REPUBLIC OF GHANA

MINISTRY OF ROADS AND HIGHWAYS

MANUAL FOR LOW VOLUME ROADS

PART C - HYDROLOGY, DRAINAGE DESIGN AND ROADSIDE SLOPE STABILISATION

2019

MESSAGE FROM THE MINISTER

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

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

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

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

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

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

PART C

Manual for Low Volume Roads

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

FOREWORD

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

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

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

Manual Updates

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

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

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

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

PREFACE

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

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

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

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

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

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

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

Ing. Bernard Badu
Director of the Department of Feeder Roads

ABBREVIATIONS, ACRONYMS AND INITIALISMS

>	:	Greater than
<	:	Less than
%	:	Percentage
A	:	Area of catchment
ABW	:	Allowable Back Water
AC	:	Asphalt Concrete
AHWEL	:	Allowable Headwater Elevation
AHW	:	Allowable Head Water
AfCAP	:	Africa Community Access Partnership
ARF	:	Area Reduction Factor
B	:	Width (width) of a box culvert
BEDEL	:	Bed elevation
C	:	Runoff Coefficient
C_A	:	Overall runoff Coefficient
CERGIS	:	Centre for Remote Sensing & Geographic Information Services
CSIR	:	Council for Scientific and Industrial Research
d	:	Distance
d_c	:	Critical depth
d_r	:	Distance from remotest part of catchment
D	:	Diameter of pipe culvert
D_{tw}	:	Depth of flow
DFR	:	Department of Feeder Roads
DFID	:	UK Government's Department for International Development
e.g.	:	For example (abbreviation for the Latin phrase exempli gratia)
F_p	:	Peak Flow Factor
F_s	:	Shape Flow Factor
FHWA	:	Federal Highways Administration (USA)
g/m^2	:	Grams per square metre
GHA	:	Ghana Highway Authority
GIS	:	Geographic Information System
H	:	Height (of box culvert)
H_e	:	Head at culvert inlet
h_o	:	Head at culvert outlet
hr	:	Hour
HFL	:	High Flood Level
HW	:	Headwater
HWEL	:	Headwater Elevation
I	:	Rainfall Intensity

IDF	:	Intensity-Duration-Frequency
i.e.	:	That is (abbreviation for the Latin phrase id est)
ILO	:	International Labour Organisation
IL_D	:	Drift Invert Level
IL_o	:	Downstream Channel Invert Level
INVEL	:	Invert Level
K_e	:	Entrance Loss
kg	:	kilogram
kg/m^2	:	kilogram per square metre
km	:	kilometre
km^2	:	square kilometres
km/h	:	kilometres per hour
kN	:	kilonewtons
kN/m_2	:	kilonewtons per square metre
L	:	Length of Stream
LVR	:	Low Volume Road
LVSR	:	Low Volume Sealed Road
m	:	metre
m^2	:	square metres
m^3	:	cubic metres
m^3/s	:	cubic metres per second
mm	:	millimetre
mm/hr	:	millimetre per hour
mm^2	:	square millimetre
mm^3	:	cubic millimetres
m/s	:	metres per second
MRH	:	Ministry of Roads and Highways
MRM	:	Modified Rational Method
MSA	:	Meteorological Services Agency
n	:	Manning's Roughness Coefficient
N	:	Number of barrels/cells (of a culvert)
P	:	Areal Rainfall Depth
P_w	:	Wetted Perimeter
q	:	Unit Discharge
Q	:	Peak Flow
Q_d	:	Design Discharge
Q_m	:	Mean Flow
R	:	Hydraulic Radius
S	:	Local river bed Slope
S_c	:	Catchment Slope
Sec	:	Second

t	:	Time
T_B	:	Base Time
T_c	:	Time of Concentration
T_s	:	Land Use
TW	:	Tailwater
TWEL	:	Tailwater Elevation
V	:	Average Velocity of Flow
V_c	:	Critical Velocity
V2	:	Volume 2
W	:	Channel Width
WBS	:	Work Breakdown Structure
WinTR20	:	Windows version of Technical Release 20
yr	:	Year

GLOSSARY OF TECHNICAL TERMS

Apron

The flat paved area constructed at the inlet or outlet of cross-drainage structures in order to prevent erosion.

Camber

The convex or arched shape of a road intended to ensure drainage from the road centre-line to side drains.

Catch Water Drain/Cut-off Drain/Auxiliary Drain:

A ditch constructed on the uphill side designed to intercept or collect and drain away surface runoff water flowing towards the road from the uphill side and lead it to a suitable point of disposal. Also known as a Cut-off Drain or Auxiliary Drain.

Complementary Intervention

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

Culvert

A structure constructed under the road, designed to allow to safely cross under the roadway.

Crossfall

A transverse gradient across the road surface that allows water to flow towards the lower edge.

Crown

The vertical height between the bottom of the drain and the highest point of the road surface.

Cut-off Wall / Curtain Wall

A vertical wall under the apron to prevent water seeping under the structure and undermining it. Also known as a Curtain Wall.

Drainage Factor

Product of the height of the crown of the road above the bottom of the drain and the horizontal distance from the centreline of the road to the bottom of the drain.

Ford / Drift

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

Freeboard

The height difference between the top flood water level and the soffit.

French Drain

A subsoil drainage structure comprising a trench filled with gravel or rock or containing a perforated pipe that, in the context of roads, redirects surface water and groundwater away from the road pavement structure.

Gradient

The longitudinal slope (typically between 2% and 5% for a culvert).

Headwall

Retaining wall at the entry or exit of the culvert to retain and protect the embankment or retained soil and/or gravel.

Hydraulics

The study of the motion of water in relation to adjacent materials and structures. Hydraulic analysis is carried out to estimate the size of drainage structure that will allow the calculated peak or maximum flow for particular catchments to pass.

Hydrology

The study of the movement of water in relation to land. Hydrological analysis is used to estimate the peak flows from a specific catchment at a given location.

Invert

The lowest surface of the internal cross section of a drainage channel. This usually varies through the length of the culvert or other cross-drainage structure.

Mitre drain

Drain that take water from the side drain into the natural drainage of the surrounding environment. Also referred to as an off-shoot or turn-out.

Meadow drain

Construction for drainage purposes, a continuous depression, typically shallow, in the surface that avoids abrupt changes in surface profile.

Permeability

A measure of the ease with which water passes through a material

Scour Check

A small structure placed across the drain on steep gradients. Designed to prevent erosion of drain invert and slopes by slowing down the flow of water and reducing the amount of suspended material in the water.

Soffit

Top inside edge of an enclosed drainage channel

Vented Ford / Vented Drift / Vented Causeway

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 following periods of heavy rainfall.   Also known as a Vented Drift or Vented Causeway.

Weep holes

Holes intended to relieve uplift pressure and reduce hydrostatic pressure behind headwalls and other retaining walls.

Wetted Perimeter.

The total length in cross section of the face of a channel that is in contact water.

Wingwall

Retaining wall at the side of the culvert or large structures to retain and protect the embankment or retained soil.

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1. INTRODUCTION

1.1 General

It is said that the three most important aspects of road design are drainage, drainage, and drainage. Directly or indirectly, water is often the cause of damage to or failure of roads and associated structures. It is therefore necessary to give careful consideration to all aspects of drainage design.

The drainage associated with any road can be divided into two broad categories: drainage of the catchment area traversed by the road; and 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. In order to minimise erosion damage to the road, the drainage system and to the adjacent land, rainwater should be discharged away from the road as frequently as reasonably possible.

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

1.2 Purpose and structure of the Manual

The Manual for Low Volume Roads (LVRs) promotes the rational, appropriate, and affordable provision and maintenance of LVRs in Ghana. 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 Ghana 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 Ghana Highways Authority (GHA) and Department of Feeder Roads (DFR). 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 and construction of low volume traffic earth, gravel or paved roads. It is appropriate for roads carrying average daily traffic of up to about 300 vehicles with 4 wheels or more in the base year and less than about 1 million equivalent standard axles over their design life. The Manual 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 Stabilisation; and
- Part D: Surfacings for Low Volume Roads.

Part C of the Manual for Low Volume Roads provides the design standards and guidelines for hydrological and hydraulic design of drainage structures in Ghana. The standards and guidelines are intended for use by Drainage Design Engineers in DFR, GHA, DUR, and private sector consultants.

Guidance on the selection and design of drainage structures is included in Chapters 3 to 6. This is followed in Chapter 7 by guidance on issues related to roadside slope stabilisation, before Chapter 8 then provides guidance on practical aspects related to the construction of drainage structures. Appendix 1 provides examples of hand calculations used in support of worked examples for the hydraulic design of culverts. Standard Drawings for all side drains and culverts are presented in Appendix 2, which is provided as a separate A3 volume.

1.3 Scope

The design standards and guidelines lay out the steps required to design storm drainage structures in a manner that eliminates or manages the risk of flooding of rural LVRs and adjoining areas. The purpose is typically to ensure that rural roads are motorable all year round. 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 Manual covers the main stages involved in the provision of drainage structures, namely:

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 stabilisation.

The Manual is consistent with, and supplements, various other sources of practical guidance prepared for the Ghana context, including:

- Guidance Notes for the Design of Drainage Structures on Rural Feeder Roads (DFR, 2005);
- Guidance Notes for the Design of Rural Feeder Roads (DFR, 2006);
- Practitioners Guide to Rural Roads Improvement and Maintenance (MLGRD, 2014) and
- Ghana compilation guides on: Soils and Natural Gravels (DFR, 2006); The Properties and Production of Concrete (DFR, 2006); and Surfacing and Pavement Options (DFR, 2007)

1.4 Typical drainage features

The typical drainage features of a road are shown in Figure C.1.1.

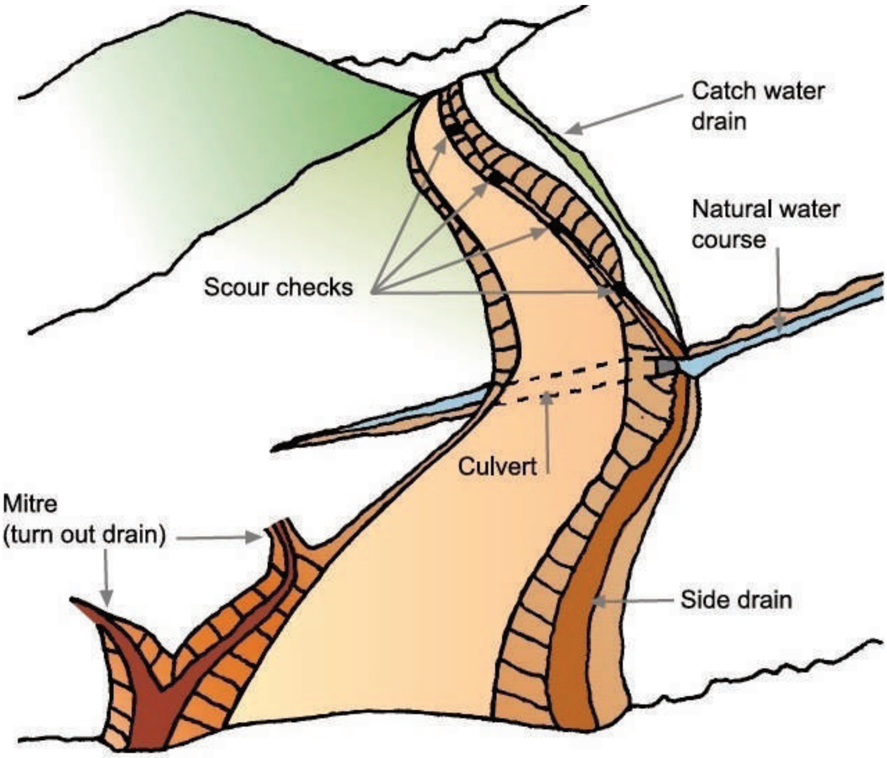


Figure C.1.1 Typical road drainage layout

Typical drainage structures are shown in Figures C.1.2 to C.1.7.

Cross culvert: This is a structure 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.

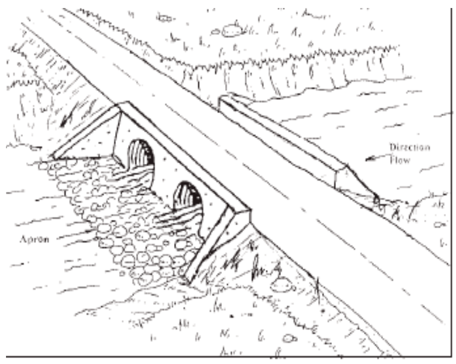


Figure C.1.2 Watercourse culvert

Relief culvert: This 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.

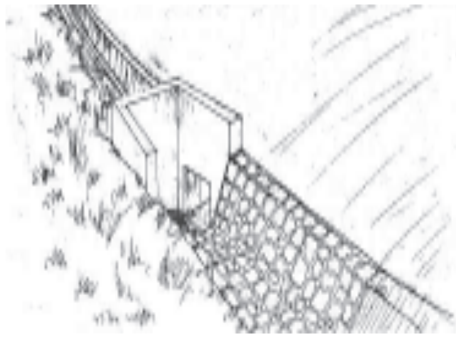


Figure C.1.3 Side drain relief culvert

Access Culvert: This is designed to allow water from the side drains to pass safely under the road at junctions and side access points.

