impact Assessment
EMP	:	Environmental Management Plan
ENS	:	Engineered Natural Surfaces
EOD	:	Environmentally Optimized Design
ERA	:	Ethiopian Roads Authority
esa	:	Equivalent Standard Axles
ESIA	:	Environmental and Social Impact Assessment
FACT	:	Fine Aggregate Crushing Test
GM	:	Grading Modulus
GWC	:	Gravel Wearing Course
GVW	:	Gross Vehicle Weight
HDM 4	:	World Bank's Highway Development and Management Model Version 4

HPS	:	Hand Packed Stone
HVR	:	High Volume Road
i.e.	:	That is (abbreviation for the Latin phrase id est)
ILO	:	International Labor Organization
IMT	:	Intermediate Means of Transport
InSAR	:	Interferometric Synthetic Aperture Radar
IRI	:	International Roughness Index
IRR	:	Internal Rate of Return
kip	:	kilopound. 1 kip = 4.448 kN
km	:	Kilometer
km/h	:	Kilometers per Hour
lat	:	Latitude
LBM	:	Labor-Based Methods
LCC	:	Life Cycle Cost
LEF	:	Load Equivalency Factor
long	:	Longitude
LVR	:	Low Volume Road
LVSR	:	Low Volume Sealed Road
m	:	Meter
m^3	:	Cubic Meters
M	:	Million
MAP	:	Mean Annual Precipitation
MCB	:	Mortared Clay Brick (fired)
MCS	:	Mortared Cobblestones
MDS	:	Mortared Dress Stone
Mesa	:	Million Equivalent Standard Axles
mm	:	Millimeter
mod	:	Modified
MPa	:	Megapascal (a unit of pressure equal to 1000 kilopascals (kPa)
MS	:	Mortared Stone
MSSP	:	Mortared Stone Setts or Pavé
NBP	:	Non-Bituminous Pavement
NMT	:	Non-Motorized Transport
NRC	:	Non-reinforced Concrete
NRCP	:	Non-reinforced Concrete Pavement
OMC	:	Optimum Moisture Content
ORN	:	Overseas Road Note
Pen.	:	Penetration
PI	:	Plasticity Index
PM	:	Plasticity Modulus
PSD	:	Particle Size Distribution

RC	:	Reinforced Concrete
RED	:	Road Economic Decision Model
ReCAP	:	Research for Community Access Partnership
Ref	:	Reference
RI	:	Roughton International
SADC	:	Southern Africa Development Community
SAR	:	Synthetic Aperture Radar
SBL	:	Sand Bedding Layer
SE	:	Super Elevation
SID	:	Spot Improvement Design
SME	:	Small and Medium scale Enterprise
SSP	:	Stone Setts or Pavé
ToR	:	Terms of Reference
TRL	:	Transport Research Laboratory
TQMS	:	Total Quality Management System
UK	:	United Kingdom
USA	:	United States of America
USCS	:	Unified Soil Classification System
USD	:	United States Dollar
USGS	:	United States Geological Survey
UTRCP	:	Ultra-thin Reinforced Concrete Pavement
VEF	:	Vehicle Equivalency Factor
vpd	:	Vehicles per Day
VOCs	:	Vehicle Operating Costs
WBM	:	Water Bound Macadam
Yr	:	Year

UNITS OF MEASUREMENT

The units of measurement used in this manual are based on International System (SI) units, with some exceptions. The basic units of measurement are summarized below.

Item	Unit	Symbol	Common Multiples
Length	meter	m	1000 meter (m) = 1 kilometer (km) 1 meter (m) = 100 centimeters (cm) 1 meter (m) = 1000 millimeters (mm)
Mass	kilogram	kg	1000kg = 1 tonne (ton) 1kg = 1000 grams (g)
Area	square meter	m^2	10,000 m^2 = 1 hectare (ha)
Volume (Solids)	cubic meter	m^3	1 m^3 = 10,000 cm^3
Volume (liquids)	liter	l	1000 liters (l) = 1 m^3 1 liter (l) = 1000 milliliter (ml)
Density	kilogram per cubic meter	kg/m^3	1000 kg/m^3 = 1 ton/m^3 1kg/m^3 = 1000g/cm^3
Force	Newton	N	1000N = 1 kilonewton (kN) 1N = 1 kgm/s^2
Pressure and stress	Newton per square meter	N/m^2	1 kN/m^2 = 1000N/m^2
Speed	meter per second kilometer per hour	m/s km/h	
Temperature	degree Celsius	°C	

SI units may be converted to the Imperial System using the following factors.

Item	Measure and Unit		Conversion Factor and Unit	
Length	1	meter (m)	3.2808	Feet (ft)
	1	Meter (m)	39.3701	Inches (In)
	1	Meter (m)	1.0936	Yard (Y)
	1	Kilometer (km)	0.6214	Miles
Mass	1	Kilogram (kg)	2.2050	Pounds
Area	1	m^2	10.7639	Square ft
	1	m^2	1.1960	Square Yard
	1	Hectare (ha)	2.4711	Acres
Volume (Solids)	1	m^3	35.3147	Cubic feet
	1	m^3	1.3080	Cubic yards
Volume (liquids)	1	Liter (l)	0.2200	Gallons (UK)
	1	Liter (l)	0.2642	Gallons (USA)
Density	1	kg/m^3	0.0624	$Pounds/ft^3$
Force	1	Newton (N)	0.2248	Pounds - force
Speed	1	m/s	3.2808	ft/s
	1	km/h	0.6214	Miles/h

Source: LSFRP Design Manual, 2016.

GLOSSARY OF TECHNICAL TERMS

Aggregate (for construction)

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

Asphalt

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

Atterberg Limits

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

Base Course

The upper layer of the road pavement.

Binder, Bituminous

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

Binder, Modified

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

Bitumen

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

Bitumen, Cutback

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

Bitumen, Penetration Grade

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

Bitumen Emulsion

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

Bitumen Emulsion, Anionic

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

Bitumen Emulsion, Cationic

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

Bitumen Emulsion Grades

- Premix grade: An emulsion formulated to be more stable than spray grade emulsion and suitable for mixing with medium or coarse graded aggregate with the amount smaller than 0.075 mm not exceeding 2%.

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

Cape Seal

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

Cement (for construction)

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

Chip Seal, Single

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

Chip Seal, Double

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

Collapsible Soil

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

Complementary Interventions

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

Concrete

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

Concrete Block Paving

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

Crown Height

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

Crushed Stone

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

Design Speed

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

Dispersive Soil

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

Distributor

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

Expansive Soil

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

Filler

Mineral matter composed of particles smaller than 0.075 mm.

Fog Spray

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

Gravel

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

Labor-Based Construction

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

Level of Service

A qualitative measure used to categorize traffic flow based on performance indicators such as vehicle speed, vehicle density, congestion, etc.

Low Volume Road

Roads carrying up to about 300 vehicles per day (with four or more wheels) and less than 1 million equivalent standard axles over their design life.

Otta Seal

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

Paved Road

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

Prime Coat

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

Ravelling

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

Reseal

A surface treatment applied to an existing bituminous surface.

Rejuvenator

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

Seal

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

Selected Layer

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

Site Investigation

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

Slurry (Slurry Seal)

A mix of suitably graded fine aggregate, cement or hydrated lime, bitumen emulsion and water, used for filling the voids in the final layer of stone of a new surface treatment or a maintenance treatment.

Slurry-Bound Macadam

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

Standard
A standard is a set of norms described as design guidelines or specifications.

Sub-base
The layer in the road pavement below the base course.

Subgrade
The native material underneath a constructed road pavement.

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

Surfacing
The layer with which traffic makes direct contact.

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

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

Wearing Course
The upper layer of a road pavement which is expected to wear under the action of traffic.

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

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1. INTRODUCTION

1.1 General

It is often said that the three most important aspects of road design are drainage, drainage, and drainage! Water is often the cause, whether directly or indirectly, of roadway destruction or pavement failure. It is therefore necessary to give careful consideration to proper design of drainage structures for pavements and other road facilities.

The drainage associated with any road can be divided into two broad categories: the drainage of the catchment area traversed by the road, and the drainage of the road reserve itself. It is essential that adequate provision be made throughout the road to efficiently collect and discharge rainwater falling onto the area of the road reserve. Rainwater should be discharged as frequently as possible away from the road to minimize erosion damage to the road, the drainage system and to the adjacent land.

Hydrological data and other information are required for the hydraulic analysis and design of drainage structures and for sizing up the different components of these structures.

1.2 Purpose of the Manual

The Manual for Low Volume Roads (LVRs) promotes the rational, appropriate, and affordable provision and maintenance of LVRs in Liberia. In doing so it aims to make cost effective and sustainable use of local resources. The Manual reflects local experience and advances in LVR technology gained in Liberia and elsewhere. It is fully adaptable for different clients and users and has application for roads at a national and district level administered by the Ministry of Public Works and local authorities. The Manual caters for interventions that deal with individual critical areas on a road link (spot improvements) through to providing complete designs for new rural roads.

The Manual is intended for use by roads practitioners responsible for the design, construction and maintenance of low traffic earth, gravel or paved roads. It is appropriate for roads which are required to carry up to 300 vehicles per day (with four or more wheels) and less than 1.0 million equivalent standard axles (mesa) per traffic lane over their design life. It is divided into the following Parts:

- Part A: Policy, Geometric Design and Road Safety;
- Part B: Materials, Pavement Design and Construction;
- Part C: Hydrology, Drainage Design and Roadside Slope Stabilization;
- Part D: Surfacings for Low Volume Roads; and
- Part E: Maintenance.

Part C of the Manual for Low Volume Roads provides the design standards and guidelines for hydrological and hydraulic design of drainage structures. The standards and guidelines are intended for use by Drainage Design Engineers in the Ministry of Public Works, county councils and private sector consultants.

1.3 Scope

The design standards and guidelines cover extensively the steps required to design storm drainage structures to minimize or eliminate flooding and erosion of rural LVRs and adjoining areas. The purpose is to ensure that rural roads are motorable all year round by providing adequate drainage structures across and along the road corridor to keep the road surface free of surface runoff after heavy downpours and to safety discharge the runoff. In cases where submersible structures are provided on very lightly trafficked roads, interruptions to access should be limited to a matter of hours, rather than days, following heavy rainfall.

The main stages involved in the provision of drainage structures are covered in the manual. They are:

a) data collection;
b) hydrological studies (estimation of peak flows);
c) selection of the appropriate drainage structure for a particular site;
d) hydraulic analysis (estimation of the size of drainage structure);
e) structural design, including the use of materials; and
f) Construction.

In addition to addressing each of these stages on a step by step basis, Part C provides detailed guidance on related aspects of roadside slope stabilization.

1.4 Typical drainage features

The typical drainage features of a road are shown on Figure C.1.1. Typical drainage structures are shown in Figures C.1.2 to C.1.7.

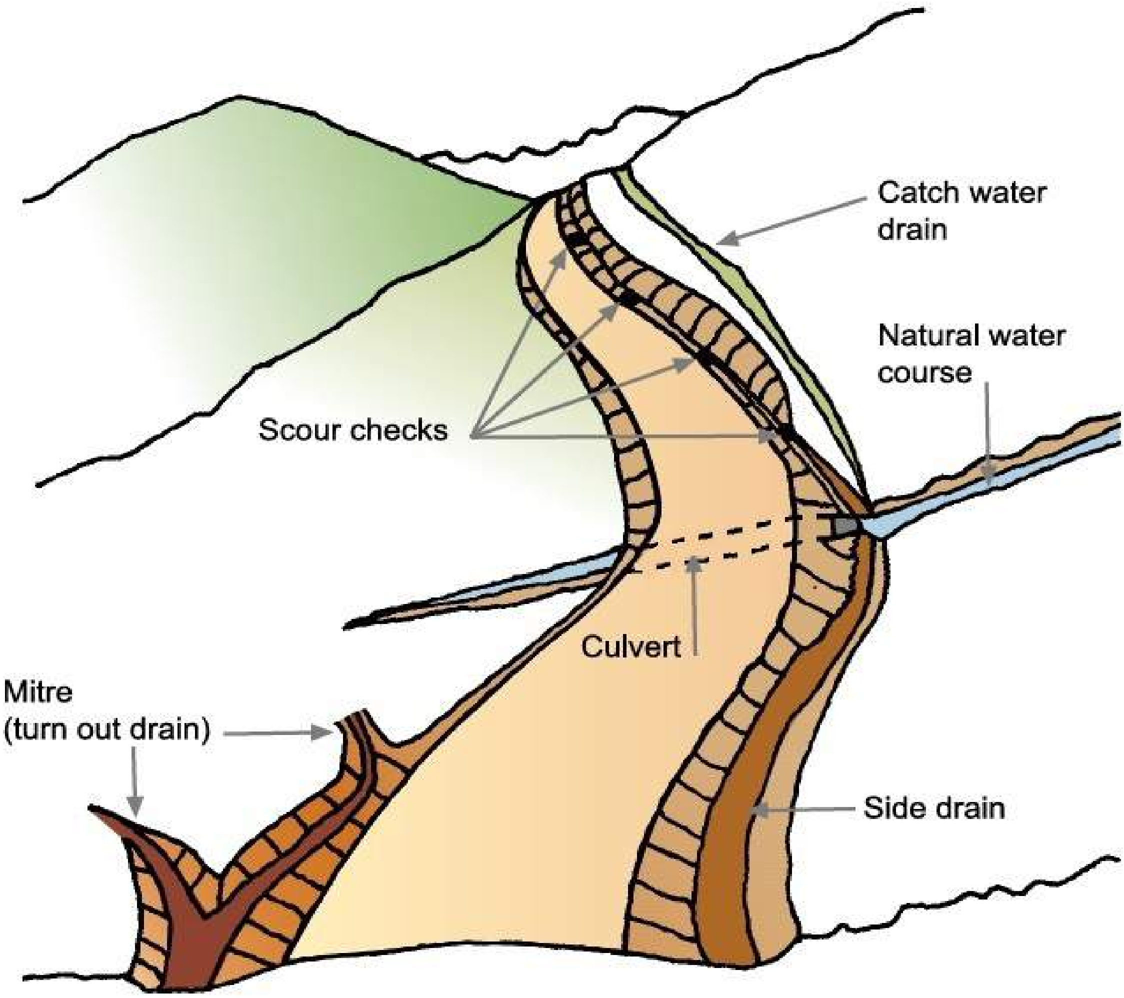


Figure C.1.1: Typical road drainage layout

Cross culvert: The culvert is a structure constructed under the road and is designed to allow water from the drains and/or natural watercourse to safely cross under the roadway. A cross-culvert may have a circular, rectangular or U-shaped opening.
A **box culvert** is a reinforced concrete cross-culvert with a rectangular opening comprising a base slab, side walls and top slab designed to accommodate higher flows of water than normally possible with a pipe culvert.

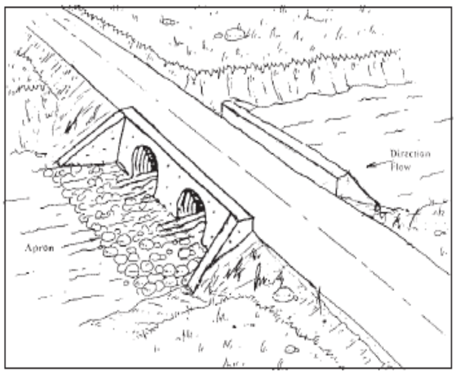


Figure C.1.2: Stream culvert

Relief culvert: The relief culvert is a structure constructed under the road and is designed to allow water from the side drains to safely cross under the roadway into the natural drainage of the surrounding environment. Relief culverts may have a circular, rectangular or U-shaped opening.

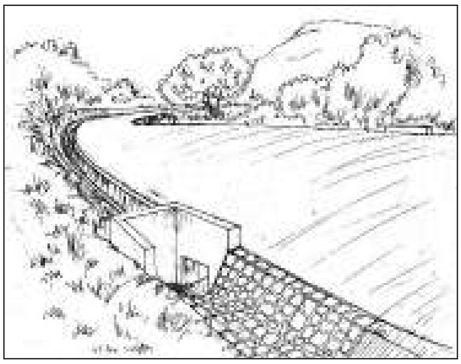


Figure C.1.3. Side drain relief culvert

Access culvert: The access culvert is a structure constructed under the road and is designed to allow water from the side drains to cross the road at junctions and accesses.

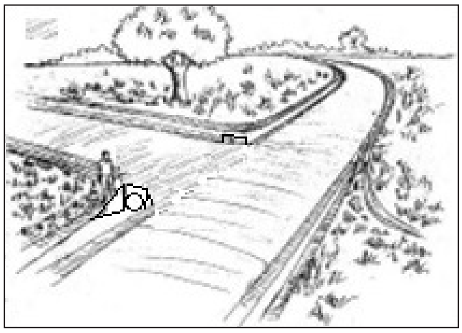


Figure C.1.4: Side drain access culvert

Drift: A drift is a low-level structure constructed to allow water from the drains and/or natural watercourse to safely cross over the road at bed level.

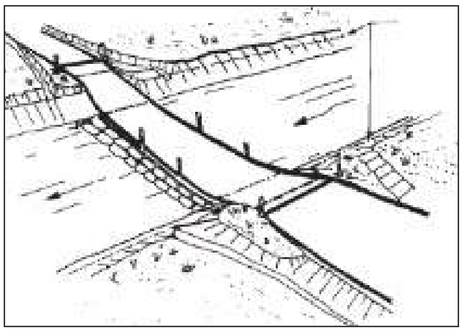


Figure C.1.5: Drift

Vented ford (vented drift or causeway): Vented ford is a medium level structure designed to allow the normal flow of water in a natural watercourse to pass safely through openings below the roadway and to be overtopped during periods of heavy rainfall.

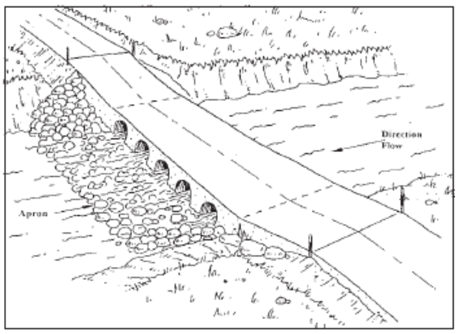


Figure C.1.6: Vented Ford

Bridge: A structure with at least one span more than 6 meters long providing a means of crossing safely above water, railway or other obstruction whether natural or artificial.

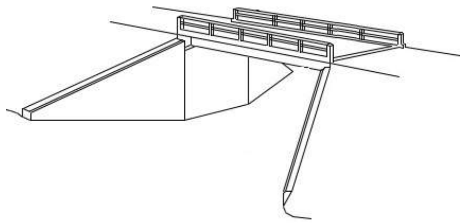


Figure C.1.7: Single span bridge

1.5 Summary of standards and departures from standards

1.5.1 Design Standards and Storm Return Period

Drainage systems cannot be designed for the very worst conditions that might occur on extremely rare occasions. This would be too expensive. The various standards for the design of drainage systems are therefore based on different levels of risk that are attached to the likely occurrence of the different storm intensities for which they are designed, assuming that appropriate routine maintenance is carried out. Storm events are defined by the intensity and duration of rainfall and are extremely variable in nature over periods of many years. Thus, a statistical distribution of storm severities shows that very severe storms are quite rare and that less severe storms are more common. The risk of a severe storm occurring is defined by the statistical concept of its likely return period, which is directly related to the probability of such a storm occurring in any one year. Thus, a very severe storm may be expected, say, once every 50 or 100 years but a less severe storm may be expected every 5 or 10 years. This does not mean that such storms will occur on a regular basis. A severe storm expected once every 100 years has, on average, a probability of 1 in 100 (or 0.01 or 1%) of occurring in any particular year. Similarly, a storm of lower intensity that is expected to occur, on average, once every 5 years has a probability of 1 in 5 (or 0.2 or 20%) of occurring in any one year. The operative words here are "on average" and there is always a finite probability that the worst storm for (say) 200 years may occur at any time.

Most drainage structures are likely to be severely damaged if their capacity is exceeded for any length of time. Hence capacity is the most important aspect of the design. In general, the more severe the storm for which the structure is designed, the more expensive it is to build. Drainage standards are therefore defined by the level of risk. This is done using the concept of return period of the maximum storm for which they are designed.

1.5.2 Level of risk

There are three factors that determine the level of risk that is appropriate for each structure namely:

- the standard of the road (i.e. the traffic level);
- the cost of the drainage structure; and
- the severity of the consequences should the road become impassable because of a failure of the drainage system.

If a drainage structure on a road carrying high levels of traffic is damaged or fails completely, the disruption and associated costs to the traffic can be very high. Therefore, the structures on such a road are designed for low risk (i.e. for storms of long return periods). On the other hand, if a drainage structure should fail on a road carrying low levels of traffic, the likely disruption to traffic and the associated costs are correspondingly less, and hence the higher cost of designing the drainage for low risk cannot be justified. The drainage is therefore designed for shorter storm return periods. Table C 1.1 below gives guidance to the storm design return periods in years for different drainage structures.

Table C.1.1: Design Return Periods

Type of drainage structure	Design Return Period (years)	Maximum Return Period (years)
Unlined side drains/ditches	2	5
Lined side drains/ditches	5	10
Drifts	2	5
Vented drifts	5	10
Pipe culverts	10	25
Minor box culverts	10	25
Major box culverts	25	50
Small bridges, span < 30 m	25	50
Major bridges, span ≥ 30 m	50	100

1.5.3 Departures from standards

It is fundamental to the concept of setting standards that they should be applied at all times. However, the basic standards for drainage structures and drainage design cannot be precisely defined because sufficient data may not be available to carry out the designs in the ideal way. (Site specific reasons and other constraints may also prevent ideal design conditions). As a result, the designer must often use simpler and apparently less accurate methods. The designer must select the most appropriate method and exercise a degree of engineering judgement in selecting the result for the design.

The same arguments do not apply to the detailed engineering design of the components of the drainage system once the maximum water flow has been estimated. For detailed design, standard drawings and specifications are provided. If the designer wishes to depart from these standards, then written approval is required from the Ministry of Public Works. The designer must submit all proposals for departures from the standards to the relevant client officer in the Ministry for evaluation.

1.6 Design standards for culverts

1.6.1 Culvert sizes

The minimum requirements for culvert sizes are summarized in Table C.1.2.

Table C.1.2: Minimum culvert size

Watercourse culvert	Minimum size (mm)	Access and relief culvert	Minimum size (mm)
Pipe	900 diameter	**Pipe**	600 diameter
U-Culvert	1200 wide x 900 high	**U-Culvert**	900 wide x 700 high
Rectangular	1000 x 1000	**Rectangular**	1000 x 1000

The minimum specified culvert sizes are to facilitate cleaning of the culvert barrel. U-culverts are preferred to pipe culverts for access and relief culverts as they have larger openings. The maximum diameter for a pipe culvert is 1.8 meters. All culverts on LVRs should have a minimum cover of 600 mm.

1.6.2 Minimum and maximum velocities in culverts

A culvert generally increases the velocity of flow above the velocity of flow in the natural channel. High velocities are most critical just downstream from the culvert outlet, and the erosion potential from the energy in the water must be considered in culvert design. The minimum and maximum velocities of flow in culverts is are summarized in Table C.1.3.

Table C.1.3: Minimum and maximum velocities in culverts

All culverts	Velocity, m/s	Remarks
Minimum	1.0	Check against siltation/sedimentation
Maximum	3.0	Check against erosion

1.6.3 Freeboard

The minimum required freeboard for box culverts is summarized in Table C.1.4.

Table C.1.4: Freeboard for box culverts

Size of culvert	Freeboard (m)
Minor culverts: spans ≤ 2.0 m	0.30
Major culverts: spans > 2.0 m	0.60

1.6.4 Multiple barrel culverts

Experience has shown that multiple barrel culverts at one location rarely function according to the design assumptions. The barrels tend to plug with debris. During construction, multiple barrel culverts should be fitted within the natural dominant channel with minor widening of the channel to avoid conveyance loss through sediment deposition in some of the barrels. Generally, multiple barrels are to be avoided where:

- the approach flow is of high velocity (a situation that requires a single barrel or special inlet treatment);
- a high potential exists for debris; and
- a meander bend is present immediately upstream.

1.6.5 Culvert material selection

The Drainage Engineer should consider replacement costs, durability and availability in selecting material for the construction of culverts. While metal culverts are imported, cement for manufacturing of culverts is obtained locally. Concrete culvert pipes are therefore normally preferred. Metal culverts may be subject to vandalism, where culvert parts are taken by people and sold as scrap, a raw material for fabrication of farm implements and other metallic parts. The use of concrete is a good deterrence for this malpractice.

1.6.6 End treatment (inlet and outlet)

The following inlet/outlet treatments are recommended for the hydraulic design:

a) Headwalls to:
   - provide embankment stability and embankment erosion protection;
   - provide protection from buoyancy; and
   - shorten the required structure length.

b) Wingwalls to:
   - increase hydraulic performance of the culvert (flare angle between 30° and 60°);

- retain the roadway embankment to avoid a projecting culvert barrel;
- protect side slopes where the channel slopes are unstable; and
- streamline water flow where the culvert is skewed to the normal channel flow.

c) Aprons to reduce scouring from high headwater depths or high velocity.

d) Cut-off Walls to:

- protect the culvert inlet from piping; and
- protect the culvert outlet from erosion.

e) Weep Holes to relieve uplift pressure and reduce hydrostatic pressure behind headwalls.

1.6.7 Culvert alignment and grade

It is recommended that culverts be placed on the same alignment and grade as the natural streambed, especially on year-round streams. This helps maintain the natural drainage system and minimizes downstream impacts.

In some cases, it may not be possible or feasible to match the existing grade and alignment. This is especially true in situations where culverts are conveying hillside runoff. If following the natural drainage course results in skewed culverts, culverts with horizontal or vertical bends, or requires excessive solid rock excavation, it may be more convenient to alter the culvert profile or change the channel alignment up or downstream of the culvert. This is best evaluated on a case-by-case basis with potential environmental impacts being balanced with construction and functional issues.

To achieve hydraulic efficiency, culverts should be installed on the main course of the stream channel. Thus, the actual location of the culverts within stream channels and flood plains must be determined by the Drainage Engineer on site, after evaluation of the overall site conditions.

1.6.8 Stability of stream channels

Culverts should be installed on the main course of the stream channel or flood plain. After installation of a culvert, it is important that the site conditions are kept stable at both upstream and downstream of the culvert. Changing of the site conditions may end up changing the site contours and divert the main course of the river channel away from the culvert location and consequently reduce the hydraulic efficiency of the culvert system.

1.6.9 Outlet protection

Outlet protection consists of the construction of an erosion resistant section between a culvert outlet and a stable downstream channel. Erosion at an outlet is chiefly a function of soil type and the velocity of the culvert discharge. Therefore, in order to mitigate erosion, the design must stabilize the area at the culvert outlet and reduce the exit velocity to a velocity consistent with a stable condition in the downstream channel. The design procedure for outlet protection consists of:

- the calculation of the discharge velocity for the design flow;
- an assessment of the erosion potential at the outlet and other critical site factors; and
- the selection of an appropriate design which protects the site.

1.7 Design standards for open drains

1.7.1 Dimensions of drains

The minimum and maximum dimensions for drains and the required freeboard are summarized in Tables C.1.5 and C.1.6.

Table C.1.5: Minimum and maximum dimensions of drains

Type of drainage structure	Minimum width (m)	Side slope (H : V)
Open – trapezoidal - lined	0.3	1:1 – 1:2
Open – trapezoidal - unlined	0.3	1:3 – 1:5

Table C.1.6: Freeboard for drains

Type of drainage structure	Freeboard (m)
Open drains	0.30

1.7.2 Carrying capacity and flow velocity

The roughness coefficients to be applied for the calculation of carrying capacity of drains are summarized in Table C.1.7.

Table C.1.7: Manning's Roughness Coefficient, n

Material in the drain	Roughness coefficient
Concrete lined channel.	0.013 – 0.015
Sandcrete block	0.015 – 0.020
Masonry	0.017 – 0.030
Earth (new)	0.018 – 0.030
Earth (existing)	0.022 – 0.060

The maximum and minimum permitted flow velocities in drains are summarized in Table C.1.8.

Table C.1.8: Flow velocities in drains

Drainage Structure	Minimum (m/s)	Maximum (m/s)
Open earth drains (no lining)	0.3	0.6
Stone or block masonry	0.6	1.8
Plain/Reinforced concrete	0.6	3.0
Dry compacted gravel or clay	0.6	1.0

1.8 Subsoil drainage

Where it is evident during the pavement investigations that a high water level exists, a drainage blanket or French Drain should be provided in addition to the roadside drains. The purpose of the subsoil drainage is to drain off all the water that will collect behind and underneath the surface drains to prevent building up of water pressures and ingress of water into the pavement structure.

1.9 Environmental and safety considerations

1.9.1 Relevance

In the design and construction of drainage structures, it is necessary that consideration be given to the environmental impact and safety of the structure to the inhabitants within the communities where the project is being undertaken.

1.9.2 Environmental considerations

The following environmental considerations should guide the design and construction of drainage structure:

- avoid excessive erosion;
- avoid sedimentation caused by low outlet velocities;
- avoid stream degradation caused by improper culvert placement; and
- avoid destruction of upstream vegetation due to excessive ponding.

1.9.3 Safety considerations

Safety considerations are vital in all designs including design of drainage structures. Traffic and child safety are major considerations that the Drainage Engineer should seek to address in the design and construction of drainage structures.

Traffic safety

An exposed culvert end acts as an unyielding obstruction likely to bring a vehicle to an abrupt stop, causing considerable damage to the vehicle and deceleration forces on the occupants. Similarly, humps at culvert crossing locations can be dangerous to vehicles on the road and bring about deceleration forces on the occupants and loss of control.

The hazard presented by culverts in private and access road entrances should be minimized by placing them as far from the roadway as practicable and avoiding the use of vertical headwalls.

Child safety

Streams and culverts will always attract children. Therefore, sudden deep drop-offs in the streambed should be avoided, particularly if submerged during normal flows. Where the invert must be placed unusually low to accommodate future deepening of the channel, and it is not feasible to fence off the culvert, consideration should be given to placing fill inside the culvert to reduce the depth of water. The fill should be protected from scouring by a temporary layer of riprap or gabions.

1.10 Classification of road drainage

1.10.1 Components of road drainage

The drainage of low volume roads, like other roads, can be classified into four different parts namely: (i) surface drainage; (ii) subsurface drainage; (iii) slope drainage; (iv) drainage of retaining structures (See Figure C.1.8). Cross drainage is required for rivers and other watercourses that the road must cross.

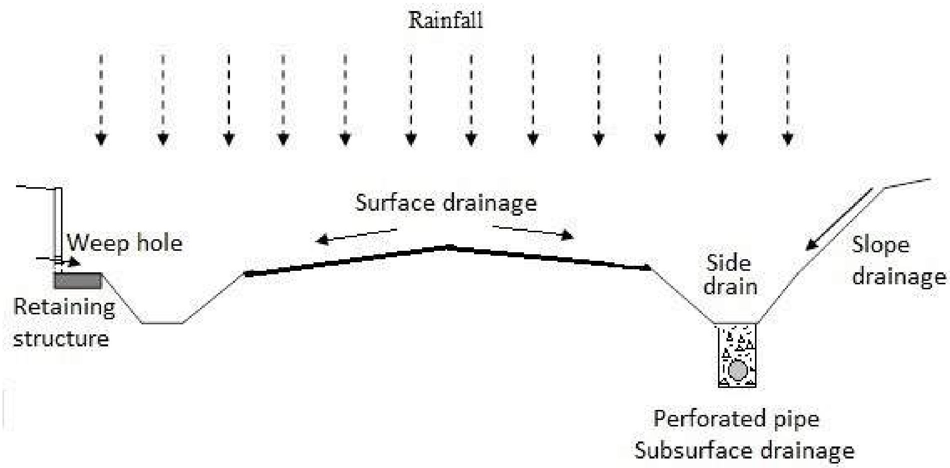


Figure C.1.8: Typical cross section of a road showing drainage types

Surface drainage

Surface drainage is the drainage of surface runoff produced by rainfall on the roadway and the adjoining areas.

Subsurface drainage

Subsurface drainage is intended to reduce the groundwater level and to intercept and drain water infiltrating from the adjoining areas and road surface or rising from the subgrade. Where wet spots are encountered in the subgrade due to seepage through permeable strata underlain by an impervious material, intercepting subsurface drains are used.

Moisture in the road embankment may be either "free-water" or "capillary moisture", both of which the Engineer must consider in the design. French drains are provided to drain subsurface water and prevent it from reaching the pavement structure. In general, the water table should be prevented from rising to within 0.60 m below the sub-base. Good subsurface drainage is particularly important in swampy areas to prevent excessive moisture in the upper subgrade, which ultimately would cause loss in stability through low resistance to wheel loads.

Slope drainage

Slope drainage is drainage of surface runoff from slope surfaces. Measures are required to prevent erosion of the slopes or slope instability, which may be caused by surface water on cut slope, fill slopes and natural slopes or by ground water seeping to the surface of the slope.

Drainage of retaining structures

Drainage of retaining structures is for the purpose of removing trapped water from the backfill behind retaining walls. This is normally achieved by installing weep holes in the wall. Drainage is also required for bridge decks to ensure rainwater is effectively shed from the surface of the road.

1.10.2 Internal and external drainage

Road drainage may also be considered in terms of internal and external drainage. These two inter-related aspects of drainage need to be considered during road design.

Internal drainage

Internal drainage is required to minimize the quantity of water that remains within a road pavement and underlying subgrade that may be moisture sensitive. This is achieved by maximizing the ability of the road to lose water to an external drainage system. The design of internal drainage includes minimizing the quantity of water that gets into a road pavement in the first place.

Effective internal drainage is critical for the performance of LVRs, particularly Low Volume Sealed Roads (LVSR). Natural gravel materials used in the upper pavement layers of LVSRs may be moisture sensitive, and the in situ strength depends on keeping the material dry.

Guidance on Internal Drainage is included in Chapter 2.

External drainage

External drainage is required to drain surface runoff or surface water to minimize the quantity of water on the road surface or to keep the roadway free from flooding.

The four components of external drainage are explained below:

a) The process of determining the quantity of water that falls upon the road itself (rainfall) and associated works required to channel the water away from the road by the drainage system.
b) The process of determining the quantity of water that is generated by the local catchment which flows in the streams, rivers and natural drains that the road must cross. This is water that falls as rainfall over the local catchment near or away from the road.
c) Overland flows generated by the local catchment.
d) Design of the individual engineering features of the drainage system to accommodate the flow of water.

Guidance on the selection and design of drainage structures is included in Chapters 3 to 6. Guidance on the construction of drainage structures is included in Chapters 8 and 9.

2. INTERNAL DRAINAGE

2.1 Sources of moisture entry into a pavement

Moisture movement occurs in various ways in road pavements. Examples of water ingress and egress and their causes are listed in Table C.2.1. Ingress of water must be controlled to ensure the stability of the pavement layers, particularly if the pavement is constructed of natural gravels which may be moisture sensitive.

Table C.2.1: Typical causes of water ingress and egress

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

2.2 Permeability

Permeability is a measure of the ease with which water passes through a material and is one of the key material parameters affecting drainage. Moisture ingress and egress is 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 C.2.2.

Table C.2.2: Typical material permeabilities

Material	Permeability	Description
Gap-graded crushed rock	> 30 mm/s	Free draining
Gravel	> 10 mm/s	
Coarse sand	> 1 mm/s	
Medium sand	1 mm/s	Permeable
Fine sand	10 μm/s	
Sandy loam	1 μm/s	Practically impermeable
Silt	100 nm/s	
Clay	10 nm/s	Impermeable
Bituminous surfacing[1]	1 nm/s	

1. Applies to well-maintained double chip seal. Thicker asphalt layers can exhibit significant permeability as a result of a linking of air voids. Permeability increases as the void content of the mix increases, with typical values ranging from 300 μm/s at 2% air voids to 30 μm/s at 12% air voids. Typically, a 1% increase in air voids content will result in a three-fold increase in permeability (Waters, 1982).

Source: Lay 1998.

2.3 Side drainage and crown height above drain invert.

Side drainage is one of the most significant factors affecting the performance of paved roads. The critical dimension is the height of the crown of the road above the bottom of the drain. The crown height, h, correlates well with the service life of pavements constructed from natural gravels. A minimum height of 0.75 m is recommended. The classification of road drainage is shown in Table C.2.3.

Table C.2.3: Classification of road drainage

Classification	Crown Height
Very good	$h > 0.90$ m
Good	$0.75 \text{ m} < h < 0.90$ m
Moderate	$0.60 \text{ m} < h < 0.75$ m
Poor	$0.40 \text{ m} < h < 0.60$ m
Very poor	$h < 0.40$ m

Note: Classification can move up one class if:

1. the longitudinal gradient >1%; and/or
2. the drain is lined, and the lining connects to the surfacing; or
3. the ground is free draining.

Irrespective of climatic region, if the site has effective side drains and adequate crown height, then the in situ subgrade strength will remain above the design value. If the drainage is poor, the in situ strengths will likely fall below the design value due to moisture ingress.

Figure C.2.1: A well-drained pavement

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.

Avoiding a 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. 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.

It is desirable for good internal drainage that a permeability inversion does not occur. This is achieved by ensuring that the permeability of the pavement and subgrade layers are at least equal or are increasing with depth. For example, the permeability of the base must be less than or equal to the permeability of the sub-base in a three-layered system.

Where permeability inversion is unavoidable, the road shoulder should be sealed to an appropriate width to ensure that the lateral wetting front does not extend under the outer wheel track of the pavement.

Ensuring proper shoulder design

When permeable base course materials are used, attention must be given to the drainage of this layer. Ideally, the base course 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. A suitable value for paved roads is about 2.5% to 3% for the carriageway, with a slope of about 4% to 6% for the shoulders. Increased crossfalls, typically about 5%, are required for unsurfaced roads.

Exaggerated internal crossfall

Lateral drainage can be encouraged by constructing the internal pavement layers with an exaggerated crossfall, especially where a permeability inversion occurs. This can be achieved by constructing the top of the sub-base with a crossfall of 3-4% and the top of the subgrade with a crossfall of 4-5%. Although this is not an efficient way to drain the pavement it is inexpensive and therefore worthwhile considering, particularly as full under-pavement drainage is rarely likely to be economically justified for LVRs. Figure C.2.2 illustrates the recommended drainage arrangements for an LVR.

If it is too expensive to extend the base course and sub-base material across the shoulder, drainage channels at 3 m to 5 m intervals should be cut through the shoulder to a depth of 50 mm below sub-base level. These channels should be back-filled with material of road base quality, but which is more permeable than the base course itself. The channels should be given a fall of 1 in 10 to the side ditch. An alternative (and preferable) option is to provide a continuous layer of pervious material of 75 mm to 100 mm thickness laid under the shoulder such that the bottom of the drainage layer is at the level of the top of the sub-base.

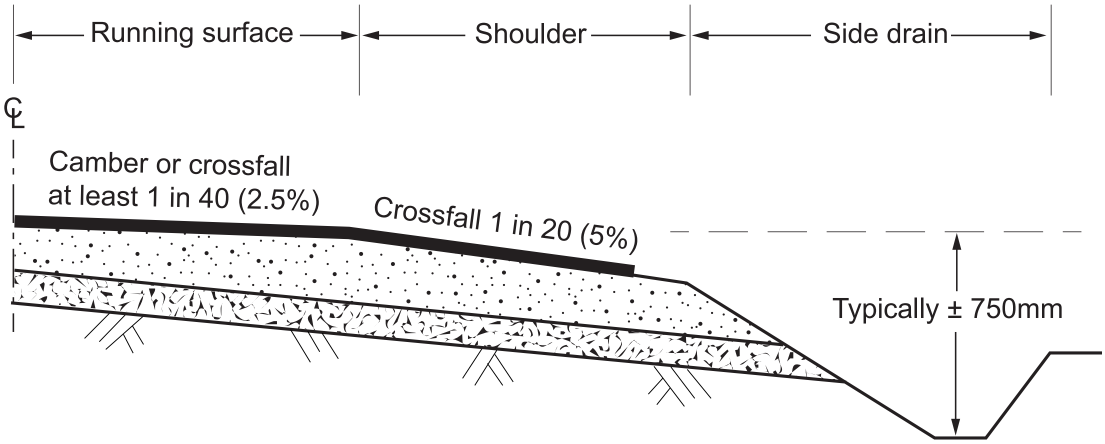


Figure C.2.2: Recommended drainage arrangements

2.4 Sealing of shoulders

A common problem associated with the use of unsealed shoulders is water infiltration into the base and sub-base. This is illustrated in Figure C.2.3. Reasons for water ingress include:

a) rutting adjacent to the sealed surface;

b) build-up of deposits of grass and debris; and

c) poor joint between the base and shoulder (more common when a paved shoulder has been added after initial construction).

It is recommended that, wherever possible, shoulders of paved roads should be sealed. The advantage of sealed shoulders is that they:

- provide better support and moisture protection for the pavement layers and reduce erosion of the shoulders (especially on steep gradients);
- improve pavement performance by ensuring that the zone of seasonal moisture variation does not penetrate to under the outer wheel track;
- reduce maintenance costs by avoiding the need for re-gravelling at regular intervals; and
- 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.

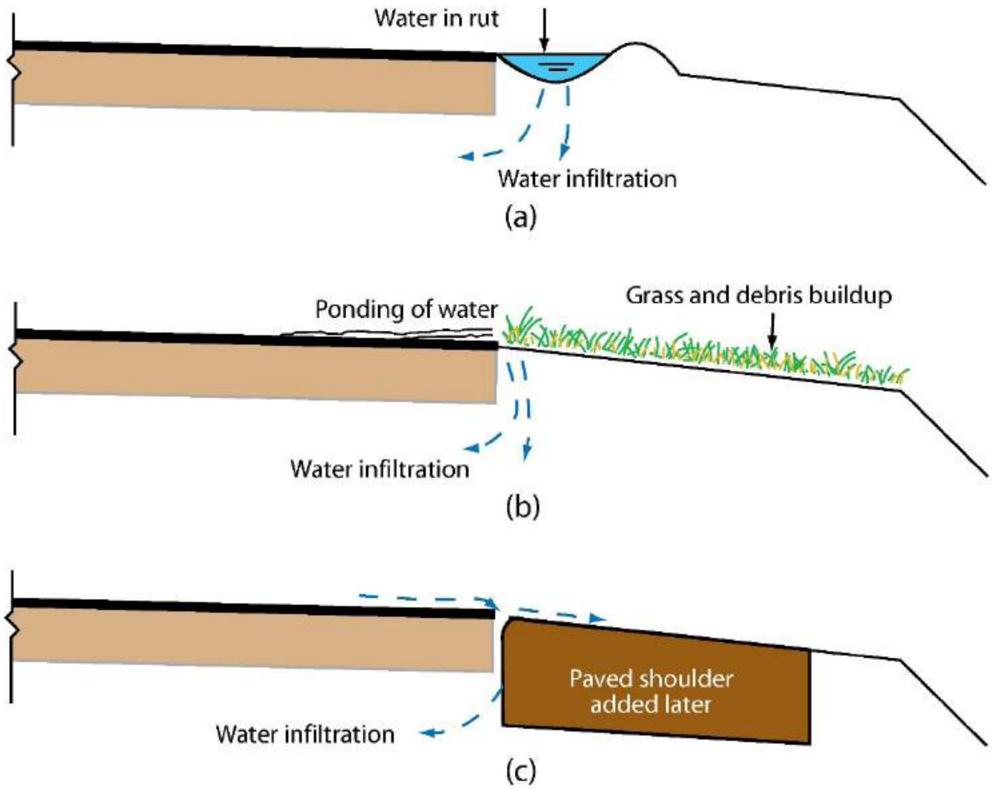


Figure C.2.3: Typical deficiencies associated with pavement shoulder construction

(Adapted from Birgisson and Ruth, 2003)

2.5 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. It is illustrated in Figure C.2.4. This "boxed" construction is a common cause of road failure due to the reduction in strength and stiffness of the pavement material and the subgrade below that is required to support the traffic loading.

Figure C.2.4: Infiltration of water through a permeable surfacing

2.6 Adopting an appropriate pavement cross-section

The two moisture zones in the pavement which are of critical significance are the equilibrium zone and the zone of seasonal moisture variation. These are shown in Figure C.2.5, with the right side of the diagram having a sealed shoulder and the left side an unsealed shoulder.

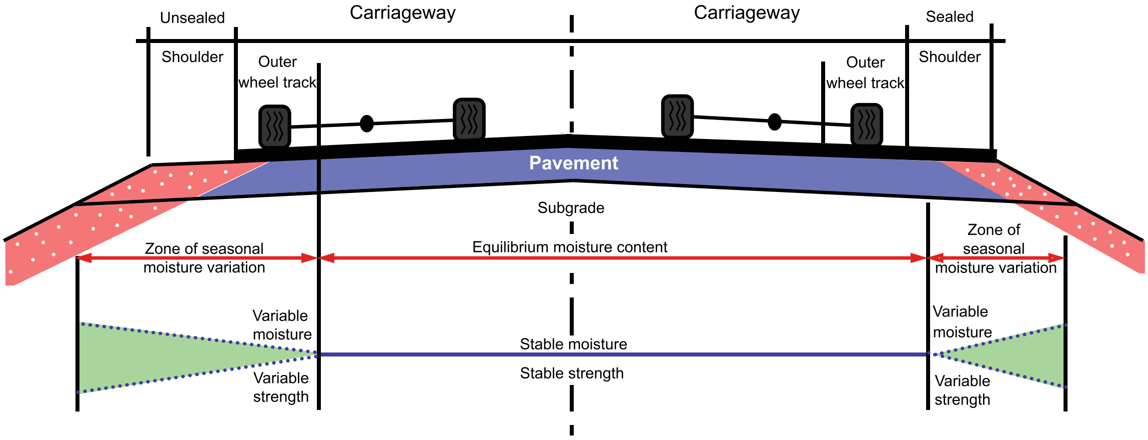


Figure C.2.5: Moisture zones in a typical LVR

From research work carried out in tropical regions (e.g. O'Reilly, 1968; Morris and Gray, 1976; Gourley and Greening, 1999), it has been found that:

- In sealed pavements over a deep water table, the moisture content in the equilibrium zone normally reaches an equilibrium value after about two years from construction and remains constant thereafter.
- In the zone of seasonal variation, the pavement moisture does not reach an equilibrium and fluctuates with variation in rainfall. Generally, this zone is wetter than the equilibrium zone in the rainy season and it is drier in the dry season.

Therefore, the edge of the pavement is of extreme importance to ultimate pavement performance, with or without paved shoulders, and is the most failure-prone region of a pavement when moisture conditions are relatively severe. In order to ensure that the moisture and strength conditions under the outer wheel track will remain stable and largely independent of seasonal variations, the shoulders should be sealed to a width of between about 1.0 and 1.2 m from the edge of the carriageway.

The drainage measures highlighted above are all aimed at:

- preventing water from entering the pavement in the first place;
- facilitating its outflow as quickly as is reasonable, given the cost implications; and
- ensuring that the presence of water in the road pavement for an extended period does not cause failures.

It should be noted that the adoption of any single measure on its own is unlikely to be as effective as the adoption of various complementary measures applied simultaneously. Such an approach forms part of the philosophy of minimizing the risks associated with using locally occurring natural materials in the pavements of LVRs.

3. HYDROLOGICAL STUDIES

3.1 Introduction

Hydrological studies are carried out on road projects to:

- determine catchments for culverts and roadside drains;
- determine the runoff coefficient of the ground cover of the catchment;
- determine rainfall intensities for the project areas;
- estimate peak/maximum flows and establish the High Flood Level (HFL) in order to design an appropriate drainage structure.

3.2 Estimation of peak flow

Estimation of the peak flow (also referred to as the maximum discharge) is critical to the proper sizing of any drainage structure. It is necessary to follow appropriate steps in the determination and estimation of the peak flow prior to sizing of the drainage facility. The peak flows can be determined using different methodologies depending on the available data and size of the catchment area. The methods used include:

1. Field Observations:
   a) direct observation of the size of the stream channel or watercourse
   b) direct observation of erosion and debris
   c) history and local knowledge, and
   d) replicating successful practice.
2. Rational Method
3. Modified Rational Method
4. WinTR20 Computer Software

3.3 Field observation method

This method is appropriate in situations where data on rainfall and topographic maps are not readily available, for example in parts of Liberia. The method requires an experienced Drainage Engineer to make informative observations of the site conditions as well as an experienced Geodetic Engineer to carry out field surveys of the existing drainage structures. There are four field observation methods which are described below.

Direct observation of the size of the stream channel or watercourse

The cross-sectional area of the stream channel or watercourse is determined and the cross-sectional area of the opening of the drainage structure is equated to that of the watercourse. Three assumptions are made:

- That the water level does not rise above the stream channel thus causing overtopping of the channel and flooding of the adjoining areas;
- That the watercourse is free to drain and flows at a speed determined by the gradient and nature of the channel; and
- That no erosion of the channel has occurred.

It is necessary to interview the local inhabitants about the performance of the watercourse to ascertain the accuracy of this method.

The peak flow is computed from the expression:

$$Q = V + A$$

Where,

Q = Peak flow, m^3/s

V = d/t = Average velocity of flow, m/s

d = Distance, m

t = Time, sec.

A = W x D = Cross sectional area of channel, m^2

W = Width at widest section of channel

D = Depth of channel at widest section

The velocity of flow can be estimated by dropping a floating object into the flow and measuring how fast is travels down the stream.

Direct observation of erosion and debris

Flood waters usually carry debris which is trapped around trees, existing structures and other vegetation near the watercourse. This debris leaves a visible water mark on the structures and trees which can be taken as the high flood level.

Erosion at existing drainage structures gives an indication of the actual cross sectional area of the watercourse, which can be equated to the cross sectional area of the opening of the new drainage structure.

History and local knowledge

Historical information and data on high flood levels at a drainage crossing point is very useful in the estimation of maximum flows. Interviews with local inhabitants within the vicinity of the crossing point about high flood levels of past storms is necessary to establish the average height of flood levels to aid in determining the peak discharges at the different locations.

Replicating successful practice

The Drainage Engineer should study the performance of existing drainage structures at the upstream and downstream ends of the crossing point. If the roadway at such locations had not experienced any overtopping over the years from heavy downpours, it is an indication of the successful performance of the drainage structures. It can thus be assumed that the parameters (catchment area, runoff coefficient, rainfall intensity, etc.) used in the design of the drainage structures are valid and can thus be adopted in sizing any new structures.

3.4 The Rational Method

3.4.1 The Rational Method formula

The Rational Method is the most well-known method for estimating peak flow. It is based on direct relationship between rainfall and runoff. The maximum discharge is given by the expression

$$Q = 0.278 * C * 1 * A$$

Where:

Q = Maximum Discharge or Peak Flow, (m^3/s)

C = Runoff Coefficient

I = Rainfall Intensity, (mm/h)

A = Catchment Area, (km^2)

The Rational Formula gives satisfactory discharge results for small catchment areas. Experiments show that this method of estimating discharge runoff begins to lose its accuracy when the drainage area exceeds 10 square kilometers. (Source: Ethiopia Roads Authority Manual for Low Volume Roads, Part E).

Figure C.3.1 is a flow chart showing the steps the Drainage Engineer needs to follow in carrying out the hydrological studies to estimate the peak flow using the Rational Method.

3.4.2 Administrative process

In order to implement the Rational Method, it is necessary to carry out various administrative processes. These processes are necessary to equip the Design Engineer with the relevant data. Two administrative processes are discussed below, namely; (i) collection of existing documents and (ii) field data collection.

Collection of existing documents

The Design Engineer should collect existing data and reports from relevant agencies as part of an initial desk study. This desk study will help in proper planning of the field studies. The following existing data and reports should be collected:

- Topographic maps should be obtained from the survey department or any organization or agency responsible for the development of topographic maps in the country to help identify the stream/river crossings and possible culvert locations and estimation of the catchment areas. The catchment areas are delineated on the topographic maps using a planimeter. The length of streams is determined from the topographic maps using the map measurer. In Liberia where topographic maps are not always readily available, aerial photograph may be used to obtain the needed data, or digital ground modeling software may be used if available.
- Geological maps should be obtained from the geological service department to help identify soil types for determining the runoff coefficient. Runoff coefficients may be obtained from existing literature and adopted for similar soil types in the country.
- Rainfall data should be obtained from the Meteorological Services Department to determine the return periods and rainfall intensity. This is an important parameter considering the severe rainfall pattern in the country. The mean annual rainfall for the major towns and cities in the different districts may be obtained from existing records.
- Existing literature and designs should be obtained from the relevant roads agencies and authorities to help estimate the capacity and design of the drainage structures.

Field data collection

The field studies should involve detailed surveys of existing drainage structures and conditions within the project area. The surveys should include the following:

a) Inventory and condition surveys of existing drainage structures

A detailed condition survey and inventory of existing drainage structures (culverts, drifts, roadside drains, etc.) should be carried out as part of the field surveys. This gives the Design Engineer first-hand information on the need to either maintain or replace the existing drainage structure. It involves visual observation and inspection of the drainage conditions of the area and the condition of existing drainage structures. An inventory form should be developed for collection of field data. The information collected should include, size, location, shape, length, material and condition of the drainage structure.

b) Assessment of High Flood Level (HFL)

To establish the high flood level during the field surveys, people living along the corridor of project road should be interviewed to find out water levels during periods when flooding was experienced due to heavy rainfalls. The Engineer should also look out for water marks on structures sited within the projected area.

c) Land use and soil type

Information on land use and vegetation cover is necessary for the estimation of the runoff coefficient, which is an input in estimating the Peak Flow. The Design Engineer should observe and note the soil type and vegetation cover during the field survey to help establish the runoff coefficient from relevant tables.

d) Topographic surveys

Detailed topographic surveys may be required to establish the location and elevations of existing and new drainage structures e.g. bridges, culverts, drifts, manholes, catch pits, etc. and other structures such as buildings, railway lines, utility service lines, etc. The invert levels of the drainage structures at the inlet and outlets should be noted. Levels should be taken within the existing drainage channels and streams upstream and downstream to help develop the profiles of the channels. The cross sections of the drainage channels, streams, etc. upstream and downstream should be recorded. Areas prone to flooding should be identified and surveyed. Detailed topographic surveys must be carried out upstream and downstream of locations where a drainage structure may be required.

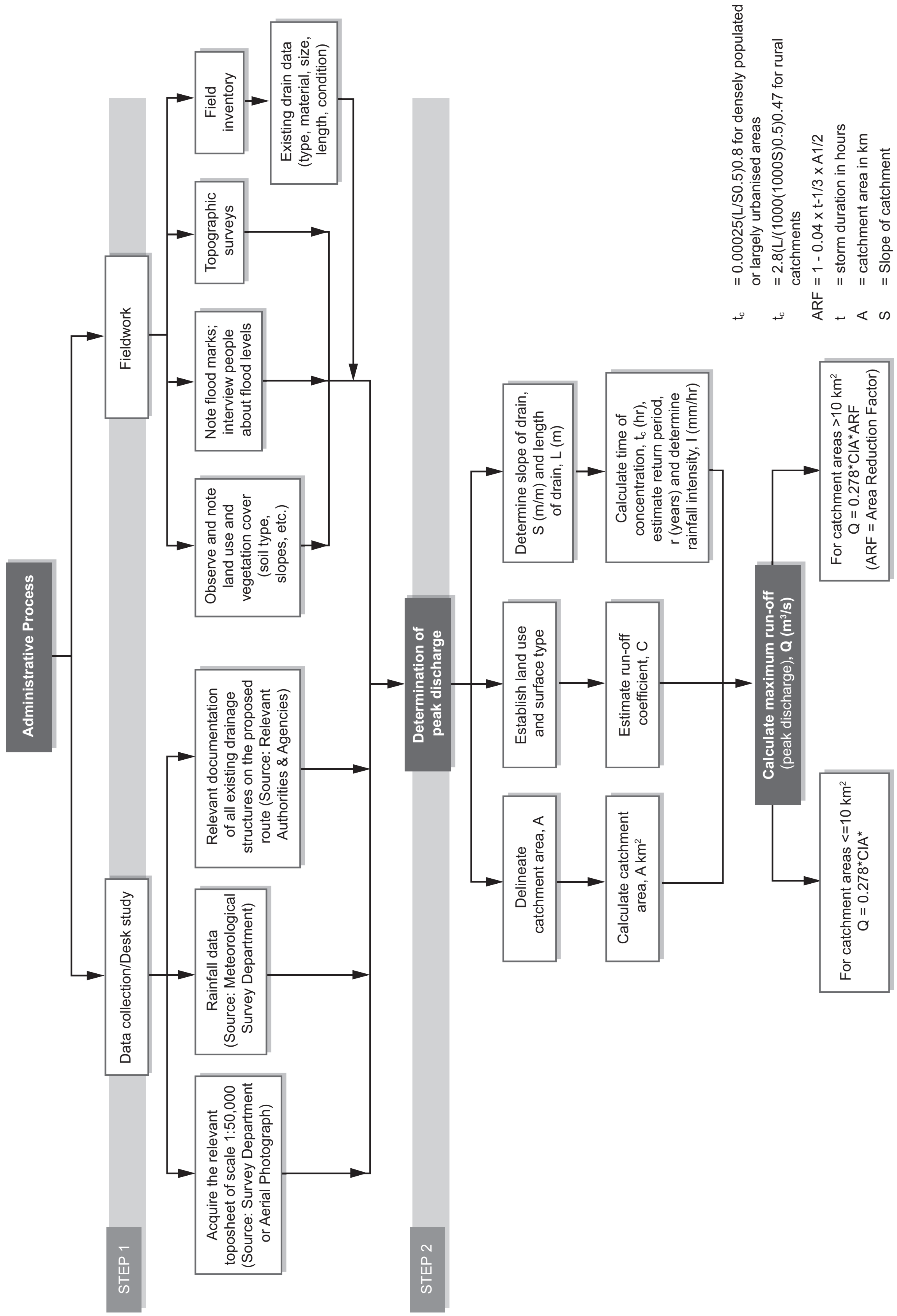


Figure C.3.1: Flow chart for estimating maximum flows/discharge

3.4.3 Runoff Coefficient, C

The runoff coefficient is dependent on the ground or vegetation cover and road surface. Hence the need to note the vegetation/ground cover and the land uses within the catchment during the field surveys. The geological map of the area should give an indication of the soil type, which is a determining factor of the runoff coefficient. Values of runoff coefficient for different surfaces for use in the Rational Formula are shown in Table C.3.1. For catchments with different vegetation cover and land surface area (A), the effective runoff coefficient is determined from the expression:

$$C = \frac{C_1A_1 + C_2A_2 + C_3A_3 + \cdots + C_nA_n}{A_1 + A_2 + A_3 + \cdots + A_n}$$

Table C.3.1: Values of Runoff Coefficient 'C'

Surface description	Runoff Coefficient, C
Concrete or asphalt pavement	0.9 - 1.0
Bituminous macadam and double bituminous surface treatment	0.7 – 0.9
Gravel surface road and shoulder	0.3 – 0.6
Residential area - city	0.3 – 0.6
Residential area – town	0.2 – 0.5
Rocky surface	0.7 – 0.9
Bare clay surface	0.7 – 0.9
Forest land (sandy to clay)	0.3 – 0.5
Flat cultivated areas (not flooded)	0.3 – 0.5
Steep or rolling grassed areas	0.5 – 0.7
Flooded or wet paddies	0.7 – 0.8

Source: Bureau of Design, USA. Design Guidelines– Criteria and Standards, Page 745.

The Engineer is required to consider the future land use in selecting the value of C. The average of the minimum and maximum values is recommended.

3.4.4 Time of Concentration, t_c

The time of concentration, t_c is also referred to as "inlet time". It is the time taken for a particle of water to travel from the remotest part of the catchment area to the entry point or inlet of a culvert. This can be determined from the parameters gathered from the topographic maps.

Where,

d_r = Distance from the remotest part of the catchment

V_f = Velocity of flow, m/s

Alternatively, the time of concentration can be computed from the equations:

$t_c = 0.00025(L/S^{0.5})^{0.8}$ for densely populated or largely urbanized areas

$t_c = 2.8(L/(1000(1000S)^{0.5})^{0.47}$ for rural catchments

Where,

L = Length of stream, m

S = Slope of the catchment.

3.4.5 Rainfall Intensity

Rainfall along with catchment characteristics determines the flood flows upon which storm drainage design is based. Although rainfall intensity varies during precipitation events, many of the procedures used to derive peak flow are based on assumed constant rainfall intensity.

Rainfall intensity, I, is defined as the rate of rainfall and is typically given in units of millimeters per hour (mm/h). Intensity – Duration – Frequency (IDF) curves are provided by national meteorological services for use in the determination of rainfall intensity, I, in a specific area within the country (see Appendix C.1).

3.4.6 Return Period

The required Return Periods for the design of drainage structures in Liberia are given in Table C.1.1. For example, a 10-year return period is recommended for minor culverts on rural roads.

3.4.7 Catchment Area

The Catchment Area, A, is the area of the watershed that contributes runoff to the drainage crossing. Topographic maps covering the project area must be obtained from the National Survey Department or Geographical Information Systems (GIS) Offices and a desktop study carried out. Where suitable maps are not available, the catchment area can be estimated from field surveys, aerial photographs or existing computer ground modeling software. Main stream crossings should be identified and areas contributing to such streams mapped out to establish the watershed for each stream crossing. The catchment area for each of the culvert positions should be determined using a digital planimeter or a similar method. An example of a catchment area map is shown in Figure C.3.2. The procedure for estimating the catchment size is further explained in Appendix C.2.

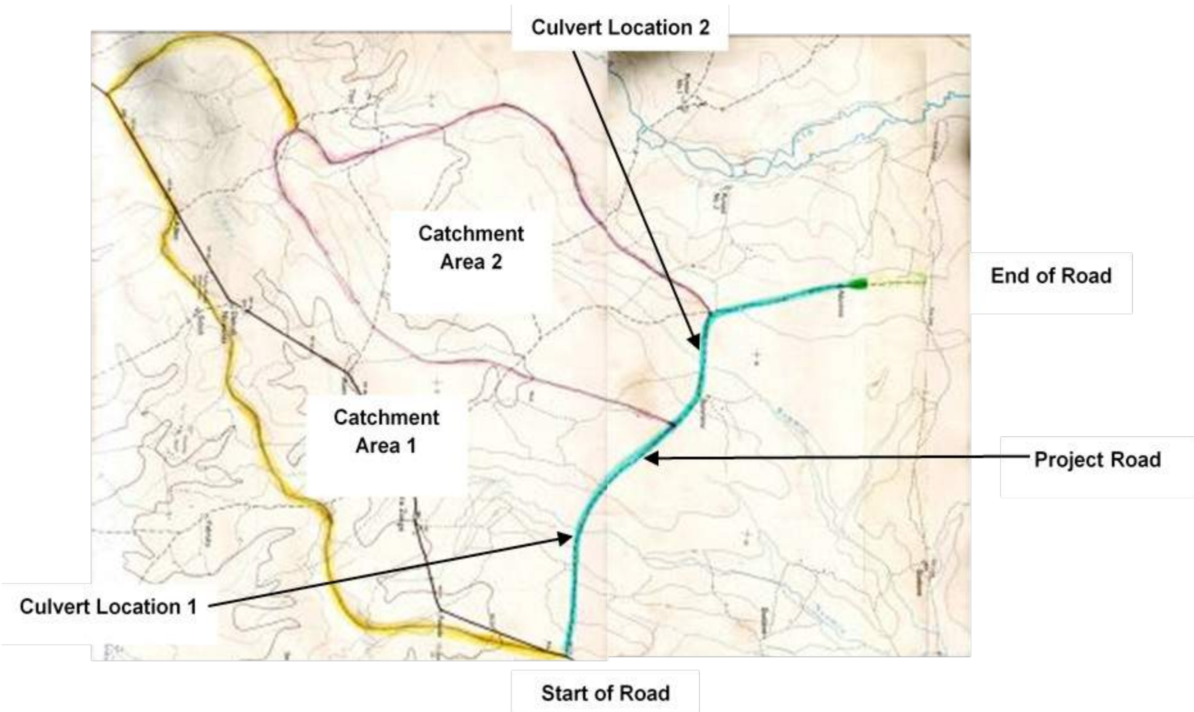


Figure C.3.2: Example of a catchment area on a topographic map

3.5 Modified Rational Method

In the Rational Method it is assumed that the intensity of the rainfall is the same over the entire catchment area. The consequence of applying the method to large catchments is an over-estimate of the flow and therefore a conservative design. The Modified Rational Method takes into account the spatial variation of rainfall intensity across a larger catchment. The effective area of the catchment is reduced by multiplying by the Areal Reduction Factor (ARF) given by the following equation:

$$Q = 0.278 * C * I * A * ARF$$

Where,

Q = Maximum Discharge or Peak Flow (m^3/s)

C = Runoff Coefficient

I = Rainfall Intensity (mm/h)

A = Catchment Area (km^2)

$$ARF = 1 - 0.04t^{-1/3} A^{1/2}$$

t = storm duration in hours.

3.6 Technical Release 20 (TR 20)

The Natural Resources Conservation Service (formerly Soil Conservation Service) of America developed the runoff curve number as a means of estimating the amount of rainfall appearing as runoff. Technical Release 20 (TR 20) is a computer based hydrological model that employs the runoff curve number to provide estimation of peak discharges and runoff from complex catchments areas or watersheds. The software requires much of the same basic data as the Rational Method namely catchment area, a runoff factor, time of concentration, and rainfall. The current version is the WinTR-20 which uses the windows platform. The basic data inputs include:

- sub-area watershed characteristics;
- stream reach hydraulics;
- storage structure hydraulics; and
- evaluation of storm amounts and distribution.

WinTR-20 Hydrological Model Basic Tutorials can be found on the internet.

3.7 Design for climate resilience

Climate change will affect roads and highways in many different ways. It will result in higher temperatures and more intense storms in some areas. It will be necessary to cope with more frequent floods, and faster and more destructive water velocities. Much of the historic data on which hydrological design depends will no longer be valid. Until new models are developed and verified, one of the simplest and important actions that can be taken is to design drainage structures based on estimates of storm characteristics with higher return periods. This is essentially increasing the safety factor. In addition, there are various other strategies that can help to increase climate resilience. In general, these include:

- identifying the most vulnerable areas and essentially increasing the 'safety factor' inherent in their design;
- ensuring that the drainage systems are well maintained and functioning correctly; and
- local realignment in critical areas or high priority roads where the consequences of failure and closure are severe (this is usually only considered as part of a repair and rehabilitation project after storm damage has occurred).

Increasing the safety factor includes:

- using drifts and vented drifts that can be safely overtopped instead of culverts that can become blocked by debris;
- adding additional protection to culverts that might be blocked by debris;
- better surface drainage so that water is dispersed off the road more frequently; and
- reducing water concentration by means of additional cross drains and turn-out drains to lower the volume of water that each one needs to deal with.

Erosion is a serious problem in many areas and adverse climate change will make matters worse. There are also likely to be more severe geotechnical problems (e.g. slope stability) caused by climate change. These are dealt with in chapter on slope stabilization.

Ensuring that the drainage system is working correctly is essentially a maintenance issue, although there may be examples of poorly designed culverts with improper alignment or grade relative to the channels and ditch lines that will need to be repaired or replaced, usually after failures have occurred.

Further information on increasing resilience of rural roads to the effects of climate can be found in the Climate Adaptation Guidelines and Handbook prepared by the CSIR Consortium and available on the ReCAP website.

3.8 Worked Examples

Worked Example 1

Your organization has recently won a contract to carry out spot improvement works on a gravel feeder road in Liberia. The terrain is generally rocky. Investigations show that it takes 42 minutes for water to travel from the remotest part of the catchment to any of the culvert locations.

(i) Identify the road link on a 1:50,000 topographic map.

(ii) Identify the culvert locations

(iii) Map out the catchment area for each culvert location

(iv) Compute the maximum flow for a 10-year return period assuming the catchment area is 2 km^2.

Solution

Step 1: Obtain a 1:50,000 topographic map from the Survey Department that covers the project area

Step 2: Identify the road link on the topographic map.

Step 3: Identify on the road link all river crossings. These are the proposed culvert locations on the road.

Step 4: Delineate the catchment area for each identified culvert location. This is achieved by studying the contours on the topographic map. The area can be obtained by tracing over the catchment area using a digital planimeter. (The Ghana Department of Feeder Road's Guideline Notes for Design of Drainage Structures provides step by step guidance for determining the catchment areas from 1:50,000 topographic maps for different terrain- See Appendix C.2).

Step 5: The maximum flow is given by the expression:

$$Q = 0.278 \times C \times I \times A$$

Where: Q = Maximum Discharge or Peak Flow, (m^3/s)

C = Runoff Coefficient

I = Rainfall Intensity, (mm/h)

A = Catchment Area, (km^2)

Determining the parameters:

1. Area (A), is as computed in step 4 but given as 2 km^2 in the question.
2. Runoff coefficient (C), is dependent on the vegetation cover and land use. It is stated in the question that the land is generally rocky. Thus, from Table C.3.1, C is between 0.7 – 0.9. It is advisable to use the upper limit to care of the future change in land use. Hence C = 0.9.
3. Rainfall intensity (I), is dependent of the time of concentration and rainfall return period. This is determined from IDF tables or graphs for the area. From the question the time of concentration is given as 42 minutes (0.7 hours) and the return period is given as 10 years. From the IDF Curves (Appendix C.1 – Lungi Station), the rainfall intensity I = 80 mm/h.
4. The Maximum Discharge or Peak Flow, Q, is determined as follows:

$Q = 0.278 \times C \times I \times A$

$Q = 0.278 \times 0.9 \times 80 \times 2$

$\mathbf{Q = 40\ m^3/s}$

Worked Example 2

Your organization has been contracted to undertake design studies to upgrade the existing gravel feeder road in Liberia to double sealed bituminous surface. The catchment is generally in flat cultivated area. Site investigations have revealed that it takes 60 minutes for water to travel from the remotest part of the catchment to any of the culvert locations.

(i) Identify the road link on a 1:50,000 topographic maps.
(ii) Identify the culvert locations.
(iii) Map out the catchment area for each culvert location.
(iv) Compute the maximum flow for a 25 year return period assuming the catchment area 11.5 km^2.

Solution

Step 1: Obtain a 1:50,000 topographic map from the Survey Department that covers the project area.

Step 2: Identify on the road link on the topographic map.

Step 3: Identify on the road link all river crossings. These are the proposed culvert locations on the road.

Step 4: Delineate the catchment area for each identified culvert location. This is achieved by studying the contours on the topographic map. Refer to the Ghana DFR Guideline Notes for Design of Drainage Structures provides step by step guidance for determining the catchment areas from 1:50,000 topographic maps for different terrain (Appendix C.2). The area can be obtained by tracing over the catchment area using a digital planimeter.

Step 5: The maximum flow is given by the expression:

$$Q = 0.278 \times C \times I \times A \ \ x\, ARF$$

Where:

Q = Maximum Discharge or Peak Flow, (m^3/s)

C = Runoff Coefficient

I = Rainfall Intensity, (mm/h)

A = Catchment Area, (km^2)

ARF = Area Reduction Factor:

t = storm duration in hours

Determining the parameters:

1. Area (A), is as computed in step 4 but given as 11.5 km^2 in question. This is greater than 10 km^2 so an area reduction factor (ARF) must be applied.
2. Runoff coefficient (C), is dependent on the vegetation cover and land use. It is stated in the question that the land is generally rocky. Thus from Table C.3.1, C is between 0.7 – 0.9. It is advisable to use the upper limit to care of the future change in land use. Hence C = 0.9.
3. Rainfall intensity (I), is dependent of the time of concentration and rainfall return period. This is determined from the IDF graphs for the area. From the question the time of concentration is given as 60 minutes and the return period is given as 25 years. From the IDF Curves for Bonthe Station (Appendix A.1), the rainfall intensity I = 24 mm/h.
4. $Q = 0.278 \times C \times I \times A \times ARF$

   $ARF = 1 - 0.04t^{-1/3} A^{1/2} = 1 - 0.04 \times 1^{-1/3} \times 11.5^{1/2} = 1 - 0.14 = 0.86$

   $Q = 0.278 \times 0.9 \times 24 \times 11.5 \times 0.86 =$ **59 m^3/s**

4. TYPES OF DRAINAGE STRUCTURES

4.1 Introduction

The road camber or crossfall is designed to remove water from the surface or pavement across the shoulder into the side drain (longitudinal ditch). The flow is intended to be in sheet form to minimise erosion of the road surface and the shoulders. It is therefore important to ensure control of a uniform camber both lateral and in longitudinal profile during construction. In many cases, where the longitudinal alignment grade is high, there is tendency for water to flow along the road, a condition that encourages erosion in unpaved or unsealed roads. Such conditions may require sealing steep sections. The recommended camber for LVRs with gravel or earth surface is 5%-7%. For sealed LVRs, 3% camber is recommended.

4.2 External drainage

4.2.1 Function of external drainage

An effective external drainage system must fulfill several functions:

- prevent or minimize the entry of surface water into the pavement;
- prevent or minimize the adverse effects of sub-surface water;
- remove water from the vicinity of the pavement as quickly as possible, and
- allow water to flow from one side of the road to the other.

This must be achieved without endangering the road or adjacent areas through increased erosion or risk of instability. Thus, an external drainage system consists of several complementary components:

- surface drainage to remove water from the road surface quickly;
- side drainage to direct water away from the road and prevent water from reaching the road;
- turnouts to take the water in the side drains away from the road;
- cross drainage to collect water from a watershed area and the side drains at a point where the road blocks the natural channel, allowing the water to cross the road;
- relief culverts to allow discharge across the road from a side drain flowing to capacity;
- sub-surface drains to cut off sub-surface water and to lower the water table when required;
- interceptor drains to collect surface water before it reaches the road; and
- erosion control (often simple scour checks) to slow down the water in the side drains and prevent erosion in the drains themselves and downstream of drainage outlets or crossings.

4.2.2 General principles of external drainage

Conservation of the natural drainage system around the road alignment is one of the most important concerns during design and construction. By effectively creating a barrier to natural surface drainage that is only punctuated at intervals by constructed drainage crossings, road construction can lead to significant local changes in water flows. The road construction will reduce the naturally available flood storage and increase the water level upstream of the road. Furthermore, road drainage reduces the time taken to reach maximum flow by shedding water from impermeable surfaces relatively quickly, particularly in the case of paved roads. The percentage of impermeable area will increase but the total catchment area will still be the same. Therefore, in addition to constructing a drainage system to convey the design runoff without surcharge, blockage by sediments, or scour, attention must be paid to strengthening those parts of the natural slope drainage system that experience increased runoff, and hence erosion potential, as a result of road construction. The main ways of doing this are to:

- control road surface drainage;
- design culverts or fords that convey water and debris load efficiently;
- optimize the frequency of drainage crossings to prevent excessive concentration of flow;
- protect drainage structures and stream channels for as far downstream as is necessary to ensure their safety and prevent erosion of land adjacent to the watercourse; and
- plant vegetation on all new slopes and poorly-vegetated areas, around the edges of drainage structures and along stream courses, without impairing their hydraulic efficiency or capacity.

4.3 Side drains (longitudinal ditches)

The side drains intercept water from the slopes and convey it to where it can be carried under the roadway or away from the road into the natural drainage of the surrounding environment. In some cases, they also serve to take water from subsurface drainage beneath the road and discharge it away from the road. Ditches may be shaped as V, trapezium and U-shaped. These shapes are illustrated in Figure C.4.1. It is advisable to limit the use of V-shaped drains to only when conditions do not permit the use of other types as they encourage erosion and have lower capacity.

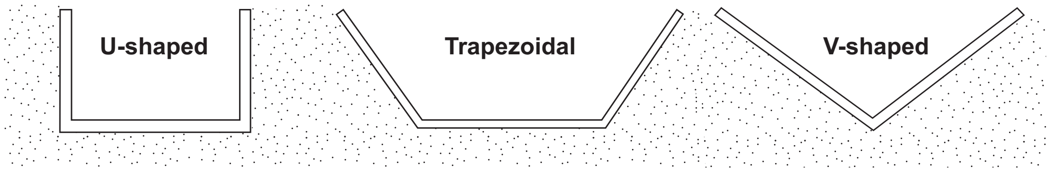


Figure C.4.1: Typical side drain cross-sections

Drains must be designed for capacity, shape, efficiency, and to avoid ponding, silting and erosion. The use of a trapezoidal cross-section facilitates maintenance by hand and is acceptable from the point of view of traffic safety. It is possible to dig and clean a trapezoidal drain with hand tools and the risk of erosion is lower. The minimum recommended width of the bottom of the side drain is 500 mm. The trapezoidal shape carries a high flow capacity and, by carefully selecting the gradients of its side slopes, it will resist erosion. The rectangular shaped drain requires less space but needs to be lined with rock, brick or stone masonry, or concrete to maintain its shape. The rectangular shaped drain is more hazardous to a vehicle leaving the carriageway.

In very flat terrain and reasonable soils (soils not highly susceptible to erosion) it may be best to use wide unlined "meadow drains". These are shallow and continuous depressions in the surface that avoid abrupt changes in surface profile. When properly designed, their capacity is high, and the flow velocity is low so that erosion is controlled but siltation does not occur.

Figure C.4.2: Concrete lined trapezoidal drain

Figure C.4.3 Concrete lined U-drain

4.4 Turn-outs (mitre drains)

Turn-outs are also referred to as off-shoots or mitre drains. They are designed to take water from the side drain into the natural drainage of the surrounding environment. Turn-outs are designed in the same way as the side drains.

The minimum width of turn-outs should be 0.60 m and the cross-section should have at least the same capacity as the side drain. When possible, turn-outs should have a steeper slope than the side drains. Turn-outs should be constructed at an angle to the road center-line and be provided at frequent intervals. The angle between the turn-out and the side drain should never be greater than 45 degrees. An angle of 30 degrees is ideal. Some excavated soil should be used to block the downhill side of the drain to ensure water flows into turn-out. Alternatively, when the side drain is constructed, gaps should be left in the drain at the location of the turn-out.

Figure C.4.4: Turn-out on an LVR

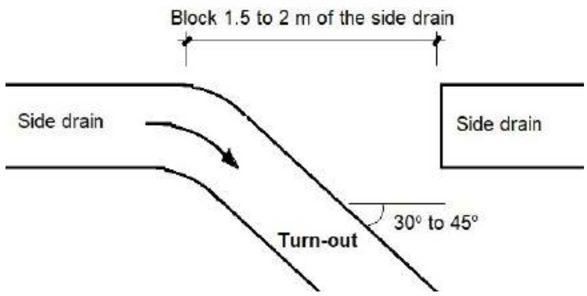


Figure C.4.5: Turn-out alignment

The maximum spacing for turn-outs is given in Table C.4.1. This ensures that the quantity and velocity of water being discharged at each outlet is small and does not cause erosion damage to the drainage system or the adjoining land.

Table C.4.1: Turn-out spacing

Side drain gradient, S (%)	Spacing (m)
1 or less	50
1 - 2	40
2 -5	25
5 - 10	15
more than 10	10

Source: TRL Overseas Road Note 2

4.5 Catch water drains (cut-off ditches/drains)

Catch water drains are usually on the hill side of the road or cut slope to intercept surface runoff from the hill side and transfer it to the natural drains on the milder slopes to prevent erosion that would be caused by permitting the runoff to spill on the road or cut surfaces. The design of catch water drains is carried out in the same way as open channel design. Further details of catch water drains are given in Chapter 7.

4.6 Cross drainage structures

Culverts are conduits beneath the road surface that transfer runoff water from one side of the road way to the other. Culverts can be used to take runoff water from the side drain (relief culvert) or where the road intercepts a natural watercourse (watercourse culvert). Culverts can also be used to carry surface runoff in the side drains at junctions or property accesses (access culverts). Culverts and bridges have similar requirements in design, but in general when the span is less than 6 m the structure is considered a culvert and when the span is more than 6 m, it is considered a bridge. When the depth of the structure is less than 4 m it is considered a culvert, and where the depth is greater than 4 m it is considered a bridge.

The culvert barrel can be in various sizes or shapes (elliptical, circular, square, rectangular, u-shape or semi-circular). The most common culvert types are the circular reinforced concrete pipe and box culverts. U-culverts are used as relief culverts as they require less depth than circular culverts. U-culverts require good carpentry skills for forming the shape and take more time in construction than the same size of pre-fabricated pipe culvert. However, when properly constructed, U-culverts can offer better hydraulic results than the same size of pipe.

Basic guidelines for the use of culverts include:

1. Culverts channel water under the road, avoiding the need for vehicles to drive through the watercourse.
2. Culverts are the most commonly used structures on LVRs. They can vary in number from about one per km in dry areas and gently rolling terrain up to six or more per km in hilly or mountainous terrain with high rainfall. In flat areas with high rainfall, the frequency may increase to allow water to cross the road alignment in manageable quantities.
3. In addition to well defined water crossing points, culverts should normally be located at low points or sags in the road alignment.
4. Culvert alignment should follow the direction of the watercourse.
5. Culvert invert levels should be approximately in line with the water flow in the stream bed, otherwise drop inlet and/or long outfall excavations may be required.
6. The gradient of the culvert invert should be between 2% and 5%. Shallower gradients may result in silting whereas steeper gradients result in scour.
7. The minimum culvert diameter recommended is 900 mm. Cross culverts smaller than 900 mm in diameter should not be installed as they are difficult to clean. U-culverts or box culverts should be used where there is insufficient depth for a 900 mm diameter pipe culvert. The top slab of the U-culvert or box culvert can be designed to carry direct traffic loads.
8. Culverts should be placed on a good foundation material to prevent settlement that may cause damage. On soft ground, it is necessary to use concrete to ensure adequate foundations.
9. Headwalls are required at the inlet and outlet to direct the water in and out of the culvert and prevent the road embankment sliding into the watercourse. Wingwalls at the ends of the headwall may be used to direct the water flow and retain the road formation.
10. Aprons with buried cut off walls are required at the inlet and outlet to prevent water seepage, scouring and undercutting.
11. Culverts concentrate the flow at the inlet and outlet, so attention is needed to the design of erosion protection works. The watercourse must be protected from erosion downstream of the structure.
12. Culverts are generally provided where the ground level is low, and the road level is high, and therefore they do not affect the road profile. Culverts are relatively safe and allow a conformable ride.
13. Culverts can exist in multiples to enable larger stream flows to be accommodated using standard unit designs.
14. Common pipe culverts are generally less expensive than bridges or drifts.
15. Culverts require more routine maintenance than drifts.
16. Drifts should be used in preference to culverts when silt supply is high.
17. Culverts may be difficult to use where there is rock, and excavation is difficult. Such places are more suited to the use of drifts.
18. The choice of a culvert type and material depends on hydraulic performance for the specific site, and the economics and availability of different culvert types or materials.

The structural design of culverts requires an assessment of the loading that will be applied to the culvert in terms of earth and traffic loading. The height of fill (H) may determine the structural details of the culvert. The minimum cover to a pipe culvert should be 0.60 m to dissipate the stresses from the traffic loads to allowable limits. When the minimum cover is not achieved, a 0.15 m concrete surround should be provided to carry the traffic load and avoid damage of the pipe culvert. Details of concrete culvert pipes with mass concrete surround and compacted gravel bedding are given in Figure C.4.6 and Figure C.4.7. Damage to a culvert due to inadequate cover is shown in Figure C.4.8.

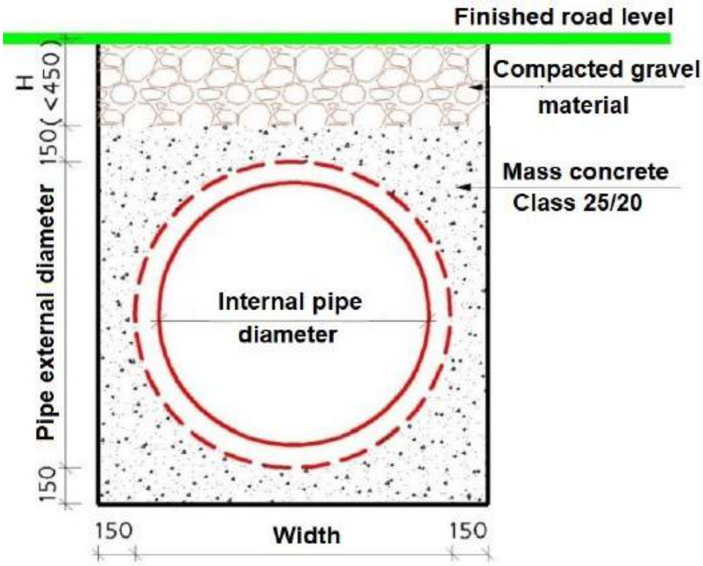


Figure C.4.6: Typical section of pipe culvert with concrete surround

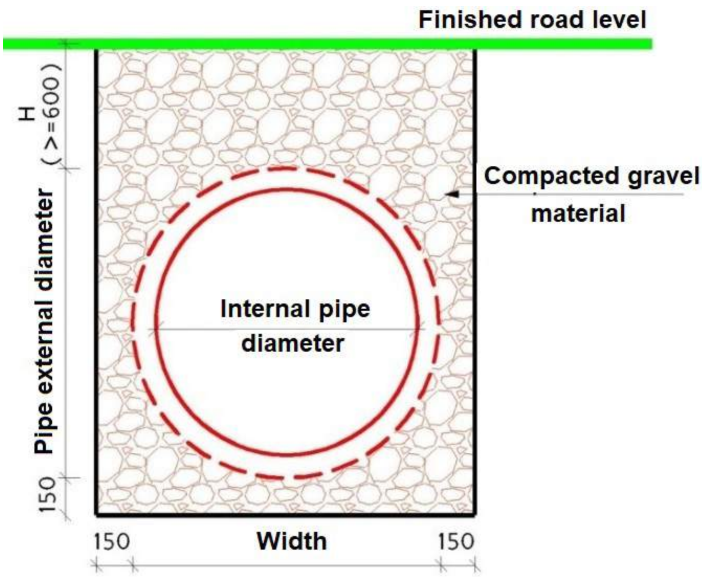


Figure C.4.7: Typical section of pipe culvert with gravel bedding

Figure C.4.8: Damage to pipe culvert due to inadequate cover

Cross drains (relief culverts) are required where it is not possible to place turn-out drains at the permitted maximum spacing to remove water from the side drain (see Table C.4.1). The maximum spacing of cross drains (with no turn-out drains) is given in Table C.4.2. If it is not possible to meet this requirement, erosion control measures such as drain lining are required.

Table C.4.2: Recommended spacing between standard cross drains

Longitudinal gradient of road and drain (%)	Recommended interval of cross drainage (m)
2	200
3-4	150
5	135
6	120
7-8	100
9-10	80
11-12	60

For relief culverts or access culverts, a minimum diameter of 600 mm pipe culvert may be used. However, a 700 x 900 U-culvert is preferred as it is easier to clean. The most important aspect in the design of relief culverts is the frequency along the road alignment and the slope of the culvert (which should be a minimum of 2% to ensure self-cleaning).

Figure C.4.9: Typical U-culvert

Figure C.4.10: Typical access culvert at a junction

4.7 Low-level water structures

4.7.1 General

This category of structures exists where conditions may not be suitable for the use of culverts or bridges. The design of low-level water structures involves similar hydrologic analysis and hydraulic designs to culverts and bridges. However, drifts and vented fords (pipe drifts, causeways) permit water to flow over the road surface.

The geometric dimensions of a drift depend on the volume of water expected to cross the road and the geometry of the intended crossing place. Typical arrangements for drifts and vented drifts are shown in Figure C.4.11 and Figure C.4.12.

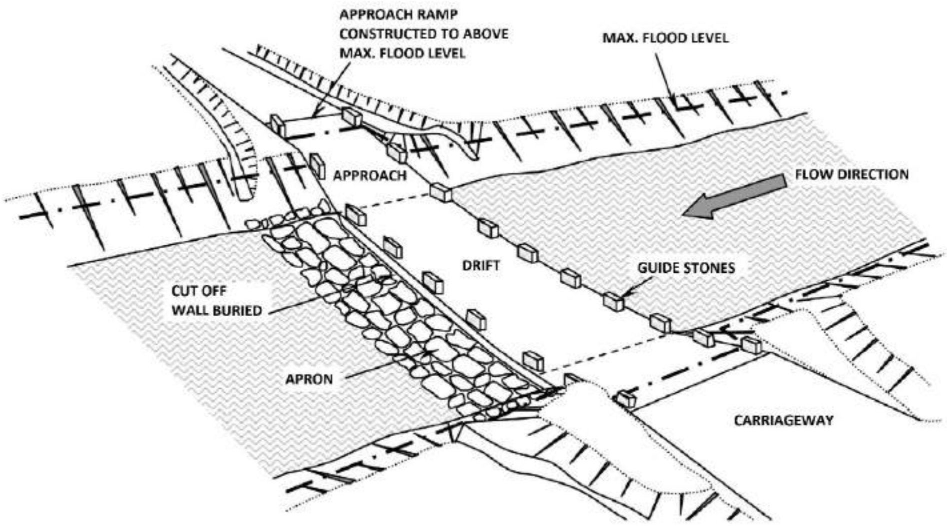


Figure C.4.11: Typical layout of a drift

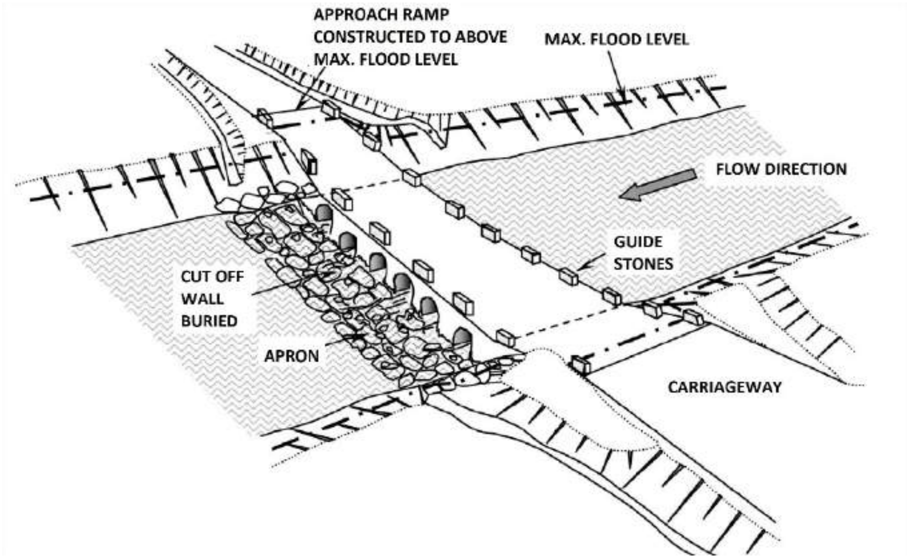


Figure C.4.12: Typical layout of a vented drift

Figure C.4.13: Large vented drift

Figure C.4.14: Simple drift using hand packed stone

4.7.2 Drifts

The basic considerations for use of drifts are:

- Stream drifts are structures which provide a firm place to cross a river or stream. Relief drifts transfer water across a road without erosion of the road surface. Water flows permanently or intermittently over a drift; therefore, vehicles are required to drive through the water in times of flow.
- Drifts are particularly useful in areas that are normally dry with occasional heavy rain causing short periods of flood water flow.
- Drifts provide a cost effective method for crossing wide rivers which are dry for most of the year or have very slow or low permanent flows.
- Drifts are particularly suited to areas where material is difficult to excavate, thus making culverts difficult to construct.
- Drifts are suited in flat areas where culverts cannot be buried because of lack of gradient.
- Drift efficiency is directly related to the drift invert slope and cross sectional area.
- Drifts have low maintenance requirements.
- The drift approaches must extend above the maximum design flood level flow to prevent erosion of the road material.
- If necessary, guide stones should be provided on the downstream side of the drift and be visible above the water when it is safe for vehicles to cross the drift.
- Buried cut off walls are required upstream and downstream of the drift to prevent under cutting by water flow or seepage.
- Approach ramps should be surfaced with a non-erodible material and provided to the drift in the bottom of the watercourse with a maximum gradient of 10% (7% for roads with large numbers of heavy trucks).
- Drifts should not be located near or at a bend in the river.
- Some form of protection is usually required downstream of a drift to prevent erosion, but drifts generally cause less erosion from their discharge than culverts.
- Drifts force vehicles to slow down resulting in longer journey times.

In general, drifts should be used where the following conditions apply:

- The difference in elevation between the invert of the side drain and/or natural watercourse and the roadway shoulder break point is less than 300 mm - Where the water level is estimated to exceed 200 mm, the approaches must be lengthened to accommodate the high water level.
- The subgrade material is rocky and difficult to excavate.
- There is evidence that the natural soils of the side drain and/or watercourse comprise mainly silt and could lead to the rapid blocking of a culvert.
- Where discharge, if concentrated, may lead to erosion of agricultural land.
- Where the cost of a culvert of similar capacity is significantly higher than the cost of a drift.

4.7.3 Vented fords and causeways

The key features of vented fords and causeways are:

- These structures are designed to pass the normal dry weather flow of the river through pipes below the road. Occasional larger floods pass through the pipes and over the road, which may make the road impassable for short periods of time.
- The level of the road on the vented ford should be high enough to prevent overtopping except at times of peak flows (There should be enough pipes to accommodate the normal flow). The location of pipes in the drift depends on the flow characteristics of the river.
- Vented fords should be built across the whole width of the water-course.
- A vented ford requires approach ramps, which must be surfaced with a non-erodible material and extend above the maximum flood level.
- The approach ramps should not have a steeper grade than 10% (7% where there is significant heavy vehicle traffic).
- Watercourse bank protection is required to prevent erosion around the structure.

- The upstream and downstream faces of a vented drift require buried cut-off walls (preferably down to rock) to prevent water undercutting or seeping under the structure.
- An apron downstream of the pipes and area of overtopping is required to prevent scour by the water flowing out of the pipe culvert or over the structure.
- The watercourse downstream from the structure must be protected from erosion. There will be considerable turbulence immediately downstream of the structure in flood conditions.
- The longitudinal alignment of the vented ford should be a slight sag curve to ensure that water flows across the center of the vented drift and not along it at the start and end of overtopping.
- Construction materials for vented fords can be riprap, gabions, and reinforced concrete.
- There should be guide stones on each side of the structure to mark the edge of the carriageway and indicate when the water is too deep for vehicles to cross safely. Guide stones should be painted with reflective paints to make them visible both day and night for road users and should be 0.6 m high.
- At locations where there is a high risk of floating timber or other debris causing damage to or blockage of a vented ford, consideration should be given to including upstream guide slides that help direct such debris safely up and over the main structure.

4.8 Small bridges

There are several different elements to a simply supported deck bridge. These are a superstructure (comprising deck, parapets, guide stones and other road furniture) and substructure (comprising abutments, wingwalls, foundations, piers and cut off walls). Bridges can be single span or multi span, with several openings for water flow and intermediate piers to support the superstructure. Key considerations for bridges are:

- Bridges are generally the most expensive type of road structure, requiring specialist engineering advice and technically approved designs.
- Bridges should not significantly affect the flow of water (i.e. the openings must be large enough to prevent water backing up and flooding or over-topping the bridge).
- The main structure should always be above flood level, so the road is always passable.
- Abutments are needed to support the superstructure and retain the soil of the approach embankments. Wingwalls provide support and protect the road embankment from erosion.
- Embankments must be carefully compacted behind the abutment to prevent soil settlement which would result in a step on the road surface at the start and end of the bridge.
- Weep holes are needed in the abutment to allow water to drain out from the embankment and avoid a build-up of ground water pressure behind the abutment.
- The shape of the abutments and piers affects the volume of flow through the structure and the amount of scouring.
- Bridges require carefully designed foundations to ensure that the supports do not settle or become eroded by the water flow. On softer ground this may require piled foundations.
- Water from the roadside drains should be channelled into the watercourse at the bridge site or erosion of the bank or scour of the abutment structure may occur.
- Guide stones or curbs should be placed at the edge of the carriageway for vehicle safety.
- If the crossing is to be used by pedestrians, proper protected footways should be provided on both sides of the carriageway.
- Reinforced concrete parapets are preferred to steel guard rails. They should be flared away at the ends and ramped for safety reasons. Warning or guard posts should be provided on the bridge approach because vehicles need to slow down for safety.
- Consideration should be given to the safety of pedestrians and other non-motorized traffic using the bridge at night. when levels of mixed traffic are high (even for short periods) and/or the bridge is long, this may necessitate the provision of a mid-span refuge area for pedestrians.

4.9 Chutes and stilling basins

Chutes are channels that are designed to carry surface runoff down a slope face and discharge the water to a stable outlet area without causing erosion. Chutes may be constructed of rock, concrete or half-round pipe. Chutes can convey runoff from diversion dikes, infiltration trenches, slope steps, benches, or other runoff control facilities. Chutes discharge into a stabilized watercourse, sediment trap, or stabilized area.

Chutes may be used on slopes 2H:1V or flatter to carry water down the face of erodible slopes, usually from runoff collection structures at the top to stable discharge areas at the bottom. Chutes are permanent structures that are effective in many situations where concentrated runoff would otherwise cause slope erosion.

Chutes must be placed on undisturbed soil or well-compacted fill. Energy dissipaters within the chute at the outlet end should be provided to protect against scour when necessary. The slopes of the energy dissipater should be no steeper than 2H:1V.

A stilling basin is a hydraulic structure or excavation at the foot of a chute to reduce the energy of the descending runoff. This reduces the danger of scour at the toe of the chute or riverbed, which can affect the stability of the structure.

4.10 Drainage in flat terrain

In flat terrain where obtaining minimum slopes may not be possible and where water flow at the outlet of a culvert may be constrained by downstream flow restrictions, considerably more care is needed to ensure sufficient flow to minimize siltation. Usually it is sufficient to ensure that the slope of the culvert is not less than 1% or, if it is greater, equal to the slope of the watercourse itself. Some engineering work may be required to ensure the downstream flow is not restricted.

In flat terrain that is liable to seasonal flooding, the road will usually be on an embankment and culverts are required to allow cross flow when the flood water ebbs or flows. Under these circumstances the flow can be relatively slow provided that enough culverts are available, but insufficient culverts can lead to rapid flow along the side of the embankment and consequent scouring. The best method of estimating this is by asking the local people how long the water usually takes to dissipate from peak flood condition after the rain.

Figure C.4.15: Poor drainage on flat ground

5. HYDRAULIC ANALYSIS

5.1 Introduction

Hydraulic analysis is carried out to estimate the size of drainage structure that will pass the calculated peak or maximum flow for a particular catchment. Guidance is provided in this chapter on the sizing of drains, culverts, drifts and small bridges.

5.2 Side drains

5.2.1 General requirements

The capacity of a side drain is estimated using the Manning's formula. The important dimensions needed are the depth and width. Using the geometry of the cross section of the drain (triangle, trapezium etc.), the relationship for depth and width can be established. The invert of a side drain should not be less than 300 mm below the shoulder break point. The width of the drain depends on the road class, the expected runoff, and conditions of the subgrade material. The side and back slopes are determined by soil conditions and cost of construction. Side slopes range from 1: 1.5 to 1:6 (see Part A of the Manual).

In designing the drain, the limiting values of velocity must be considered to prevent erosion or siltation of the drain. Side drain slopes should not be less than 1 in 200 or 0.5% in order to prevent ponding or silting.

5.2.2 Manual approach

The flow capacities of side drains can be determined from the expression:

$$Q = VA$$

Where,

Q = Discharge in m^3/s

V = Mean Velocity in m/s

A = Flow Area in m^2

The mean velocity "V" is obtained from the Manning's Equation:

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

Where,

V = Mean Velocity in m/s

R = Hydraulic Radius in m

S = Water surface Slope in m/m

n = Manning's Roughness Coefficient

$R = A/P$

Where,

A = Flow Area in m^2

P = Wetted Perimeter in m, measured at right angles to the direction of flow.

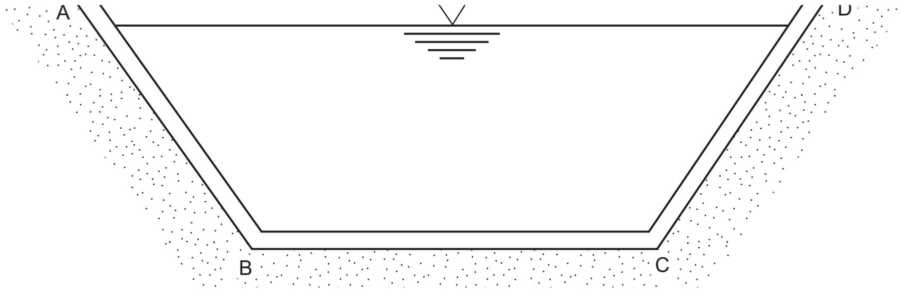


Figure C.5.1: Wetted Perimeter

Wetted Perimeter, P = AB + BC + CD i.e. the face of the channel that is covered with water as shown in Figure C.5.1.

Hydraulic Radius, "R" is a shape factor that depends on the channel, river or culvert dimensions and the shape of the flow.

Manning's Roughness Coefficient, "n" is based on field observations, survey data and engineering judgement and should be applied over a sufficient distance to establish uniform flow. It depends on the carrying capacity of the drainage structure material (see Table C.1.7).

Water Surface Slope, "S" represents the loss in head by a drop in the gradient.

Mean Velocity, "V". Mean velocities for different surfaces are shown in Table C.5.1.

Table C.5.1: Mean Velocity

Surface type	Mean velocity of flow (m/s)
Cement concrete	0.6 - 3.0
Asphalt concrete	0.6 - 1.5
Stone or block pitching	0. 6 - 1.8
Hard gravel or clay Coarse grained soil	0. 6 - 1.0
Gravelly sandy soil	0.3 – 0.6
Sand or sandy soil with a considerable large clay content	0.2 – 0.3
Sand or silt	0.1 - 0.2

Source: Ghana Highway Authority Road Design Guide, March 1991, Page 78.

Figure C.5.2 is a flow chart of the process of determining the flow capacity of a side drain.