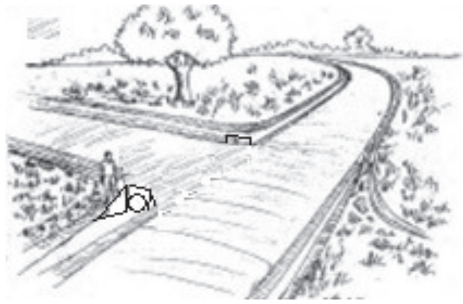


Figure C.1.4 Side drain access culvert

Drift (Ford): This is a low-level structure designed to allow water from the drains and/or natural watercourse to safely cross over the road at bed level.

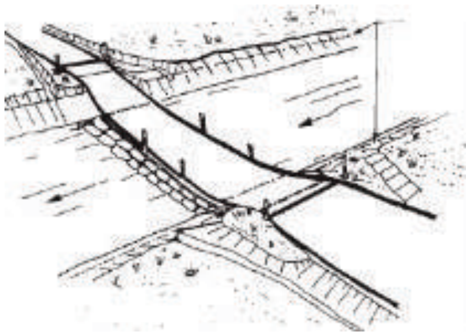


Figure C.1.5 Drift (ford)

Vented Ford (Vented Drift or Causeway): This 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 at times during periods of heavy rainfall.

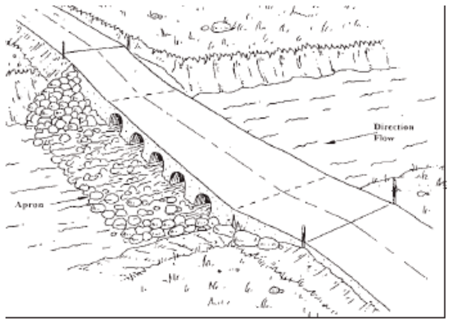


Figure C.1.6 Vented ford/drift

Bridge: This is a structure with at least one span more than 6 metres long that provides a means of the roadway crossing safely above water, a railway or some other obstruction whether natural or artificial.

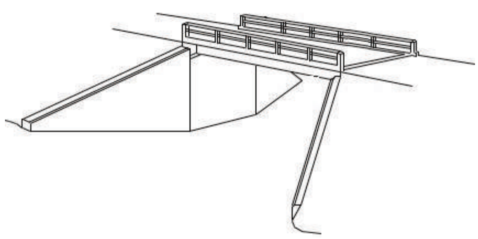


Figure C.1.7 Single span bridge

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 not represent good value for money. The various standards for the design of drainage are based on different levels of risk that are attached to the likely occurrence of the different storm intensities for which they are designed, assuming that appropriate routine maintenance is carried out. Storm events are defined by the intensity and duration of rainfall and are extremely variable in nature over periods of many years. A statistical distribution of storm severities shows that very severe storms are quite rare and less severe storms 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 such 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" as there is always a finite probability that the worst storm for (say) 200 years may occur at any time.

Unless designed to be submersible, most drainage structures are likely to be severely damaged if their capacity is exceeded. Hence their capacity is the most important aspect of their design. In general, the more severe the storm for which the structure is designed, the more expensive it is to build. 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. These are:

a) the standard of the road (i.e. the traffic level);
b) the cost of the drainage structure; and
c) 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. The structures on such a road are therefore designed for low risk, i.e. for storms of relatively 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 lower, so the higher cost of designing the drainage for low risk cannot be justified and the design is based on shorter storm Return Periods. Table C.1.1 provides indicative guidance to the storm Design Return Periods in years for different drainage structures on LVRs.

Table C.1.1 Design Return Periods

Type of drainage structure	Design Return Period, yrs	Maximum Return Period, yrs
Unlined Side Drains/Ditches	2	5
Lined Side Drains/Ditches	5	10
Drifts	2	5
Vented Fords	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. In addition, site specific factors and other constraints may also prevent ideal design conditions. As a result, the designer must often use simpler and potentially less accurate methods. The designer must select the most appropriate method and in so doing exercise a degree of engineering judgement.

The same arguments do not apply to the detailed engineering design of the components of the drainage system once the maximum water flow has been estimated. For these, standard drawings and specifications are provided. If the designer wishes to depart from these, then written approval is required from the responsible road authority. The designer must submit all proposals for such departures from 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 summarised in Table C.1.2.

Table C.1.2 Minimum culvert size

Culvert type	Watercourse culvert minimum size (mm)	Access and relief culverts minimum size (mm)
Pipe	900 Diameter	600 Diameter
U-Culvert	1200 wide x 900 high	900 wide x 700 high
Rectangular	1000 x 1000	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 metres.

All culverts on LVRs should have a minimum cover of 600 mm.

1.6.2 Minimum and maximum velocities in culverts

The velocity of flow in a culvert is generally higher than that in the equivalent natural channel. High velocities are particularly critical just downstream from the culvert outlet, where the erosion potential from the energy in the water must always be considered in culvert design.

Table C.1.3 Minimum and maximum velocities in culverts

All Culverts	Velocities, m/s	Remarks
Minimum	1.0	Check against siltation/sedimentation
Maximum	3.0	Check against erosion

Low flow velocities give rise to the risk of siltation, particularly where no measures are in place to trap sediment before the water enters the culvert. The minimum and maximum velocities of flow in culverts is are summarised in Table C.1.3.

1.6.3 Freeboard

Freeboard is the height difference between the top flood water level and the culvert soffit (its top inside edge). This space is needed to allow twigs, small branches and other floating debris to pass through. The minimum required freeboard for culverts varies with culvert size and is summarised in Table C.1.4.

Table C.1.4 Freeboard for 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

The provision of multiple barrel culverts at one location rarely results in overall flow rates being reliably as high as would be anticipated based on simplistic design assumptions. This is because barrels have a higher probability of being obstructed by debris.

During construction, as multiple barrel culverts are fitted within the natural dominant channel, minor widening of that channel should be undertaken. This serves to reduce the risk of conveyance loss through sediment deposition in some of the barrels.

Generally, multiple barrels are to be avoided where:

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

1.6.5 Selection of culvert material

In selecting material for the construction of culverts, the Drainage Engineer should consider: cost; durability; skills and experience in their use; as well as the risk of theft. Higher level policy decisions may also be made on the basis of an economic rather than financial appraisal. Such economic analysis takes account of all applicable taxes, duties and subsidies, as well as the opportunity cost of labour. In the Ghana context, the choice between corrugated metal and concrete culverts is influenced, but not necessarily determined, by the fact that:

- corrugated metal culverts are imported, while cement for manufacturing of culverts is obtained locally; and
- Corrugated metal can be used for a range of purposes, giving rise in some locations to a risk of theft.

1.6.6 End treatment (inlet and outlet)

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

a) Headwalls to:
  - provide embankment stability and protection against embankment erosion;
  - 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 arising from high headwater depths or high velocity.

d) Cut-off Walls to:
  - protect the culvert inlets from piping (the development of erosion channels outside the culvert structure); and
  - protect culvert outlet from erosion.

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

1.6.7 Culvert alignment and grade

To achieve hydraulic efficiency, culverts should be installed on the main course of the stream channel. This serves to maintain the natural drainage system and while minimising downstream impacts. 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.

In some cases, it may not be possible or feasible to match the existing grade and alignment. This can arise in situations where culverts are conveying hill-side 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 upstream or downstream of the culvert. This is best evaluated on a case-by-case basis with potential environmental impacts being balanced against construction and functional issues.

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 both upstream and downstream of its location. 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, the velocity of the culvert discharge, and the extent and nature of any material carried in the water. In order to mitigate the risk of erosion, an adequate design must therefore stabilise 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 to an acceptable degree.

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 summarised in Tables C.1.5 and C.1.6.

Table C.1.5 Minimum and maximum dimensions of drains

Type of drainage structure	Min. 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 summarised in Table C.1.7.

Table C.1.7 Carrying capacity of drains (Manning's roughness coefficient)

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 summarised 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 there is a risk of excess water in the subsoil, a drainage blanket or French Drain should be provided in addition to the roadside drains. The purpose of such subsoil drainage is to prevent building up of water pressures and ingress of water into the pavement structure by draining off water that would otherwise collect behind and underneath the surface drains.

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 both to the environmental impact of the structure and to its safety to road users as well as to local inhabitants.

1.9.2 Environmental considerations

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

- 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 the 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 result in discomfort for the occupants, damage to the vehicle, and a potential associated loss of control.

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

Child safety

Streams and culverts tend to attract children. Sudden deep drop-offs in the streambed should therefore 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. Such 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 LVRs, like other roads, can be classified into four different parts namely: (i) surface drainage; (ii) subsurface drainage; (iii) slope drainage; and (iv) drainage of retaining structures. These are illustrated in Figure C.1.8. In addition, cross drainage is required for rivers and other watercourses that the road must cross.

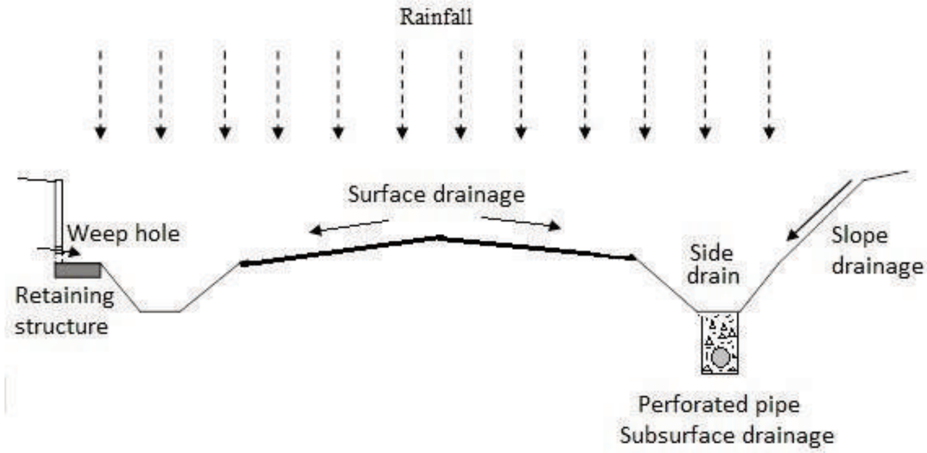


Figure C.1.8 Typical cross section of a road showing drainage types

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 the 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 stored 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 minimise the quantity of water that remains within a road pavement and underlying subgrade that may be moisture-sensitive. This is achieved by maximising the ability of the road to lose water to an external drainage system. The design of internal drainage includes minimising 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 (LVSRs). 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 C.2.

External drainage

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

The design of external drainage entails the processes of:

a) Determining the quantity of rainfall that falls upon the road itself and the drainage works required to channel that water away from the road;
b) Determining the quantity of water that is generated by the local catchment which flows in the streams, rivers and natural drains that interact with the road. This is water that falls as rainfall over the local catchment near or away from the road;
c) Identifying any risks related to other overland flows generated by the local catchment; and
d) Designing the individual engineering features of the drainage system to accommodate the anticipated flows of water.

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

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 due to pavement failure
	Penetration through intact layers
From the subgrade	Artesian head in the subgrade
	Pumping action at formation level
	Capillary action in the 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 vapour phase onto underside of an impermeable surface
Through the pavement surface	Through cracks under pumping action through the intact surfacing
Into the subgrade	Soakaway action
	Subgrade suction
To the road margins	Into lateral/median drains under gravitational flow in the 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. It is one of the key material parameters affecting internal 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 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 C2.2 Typical material permeabilities

Material	Permeability	Description
Gap-graded crushed rock	> 30 mm/s	Free draining
Gravel	> 10 mm/s	
Coarse sand	> 1 mm/s	

Material	Permeability	Description
Medium sand	1 mm/s	Permeable
Fine sand	10 µm/s	
Sandy loam	1 µm/s	Practically impermeable
Silt	100 nm/s	
Clay	10 nm/s	Impermeable
Bituminous surfacing[1]	1 nm/s	

NOTE:

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

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.

This classification of road drainage is shown in Table C.2.3. The crown height, h, correlates well with the actual service life of pavements constructed from natural gravels.

Table C.2.3 Classification of road drainage

Classification	Crown Height
Very good	h > 0.90 m
Good	0.75 m < h < 0.90 m
Moderate	0.60 m < h < 0.75 m
Poor	0.40 m < h < 0.60 m
Very poor	h < 0.40 m

NOTE: Classification can move up one class if:

- The longitudinal gradient >1%; and/or
- the drain is lined, and the lining connects to the surfacing; or
- 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.

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 critically important 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 give rise 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.

Good internal drainage is achieved by ensuring that the permeability of the pavement and subgrade layers are at least equal or are increasing with depth. For example, in a three-layered system the permeability of the base must be less than or equal to the permeability of the sub-base.

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 road base materials are used, attention must be given to the drainage of this layer. Ideally, the road base and sub-base should extend right across the shoulders to the drainage ditches. In addition, proper crossfall is needed to assist the shedding of water into the side drains. 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 there is a permeability inversion. 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 a LVR.

If it is too expensive to extend the road base 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 road base itself and should be given a fall of 1 in 10 to the side ditch. An alternative and preferable option would be 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.