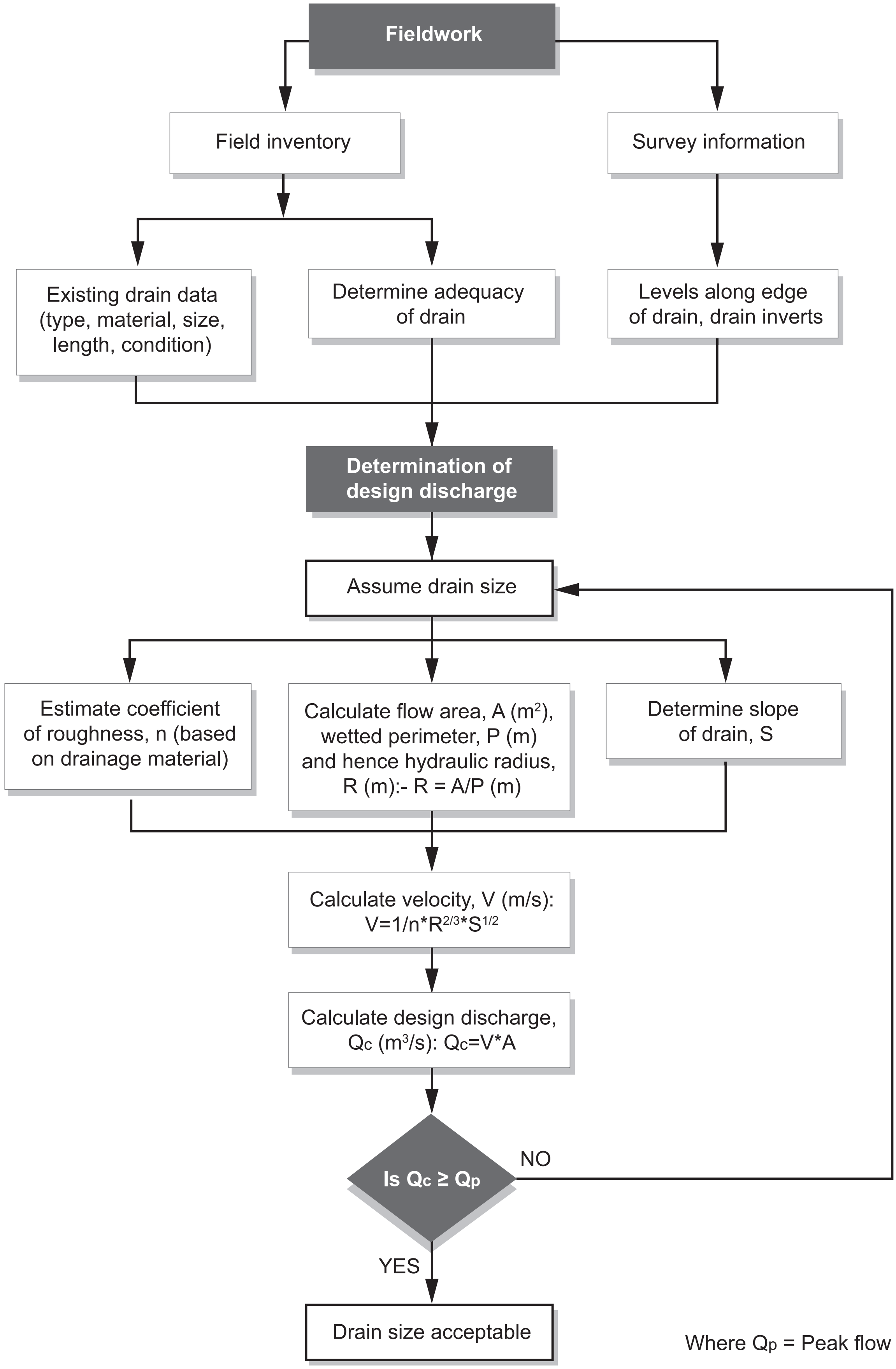


Figure C.5.2: Flow chart for estimation of flow capacities of side drains

5.2.3 Computer based approach

The flow capacities of side drains can be determined using the computer based software, HY-22 for open channel analysis. The process is summarised as follows:

Select channel type:

Rectangular, circular, trapezoidal.

Input data:

Enter items 3 and 4 and any other four items.

1. Channel slope, m/m
2. Channel Bottom Width, m
3. Left Side Slope (horizontal to 1)
4. Right Side Slope (horizontal to 1)
5. Manning's Coefficient
6. Discharge, m^3/s
7. Depth, m

➢ **Click on Calculate**

Output data:

1. Cross section area, m^2
2. Average Velocity, m/s
3. Top Width, m
4. Hydraulic Radius, m
5. Froude Number.

5.3 Culverts

5.3.1 Overview

The hydraulic performance and hence design of culverts may often be complex and depends on flow characteristics. There are different methods of estimating the size of culverts that will pass the design flow. In flat terrain, where there is a high risk of silting, a factor of safety of 2 should be allowed in the design of the culvert.

5.3.2 Use of hand calculations and nomographs

A simple and systematic procedure used for selection of a culvert size is the use of culvert nomographs. The culvert opening is estimated using the nomographs for:

- concrete pipes, and
- concrete box culverts.

A simple flow chart for estimating the size of culverts is as shown in Figure C.5.3. This approach uses a trial and error method as well as the nomographs for inlet control computations.

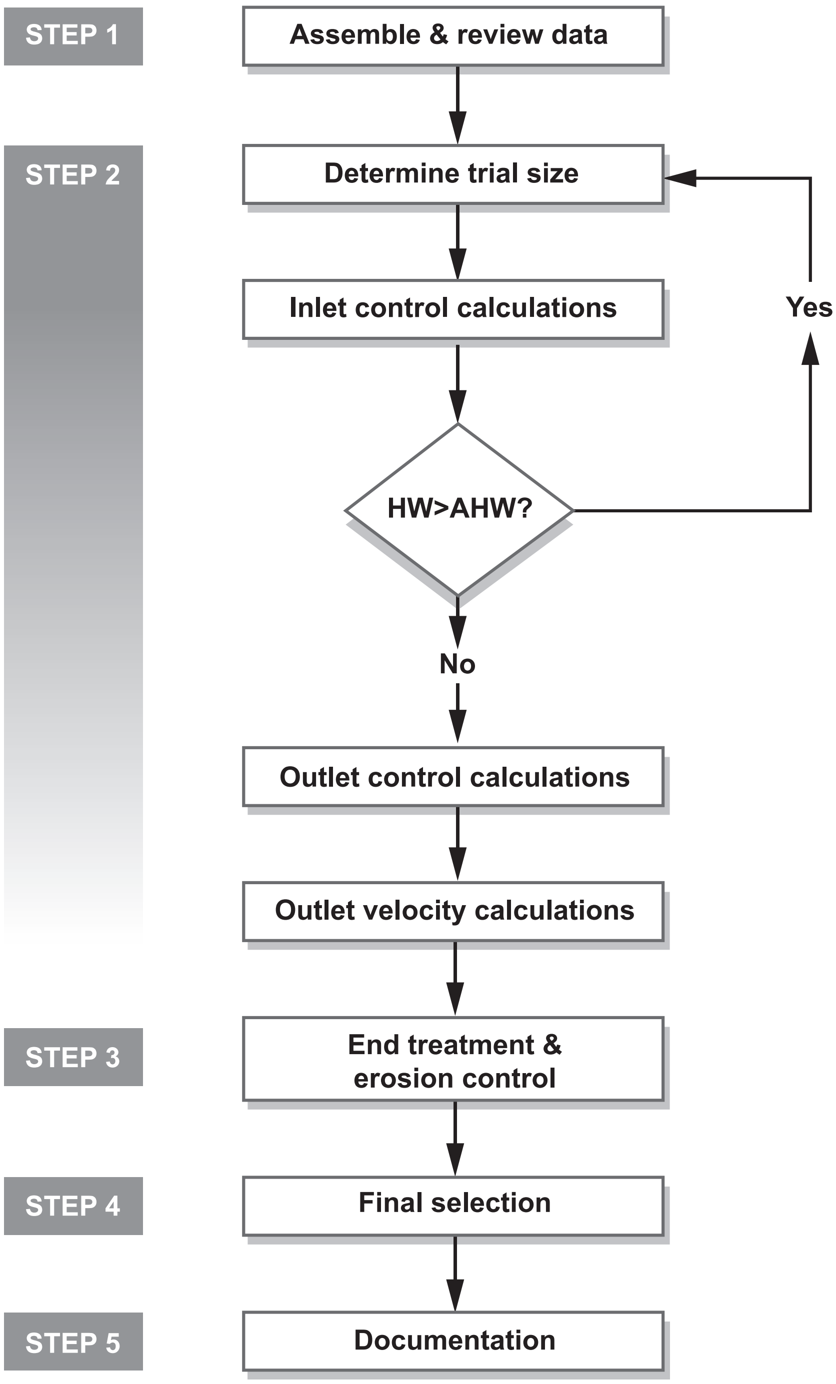


Figure C.5.3: Flow chart for estimation of flow capacities of culverts

The steps in the flow chart are explained as follows:

STEP 1: Assemble and review data

- From field notes determine the existing culvert sizes, High Water level (HWL), etc.
- Obtain the plan, profile and cross sections of the road and stream.
- Review the position of the culvert on aerial photographs and Google Earth.

Enter existing culvert data in columns 9 to 13 of the hand calculation forms as shown below (See Appendix C.3 for full hand calculation form).

Station	Design data							Culvert data				
	Q m^3/s	d m	d_e m	AHW m	Skew No.	L m	S m/m	Description	D or B x D m	N	Q/N m^3/s	A m^2
1	2	3	4	5	6	7	8	9	10	11	12	13

STEP 2: Culvert selection

Design data

Col. 1: Enter culvert station/location

Col. 2: Enter design discharge, Q (m^3/s)

Col. 3: Enter flood depth in natural channel, d, usually based on field observation

Col. 4: Enter depth of culvert invert below streambed, d_e

Col. 5: Allowable Headwater Depth (AHW) = Col. 3 + Col. 4 + Allowable Backwater Depth (ABW)

Col. 6: Skew Number

Col. 7: Culvert length, allowing for skew (if any)

Col. 8: Culvert slope

Culvert data

Col. 9: Description of trial culvert

Col. 10: Enter trial size of culvert [diameter of pipe culvert (D) or B x D for rectangular culvert]

Col. 11: Number of barrels, N

Col. 12: Enter Col. 2 ÷ Col. 11

Col. 13: Area, A per barrel

INLET CONTROL			OUTLET CONTROL									
Q/NB $m^3/s/m$	HW/D	***HW m***	K_e	H m	d_c m	$(d_c+D)/2$ m	TW m	h_o m	LS m	***HW m***	GOVG HW m	VEL V_o m/s
14	15	16	17	18	19	20	21	22	23	24	25	26

Inlet control

Col. 14: Enter Q/NB for box culvert only

Col. 15: HW/D from inlet control charts (Figure C.5.6 and Figure C.5.7).

Col. 16: HW for inlet control = D x Column 15. If HW is greater or significantly less than AHW, try other size until HW is acceptable before going on to outlet control calculation.

Outlet control

Col. 17: Entrance loss coefficient from Table C.5.2.

Col. 18: Head, H obtained from Figures C.5.8 and C.5.9.

Col. 19: Critical depth dc, from Figure C.5.10 and Figure C.5.11.

Col. 20: Calculate and enter (dc+D)/2.

Col. 21: Tailwater depth TW = Col. 3 + Col. 4.

Col. 22: h_o = the larger of $(d_c+D)/2$ and TW i.e. larger of Col. 20 and Col. 21.

Col. 23: LS = culvert length x culvert slope i.e. Col. 7 x Col. 8.

Col. 24: Headwater depth, HW = Col. 18 + Col. 22 – Col. 23. If HW is negative, enter zero.

Controlling HW

Col. 25 Enter governing HW i.e. larger of Col. 16 and Col. 24.

If outlet control governs and HW > AHW, repeat from Col.10 with larger sizes until HW is equal to or slightly less than AHW.

If HW is equal to or slightly less than AHW, size is acceptable. Go to next step.

Outlet velocity

Col. 26 If downstream conditions are such that the culvert outflow may cause a significant erosion or sedimentation problem, calculate V_o for inlet or outlet control, whichever is governing, and enter in Col. 26.

STEP 3: Special erosion control measures

Special erosion control measures are required if the field investigation indicates the channel is degrading or there is a risk of significant environmental or other damage due to erosion or sedimentation caused by the culvert outflow velocity.

STEP 4: Final culvert selection

- Repeat the procedure for alternative culvert types, shapes, materials or number of barrels as required.
- Select the optimum design based on cost and other practical considerations.
- Check that the design is consistent with existing culverts of known adequacy in the project area, considering differences in hydraulic conditions.

STEP 5: Documentation

- File the information gathered for future reference. The amount of information and details should be commensurate with the importance of the culvert and potential damage claims.

The detailed steps involved in the estimation of size of culverts and flow velocity are summarized in the flow charts in Figure C.5.4 and Figure C.5.5. The approach uses a trial and error method as well as the nomographs for inlet control computations. A form for the hand calculation is included in Appendix C.3.

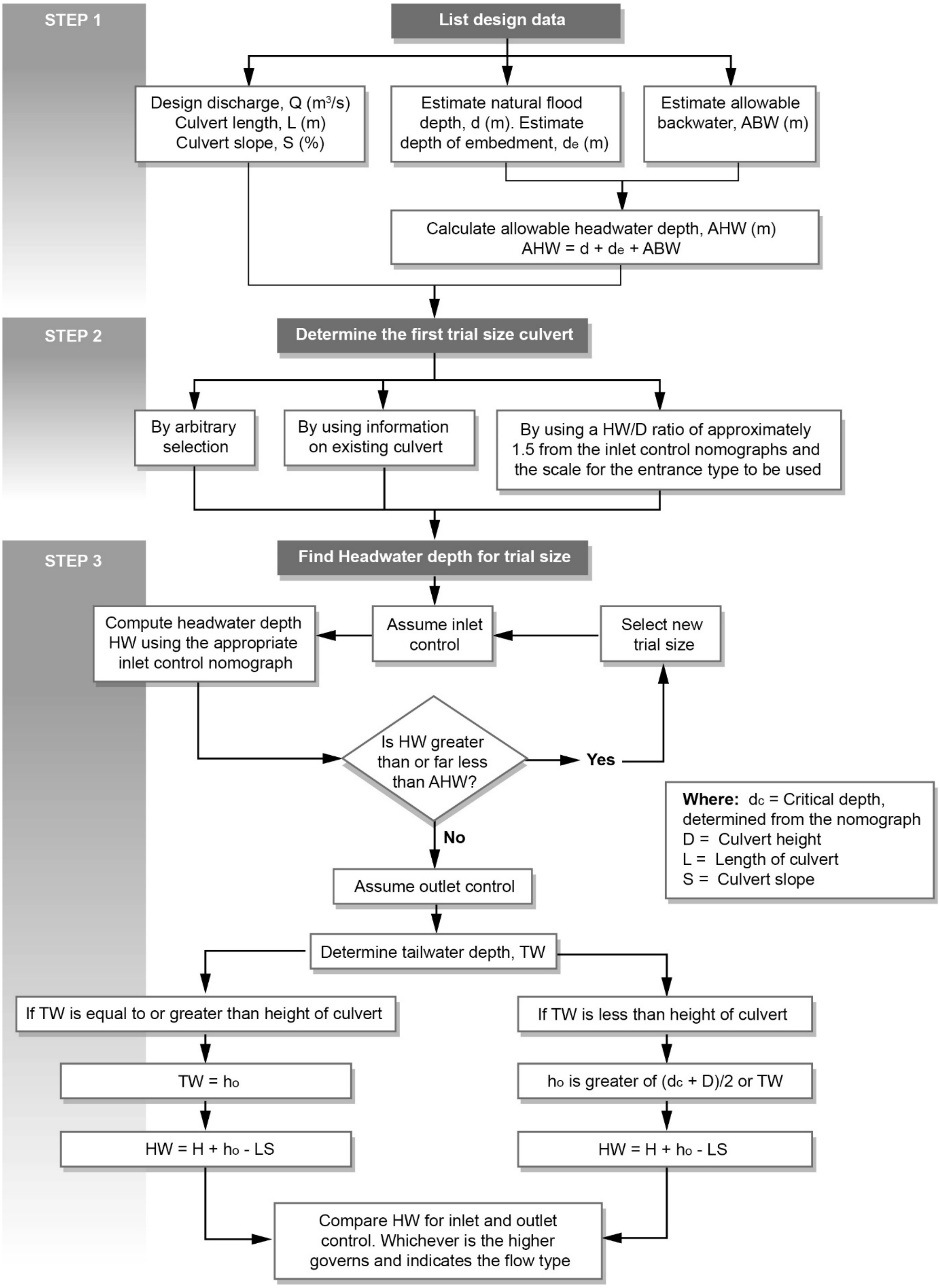
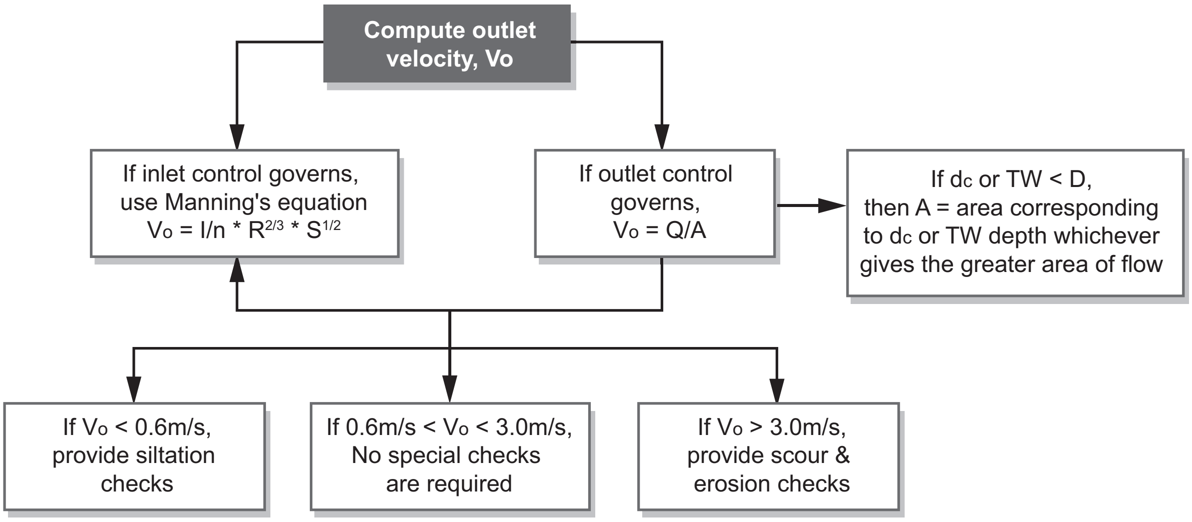


Figure C.5.4: Flow chart for estimation of flow capacities of culverts



Where n = Manning's Roughness Coefficient
S= Culvert Slope
R= Hydraulic Radius

Figure C.5.5: Flow chart for estimation of flow velocity

Figure C.5.6 is used to determine headwater depth (HW) of a pipe culvert. The steps are as follows:

i. Determine the pipe diameter (D) and design discharge (Q) on the scales.
ii. Join the values of D and Q with a straight line. Extend the line to meet scale (1).
iii. From the point on scale (1) draw a horizontal line to meet scale (2) and scale (3).
iv. Depending on the entrance type as described on the chart, the value of H_e/D is equal to the value on either scale (1) or scale (2) or scale (3).
v. H_e which is the headwater depth (HW) is equal to D multiplied by the value on the scale line.

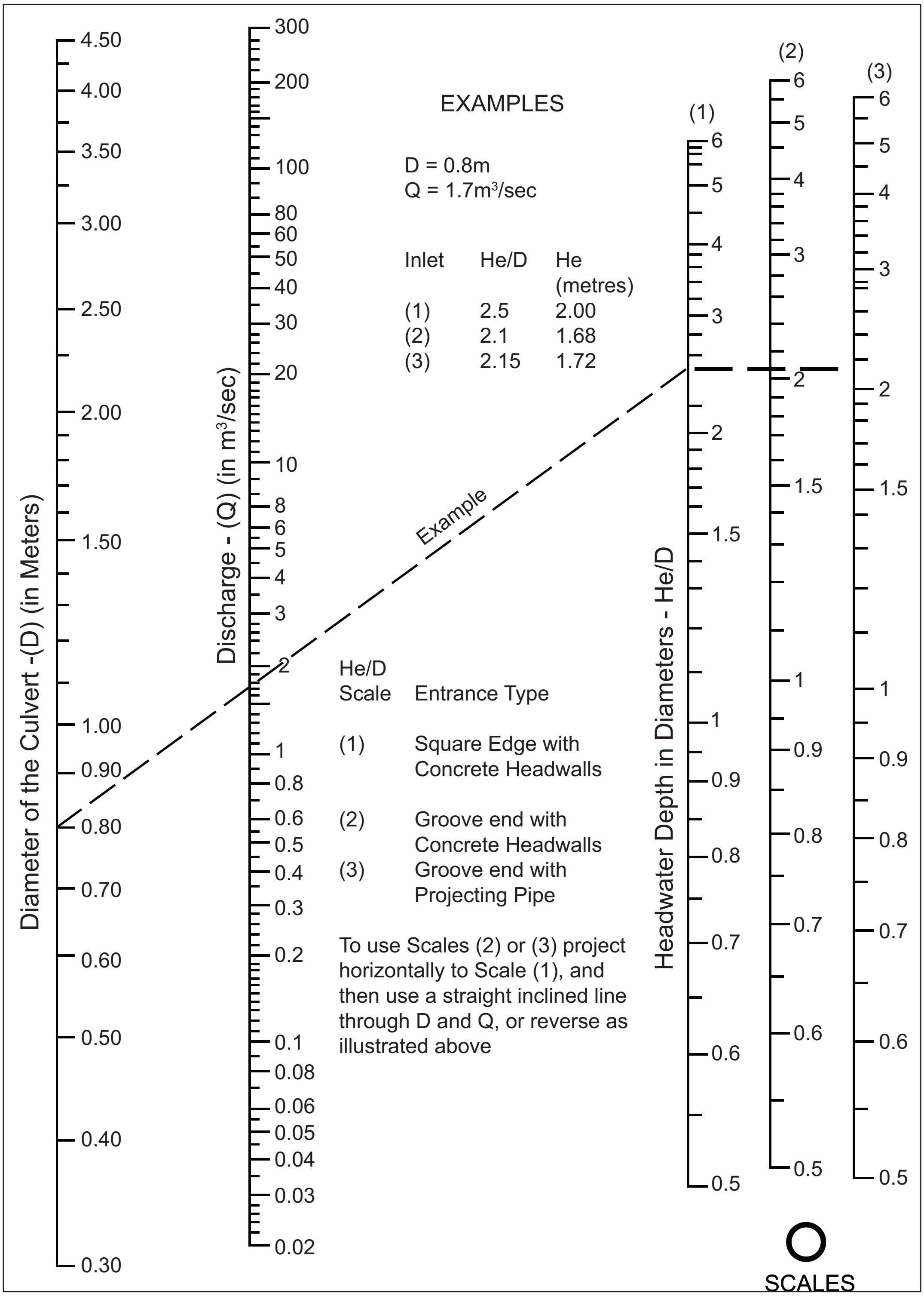


Figure C.5.6: Headwater depth and capacity for concrete pipe culverts with inlet control

(Adapted from Chart 1A, FHWA, 2012)

Figure C.5.7 is used to determine headwater depth (HW) of box culverts. The steps are as follows:

i. Determine the height of box (D) and ratio of design discharge (Q) to width (B) of box i.e. Q/B.
ii. Join the values of D and Q/B with a straight line. Extend the line to meet scale (1).
iii. From the point on scale (1) draw a horizontal line to meet scale (2) and scale (3). Depending on the wingwall type as described on the chart.
iv. The value of H_e/D is equal to the value on either scale (1) or scale (2) or scale (3).
v. H_e which is the headwater depth (HW) is equal to D multiplied by the value on the scale line.

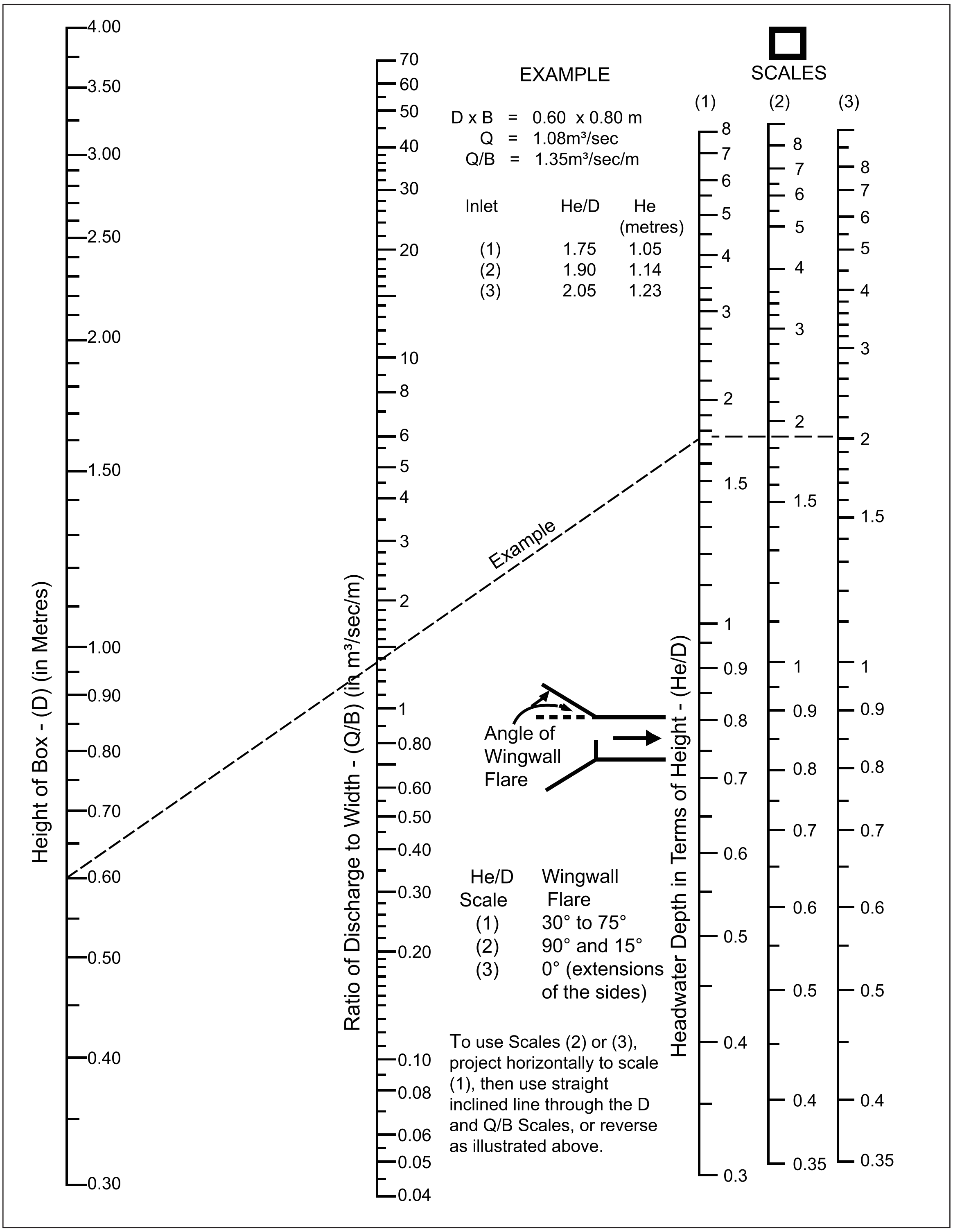


Figure C.5.7: Headwater depth and capacity for concrete box culverts with inlet control

(Adapted from Chart 8A, FHWA, 2012)

Figure C.5.8 is used to determine head (H) of pipe culverts flowing full. The steps are as follows:

i. Determine the length (L) of culvert on the scale depending on the entrance loss Ke and diameter (D) of the pipe culvert.

ii. Join the two points with a straight line to cross the turning line.

iii. Determine the design discharge (Q) on the scale at the extreme left. Join the discharge and the point of crossing on the turning line with a straight line. Extend the line to meet the head scale on the extreme right

iv. Determine the value on the head scale which is equal to the head (H) of pipe culverts flowing full.

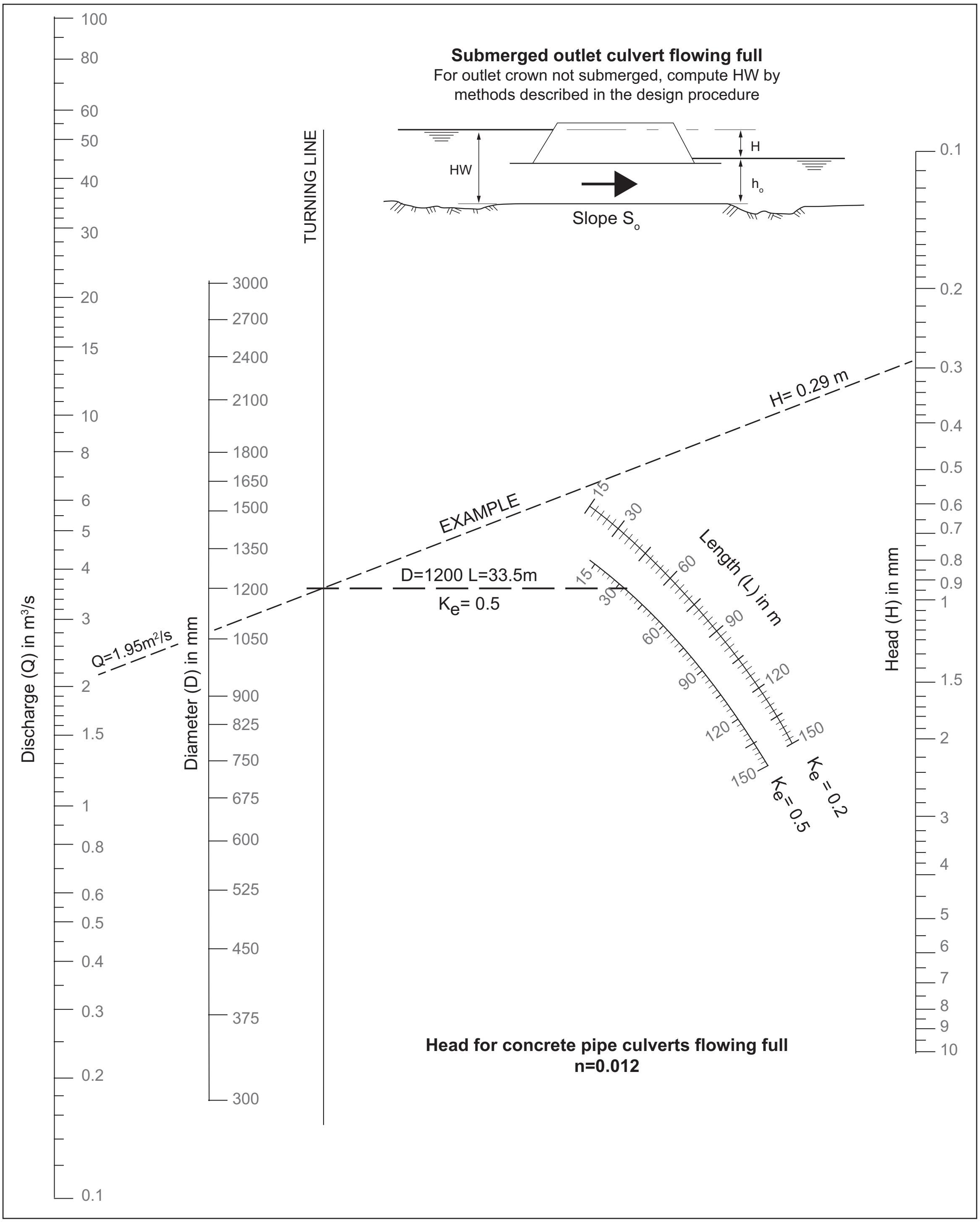


Figure C.5.8: Head for concrete pipe flowing full, n = 0.012

(Adapted from Chart 5A, FHWA, 2012)

Figure C.5.9 is used to determine head (H) of box culverts flowing full. The steps are as follows:

i. Determine the length (L) of culvert on the scale depending on the entrance loss K_e
ii. Determine cross-sectional area or dimensions of the box culvert.
iii. Join the two points with a straight line to cross the turning line.
iv. Determine the design discharge (Q).
v. Join the discharge point on the scale (extreme left) and the point of crossing on the turning line with a straight line.
vi. Extend the line to meet the head scale on the extreme right and determine the value which is equal to the head (H) of pipe culverts flowing full.

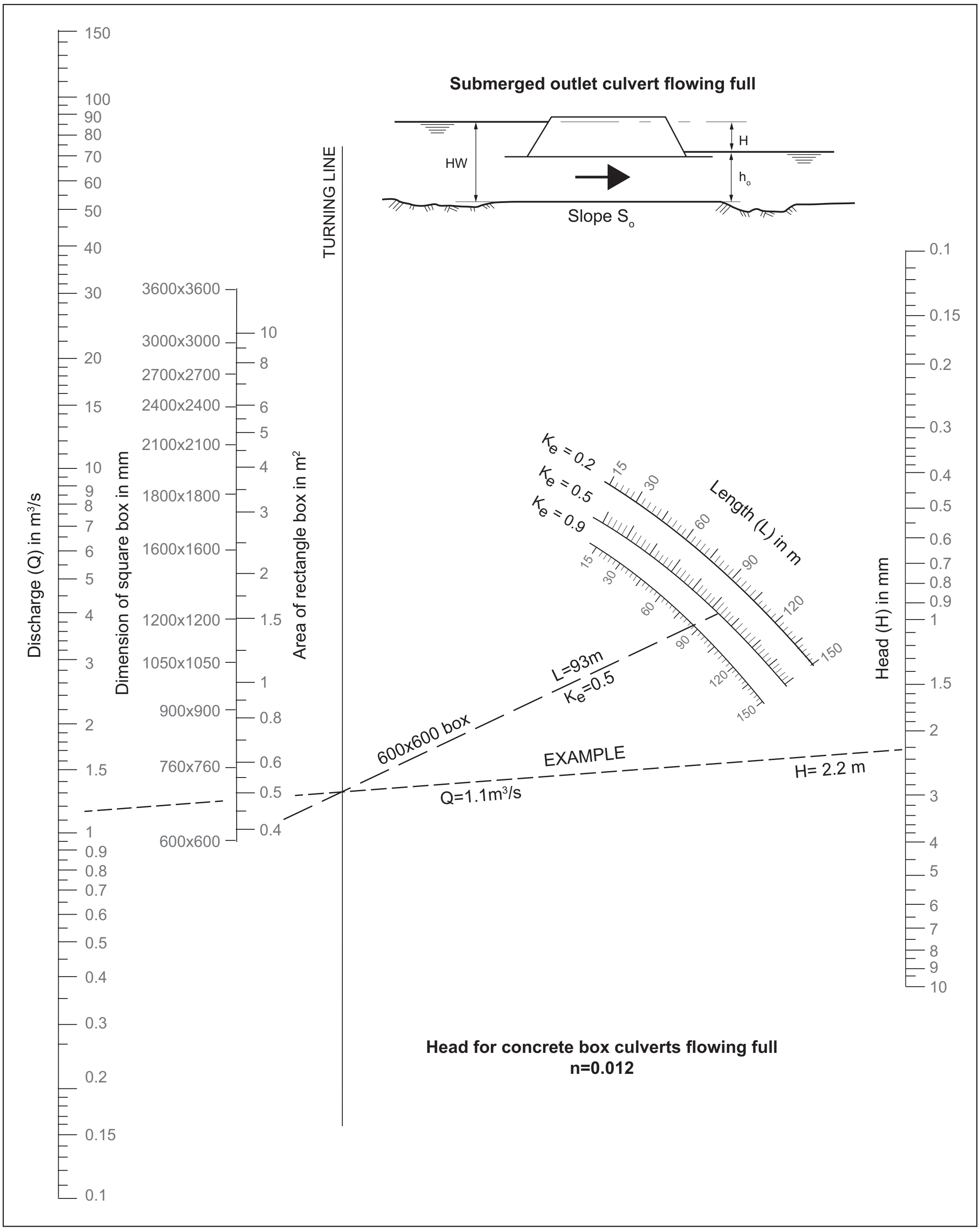


Figure C.5.9: Head for box culvert flowing full, n = 0.012

(Adapted from Chart 15A, FHWA, 2012)

Figure C.5.10 is used to determine critical depth (d_c) for pipe culverts. The steps are as follows:

i. Determine the design discharge (Q) and size of pipe culvert.

ii. Draw a vertical line from the discharge value on the horizontal axis to intersect the value of the pipe size.

iii. From the intersection point, draw, a horizontal line to meet the vertical axis.

iv. The value on the vertical axis is the critical depth (d_c) for the pipe culvert.

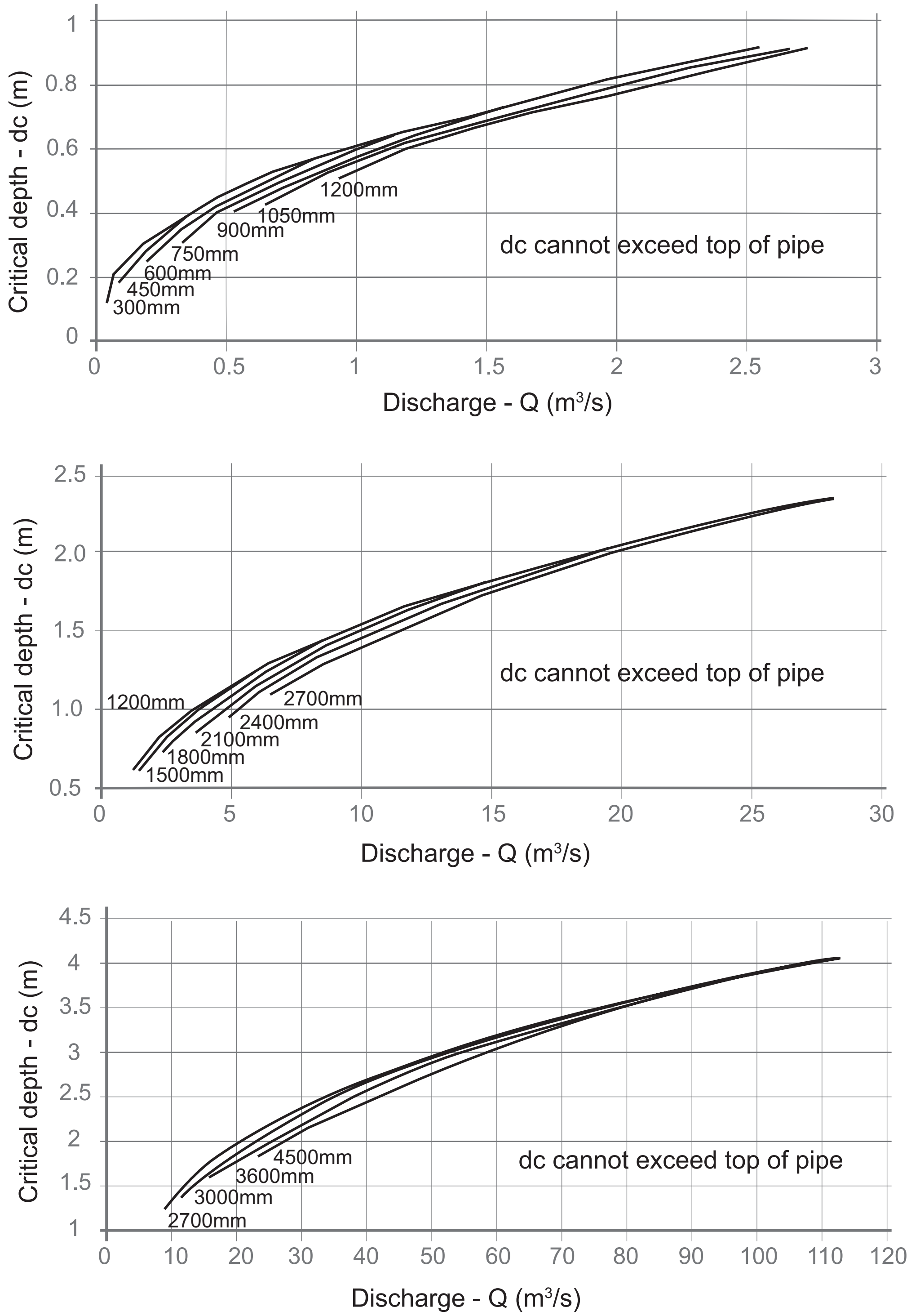


Figure C.5.10: Critical Depth (d_c) for circular pipes

(Adapted from Chart 4A, FHWA, 2012)

Figure C.5.11 is used to determine critical depth (d_c) for box culverts. The steps are as follows:

i. Determine the ratio of design discharge (Q) to width of box culvert (B) i.e. Q/B.
ii. Draw a vertical line from the value of Q/B on the horizontal axis to meet the graph.
iii. From the intersection point, draw, a horizontal line to meet the vertical axis.
iv. The value on the vertical axis is the critical depth (d_c) for the box culvert.

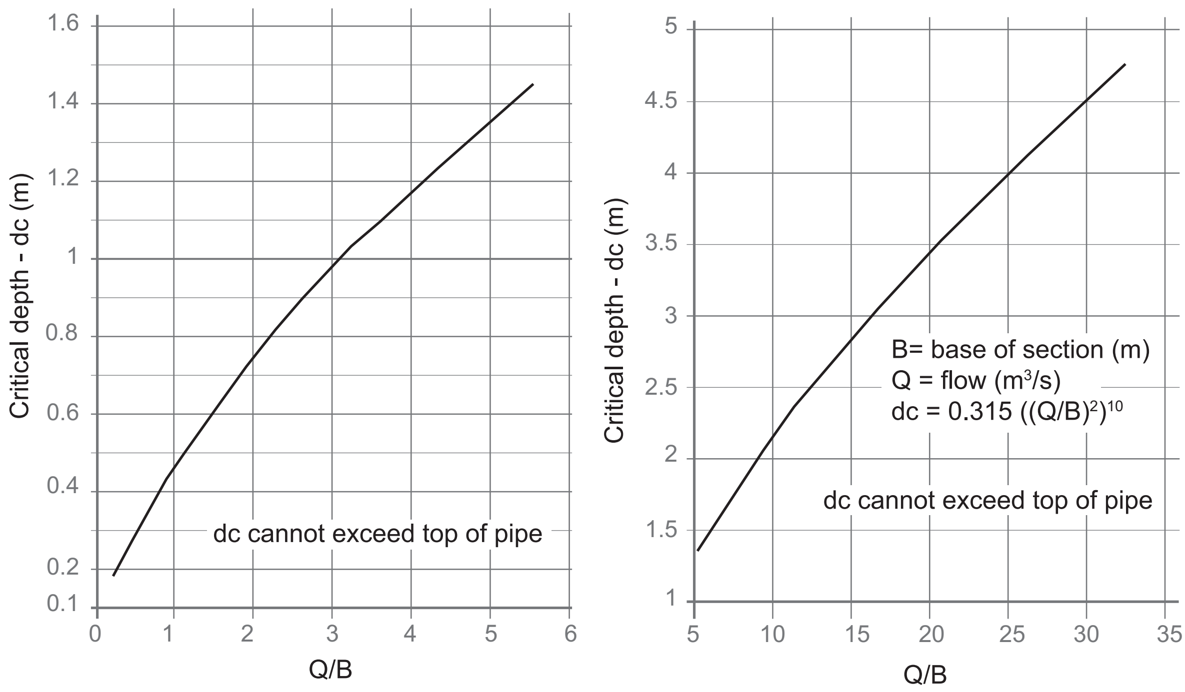


Figure C.5.11: Critical Depth (d_c) for rectangular section

(Adapted from Chart 14A, FHWA, 2012)

Values of coefficient k_e to apply to the velocity head ($V^2/2g$) for determination of head loss at the entrance to a culvert operating under outlet control are given in Table C.5.2..

Then entrance head loss $h_e = K_e \frac{V^2}{2g}$

Table C.5.2: Entrance loss coefficients

Type of barrel and inlet	Ke
Pipe, concrete	
Projecting from fill, socket end	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwall	
Socket end of pipe	0.2
Square-edge	0.5
Rounded (radius =1/12D)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope (standard precast)	0.5

Type of barrel and inlet	Ke
Beveled edges, 33.7° or 45° bevels	0.2
Side-tapered or slope-tapered inlets	0.2
Box, reinforced concrete	
Headwall	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edges crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or bevelled top edge	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-tapered or slope-tapered inlet	0.2
Projecting	
Square-edged	0.7*
Beveled edges, 33.7° or 45° bevels	0.2*

**Estimated*

Refer to Greenville County, South Carolina: Storm Water Management Design Manual, January 2013 Appendix C for sketches of the inlet types shown in Table C.5.2.

5.3.3 Use of computer based approach - HY-8

HY-8 is computer based software used for culvert analysis to size culverts that will pass the design flows without overtopping (i.e. flooding) of the road surface. The software is Windows based and user friendly. The major input data requirements for the HY-8 software are placed under the following subjects:

- Crossing Properties;
- Discharge Data;
- Tailwater Data;
- Roadway Data; and
- Culvert Properties.

There are two entry options for this: select either Culvert Data or Site Data.

(i) For Culvert Data, it is necessary to enter the culvert slope, type of culvert material (concrete, steel, etc.), and the Manning's coefficient (n) from Table C.1.7 based on the culvert material.

(ii) For Site Data, there are two options: either the user selects Culvert Invert Data or Embankment Data and enters the parameters accordingly from the pull down menus.

After entering all data, select Analyze Crossing from the menu bar for output results.

5.3.4 Culvert alignment

When a natural stream crosses the road at an angle, it is better to construct a skew crossing or to realign the road, so that a 90° crossing can be constructed, rather than change the direction of the channel. If the existing channel bed is altered, a lot of erosion problems can be expected. It can sometimes be difficult to maintain an adequate gradient over the longer length required in the case of a skewed culvert. Such concerns are less important in the case of relief culverts in sidelong ground, where there is no existing watercourse, and where it is easier to ensure an adequate crossfall, even over a longer culvert length.

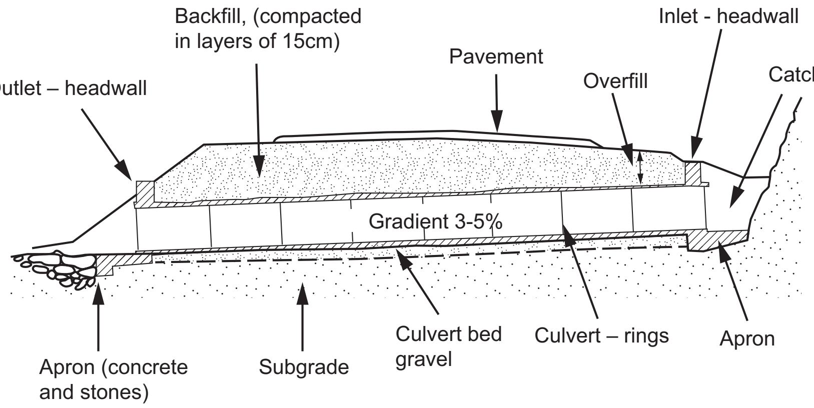
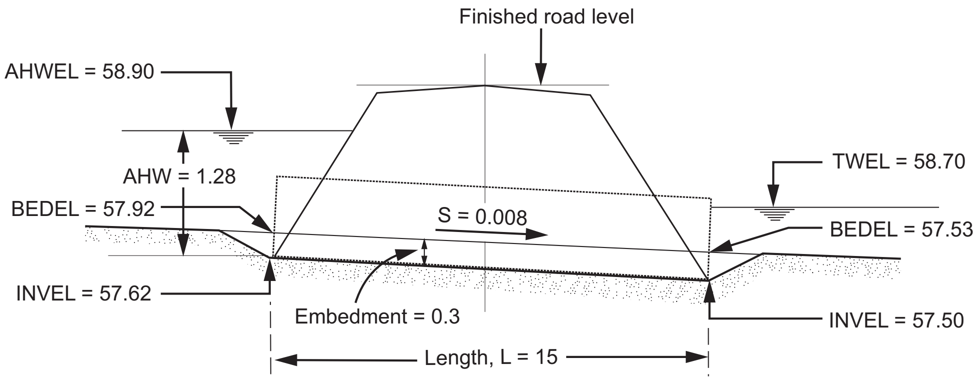


Figure C.5.12: Relief Culvert

5.3.5 Worked Examples

Worked Example 1

The figure below illustrates a pipe culvert crossing a road on an embankment at KM 5+450 on a low volume road.



Given:

- 10-year design flow i.e. Q_{10} = 1.3 m^3/s
- Channel is non-erodible, with:
  - Slope of bed S_o = 0.008 m/m
  - Natural bed elevation at Culvert Outlet = 57.53
  - No restrictions on Outlet Velocity.
- Allowable Head Water Elevation (AHWEL) = 58.90.
- Allowable Head Water (AHW) = 1.28 m.
- Skew No. = 90.
- From field investigation, approx. 10-year flood depth is 0.9 m above average stream bed, and there is no flow through most of dry season.
- Culvert Length L is approx. 15 m.
- Invert is to be embedded 0.3 m below streambed.
- There is an existing open footing culvert at the site 1.60 m x 1.5 m x 20 m long, which is hydraulically adequate except for some scour.

Task: Determine the required size of conventional concrete pipe culvert with headwall and square edge.

Solution:

Step 1: Assemble and review the available data

Step 2: Culvert selection

See the hand calculation spreadsheet in Appendix C.3 for the solution to the question. The columns are as explained in Section 5.3.2, Step 2 – culvert selection.

The columns requiring explanation are included below:

Col. 10 Trial Size: The existing open footing culvert has a cross-sectional area of 1.60 m x 1.5 m = 2.4 m^2. Try a 2 x 1.20 m pipe culvert giving slightly smaller area of 2.26 m^2. The 1.2 m height is slightly less than the AHW, which is acceptable.

Col. 16 Since HW < AHW, the size is acceptable

Col. 25 Col. 24 is larger than Col. 16, thus outlet control governs, and the 2 x 1.20 m pipe culvert size is satisfactory.

Col. 26 Outlet Velocity

Since flow is under outlet control, the outlet velocity V_o is determined from the equation, Q = VA thus,

$$V_o = Q/A$$

Where,

V_o = Outlet Velocity

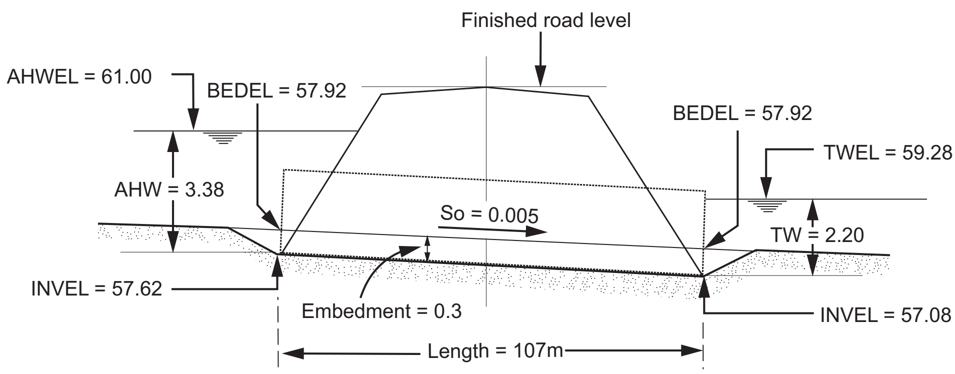
Q = Maximum Discharge

A = Area corresponding to the Tailwater Depth, TW.

The hand calculation spreadsheet is included as Appendix C.3.

Worked Example 2

The figure below illustrates a box culvert in inlet control located at KM 1+300 on an LVR.



Given:

- 50-year design flow i.e. Q_{50} = 28.3 m^3/s.
- Channel is non-erodible, with slope of bed S_o = 0.005 m/m.
- Natural bed elevation at culvert outlet = 57.38.
- No restrictions on outlet velocity.
- Allowable head water elevation (AHWEL) = 61.0.
- Allowable head water (AHW) = 3.38 m.
- Skew no. = 90.
- From field investigation, approx. 50-year flood depth is 1.9 m above average stream bed, and there is no flow through most of dry season.
- Culvert length, L (excluding wingwalls) approx. 107 m.

- Invert is to be embedded 0.3 m below streambed.
- There is an existing open footing culvert at the site 6.10 m x 1.83 m x 107 m long, which is hydraulically adequate except for some scour.

Task: Determine the required size of conventional reinforced concrete (RC) box culvert with 45° wingwalls and 45° beveled inlet top.

Solution:

STEP 1: Assemble and review the available data

STEP 2: Culvert Selection

See the hand calculation spreadsheet in Appendix C.3. for the solution to the question. The columns are as explained in Section 5.3.2. Only columns requiring explanation are included below:

Col. 10 Trial Size: The existing open footing culvert has a cross-sectional area of 6.10 m x 1.83 m = 11.16 m^2. Although the proposed box culvert will be much longer, it will withstand higher velocities, therefore try a 3.5 m x 3.0 m box culvert giving slightly smaller area of 10.50 m^2. The 3.0 m height is slightly less than the AHW, which is acceptable.

Col. 16 The second trial size 3.0 m x 3.0 m is satisfactory since HW < AHW. A 3.5 m x 3.0 m box culvert would have been needed if the culvert had not been embedded 0,3 m.

Col. 25 Col. 16 is larger than Col. 24, thus inlet control governs, and the 3.0 m x 3.0 m size is satisfactory.

Col. 26 Outlet Velocity: Although in this example there is no potential downstream problem, the outlet velocity will be calculated to illustrate the procedure. See Table C.5.3. The culvert is operating in inlet control.

(n = 0.012, s = 0.005 m/m, 3.0 m x 3.0 m box culvert)

Table C.5.3: Calculations of outlet velocity for inlet control culvert

Trial d (m)	D/d *	A_o/A *	A_o (m^2)	P_o/P *	P_o (m)	R_o (m)	V_o (m/s)	Q_o (m^3/s)	Remarks
1	2	3	4	5	6	7	8	9	10
2.5	-	-	7.5	-	8.0	0.94	5.65	42.4	Q_o > 28.3 m^3/s
2.0	-	-	6.0	-	7.0	0.86	5.33	32.0	Q_o > 28.3 m^3/s
1.8	-	-	5.4	-	6.6	0.82	5.16	27.9	Q_o < 28.3 m^3/s
									V_o = 5.2 m/s

*Columns not required for rectangular culverts:
Col. 6 $P_o = b + 2d$
7 R_o = col. 4 ÷ col.6
8 $V_o = 1/n \{R^{0.667} S^{0.5}\}$
9 Q_o = col. 4 x col. 8

5.4 Simplified method for culvert design

5.4.1 Single bore culverts

For very low traffic LVRs (Class F2 and F3) the following simplified method may be used to select the culvert size. The size of pipe is selected from Table C.5.4 to accommodate the calculated flow, Q. The data are based on a flood return period of 10 years, which is the recommended return period to be used for the design of pipe culverts on low traffic LVRs. It is assumed that inlet control condition prevails, the bed slope is 1% and the wingwalls have a 45° flare.

Table C.5.4: Capacities of pipe culverts

D (mm)	H_w (m)	Q (H_w/D = 1.0)	H_w (m)	Q (H_w/D = 1.2)
900	0.9	0.9	1.1	1.2
1200	1.2	1.8	1.4	2.4
1500	1.5	3.2	1.8	4.2
1800	1.8	5.0	2.2	6.5

Q = Maximum Discharge, m^3/s

D = Internal Diameter of Pipe, m

H_w = Headwater elevation, m

It is recommended to use H_w = D for pipe culverts on LVRs since the cover to culverts is normally small.

5.4.2 Multiple bore culverts and vented fords

For multiple pipe culverts the estimated flow capacities are given in Table C.5.5. The data are based on a flood return period of 10 years, with inlet control, bed slope of 1% and 45° wingwalls flare.

Table C.5.5: Capacities of multiple pipe culverts

D (mm)	Q_{single}	Q_{double}	Q_{triple}
900	0.9	1.8	2.6
1200	1.8	3.6	5.4
1500	3.2	6.3	9.5
1800	5.0	10.0	14.9

Q = Maximum Discharge, m^3/s

D = Internal Diameter of Pipe, m

Where multiple pipes are to be installed the minimum space between the center-line of adjacent pipes should be at least 2 pipe diameters. Where space restrictions require the installation of pipes at closer spacing, the factors in Table C.5.6 should be used to reduce the flow rates through the pipes.

Table C.5.6: Pipe spacing and Flow Reduction Factor

Spacing between pipe centers	Flow Reduction Factor
More than 2.0 pipe diameters	1.0
1.5 - 2.0 pipe diameters	0.9
Less than 1.5 pipe diameters	Not recommended

Due to the difficulty of ensuring adequate compaction under and between the pipes, a bedding of lean concrete should be used where space restrictions require the installation of pipes at close spacing.

The design flow for a multi-bore culvert should be taken to be the maximum flood flow. As vented fords are designed to be overtopped during peak flows the pipes should be designed to pass the normal flow and small floods. Overtopping will only occur for the higher flow rates and the designer must decide what level of flow the pipes will pass before overtopping occurs. The overtopping flow depends on the duration, size and regularity of high flows and the total number of pipes that can be fitted into the structure.

5.4.3 Box culverts

Flow capacities for standard box culverts are given in Table C.5.7. The data are based on the flood return periods for the design of box culverts that are given in Table C.5.8. It is recommended to use H_w = H for box culvert design on LVRs as the cover to culverts is normally small.

Table C.5.7: Capacities of standard box culverts

B (m)	H = H_w (m)	Q_{single} (m^3/s)	Q_{double} (m^3/s)	Q_{triple} (m^3/s)
1.0	0.6	0.7		
1.0	0.8	1.1	2.2	
2.0	1.0	3.0	6.1	
2.0	1.2	4.0	8.1	
2.0	1.4	5.1	10.2	
2.0	2.0	8.9	17.8	26.7
3.0	2.0	13.4	26.7	40.1
3.0	3.0	25.0	50.1	75.1
4.0	3.0	33.4	66.8	100.1
4.0	4.0	52.1	104.3	156.4

Q = Maximum Discharge, m^3/s

B = Width of the box culvert (m)

H = Height of the box culvert (m)

H_w = Headwater elevation, m

The following are assumed: Inlet control, wingwall flare 30° to 75°, and vertical headwall.

Table C.5.8: Return period (years) for box culverts

B (m)	H = H_w (m)	Single	Double	Triple
1.0	0.6	10		
1.0	0.8	10	10	
2.0	1.0	10	10	
2.0	1.2	10	10	
2.0	1.4	10	10	
2.0	2.0	10	25	25
3.0	2.0	10	25	25
3.0	3.0	10	25	25
4.0	3.0	25	25	50
4.0	4.0	25	25	50

5.5 Drifts

The design process for a drift includes the following steps:

STEP 1: Collect the stream data

i. Design Discharge (Q), m^3/s
ii. Design Return Period - 10 years
iii. Downstream Channel Invert Level (IL_o)
iv. Catchment Slope (S_c)
v. Local River Bed Slope (S) = 0.7 x S_c

STEP 2: Determine the tailwater data

vi. The Depth of Flow (D_{tw}) = $[(Q \times n)/(K \times S^{0.5})]^{0.45}$, m
a. Where,

n = Manning's Coefficient = 0.05

Coefficient K = 4.48

Q = Design Discharge

S = River Bed Slope

i. The Channel Width (W) = 5.6 x $D_{tw}^{0.26}$, m
ii. The Tailwater Level (TWL) = $IL_o + D_{tw}$, m
iii. The Velocity in Channel (V) = Q/ (D_{tw} x W), m/s

STEP 3: Design the drift

iv. Assume a length for the drift (L), m. This is the length of the drift in the direction of travel along the road. A good assumption can be made from site observations at the proposed location.
v. Calculate the unit discharge (q) = Q/L $m^3/s/m$
vi. Calculate the critical depth (D_c) = $(q^2/g)^{0.33}$
vii. Calculate the critical velocity (V_c) = q/D_c
If V_c > 1.50 m/s, increase the length of the drift
viii. The head (H_c) = $D_c + V_c^2/2g$
ix. The maximum depth of water is the greater of either (TWL – IL_D) or $H_{c.}$

The drift invert level (ILD) is determined on site depending in the site conditions. The invert level should normally be at the road level or slightly below. If the drift is too high there will be increased erosion downstream and around the outside of the drift if it is overtopped. The approach ramps at each end of the drift must be long enough to accommodate the calculated maximum depth of water.

5.6 Vented fords

Figure C.5.13 shows the crossing profile of a vented ford. HW is the depth of headwater, P is the height of the ford above the channel bottom, H is the upstream head, h is water depth at the center of the ford, and D is the diameter of pipe or the height of vent.

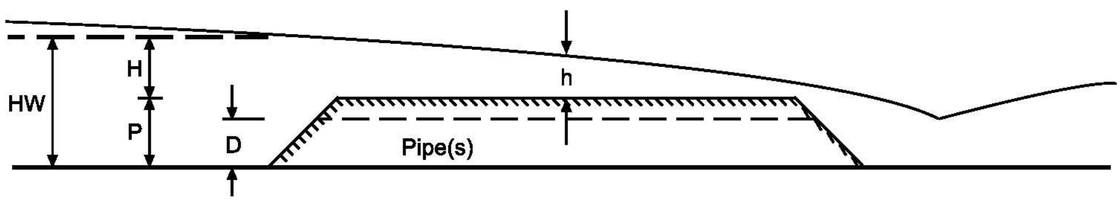


Figure C.5.13: Crossing profile of a vented ford

Design considerations for vented fords are summarized in Table C.5.9.

Table C.5.9: Design considerations for vented ford

Considerations	Criteria
Depth of cover above pipes	Minimum 31 cm recommended.
Exit velocity of pipes (vents)	Limit exit velocity of the flow not to exceed 3.0 m/s. Exit velocity, $V_e = Q_{vent}$ /(flow area of pipe).
Pipes	Pipes should be anchored in the ground and both ends beveled or mitered to reduce debris accumulation. Minimum size is 60 cm diameter.
Guard rails	Guard rails are not recommended to avoid catching debris and floating materials during a flood.
High streamflow	Road surface is raised above streambed to accommodate the flow.
Streambed erosion protection	Riprap is placed upstream and downstream to reduce the scour in erodible channel.

A vented ford is designed to have a flow capacity of $Q_{vent} = Q_e - Q_{top}$ (1)

where, Q_e is the total design flow from hydrological analysis, and Q_{top} is the flow over the ford.

The flow over the ford can be calculated from the equation, (2)

where, L is the length of the ford normal to the flow (i.e. the width of the ford at the road level).

Considering H = h/0.6 and assuming a maximum allowable water depth (h) of 0.31 m over the ford, H becomes 0.517 and the equation can be rearranged as:

$$Q_{top} = 1.603L^{0.0251} \quad (3)$$

After the discharge through the vent (Q_{vent}) is determined (Equation 1) the number and size of pipes is selected. A single pipe may be considered first. If a computed trial size is larger than the design height of the low water surface crossing or availability of pipe size, multiple culverts should be used. The design discharge flowing through each pipe is equal to the total discharge through the vent divided by the number of pipes.

The pipe exit flow velocity should not exceed 3 m/s for scour control and channel protection. The exit velocity is computed by:

$$V_e = \frac{Q_{vent}}{\frac{\pi D^2}{4}} \quad (4)$$

6. STRUCTURAL DESIGN

6.1 Introduction

Runoff through drainage structures can cause many problems to the existing channel both at the inlet and especially the outlet. It is therefore necessary to design the structural elements to minimize or eliminate damage caused by the runoff. Table C.6.1 shows the structural elements that need attention for the different drainage structures.

Table C.6.1: Critical design aspects

Structural Element	Drift	Culvert	Vented Drift	Major Culvert	Bridge
Foundations	✓	✓	✓	✓	✓
Structural slabs	✓		✓	✓	
Cut-off walls	✓	✓	✓	✓	✓
Pipes		✓	✓		
Headwalls & wingwalls		✓	✓	✓	✓
Apron	✓	✓	✓		
Approach ramps			✓	✓	✓
Downstream protection	✓	✓	✓	✓	✓
Arches				✓	
Deck				✓	✓
Abutments					✓
Piers					✓
Bearings & Joints					✓

6.2 Scour

Scour is the erosion of material from the river sides and bed due to water flow. Damage due to scour is the most likely cause of structural failure of drainage structures. Minimizing or eliminating the effects of scour should therefore receive the most attention when designing any drainage structure. Scour can occur during any flow, but the risk is generally greater during floods.

There are three major types of scour to be considered:

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

The latter two scour types are the most important to consider when designing a structure. The amount of scour at a structure is affected by the following factors:

- **Slope, alignment and bed material of the stream**: the amount of scour is dependent on the speed of the water flow and the erodibility of the bed material. Higher water velocities result in more scour.

- **Vegetation in the stream**: any vegetation growing permanently in the stream can improve the strength of the river bed, reducing scour. The vegetation can also reduce the speed of the water.
- **Depth, velocity and alignment of the flow through the bridge**: the faster the flow, the more scour will occur. If the flow is not parallel to the constriction more scour will occur on one side of the constriction.
- **Alignment, size, shape and orientation of piers, abutments and other obstructions**: water is accelerated around these obstructions, creating vortices with high velocities at abrupt edges on the obstruction, increasing the scour depth.
- **Trapped debris**: debris can restrict the flow of water and cause an increase in water velocity. It is important that structures are designed to minimize the chances of debris being trapped and to ensure that inspections and maintenance are carried out after flood periods to remove any lodged debris.
- **Amount of bed material in the water**: if the water is already carrying a large amount of material eroded from further upstream a greater amount of scour will occur at the structure. A typical example is saltation where runoff in the side drain carries along smaller stone particles which erode the earth ditches.

The site of the proposed structure and the watercourse upstream and downstream must be inspected for evidence of existing scour, erosion or deposition in the watercourse and banks.

It is difficult to accurately predict the level of scour that may be experienced for a particular design. There are many formulae for predicting the amount of scour around a structure but these formulae, in general, require detailed knowledge of the river and bed characteristics. They are also based on empirical data and often give different design scour depths. Engineering judgement is required. This manual proposes a number of 'rules' for designing to resist scour. It must be stressed that these rules are not infallible and local knowledge should also be considered when designing a structure.

Rule 1 - Provide minimum foundation or cut-off wall depths

Regardless of the required depth for foundations determined by the ground conditions and predicted scour, the minimum foundation and cut-off wall depths shown in Table C.6.2 should be provided. The depth is measured from the lowest point in the bed of the watercourse at the crossing point. These depths can only be reduced where firm rock is encountered at a shallower depth and the foundations are firmly keyed into the rock.

Table C.6.2: Foundation depths

Structure	Foundation depth	Cut-off wall depth
Drift	Not applicable	Min. 0.6 m
Relief culvert	Not applicable	Min. 0.6 m
Watercourse culvert	Not applicable	Min. 0.6 m (headwalls and wingwalls)
Vented drift	Not applicable	Min. 0.6 m
Large bore culverts	3.0 m	Min. 0.6 m
Bridges	3.0 m	Not applicable

Rule 2 - Create a minimal constriction to the water flow

The amount of scour experienced at a structure is proportional to the restriction in the normal water flow. If the flow is unconstrained then scour will not occur. Where the flow is constrained the design of the structure, particularly the level of foundations, should allow for a lowering of the river bed level due to scour. The amount/depth of scour that will occur depends on the following three factors:

- Constricted flow width;
- Maximum flow rate; and
- The type of material forming the sides and bottom of the watercourse.

6.3 Foundations

The strength and durability of any structure is determined by the quality of its foundations and the bearing capacity of the soil.

For small, simple structures such as drifts, culverts and vented fords it is sufficient to construct the structure on well drained, firm soil. These conditions can be determined on site by checking for footprints when walking over the proposed location. If more than a faint footprint is left it is necessary to improve the ground before construction commences.

If the ground conditions are poor at the proposed level of the structure's foundation, it is necessary to continue excavation to firm material that can provide sufficient bearing capacity. The designer then has several options for the construction of the structure:

- Alter the design to lower the level of the foundations;
- Replace the poor excavated material with new material that has a better bearing capacity (e.g. a well graded sand and gravel) that is compacted into the excavation in 300 mm layers;
- Stabilize the poor material by mechanical or chemical means; or
- Provide a deep foundation (not covered by this manual).

For all structures it is necessary to start the construction on a well-drained, level base. The excavations for all structures, apart from those built on rock, should be dug an additional 300 mm below the proposed foundation level. A 300 mm layer of sand and fine gravel (free from clayey or silty material) complying with the requirements of base soil category 4 in Table 6.3 should be placed and levelled in the bottom of the excavation and compacted in 150 mm layer thickness to 95% of the maximum density of the material to provide a good base for the structure. Alternatively, at least 100 mm of lean concrete blinding (15 MPa) should be laid to provide a firm clean working platform.

Table C.6.3: Base soil requirements

Base Soil category	% finer than No. 200 sieve (0.075 mm) (after regrading, where applicable)	Base Soil description
1	> 85	Fine silt and clays
2	40 – 85	Sands, silts, clays, and silty & clayey sands
3	15 – 39	Silty & clayey sands and gravel
4	< 15	Sands and gravel

Source: United States Department of Agriculture (1994)

A simplified method for calculating the load exerted by the foundations of a vented ford or large bore culvert on the ground is to calculate the load of the structural fill material and multiply by a safety factor shown in Table C.6.4.

Table C.6.4: Bearing safety factor

Material	Load per meter of fill	Safety factor
Concrete/gravel	25 kN	1.5
Earth	20 kN	1.5

Example

The central section of a vented ford is 2 m high (from its foundation level) and has masonry walls with an earth fill inside. What is the foundation loading?

The load exerted on the soil below the structure will be: 2 x 20 x 1.5 = **60 kN/m²**

Where a foundation is to be built on rock which may be sloping down to the watercourse, it is necessary to form a level platform for the foundation. This may be achieved by either breaking out the rock to give a level foundation or building up the foundation to level by placing concrete around drilled and grouted mild steel bars. The preferred option which should be adopted, unless the rock is too hard to break out, is to break out a level platform.

6.4 Cut-off walls

Cut-off walls, also called curtain walls, should be provided at the edge of drainage structures. They prevent water eroding the material adjacent to the structure, which would eventually cause the structure to collapse. The location of cut-off walls for the various structures is shown in Table C.6.5.

Table C.6.5: Cut-off wall locations

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

The absence of cut-off walls at the inlet of the structure could allow water to seep under the apron and structure causing settlement and eventually collapse of the structure. At the downstream end of the structure the flowing water could erode the material next to the apron, eventually eroding under the apron and causing it to collapse. The benefits of a cut-off wall for culverts are illustrated in Figure C.6.1.

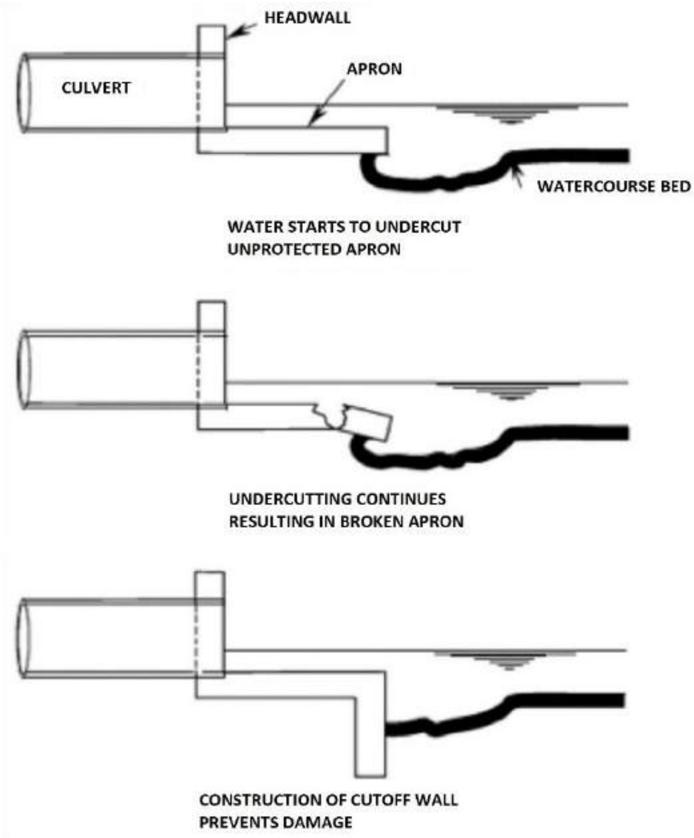


Figure C.6.1: Benefits of cut-off walls

The depth of the cut-off walls depends on the ground conditions. Where a rock layer is close to the ground surface the cut-off walls should be built down to this level. If there is no firm stratum near the surface the cut- off walls should extend the minimum dimensions listed in Table C.6.2. The method of construction of the cut-off wall should be similar to the construction method and material used for the remaining parts of the structure. This facilitates the construction and reduces cost.

6.5 Pipe culverts

Concrete pipes are usually used as culverts (watercourse, relief and access culverts) on LVRs. Metal (steel) and timber pipe culverts are rarely used as culverts on LVRs in Liberia. Precast reinforced concrete pipes are mostly used as watercourse and relief culverts in Liberia.

Figure C.6.2 shows a typical concrete pipe culvert under construction.a

Figure C.6.2: Reinforced concrete pipe culvert

There are varying standard size of concrete pipes used as cross culverts on LVRs. An oversized culvert, designed to avoid pipe repairs or failure as well as prevent environmental damage, can be cost-effective in the long run. The minimum recommended size is 900 mm diameter pipe. A 600 mm diameter pipe may be used for relief culverts in flat ground where the invert depth is limited, but U-culverts should be used in preference to smaller diameter pipe culverts in these circumstances. Humps built into the road over culverts to provide the minimum cover are strongly discouraged.

The design process for culverts requires data on the culvert catchment area and predicted rainfall intensity. The methodology for sizing of culverts is given in Chapter 5. Standard drawings for culverts are included in Appendix C.4.

6.6 Headwalls and wingwalls

6.6.1 General

Headwalls and wingwalls are required at each end of a culvert and serve several different purposes:

- they direct the water in or out of the culvert;
- they retain the soil around the culvert openings; and
- they prevent erosion near the culvert and seepage around the pipe which causes settlement.

6.6.2 Culvert headwall

The headwall can be positioned at different places in the road verge or embankment as shown in Figure C.6.3.

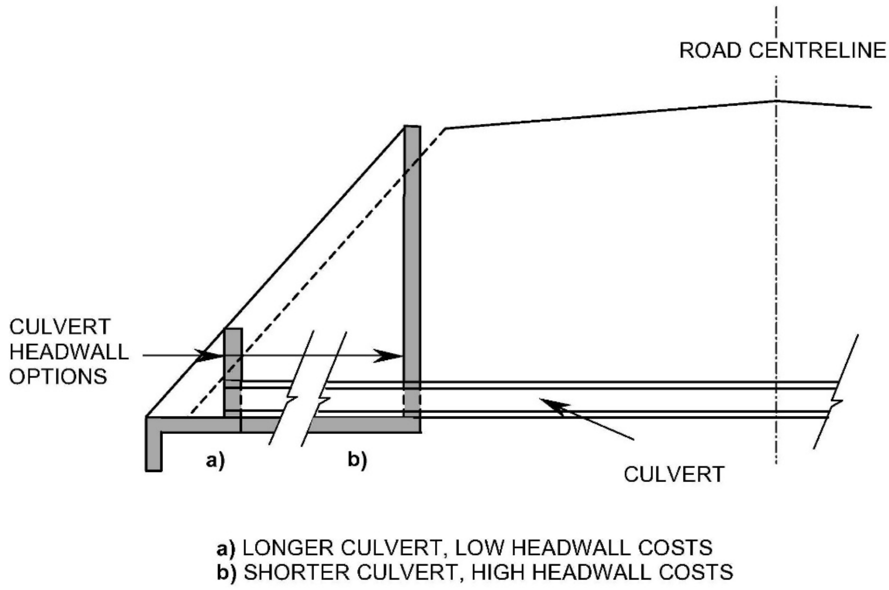


Figure C.6.3: Possible culvert headwall positions

The closer the headwall is placed to the road on an embankment the larger and more expensive it will be. The most economical solution for headwall design is to make it as small as possible. Although a small headwall will require a longer culvert, the overall structure cost will normally be lower. If, due to special circumstances at a proposed culvert site, a large headwall with wingwalls is required it should be designed as a bridge wingwall.

Where a road is not on an embankment the size of the headwall will be small regardless of position. In this case the position of the headwalls is determined by the road width and any requirements of national standards. The headwalls should be positioned at least 1 meter beyond the edge of the carriageway width to prevent a restriction in the road and reduce the possibility of vehicle collisions (see Figure C.6.4).

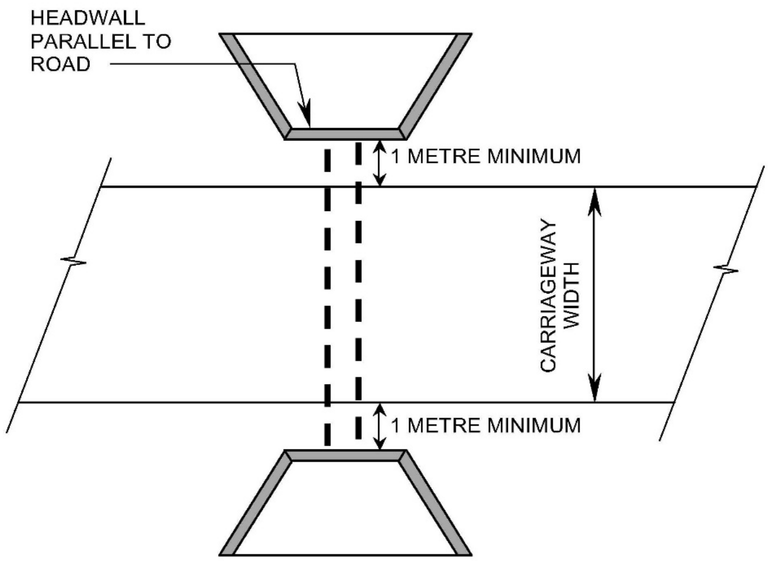


Figure C.6.4: Position of culvert headwalls

Other important considerations for headwalls include:

- Headwalls should project above the road surface by 300 mm and be painted white so that they are visible to drivers.
- Headwall faces should always be at right angles to the line of the culvert, at both inlet and outlet, to present minimal disruption to the water flow, including on skew culverts.
- Headwall and adjacent works must be designed so that the culverts can be de-silted manually under maintenance arrangements. Cleaning can be difficult with a drop inlet arrangement.

6.6.3 Wing walls

Wing walls are used to retain the soil behind the culvert and abutments of bridges to help guide the flow through the structure in flood conditions and safely retain the backfill material without risk of erosion. There are two basic reference layouts for wingwalls, either parallel to the road or parallel to the watercourse (see Table C.6.6). However, wingwalls are usually constructed at an angle between these two arrangements. Wingwalls should always be constructed to the toe (bottom) of the slope and not part way down. Wingwalls that do not extend to the bottom of the slope are likely to suffer from erosion around the ends.

Table C.6.6: Wingwall positions

Wingwalls parallel to watercourse	Wingwalls parallel to road
Foundations on same level	Foundations can be stepped but are more difficult to construct
Wall more susceptible to erosion from watercourse	Wall mostly away from watercourse
Wall size smaller than wall parallel to road	Wall size larger than wall parallel to watercourse
Larger amount of fill to be moved, placed and compacted	Reduced amount of fill required to be moved, placed and compacted

The relative availability and cost of fill material and materials to construct the wingwalls will determine the most appropriate arrangement. In general, to ensure the cheapest option, the design should ensure the smallest wingwalls are chosen for the structure and its location. Where wingwalls are chosen that run parallel to the road it is necessary to take suitable measures to prevent water in the carriageway side drains causing erosion around the wall at their outfall. This usually requires a lined channel or cascade at the base of the wingwall.

6.6.4 Aprons

Purpose of aprons

An apron is required at the inlet and outlet of culverts and downstream of drifts and vented fords to prevent erosion. As the water flows out of or off a structure it will tend to erode the watercourse downstream, causing undercutting of the structure. This process is described in Section 6.4.

Aprons should be constructed from a material which is less susceptible to erosion than the natural material in the stream bed. A typical concrete culvert apron is shown in Figure C.6.5.

Figure C.6.5: Concrete apron at culvert outlet

Drift aprons

Where the discharge velocity across the drift is less than 1.2 m/s, which may be experienced for relief drifts, a coarse gravel layer (10 mm) will provide sufficient protection downstream of the drift. For discharge velocities greater than 1.2 m/s more substantial protection is required which utilizes larger stones. The width of the apron should be at least half the width of the drift and extend across the watercourse for the whole length of the drift.

Vented ford aprons

The apron for vented fords should extend the whole length of the structure including downstream of the approach ramps to the maximum design level flood. The other design requirements for vented ford aprons are the same as culvert aprons.

Culvert aprons

Aprons should be provided at both the inlet and outlet of culverts (see Figure C.6.6). They should extend the full width between the headwall and any wingwalls. If the culvert does not have wingwalls the apron should be twice the width of the culvert pipe diameter. The apron should extend a minimum of 1.5 times the culvert diameter beyond the end of the pipe. Cut-off walls should also be provided at the edge of all apron slabs. The choice of apron construction depends on the type of material used for construction of the culvert. The apron may be constructed from gabion baskets, cemented masonry or concrete.

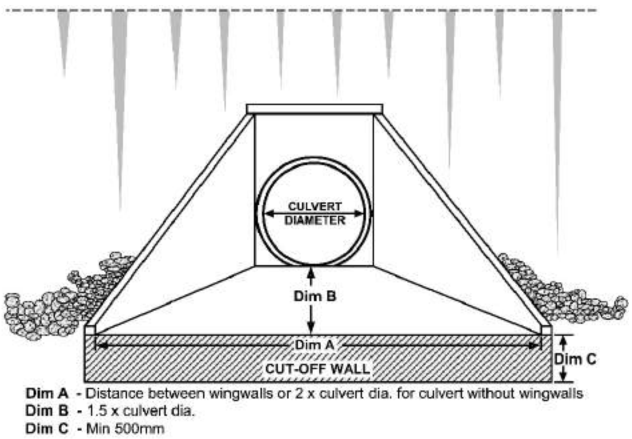


Figure C.6.6: Culvert apron

6.6.5 Approach ramps

The approaches to drift, fords, large bore culverts and bridges must allow vehicles to reach the structure without losing traction or getting stuck on the crossing. The approach slopes should not be so steep that vehicles get stuck at the bottom of the slope. A maximum gradient between 5% and 10% is determined by the vehicles that are using the road. A gradient of 10% may be used if the only vehicles using the road are cars and light trucks. A gradient of 7.5% may be used for medium size trucks and small minibuses and a gradient of 5% if buses and large trucks (>10 tonnes) are expected to travel along the road. Allowance should be made for the fact that heavier vehicles may use the road following improvement of the route. Approaches steeper than 5% require the running surface to have a concrete or cement bound masonry slab to allow vehicles to maintain traction particularly during wet periods. The running surface of the approach way should be designed as a structural slab of either concrete (150 mm thick with light steel mesh reinforcing) or cement bonded stone paving. The slab should also have a 2-3% crossfall in the direction of water flow to ensure that the deck drains quickly after rainfall.

The approach way is subjected to similar erosion characteristics as the main structure. It is therefore necessary to surface the approach ways with the same material as the main structure, at least to the height of the maximum flood level, to ensure damage does not occur. If the structure is designed to be overtopped the approach ways must be constructed higher than the maximum flood level to ensure that the water does not erode around the ends of the structure leaving it inaccessible.

It is necessary to provide cut-off walls along the sides of the approach ways to protect against scour. The sides of the approach ways should be faced to ensure erosion does not occur. They may be constructed from:

- masonry (most appropriate for higher walls);
- gabion baskets; or
- concrete (for low walls up to 0.5 meter).

The fill material in the approach way should be chosen from one of the three options shown in Table C.6.7.

Table C.6.7: Fill material in the approach way

Well compacted sand and gravel	Rubble masonry	Lean concrete mix with plums
Sand and gravel may be readily available in the watercourse around the crossing site. These may be stockpiled during the initial stages of construction by labor. The material to be used as a fill should be well graded and placed in 100 mm layers which are well compacted before subsequent layers are placed.	If a well graded mix of sand and gravel is not available, it may be more economic to use rubble masonry rather than breaking rocks to create a well graded material. Broken man-made bricks can be used in addition to, or instead of, natural stone provided they meet needed requirements. Rubble masonry should be bound together with a 1:8 cement-sand mortar.	A concrete mix of 1:4:8 (cement, sand and aggregate) can be used with large plums up to 200 mm in size. This option will have the highest cement requirement, and hence cost. However, it may be the most beneficial fill option if there are small quantities of sand, aggregate and large stones near the bridge site.

Approach ways are susceptible to scour from water flowing from the carriageway side drains into the watercourse due to the increased slope. A lined channel should therefore be provided at the edge of the approach way to ensure that erosion does not occur. The approach ways should be constructed separately from the main structure to allow for thermal expansion of the structure and slight ground movements, particularly for the structural slab. If they are constructed integrally with the main structure any slight settlement or thermal effects could cause cracks in the structure which would weaken it against damage from water. The approach ways therefore require an end wall and cut-off wall next to the main structure. The gap between the two structures should be very small (no greater than 10 mm). The edges of the approach ways should be marked by guide stones to show drivers the location of the edge of the carriageway. These guides should be 300 mm high and painted white.

6.7 Downstream protection

6.7.1 Overview

Erosion of the watercourse is likely to occur around a structure due to a constriction of the water flow. The constriction causes the water velocity to increase as it passes through or over the structure and this high velocity can be maintained well downstream of the structure. Section 6.6.4 discusses the use of aprons downstream of a structure to prevent erosion and undercutting of the structure. However, in small constrained channels severe erosion may still occur after the apron, particularly where the watercourse is on a gradient. It is therefore often necessary to provide additional protection to the watercourse to reduce the velocity of the water and prevent erosion.

Figure C.6.7 shows a gully that has been formed due to water eroding soft material downstream of a culvert as the watercourse was unprotected. For slow flowing water it is unlikely that any protection will be needed, but for faster flowing water the maximum allowable velocity will depend on the bed material and the amount of silt or other material already being carried in the water.

Figure C.6.7: Erosion of channel downstream of culvert

Erosion can occur in any channel regardless of the presence of a structure. It is therefore not possible to state how far downstream of a structure channel protection should extend. However, the following issues should be considered:

- the general erodibility of the bed, which is based on the type of channel material and the gradient;
- the likelihood of damage to the structure if erosion occurs downstream; and
- the potential effects of erosion on downstream areas (e.g. damage to buildings or farming land).

Maximum water flow velocities that can be tolerated without channel protection related to the type of bed material are shown in Table C.6.8.

Table C.6.8: Maximum water velocities

Bed material	Maximum water velocities without channel protection	
	Clear water (m/s)	Water carrying silt (m/s)
Stiff clay	1	1.5
Volcanic ash	0.7	1
Silty soil / sandy clay	0.6	0.9
Fine sand / coarse silt	0.4	0.7
Sandy soil	0.5	0.7
Firm soil / coarse sand	0.7	1
Graded sand and gravel	1.2	1.5
Firm soil with silt and gravel	1	1.5
Gravel (5 mm)	1.1	1.2
Gravel (10 mm)	1.2	1.5
Course gravel (25 mm)	1.5	1.9
Cobbles (50 mm)	2	2.4
Cobbles (100 mm)	3	3.5
Well established grass in good soil	1.8	2.4
Grass with exposed soil	1	1.8

There are several methods for providing erosion protection to the watercourse. The choice of method depends on the availability or cost of different materials, the size of the watercourse and level of protection required.

6.7.2 Rip-Rap

Rip-rap comprises stones placed in the river bed to resist erosion. the stones must be large and heavy enough that they will not be washed away during floods. rip-rap should consist of well graded rocks placed as tightly as possible to improve the resistance to erosion. The rocks should be strong and not likely to crumble. Angular rocks have the best performance, due to the interlock that is formed between the rocks. Round rocks can be used if they are not to be placed on the sides of a watercourse which has a gradient steeper than 1:4. Flat slab stones should be avoided as they can be easily dislodged by the water flow. Table C.6.13 shows the sizes of stone that should be used for rip-rap.

Table C.6.9: Stone sizes for rip-rap bed protection

Water velocity (m/s)	Rock size dia. (m)	Rock mass (kg)	Minimum % of rock meeting specified dimensions	Thickness of rip-rap (m)
Less than 2.5	0.40	100	0%	0.5
	0.30	35	50%	
	0.15	3	90%	
2.5 - 3	0.55	250	0%	0.75
	0.40	100	50%	
	0.20	10	90%	
3 - 4	0.90	500	0%	1.0
	0.70	250	50%	
	0.40	35	90%	

6.7.3 Masonry slabs

In areas where outlets from culverts are on a steep slope it may not be possible to place rip-rap as it will be washed down the slope. Masonry slabs, cascades or channels may be constructed on the steep section of the outfall to control erosion. Where the water velocity is high it is necessary to use mortar in the slab as hand pitched stones are likely to be washed out. It is not necessary to make the slab smooth as a rough slab will help to reduce the energy in the water. Large stones may be fixed in the slab which project above the standard level to create more turbulence to slow the water speed (Figure C.6.8). Masonry cascades or step structures can incorporate a series of 'ponds' or sumps (stilling basins) to help dissipate energy.

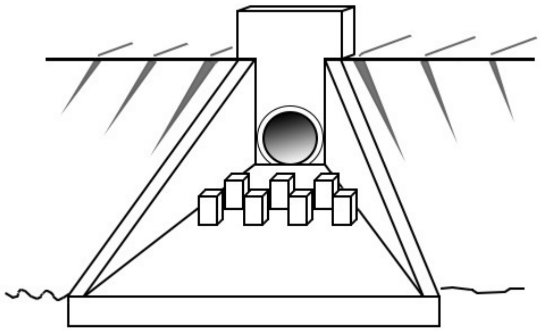


Figure C.6.8: Energy dissipating apron

In flatter areas, up to a 5% gradient, it is normally possible to use hand pitched masonry for aprons on culverts on small watercourses, providing it is well placed with any large flat stones bedded on their edges.

6.7.4 Gabions

Gabions can be used to protect the bottom or banks of a watercourse. As the stones are confined by the wire cages much smaller stones than those used for rip-rap can be used. The disadvantage of gabions is that they have the additional cost of the wire for the cages when compared with rip-rap, and the ability to move and place the stones manually may outweigh the cost of the wire. As gabions can be made in different sizes they can be used for a wide range of different shaped watercourses. They can also withstand limited ground movements and therefore accommodate any small changes in the riverbed. Figure C.6.9 shows the use of gabions and mattresses for protecting the watercourse.

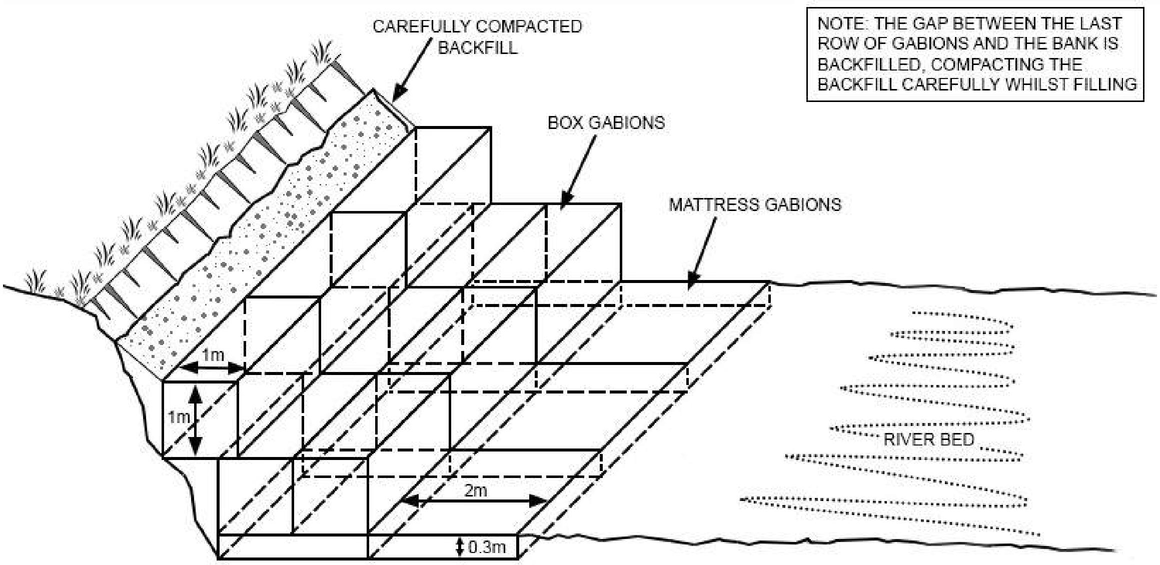


Figure C.6.9: Gabion protection on steep banks

The size of the gabions depends on the velocity of the water flow. For all flow velocities the smallest gabion used for a wall is 0.5 x 0.5 x 1 m. The gabion mattress in the bottom of the watercourse should be 200-300 mm thick for water velocities up to 3 m/s and 500 mm thick for velocities over 3 m/s. It is important that the mattresses are securely wired together to ensure that they do not slide down the bank and cause the water to erode the watercourse banks behind them.

6.7.5 Vegetation

Vegetation is often the best option for erosion protection in small watercourses as once established it slows down the speed of the water flow and holds erodible soil together. It is a cost effective protection method where suitable local plants are available. The use of vegetation to control erosion is sometimes called bio-engineering. Bio-engineering covers a wide range of techniques that use vegetation, for example the control of erosion and stabilization of engineering structures. This manual describes the use of bio- engineering to control erosion downstream of water crossings. It is not sufficient to randomly plant any vegetation as the conditions must be correct for the plants to grow and they must produce the desired anti-erosion effect.

The most basic form of vegetation erosion control is to allow the region's natural grasses to grow in the water channel. They may grow naturally without any assistance if they are already well established in the channel. However, if some erosion has occurred in the channel it may not be possible for the grass to establish itself without assistance. In these cases, it is necessary to cultivate the grass in a nursery or near the site at the roadside if it will not be damaged by vehicles or cattle. Once the grass is established it can then be transplanted into the water channel. The replanting may be by individual plants or by turfing techniques. Natural fiber matting may also help to establish plant growth. The timing of the planting is dependent on the rainy season. Plants need to get established in the watercourse while there is moisture in the soil. It may be necessary to regularly water the plants until they are established in their final situation. However, they are not able to grow during periods when the channel is full of water and it is unlikely that the grass will grow in the base of the watercourse if water is flowing throughout the year. In these cases, it may be possible to plant the grass on the edges of the channel and an aquatic plant in the base of the channel. The choice of plant will again be based on local knowledge, but it is likely that plants found in other watercourses with similar conditions nearby will be the most appropriate. The local agricultural or botanical institutions should be able to provide guidance on plant selection.

In areas where hand pitched stone is proposed to protect the channel downstream from a culvert, it may be reinforced with plants rather than cement or mortar, to bind the stones together. Stones should be placed in the river bed in the same manner as for standard hand pitched stone slabs. Any small gaps that remain between the stones should then be filled with soil and grass planted approximately 150 mm apart. The exact distance will depend on the shapes and gaps between the stones. When the grass is planted the workers should ensure that the roots are deep enough to enter the soil beneath the stone pitching. In channels with a permanent water flow the grass should only be planted towards the sides of the channel, as it may not grow under water in the center of the channel.

Further information on bio-engineering techniques is given in Section 7.7.

6.7.6 Steep channels

In areas where water is flowing down steep hillsides and high embankments and crossing a road through a culvert, it is necessary to provide protection to the slope above and below the road. This is particularly important when a road is winding up a hill and a watercourse crosses the road a number of times. In these locations it may not be possible to channel all the water down steep inclines at the hairpins. Water flowing downhill has a large amount of energy which must be 'lost' if erosion is to be prevented. The most appropriate method in these cases is to construct a step waterfall or cascade to dissipate the energy. A cascade is a drainage channel with a series of steps, sometimes with intermediate silt traps or ponds, to take water down a steep slope,

Figure C.6.10 and Figure C.6.11 show details of gabions being used for erosion control of a steep watercourse.

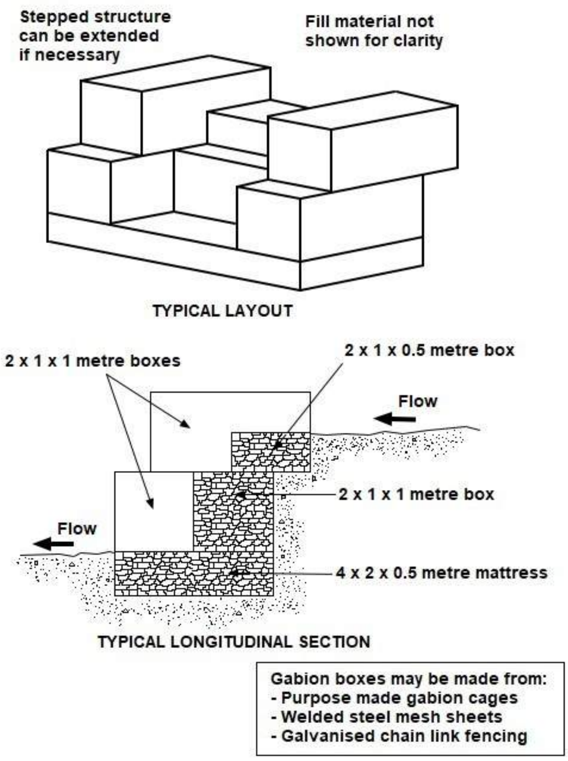


Figure C.6.10: Gabion cascade for steep watercourse

Figure C.6.11: Gabion cascade

6.8 Drifts

6.8.1 General considerations

The primary objective in the design of a drift is to provide a suitable surface for vehicles to drive across while creating minimal disturbance to the water flow. Drift slabs should therefore follow, as closely as possible, the bed of the watercourse. The drift slab surface should be no more than 200 mm above the existing bed level. However, it is desirable to construct the drift with a finished level at the same level as the river bed. Slabs which are constructed more than 200 mm above the existing bed level are likely to cause severe erosion downstream of the drift, requiring frequent maintenance.

NOTE: If the site selected for the drift appears to suffer from silting the final level of the drift could be raised 200-300 mm above the natural river bed. This raising of the level will cause water to flow slightly faster over the drift and reduce the potential for the drift to silt up.

If the river is flowing in a channel with banks on each side it is necessary to ensure that there is a suitable approach slope from the road on each side to the drift in the bottom of the river bed. Guidance on the design of approach ramps is given in Section 6.6.5.

Although vehicles may not be able to cross the drift during periods of high water it is essential that the drift slab extends beyond the highest flood level to ensure that scour and erosion will not take place at each end of the drift. It may, therefore, be necessary to construct the drift slab to the top of the river banks at the end of the approach slope.

To reduce the cost of construction it may be possible to reduce the width of the drift slab so that it is narrower than the normal road width. Vehicles would not be able to pass each other on the drift so the designer must ensure that there is sufficient space on each side of the drift to allow vehicles to wait. To prevent vehicles driving off the drift and possibly getting stuck in the soft or loose river bed, or vehicles attempting to pass each other on the drift, guide stones should be placed along the edges of the approaches and across the drift.

The width of the central or flat middle section of the drift should minimize disturbance to the water flow. The construction of the road will cause a larger amount of water to flow across the drift due to water flowing off the road along the side drains. Drifts should be constructed with the central flat sections of the following length:

- River crossings: width of the watercourse.
- Relief and perennial stream drifts: width of the dry bed with minimum dimension of 2 meters.

Figure C.6.12: Drift with flow in stream

Figure C.6.13: Drift with low water level

6.8.2 Drift slab construction

There are four possible solutions for constructing the drift slab, in descending cost:

- Concrete slab;
- Cement bonded stone paving;
- Dry pitched stone paving; and
- Gabions with gravel or broken stone.

The main factors affecting the choice of construction method are:

- The nature of the river bed;
- The expected volume and flow rates of the water;
- The availability of different construction materials; and
- The cost of labor.

If large volumes of fast flowing water are expected it is necessary to use a concrete slab or cement bound stone paving as the water will erode gravel and dislodge hand pitched stones. In cases of slower flowing water or small streams, hand pitched stone or gabions are likely to be acceptable and a cheaper option.

6.8.3 Concrete slab

Although concrete slabs are expensive, they are a long lasting, low maintenance solution. The concrete slab should extend the full width of the drift between the cut-off walls with a minimum thickness of

200 mm. In areas where stone is locally available 'plums' may be put in the slab to reduce the amount of cement required and hence reduce the overall cost. Where plums are used, they should not have a dimension greater than 75 mm (100 mm where the slab is 300 mm or thicker) and should be placed as far as possible in the middle of the slab. The slab should be reinforced with a light steel mesh to control cracking in the concrete.

6.8.4 Cement mortar bonded stone paving

Stone paving offers a cheaper alternative to a concrete slab in areas where masonry or locally manufactured blocks of sufficient strength are available. The slab should be a minimum of 300 mm thick which may require more than one course of paving to be laid. The blocks should be laid in an arrangement to ensure that the different courses interlock with each other.

6.8.5 Hand pitched stone

In areas where masonry stone is widely available this option is likely to be cheaper than constructing a concrete slab. However, it is only suitable for low velocity flows and can take a considerable length of time to construct for larger crossings. It is essential that the stones are well placed to ensure that they are interlocked to prevent them being washed out by the water. The whole structure can be washed away if the water can wash out one stone, as this weakens the remaining structure. Larger stones are better than smaller ones as they are less likely to be washed away. The best stones to use are angular and flat faced and should be placed on their edge, to give the greatest interlock between stones. Hand pithed stone drifts are only suitable on very low traffic roads.

6.8.6 Gabion and gravel

This option is the cheapest and quickest option for constructing a drift. The drift basically consists of a gabion basket on the downstream side which acts as a dam to prevent the gravel being washed away. Smaller stones may be used in the gabion than for hand pitched stone and maintenance does not require specialist skills. However, the gravel may not be able to withstand large flows of water.

Where gravel may be washed away, but there is a reasonable amount of gravel in the riverbed, it may be possible to protect the riverbed and trap gravel and sand in the top of a gabion mattress to create a vehicle running surface. Gabion mattresses are similar to gabion baskets except that they are a flatter section; usually 250-300 mm deep and cover a wider plan area. Sand and gravel will tend to be trapped on the top of the gabions which will reduce wear of the wire by traffic.

An additional measure to stabilize the face of the gabion and the retained material is to insert natural fiber matting in the top and face of the gabion. This also encourages vegetation growth for improved stabilization.

6.9 Bridge design

6.9.1 General

A bridge provides passage across a river, valley or other gap across the road without closing the gap. It is not possible to provide a standard design for a bridge since all are unique to the site. Bridge design is a specialized area and requires the expertise of a Bridge Engineer to carry out detailed design of the bridge elements. A bridge is basically an extension of a road, although a more sophisticated and expensive part. At a cost of up to 100 times or more than that of an equivalent length of road, it is important that careful attention be paid to its design and construction. Bridges are critical elements of the road system. A bridge collapse not only disrupts the serviceability of the whole of the road network but can also endanger life to a much greater extent than other components of the road. The possible consequences of structural failure must be considered into account and given due emphasis in the design process. The main components of a bridge structure are:

1. Foundation;
2. Abutments and piers;
3. Bearings;
4. Decking which may be made of slab or timber, beams, girders etc.; and
5. River training works (wingwalls, aprons, etc.).

Bridges found on LVRs are often for the purpose of major river crossings. The key design considerations are bridge location, hydraulic performance (which involves a study of estimated capacity of the water opening, flood elevations and span length), and channel protection.

6.9.2 Location of bridges

The following are guidelines for the location of bridges:

1. Where re-alignment is needed, the proposed alignment of the bridge should be as close as possible to the general direction of the road.
2. The section of the river at the crossing should have suitable geological conditions for construction of supports (abutments and piers) and approach embankment. Rock or another hard surface close to the riverbed level is desirable.
3. Avoid design in locations where the river is undergoing change.
4. The crossing should be at the narrowest section of the river and the smallest width of flood plain if possible.
5. The axis of the bridge should be normal to the river flow.
6. Avoid conditions which require excessive underwater construction.
7. Avoid locations where expensive river training works are needed.
8. Avoid conditions which require sharp curves in the approaches.

6.9.3 Bridge hydraulics

Flood estimation

The following guidelines should be used in the selection of the design flood for road bridges:

- The 25 to 50-year frequency flood should be the design flood for feeder and secondary roads, depending on the importance of the road.
- The 10-year frequency flood should generally be used as the design flood for temporary bridges.

The discharge is analyzed by using the area of the bridge cross section filled by the design flow called the “discharge section” or the “active channel section”. This area is determined by:

Area (A) = Design flow (Q) /Average velocity of the river in channel before construction (V)

This area is adjusted upward by a factor to take consideration of the reduction of space by the construction of piers and the space where stream flow is disturbed by eddies around these locations. The overall open space then takes account of the desired freeboard.

The flood elevation can be determined by the method of channel analysis if the cross section profile of discharge area is known. It can also be obtained by visual assessment on site. The space between the design flood elevation and the lowest part of the superstructure must allow for backwater effects and the floating debris. Table C.6.10 provides a guide for vertical clearance for bridges. These clearance measurements should be increased on rivers with a history of unusually large floating items.