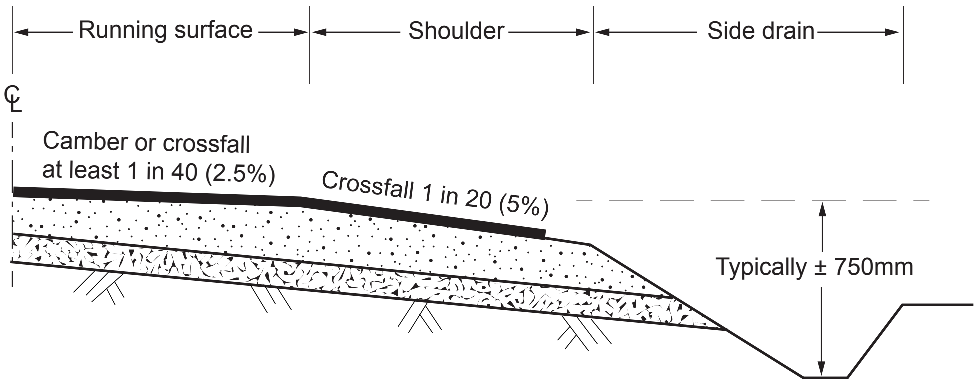


Figure C.2.2 Recommended drainage arrangements

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:

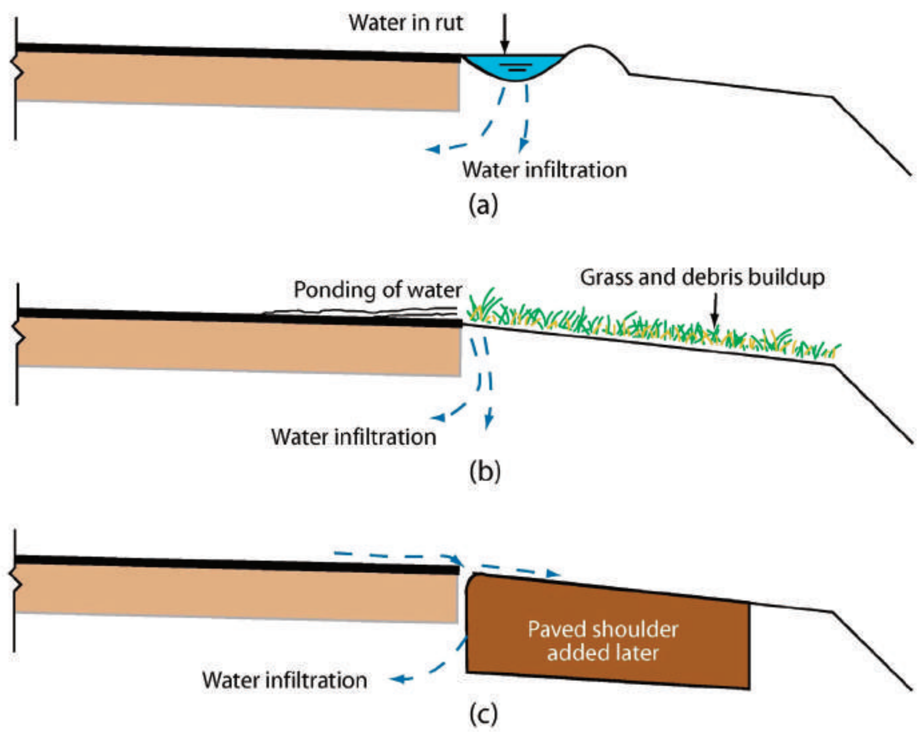
a) rutting adjacent to the sealed surface;

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 should be sealed. The advantage of sealed shoulders is that they:

- provide better support and moisture protection for the pavement layers and also 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 relatively soft.



SOURCE: Adapted from Birgisson and Ruth, 2003

Figure C.2.3 Drainage deficiencies associated with pavement shoulder construction

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. As illustrated in Figure C.2.4, this type of construction has the undesirable feature of trapping water at the pavement/shoulder interface and inhibiting flow into drainage.

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 sustain the traffic loading.

Figure C.2.4 Infiltration of water through a permeable surfacing

2.6 Adopting an appropriate pavement cross-section

The two significant moisture zones in the pavement are the equilibrium zone and the zone of seasonal moisture variation. These are shown in in Figure C.1.1, with the right side of the diagram having a sealed shoulder and the left side an unsealed shoulder.

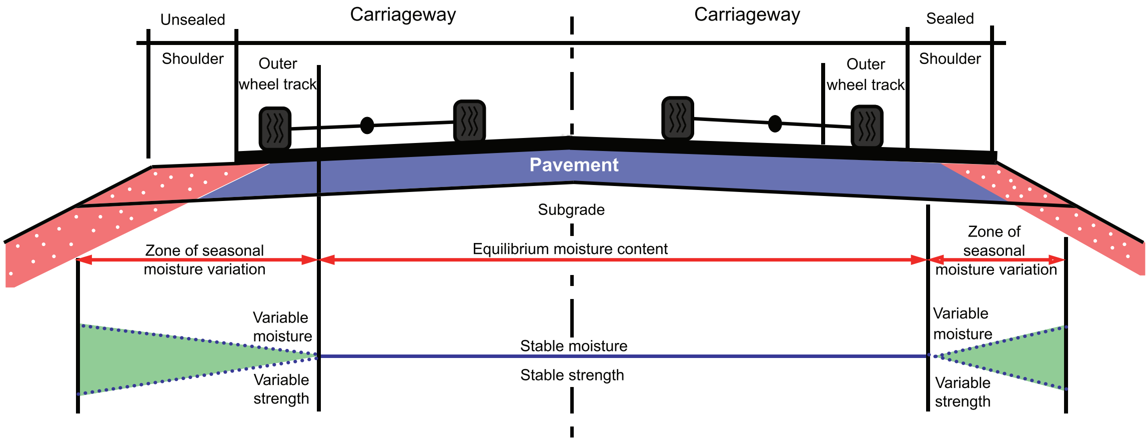


Figure C.2.5 Moisture zones in a typical LVR

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 contents in the equilibrium zone normally reaches an equilibrium value after about two years from construction and remain constant thereafter.
- In the zone of seasonal variation, the pavement moisture does not reach an equilibrium and fluctuates with rainfall. Generally, this zone is wetter than the equilibrium zone in the rainy season and drier in the dry season.

Whether or not the shoulders are paved, the edge of the pavement is therefore of extreme importance to ultimate pavement performance. When moisture conditions are relatively severe, this is the most failure-prone region of a pavement. In order to ensure that the moisture and strength conditions under the outer wheel track will remain fairly stable and largely independent of seasonal variations, the shoulders should be sealed to a width of between about 1.0 and 1.2 m from the edge of the 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 for an extended period of time does not result in 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 minimising the risks associated with using locally occurring natural materials in the pavements of LVRs.

3. HYDROLOGICAL STUDIES

3.1 Introduction

3.1.1 Purpose of hydrological and drainage studies

As an important step in the process of ensuring that all parts of the road are maintained in an excellent drainage condition, hydrological and drainage studies are carried out on project roads in order to determine:

- catchments for culverts and roadside drains;
- the runoff coefficient of the ground cover of the particular catchment;
- rainfall intensities for the project areas;
- peak/maximum flows; and
- the High Flood Level (HFL).

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. The different methodologies that can be used to determine the peak flows are listed here and then explained in the subsequent section.

The choice of appropriate methodology to use depends primarily on the availability of data, and the extent of the catchment area. The most commonly used methods and tools 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 past practice.
2. The Rational Method
3. The Modified Rational Method
4. The WinTR-20 Model
5. The DFID Hydrological Model.

3.3 Field observations method

This method is appropriate in situations where data on rainfall and topographic maps are not readily available. 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 as follows:

(i) 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 to cause 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 local inhabitants about the performance of the watercourse in order to ascertain the degree of confidence that can reasonably be placed in 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.

$A = W \times D$ = 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 it travels down the stream.

(ii) Direct observation of erosion and debris

Flood waters usually carry debris. Some of this can be left 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 normally 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.

(iii) History and local knowledge

Historical data on, and local information about, high flood levels at drainage crossing point can be very useful in deriving or validating the estimation of maximum flows. Interviews with local inhabitants within the vicinity of the crossing point is necessary to establish the history of flood levels as an aid to determining peak discharges at the different locations.

(iv) 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 has not experienced any overtopping over the years following heavy upstream downpours, this 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 the new structures.

3.4 The Rational Method

3.4.1 The Rational Formula

For small catchments, the Rational Formula is the most commonly used approach for estimating peak flows. The maximum discharge is given by the expression

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

Where:

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

C = Runoff Coefficient

I = Rainfall Intensity, (mm/hr)

A = Catchment Area, (km^2)

This formula is based on a series of theoretical simplifications needed to justify this implied direct linear relationship between rainfall and runoff. These include assumptions that:

- The peak flow at the inlet of the cross drainage structure is reached at the moment that the entire watershed is contributing to that flow. This is known as the Time of Concentration (t_c).
- The Rainfall Intensity associated with a storm varies with the applicable t_c, but is otherwise uniform over time.
- The Runoff Coefficient as estimated for a specific catchment is independent of other variables such as the moisture content of the soil, and the rainfall intensity and duration.

None of these assumptions holds strictly true, and the application of the method to large catchments can result in significant over-estimation of peak flows. Nevertheless, they are sufficiently valid to allow the Rational Method to be used in the Ghana context for catchments of up to 10 km^2. Beyond this, the inclusion of an Area Reduction Factor (ARF) results in the Modified Rational Method, which can be used for larger catchments.

Figure C.3.1 presents 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 that equip the Design Engineer with the relevant data. Two such processes are described below, namely: (i) collection of existing documents; and (ii) field data collection.

(i) Collection of existing documents

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

- **Topographic maps**. These are used to help identify any stream/river crossings and possible culvert locations and in the estimation of catchment areas. A planimeter is used to help calculate the approximate area of a catchment, and a map measurer to determine the length of streams. Topographic maps can be obtained from the Survey Department or the Centre for Scientific and Geographical Information Systems (CERGIS) at the Department of Geography, University of Ghana, Legon. Where topographic maps are not readily available, aerial photographs could be used to obtain the needed data. In some cases, digital ground model data may be available in a format that can readily be used by standard design software.

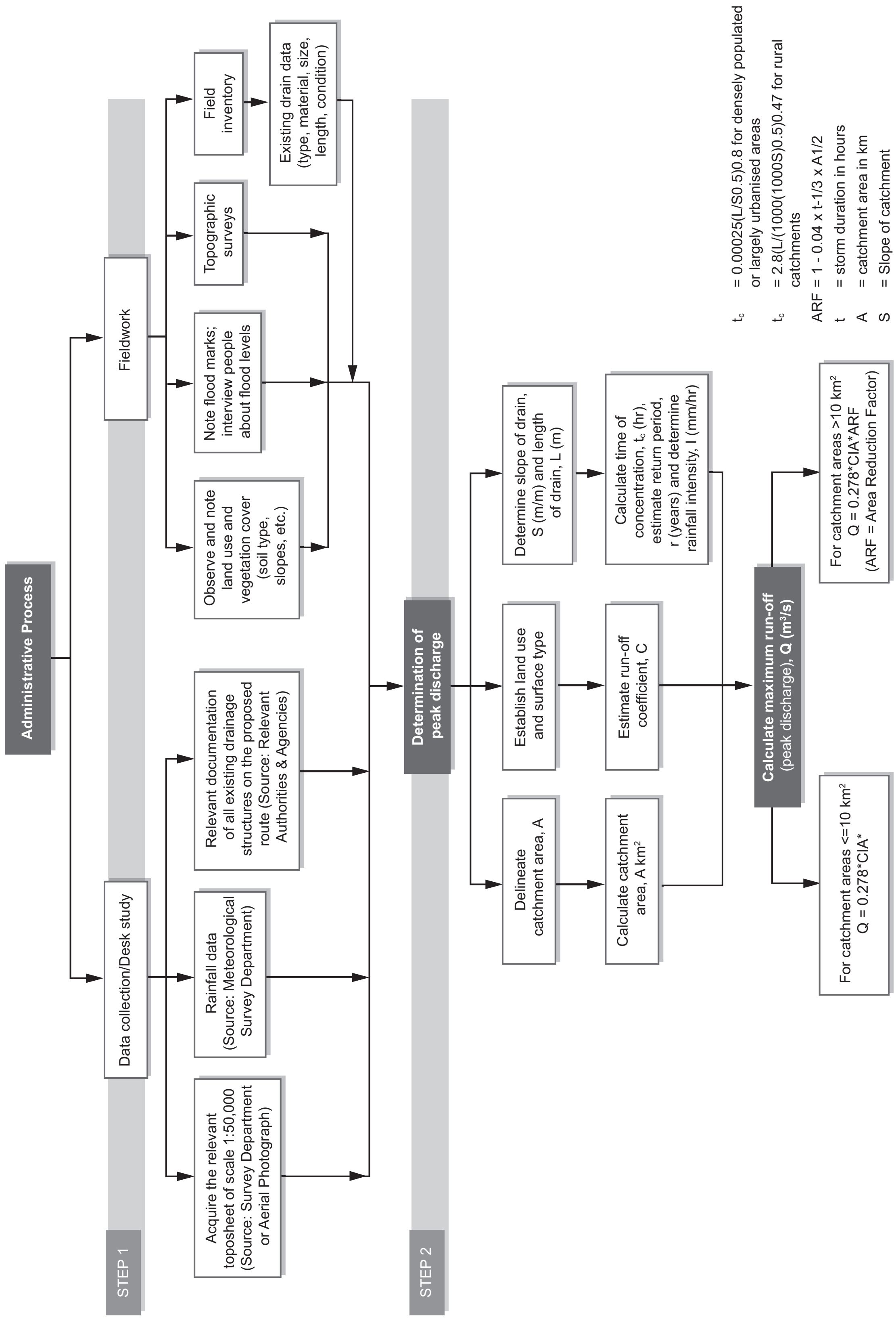


Figure C.3.1 Flow chart for estimating maximum flow/discharge

- **Geological maps**. These should be obtained from the Geological Service Department in Accra. By helping to identify soil types at specific locations they facilitate the determination of appropriate runoff coefficients. Runoff coefficients may be obtained from existing literature and adopted for similar soil types in the country.
- **Rainfall data**. These should be obtained from the Meteorological Services Agency, Accra to help determine the return periods and rainfall intensity. The Intensity-Duration-Frequency (IDF) curves developed for Ghana by J. B. Dankwa in 1974 have been reviewed in light of subsequent changes in the climatic conditions. The revised IDF curves that reflect current and projected rainfall patterns are now available at the Ministry of Roads and Highways (MRH) and the Meteorological Services Agency (MSA).
- 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.