Table C.6.10: Vertical clearance from design flood level

Discharge (m^3/sec)	Vertical clearance (mm)
< 0.3	150
0.3 to 3	450
3 to 30	600
30 to 300	900
> 300	1200

6.9.4 Channel Protection

Scour

Scouring is one of the major characteristics of waterways, whether for permanent streams or runoff. It is the term used for removal of bed or embankment materials by stream or tidal currents. It may occur naturally as a result of channel construction or where changes occur in the flow regime (direction or volume of flow). Scour can occur in streams without a bridge or when construction constricts flow, or where changes of flow pattern occur at piers and abutments. Local scour at bridge locations is caused by bends in channels, shapes of the piers, obstruction by piers and abutments. Therefore, a scour analysis should be performed and used in design of foundations for all bridges with a pier in the water way or where the abutments are close to the active channel.

Protection of foundations from scour

Foundations can be protected from scour by:

- Placing the footing of piers below the estimated lowest level of the scour;
- Providing protection against local scour by using rip-rap protection or gabion mattress;
- Using piles or columns under the piers and protecting against local scour;
- Protecting abutments that sit close to the channel with rip-rap protection; and
- Protecting embankment slopes adjacent to structures subject to erosion with stone pitching or rip-rap protection.

Guidance on estimating scour depth can be found in Overseas Road Note 9 “A Design Manual for Small Bridges”.

6.9.5 Bridge structural design considerations

Load specifications

For major bridge projects on higher standard rural feeder roads, where the commercial traffic level is relatively high, and the bridge is part of a permanent structure, standards of load specifications should be adopted in line with international practice (until local standards have been set following extensive studies).

For minor bridges, load specifications for a standard axle load of 8.16 tonnes (metric) may be used.

Loads to be considered

For design of road bridges, the following loads, forces and stresses should be considered where applicable:

1. **Dead load**: The designer must calculate the weight of all components of the bridge. This must include loads applied by sidewalks and utilities. An allowance of 10-15% may sometimes be applied for such details as bolts, rivets, appurtenances, paintwork etc. Some typical values of unit mass useful in dead weight calculations are given in Table C.6.11.

Table C.6.11: Typical values of unit mass

Material	kg/m^3	lb/ft^3
Steel	7850	490
Concrete (plain or reinforced)	2400	150
Loose sand	1600	100
Macadam	2240	140
Guard rails, fastening	3200	200
Timber	560 - 960	35 - 60
Asphalt paving	2240	140

2. **Live load**: Weight of vehicles passing on the road. The designer must apply appropriate live load specifications depending on the road and nature of traffic in consultation with the Chief Engineer.
3. **Sidewalk loading**: Slabs, beam and girders must be designed for sidewalk live loads ranging 140 – 290 kg/m^2 or (30 – 60 lb/sq ft).
4. **Impact load:** Impact loads are due to effect of vehicle breaking. The allowance for impact loads is a function of the live load and for bridges with span range of 3 m – 45 m. It is determined by the ratio:
   - For concrete bridges: 4.5/(6+L)
   - For steel bridges: 9/(13.5+L)
5. **Wind load**: Wind load is applied as uniformly distributed load for only exposed areas. The forces are usually for a base wind velocity of 160 km/hour (100mph). This load can only apply after evaluation of the location of the specific bridge with respect to the general terrain, ground levels and whether high speed winds have been a problem in such location. Wind load may be applicable for low weight structures like long spanning pedestrian bridges, floating bridges, suspended or suspension bridges.
6. **Passive earth pressure:** This is determined using Rankine's formula. Live load surcharge pressure equivalent to 0.5 m of earth added can be considered where there is no rigid approach pavement provided. It is recommended that an approach pavement is provided before entry onto the bridge deck.

   The designer must ensure that there is effective drainage of the backfill material. Drainage is provided by the use of weep holes and crushed rock, perforated pipe drains and gravel.
7. **Seismic stresses:** This can only apply if there is anticipated earthquake pressure. Seismic design is beyond the scope of this manual.
8. **Buoyancy or uplift:** The designer must make provision for adequate attachment of the superstructure to the substructure without restricting the ability of the superstructure in responding to lateral stresses caused by aspects like temperature changes.
9. **Temperature stresses**: The designer must ensure provisions are made for movements resulting from temperature changes. This is done by providing expansion gaps.

Some considerations for the design of a bridge

The bridge designer should pay attention to the following:

- The class of road, traffic and selection of appropriate load;
- The type of materials to use and type of materials available;
- The general terrain, geology around the crossing, depth of stream and profile of stream;
- The cost of the bridge;
- The aesthetic aspects of the bridge; and
- The feasibility of the project.

Steps in structural design of a bridge

The steps below provide some good practices to be followed in carrying out design of a concrete bridge. The designer can carefully adjust the design steps for use on composite bridges involving the use of two or more materials (e.g. steel beams, concrete beams, timber beams, timber decks, concrete decks).

1. The bridge type is selected, and a proposed scheme made, with associated sketches.
2. Choose type of concrete and steel reinforcement.
3. Calculate the dead load of each element of the bridge.
4. Consider the type of traffic and live load.
5. Calculate the total load (live load and dead load).
6. Calculate shear force, shear stress, and maximum moment on the slab.
7. Calculate shear force, shear stress and maximum moment on beams.
8. Check and control the overall depth of the slab and beams.
9. Check and control stresses in the steel reinforcement (tension, compression and distribution bars and stirrups).

10. Check and control the development length and bond stress.
11. Conduct an overall check of each step.

The design of bridges is specialized field and the design should only be undertaken by individuals with the required qualifications and experience. The consequences of the failure of a bridge are high. Futher guidance can be found in Overseas Road Note 9 “A Design Manual for Small Bridges”. If there are any doubts about the design of a bridge the engineer should seek guidance from the MPW.

6.10 Sedimentation and erosion control in channels

6.10.1 Overview

If a drainage structure is properly designed, there will be little or no erosion. Culverts and other drainage channels must be designed for velocities that will not cause scouring and will make the structure self -cleaning to prevent sedimentation. Where these velocities can’t be controlled, then additional protective measures must be provided. There are various methods of reducing erosion, the most common being to build simple scour-checks in channels, lining of channels with resistant materials, providing catch pits for water detention, use of stone mattresses, and use of rip-rap-protection.

The simplest way of controlling erosion on road projects is by avoidance. This can be achieved by:

- reducing the area of ground that is to be cleared;
- quickly replanting cleared areas, maintaining the planted areas and specific bio-engineering measures;
- avoiding erosion sensitive alignments; and
- controlling the rate and volume of water flows in the area.

6.10.2 Scour checks

Scour checks (sometimes called check dams) reduce the speed of water in a drain and help prevent it from eroding the road structure. Scour checks must be provided in longitudinal drains with gradient steeper than 4%. They should not be provided on flatter slopes as they cause excessive sedimentation in the drain. The top level of the scour check should be a minimum of 200 mm below the edge of carriageway. The distance between the scour checks depends on the road gradient and the erosion potential of the soils.

Table C.6.12 provides a guide for scour check spacing.

Table C.6.12: Scour check spacing

Drain gradient, S (%)	Scour check spacing (m)
5	20
6	15
7	10
8	7.5
9	6
10	5
11	4
12	4

The scour check acts as a small dam. when naturally silted up on the upstream side it effectively reduces the gradient of the drain, and therefore the velocity of the water. Scour checks are usually constructed with natural stone, masonry, concrete or wooden or bamboo stakes. By using building materials available along the road, they can be constructed at low cost and be easily maintained.

After the scour check has been constructed, an apron should be built immediately downstream using stones. The apron will help resist the forces of the waterfall created by the scour check. Sods of grass should be placed against the upstream face of the scour check wall to prevent water seeping through it and to encourage silting on the upstream side. The goal is to establish a complete grass covering over the silted scour check to stabilize it.
An example of a scour check is shown in Figure C.6.14. Typical design details for scour checks are given in Figure C.6.15. and Figure C.6.16.

Figure C.6.14: Scour check made from wooden stakes

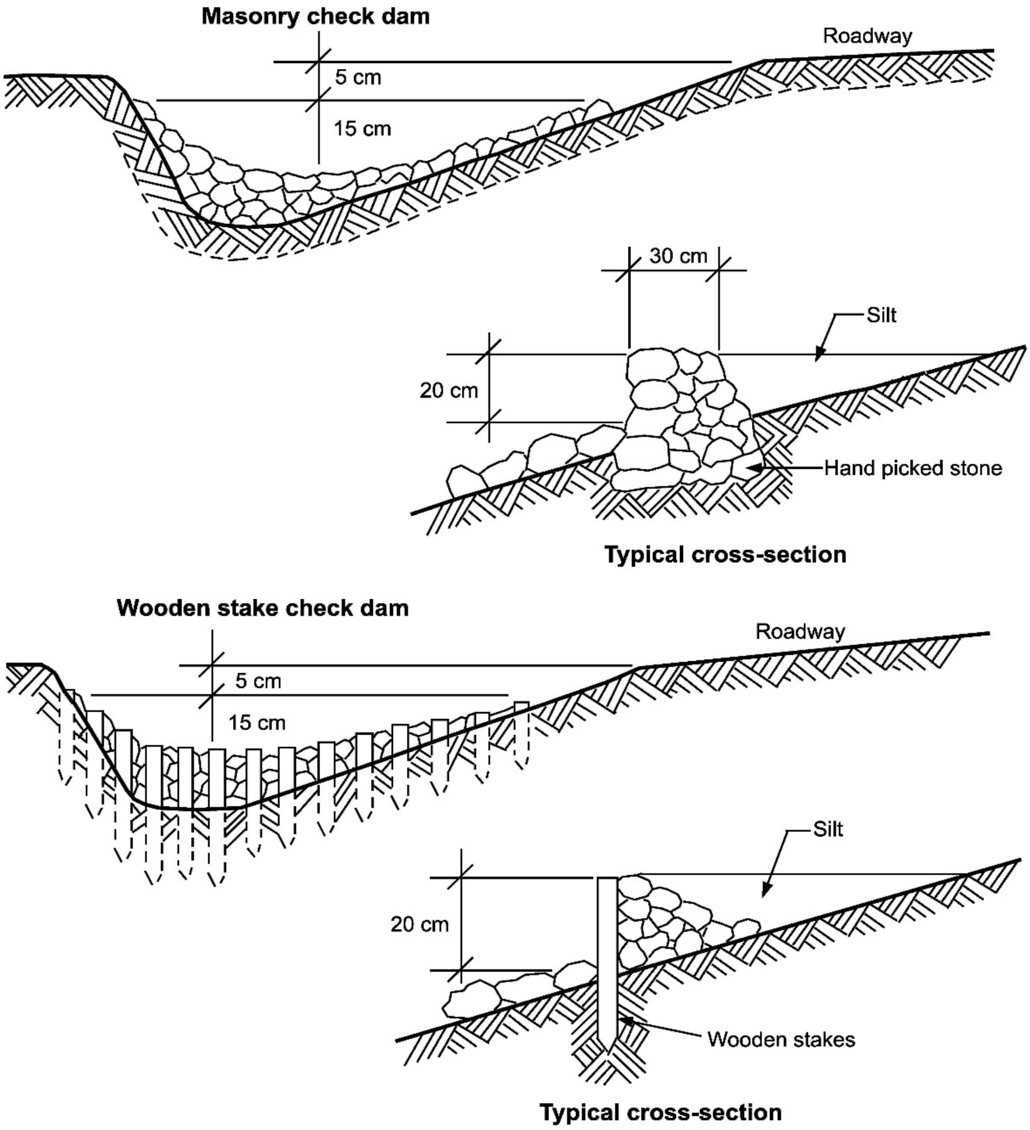


Figure C.6.15: Typical design of scour check

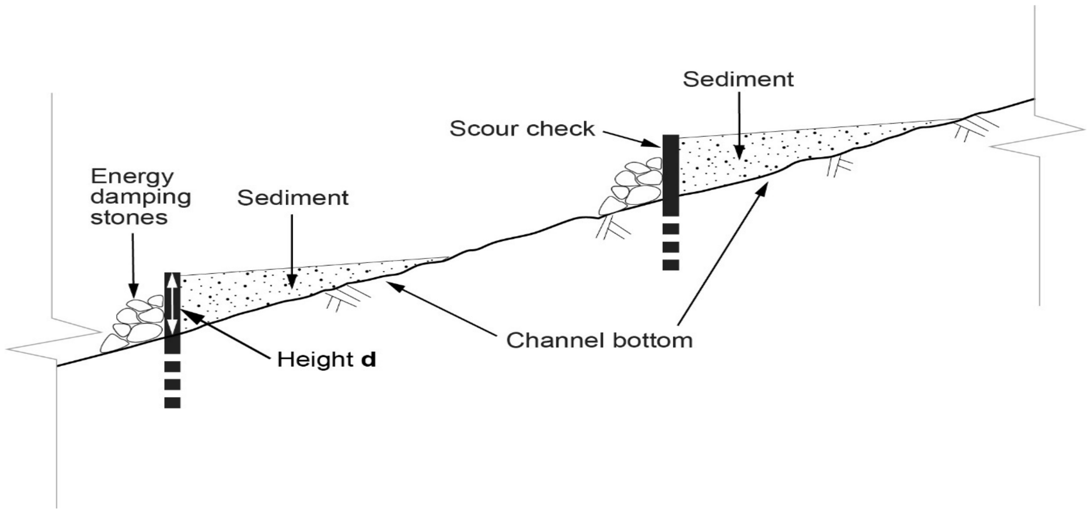


Figure C.6.16: Typical section through a channel with scour checks

6.10.3 Replanting

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

The engineering functions of vegetation in erosion protection measures include:

- retaining material from moving over the soil surface;
- armoring the surface against erosion and abrasion;
- reinforcing the soil by increasing its effective shear strength; and
- draining the soil profile by taking water into the roots.

6.10.4 Slope protection

Avoiding erosion by stabilizing slopes requires good engineering design of the slope form and drainage. Chapter 7 on roadside slope stabilization describes this approach in detail.

6.10.5 Sedimentation control

Deposition of silt in drains and culverts can eventually block them. The blocked drains and culverts may then contribute to the flooding and waterlogging problems. The following steps can minimize the risk of siltation of drainage structures:

- Ensure a minimum flow velocity of 0.6 m/s to ensure self-cleansing of the drainage structure - where possible ensure a flow of 1.0 m/s;
- Ensure that the culvert is set at the right level: too low will cause silting to occur;
- Where the minimum velocity is not attainable, silt traps should be introduced to minimize the amount of silt entering the structure (a typical silt trap is a catch-pit with a sump of at least 150 mm deep);
- Ensure regular desilting; and
- Avoid sharp turns in drains and areas of “dead” water: silting is common where drop inlets are provided for culverts.

7. ROADSIDE SLOPE STABILIZATION

7.1 Introduction

Failures of natural slopes, cut slopes, and embankments in hilly and mountainous areas disrupt traffic flow and create a considerable problem to road users during the rainy season. These failures typically occur where a natural slope is too steep, a cut slope in soil and/or weathered rock contains weak materials or adverse joints or where fill material is not properly compacted or is constructed too steeply. In all cases, a rise in groundwater, or temporary saturation of surface soil layers during heavy rain, will lead to an overall reduction in stability and possibly failure.

Figure C.7.1: Slope failure along adverse rock joints (arrow indicates joint surface)

Slope instability affecting the Right of Way usually occurs either above or below the road, though in Liberia the vast majority of slope problems relate to shallow failures and erosion above the road. Instability below the road can affect both fill slopes and the natural ground below and beneath them. In rare instances, the entire carriageway may be affected by deeper movement involving the slopes below and above the road and the ground upon which the road is constructed.

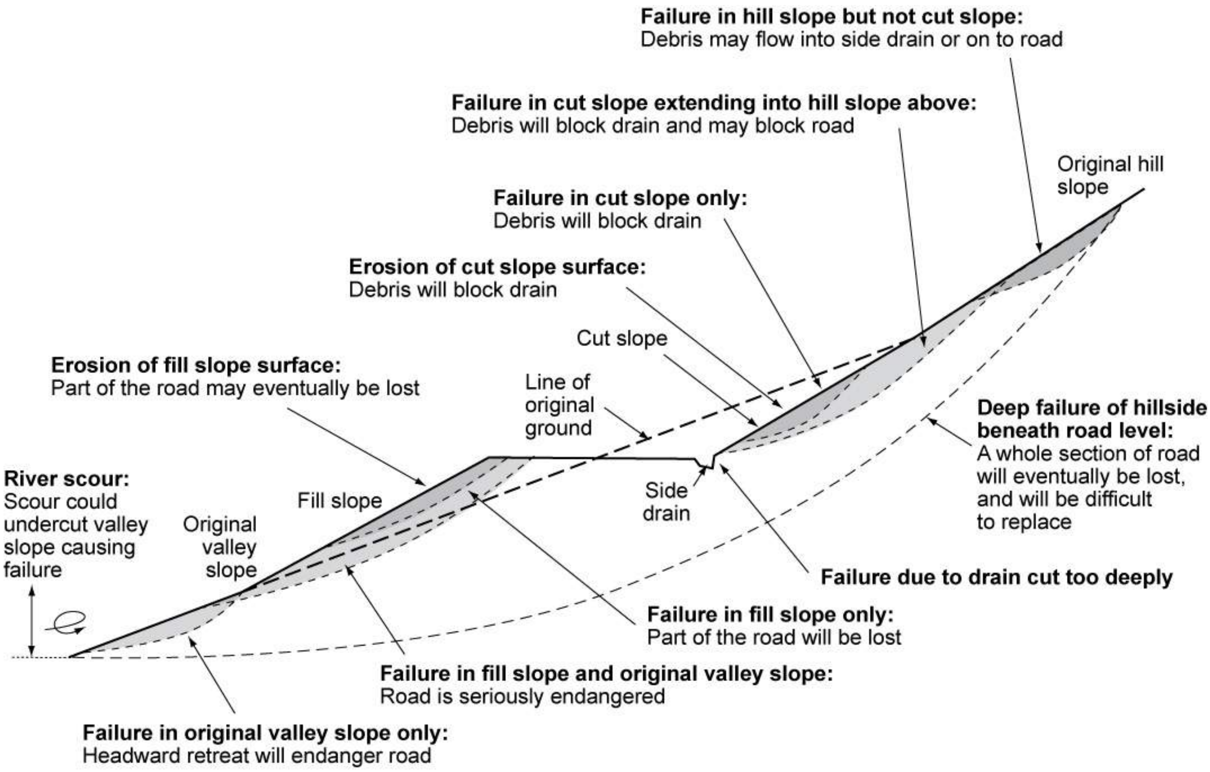


Figure C.7.2: Slope instability affecting roads in hilly and mountainous terrain

7.2 Slope instability above the road

7.2.1 Causes and mechanisms

The types of slope failures and erosion problems commonly observed above a road alignment include:

- Erosion of cut slope surfaces;
- Shallow failures in the cut slopes; and
- Deeper failures in cut slopes and hillslopes.

Once a cut has been formed the slope is exposed to erosion. This may lead to the formation of gullies, and ultimately to slope failures. Erosion is especially common on cuts in weak and deeply weathered rocks. The residual soils found on many hillsides in Liberia are vulnerable to erosion in steep excavations without protective vegetation cover. Failures in cut slopes usually occur due to a change in slope geometry, progressive weathering or the introduction of water, either through land use or very heavy rainfall. Many of the failures in cut slopes occur in the form of minor slips. Sometimes, however, these small slips may develop into larger failures, affecting a much larger area. These failures often develop at the toe of the slope and propagate upslope. Most failures affecting the natural hillside above roads are caused by the removal of support in the road cutting, combined with the effects of groundwater rise during heavy or prolonged rainfall.

Notwithstanding the above, it is also common to find that slope excavations expose an upper weathered mantle of perhaps 1-3 meters depth overlying less weathered rock. The mantle is inherently weaker but is often incorrectly cut to the same angle as the stronger material beneath it. During heavy rainfall this upper section of the cut slope is prone to shallow failure (Figure C.7.3) and, given its location at the top of the excavation, it is difficult to treat. It is better, therefore, to allow for this in the earthworks design (Section 7.2.2).

Figure C.7.3: Failure of the upper portion of the cut slope in the weathered mantle

Deep-seated failures are defined as those that involve bedrock. However, where the depths of tropical weathering are high, from an engineering perspective, they can be regarded as failures deeper than 5 meters. Slope failures involving rock often take place along adverse joints and may be triggered by the build-up of water pressures within the jointed rock. However, the majority of large landslides usually occur in the weathered mantle above less weathered rock (Figure C.7.4).

Figure C.7.4: Slope failure in weathered mantle

7.2.2 Prevention and mitigation

Cut slope design

Cut slopes should be designed according to the required height of the excavation and the strength of the materials exposed. However, there are many other factors that will exert control on the final geometry. Cut slopes that are only a few meters in height are likely to expose overburden soils and weathered mantle that may only be stable at relatively shallow angles. As the depth of cutting increases, stronger materials will theoretically be exposed that will stand at steeper angles. Consequently, a compound slope design might apply, with a steeper lower segment and a gentler upper segment. Conversely, however, deeply-weathered slopes are common in the humid tropics, and it may be the case that excavations do not expose rock until several meters beneath a weathered soil cover.

Groundwater levels and slope drainage will also exert a significant influence on the stability of the excavation, and it may be necessary to design the earthworks accordingly or make provision for drainage measures (Section 7.5). In-built up areas and areas of highly productive agricultural land there will be a desire to limit the land-take by designing steep excavations, possibly supported by retaining walls.

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

One of the simplest ways to decide upon a suitable cut slope design 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. Any cut slope where failure would result in large rehabilitation costs or would threaten public safety should be designed using more rigorous techniques. Situations that warrant engineering geological assessment include deep cuts, cuts with complex geological structure (especially if weak zones are present), cuts where high groundwater or seepage pressures are likely, cuts involving soils with low strength, cuts in landslide debris, and cuts in rock formations susceptible to landslides. Cut slopes in colluvium or landslide debris are especially vulnerable to failure if cut too steeply (Figure C.7.5).

Figure C.7.5: Colluvium/landslide material

For LVRs it is important to minimize earthworks and retaining wall costs. Table C.7.1 shows the range of recommended cut slopes for weathered rock, residual soils and transported soils commonly encountered in excavations. The recommended cutting angles are based on weathering grades that range from I (fresh rock) to VI (residual soil) according to the classification shown in Figure C.7.6. Note that the cutting angles do not take account of any adverse jointing, either in rock or residual soil profiles. These need to be assessed on a case-by-case basis to ensure that rock masses do not become destabilized along dominant adverse joint sets.

The cutting angles provided in Table C.7.1 should be used as a provisional guide, with variations made according to materials and drainage conditions and the history of cut slope stability in the local area.

Table C.7.1: Cut angles for rock, residual soil and transported soils

Material type	Weathering Grade	Slope face angle (degrees)	Slope face gradient (vertical : horizontal)
Competent rock	I - II	80 - 85	6:1 to 10:1
Weathered rock	III - IV[1]	60 - 75	2:1 to 4:1
Coarse-grained residual soil	V - VI	45	1:1
Fine-grained residual soil	V - VI	35[2]	1:1.5 to 1:2[2]
Coarse-grained taluvium and river terrace deposits	n/a	40	1:1.2
Fine-grained colluvium	n/a	35[2]	1:1.5 to 1:3[2]

[1] weathering grade IV is borderline soil/rock.

[2] depends on clay content and water table.

Source: Hearn (2011).

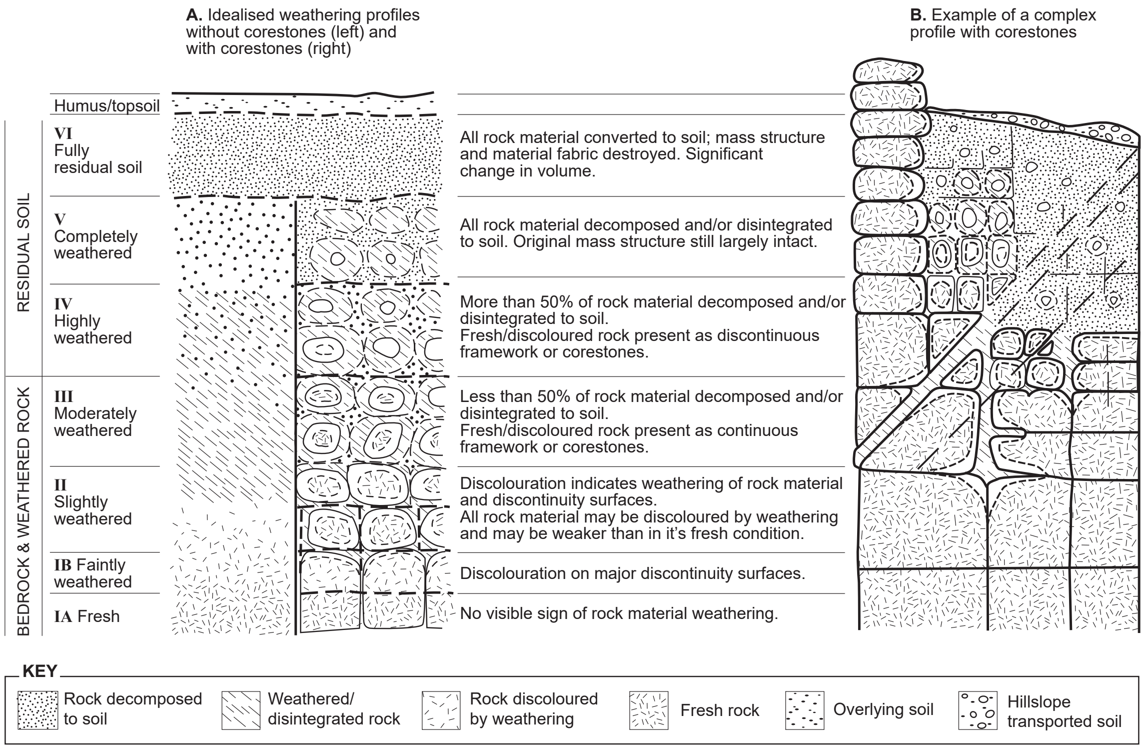


Figure C.7.6: Weathering Grade Classification

Source: Modified from Fookes (1997) and reproduced from Hearn (2011).

Despite the recommendations in Table C.7.1, red-brown residual soils are often cut to much steeper angles, sometimes up to 3V:1H or more and they remain stable for considerable periods of time (Figure C.7.7 and Figure C.7.8).

The box below summarizes some of the findings of a study of cut slopes along a 270 km-long railway in Liberia in 1962 reported in Aid (1971), which is assumed to be the Buchanan-Nimba County iron ore railway constructed by LAMCO between 1960 and 1963. The study illustrates the significant variability in the depth of weathered soils in the country and the instability that can result when they are cut too steeply.

Early experience in cut slope design and stability along the LAMCO Buchanan-Nimba County railway

Some cuttings were up to 28 m deep before bedrock was encountered. Soils were predominantly ferralitic (tropically-weathered soils dominated by hydrated oxides of iron and aluminum). Slopes up to 30 ft (9 m) were cut to approximately 2.V:1H (60-65°). Cut slopes deeper than this were provided with a 7 ft (2 m) wide bench at 23 ft (7 m). Additional benches were provided at 23 ft intervals, as required. 20% of slopes higher than 33 ft (10 m) apparently failed during construction. One slope that was 72 ft (30 m) deep failed and had to be cut back to 1V:1.25H (39°). Relic joints within the ferralitic soil and groundwater seepage were frequent causes of instability. Slopes cut to less that 1:1 were subject to erosion. Sandy silty soils were found to be 5-10 times more erodible than lateritic gravel soils.

The highest concentrations of iron and aluminium oxides give rise to laterites, a residual ferruginous material generally occurring as a strong ferricrete that resembles rock. Lateritic rock and welded laterite gravel can stand steeply in roadside excavations (Figure C.7.7 and Figure C.7.8).

Figure C.7.7: Cut slope in WG V-VI material at 3V:1H and still intact after 40 years

Figure C.7.8: Welded laterite gravels standing steeply in roadside cut slope

The observations made along the LAMCO railway illustrate how variable soil conditions can be and how stability can be significantly influenced by groundwater condition and relict joints. These observations, including those relating to erosion, should be borne in mind when designing excavations. Normally, the strength of in situ soils is greater than that determined from laboratory testing, and it may be prudent to experiment with steeper slope angles to see how they perform over a wet season before finalizing the earthworks design.

For deep excavations in rock (perhaps deeper than 5-6 m), it is common to bench the cut slope. Table C.6.2 lists some of the advantages and disadvantages of benched profiles. The maximum bench height and the minimum bench width should normally be 6 m and 2 m respectively, though narrower benches (minimum 1 m width) are sometimes used. The advantages of a benched rock slope include the ability to control drainage on the slope (an inward gradient of 3-5% is usually provided) and to confine small failures to individual benches, thus reducing the quantities of debris that reach road level. In hard rock effective drainage can be achieved via an excavated channel at the back of each step. In permeable and erodible rock, a lined channel (mortared masonry or concrete) is required. Runoff will need to be conveyed to road level via chutes and cascades. Any debris that blocks these drains or any deformation or cracking to the drains themselves can result in uncontrolled runoff, seepage, erosion and localized slope failure. Therefore, if benches are used, they must be regularly inspected and maintained. Benches are not recommended for very weak rocks, colluvium and other debris slopes. Cut slopes in residual soils can be benched (Figure C.7.9) if they are provided with sufficient drainage to prevent softening of the soil. Outward-sloping benches are not recommended in residual soils as they will tend to concentrate runoff on the cut face below, encouraging erosion.

Figure C.7.9: Benched cut slope in residual soils

Table C.7.2: Advantages and disadvantages of benched cut slopes

Advantages	Disadvantages
▪ Benches slow down the rate of surface runoff, and therefore reduce surface erosion. ▪ Benches permit the construction of mid-slope longitudinal drains much more easily, and these can form part of an overall slope drainage system. ▪ Where excavation is to be undertaken in softer materials, such as weathered rock, benching can help prevent long erosion furrows from developing by interrupting and controlling the flow of surface runoff. ▪ Shallow failures are usually limited to one bench at a time. ▪ Shallow failures are usually contained on the bench below and are thus often prevented from reaching the road. ▪ Benches offer advantages in terms of access for drilling equipment and excavation plant. ▪ Benches permit access to the slope face for maintenance purposes.	▪ The cut faces in a benched slope profile are steeper than a continuous slope cut to the same overall angle. This may encourage localized failures to occur in soft materials and may create conditions of instability in adversely-jointed rock that might otherwise not occur. ▪ Conversely, if the risers of a benched cut slope are cut to the same slope as a continuous cut the overall height of cut will usually be greater. ▪ Vegetation is less easy to establish on a benched slope profile to the same overall angle (i.e. where steeper risers are required between benches). ▪ Defective bench drainage systems due to erosion or blockage can lead to uncontrolled rainfall runoff and concentrated erosion that ultimately leads to slope failure. ▪ Benches are nearly always inadequately maintained on low cost roads as they are not of primary concern to road maintenance crews. This can quickly lead to drainage failures.

Source: Hearn (2011)

7.3 Slope stabilization above the road

7.3.1 Soil slopes

If a cut slope or natural hillside above the road fails, there are several measures that can be taken. If access and land availability permit, the debris can be removed, and the slope cut back to a reduced angle, thus increasing its stability. If this is not possible then drainage measures can be introduced to lower the water table, though drainage measures on their own may be insufficient. If this is the case, then retaining walls and revetment walls will need to be considered. These can be designed to either retain the soil mass or to provide protection against erosion and shallow failure. It is important to know the depth and mechanisms of instability before designing walls to support or protect a failed or eroding slope. Walls are usually constructed from gabion or mortared masonry, but mass concrete and reinforced concrete structures are sometimes used. It is normal practice to combine retaining structures with slope drainage if groundwater or surface soil saturation is one of the principal causes of slope instability.

Figure C.7.9 illustrates forms of slope instability affecting mountain roads.

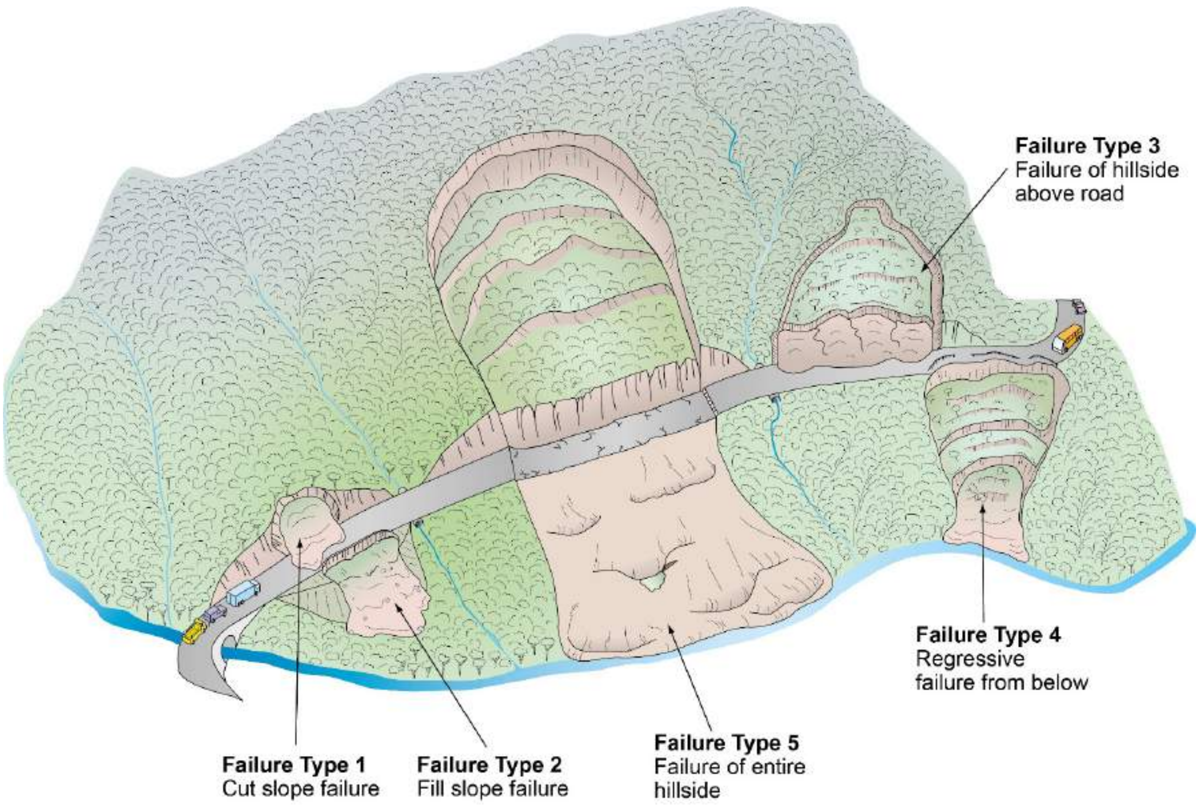


Figure C.7.10: Some common and less common forms of slope instability

Source: Hearn (2011).

Most slope stability problems relate to shallow landslides in cut slopes (failure types 1 and occasionally 3). Table C.7.3 outlines some typical slope management responses to these hazards. For existing roads, the three right-hand columns apply (*stabilization*, *protection*, or *accept*). Techniques of stabilization and protection are summarized in Table C.7.4. For LVRs, the priority is to find low cost solutions, maximizing the use of surface drainage and earthworks and locally-available materials, for example through the construction of gabion and masonry walls (Section 7.6), and the use of bio-engineering measures (Section 7.7). Most cut slope failures are shallow, and the most common forms of treatment comprise removal, trimming and drainage. Bio-engineering should be considered as an integral part of the solution. The option to accept the hazard should only be taken when the cost of stabilization or protection is too prohibitive compared to the risk posed by the hazard itself. For example, a slow-moving landslide on an LVR may be easily manageable by refilling and regrading compared to the cost of attempting to stabilize it.

7.3.2 Rock slopes

Figure C.7.11 shows a range of measures commonly used to stabilize or mitigate rock fall and rock slide hazards along mountain roads. For LVRs, prescriptive codes 11-16 (right hand column) may be the only practical options based on cost considerations.

Where adversely-orientated rock joints are steep-dipping the simplest approach will be to reduce the slope of the excavation so that failure cannot take place along these joint sets. This requires land to be available, which is not always the case. Furthermore, cutting the slope back to too shallow an angle creates a large quantity of excavation materials to be disposed of and may then create problems of erosion in the freshly-cut slope surface (though this shouldn't be so much of a problem if the main slope material is rock). Where adversely orientated rock joints are moderately dipping (perhaps 40° or less) a simple earthworks solution may not be practicable. The most likely options in such situations are described below.

Where the hazard is from falling rock:

- Either remove the unstable rock or realign the center-line away from the cut slope to create more room so that a space can created for rocks to fall into without impacting the carriageway.
- In combination with the above, or alone, construct a rock trap wall. If space permits, then gabion is the best option as it absorbs impact without failure. If space does not permit, then a reinforced concrete rock trap wall may be a suitable alternative.
- Hang wire netting over the slope, anchored into the natural ground above the cutting to control the fall of rock debris.
- Protect the rock face using shotcrete (wire mesh and concrete) bolted into the rock face.

Where the hazard is from sliding rock:

- Construct a retaining wall, sufficient to retain the rock mass.
- Use rock dowels or bolts to effectively reinforce the rock mass by anchoring surface rock layers into underlying stable rock layers.

For new roads the road should be located away from hazardous areas such as these. In the case of existing roads, it may be easier and cheaper to carry out local realignment. If this is not possible then expert guidance should be sought before deciding on the best course of action. Until a decision is made, it may be necessary to close the road during and immediately after heavy rain, and prevent any road widening through additional excavation into the hillside.

Table C.7.3: Slope hazard management options

Failure type	Engineering management				
	Avoid	**Remove**	**Stabilize**	**Protect**	**Accept**
1	These failures are often triggered by slope excavation and therefore avoidance during route selection is usually not an option.	Removal of slipped debris is an option in the case of the smaller failures if the remaining slope is stable and can be protected against erosion.	Can be achieved usually through earthworks, drainage and retaining structures.	Catchwalls or fences may be provided to protect road from falling debris	Small failures onto the road can be accepted if source removal, stabilization or protection is too difficult or costly compared to the damage inflicted.
2	Shift the road into hillside to avoid unstable fill slope below. However, this may initiate type 1 and 3 failures.	Not usually practicable where large fill slopes are involved. Also, either partial or complete removal will result in loss of road width.	Through excavation and recompaction, improved drainage or by the use of retaining structures founded beneath failure surfaces.	Construction of road edge and road fill retaining walls founded beneath failure surfaces may isolate the road from the failing fill slope.	Ongoing movements that cause progressive loss of all or part of the road cannot usually be accepted without loss of road function.
3	Often caused by slope excavation. Avoidance during route selection is usually not an option. In the worst cases, where landslides frequently cause road blockage, realignment might be cost-effective in the longer term, if a suitable alternative exists.	Not usually practicable given large volumes, access difficulties and uncertainties over the stability of the remaining slope.	May not be practicable or economically feasible to achieve stabilization in the case of the larger slope failures, though improvements can be achieved through earthworks, drainage and retaining structures.	Catchwalls or fences may be provided to protect road from rock fall debris, but these are unlikely to be appropriate for soil slope failures.	It is usually only feasible to accept ongoing movements if these are slow and can be accommodated by maintenance.
4	Avoid through alignment selection (new roads) or realignment (existing roads) if a suitable alternative exists. Roads are often shifted into the hillside to avoid developing problems below. However, this may initiate type 1 and 3 failures.		If the slope failure is local to the road, then stabilization by retaining structures and drainage may be possible. though unlikely at low cost.	Construction of road edge and road fill retaining walls founded beneath failure surfaces may isolate the road from the slope failure below.	Ongoing movements that result in the progressive loss of all or part of the road cannot usually be accepted without loss of road function.
5	Avoid through alignment selection (new roads) or realignment (existing roads) if a suitable alternative exists.		Stabilization of large landslides is usually beyond the scope of low cost roads.	Road cannot usually be protected against ground movements.	It is usually only feasible to accept ongoing movements if these are slow and can be accommodated by road surface and drainage repairs. Consider a gravel road surface.

Source: Hearn (2011).

Table C.7.4: Stabilization and protection options for slopes above the road

Instability	Stabilization options	Drainage options	Protection options
1. Erosion of the cut slope surface	None	▪ Usually none; ▪ Occasionally a cut-off drain above the cut slope can reduce water runoff; however, these are difficult to maintain and can contribute to instability if blocked or otherwise disturbed.	In most cases, bio-engineering is adequate, usually grass slip planting; Where gullies are long, or slopes are very steep, small check dams may be required; Sometimes a revetment wall at the toe helps to protect the side drain.
2. Failures in cut slope	▪ Reduce the slope grade and if this is feasible, then add erosion protection; ▪ A retaining wall to retain the sliding mass; ▪ For small sites where the failure is not expected to continue, a revetment might be adequate.	▪ A subsoil drain may be required behind a wall if there is evidence of water seepage; ▪ Herringbone surface drains may be required if the slope drainage is impeded.	Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
3. Failures in cut slope and hill slope	▪ Reduce the slope grade, and if this is feasible, then add protection; ▪ A retaining wall to retain the sliding mass. This may need to be quite large, depending on the depth of the slip plane.	▪ A subsoil drain may be required behind a wall if there is evidence of water seepage; ▪ Herringbone surface drains may be required if the slope drainage is impeded.	Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil
4. Failures in hill slope but not cut slope	▪ Reduce the slope grade, and if this is feasible, then add protection; ▪ A retaining wall to support the sliding mass, as long as foundations can be found that do not surcharge or threaten the cut slope.	▪ A subsoil drain may be required behind a wall if there is evidence of water seepage; ▪ Herringbone surface drains may be required if the slope drainage is impeded.	Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil
5. Deep failure in the original ground underneath the road	▪ Consider re-alignment of road away from instability ▪ If slow moving, short term option may be to repave or gravel the road.	▪ Ensure roadside drainage is controlled	Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.

Source: Based on Hunt et al. (2008).

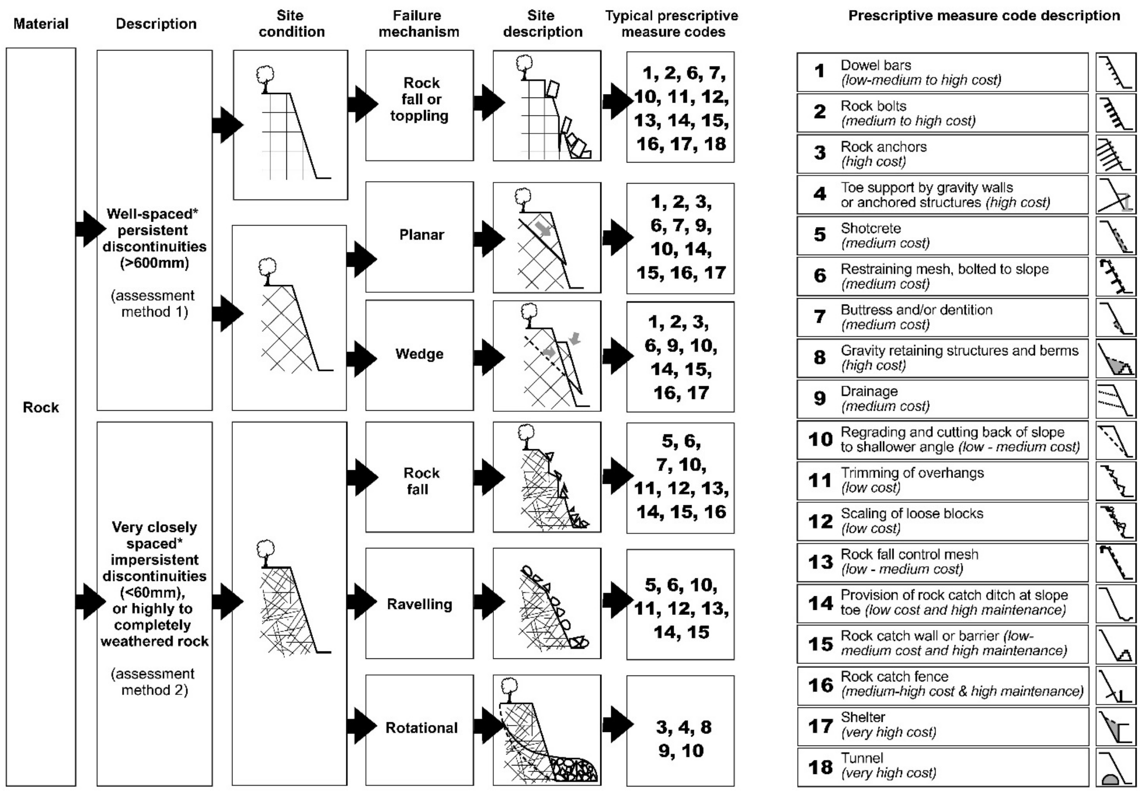


Figure C.7.11: Rock slope materials, failure mechanisms and remedial measures

Source: Hearn (2011).

7.4 Slope instability below the road

7.4.1 Causes and mechanisms

Figure C.7.10 illustrates some of the more serious instability and landslide problems affecting the slopes below some mountain roads. Most problems are associated with failure type 2, i.e. fill slope failures. Other common problems, not illustrated on Figure C.7.10, relate to scour below culverts and side drain turnouts and failure of fill slope retaining walls. Uncontrolled runoff below a road located on sloping ground can quickly lead to incision and shallow slope failure, sometimes causing undermining to below-road retaining and drainage structures if not corrected.

Erosion

Most earth embankments or fill slopes face light to moderate erosion problems arising from rainfall splash and surface runoff from the road. In some cases, agricultural activities can cause problems on fill slopes, especially if irrigated. Rills and gullies can form easily on poorly compacted and unprotected embankment shoulders and slopes (Figure C.7.12). The degree of erosion is normally a function of material type and its compaction, rainfall or runoff intensity, slope angle, length of slope, and vegetation cover.

Figure C.7.12: Severe erosion of embankment material

Slope instability

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

7.4.2 Prevention and mitigation

Fill slope design

When deciding on the fill slope batters for design, it is important to consider the type of material that is going to be used and the ground upon which the fill is to be constructed. Design should also consider the stability of existing fill slopes in the surroundings of the project site. Fill slope batters commonly used are summarized in Table C.7.5. Fill slope batters of 1V:1.5H should be the steepest fill slopes constructed in granular soils. The stability of fill slopes constructed to this angle can be increased by using rock fill as the construction material, and batters of 1V:1.25H or even 1V:1H may be achievable with hand-paced, interlocking angular rock. Fill slopes that are required to be steeper than this, due to topographic and land-take constraints, will require support by retaining walls (Section 7.6). The Feeder Roads Design Manual (MPW 2012) recommends embankment side slopes of between 1V:2H and 1V:4H, with the slacker gradient allowing the adjacent embankment slope to be combined with the provision of shoulders.

In general, for LVRs, fill slopes 3 m or less in thickness, with 1V:2H or flatter side slopes, may be designed based on the strength characteristics of a compacted free draining granular fill and engineering judgement. This is provided there are no problematic soils present, such as expansive or collapsible soils, organic deposits, or soft and loose sediments beneath the fill slope. Fill slopes over 3 m in thickness, or any embankment to be constructed on soft soils, in unstable areas, or those comprising light weight fill, require site-specific engineering geological assessment. Moreover, any fill placed near or against a bridge abutment or foundation, or that might impact on a nearby structure, will likewise require design by a specialist engineer.

Table C.7.5: Recommended fill slope angles

<table>
<tr><th rowspan="2">Fill Material</th><th colspan="2">Fill slope height</th></tr>
<tr><th>< 5 m</th><th>5-10 m</th></tr>
<tr><td>Rock fill</td><td colspan="2">1V:1.5H</td></tr>
<tr><td>Well-graded sand, gravel and sand or silt mixed with gravel</td><td>1V:1.5H</td><td>1V:2H</td></tr>
<tr><td>Poorly-graded sand</td><td>1V:2H</td><td>1V:3H</td></tr>
<tr><td>Sandy clay soils, silty clay and stiff clay soils (excluding expansive clay)</td><td>1V:3H</td><td rowspan="2">Not recommended</td></tr>
<tr><td>Soft clay/plastic clay (excluding expansive clay)</td><td>1V:3H</td></tr>
</table>

NB the use of expansive clays is not recommended; slopes may need to be modified for traffic safety reasons.

Source: Ethiopia Manual for Low Volume Roads, ERA (2017).

Fill slope stabilization and protection

Table C.7.6 provides some slope stabilization and protection options appropriate for fill and valley slopes below the road. As with slopes above the road, each situation will require careful investigation prior to deciding on the most appropriate course of action. It is usually the case that fill slope failures on steeply-sloping ground can only be reliably remedied through the construction of retaining walls with foundation levels below the zone of movement, and preferably on rock, or in in situ soils where allowable bearing pressures permit.

Table C.7.6: Stabilization and protection measures for slope instability below the road

Instability	Stabilization options	Drainage options	Protection options
1. Erosion of the fill slope surface	None	▪ Ensure roadside drainage is controlled	▪ Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
2. Failures in fill slope	▪ Re-grade or remove, replace and compact fill; ▪ Before replacing fill, cut steps in original ground to act as key ▪ between fill and original ground; ▪ A new road retaining wall may be the only option	▪ Ensure roadside drainage is controlled	▪ Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
3. Failure in fill slope and original valley slope	▪ Re-grade or remove, replace and compact fill; ▪ Before replacing fill, cut steps in original ground to act as key between fill and original ground; ▪ A new road retaining wall may be the only option.	▪ Ensure roadside drainage is controlled	▪ Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
4. Failure in original valley slope	▪ Re-grade if sufficient space between road and valley side; ▪ A new road retaining wall may be the only option.	▪ Ensure roadside drainage is controlled	▪ Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
5. Removal of support from below by river erosion	▪ May need extensive river training works to prevent further erosion.	▪ None	▪ Slope protection (walls and rip-rap etc.) may be necessary

Source: Based on Hunt et al. (2008).

7.5 Drainage

Several surface drainage systems can be considered on earthworks slopes and natural slopes, depending on slope geometry, materials and potential failure mechanisms (Table C.7.7). Some of these are illustrated schematically in Figure C.7.13. Some typical details for surface drains are provided in Figure C.7.14. Note that deeper drainage systems involving counterfort drains and horizontal drains are rarely used on LVRs due to their high expense and the need for ground investigation for their effective design.

Table C.7.7: Common techniques of slope drainage

Function	Type	Advantage	Limitation
Interception of surface runoff	Unlined cut-off drain (open ditch)	Cheap, can in some instances prevent ingress of surface runoff into landslide masses	Prone to leakage and erosion; may act as incipient tension crack beyond slope crest; requires frequent inspection for damage/blockage; access may be difficult for maintenance
	Lined cut-off drain (Type 1 on Fig C.7.14)	As above, though less prone to erosion and leakage	Requires frequent inspection for damage/blockage; access may be difficult for maintenance. Concentrates flow and erosion if ruptured by ground movement
	Sub-soil drain (Type 2 on Fig C.7.14)	Usually not prone to erosion and leakage and can tolerate some ground movements while continuing to function	May become clogged with silt. Can be surcharged during large surface flows. May encourage water to enter the slope if not constructed properly or where excessive ground movements create 'sags' in vertical alignment or tears in the polythene; access may be difficult for maintenance
	Lined cut-off drain with subsoil drain (Type 3 on Fig C.7.14)	Combines surface and subsurface drainage. Can accommodate large surface flows	Requires frequent inspection for damage/blockage; access may be difficult for maintenance
	Bench drain on cut slopes Berm drain on fill slopes	Collects and discharges surface runoff from one bench or berm to the next. Reduces the tendency for large quantities of water to pond and seep into the slope material	Will crack and dislocate following any ground movements, may become blocked by falling debris or silt if not properly maintained

Function	Type	Advantage	Limitation
Reduction of shallow sub-surface water and drainage of seepages	Herringbone drain	Depending on depth, usually able to intercept water up to 1.5 m below slope face; can be used to drain seepage areas; can accommodate some slope movement; can be used to help stabilize shallow slope failures up to 2 m deep	May have very limited effect on overall stability of deep-seated failures. May create shallow instability during construction, hence preference to minimize branch length
	Counterfort or trench drain	Generally, able to intercept water up to 3 m depth below slope face; can act as a 'buttress' if base is below slip surface	Usually needs to be machine dug; difficult to construct in boulder material
Interception of deep water table	Drilled sub-horizontal drain	Only feasible method of intercepting groundwater at depth	Relatively high cost; drilling equipment required; may not always be successful

Source: Hearn (2011).