(ii) 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. This gives the Design Engineer first-hand information on the need to either maintain or replace existing structures. An inventory form should be used to facilitate collection of field data. The information collected should include the 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 determine reported water levels during periods when flooding were experienced due to heavy rainfalls. The Engineer should also look out for water marks on structures sited within the projected area, as well as debris lines in nearby vegetation.

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 are required to establish the location and elevations of existing and proposed new drainage structures (e.g. bridges, culverts, drifts, manholes, catch pits, etc.) and other structures such as buildings, railway lines, and utility service lines. 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 drainage structure may be required.

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 which is determining factor of the runoff coefficient. Values of runoff coefficient for the 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:

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

Recommended values of 'C' for use in the Rational Formula are shown in the Table C.3.1.

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.

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 hydraulically most remote 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.

$$t_c = \frac{d_r}{V_f}$$

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

S = Slope of the catchment.

3.4.5 Rainfall intensity, I

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

Rainfall intensity, I, is defined as the rate of rainfall and is typically given in units of millimetres per hour (mm/hr). IDF curves are normally provided by national meteorological services for use in the determination of rainfall intensity in a specific area within the country. As detailed above the Dankwa IDF curves used in Ghana since 1974 have recently been updated and are available at the MRH and the MSA.

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, A

Catchment Area, A, is the area of the watershed that contributes runoff to the drainage crossing. Topographic maps covering the project areas must be obtained from the Survey Department or CERGIS, and a desktop study carried out. Where these are not available, the catchment area can be estimated from field surveys, existing contour maps, aerial photographs, satellite imagery or existing digital ground models. 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 of each of the culvert position should be determined using a digital planimeter. The existing Department of Feeder Road's (DFR'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. An example of a catchment area map is shown in Figure C.3.2.

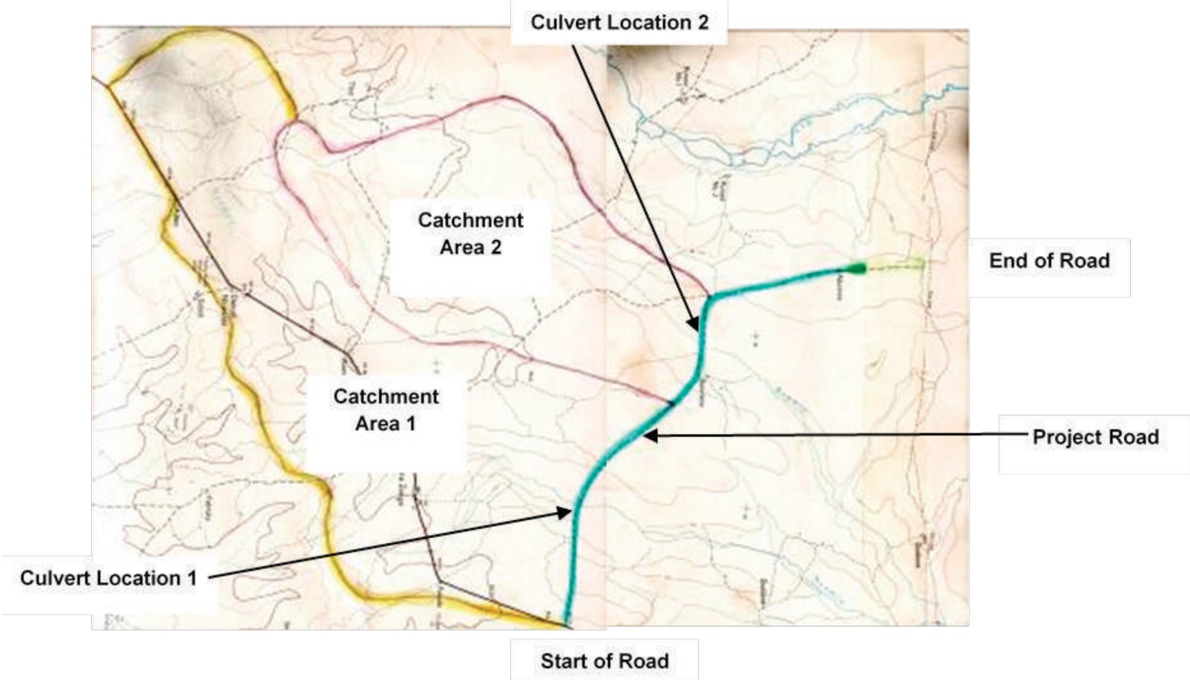


Figure C.3.2 Example of a catchment area map

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. Where sufficient localized data is not available, the Modified Rational Method is considered acceptable for larger catchment areas i.e. greater than 10.0 km^2.

A simple modification is made to take 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/hr)

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.

The WinTR20 Hydrological Model Basic Tutorials can be found on the internet.

3.7 DFID hydrological model for Ghana

DFID supported detailed studies in 2006 on Feeder Roads in Ghana that included the development of a guideline for design of Feeder Roads in Ghana.

The drainage model is a spreadsheet developed for estimation of design flow for a 10-year return period. Input data are catchment characteristics obtained from 1:50,000 maps and field inspection, just as with the rational method. Rainfall data are obtained from the IDF curves available from the MSA. The method of flow estimation is quite different from that used in the Rational Method. However the same parameters are used, namely the runoff coefficient (C), catchment area (A) and rainfall intensities (I), albeit with some adjustments. The estimation of peak flow using the DFID approach is given by the expression:

$$Q_p = F_p * F_s * Q_m$$

Where,

Q_p = Peak Flow

F_p = Peak Flow Factor

F_s = Shape Flow Factor

Q_m = Mean Flow =

$$\frac{(C_A * P * A)}{360} * T_B$$

C_A = Overall Runoff Coefficient

P = Areal Rainfall Depth

A = Catchment Area

T_B = Time Base =

$$\frac{(C * \sqrt{A}\,)}{S^2} + T_S$$

C = Rainfall Zone

S = Slope Class

T_S = Land Use

This is the tool most commonly used by DFR for the estimation of peak flows.

3.8 Design for climate resilience

Climate change will affect roads and highways in many different ways. Globally, the accepted characteristics include higher temperatures and more extreme weather events. These are often experienced as higher rainfall, more intense peaks in precipitation, and more sustained periods of both rainfall and drought. A number of online tools are available that facilitate the process of understanding past trends and anticipating future risks.

One such tool is the Climate Change Knowledge Portal at http://sdwebx.worldbank.org. In the case of Ghana, this suggests that, over the past 100 years, annual precipitation levels in Ghana have reduced by about 5%, with (approximately) 10% increases of monthly rainfall in July and August being more than offset by reductions during other months.

Looking ahead, the most compelling feature of the climate projection for Ghana is the continued steady increase in maximum day temperatures. This gives rise to an increased risk of:

- more intense local storms as raising temperatures increase the atmosphere's potential carrying capacity of moisture; and
- drought, as rates of evaporation and transpiration increase, without an associated overall increase in precipitation rates.

In light of this projection, which is also consistent with recent anecdotal experience in Ghana, one of the simplest and important actions that can be taken is to design individual 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 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 on 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).

Practical means of increasing the safety factor include:

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

In areas prone to water shortages, various mitigating measures could potentially be taken as a Complementary Intervention. These include converting used borrow pits into ponds or reservoirs in areas where local development plans have identified a need for increased water storage.

The most significant risks are likely, in most of the country, to relate to increased intensity of local storms. Whatever precautions are taken, the risk will always remain of a storm intensity that exceed what the structure has been designed for. In the case of submersible structures such as vented fords and drifts, this should not be a problem. In the case of larger and higher structures such as minor bridges consideration should be given to designing the approach embankments in a manner that ensures that they will overtop and potentially fail sacrificially before the structural integrity of the bridge itself is compromised.

Erosion is a serious problem in many areas and climate change is likely to make matters worse. There are also likely to be more severe geotechnical problems (such as those related to slope stability) caused by climate change. These are dealt with in Chapter C.7 on roadside slope stabilisation.

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 (CSIR, 2018).

3.9 Worked examples

Worked Example 1

Your Organisation has recently won a Contract to carry out Spot Improvement works on the Mpatasee – Daku gravel feeder road in Ashanti Region. 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 maps.
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 a 2 km^2 catchment area.

Solution

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

Step 2: Identify on the topographic map the road link Mpatasee – Daku.

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 existing 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. 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:

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

Where:

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

C = Runoff Coefficient

I = Rainfall Intensity, (mm/hr)

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 the rainfall Return Period. This determined from the revised IDF tables or graphs for the particular area. From the question the time of concentration is given as 42 minutes (0.7 hrs) and the return period is given as 10 years. Thus, from the IDF Table presented as Table C.3.2, the rainfall intensity I = 81.31 mm/hr.
4. The Maximum Discharge or Peak Flow, Q, is determined as follows:

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

$Q = 0.278 \text{ x } 0.9 \text{ x } 81.32 \text{ x } 2$

$Q = 41 \text{ m}^3/\text{s}$

Table C.3.2 Intensity-Duration-Frequency table (Kumasi)

KUMASI	Return period (yrs)							
Duration (hrs)	5	10	15	20	25	50	75	100
0.2	115.07	129.50	137.41	143.00	147.47	159.76	166.46	172.04
0.4	92.88	104.60	110.92	115.42	119.03	128.95	134.36	138.87
0.7	72.20	81.31	86.22	89.72	92.53	100.24	104.44	107.95
1	59.15	66.62	70.64	73.51	75.81	82.12	85.57	88.44
2	37.11	41.80	44.32	46.12	47.56	51.53	53.69	55.49
3	27.16	30.58	32.43	33.75	34.80	37.70	39.28	40.60
6	15.18	17.09	18.12	18.86	19.45	21.07	21.95	22.69
12	8.16	9.19	9.74	10.14	10.45	11.33	11.80	12.20
24	4.29	4.83	5.13	5.33	5.50	5.96	6.21	6.42

Worked Example 2

Your Organisation has been contracted to undertake design studies to upgrade the existing gravel feeder road and farm track from Ashale Botwe to the intersection with trunk road between Frafraha and Oshiyie in the Greater Accra Region of Ghana 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 map.
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 a 11.5 km^2 catchment area.

Solution

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

Step 2: Identify on the topographic map the road link Ashale Botwe to the intersection with trunk road between Frafraha and Oshiyie

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 existing Department of Feeder Road's (DFR'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. 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:

$$Q = 0.278 \times C \times I \times A \ \text{x} \ ARF$$

Where:

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

C = Runoff Coefficient

I = Rainfall Intensity, (mm/hr)

A = Catchment Area, (km^2)

A = Area Reduction Factor:

t = storm duration in hours

Table C.3.3 Intensity-Duration-Frequency table (Accra)

ACCRA	Return period (yrs)							
Duration (hrs)	5	10	15	20	25	50	75	100
0.2	128.47	159.76	180.98	196.62	210.03	255.83	287.11	310.57
0.4	103.70	128.95	146.08	158.71	169.53	206.50	231.75	250.69
0.7	80.61	100.24	113.56	123.37	131.78	160.52	180.15	194.87
1	66.04	82.12	93.03	101.08	107.97	131.51	147.59	159.65
2	41.44	51.53	58.38	63.42	67.74	82.52	92.61	100.17
3	30.32	37.70	42.71	46.40	49.57	60.38	67.76	73.30
6	16.94	21.07	23.87	25.93	27.70	33.74	37.87	40.96
12	9.11	11.33	12.83	13.94	14.89	18.14	20.35	22.02
24	4.79	5.96	6.75	7.33	7.83	9.54	10.71	11.58

Determining the parameters:

1. Area (A), is as computed in step 4 but given as 11.5 km^2 in the question. This is greater than 10 km^2 so an area reduction factor (ARF) has to 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 C2.1, 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 revised IDF tables or graphs for the particular area. From the question the time of concentration is given as 60 minutes and the return period is given as 25 years. Thus, from Table C.3.3, the rainfall intensity I = 107.97mm/hr.

4. $Q = 0.278 \times C \times I \times A \ \text{x ARF}$

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

   $Q = 0.278 \text{ x } 0.9 \text{ x } 107.97 \text{ x } 11.5 \text{ x } 0.86 = 267 \ m^3/s$

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 (where there is one) into the side drain. The flow is intended be in sheet form in order to reduce the risk of erosion of the road surface and shoulders. It is therefore important to ensure control of a uniform camber both laterally and in longitudinal profile. 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. To manage this risk, it may be necessary to seal steep sections.

The recommended camber for unsealed rural feeder roads is 5% - 7%. Camber on a paved LVR should be 3%. In exceptional circumstances this may be reduced to 2.5% in the case of asphalt and 2.0% in the case of concrete. For these to be sufficient to reliably ensure drainage of the road surface, it is important that specified limits regarding both compaction and surface irregularities are consistently achieved during construction.

4.2 External drainage

4.2.1 Function of external drainage

An effective external drainage system must:

- prevent or minimise the entry of surface water into the pavement;
- prevent or minimise the adverse effects of sub-surface water;
- remove water from the vicinity of the pavement as quickly as possible; and
- where necessary, and in a planned manner, 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 as follows:

- 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, to provide a means of collecting and removing some of the material carried in the water, and thereby to limit erosion both 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 considerations 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 appropriately along stream courses, without impairing their hydraulic efficiency or capacity.