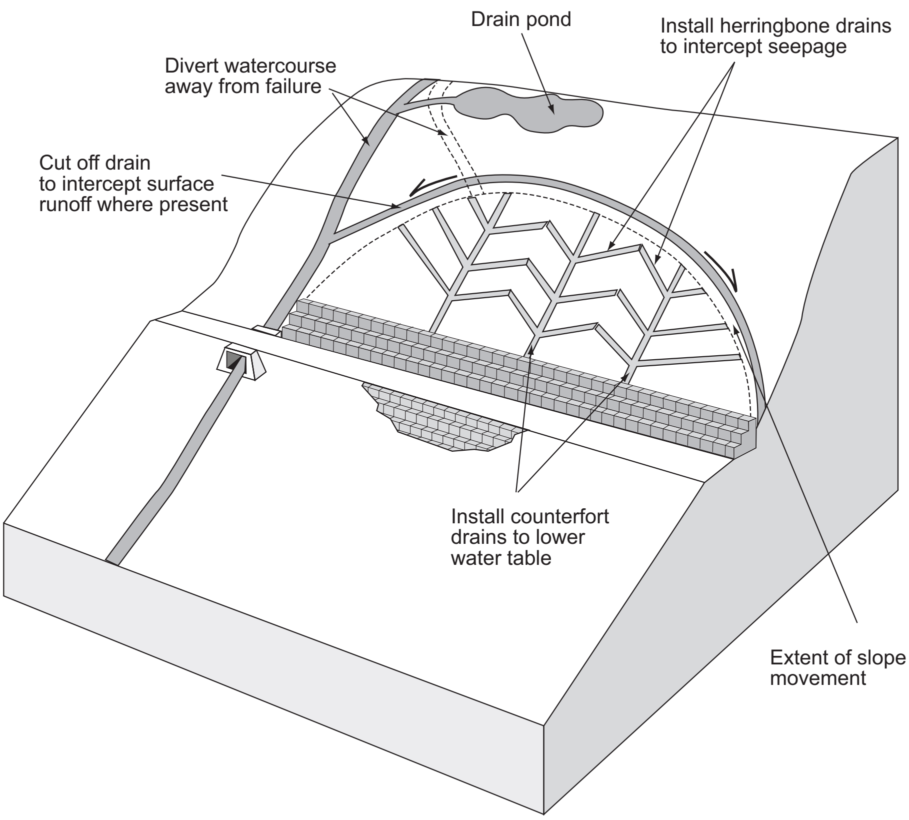


Figure C.7.13: Typical slope drainage measures

Source: Hearn (2011).

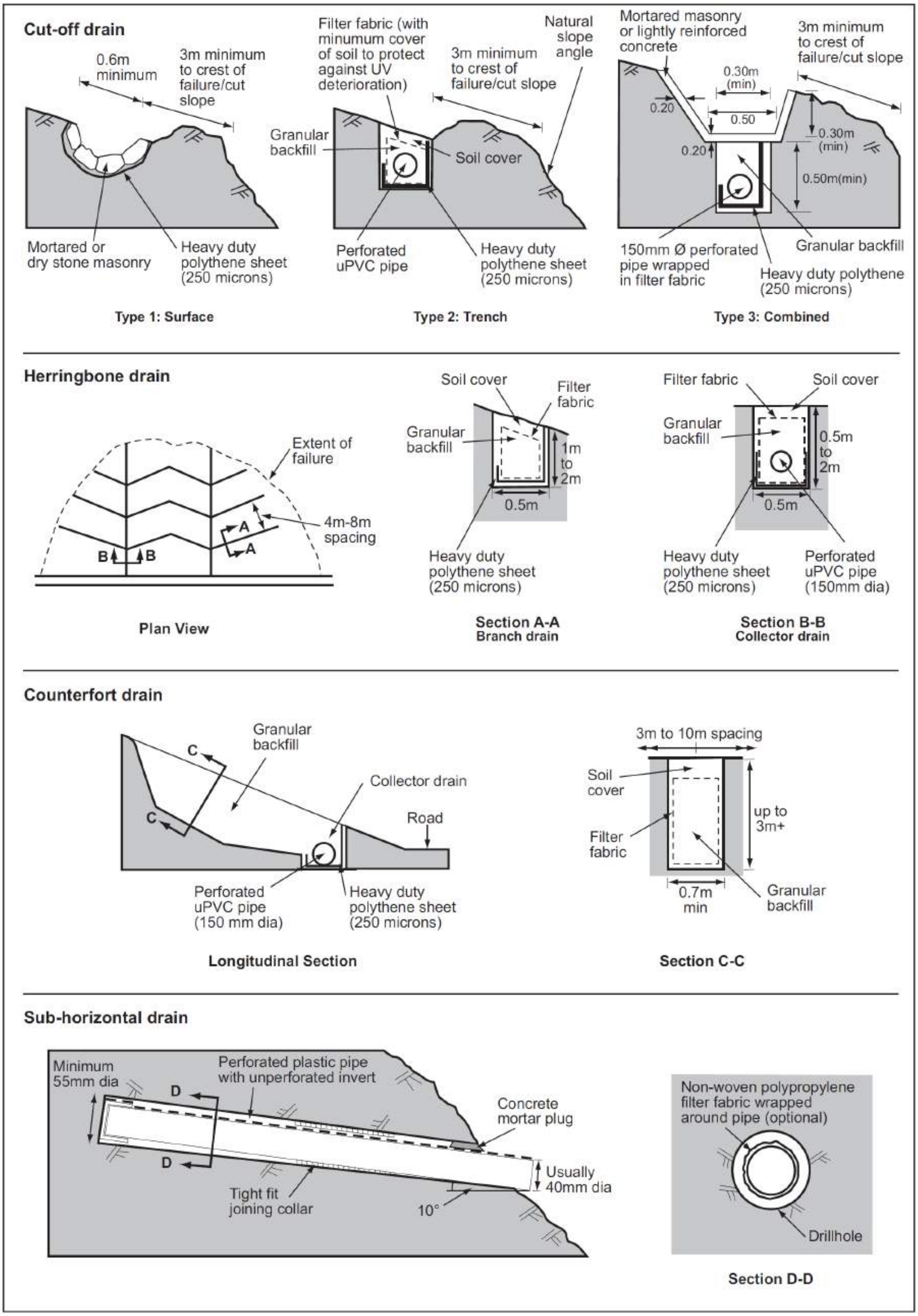


Figure C.7.14: Typical slope drainage details

Source: Hearn (2011).

French drains (type 2 on Figure C.7.13) are ordinarily 600 mm wide by 1000 mm deep and allow deeper seepage paths to be intercepted. They are filled with gravel wrapped in filter fabric with an impermeable membrane to the base and the downslope side, and they normally contain a slotted PVC pipe towards the base of the drain. These drains require careful attention to detail during installation.

It is sometimes common practice to construct stepped channels to convey water down a slope, and these 'cascades' are usually constructed in gabion, masonry (dry and mortared) and concrete. Cascades should not normally be built on slopes steeper than 50° because shooting (high-speed) flow will occur whereby individual steps are bypassed.

7.6 Retaining walls

7.6.1 Use of retaining walls

The use of retaining walls on LVRs is relatively uncommon, with preference usually being given to less costly earthworks-based designs. Detailed horizontal and vertical alignments can also sometimes be adjusted to avoid or minimize the need for retaining walls. However, the use of retaining walls is sometimes unavoidable, for example to a) support road fill on steep slopes, b) retain slipped material or c) support cut slopes that have to be cut more steeply than the exposed material can stand unsupported. In rarer cases, retaining walls are used to support the road where it crosses unstable ground, as described in Section 7.3.

Most retaining wall failures are due to them having been applied in situations for which they were not designed. It is important, therefore, to design retaining structures to accommodate the slope and ground conditions for which they are intended. Ground investigations will normally be required to inform the design. Trial pits are the most effective means of investigating ground conditions at shallow depth, such as the location of most slip surfaces that affect mountain roads. Geological interpretations are required where slip surfaces are deeper and site conditions are complex.

In some circumstances, for example where high walls are required and where foundation soils are predominantly clay, reinforced concrete walls may be preferred to masonry walls of the same height due to consideration of bearing pressures. Figure C.7.15 illustrates the range of wall types sometimes adopted on mountain roads. Gabion, dry stone and mortared masonry are by far the most common of wall types found on LVRs, and they are briefly described below.

7.6.2 Gabion walls

Gabion walls are built from gabion baskets wired together. A gabion basket is made of steel wire mesh in the shape of a rectangular box. The wire should be galvanized, and sometimes PVC-coated for greater durability and protection from sunlight. The baskets usually have a double twisted, appropriately sized, hexagonal mesh, which allows the gabion wall to deform slightly without the box breaking or losing its strength. The manufacturer's specifications for mesh size, galvanizing, wire diameter, panel frames, basket connectors and the twisted connections (usually minimum three half turns) need to be adhered to. Stone fill should be dressed block-shaped with a dimension at least twice the size of the mesh size and should be of sound rock. Rounded river stone should be avoided wherever possible.

Gabion walls are cost-effective because they employ mainly locally available rock and low-skilled labor. Gabion is commonly used for walls of up to 6 m in height, although greater heights have been constructed. Gabion walls are usually the preferred option where the foundation conditions are variable and where clay soils form the foundation material. In such situations the base of the wall should be made as wide as possible to spread the load and reduce bearing pressures. Good drainage and free-draining backfill are essential along with the use of filter fabric to prevent the migration of fines. Where slope movements and differential settlement are anticipated gabion walls are likely to perform better than other wall types because they can accommodate some deformation without structural failure. Gabions are not preferred as retaining walls immediately below and adjacent to sealed roads due to the potential for settlement. This settlement gives rise to movement of the backfill and subsequent pavement cracking. Care should be taken to locate the base of the wall on a good foundation, to reduce the potential for differential settlement.

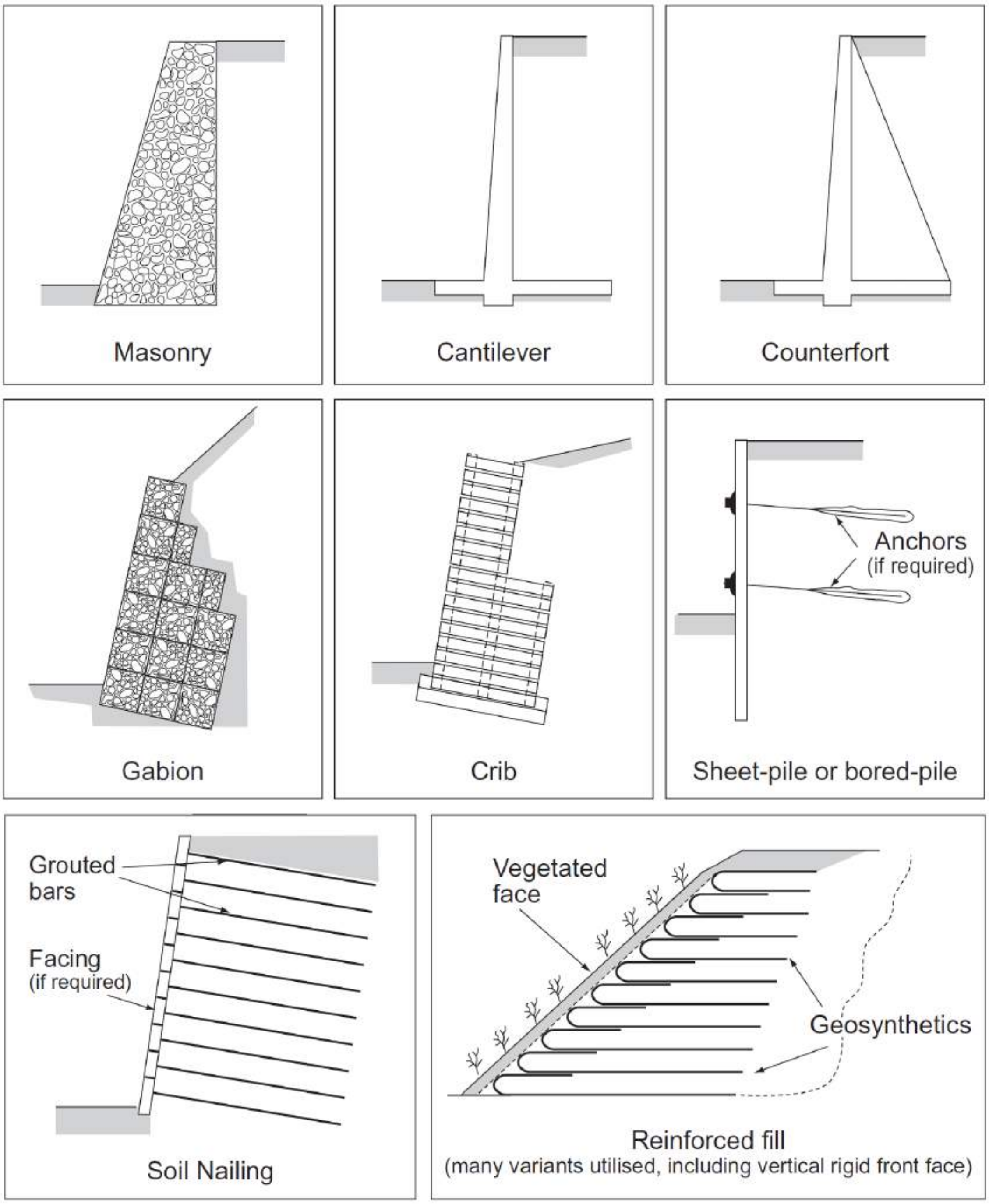


Figure C.7.15: Typical retaining structures

Source: Hearn (2011).

7.6.3 Dry stone walls

Dry stone walls are constructed from dressed stone without any mortar to bind them together. The stability of the wall is provided by the interlocking of the stone blocks. The advantage of dry stone walls is that they are free-draining. Their durability depends on the soundness of the stone used and the quality of construction.

Any differential movement of the foundation will lead to loss of strength and failure. Although dry stone walls of up to 2 m in height have been constructed to support road fill on LVRs in the past, their use in this situation is now less common with mortared masonry walls being preferred (see below). Dry stone walls are best suited to slope protection above the road, often associated with bio-engineering works

(Section 7.7). Revetments of 1 to 1.5 m high in dry stone (sometimes referred to as ‘breast’ walls) can provide important protection to the toe of a slope from shallow movement in surface soils and erosion.

7.6.4 Mortared masonry walls

Masonry walls are brittle and cannot tolerate settlement. They are suited to uneven founding levels but perform equally well on a flat foundation. They are used as cut slope and fill slope retaining walls but also as a form of revetment or ‘breast’ wall (Figure C.7.16) in a slope protection capacity. Mortared masonry walls tend to be more expensive than gabion walls, though there are exceptions to this.

If the wall foundation is stepped along its length, movement joints should be provided at each change in wall height so that any 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 75 mm diameter and placed at 1.5 m centers with a slope of 2% towards the front of the wall. A geotextile filter should be placed at the back of the weep holes to permit free drainage of water but prevent migration of the backfill.

Masonry walls constructed on rock are usually provided with a concrete levelling foundation. Masonry walls constructed onto anything other than rock are usually provided with a reinforced concrete foundation. However, there are many cases where masonry has been constructed directly onto the subgrade. If the subgrade is sufficiently strong and of consistent bearing capacity without the potential for seepage erosion then such walls can perform as required, but there are many cases where walls have failed because of poor subgrade preparation and inadequate wall foundation. Generally, masonry walls should not be founded on clay soils. On weak subgrades their cross-section should be widened so as to distribute the bearing pressures over a wider wall footprint.

Figure C.7.16: Mortared masonry wall providing shallow support and slope protection

7.6.5 Reinforced fill

Reinforced fill structures usually combine a rigid vertical facing to the front of the fill with reinforcing strips or mesh attached to it that are constructed in layers within the compacted backfill. The facing can be constructed from interlocking pre-cast concrete panels, rigid steel grids, polymeric geogrids or gabion boxes. The latter are normally cheaper to construct though are less robust. Alternatively, a suitable geotextile can be used to provide a sloping front face, where space permits. As backfilling proceeds, the horizontal strips or mesh are laid across the length of the wall at selected vertical intervals, the strips or mesh being securely connected to the front face. As a rough guide the width of these strips or mesh is normally about 0.6H – 1.0H, where H is the height of the wall. Alternatively, geogrids can be used throughout. The backfill must be properly compacted, inert (particularly if gabion mesh is used) and preferably granular. If polymeric geogrids are used, care must be taken to reduce the possibility of loss or damage due to theft and deterioration from ultraviolet light.

These structures are uncommon on LVRs, mostly for reasons of cost and lack of design expertise. The main advantage of reinforced fill structures over conventional walls is that they allow distribution of loads over a wider section, thereby reducing bearing pressures (Figure C.7.17). For new road construction the required construction widths are usually available to allow reinforced fill construction. For existing roads, where the road needs to be kept open to traffic as well, this may not always be the case. Guidance on the design of reinforced fill structures can be found in CEDD (2017).

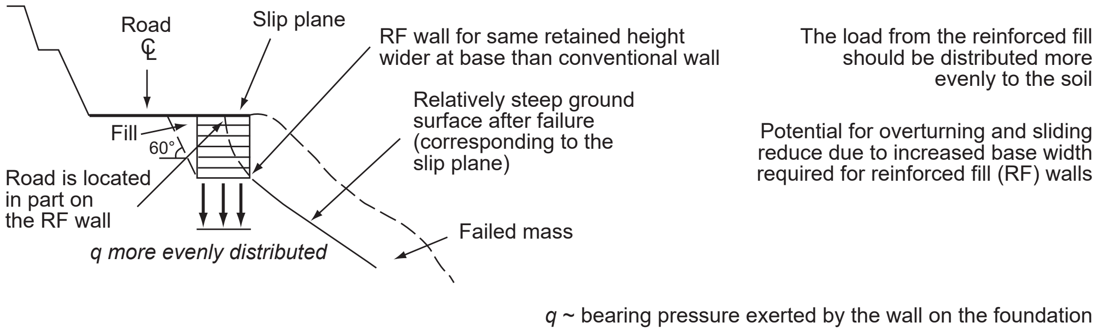


Figure C.7.17: Distribution of bearing pressure on reinforced fill walls

Source: Hearn (2011).

7.7 Bio-engineering techniques

7.7.1 Overview

Certain types of plants, due their ground cover and dense root systems provide the maximum protection against erosion. For any given site it will be necessary to determine which species are the most tolerant of the ground conditions and climate that prevail and which of these can be grown most quickly. A forester or agricultural specialist should be consulted for advice in this regard. Vegetation is unlikely to have a significant impact on slope stability because slip planes are usually (much) deeper than the 0.5 m depth over which roots normally penetrate and therefore bind the soil together. However, some species, such as vetiver, have very deep root systems and can offer greater potential for shallow slope stabilization. Vetiver has been used successfully as an earthworks erosion protection measure in other parts of the continent (www.vetiver.org).

7.7.2 Selection of plant species

The plant must be of the right type to undertake the bio-engineering function required. The following points should be considered (Hunt et al 2008):

- There is no single species or technique that can resolve all erosion protection problems. It is always advisable to use local species which do not invade and harm the environment.
- Grasses that form dense clumps generally provide robust slope protection in areas where rainfall is intense. They are usually best for erosion control, although most grasses cannot grow under the shade of a tree canopy.
- Shrubs (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.
- In most cases the establishment of full vegetation cover on unconsolidated fill slopes may take one to two rainy seasons. The establishment of full vegetation on undisturbed cut slopes in residual soils and colluvial deposits may need 3 to 5 rainy seasons. Less stony and more permeable soils have faster plant growth rates, and drier locations have slower rates.
- Plants cannot be expected to reduce soil moisture significantly at critical periods of intense and prolonged rainfall.
- Grazing by domestic animals can destroy plants if it occurs before they are fully grown. Once established, plants are flexible and robust. They can recover from significant levels of damage (e.g. over-grazing, flooding and debris deposition).

The use of grass-planting and other techniques of bio-engineering as an erosion-control measure on earthworks slopes in Liberia was first applied by ArcelorMittal Liberia Ltd (AML) in 2009. Research carried out by AML (Poilecot 2015) identified several grass species with soil conservation potential in the humid tropical environment of Liberia. These are listed below along with their common name, where indicated.

- Anadelphia leptocoma (thatch grass)
- Andropogon gayanus (blue grass; gamba grass)
- Andropogon macrophyllus (American carpet grass)

- Axonopus compressus
- Ctenium newtonii
- Cynodon dactylon (couch grass; star grass)
- Eragrostis atrovirens
- Hyparrhenia diplandra
- Hyparrhenia rufa
- Imperata cylindrica (cotton wool grass; spear grass)
- Loudetia phragmitoides (erapo grass)
- Melinis minutiflora (molasses grass; stink grass)
- Paspalum scrobiculatum (bastard millet; koda millet)
- Pennisetum polystachion (mission grass)
- Pennisetum purpurem (elephant grass; napier grass)
- Rhylachne rottboellioides
- Schizachyrium rupestre
- Sporobolus dinklagei
- Sporobolus pyramidalis (catstail dropseed)

Many of these grasses are used for thatching and weaving, several are good for animal foraging and some have medicinal value. Most are applied either by direct seeding or through the use of rooted stem cuttings and grass slips.

7.7.3 Site preparation

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

7.7.4 Recommended techniques

Tables C.7.8 and C.7.9 outline the different types of bio-engineering techniques recommended for slopes and soil materials above and below the road. Bio-engineering techniques commonly used for erosion control in the vicinity of the Right of Way (ROW) in general are given in Table C.7.10. Some of the bio-engineering techniques referred to in these tables are illustrated in Figure C.7.17.

Table C.7.8: Bio-engineering techniques for slopes above the road

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

Table C.7.9: Bio-engineering techniques for slopes below the road

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

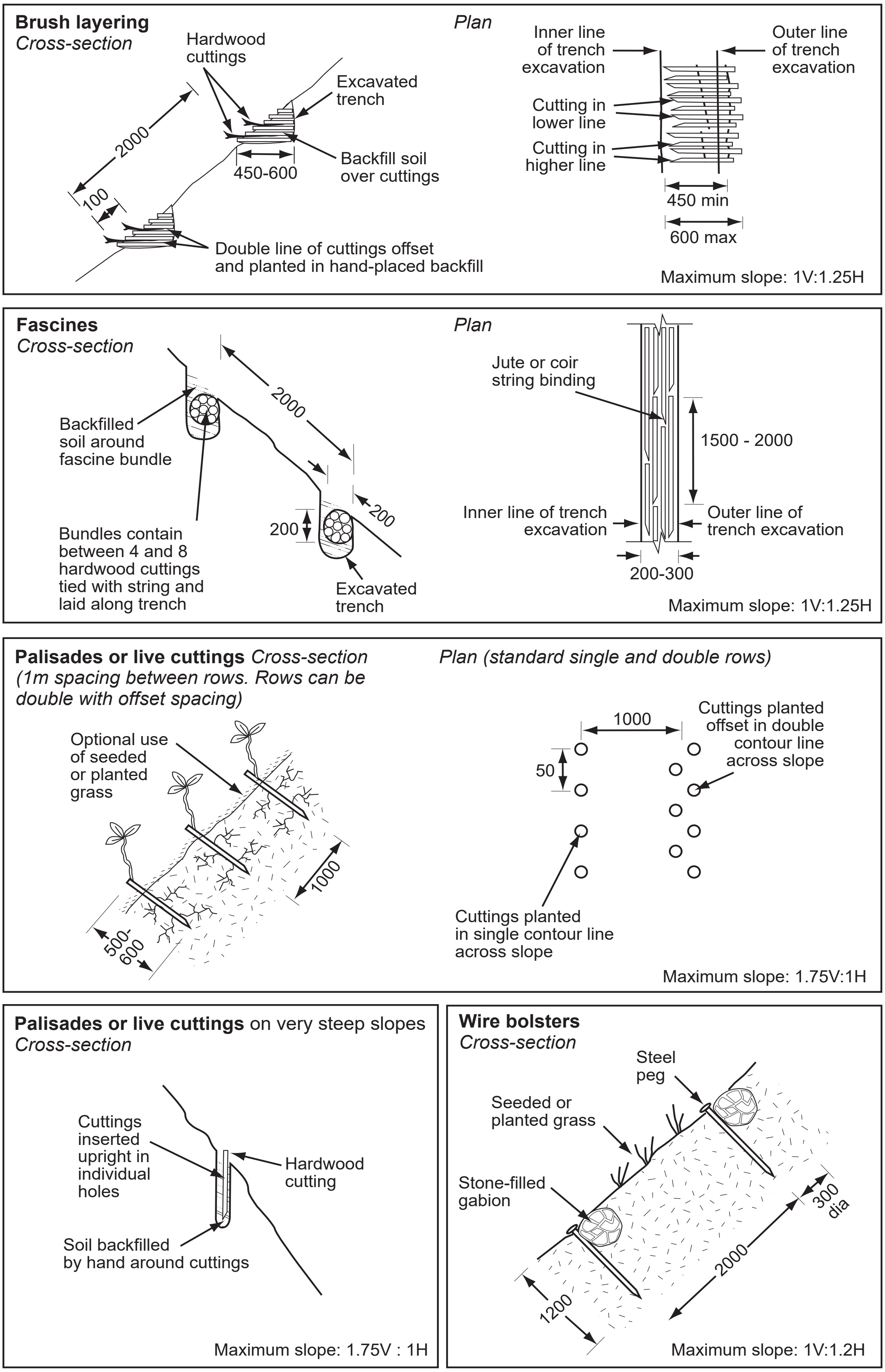
Table C.7.10: Bio-engineering techniques for slope improvement in the ROW

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

7.8 Useful dos and don'ts

The following advice is given when considering slope stability problems and solutions for LVRs:

- Always determine the cause, mechanism, depth and extent of the slope instability or erosion problem before deciding upon the design approach.
- Assign a priority to each road or section of road to identify critical areas.
- Regularly inspect side slopes, culvert outlets and side drains to identify potential problems.
- Identify areas where land use activities are adversely affecting engineering performance and take steps to rectify the situation.
- Identify areas where engineering works are adversely affecting land use and take steps to rectify the situation.
- When scheduling retaining structures in unstable areas try to locate rock or high strength in situ soil horizons beneath the slip surface for foundation purposes.
- Ensure adequate bearing capacity for all structures using in situ tests (e.g. DCP) and ground investigation.
- When building gabion (or masonry) structures in stream channels, ensure adequate foundation beneath potential scour depths and key structures adequately into channels banks or side slopes.
- Drainage control is a major factor in the maintenance of natural and man-made slopes. Drainage must be diverted away from vulnerable areas and every attempt made to slow down but not impede or pond drainage.
- Select safe and appropriate areas for spoil disposal.
- Bio-engineering can significantly enhance the slope environment but is ineffective if the depth of movement or potential movement is greater than about 0.5 meters.



All dimensions in mm

Figure C.7.18: Typical bio-engineering details

8. CONSTRUCTION MATERIALS

8.1 Introduction

There are various materials available that can be used for the construction drainage structures on LVRs. Notable materials are concrete (plain and reinforced), timber, block masonry, stone masonry and steel. This chapter focuses on concrete (plain and reinforced) which is the most widely used material in the construction of drainage structures on LVRs in Liberia and most West African countries. This chapter also highlights the uses of stone and timber as construction materials and their characteristics. The main source of information here is the Ethiopian and Tanzania Manuals for LVRs and the Labor-Intensive Public Works Manual - Practitioners Guide to Rural Roads Improvement and Maintenance developed by the Local Government Ministry in Ghana.

8.2 Concrete (plain and reinforced)

8.2.1 Introduction

Concrete is a mixture of predetermined quantities of dry materials (cement, stone and sand) and water. It is the most widely used construction material for drainage structures e.g. culverts, U-drains, bridge elements (abutments, decks, etc.).

8.2.2 Concrete strength

Concrete can be poured into almost any shape when the ingredients are first mixed. After placement, it gains strength through the curing process for up to 28 days. It hardens to a strong mass possessing great durability, capable of carrying great loads and resistant to the effects of weather. However, its final strength depends on the way it is mixed, handled, compacted and cured.

The strength of concrete depends on:

- The proportions of cement, aggregate (gravel/sand) and water in the mix. The proportions vary, depending on the purpose for which the concrete will be used. Within certain limits, more of the cement ‘glue’ makes a stronger concrete. However, cement is very expensive, and therefore no more cement is used than is necessary to produce the strength of concrete required for the structure.
- The use of clean, strong, well-graded aggregate. The aggregates form up to 80% of the concrete and so it is important that they are of the right quality. The larger aggregate particles form the ‘skeleton’ of the concrete and provide the compression strength. The smaller size particles of gravel and the sand fill the spaces between the bigger particles. The adhesive cement/water mix fills the smallest spaces between the aggregate particles and glues them all together into the finished concrete. The aggregates must not contain impurities, soil, twigs etc., as these will reduce the cement strength.
- The use of the correct volume of clean water, free of dissolved organic matter, or other impurities. Salt water is unsuitable.
- The compaction of the concrete after pouring, to remove all air voids.
- The curing of the concrete to retain the water used for mixing.
- The temperature at which the concrete has hardened.
- The type of cement used.

When the correct proportions of aggregate and cement are mixed with the right volume of water, the mixture will change from a plastic form into solid concrete over a period of about one hour. It is during this hour, before it starts to harden, that the plastic concrete is workable and must be poured into its final shape. However, it will take about 4 weeks for the concrete to reach 95% of its full strength. Concrete has two kinds of strength:

Compressive strength, which is the ability to resist being crushed by a weight without breaking or bending. Correctly proportioned concrete has high compressive strength without any extra reinforcing.

Tensile strength, which is the ability to resist being pulled apart without stretching or snapping. The tensile strength of concrete is lower, perhaps only 20% of its compressive strength, and so in places where the concrete is subjected to tensile loads, it must be reinforced. This is done by placing steel rods (reinforcing rods) into the place where the plastic concrete will be poured, so that the rods are encased in the concrete. The rods are placed in line with the direction of the pulling loads and so give the concrete tensile strength. This is known as reinforced concrete. Angular aggregates give a higher tensile strength than round aggregates such as river gravel.

8.2.3 Constituents of concrete

The three ingredients of the mixture that forms concrete are:

- Cement, which is an adhesive, composed principally of limestone and special clay. It is the active ingredient and constitutes about 10%-15% of the concrete by weight. The cement, in combination with water, forms a strong matrix which surrounds and binds the aggregate together. As the concrete mix sets and hardens it gains strength and durability.
- Aggregate (coarse and fine), which is a well-graded mixture of gravel (coarse) and sand (fine), up to 40 mm in diameter. Aggregates are inert materials, usually of mineral origin, which constitute the bulk of the concrete (about 75%-85%). They are usually chosen from local sources for low cost, but their size range, shape, density, hardness and surface properties have important effects on the resulting concrete.
- Water, which causes a chemical reaction to take place when it is added to a mixture of cement and aggregate. It constitutes about 5% of the concrete by weight. Initially, it gives the concrete workability, allowing it to flow and take up the shape in which it is molded. Over time, the water combines chemically with the cement in a process called hydration, which causes the concrete to set and develop strength.

Figure C.8.1 shows the percentage composition of the different components of concrete mix.

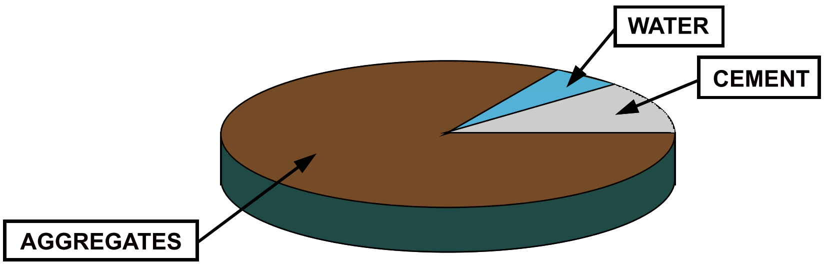


Figure C.8.1: The constituents of concrete

The suitable raw material (stones and sand) used the production of concrete are found in almost everywhere in the country. Cement and reinforcing bars are widely manufactured to standards that are internationally recognized. They are readily available at the market.

Cement

The most commonly used cement for concrete works at most construction sites is the Ordinary Portland Cement. It is usually sold in cement paper bags of 50kg weight which is equivalent to 0.035 m^3. Portland cement is produced mainly from finely ground limestone and clay. These materials are burnt at high temperatures to form cement clinker. A small quantity of gypsum is added to the cooled clinker to control the rate of setting. The clinker is then ground to a fine powder to produce Portland cement.

Cement must be stored in a dry, well-ventilated area, on a raised platform to prevent ground moisture entering. Bags of cement should be stacked close together but keep a clear space between the sacks and the walls (see Figure C.8.2). Lumps in cement are signs that the cement has been exposed to moisture. Stack the bags so that the first batch in can be the first out.

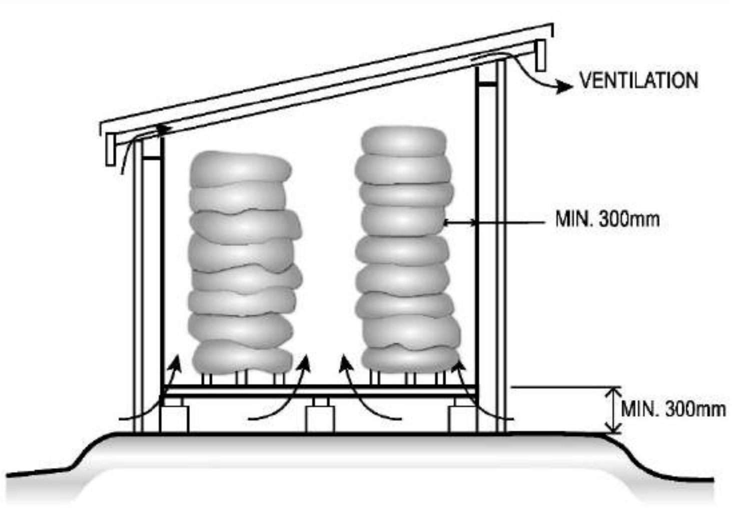


Figure C.8.2: Storage of cement

If it becomes necessary to store sacks of cement in the open, you should;

- make a wooden platform about 300 mm off the ground supported on bricks or timber;
- cover the bags with a tarpaulin or plastic sheeting (If more than one sheet is used, overlap them so that water runs off without wetting the bags.); and
- weigh down the sheeting at the bottom and on top with reasonable weights that will not allow the wind to blow the sheeting away.

Water

The mixing water used should be clean and free from salts. It can be taken from rivers, lakes or wells or from a treated water supply. Salt water may be used for plain concrete, though it will affect the rate of setting. However, salt water should not be used for reinforced concrete. River water containing sediments can be used if the sediments are first allowed to settle out in a tank or drum until the water is clear.

Aggregates

There are two types of aggregates used in making or producing concrete which are identified by the particle sizes:

- Fine aggregates usually called sand, has particle sizes between 0.5 mm and 2.0 mm.
- Coarse aggregates usually called stones has particle sizes between 5 mm and 20 mm. The aggregate size can increase to 40 mm. Coarse aggregates may be naturally occurring gravel, or more commonly crushed or hand-broken quarry stone (see Figure C.8.3).

Figure C.8.3: Aggregate being crushed by hand

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

Reinforcement

Reinforcement is normally in the form of steel bars. Three characteristics are of primary importance: enough strength that a small amount of reinforcement can be used to carry the tensile and shear forces; enough ductility that the rods can be bent without breaking, and, if a member is overloaded, that the structure will deform without failing; and sufficient bond between the reinforcing steel and the concrete that forces can be transferred between them.

Two types of steel reinforcement are in common use: mild steel and hot rolled high-yield steel. Mild steel bars are round, while high yield bars have a deformed surface to improve the bond with the concrete. Typical reinforcement sizes range from 6 mm to 32 mm in diameter. Reinforcing steel is usually available both in rod and mesh forms. Reinforcement bars are cut to the required length and bent to the required shape; they are then tied together in the arrangements shown on the drawings using binding wire and spacer blocks

On site, reinforcement should be kept straight until needed and should be stored clear of the ground to prevent contamination with soil. An example of a steel reinforcement cage being assembled on site is shown in Figure C.8.4.

Figure C.8.4: Steel reinforcement cage being assembled

The important points that require checking and supervision for reinforcement works are:

- All reinforcing bars and mesh must be fixed in the positions shown on the engineer's drawings.
- The bars must be properly tied together with binding wire and the long ends of the wires must be clipped off.
- Sufficient spacers must be securely fixed to make sure that the correct concrete cover to the reinforcement is obtained in the whole structure. Spacers shall be made of well cured mortar. Wooden blocks, pieces of stone or brick must not be used for spacers.
- All reinforcement bars must be encased in concrete of a sufficient thickness and density to ensure bonding, to prevent corrosion and to protect against fire damage. A nominal concrete cover of 20 mm to 50 mm must be adhered to which is dependent on bar diameter and environmental conditions, as can be seen in Table C.8.1.
- Steel must not be trampled or pushed out of position by the concreting gang or the placing equipment.

Table C.8.1: Concrete cover to reinforcement bars

Environmental conditions	Bar diameter, d1 (mm)	Min. dimension for ≥ B25 min cover (mm)	Nominal dimension for ≥ B25 nominal cover (mm)
Structural elements to which external air has regular or permanent access e.g. side drains.	up to 20 25 28	20 25 30	30 35 40
Structural elements which are permanently under water or enclosed in subsoil e.g. culverts.	up to 20 25 28	20 25 30	30 35 40

Environmental conditions	Bar diameter, d1 (mm)	Min. dimension for ≥ B25 min cover (mm)	Nominal dimension for ≥ B25 nominal cover (mm)
Structural elements which are exposed to especially corrosion-inducing influences on steel or concrete, e.g. coastal areas.	up to 28	40	50

8.2.4 Types of concrete

There are three principal types of concrete used in drainage structural works. These are determined by their mix proportions as described in the Table C.8.2.

Table C.8.2: Types of concrete

Concrete Class	Mix Proportion (Cement:Sand:Stone)	28 Day Strength, N/mm^2	Description
Lean	1:4:8	<15	This is a meagre mix with low cement content. It is used for blinding the foundation excavations, for structures, where it acts as a clean working surface prior to placing mass or structural concrete.
C15	1:3:6	15	This is appropriate for gravity structures where reinforcing steel is not used. A large sized stone (up to 50 mm) is therefore permitted. Larger stones would create mixing difficulties.
C20 C25 C30	1:2:4 1:1.5:3 1:1:2	20 25 30	Concrete for use in reinforced structures and load bearing applications such as culvert rings, headwalls, wingwalls, and bridge decks. High strength concrete with maximum aggregate size 20 mm to allow the concrete to easily pass around steel reinforcement.

Table C.8.3 gives the material requirements for producing 1 m^3 of concrete.

Table C.8.3: Material requirements 1 m^3 concrete

Concrete Class	Number of 50kg Cement bags	Weight (kg)	Fine (m^3)	Coarse (m^3)
Lean	3.3	166	0.47	0.94
C15	4.3	215	0.46	0.92
C20	6.0	300	0.42	0.84
C25	7.3	365	0.38	0.76
C30	8.1	405	0.35	0.70

8.2.5 Batching of concrete

The proportions of cement, sand, gravel aggregate and water used to produce concrete (called batching) can vary, depending on the different uses and required strength of the finished concrete. The mixed proportions are defined by the ratios by volume of the cement, sand and coarse aggregate components used in the mixture. When batching by volume is to be used, the mix proportions should be measured using a gauge box with dimensions as shown in Figure C.8.5. The gauge box has a volume of 0.036 m^3, equivalent to one 50kg bag of cement.

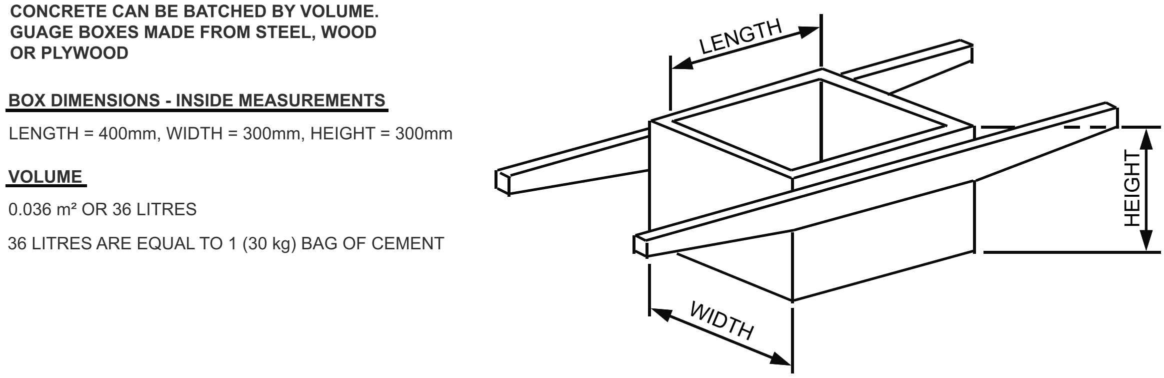


Figure C.8.5: Gauge box made of steel or wood

Table C.8.4 gives the cement/water ratio, number of gauge boxes of aggregates and the yield per batch for one bag of cement.

Table C.8.4: Batch with 1 bag cement

Class of concrete	Cement/water	Batch with 1 bag cement		
		Number of boxes of aggregates		Yield per batch
		Fine	Coarse	(m^3)
Lean	1bag/25 liters water	4	8	0.30
Mass	1bag/25 liters water	3	6	0.24
Grade 15	1bag/25 liters water	3	6	0.24
Grade 20	1bag/25 liters water	2	4	0.16
Grade 25	1bag/25 liters water	1.5	3	0.14
Grade 30	1bag/25 liters water	1	2	0.12

8.2.6 Concrete production

Water required

The strength of concrete depends on the amount of water used, and more water means less strength. However, a minimum amount of water is needed to make the cement workable. Between 20 and 25 liters of water for each bag of cement is needed to produce workable and yet sufficiently strong concrete. Less water is required if the sand or gravel is already wet. For each bag of cement used in the batching process, the right volume of water should be carefully measured, using a container of known volume. Too much water in the mix is one of the main causes of weakened concrete. Workers often use too much water, because it increases the workability and makes the concrete easier to mix and pour out after mixing. However, too much water allows segregation or separation of the materials, so they are no longer evenly spread through the mixture, resulting in a reduction in strength of the concrete. The wetter the mix, the weaker will be the finished concrete. The concrete is also more likely to crack as the water dries out.

Mixing

Once the volumes have been accurately measured out in the gauge box, the components can be mixed together (dry mix) before the correct volume of water added to produce concrete. The quality of the finished concrete is only as good as the quality of the mixing. Concrete should be mixed as near as possible to the place where it will be used. Concrete may be mixed (or batched) by hand or by a mechanical mixer.

8.2.7 Transportation and placement of concrete

Transporting

Concrete should be mixed as near as possible to the site of placement and may be transported using trucks, wheelbarrows, or head pans for sites with difficult access. The wet mix should be transported within 30 minutes to allow placing before setting commences. The haul routes for wheelbarrow hauling should be kept smooth to prevent segregation during transportation. If it is not possible to prevent segregation during transportation, the concrete must be remixed before it is placed. Careful handling is necessary to avoid segregation, which occurs when the larger, heavier particles of aggregate sink to the bottom of the concrete mix, due to jolting and rough handling.

Placing of concrete

Concrete should be poured as close as possible to its final position, certainly not from more than a height of 1 m, as this will also cause segregation. Concrete must be placed in layers of maximum depth of 300 mm for hand compaction and 600 mm for mechanical compaction using vibrating poker. Each layer must be poured in one continuous operation before the previous layer has hardened, to allow the two layers to bond together. Each layer should also be compacted before the next layer is placed. Concrete should not be placed during rain, as this may wash cement from the surface layer. Over-vibration must be avoided as it can lead to segregation of the concrete paste from the aggregates.

Formwork or shuttering for concrete must be clean, smooth faced, and secure from movement or leakage when the concrete is poured. Formwork is normally constructed from timber and plywood, especially where shapes are complex. Where the same shape is repeated (e.g. for culvert barrels or headwalls) then steel formwork can be economical and efficient to use). The Formwork should also be greased with burnt oil to avoid the concrete bonding with the shuttering board. Knocking of the formwork is necessary to allow the air bubbles to escape and achieve a smooth finish after removal of the shuttering.

The top of the placed concrete should be finished smooth with a mason's trowel or float. However, any day work joints should be left rough to ensure a good bond for the next layer of concrete.

Curing of concrete

The strength, wear resistance and stability of concrete all improve with time, if conditions are favorable. This improvement is known as curing and is very rapid in the early stages but continues for a long time. Concrete, which dries too quickly will not develop its full strength. Concrete hardens as a result of hydration of the cement with water. Fresh concrete contains more than enough water to hydrate the cement completely but if the concrete is not protected against drying out, the water content, especially near the surface, will be insufficient for complete hydration. This causes cracking. Curing should start as soon as the concrete begins to harden (3-4 hours after placing). Suitable methods include: sprinkling or flooding; covering with empty cement bags, hessian bags or other fabric, sand, sawdust (50 mm thick), grass or leaves, all of which should be kept wet. The curing process that occurs within the first 28 days is very important for the final strength of the concrete. For faces cast against formwork, the formwork may be loosened after one day and left in place, dampening from time to time. All concrete should be cured for at least 7 days.

Figure C.8.6: Curing of concrete with wet hessian sacking

Figure C.8.7: Curing concrete slab by ponding

Two conditions are required for good curing:

Presence of moisture: Evaporation of water from newly placed concrete can cause the chemical process to stop. It may also cause the concrete to shrink. It is therefore important to keep the concrete – including the edges of slabs – moist during this 28-day period. The simplest method is to cover the concrete with moisture retaining materials, such as plastic sheet or moistened sack (Figure C.8.6). If these are not available, a covering of wet sand or soil about 50 mm thick will do. Continuous water sprinkling is also an excellent method therefore a laborer should be assigned just for curing of the concrete work. Shading the curing concrete completely from direct sunlight is also very beneficial. On flat surfaces such as culvert aprons, a water-filled earth or clay dam can be constructed around the concrete. This method, called ponding, retains the moisture during curing (Figure C.8.7). Concrete cured in the air without being kept moist may only have two thirds of the strength of concrete cured moist.

An even temperature: Exposure to hot sun will heat the concrete and speed up the evaporation of moisture. A large variation between day and night temperatures will cause erratic curing conditions. It is therefore important to try to maintain an even temperature in the curing concrete. Ponding, as described above, also helps maintain an even temperature while the concrete is curing. It can also be shaded from the sun with plastic sheet, tarpaulins or banana leaves etc.

Summary of the guidelines for placing, compacting and curing of concrete to assist in the attainment of a quality product is shown in Table C.8.5.

Table C.8.5: Guidelines for good quality concrete

Activity	Recommendations of good practice
Placing concrete	▪ Forms and the shutters should be cleaned before placing the concrete ▪ Concrete should be placed in layers of 300 mm depth ▪ Concrete should not be placed in heaps, as this causes separation of the stones from sand and cement ▪ Concrete should not be dropped from a height of more than 1.5 m, as this also causes separation of the stone from the sand and cement ▪ Reinforcement bars are to be placed inside the shuttering before placing the concrete
Compacting concrete	▪ Compacting is undertaken by tamping with a steel or wooden rod. It is important to remove all the air in the concrete as entrained air reduces the strength of the concrete.
Curing concrete	▪ Curing means keeping the outside of the concrete moist (wet) during the setting (hardening) of the concrete by: ú Wetting the concrete surface frequently ú Covering the surface with wet material (cloth, paper bags, sand etc.) ▪ Hardening of concrete requires at least seven days. Curing prevents cracks in the surface layer of the concrete. ▪ As cement is normally one of the most expensive items in the construction process, it should not be wasted.
Too much cement = costly; Too little cement = low strength	

8.2.8 Properties of concrete

In its freshly mixed state concrete needs sufficient workability to enable it to be placed into the formwork and compacted. The workability needed depends on the shape of the formwork to be filled, the amount of reinforcement in it, and sometimes on the method of transportation. Workability is measured on site by the slump test, which is described below. Table C.8.6 indicates the maximum workability suitable for different situations.

Table C.8.6: Maximum slump values for particular uses

Concrete use	Maximum slump
Lean concrete	100 mm
Reinforced foundations	80 mm
Other reinforced areas	50 mm

Strength and stiffness

The strength of a concrete develops slowly as the cement hydration reaction continues. After 28 days, the concrete will have attained most of its final strength, and this is the age at which the strength is specified for use in design. Concrete mixes are designed to achieve a given 28-day strength in compression, as measured by crushing tests on cubes or cylinders. Typical structural concretes have strengths in the range 25 to 40N/mm^2. For high quality control concrete crushing test samples are made regularly on site and sent to a testing laboratory for testing at 28 days.

Tensile strength and stiffness also develop as the compressive strength develops. The tensile strength of concrete is normally about one-tenth of its compressive strength. A quality control test which is used to assess the strength based on the tensile strength is suggested in field testing below.

Moisture movement

Wet cured concrete exposed to air will shrink over time. It will also expand and contract subsequently as a result of changes in ambient humidity or exposure to rain or moisture. The extent of shrinkage depends on the properties of the concrete and ambient conditions, but typically about 0.8 to 1.0 mm per meter of drying shrinkage can be expected (in all dimensions) with subsequent variations of about one-third of these values. This can cause unsightly cracking in concrete structures unless joints are provided at intervals to allow it to occur. Additional (creep) moisture movements occur as a result of the load. Creep continues over a long period of time (some months). Both creep and shrinkage can be restrained (though not prevented) by the presence of reinforcement.

Durability

The durability of concrete depends on its resistance to the major causes of deterioration: corrosion of the reinforcement, sulphate attack, chemical attack, and deterioration of the aggregate-cement bond. There are four principal agents of deterioration shown in Table C.8.7. Protection of the concrete from these agencies of deterioration can be achieved by:

- good compaction – the permeability of concrete is increased if compaction is poor or cracking occurs as a result of poor curing;
- adequate cover to reinforcement – the minimum cover is specified according to the environmental conditions and is greater for external surfaces, and surfaces which are to be tooled, closer to the sea, etc. (see Table C.8.1);
- use of low permeability concrete - by using well-compacted concrete with low water cement ratio, which reduces the ability of water to move through the concrete;
- providing a minimum cement content – in order to create a sufficiently alkaline environment to inhibit reinforcement corrosion, a minimum quantity of cement is needed; nominal mixes provide an adequate amount of cement; and
- minimizing the risk of alkali silica reaction - by limiting the alkali content of the concrete or by using non-reactive aggregates.

Table C.8.7: Agents of concrete deterioration

Agent	Description
Corrosion (rusting) of the reinforcement	Corrosion is caused by an electro-chemical reaction occurring in the presence of water and air. It occurs when water gains access to the reinforcement either through inadequate concrete cover to the reinforcement, or because of poorly mixed or poorly compacted concrete, or as a result of cracking.
Sulphate attack	Sulphates in soil, sea water and some aggregates will react with the hydrated cement resulting in expansion and damage of the concrete.
Alkali-silica reaction	“concrete cancer” is a deterioration of the concrete as a result of a reaction between alkaline fluids and reactive minerals in certain types of aggregates.

Thermal movements

The coefficient of thermal expansion of concrete is about 10 to 14 x 10^{-6} mm/°C, i.e. about 3 mm per meter for a 30°C temperature rise, which is about the same as for structural steels (12 x 10^{-6}). Thus, for long concrete structures such as multi-span bridges, expansion joints are needed to allow for seasonal temperature changes.