4.3 Side drains (longitudinal ditches)

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 from beneath the road and discharge it away from the road. Ditches may be U-shaped, trapezoidal, or V-shaped, as illustrated in Figure C.4.1. V-shaped drains have lower capacity and are prone to erosion, so it is advisable to limit their use to when conditions do not suit the use of other types.

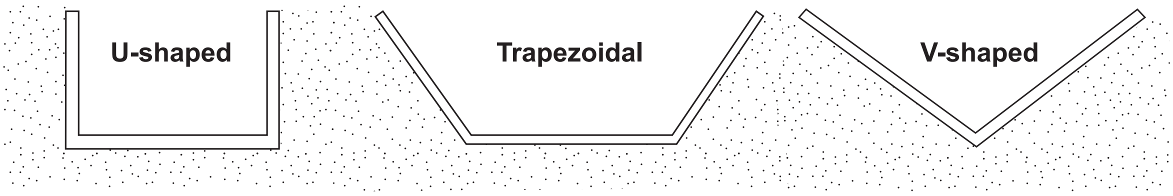


Figure C.4.1 Typical side drain cross-sections

Drains must be designed for capacity, shape, efficiency, and to avoid ponding, silting and erosion. The adoption of a trapezoidal cross-section facilitates maintenance and is acceptable from the point of view of traffic safety. It is much easier and appropriate to dig and clean a such a drain with hand tools. 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.

In very flat terrain and reasonable soils (those 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 yet siltation does not occur.

Figure C.4.2 Concrete-lined trapezoidal drain along a sealed LVR in Eastern Region

Figure C.4.3 Concrete-lined U-Drain along a gravelled LVR in Eastern Region

4.4 Mitre drains (off-shoots/turn-outs)

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

The minimum width of mitre drains should be 0.60 m and the cross-section should have at least the same capacity as side drain. When possible, they should have a steeper longitudinal slope than the side drains. The angle between the mitre drain and the side drain should never be greater than 45°. An angle of 30° is ideal. Some excavated soil should be used to block the downhill side of drain to ensure water flows into mitre drain. Alternatively, when the side drain is constructed gaps should be left in the drains immediately downhill of each turn-out location.

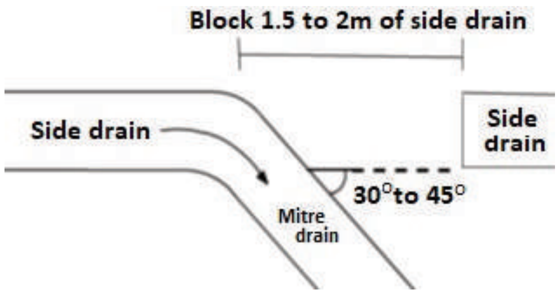


Figure C.4.4 Mitre drain along a LVR

Figure C.4.5 Mitre drain alignment

The maximum spacing for mitre drains is given in Table C.4.1. This ensures that the quantity and velocity of water being discharged at each mitre drain outlet is limited and reduces the risk of erosion damage to the drainage system or on the adjoining land.

Table C.4.1 Maximum mitre drain spacing

Side drain gradient, S (%)	Spacing (m)
1 - 2	200
4	100
6	50
8	40
10	25
12	20

SOURCE: DFR Site Supervision Pocketbook V2, Page 16

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

As illustrated in Figure C.1.1, catch water drains are usually on the uphill side of the road or cut slope. Their function is to intercept surface runoff from the hillsides 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 a catch water drain is carried out in the same way as open channel design. Further details of catch water drains are given in Chapter C.7.

4.6 Cross drainage structures

Culverts serve as conduits that pass beneath the road surface to transfer runoff water from one side to the other. They either take runoff water from the side drain (relief culverts) or where the road intercepts a natural watercourse (watercourse culverts). 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 a cross drainage structure is considered to be a bridge rather than a culvert if the span exceeds 6 m, and/or its depth exceeds 4 m.

The culvert barrel can be in various sizes and shapes: elliptical; circular; square; rectangular; U-shaped or semi-circular. The most common culvert types in Ghana are precast circular unreinforced concrete pipe culverts, and box culverts. U-culverts are well suited for use as relief culverts as they require less depth than circular culverts. However, 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. When properly constructed, U-culverts can offer better hydraulic results than the same size of pipe.

Basic guidelines for the provision of culverts include:

1. The average number of culvert crossings required on a LVR in Ghana is typically (though not necessarily) between two and three per km. However, the requirement can vary greatly, from about one per km in

dry and gently rolling terrain up to six or more per km in severe terrain with high rainfall. In flat areas with high rainfall, the frequency may also increase to allow water to cross the road alignment in manageable quantities.

2. In addition to being provided at well-defined water crossing points, culverts should normally be located at low points or sags in the road alignment.
3. Where possible, the alignment of a culvert should follow that of the watercourse, both horizontally and vertically.
4. Culvert invert levels should be approximately in line with the water flow in the stream bed. Otherwise a drop inlet and/or long outfall excavations may be required.
5. The gradient of the culvert invert should be between 2% and 5%. Shallower gradients could result in silting whereas steeper gradients may cause scouring.
6. For fully engineered roads, the minimum recommended diameter for a watercourse culvert is 900 mm, in order to facilitate cleaning. U-culverts or box culverts should be used where this would otherwise result in a hump in the vertical profile of the road. In such circumstances the top slab of the U-culverts or box culvert can be designed to carry direct traffic loads.
7. Culverts should be placed on a good foundation material to prevent settlement and damage. On soft ground, it is necessary to use concrete to ensure adequate foundations.
8. Headwalls are required at the inlet and outlet to direct the water in and out of the culvert and to 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.
9. Aprons with buried cut off walls are required at the inlet and outlet to prevent water seepage, scouring and undercutting.
10. Culverts concentrate the flow at the inlet and outlet. Attention must therefore be given to the design of erosion protection works.
11. It is particularly important to protect the watercourse from erosion downstream from the structure, where flows are faster and more turbulent than upstream.
12. Culverts are generally provided where the ground level is low and the road level is high, so they do not affect the road profile. Compared with submersible structures such as drifts, culverts are relatively safe and should allow a comfortable ride.
13. Multiple culverts can be provided at a single location to enable larger stream flows to be accommodated using standard unit designs, while taking account of the associated increased risk of blockage by debris.
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 therefore more suited to the use of drifts.
18. The choice of a culvert type and material depends on hydraulic performance for the specific site, the nature and volume of the traffic, the length of the road, the proven capacity to effect routine maintenance, and the economics and availability of different culvert types or materials.

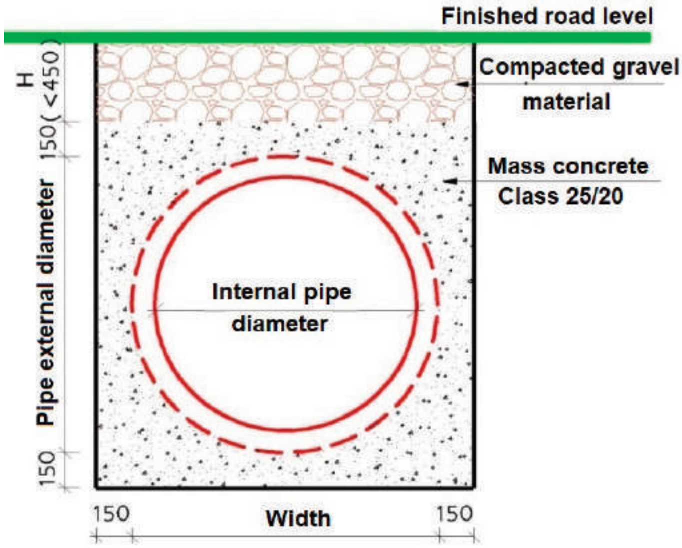


Figure C.4.6 Typical section of pipe culvert with concrete surround

The structural design of culverts requires an assessment of the loading that will be applied on the culvert in terms of earth and traffic loading. The height (H) of fill may determine or be a determinant of the structural details of culvert. Minimum cover to the pipe culvert should be 600 mm where the stresses from the traffic loads are dissipated to allowable limits. When the minimum cover is not achieved, then a 150 mm concrete surround should be provided to take the stresses from 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 and Figure C.4.7.

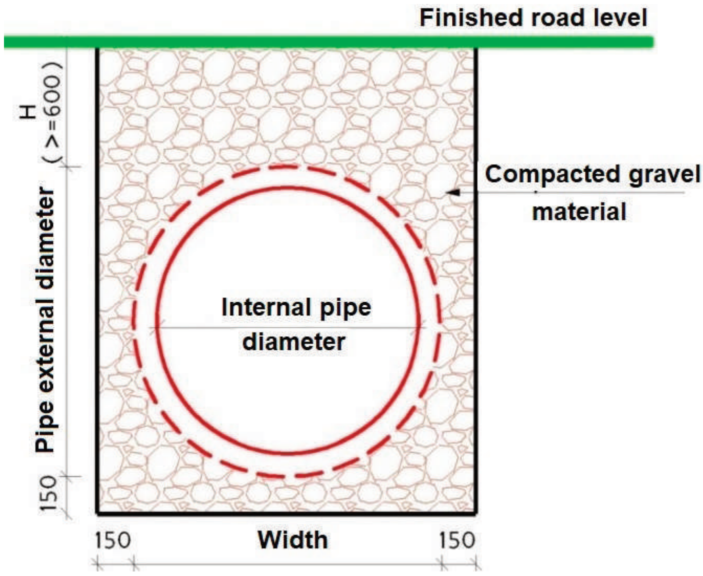


Figure C.4.7 Typical section of pipe culvert with gravel bedding

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

For relief culverts or access culverts, a minimum diameter of 600 mm pipe culvert may be used. However, a 700 mm (high) x 900 mm (wide) 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 Double U-Culvert

Figure C.4.10 Access culvert at a junction

Relief culverts are required where it is not possible to place mitre drains at the permitted maximum spacing to remove water from the side drain (see Table C.4.1).

The maximum spacing of relief culverts (when there no mitre drains are possible to discharge flows in side drains) is set out in Table C.4.2. If it is not possible to meet this requirement, erosion control measures such as side drain lining are required in addition to possible widening of the side drain.

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

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 hydrological analysis and hydraulic designs to culverts and bridges. However, drifts and vented fords (pipe drifts, causeways) permit water to flow over the surface. A drift is sometimes referred to as a ford or a causeway, particularly when vented.

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 fords are shown in Figure C.4.11 and Figure C.4.12.