8.2.9 Field testing

Presence of silt and clay in sand and coarse aggregates: Visual Test

Rub a sample of the sand between damp hands and note any discoloration caused. Clean materials will leave the hands only slightly stained. If the hands remain dirty after the sand has been thrown away, it indicates the presence of too much silt and clay.

Presence of silt and clay in sand and coarse aggregates: Bottle Test

Half fill a clear bottle or tumbler with aggregates (Figure C.8.8). Add water until it almost reaches the top, shake vigorously and then allow the aggregates to settle. After about 30 minutes there should be no fine material deposited on top of the aggregates and the water should be clear. Salt may be added to the water (one teaspoon per 0.5 liters) to speed the settlement. If the height of the silt layer is more than 6%, the sand should be washed before use in concrete.

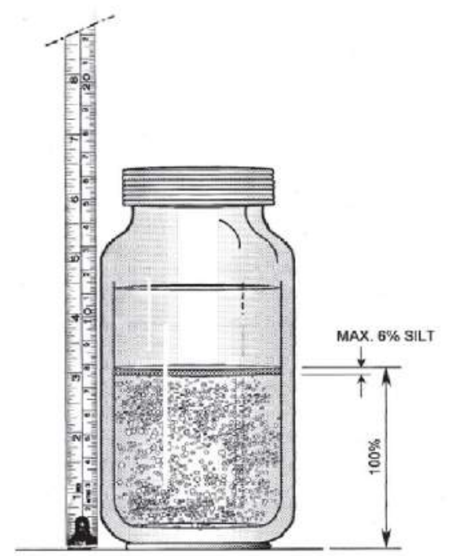


Figure C.8.8: Field Fines Test

Quality of reinforcement

Reinforcement to be used should be supplied to comply with the national standards. Before use it should be checked to ensure that all bars are straight and free from loose scale, loose rust, oil, grease and dirt.

Quality of cement

Cement to be used should be supplied to the national standards. To check that it has not deteriorated in storage, the cement should be tested by hand for hardened lumps. A small proportion of lumpy cement may be removed by sieving.

Test for workability of fresh concrete: Slump Test

The slump test is the standard method of making sure that concrete does not vary in consistency due to variations in the water-cement ratio. To undertake the test, the standard cone-shaped mold is required, and a steel rod described in Figure C.8.9.

Mold shall be made of a metal not readily attacked by cement paste and not thinner than 1.5 mm (e.g. galvanized steel). The interior of the mold shall be smooth and free from projections such as protruding rivets and shall be free from dents.

The mold shall be in the form of a hollow cone, with part of a cone having the following internal dimensions:

- Diameter of base: 200 ± 2 mm
- Diameter of top: 100 ± 2 mm
- Height 300 ± 2 mm.

Tamping rod: Made from straight steel bar of circular cross section, 16 mm diameter, 600 mm long with both ends hemispherical.

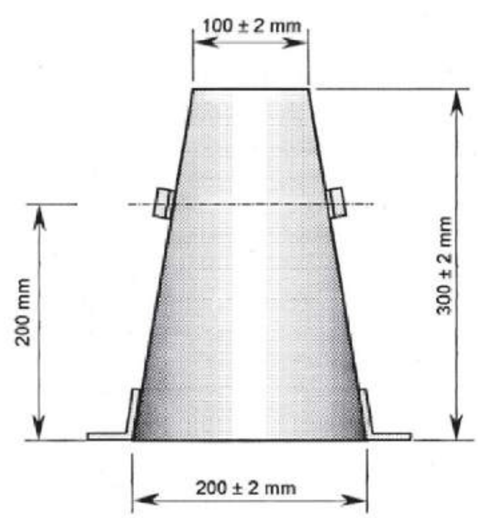


Figure C.8.9: Slump Test Mold

The procedure for the test is illustrated in Figure C.8.12. The cone must be clean and dry inside and is put on a smooth, hard surface. The cone is filled one-quarter full. Holding it firmly in place with the metal feet, rod the concrete thoroughly 25 times. Then add more concrete to about half-way and rod it another 25 times, taking care to take the rod just through into the first layer. Next add the third layer filling the cone three-quarters full, and rod again 25 times, going through into the layer below. Finally fill the cone up, rod 25 times again, going well down into the third layer and smooth off the top. The top is smoothed off level with the cone.

Wipe the metal plate it stands on clean and dry and wipe around the base of the cone. Then, carefully and keeping it quite straight, lift the cone off and put it down beside the concrete. The concrete will collapse to some extent - very dry concrete hardly at all, very wet concrete completely. Test it by measuring how far it has collapsed.

To measure the slump, rest the rod across the top of the empty cone so that it reaches over the concrete. With a ruler measure down from the underside of the rod to the top of the concrete, always measuring from the highest point on the concrete.

Examples of slump testing on site are shown in Figure C.8.10 and Figure C.8.11.

Figure C.8.10: Workable and cohesive concrete

Figure C.8.11: Collapsed slump - too much water

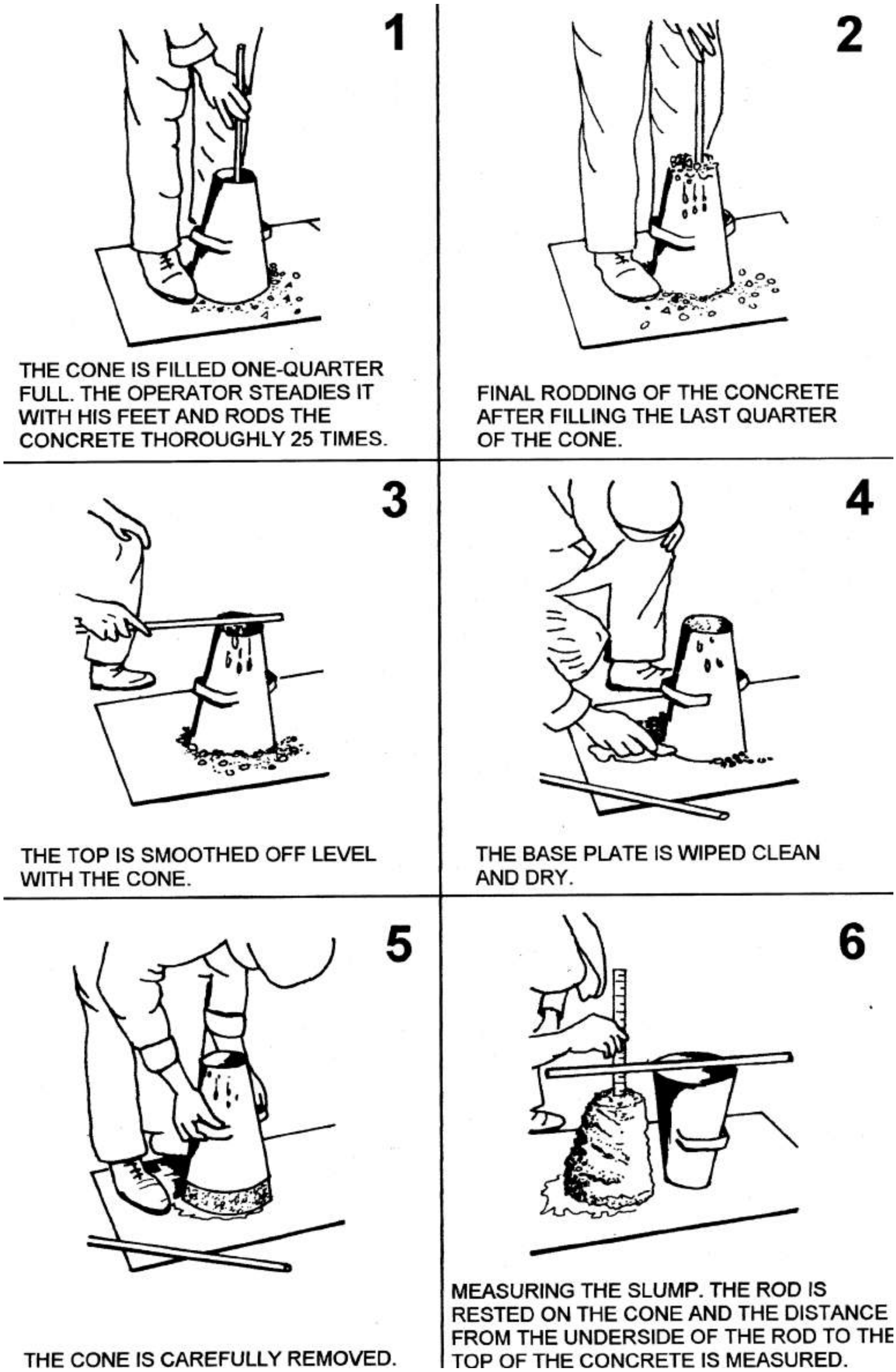


Figure C.8.12: Slump Test

Concrete Strength: Cube Tests

Test cubes should be made on the site to check that the concrete has the required strength. The cubes are used to find the crushing strength of the concrete. It is essential that the cubes are made with great care. They are normally sent to a laboratory for testing. This test is only recommended for structural concrete with a design strength of 20 N/mm^2 or above. Cubes should be cured by immersion in a tank of clean water and tested 28 days after casting. In some cases, additional cubes are cast and crushed after 7 days to indicate whether the concrete is on track to achieve the required specification strength.

Concrete Strength: Impact Tests

For a rough strength assessment for concrete of mass concrete grade, make a set of 10 briquettes from plain concrete of dimensions 100 x 200 x 50 mm. Place each brick in turn with its largest face downwards, resting on timber battens 150 mm apart. Drop a mason's 2 kg hammer from a height of exactly 0.5 m so that it strikes the upper face midway between the battens. The briquette should not break. If more than 1 in a sample of 10 bricks breaks in this test, the concrete is not of adequate strength and should not be used.

8.2.10 Uses of concrete

Reinforced concrete is used for bridge decks, piers and abutments, as well as for box culverts and culvert rings. Plain concrete may be used for drifts and causeways, and for the foundations of masonry and timber walls, piers and abutments. Typical examples of where concrete can be used in small drainage structures for LVRs are shown in Figure C.8.13 and Figure C.8.14.

Figure C.8.13: Concrete culvert ring production

Figure C.8.14: Constructing reinforced concrete slab

8.3 Stone

8.3.1 Overview

Stone is greatly valued for its aesthetic appeal, durability, and ease of maintenance although seldom used to form entire structures. Stones are classified into different types based on their geological origin. The most common types include granite, lime stone, sand stone, marble, slate and gneiss. All natural stone used for structural support, curtain walls, veneer, floor tile, roofing, or strictly ornamental purposes is called building stone. Stones are used in different ways for construction purposes on LVRs. They come in different shapes and forms and of different quality and durability. In Liberia, stone masonry is used for protection purposes against erosion e.g. gabions and for slope stabilization e.g. stone pitching on LVRs (see Figure C). Stone aggregate is a major constituent of concrete used for the construction of drainage structures.

Figure C.8.15: Stone pitched embankment slopes behind wingwalls

8.3.2 Properties of stone

Size

The most important prerequisite of a good building stone is that the stone is available in pieces of a size and shape suitable for the type of wall or structure to be built. Stones should also be small enough to be lifted and placed by hand. For use in rubble walling, a range of sizes is needed. The individual stone height may be up to 300 mm, the length should not exceed three times the height and the breadth on the base should not be less than 150 mm, or more than three-quarters of the wall thickness. A range of sizes should be used, with larger stones being used for corners (quoins) and for through (bonding) stones.

Durability

Durability is the resistance of the stone to weathering or deterioration from other causes. The structure of the stone is the most important aspect of its resistance to decay. Stone used for building should be uniform in color and texture, without soft seams or veins or other visible blemishes. The surface of a freshly broken stone should be bright, clean and sharp without loose grains and be free from an earthy appearance. Visual tests are sufficient to assess its durability characteristics. Other durability issues are shown in Table C.8.8.

Table C.8.8: Durability of stone

Frost Action	Some types of stone are seriously affected by frost, and in cold climates must not be used in positions where they can become saturated. The remedy is to protect the stone from becoming saturated by means of a coping and provide protection for the base from upward percolating water by means of a damp-proof course.
Soluble salts	Soluble salts can disfigure and ultimately cause deterioration of some sedimentary stones. Soluble salts may occur in the sands used for mortar, in the water behind retaining walls, or in road salts. The remedy is not to use a stone which is liable to react poorly to soluble salts in circumstances where it will be exposed to them.
Thermal and moisture movement	Some small variations in the dimensions of stones always occur as a result of changes in temperature and moisture. These are rarely sufficient to cause any cracking problems, but it is a good precaution to insert movement joints in mortared masonry walls at intervals of approximately 15 meters.

Note: Frost action is not expected in Liberia; weathering due to rainfall is more common.

Compressive strength

There are significant problems of strength testing of stone in rural areas. The compressive strength of dense stone is generally greatly more than that required in any small road structure. A few porous stones, like pumice or tuff may require some testing to establish that they have a suitable compressive strength. In other cases, the compressive strength can be assumed to be adequate for the small road structures described in this manual based on evidence of established local use. However, for stones subject to abrasive conditions or for use in arches, it is advisable to confirm the compression strength is a minimum of 15 MPa unless otherwise specified.

Seasoning

Certain stones such as soft limestone and sandstone increase significantly in strength and durability after quarrying. The appropriate time for seasoning depends on the quarry, and local knowledge is needed to decide on the correct seasoning time.

Porosity

Porosity is not in itself a disadvantage in most cases, but some stones are capable of absorbing substantial amounts of water; this can reduce the strength, and, in cold climates, freezing can cause disintegration of the stone. A good building stone should not absorb more than 5% of its weight in water.

Field testing

In many cases the best test of the suitability of a stone from a local quarry or other source is its previously successful use in structures in the area which have been subjected to the local climate for a long period of time. Enquiries to local builders and contractors may result in knowledge gained regarding the best sources of building stone, and any local characteristics. This information can be supplemented by additional tests as required.

Structure test

The structure of a stone from sedimentary rock sources can be tested by immersing small pieces in clear water in a glass jar for about an hour and then shaking them vigorously. If the water discolors, the stone is not well cemented and should not be used.

Water absorption

The water absorption of a stone is a measure of its porosity and of its liability to frost damage. The water absorption of a stone can be assessed by:

- weighing it when dry (stored in a dry environment for at least 5 days);
- immersing it in water for 24 hours at ambient temperature; and
- weighing it again after removing excess surface moisture. (The difference in weight should not exceed 5% of the initial weight).

Soundness test

The soundness (freedom from cracks or weaknesses) of a stone can be tested by means of the hammer test (see Figure C.8.16).

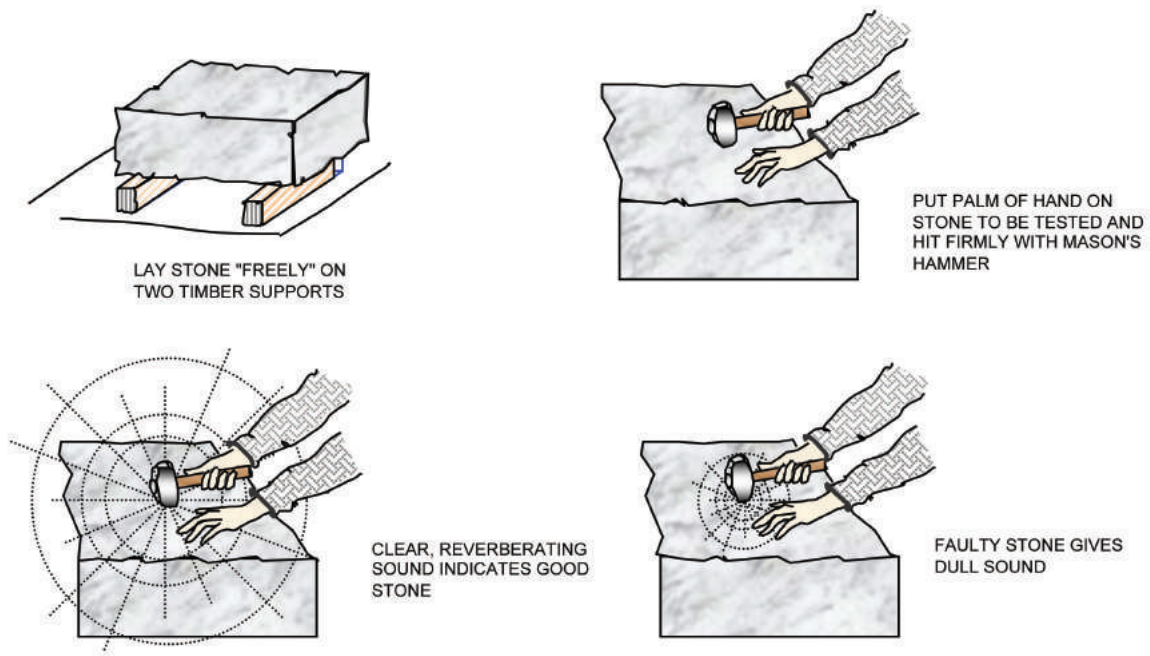


Figure C.8.16: Hammer test for stone

Acid test for weathering potential

A small sample is immersed into a 1% solution of hydrochloric acid for seven days, during which time it is frequently agitated. If the sample has retained the sharpness of its edges and corners, it will weather well.

Compressive strength

There is no adequate field test for compressive strength. This is not normally an important consideration except with blocks made from weak stones such as tuff. Where needed, testing should be entrusted to a competent laboratory.

Hardness

The surface hardness can be tested by scratching with a penknife. All types of stone are marked by a knife blade under firm pressure, but stone in which a penknife blade can make a groove exceeding 2 mm is likely to be moderately weak in compression, and compression testing may be needed.

8.4 Mortars

Stone masonry usually involves mortar jointing. The main function of mortar in masonry is to provide an even bed to distribute the load over the whole bearing area of the units, and to bond the masonry units together.

Good mortars should:

- be cohesive, spread easily and retain water so that they remain plastic while the masonry units are positioned and adjusted;
- set and develop strength rapidly after the units are in place;
- have a final strength adequate to carry the load without cracking the masonry; and
- be impermeable to moisture movement, and resistant to weathering.

Mortars are composed of clean sand and a binding agent (usually Portland cement) and often some additive (either lime or plasticizer) to improve plasticity and workability. Sand should be soft building sand free of organic particles and clay. Lime should be bagged dry hydrated lime or lime putty. A plasticizer is an admixture to the mortar used in small quantities to improve the workability of the mix or to achieve the same workability with less water, thus improving both strength and durability. Plasticizers are proprietary materials and should be used according to manufacturers' instructions.

It is important that the strength should not be greater than that of the units being joined so that movement cracking is dispersed through the mortar joints and not lead to a few wide cracks which could affect strength and weather resistance.

Table C.8.9 gives typical mortar mixes using cement-sand or cement-lime-sand.

Table C.8.9: Mortar proportions by volume

Type of concrete	Type of mortar	
	Cement : Lime : Sand	Cement : Sand
Higher strength for structural use or in contact with water	1: 0.5 : 3	1: 3
Lower strength for general use	1: 1 : 4	1: 4

Commonly used mixes are 1:4 cement-sand for structural use or where there is contact with water, and 1:6 in other cases. For a good quality mortar, the water content should be low (typically 0.4 water/cement ratio). The quantity mixed in any one batch should not be more than can be used in about one hour; during that time unused mix should be covered to protect it from evaporation.

8.5 Timber

8.5.1 General

Timber is widely used on LVRs for bridge decks and as formwork for casting concrete. It can also be used for bridge piers and abutments. Timber comes in logs and can be processed into different forms and shape to suit the specific use e.g. timber boards, timber beams, etc. Timber is most commonly utilized structurally in the form of sawn sections. Timber is generally sawn at sawmills, in or close to the forests from which the trees are extracted, and then supplied to timber wholesalers or importers, who sort, grade and treat the timber for supply to the users. Timber logs may be used for temporary works such as bridge decks over streams for traffic diversions or over side ditches to serve as access bridges.

8.5.2 Seasoning

Freshly cut timber contains a substantial proportion of water, up to 100% of its dry weight. If used in the green state it is subject to substantial shrinkage movement, as well as being prone to fungal attack. Thus, for effective structural use timber must be dried so that its moisture content is close to the equilibrium moisture content (between 10% and 20%, depending both on the type of timber and the climatic conditions). This process, which must be carried out with care to avoid distortion, is referred to as seasoning. Seasoning also increases the strength and stiffness of the timber.

8.5.3 Preservation

Timber should be carefully preserved and treated to acceptable service life before being used. Preservative treatment is needed to protect timber from fungal attack, insects and marine borers. There are several chemical treatments available, and the success of the treatment depends on effective choice of both the chemical substance used and the treatment process.

Chemical preservatives include:

- oil-based preservatives such as creosote;
- water-based preservatives such as copper/chrome/arsenite; and
- organic solvent preservatives such as pentachlorophenol.

8.5.4 Stress grading

Because of the natural variability of timber, even of pieces from the same source, careful grading, piece by piece, is essential to ensure safe and efficient use. Stress grading can be done either visually or mechanically. Visual grading involves making a visual assessment of the extent of the principal factors affecting strength - knots, fissures, grain slope, wane, distortion, and perhaps worm holes and fungal decay, and classifying the timber according to predetermined measures of each which are acceptable in the various grades. Some aspects of visual stress grading are described below. In machine grading, each piece is subjected to a bending test under load in an automated process and is graded according to its deformation; a visual assessment is carried out at the same time.

8.5.5 Properties of timber

Natural defects

Natural defects shown in Figure C.7.18 are features which develop in the living tree, which may affect its structural usefulness. Some can be accommodated within limits.

The most important are:

- knots - parts of branches which have become enclosed in the main tree; can reduce strength in tension, can be difficult to work;
- fissures - splitting separation of the fibers due to a variety of causes including: stresses in the standing tree (shakes), slits from rapid drying, resin pockets (in resin-bearing softwoods);
- wane - inclusion in the sawn timber of part of the original round surface of the log;
- insect holes; and
- grain slope - the small angle between the direction of the grain and the length of the cut timber.

Several other types of natural defect are unacceptable and should be eliminated from any timber used structurally including:

- brittle heart - this material is found in the center of some tropical trees, and should be avoided because it is of low strength and breaks with a brittle fracture; and
- fungal decay - this is discussed below.

Shape

The processes of sawing and seasoning timber create distortions which must be limited for satisfactory use. The four principal types of distortion encountered are bow, spring, twist and cup. Some suggested limits are given in the Table C.8.10.

Table C.8.10: Limits of distortion

Shape description	Limit of distortion
Bow: X	Should not exceed 15 mm per 2 m length (in a piece of 75 mm and greater in thickness
Spring: Y	Should not exceed 7 mm per 2 m length (in a piece of 250 mm or more in width
Twist: Z	Should not exceed 10 mm per 2 m length
Cup: W	Should not exceed 1 mm per 25 mm of width

The bow, spring, cup and twist of a piece of timber can be measured directly if the timber is placed on a flat surface. An average of at least 10 measurements should be taken. Some limits to distortion appropriate for tropical hardwoods and softwoods to be used structurally are shown in Figure C.8.17.

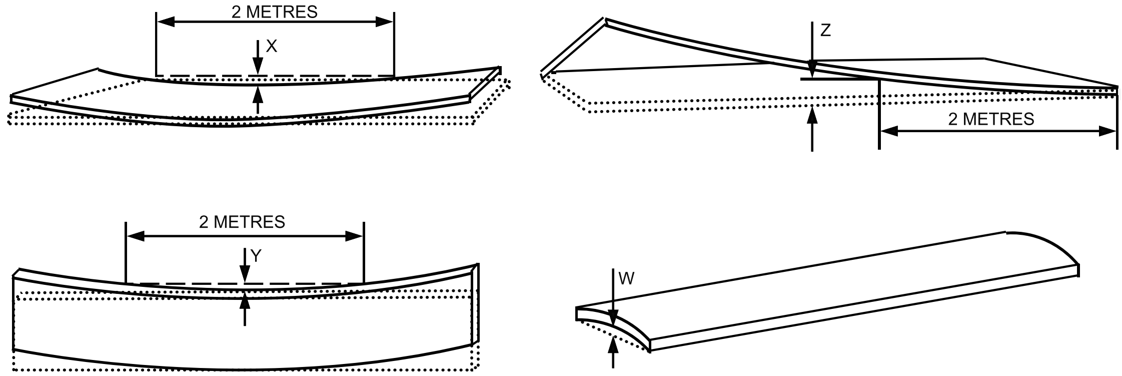


Figure C.8.17: Timber shape criteria

Moisture content

Moisture content needs to be limited to achieve the best structural properties and reduce shrinkage as well as reduce susceptibility to fungal attack. Seasoning should reduce the moisture content to within 5% of the equilibrium moisture content, which is in the range 10-12% for hot-dry regions but may be 14-18% for tropical rainforest regions.

Density

The density of timber depends on its type. Softwoods typically have densities in the range 350-480 kg/m^3, but for bridge construction those suitable have densities above 420 kg/m^3 at 18% moisture content are required. Tropical hardwoods typically have densities in the range 500 to 800 kg/m^3 or even higher, but there are many hardwoods with much lower densities. The foregoing tables divide the common species of hardwoods into two classes: heavy hardwoods with densities above 650 kg/m^3 when dried to a moisture content of 18%; and lighter hardwoods with densities less than 650 kg/m^3.

Strength and elasticity

Strength and stiffness are the most important properties from the point of view of structural utilization, and they are closely related; timbers with higher strengths generally also have higher modulus of elasticity. The strength of timber can be affected by natural defects. Typical examples are shown in Figure C.8.18.

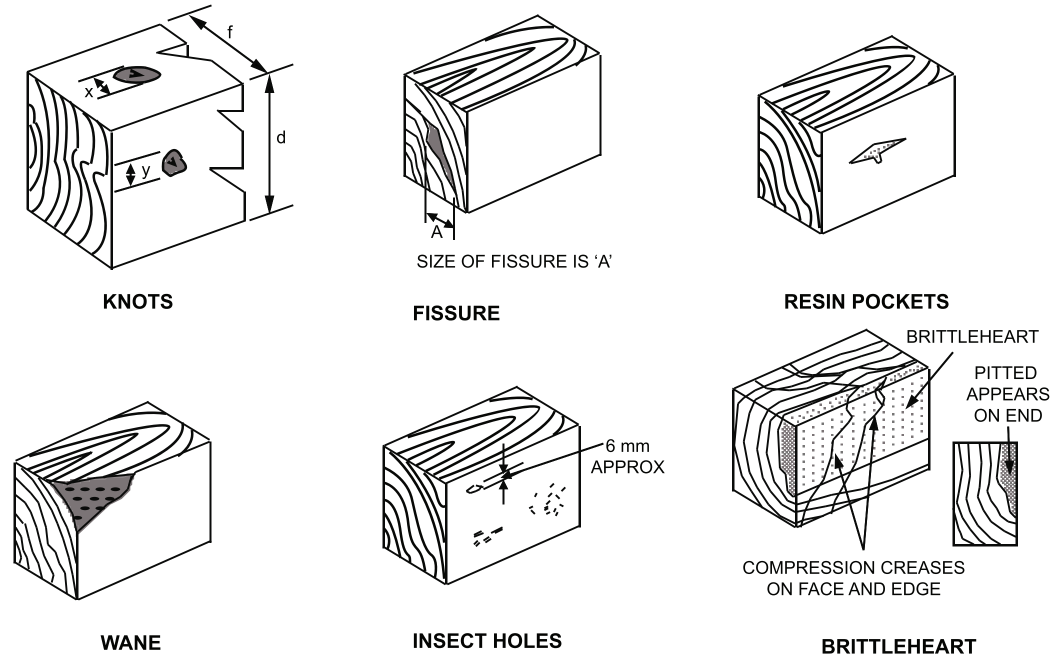


Figure C.8.18: Natural defects in timber

For structural design maximum allowable stresses in bending, tension, compression (both parallel to and perpendicular to the grain) and in shear must not be exceeded. These are normally derived from guidelines or codes of practice in which timbers are grouped into strength groups. Values of the strength and stiffness parameters are given for each group depending on the moisture content and extent of defects in the timber. Design stresses for the three principal timber groups are shown in Table C.8.11.

Table C.8.11: Design stresses for the three principal timber groups

Stress Type	Heavy Hardwoods (N/mm^2)	Lighter Hardwoods (N/mm^2)	Softwoods (N/mm^2)
Bending	15.1	8.6	5.4
Tension	9.0	5.0	3.2
Compression parallel to the grain	11.3	6.8	5.0
Compression perpendicular to the grain	2.2	1.8	1.5
Shear	2.2	1.1	0.9

Durability

The durability of timber relates primarily to its resistance to fungal attack and attack by insects or marine borers. Durability is enhanced by good timber selection, effective seasoning and preservative treatment, and maintenance after construction. It is also enhanced by good design, particularly measures to ensure that timber is protected from water. The end grain and joints are particularly susceptible.

Fungal attack can cause both staining and decay. Some fungi attack cell contents only, rather than the cell wall substance, and as a result, no structural degradation of the timber occurs. Decay is not an inherent property of the material itself but depends on the availability of food (the wood itself), moisture, air and favorable temperature conditions. Some species have more durable heartwood than others and this is related to the toxic chemicals present in the cells and cell walls of the more durable species.

The natural resistance of wood to decay can be increased by ensuring that its moisture content is below 18% (based on the oven-dry weight of the wood). In addition to using seasoned timber, the wood should be protected from dampness by moisture barriers or flashing. If timber is in contact with the ground, only the more durable heartwood or preservative-treated timber should be used.

In tropical climates, great damage is done to wood by subterranean termites. Termites must have access to the soil or to some other constant source of moisture. They can severely damage timbers in contact with the ground and may even extend attack to the roof timbers of high buildings.

Damage above ground may be prevented by ensuring that all means of access are eliminated. Metal shields or stump caps, or poisoned soil barriers, are effective in preventing the passage of termites from the foundations to other parts of the structure. Where shields are used, adequate clearance below deck level should be provided to allow easy, and regular, inspection. In areas of severe infestation, the only practical methods of control are, however, the use of termite-resistant or preservative-treated timbers.

Apart from termites, there are a number of other insects which attack timber. Moisture is an essential element for some insects' development and hence drying is an obvious protective treatment. However, preservation is generally regarded as being a broad and more positive measure particularly where the timber is to be used in structural applications.

Protection of timber submerged in salt water against attack by water-born organisms is usually based on the use of mechanical sheathing with resistant timbers, concrete or non-ferrous metal, or the use of preservatives which are resistant to leaching, such as creosote. Some tropical woods possess a natural resistance to such attack.

Shrinkage and thermal movement

Some shrinkage and expansion as a result in changes in the moisture content of the timber must be allowed for in design. The important shrinkage movements are tangential and radial, that is across the width of the timber; in these directions the movement can exceed 3% as a result of a change in relative humidity from 90% to 60%. In the longitudinal direction the shrinkage movement is very small, less than 0.1%. The coefficient of thermal expansion is 30 to 60 x 10-6 per °C across the fibers, but less than one tenth of this parallel to the fibers. Thermal expansion even of large structures is therefore not a problem.

Fire resistance

Timber is a combustible material and will ignite at temperatures of around 220 to 300°C. It produces toxic carbon monoxide and large quantities of smoke when ignited. When used in external conditions on road structures the risks are from fire caused by fuel spillage in overturned vehicles and wildfires. However, timber chars as it burns, at about 0.5-0.7 mm per minute, which helps to insulate the interior. There is no instant loss of strength in fire, nor a rapid expansion, and timber structures can safely carry their loads for some time in a fire, enabling people to escape and the fire to be extinguished. Fire retardant and fire- protection chemical treatments are available either as paints or for pressure impregnation, but they are expensive, and the paints require maintenance. Fire protection is therefore not usually applied to external structures for LVRs.

Field testing

Some visual indicators for a good quality timber are:

- The cellular tissue should be hard and compact.
- The fibrous tissues should adhere firmly together and should not clog the teeth of the saw.
- Depth of color indicates strength and durability.
- A freshly cut surface should be firm, shining and somewhat translucent, whereas a dull chalky appearance is a sign of bad timber.
- In resinous timbers those with least resin in the pores are the strongest and most durable; in non-resinous timbers those with least sap are best.
- A good timber is uniform in color, with straight grains, free from dead knots, cracks and shakes, and has regular annual growth rings.

Shape

The bow, spring, cup and twist of a piece of timber can be measured directly if the timber is placed on a flat surface (see Figure C.8.17).

Visual stress grading

Visual stress grading involves making measurements or inspections of natural defects: slope of grain, knots, fissures and resin pockets, wane and insect holes.

Minimum acceptable limits for all these characteristics are shown in Table C.8.12. Other visible defects including bark pockets, compression failures, fungal decay, and brittle heart should not be permitted in any structural timber.

Table C.8.12: Limits of visible defects for structural timber from tropical hardwoods

Property	Acceptable limit for structural timber
Slope of the grain	1 in 11
Knots: size	25% of the thickness, up to 75 mm
Knots: frequency	One sizeable knot per meter of length
Fissures and resin pockets	Moderate fissures (of greater than 1/3 the thickness but less than the thickness): not to exceed in length 20% of the length or 1.5 times the width.
Wane	Not to exceed 25% the sum of the width and thickness
Insect holes	In a square of 100 mm sides not more than 32 pinholes (< 1 mm), nor more than 4 shot holes (< 3 mm) nor more than 2 holes of 6 mm diameter

Determination of moisture content

The moisture content of wood can be measured by the oven test method. Small samples are cut from the wood to be tested, the samples are weighed and then dried in the oven at 100°C until their weight becomes constant. They are then weighed again. The moisture content, m, is calculated as a percentage as follows:

$$m\% = 100 \times \frac{\text{weight of water}}{\text{dry weight}} = 100 \times \frac{\text{initial weight} - \text{final weight}}{\text{final weight}}$$

Typical equilibrium moisture contents for different regions are shown in the foregoing text. The equilibrium moisture content of the timber to be used should be determined by a laboratory. The oven test can then be used as a check on the effectiveness of seasoning of timber delivered to site. Moisture content should be kept within 5% of the equilibrium moisture content.

Strength and elasticity: load testing

The suitability of the structural properties of a timber are normally determined by the use of standard tables of properties for the species to be used, coupled with stress grading to determine the classification of the sections available. However, in certain circumstances, the structural properties of timber can be checked by a direct load test. This is easiest to carry out when the timber is to be used in bending.

A pair of joists is set up between solid supports using the span length which will be used in the actual structure. The joists are connected to each other by cross-bracing, and a deck is placed over them. The deck is loaded uniformly, using heavy materials such as bricks or stone, until it reaches the design load. The deformation at mid-span is then measured. Under the design load it should not exceed about 1/300 of the span. The load should then be increased to 50% above the design load, and the timber should show no sign of failure.

8.5.6 Uses of timber

The principal use of timber in LVR structures is for bridge decks (see Figure C.8.19), where its structural advantages can be utilized most fully, and where it is more easily protected from moisture penetration. Timber can also be used for bridge abutments and retaining structures (though in these uses a short lifetime must be expected), and for culverts. Timber is used as formwork for concrete structures (see Figure C.8.20).

Figure C.8.19: Timber bridge deck

Figure C.8.20: Timber formwork

9. CONSTRUCTION METHODS

9.1 Introduction

Proper construction of drainage structures on LVRs is essential for the design life of the road. Poor drainage accounts for the early deterioration of roads since water in the road pavements leads to its failure. When roads become impassable it is normally at water crossing points with poor drainage conditions.

This chapter provides guidance on construction methods for a range of small structures (drifts, culverts, etc.), from the preparatory work, through planning and the various site activities to the completion of site works. It includes aspects of programming, construction, supervision and monitoring of works, whether the structure is built by a contractor, a Ministry of Public Works work force or a work group set up specifically for the task.

The focus is on the use of local labor that can be utilized for a range of tasks in the construction of small structures.

Not all issues dealt with in this chapter will arise during the construction of a structure, especially a small one. Checklists are provided and where appropriate the text refers to other documents for further reference and information.

9.2 Preparatory work

9.2.1 General

Preparatory work of a varying nature, depending on the size and complexity of the small structure, is always required in terms of planning and mobilizing all the resources required before the site work can begin. This section describes the typical preparatory work involved in undertaking the construction of small structures.

9.2.2 Minor culverts

The amount of preparatory work may often be limited due to limited resources and costs involved, and usually standardized nature of culverts. However, some aspects of the preparatory work in the following sections for larger structures may be relevant.

9.2.3 Bridges, drifts and major culverts

The size, resources and funding required for larger structures will usually necessitate considerable preparatory work before the actual site works can begin. The structural survey and design must be carried out in accordance with the guidelines elsewhere in this manual and with any locally established standards. Cost estimates, detailed drawings and bills of quantities must be prepared for the works.

If the work is contracted, appropriate contract documentation should be prepared in accordance with local standards and procedures. When a contractor will be appointed, local contractor classification, tendering, selection and award procedures should also be complied with. Arrangements should be in place for resolution of any disputes that may arise through the contract. Arrangements for management, supervision, testing, approval and audit of the works should be established. All of these issues should be clearly documented and known to the parties involved in the construction process. If there is any doubt about the responsibilities, adequacy or arrangements for any of these issues, then professional advice should be sought to rectify the situation.

The construction of any structure for a public road involves risks and responsibilities which must be appreciated and should be assigned to the most appropriate parties. Inadequate attention to some aspects of the work can result in a structure not fit for its purpose, waste of resources, or even serious damage or eventual loss of the structure.

9.2.4 Structure costing

It is usual to prepare a detailed costing of the structure, either for internal budgeting and funding purposes, and or for contracting out the work. This is achieved through preparation of a Bill of Quantities which can be priced by the Employer, Consultant, Sponsor/Donor or by the Contractor.

Table C.9.1 shows a checklist for preparing a construction program which may be used as the basis for developing a Bill of Quantities. Bills of Quantities in a national standardized format, with activity related items, will assist Employers and Contractors in pricing works and assessing value for money.

Table C.9.1: Checklist of cost components for detailed costing of a drainage structure

Direct costs	Overheads
▪ Materials ▪ Unskilled labor ▪ Skilled labor ▪ Equipment purchase ▪ Equipment operating costs ▪ Hire of equipment ▪ Tools ▪ Temporary works ▪ Services hired in	▪ Supervisory and technical staff ▪ Survey and setting out ▪ Main office, workshop costs ▪ Supervision vehicles ▪ Transport to and from site ▪ Site camp and stores ▪ Security measures and facilities ▪ Communications (telephone, mail) ▪ Insurances, bonds ▪ Banking and other charges ▪ Training ▪ Protective clothing and safety ▪ Traffic control/signs ▪ Testing ▪ Welfare, pensions, social costs
Contingency/risks (e.g. unforeseen additional work, late payment, delays)	
Profit The contractor should normally be expecting to make in the order of 10% profit on his work, after covering ALL other costs. However, this percentage will be affected by local market conditions, competition and perceived risks.	

9.3 Planning of site works

Planning is vital to the success of any project implementation. Good planning of the site works is essential and covers the entire scope of the works. As many sites of drainage structures are far-off from organizational bases, sources of some materials and skilled manpower, and communications can be difficult, it is necessary to have a good plan that gives information to project team about the activities on site. Poor planning usually leads to serious delays and increased costs to the project. Table C.9.2 and Table C.9.3 are checklists for developing a plan to ensure timely and successful implementation of projects.

Table C.9.2: Checklist for planning site works

Item	Planning activity
1	List all construction and support activities and prepare a work breakdown structure (WBS)
2	Prepare a program of works or construction program (using a Microsoft Project Software i.e. bar chart) based on the Bill of Quantities showing a logical sequence of activities and expected productivities
3	Prepare resource plan and cash flow requirements
4	Plan in recognition of the seasonal watercourse conditions and expected flood conditions. Plan adequate arrangements for damming, diverting or control of water
5	Ensure compliance with all laws and regulations regarding recruitment, labor, (permanent/ casual) employment, gender and disadvantaged groups opportunities, payment, security for payment to laborers, conditions of work.
6	Plan compliance with environmental requirements, particularly with regard to materials exploitation, replacement of felled timber, watercourse pollution and waste disposal
7	Inspect site. Check site survey. Review designs and documentation for compatibility and with the actual site conditions. Clarify any inconsistencies
8	Plan and arrange land (acquisition/lease/use) and setting up site, camp and stores. Cement to be stored in a secure, dry and well ventilated place
9	Ensure adequate site access arrangements, particularly if the structure is being built in advance of the road works
10	Plan water supply, other services requirements and sanitation arrangements
11	Plan site security (particularly against theft of hand tools & materials; cement is particularly susceptible)
12	Ensure availability and accessibility of funds and contingency finance
13	Ensure payment arrangements for sub-contractors, and suppliers are in place
14	Plan staffing, identify skills locally available or required to be imported to the site area, accommodation, logistics, transport to site, recruitment and training of workforce
15	Arrange for supplies of materials to site
16	Plan safe and adequate temporary arrangements for traffic and pedestrians where replacing an existing structure or facility
17	Plan actual/contingency arrangements for de-watering and shoring of foundations
18	Plan for concrete works with respect to weather, since both low, $< 16°C$ and high $> 32°C$ temperatures are not suitable for concrete works and curing

Table C.9.3: Checklist for preparing a construction program

Item	Construction activity
1	Site Clearing: trees, bushes, scrubs
2	Safe disposal of waste
3	Primary setting out and establishment of control/reference points
4	Remove topsoil, stockpile for re-use or disposal
5	Detailed setting out and establishment of levels and profile boards
6	Excavate foundations and any cut-off trenches
7	Temporary shoring, watercourse diversions, piling, cofferdams, de-watering/drainage
8	Drill and blast any solid rock
9	Replace "soft spots" in ground, clean and prepare foundation area
10	Construct foundations
11	Construct temporary works for superstructure
12	Erect abutments, piers, deck, wingwalls
13	Fix deck timbers and running boards where applicable
14	Erect curbs, parapets barriers and safety structures
15	Install drainage layers and features against structure
16	Backfill against and adjacent to the structure, compacting each layer according to the specifications. Attention to be paid to all compaction within 5 meters of the structure.
17	Construct road pavement/surfacing and markings, road shoulders
18	Construct road drainage features
19	Construct gabions and erosion control measures
20	Install traffic warning signs if necessary
21	Clear site, remove surplus materials and leave tidy

The outputs recommended in Table C.9.4 may be useful in estimating the resources and time required for each activity.

Table C.9.4: Recommended output

Serial No.	Activity	Output
1	Site clearance (bush clearing, tree felling, etc.)	100 – 350 m^2 / worker day
2	Removal of tree stumps	1 / worker day
3	Soil excavation (and stockpiling alongside)	2 – 5 m^3 / worker day
4	Rock (fractured) excavation (solid rock will require drilling and blasting/ splitting)	0.8 m^3 / worker day
5	Loading	8.5 m^3 / worker day

Serial No.	Activity	Output
6	Haulage by wheelbarrow 0 – 20 m 20 – 40 m 40 – 60 m 60 – 80 m 80 – 100 m 100 – 150 m	 8.5 m^3 / worker day 7.0 m^3 / worker day 6.5 m^3 / worker day 5.5 m^3 / worker day 5.0 m^3 / worker day 4.5 m^3 / worker day
7	Install only 600 or 900 mm diameter culvert lines (including excavation and backfill)	0.8 - 1.2 linear meter per worker day
8	Mix and place concrete	1.0 m^3 / worker day
9	Erect masonry work	1.0 m^3 / worker day

Source: Ethiopia Manual for Low Volume Roads, Part D (2017).

Productivity depends on a number of factors, including worker nutrition, fitness, experience and motivation, site organization, tool quality and condition, and climate. Individual small structures sites do not allow much scope for improvement of performance with experience due to the short time spans involved for individual activities. New workers under training will also be less productive. Poor quality and condition of hand tools can affect productivity by up to 25%.

Table C.9.5 is a checklist of the range of skills which may be required on a construction site of drainage structures. The more specialist skills may need to be imported into the project area. Some skills may be taught through on-the-job training. This will involve costs and loss of productivity. Workers not from the area of the structure site may require temporary accommodation and incur costs relating to travel and allowances.

Table C.9.5: Potential skill requirements

Serial No.	Activity
1	Surveying and Setting Out
2	Drilling and Blasting
3	Piling/Cofferdam
4	Carpentry
5	Masonry
6	Temporary Works
7	Steel Bending and Fixing
8	Concreting
9	Equipment Maintenance

Table C.9.6 is a checklist of hand tools and equipment that could be required to execute the works.

Table C.9.6: Checklist of hand tools and site equipment

Hand tools		Equipment
▪ Ranging rods ▪ Spirit level /Abney level ▪ Water tube level ▪ String lines, pegs ▪ Profile boards & travelers ▪ Plumb bob ▪ Tape measures ▪ Felling axes ▪ Tree felling saws ▪ Bush knives ▪ Brush hooks ▪ Ropes ▪ Pick axes ▪ Mattocks ▪ Hoes ▪ Crowbars ▪ Shovels ▪ Sledge hammers ▪ Wheelbarrows ▪ Head pans/baskets ▪ Earth 'stretchers' ▪ Carpenters tool kits	▪ Hand drills ▪ Plugs and feathers ▪ Masons trowels ▪ Masons hammers ▪ Spirit levels ▪ Straight edges ▪ Lifting tackle ▪ Buckets ▪ Mortar pans ▪ Mixing boards ▪ Water containers/drums ▪ Screeding boards ▪ Pointing tool ▪ Hand rammers ▪ Rakes/spreaders ▪ Slump test equipment ▪ Concrete cube molds and curing tank ▪ Soil density testing equipment ▪ Sandbags for water control	▪ Culvert molds ▪ Plate compacter ▪ Pedestrian vibrating roller ▪ Water bowser ▪ Water pump ▪ Concrete mixer ▪ Batching boxes ▪ Vibrating poker ▪ Piling equipment ▪ Hydraulic excavator ▪ Compressor and air tools ▪ Craneage ▪ Aggregate crushing eqp. ▪ Aggregate screens ▪ Supply and site transport ▪ Formwork/molds ▪ Traffic signs and barriers ▪ Safety helmets and equipment

9.4 Site works

9.4.1 Introduction

This section of the manual looks at the actual activities to be carried out to meet the requirements and the processes spelt out in the program of works for construction of small drainage structures on LVRs. Quality control measures are very important during site works to ensure that the works are executed according to the project specification and drawings. The following site works are described:

- Setting out techniques;
- Surface drainage;
- Side drains;
- Turn-outs;
- Scour checks, and
- Culverts.

9.4.2 Simple setting out techniques

There are different methods of setting out works for construction of drainage structures on road projects. However, for LVRs where most activities are labor-based, the simple setting out techniques are used. The example of setting out a right angle is shown in Figure C.9.1.

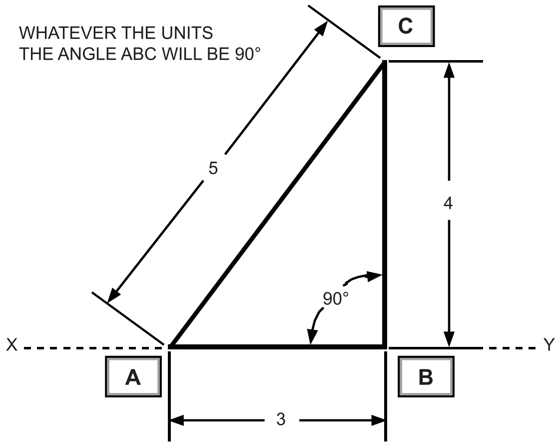


Figure C.9.1: Setting out a right angle

9.4.3 Surface drainage

Adequate surface drainage is achieved by providing very good road camber or cross slope/fall. This helps to eliminate stagnation of water on the road surface which together with the traffic can quickly cause erosion of the road surface. The reduced bearing capacity of the pavement resulting from water ingress can result in deformation of the road. Stagnant water on the road surface is shown in Figure C.9.2.

Figure C.9.2: Stagnant water due to poor camber

Road camber is the slope from the center line of the road towards the shoulders. The appropriate camber to apply varies according to the surface type of the road. For gravel or earth road which are usually LVRs, the camber can be formed to a gradient between 5% and 7% and for sealed roads between 2% and 4%. a side drain is required to drain off the surface water at the toe of the hill.

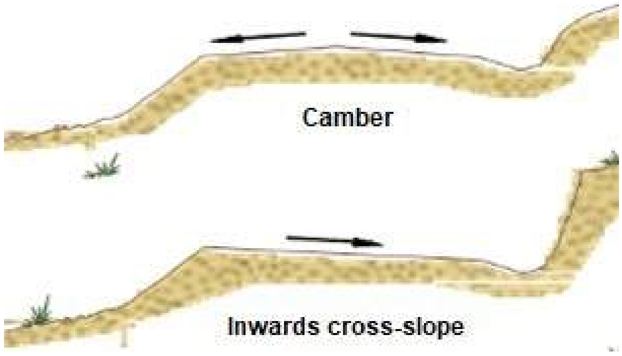


Figure C.9.3: Types of cross-slopes on LVRs

Constructing the camber

The road camber should be formed already before the surface layer is provided. Excess soils from the excavation of side drain are used to form the road camber. The excess soil is piled along the center line of the road during forming of the side drains and spread towards both sides of the road to the shoulders. Enough quantity of soil is piled up along the center and it is spread loose before compaction to form the camber with the correct slope. The loose soil is spread to a slope between 7% and 9% before compaction and the expected slope after compaction is between 5% and 7%.

The levels of the shoulders and centerline are formed by setting out with profile boards and a line level which ensures the correct camber is obtained. These levels are then transferred to pegs placed at 5 meter centers. The exact level of any point along the road can accurately be determined by stretching thin rope between the pegs. The rope serves as a useful guide for workers carrying out the leveling.

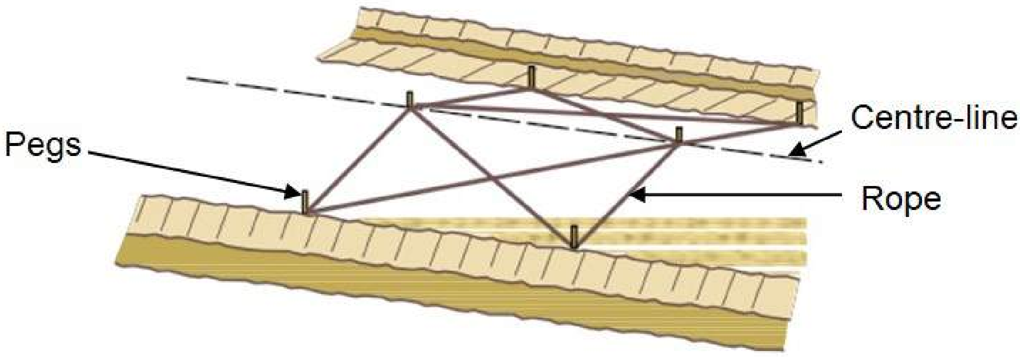


Figure C.9.4: Setting out levels for road camber

The spread loose soil is compacted at optimum moisture content to secure the camber. The final road levels are checked by using rope lines to check the level of the compacted surface to ensure it meets required standards, specifications and quality. If there are any irregularities in the levels, the soil should be removed and filled in again and compact properly. Repeat the checking process to ensure works done to the require standards and quality.

9.4.4 Side drains

Side drains or ditches are constructed to collect surface runoff from the carriageway and adjoining areas. There are usually in three forms i.e. V-Shaped, trapezoidal and rectangular or U-Shaped. The V-shaped drains are usually constructed by motor-grader using its blade. This type of drain is easily silted up and carries low capacity compared to the other shapes.

Excavation of side drain

Excavation of side drains or ditches should start after all levelling works have been completed. On LVRs where most activities are labor-based, excavation of ditches is done in two stages. The first stage involves the excavation of a rectangular ditch, followed by excavation of the side slopes of the ditch. The excavation work is set out using string line and pegs and controlled by using ditch templates. The excavated soils from the ditches are placed in the middle of the road.
The excavation of the side slope and checking of the shape of the drain using the template is shown in Figure C.9.5.

Figure C.9.5: Checking drain shape with template

9.4.5 Turn-out

Turn-outs (mitre drains) divert water away from the side drains to lower areas to help control siltation and erosion of the ditches.