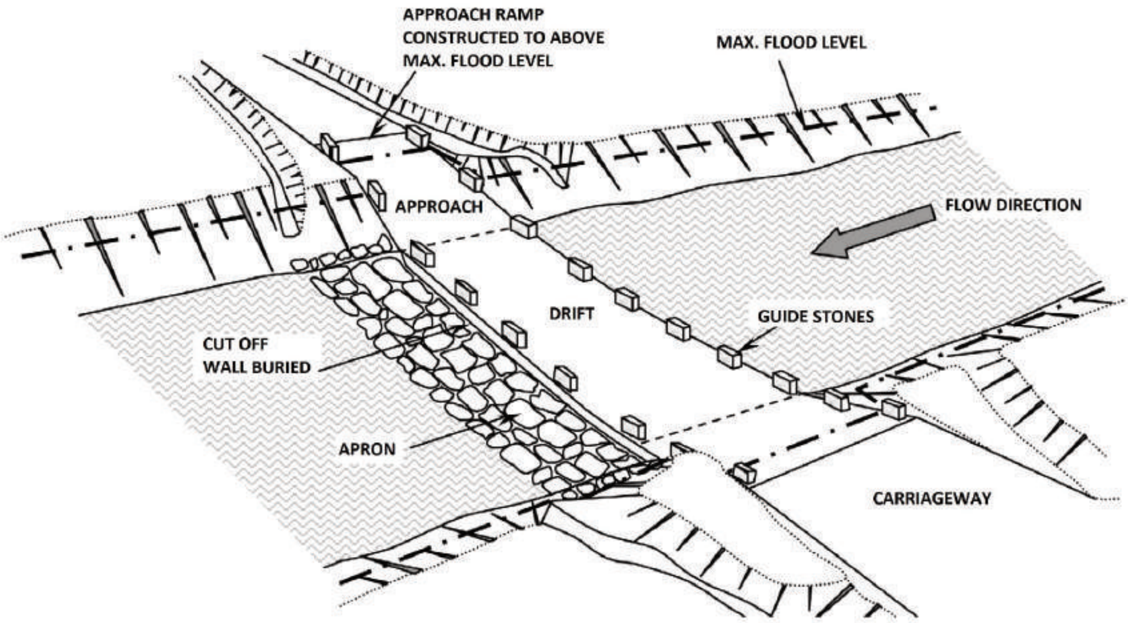


Figure C.4.11 Layout of a drift

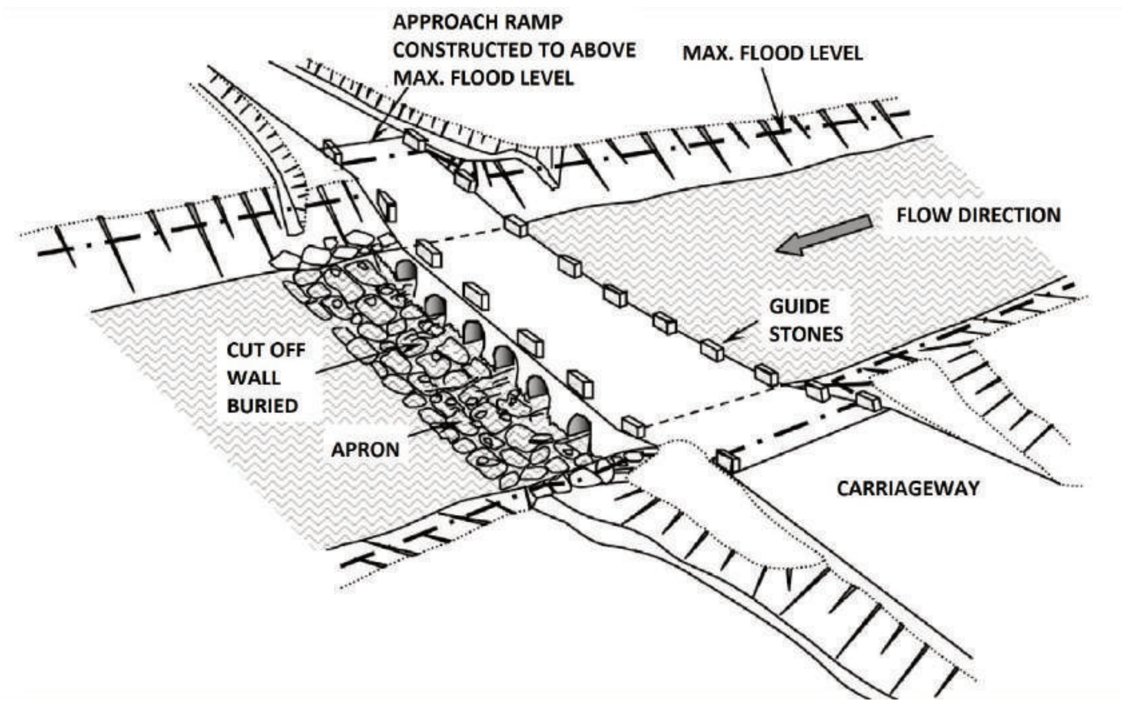


Figure C.4.12 Layout of a vented ford

Figure C.4.13 Large vented causeway

Figure C.4.14 Simple hand packed stone drift

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, so in times of flow vehicles are required to drive through the water, or in the case of very high flows, to wait until it is safe to cross.
- 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 the majority 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 to use in flat areas where culverts cannot be buried because of a 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 in order to prevent erosion of the road material.
- If necessary, guide stones or posts 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 undercutting 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%. The designed gradient will take account of the anticipated mix of traffic but should be at least 5%.
- 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.
- Drifts force vehicles to slow down resulting in longer journey times.
- Drifts are likely to cause less erosion from their discharge than culverts.

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 allow the normal dry weather flow of the river to pass through pipes below the road. Occasional larger floods will also flow over the road, which may make the road impassable for short periods of time.

- Vented causeways are the same concept as vented fords but are longer with more openings. They are used to cross wider watercourse beds.
- 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 sufficient pipes to accommodate standard flows. The location of pipes will depend 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.
- As with drifts, the approach ramps should have a gradient of between 5% and 10%, depending on the mix of vehicles using the road.
- Watercourse bank protection is required to prevent erosion around the structure.
- The upstream and downstream faces of a vented ford 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 that flows out of the culvert pipes or over the structure.
- The watercourse downstream from the structure must be protected from erosion. In flood conditions there could be considerable turbulence immediately downstream of the structure.
- The road surface longitudinal alignment of the vented ford should be a slight sag curve to ensure that, at the start and end of overtopping, water flows across the centre of the structure.
- Construction materials for vented fords can be, riprap, gabions, and reinforced concrete.
- There should be guide stones or posts 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 or posts should be painted with reflective paints to make it 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 other debris causing damage to or blockage of a vented ford, then 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.

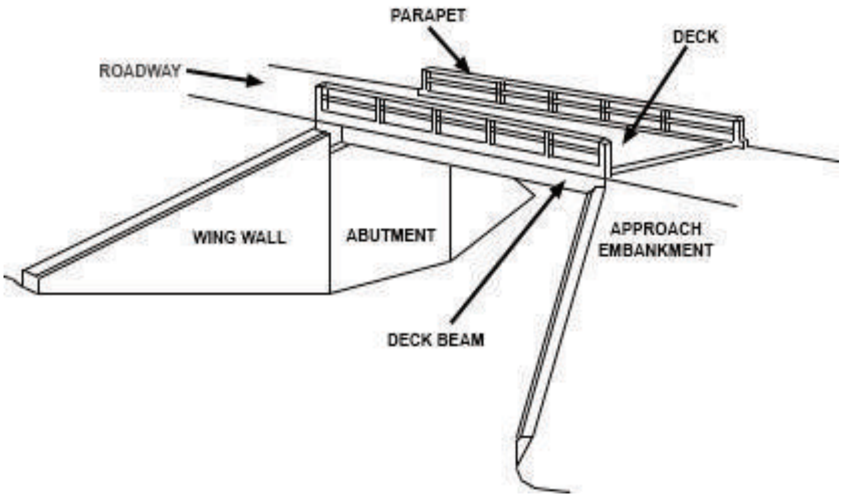


Figure C.4.15 Typical simply supported bridge deck

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 kerbs 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 stabilised watercourse, sediment trap, or stabilised area.

Chutes may be used on slopes 2H:1V or flatter to carry water down the face of erodible slopes, usually from runoff collection devices 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, drop to reduce open the energy of the descending runoff. This reduces the risk 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. It is usually 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 is usually 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 too few culverts can lead to rapid flow along the side of the embankment and consequent scouring. One method of estimating this is through asking the local people how long the water usually takes to dissipate from peak flood condition after the rain.

Figure C.4.16 Poor drainage in flat terrain

5. HYDRAULIC ANALYSIS

5.1 Introduction

Hydraulic analysis is carried out to estimate the size of drainage structure that will allow the calculated peak or maximum flow for particular catchments to pass.

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 drains, the relationship for depth and width can be established. The invert of side drains shall not be less than 300 mm below the shoulder break point. The width of the drains 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, as detailed in Part A of this Manual.

In designing the drain, consideration must be given to the limiting values of velocity to prevent erosion siltation or ponds in 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:

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

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

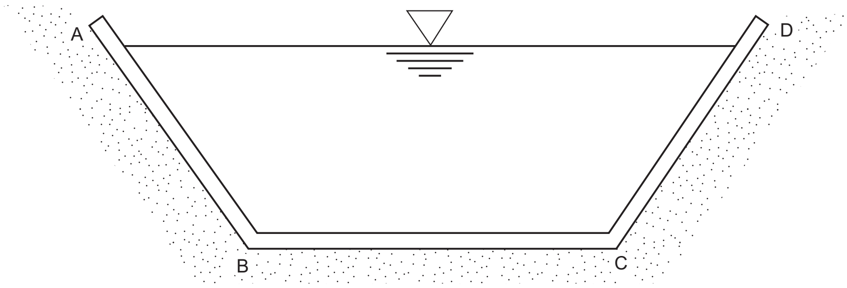


Figure C.5.1 Wetted perimeter

Wetted Perimeter, P_w = 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 'V' 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.

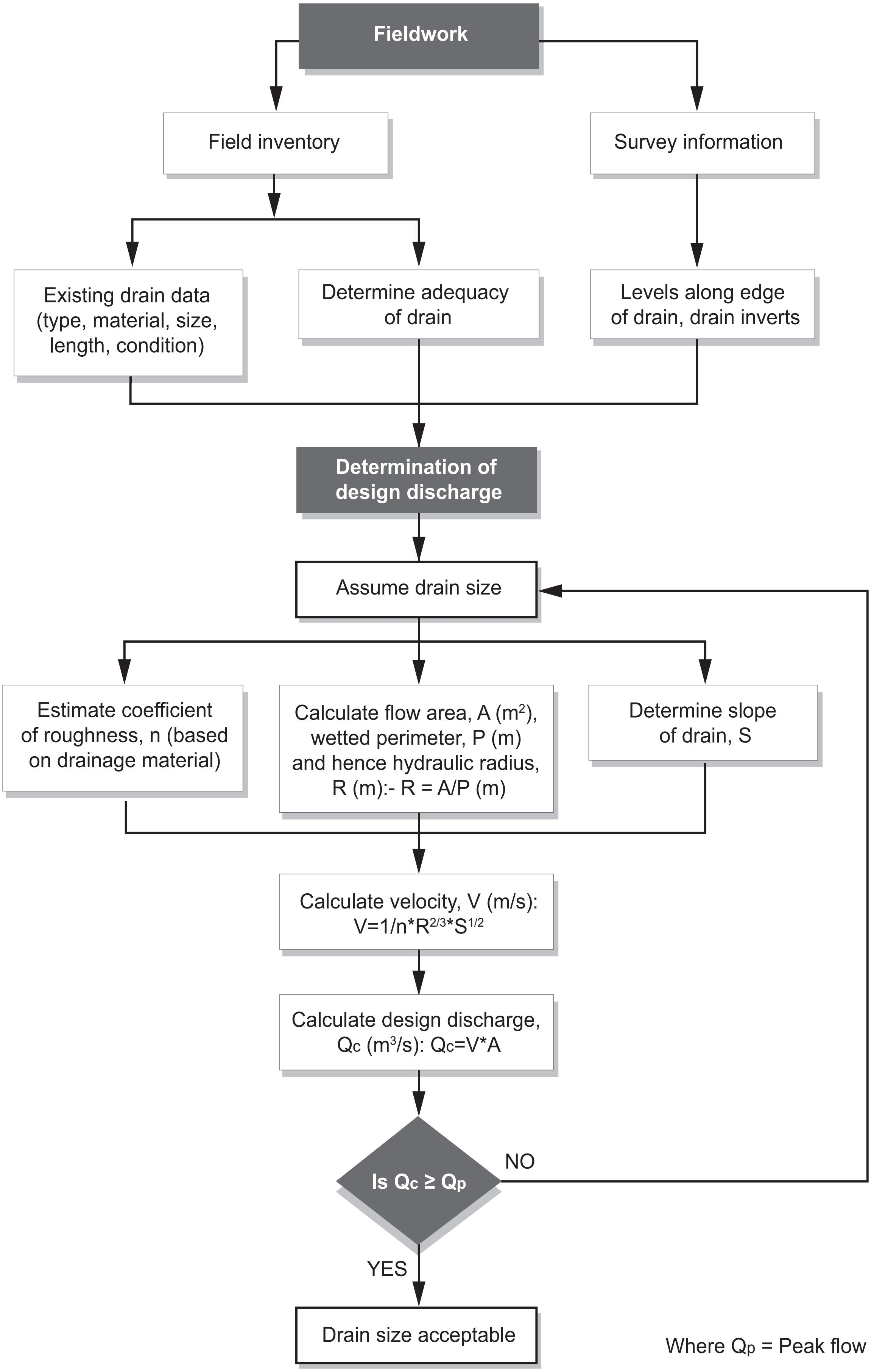


Figure C.5.2 Flow chart for estimation of flow capacities of side drains

5.2.3 Computer based approach

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

Select channel type:

Either rectangular, circular, or 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 allow the design flow to pass. 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 - Nomographs

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

- corrugated metal pipes;
- concrete pipes; and
- concrete box culverts.

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

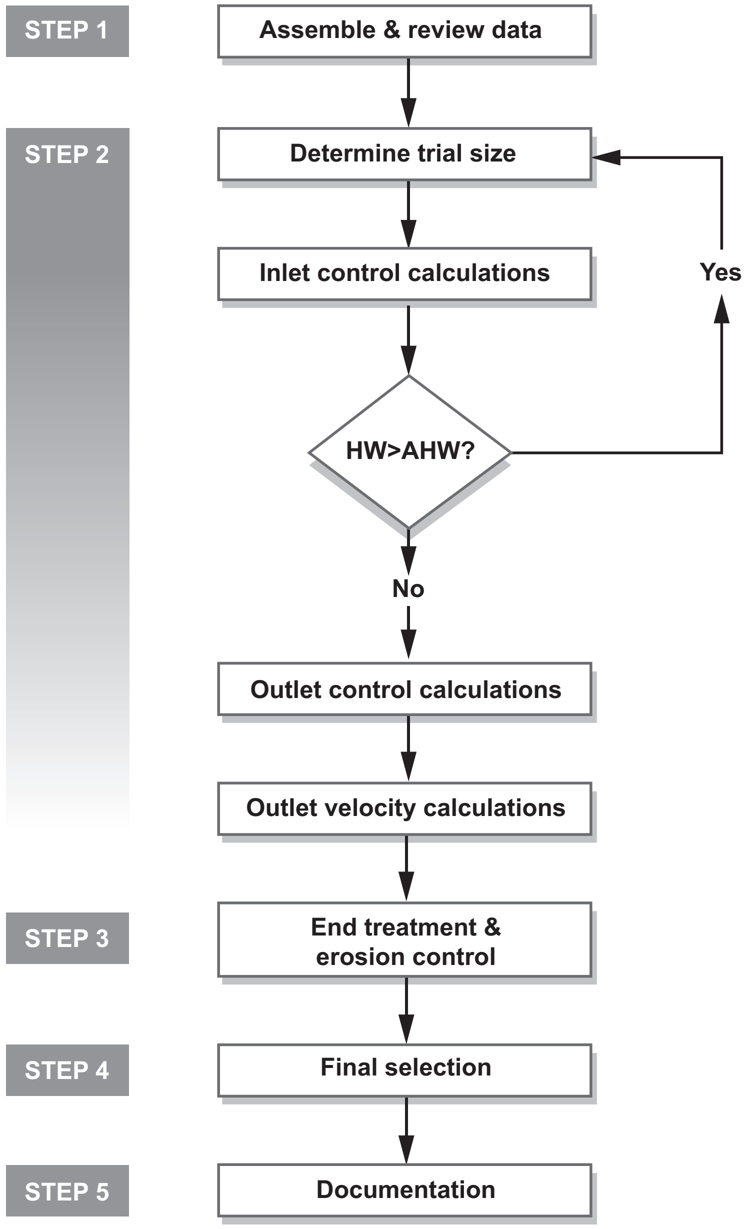


Figure C.5.3 Flow chart for estimation of flow capacities of culverts

The steps in the flow chart are explained further 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.1 for full hand calculation form).

Table C.5.2 Extract from hand calculation form (columns 1 to 13)

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 Select culvert

Design data

Col. 1 Enter culvert station/location

Col. 2 Enter design discharge, Q (m3/s)

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

Col. 4 Enter depth of culvert invert below streambed

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

Col. 11 Number of barrels, N

Col. 12 Enter Col. 2 ÷ Col. 11

Col. 13 Area (A) per barrel

Table C.5.3 Extract from hand calculation form (columns 14 to 26)

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 Co. 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.5.3.

Col. 18 Head, H obtained from Figures C.5.6 and C.5.7.

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

Col. 20 Calculate and enter $(d_c+D)/2$.

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, 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 that 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(s) on the basis of cost and other considerations.
- Check that the proposals are reasonably consistent with existing culverts of known adequacy in the project area, taking into account 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 are shown 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 as Appendix C.1.

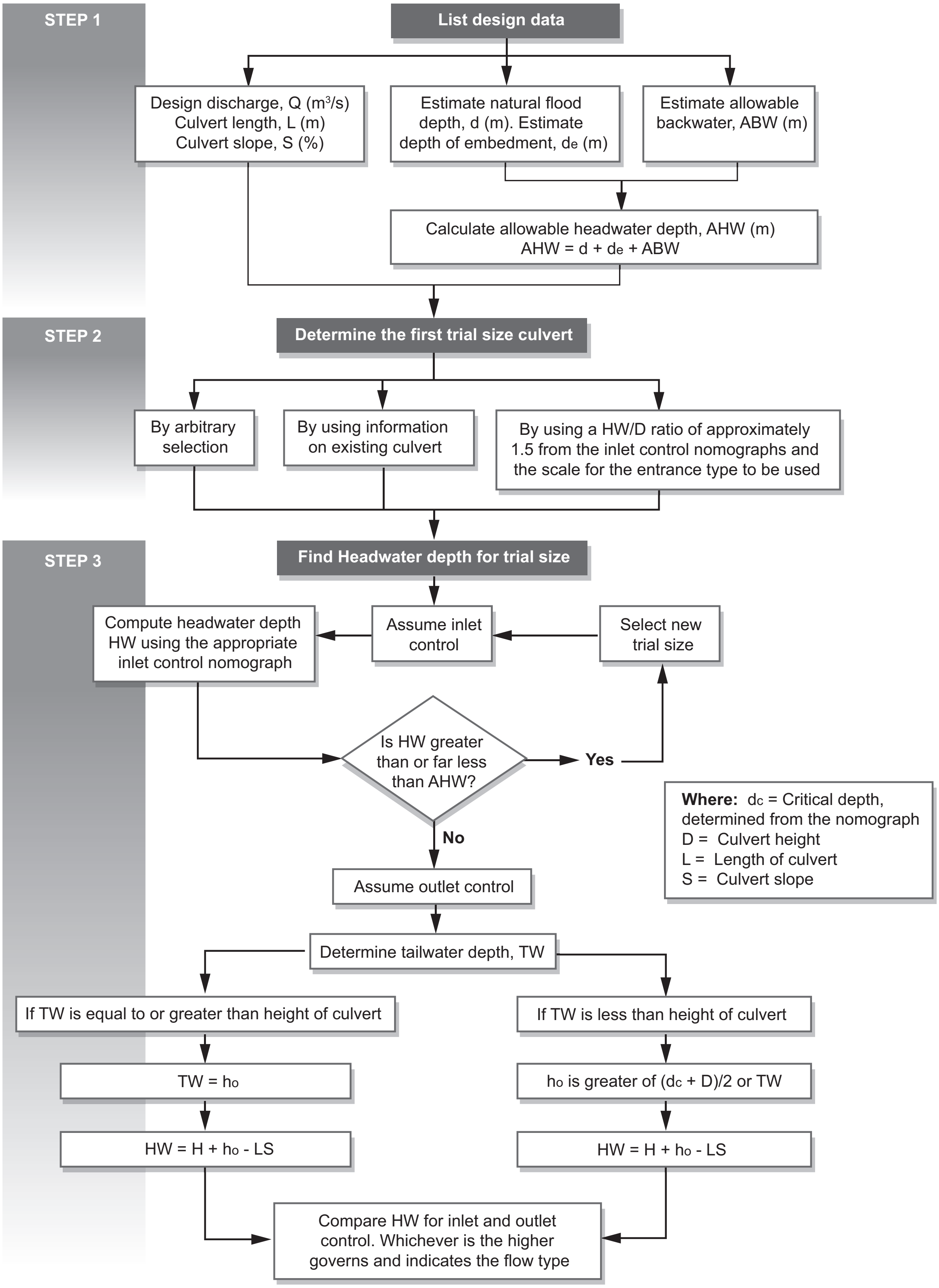
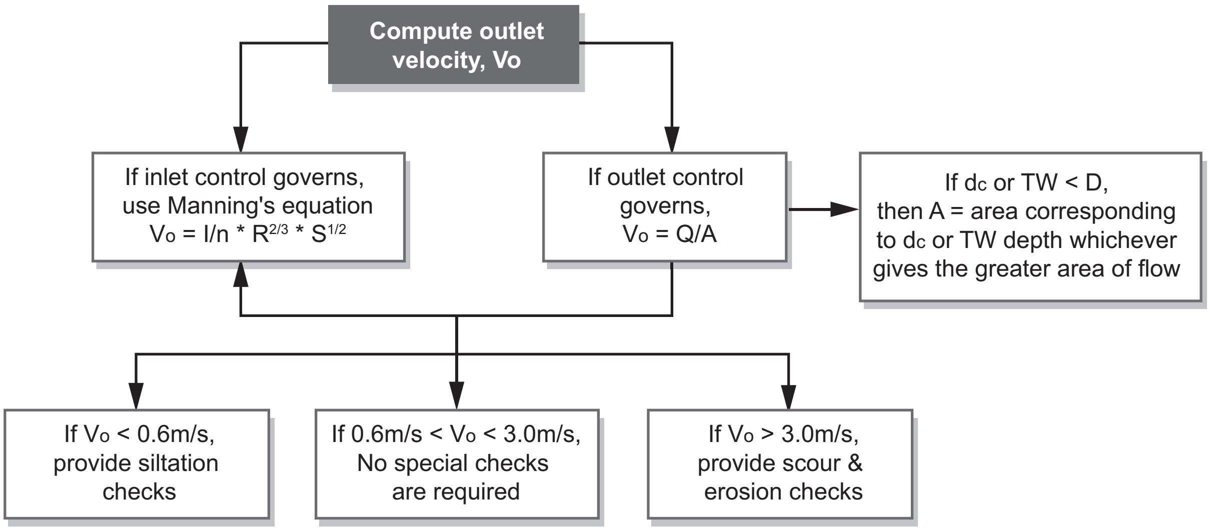


Figure C.5.4 Detailed flow chart for estimation of flow capacities of culverts

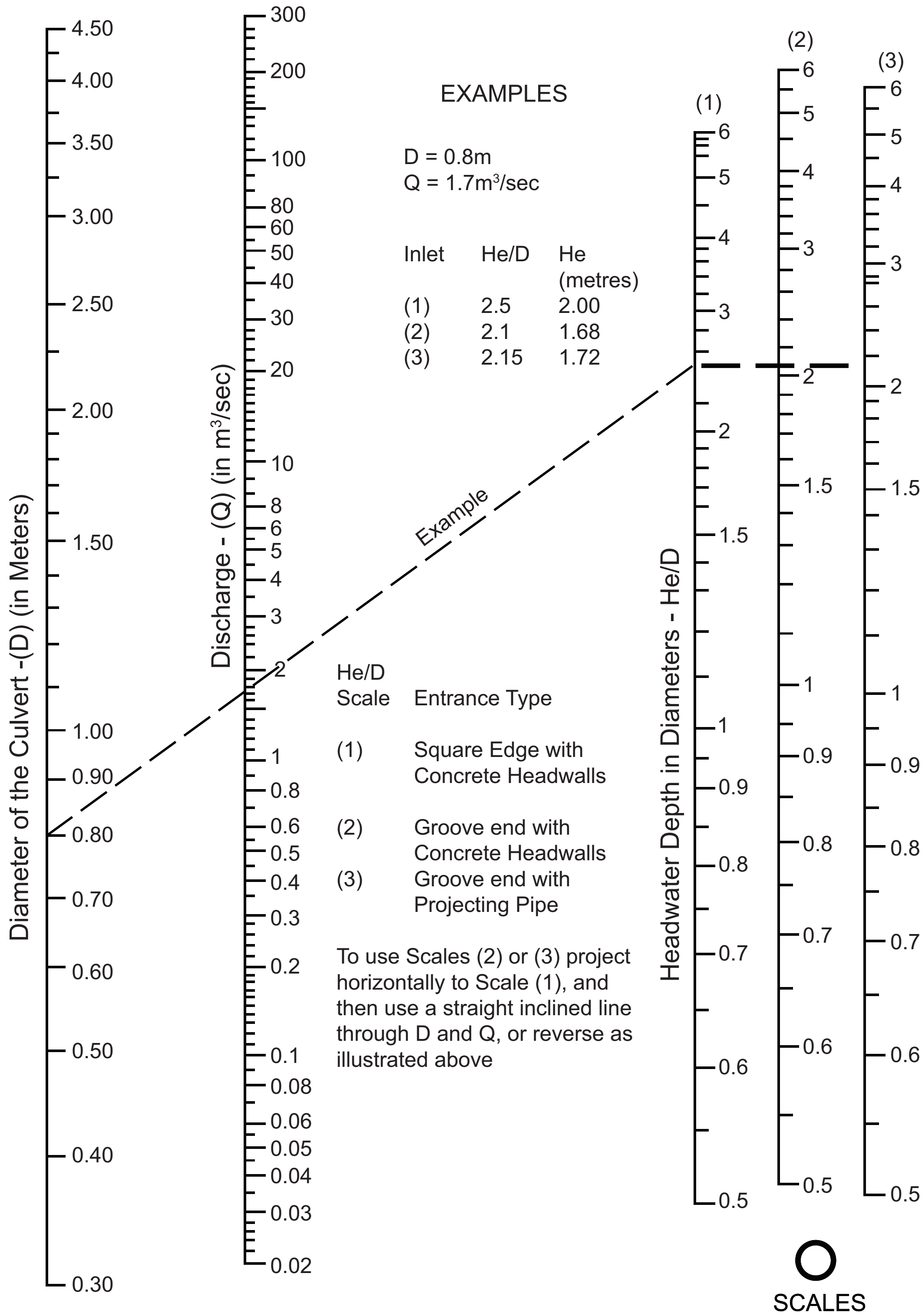


Where n = Manning's Roughness Coefficient
S= Culvert Slope
R= Hydraulic Radius

Figure C.5.5 Flow chart for estimation of flow velocity

Figure C.5.6 is used to determine headwater depth (HW) of 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.



SOURCE: Adapted from Chart 1A, FHWA, 2012

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

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

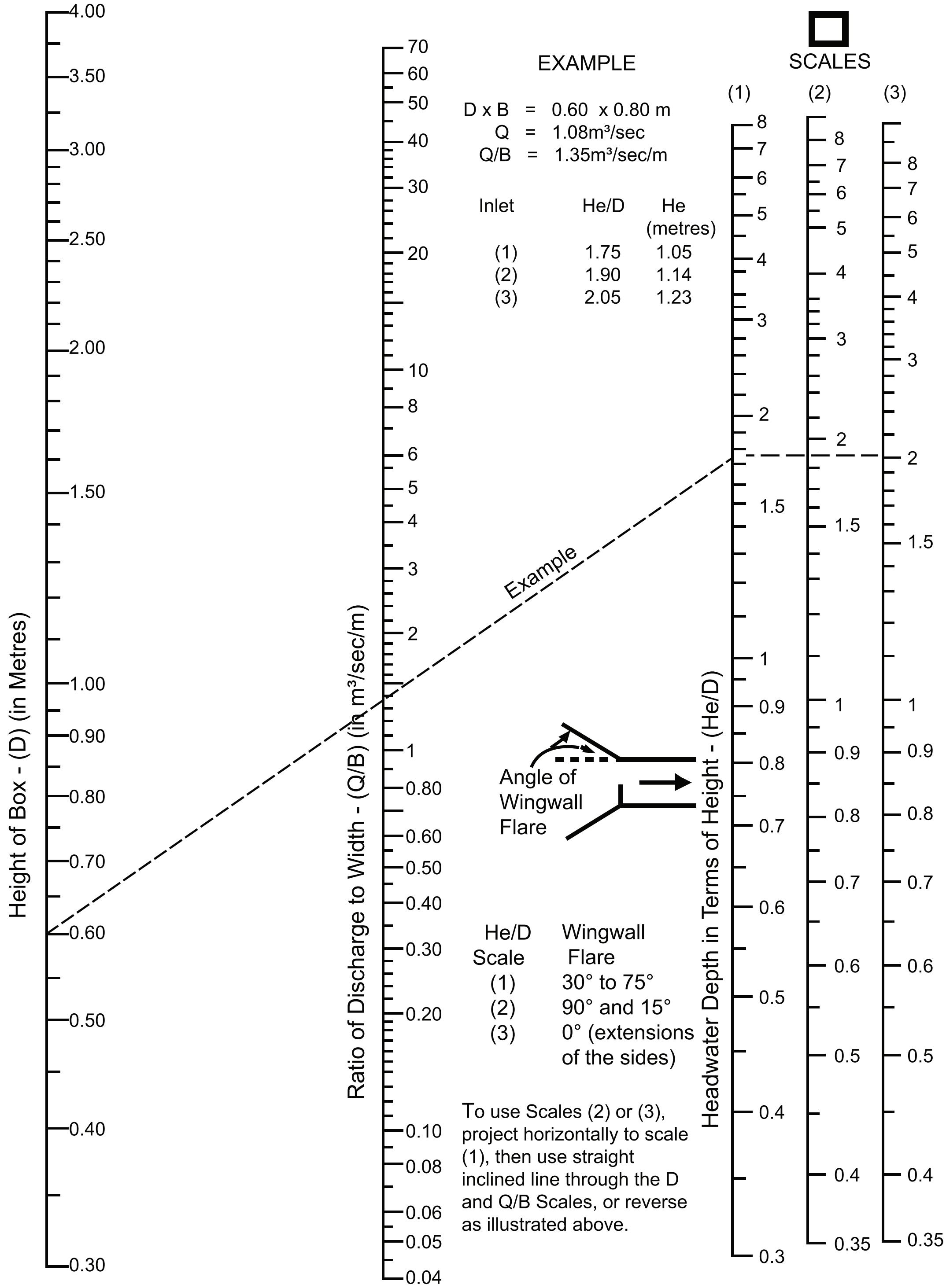
i. Determine the height of box (D) and ratio of design discharge (Q) to width (B) of box i.e. Q/B.