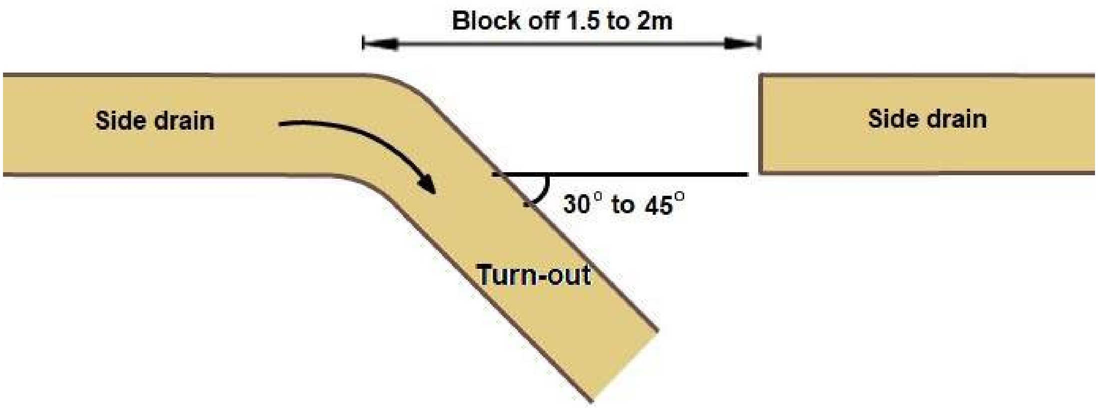


Figure C.9.6: Typical plan of a turn-out drain

The angle between the turn-out drain and the side drain should not be greater than 45° and 30° is an ideal angle. If it is necessary to take water off at an angle greater than 45°, it should be done in two or more bends so that each bend is less than 45°. The angle between the turn-out drain and the side drain can be set out using the triangles shown in Figure C.9.7.

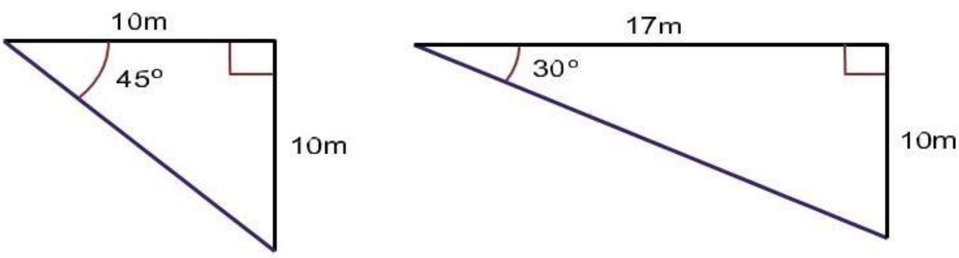


Figure C.9.7: Triangles for setting out mitre drains

Construction of turn-out drains

Construction of turn-out drains follows the same procedure for excavating side drains. Ropes and pegs are used to set out the drains with the exact depth of the turn-out drain marked on the pegs. Excavated soils from the turn-out drains should be placed in such a way that the material is not washed back into the drain after any rains during construction.

9.4.6 Culverts

Culverts are the most common cross drainage structure on LVRs. They allow water from streams and the side ditches cross the road from one section to the other. Figure C.9.8 to Figure C.9.10 show culvert layouts for the different terrain types.

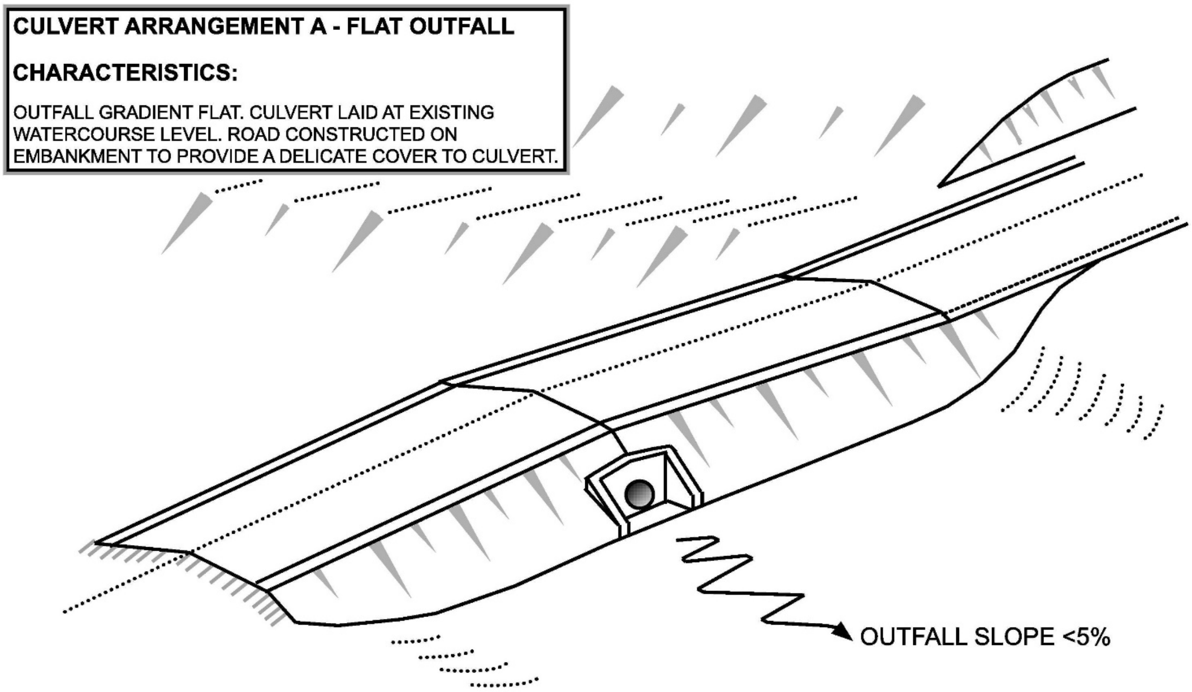


Figure C.9.8: Culvert Arrangement A - flat outfall

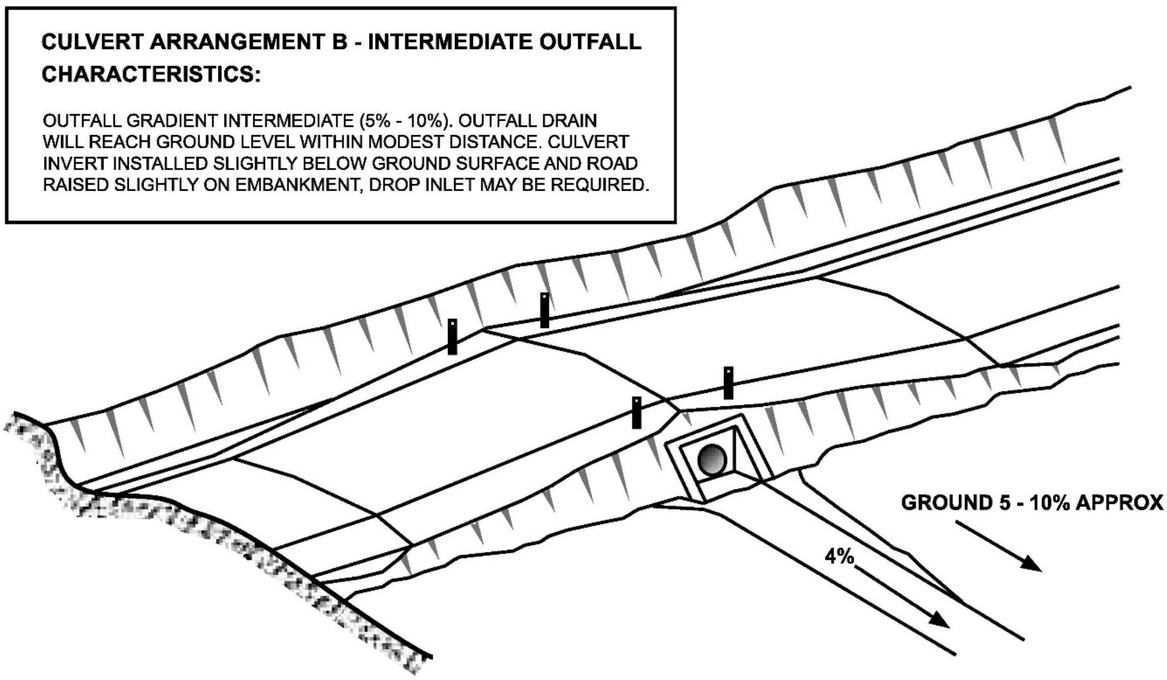


Figure C.9.9: Culvert Arrangement B - intermediate outfall

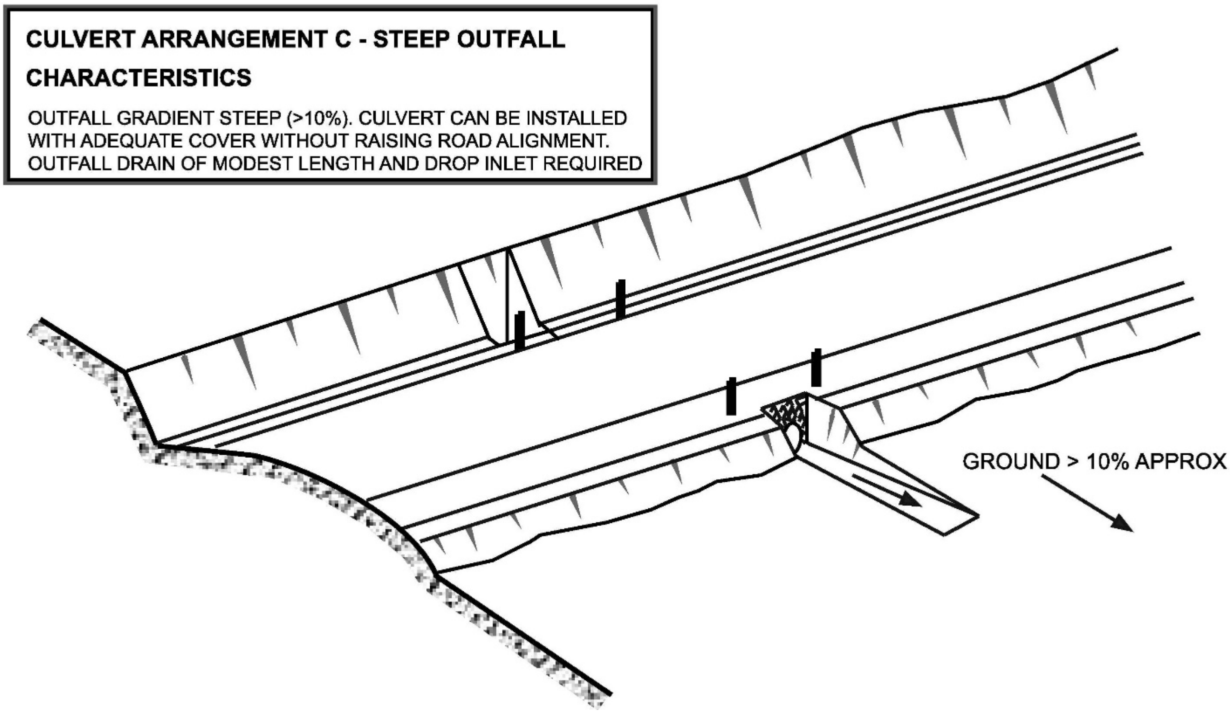


Figure C.9.10: Culvert Arrangement C - steep outfall

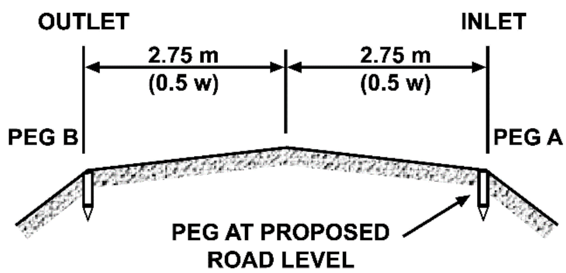
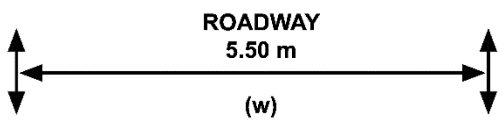
Calculating the required depth of excavation

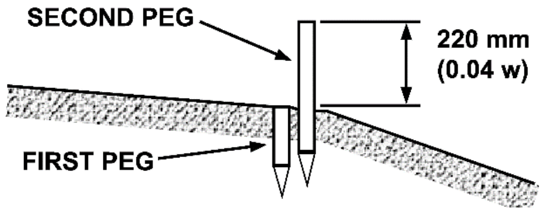
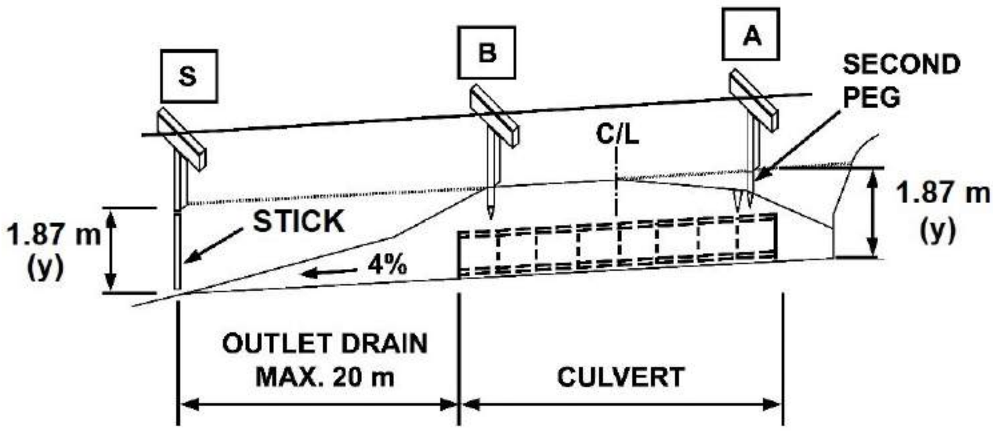
Culverts should be set out according to the design and detailed drawings. The main aspects to be set out accurately are the center-line of the culvert barrels, the culvert extent and the inlet and outlet levels of the culvert.

If the culvert site is flat, the watercourse gradient must be checked for 20 meters downstream from the location of the culvert outlet. Boning rods and an Abney Level, or a line and level can be used for this purpose. If the gradient is less than 5% (1 meter fall in 20 meters) then the culvert must be constructed in Arrangement A with the culvert inverts as close to existing ground/watercourse level as possible. Arrangement A is also constructed if the height of the embankment fill (measured from ground level to the edge of the road running surface) at the culvert site is at least 1.1 meters.

If the conditions for Arrangement A are not in place Arrangement B or C is required. Guidance on calculating the correct depth for installing a 600 mm diameter culvert under Arrangement B or C is given in Table C.9.7. The roadway width is assumed to be 5.5 meters but the dimensions in brackets are for a cross-section with roadway width of ‘w’ meters.

Table C.9.7: Calculating the depth for culvert installation

Procedure step by step	Example and explanation
STEP 1 Fix the center-line of the culvert. Establish two pegs (Peg A and Peg B) at the location of both roadway edges and at proposed finished roadway level. Make sure that pegs are on the same level (use line and level or Abney level).	
STEP 2 Measure the distance between peg A and B (5.50 m, or ‘w’ for other cross sections).	

<table>
<tr><th>Procedure step by step</th><th>Example and explanation</th></tr>
<tr><td>STEP 3
Calculate the minimum depth (d) to be excavated from the proposed road level to the underside of the culvert pipe at the inlet to ensure adequate cover (at Peg A).</td><td>Outside diam of 900 mm culvert = 1.05 m
<u>Cover to culvert pipe = 0.60 m</u>
Total depth, d = 1.65 m</td></tr>
<tr><td>STEP 4
Calculate the difference in culvert level between Peg A and B with the chosen culvert gradient (4% in this case).</td><td>Difference in level = 4% x 5.50 m = 0.22 m
(for road width w, difference in level = 0.04w)</td></tr>
<tr><td>STEP 5
Calculate the depth to be excavated from the
proposed road level to the underside of the culvert at the outlet (Peg B).</td><td>Road width (m) 5.50 m w
Inlet depth 1.65 m 1.62 m
<u>Difference in level + 0.22 m</u> <u>0.04w</u>
Depth at outlet 1.87 m y = (1.62 + 0.04w)</td></tr>
<tr><td>STEP 6
Raise the level of Peg A by the same measurement calculated under Step 4, establishing a second peg (difference in level).</td><td></td></tr>
<tr><td colspan="2">STEP 7
Find the end of the outlet drain by using boning rods and a stick of length 1.87 m (‘y’ for other cross-section road widths). Walk the boning rod S and the stick away from B until the tops of A, B and S are in line (see the sketch below).

If the length of the outlet drain SB is less than 20 meters, then establish the drain outlet peg at ground level at point S. Construct the culvert in Arrangement C (i.e. the road alignment will not need to be raised). Establish the excavation level for the underside of the culvert pipe by measuring vertically down 1.87 m (‘y’) from Peg B and the top of the second peg at point A. The excavation pegs should be 5.50 m apart (‘w’ for other cross-section road width).</td></tr>
</table>

Procedure step by step	Example and explanation
STEP 8 If the drain outlet cannot be found within 20 meters of the culvert outlet (i.e. the ground is too flat), follow the following procedure: ▪ Place a peg at ground level at the point S, 20 meters away from the culvert outlet (point B); ▪ Adjust the boning rod at point S until the tops of the 3 boning rods A, B and S are in line; and. ▪ Measure the distance from the bottom of the boning rod S to the ground level: z meters. ▪ The road level at A and B will have to be raised by 1.87 - z meters (y - z for other cross sections). To fix the culvert inlet excavation level, measure (1.87 - z) meters (y - z for other cross sections) down from the top of the second peg at point A. To fix the culvert outlet excavation level, measure (1.87 - z) meters (y - z for other cross sections) down from the peg at point B (these pegs should be 5.50 m apart, or w for other cross sections). This is Arrangement B.	

Note:

- Where pipes will be added on imported material, the excavation levels must be lowered by the thickness of the bedding material.

Establishing the depth of excavation

The method for establishing the depth of excavation for a culvert involves excavation of a trench from the inlet to outlet as shown in Figure C.9.11 and Figure C.9.12. To ensure a correct and uniform level of the trench, profile boards are placed at the inlet and outlet positions, with the level boards b meters above the level of the excavated trench. A third profile board with a fixed height is useful for controlling excavated levels between the adjustable profile boards. It is known as the travelling profile or *traveler*. During excavation along the line, from inlet to outlet points, the traveler can be used to control that the correct levels have been achieved. By placing the traveler in the sight line between inlet and outlet, it is easy to determine whether the excavation has been carried out to correct levels. If the top of the traveler is below the sight line between the two fixed profile boards, the ditch has been excavated too deep. If the traveler sticks up above the sight line the ditch needs to be dug deeper.

To provide accurate guidance for the final excavation, it is useful to dig slots at regular intervals of 4 to 5 meters along the sight line. When sufficient slots have been dug, the workers can start excavating the trench by joining up the excavated slots. The traveler is then used once again to ensure that the finished work is to the correct level and that there are no high or low spots.

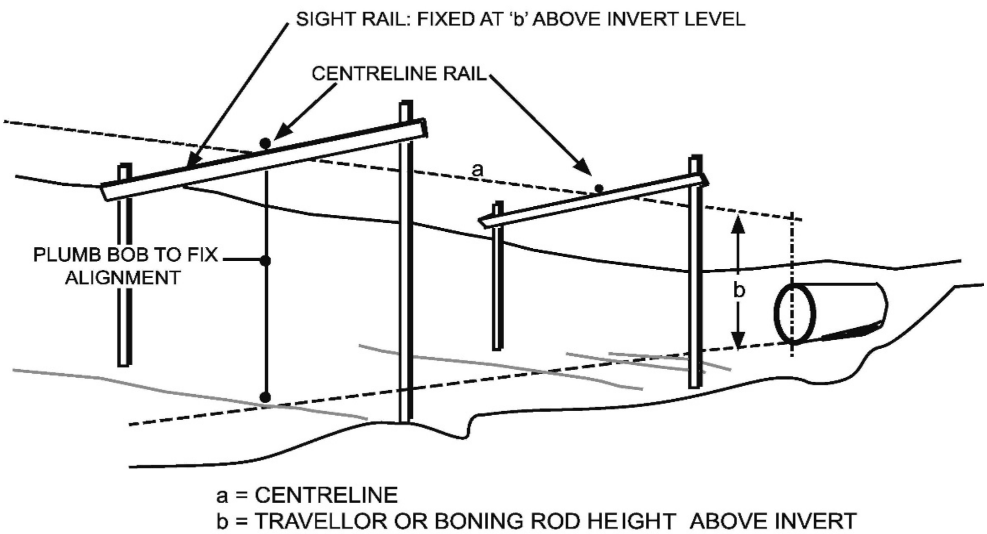


Figure C.9.11: Setting out a culvert profile

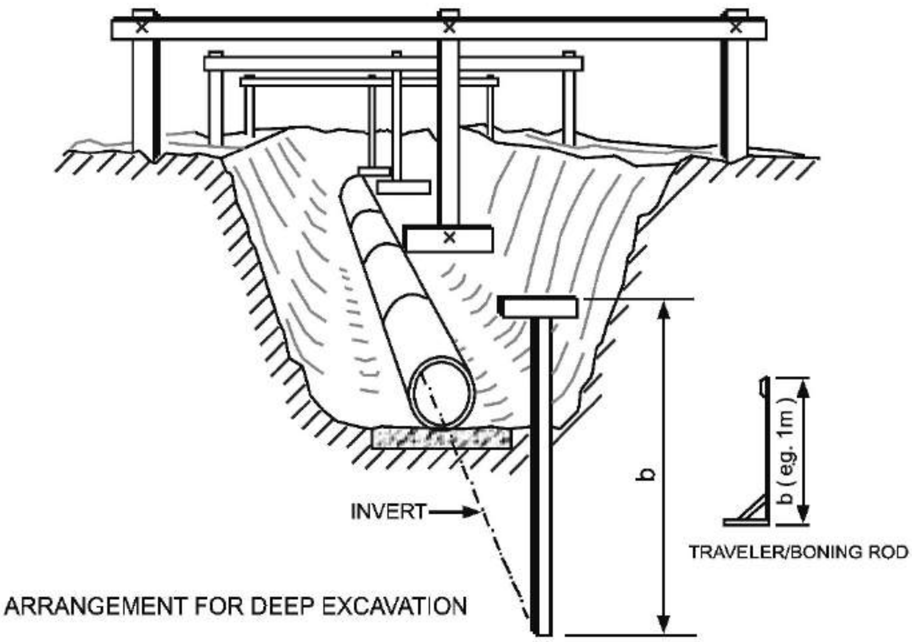


Figure C.9.12: Setting out culvert profile (deep excavation)

9.5 Site administration

The activities listed in Table C.9.8 are required to be carried out in support of the site works. It is important to keep accurate records of the actual works carried out to compare to the planned progress, resource use and expenditure. This helps to ensure value for money. Any problems encountered should be recorded along with explanations of how they were overcome. This assists in explaining any delays or cost over runs and helps to improve future planning of structures.

Table C.9.8: Checklist of site administration tasks

Serial No	Task
1	Set and review individual/gang task rates
2	Daily labor and work achievement record for site labor force
3	Daily diary of works achieved; problems encountered and methods of solving them should be recorded
4	Update work program
5	Daily checks on site stores, tools, materials, re-order as necessary
6	Daily checks and service of site equipment
7	Testing of materials, inspection and quality control
8	Prepare payrolls
9	Arrangements for payment of labor force
10	Keep a careful record of all costs
11	Reporting of progress to Employer/senior management
12	Safety and first aid arrangements

Other important aspects of site administration include:

- Weekly and daily programs should be prepared based on the Bills of Quantities, the overall works program and expected local productivities. Adjustments are required continuously based on actual experience. Weekly reports should be prepared for management monitoring purposes. Key indicators should be used to monitor the progress of the work, such as cement or worker days used, against the quantities planned.
- For structures, 'as-built' drawings should be prepared. These should particularly record differences from the original design, and important details such as actual foundation levels and concrete strengths. If there are no changes, this is also valuable information.
- A cost analysis of the completed structure should be carried out to enable cost estimating of future structures to be more accurate.
- A final inspection of the completed structure should be carried out prior to handing over to the authority that is responsible for its maintenance.
- It is advisable for an independent performance audit to be carried out on a completed structure to review the works. This should verify the structure's 'fitness-for-purpose' and value for money.

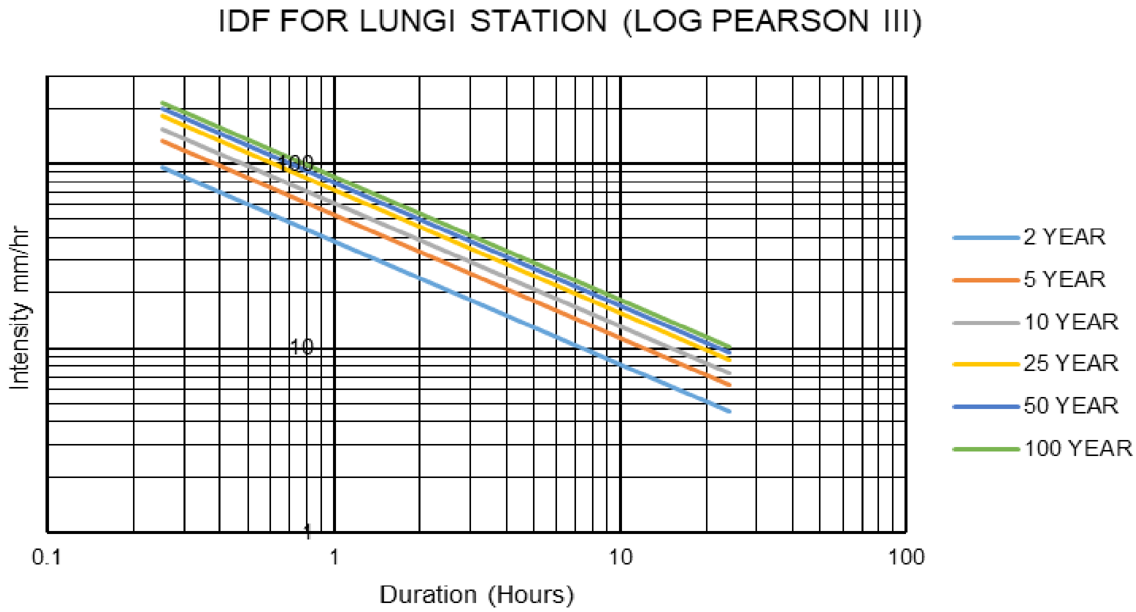
10. REFERENCES

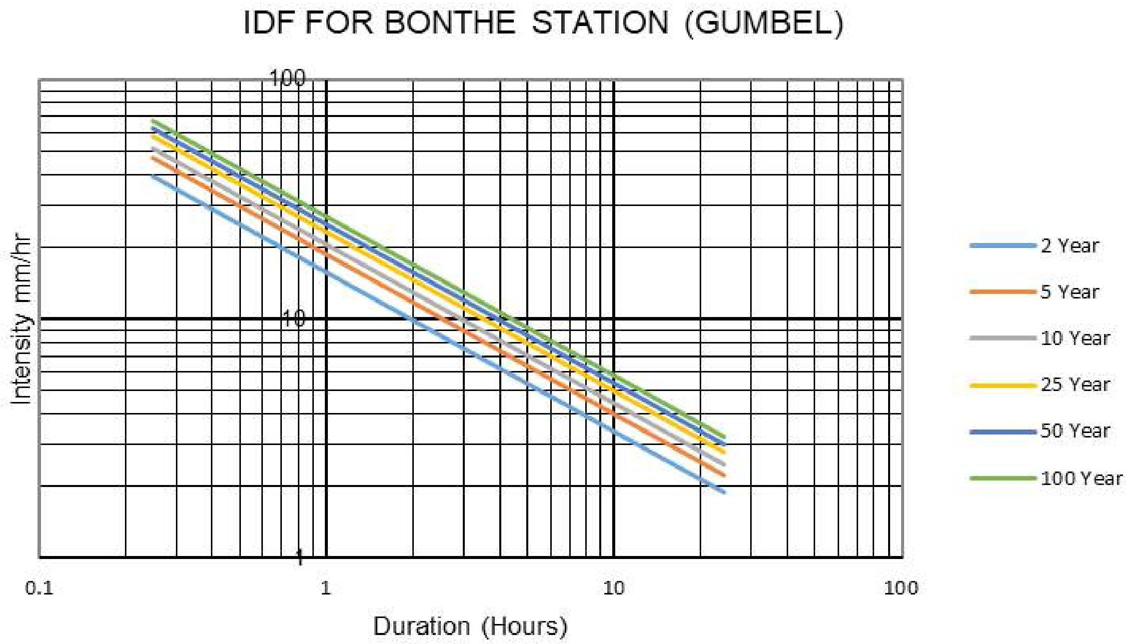
- Administração Nacional de Estradas, Mozambique, 2014. Low Volume Rural Roads Manual Volume 1, Planning and Design. Transport Research Laboratory. AfCAP.
- Building & Road Research Institute, Ghana, 1971. Laterite and lateritic soils and other problem soils of Africa. An engineering study for the Agency for International Development, Aid/csd-2164. Lyon Associates Inc, USA.
- City of Cibolo, Texas, USA, 2013. Stormwater Design Guidelines.
- Civil Engineering Development Division, Hong Kong, 2017. Guide to reinforced fill structures and slope design. Geoguide 6.
- Council for Scientific and Industrial Research, Paige-Green Consulting (Pty) Ltd and St Helens Consulting Ltd, 2018. Climate Adaptation: Risk Management and Resilience Optimisation for Vulnerable Road Access in Africa. Change Management Guidelines. AfCAP.
- Council for Scientific and Industrial Research, Paige-Green Consulting (Pty) Ltd and St Helens Consulting Ltd, 2018. Climate Adaptation: Risk Management and Resilience Optimisation for Vulnerable Road Access in Africa. Climate Adaptation Handbook. AfCAP.
- Council for Scientific and Industrial Research, Paige-Green Consulting (Pty) Ltd and St Helens Consulting Ltd, 2018. Climate Adaptation: Risk Management and Resilience Optimisation for Vulnerable Road Access in Africa. Climate Threats and Vulnerability Assessment Guidelines. AfCAP.
- Council for Scientific and Industrial Research, Paige-Green Consulting (Pty) Ltd and St Helens Consulting Ltd, 2018. Climate Adaptation: Risk Management and Resilience Optimisation for Vulnerable Road Access in Africa. Engineering Adaptation Guidelines. AfCAP.
- Department of Feeder Roads, Ghana, 2005. Guidance Notes for the Design of Rural Feeder Roads.
- Department of Feeder Roads, Ghana, 2004. Site Supervision Pocketbook.
- Department of Transport, Florida, 2016. Plans Preparation Manual Vol. 1. Reinforced Concrete Box and Three-Sided Culverts.
- Ethiopian Roads Authority, 2017. Manual for Low Volume Roads Manual. AfCAP.
- Federal Highway Administration, USA, 2012. Hydraulic Design of Culverts. Hydraulic Design Series Number 5, Third Edition. Publication No. FHWA-HIF-12-026
- Ghana Highway Authority, 1991. Road Design Guide.
- Government of Sierra Leone, 2011. National Rural Feeder Roads Policy.
- Hearn, G.J. (ed), 2011. Slope Engineering for Mountain Roads, Engineering Geology Special Publication 24, Geological Society of London.
- Hunt, T., Hearn, G., Chonephetsarath, X. and Howell, J. 2008. Slope Maintenance Manual. Ministry of Public Works and Transport, Roads Administration Division, Laos.
- Keller, G. and Sherar, J., 2003. Low Volume Roads Engineering, Best Management Practices Field Guide. USAID.
- Ministry of Local Government and Rural Development, Ghana, 2014. Labour-Intensive Public Works Manual - Practitioners Guide to Rural Roads Improvement and Maintenance. ILO.
- Ministry of Public Works, Liberia, 2012. Feeder Roads Design Manual and Specifications. Liberia-Swedish Feeder Road Programme.
- Ministry of Public Works, Liberia, 2014. Best Practice Guidelines. Liberia-Swedish Feeder Road Programme.
- Ministry of Transport, Ghana, 2007. Standard Specification for Road and Bridge Works.
- Ministry of Works, Transport and Communication, Tanzania, 2016. Low Volume Roads Manual. AfCAP.
- Poilecot, P., 2015. Guide to Liberian grasses. The Nimba Mountains, Bong and the coastal area around Buchanan. ArcelorMittal Liberia Ltd/Cirad. Developed from a report prepared by URS Scott Wilson.

- TRL, 2000. Overseas Road Note 9. A Design Manual for Small Bridges. Second Edition. Funded by DFID.
- United States Department of Agriculture, 1994. Gradation Design of Sand and Gravel Filters. Part 633, National Engineering Handbook.

APPENDIX C.1: RAINFALL INTENSITY DURATION FREQUENCY CURVES

The IDF curves were developed from rainfall data provided by the meterological services of Sierra Leone. Similar data were not available for Liberia at the time of prepartion of this manaul. It is recommmeded that the Sierra Leone IDF curves should be used for flow estimation in Liberia using the Rational Method until such data are available for Liberia. Two curves are provided, for Lungi Station and Bothe Station. The Lungi Station curves should be used in areas where high intensity rainfall is expected and the Bonthe curves for areas where lower intensity rainfall is more likely.





APPENDIX C.2: DETERMINATION OF CATCHMENT AREA

(Source: Ghana Department of Feeder Roads Guideline Notes for Design of Drainage Structures)

Ideally the flows in each watercourse would be estimated from gauge flows collected over many years. However, this is a costly and time-consuming exercise that is rarely worthwhile even for major highway bridges. So, if flow-gauging information is available use it, however, it will be necessary to estimate most flows in rivers and stream using the information available to the drainage designer including catchment characteristics and rainfall data.

Whatever method is used to estimate flows similar data on the catchment is needed. So, a site inspection field sheet should be prepared for each catchment.

- The land use can be taken directly from the field inspection sheets.
- The soil type and evidence of waterlogging can be taken directly from the field inspection sheets. Together these give an indication of permeability.

Other catchment characteristics can be measured from the 1:50,000 map(s). First the catchment boundaries are drawn on the map. This process is described below. This may be difficult in very flat catchments. Plotting catchments and approaches for flat catchments are also discussed in the next section.

The length of the catchment is measured from the most remote point in the catchment to the culvert location. The length is defined on the example below.

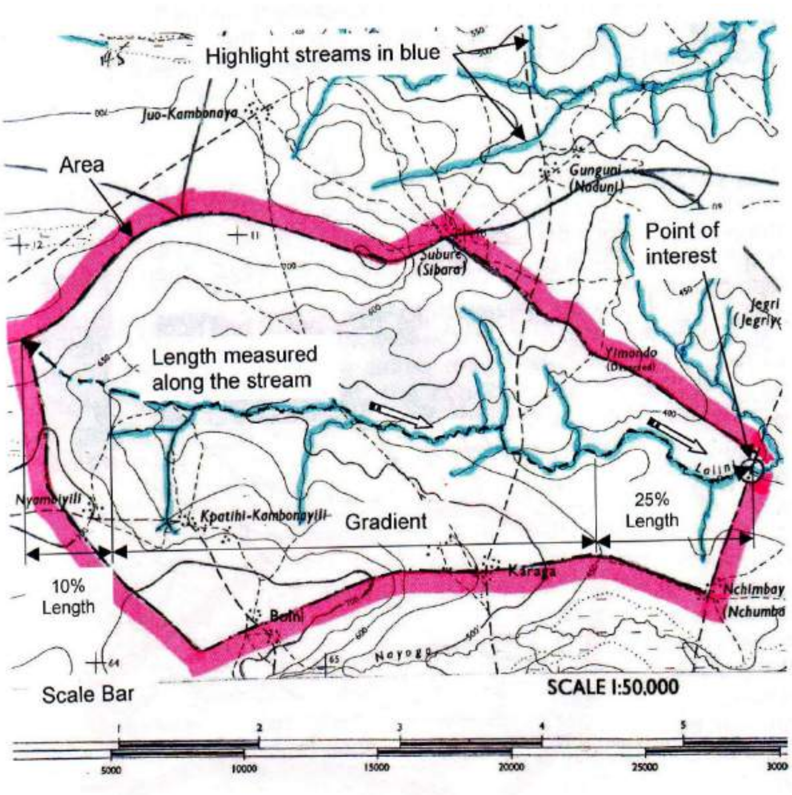


Figure 1: Measurement of catchment length

The general Slope of the catchment should be a typical value for the entire catchment and can conveniently be measured between contours at approximately the 10% and 75% points on the longest drainage path in the catchment.

Flow from the catchment is directly related to the size of the catchment. The catchment area can be determined and measured accurately from the 1:50,000 map. This is one of the few steps in estimating flows where it is possible to be precise so take care to be accurate.

The first stage is marking up each catchment on the 1:50,000 map. If using a photocopy draw a scale bar on the area before it is photocopied so that the scale can be determined even if the map is enlarged or reduced.

Identify the Catchment

The catchment boundary will follow ridges connecting the high points around the edge of the catchment. However, the contours that mark the ridges can look the same as the contours across a valley. So first highlight streams in blue as these streams mark the valleys. Then identify any high points.

Then it is a matter of joining the high points and ridges around the catchment boundary. At all times the catchment boundary will cross contours at right angles. Often paths will follow a ridge, where the ground is driest, and can be used as a guide. This is demonstrated on Figure 2.

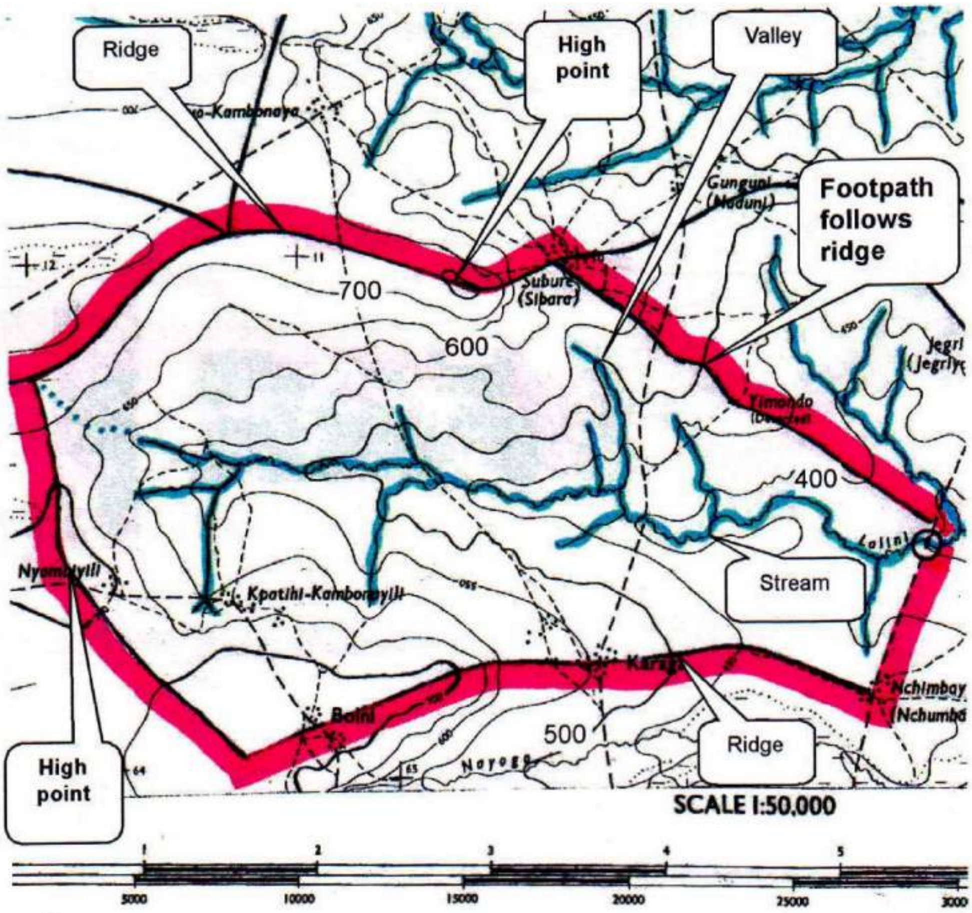


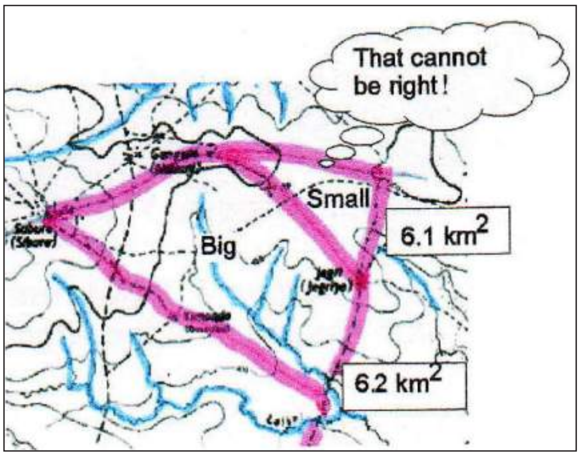
Figure 2: Marking the catchment on a 1:50,000 map

Once the catchment is marked up it should be checked:

- Does the boundary line always cross the contours at right angles? If not, correct it.
- Does the boundary follow any stream lines? If it does then correct it.
- Are there any gaps between catchments? If there are then correct it.

Measure the Catchment

The catchment area can be measured using a planimeter, by dividing the area into convenient triangles and rectangles or by tracing the catchment on a sectional sheet and counting squares. The planimeter should be checked and calibrated before use. Mark up a simple easily measured area, possibly a square 10cm x 10cm (25 km^2 at 1:50,000). Measure it with the planimeter 10 times. Check that you get the same result each time and either adjust the planimeter to get the correct area or calculate a calibration factor. Once you have measured each catchment area check it:



First perform a rough check on the size of the catchment by approximating the whole catchment to an easily measured shape such as a triangle or rectangle. This is also important as the length to width ratio gives a numerical indication of the shape of the catchment

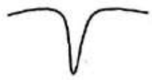
Then compare the measured catchment area with those on each side of it? Is it bigger, smaller or a similar size? How does it look on the map - bigger, smaller or a similar size? Are these consistent? If not find the error and check it.

Flat Catchments

Marking up catchments and measuring catchment characteristics is easy when there are lots of contours. In flat areas where there are very few contours it is more difficult. It will be necessary to make some assumptions. These should be logical and reasonable and should not be arbitrary.

Procedure

1. Start by finding a contour or lake edge. You will also need to work out which side of the contour is uphill, and which is downhill.
2. There will be deviations in the contour as shown on the right. If these deviations point uphill, they are valleys.



3. Assumption 1. You can assume that water would flow down the middle of that valley.
4. Assumption 2. You can assume that the valley is straight.
5. Draw a blue line along the valley and extending upstream and downstream.
6. There will be other lobes on the contour that point downhill. These are ridges.

7. Assumption 3. You can assume that the ridgeline (watershed) follows the middle of that lobe.
8. Assumption 4. You can assume that the ridge is straight.
9. Assumption 5. If two rivers flow approximately parallel, you can assume that the ridge is midway between them.
10. Assumption 6. If two rivers flow away from the same area of high ground, you can assume that the ridge is halfway between them.

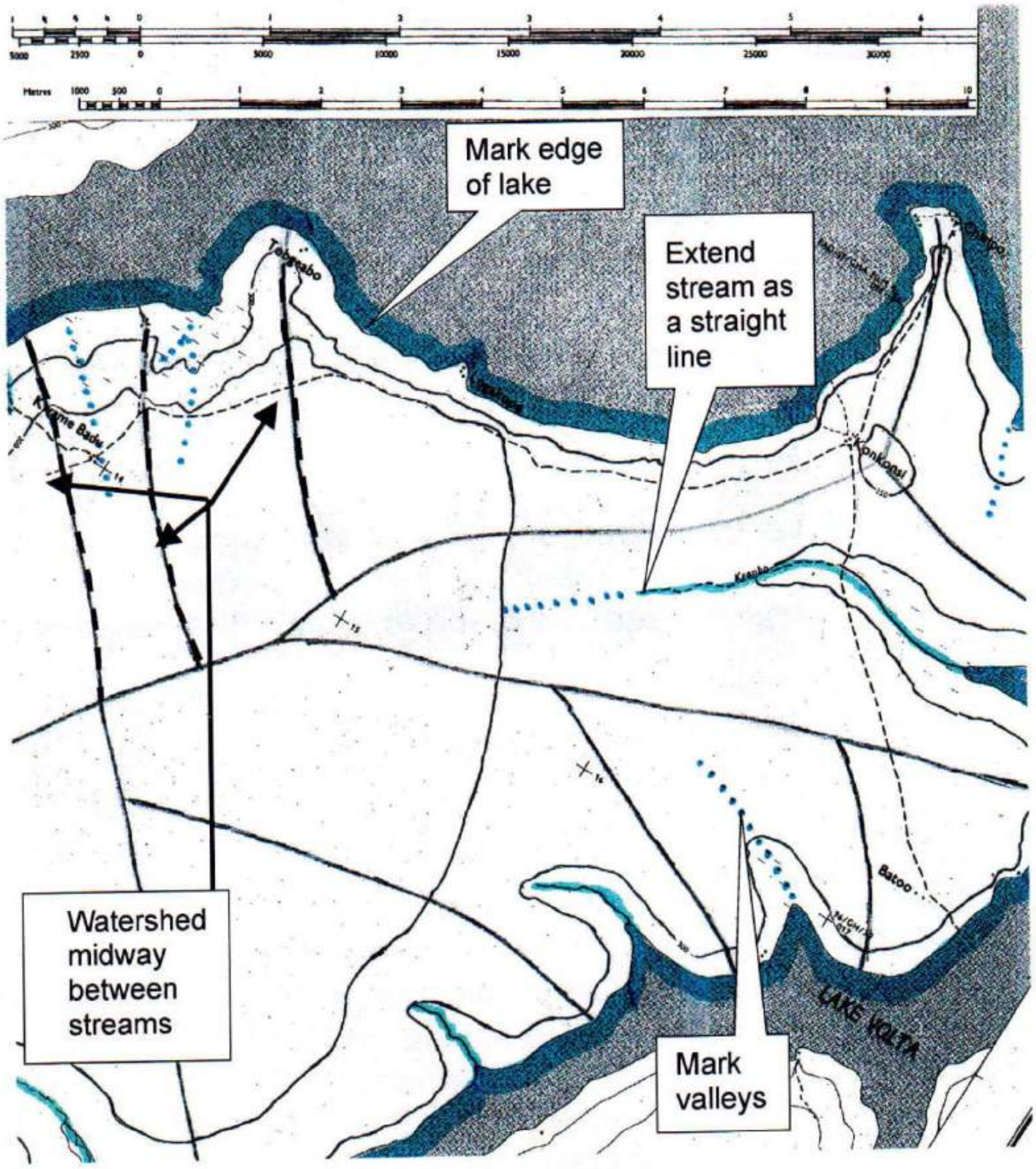


Figure 3: Marking the catchment in flat areas

APPENDIX A.3: HAND CALCULATION SHEETS

WORKED EXAMPLE 1 - HAND-CALCULATION OF CULVERTS

Location: KM 5+450

Sheet No.: 001

CULVERT DESIGN

Prepared by:..

Checked by:..

Date..

	DESIGN DATA							CULVERT DATA					INLET CONTROL			OUTLET CONTROL									
Sta.	Q, m^3/s	d, m	d_e, m	AHW, m	Skew, No.	L,m	S, m/m	Descip	D or BxD m	N	Q/N, m^3/s	A, m^2	Q/NB,$m^3/s/m$	HW/D	HW, m	K_e	H, m	d_c m	$(d_c+D)/2$m	TW,m	h_0, m	LS, m	HW, m	GOVG, HW, m	VEL, Vo, m/s
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26
KM2+100	1.3	0.9	0.3	1.28	90	15	0.01	CPC	1.2	2	0.65	2.26	-	0.74	0.89	0.5	0.2	0.63	0.92	1.2	1.2	0.15	1.25	1.25	

Governing HW = 1.25 is near equal to the AHW 1.e. 1.28. Hence, Culvert flow under Outlet Control

From Figure C.4.5 in the manual, with flow under outlet control conditions, $V_o = Q/A$

Q, maximum flow = $1.3m^3/s$

A = Area corresponding to TW depth = 2x{3.142 x (1.2x1.2)/4 } = $2.26m^2$

$V_o = 1.3/2.26$

$V_o = 0.58m/s$

Thus $V_o < 0.6m/s$ provide for siltation control

Col. 2 Peak Flow (Maximum discharge)
3 Flood depth
4 Embedment below channel invet
5 Col. 3 + col.4 + allowable backwater
7 Culvert Length (Allow for skew if applicable)
8 Culvert slop slope channel
10 D (pipes) or B x D (rectangular)

Col. 11 No. of barrels
13 Area per barrel
14 For box only
15 Determine using Figures C.4. 6
16 HW = col. 15 x D (col. 10)
17 Determine from Table 4.2
18 Determine using Figures C.4.8

Col. 19 Determine using Figures C.4.10
21 Col. 3 + col. 4
22 ho = larger of cols. 20 & 21
23 Col. 7 x col. 8
24 HW = col. 18 + col. 22 - col.23
25 Larger of cols. 16 &24
26 Outlet vel. If req'd (Subsec. 3.2.3)

WORKED EXAMPLE 2: HAND-CALCULATION OF CULVERTS

Location: KM 1+300
Sheet No.:001

CONVENTIONAL CULVERT DESIGN

Prepared by: F.O.O
Checked by: ROB
Date: 29/01/2018

	DESIGN DATA							CULVERT DATA					INLET CONTROL			OUTLET CONTROL									
Sta.	Q, m^3/s	d, m	d_e m	AHW, m	Skew, No.	L,m	S, m/m	Descip	D or BxD m	N	Q/N, m^3/s	A, m^2	Q/NB,$m^3/s/m$	HW/D	HW, m	K_e	H, m	d_c m	$(d_c+D)/2m$	TW,m	h_0, m	LS, m	HW, m	GOVG, HW, m	VEL, Vo, m/s
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26
KM 2	28.3	1.9	0.3	3.38	90	107	0.005	RC BOX	3.5 x 3.0	1	28.3	10.5	8.09	0.95	2.85	0.2	0.63	1.87	2.44	2.20	2.44	0.54	2.54	2.85	-
								GOVERNING HW WAS LESS THAN AHW; TRY ANOTHER SPAN																	
KM 2	28.3	1.9	0.3	3.38	90	107	0.005	RC BOX	3.0 x 3.0	1	28.3	9.00	9.43	1.10	3.30	0.2	0.90	1.06	2.53	2.20	2.53	0.54	2.89	3.30	5.20
							GOVERNING HW SLIGHTLY LESS THAN AHW, THEREFORE OK. FOR CALCULATION OF V_o SEE FLOW CHART																		

Col. 2	Peak Flow (Maximum discharge)	Col. 11	No. of barrels	Col. 19	Charts C4.10 & C4.11
3	Flood depth	13	Area per barrel	21	Col. 3 + col. 4
4	Embedment below channel invet	14	For box only	22	ho = larger of cols. 20 & 21
5	Col. 3 + col.4 + allowable backwater	15	Charts C4. 6 to C4.7	23	Col. 7 x col. 8
7	Allow for skew if applicable	16	HW = col. 15 x D (col. 10)	24	HW = col. 18 + col. 22 - col.23
8	Culvert slop slope channel	17	Table C4.2	25	Larger of cols. 16 &24
10	D (pipes) or B x D (rectangular)	18	Charts C4.8 & C4.9	26	Outlet vel. If req'd

APPENDIX C.4: DRAINAGE STRUCTURAL DRAWINGS

General arrangement drawings and specifications for typical structures are published as a separate volume at A3 size. Details are provided for the following standard drainage structures:

- Reinforced Concrete Pipe Culvert (Single);
- Reinforced Concrete Pipe Culvert (Double);
- Reinforced Concrete Box Culvert (Single);
- Reinforced Concrete Box Culvert (Double);
- Mass Concrete Relief Culvert; and
- Vented Drift.