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.

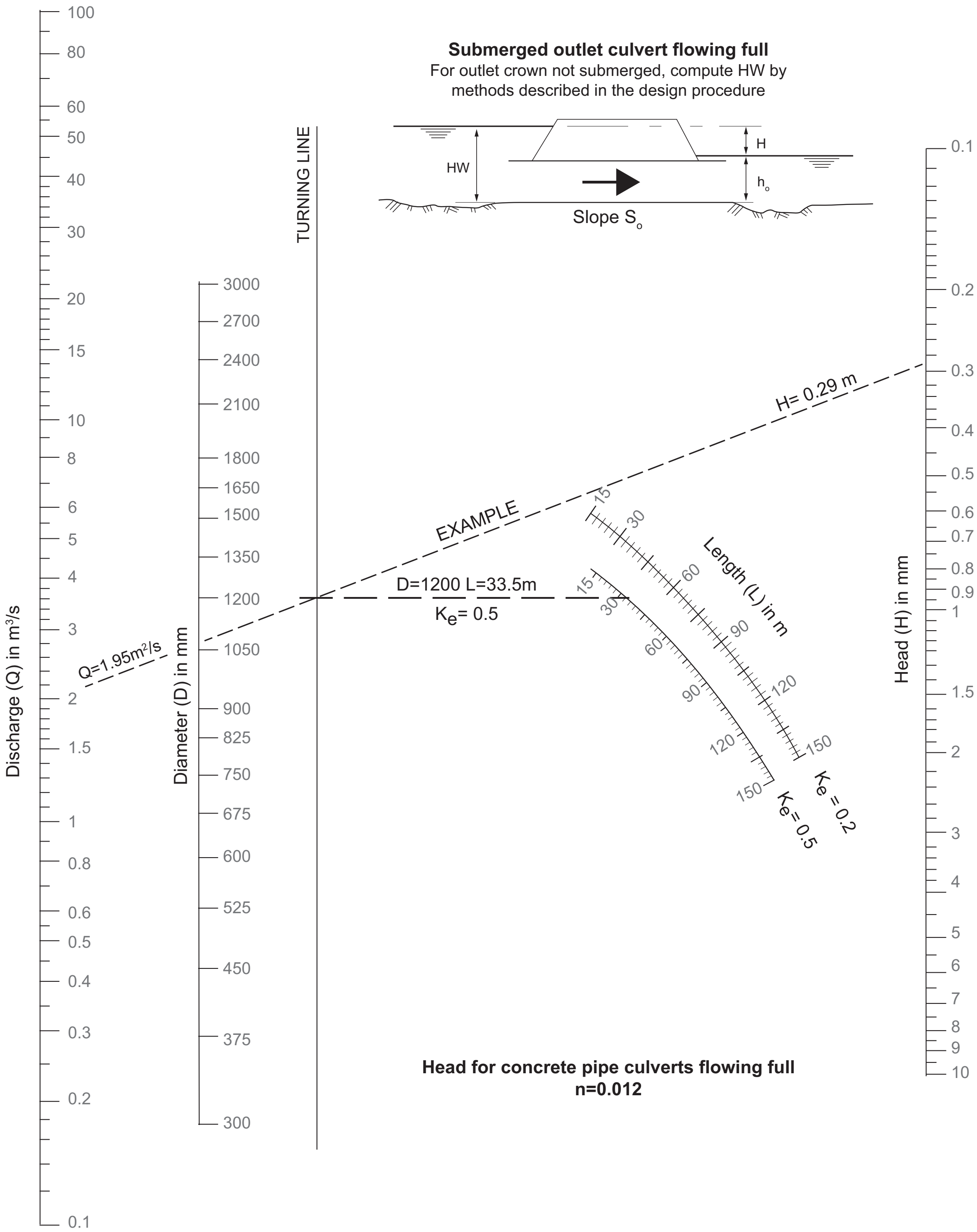


SOURCE: Adapted from Chart 8A, FHWA, 2012

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

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

i. Determine the length (L) of culvert on the scale depending on the entrance loss K_e 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.

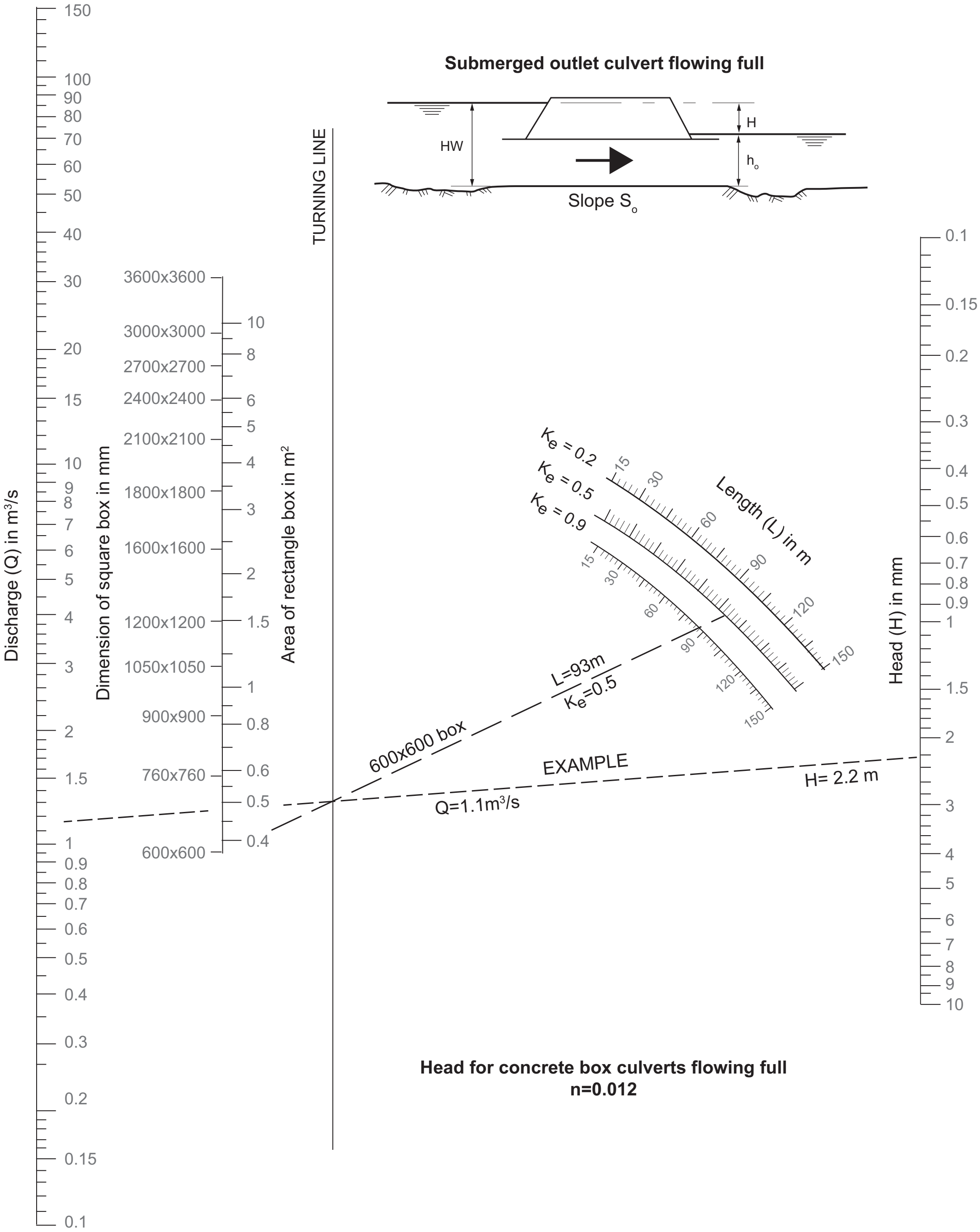


SOURCE: Adapted from Chart 5A, FHWA, 2012

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

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

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

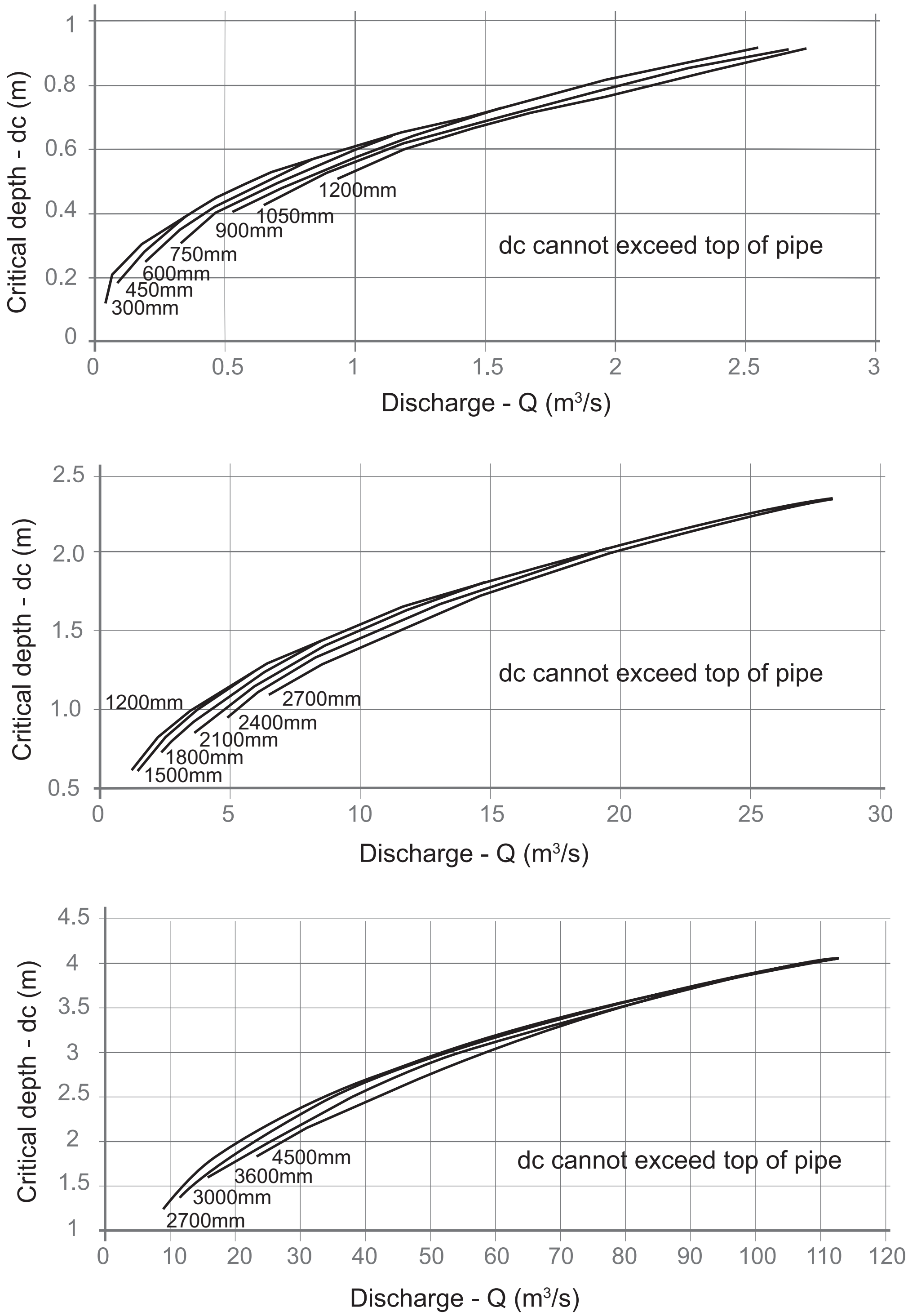


SOURCE: Adapted from Chart 15A, FHWA, 2012

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

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

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.

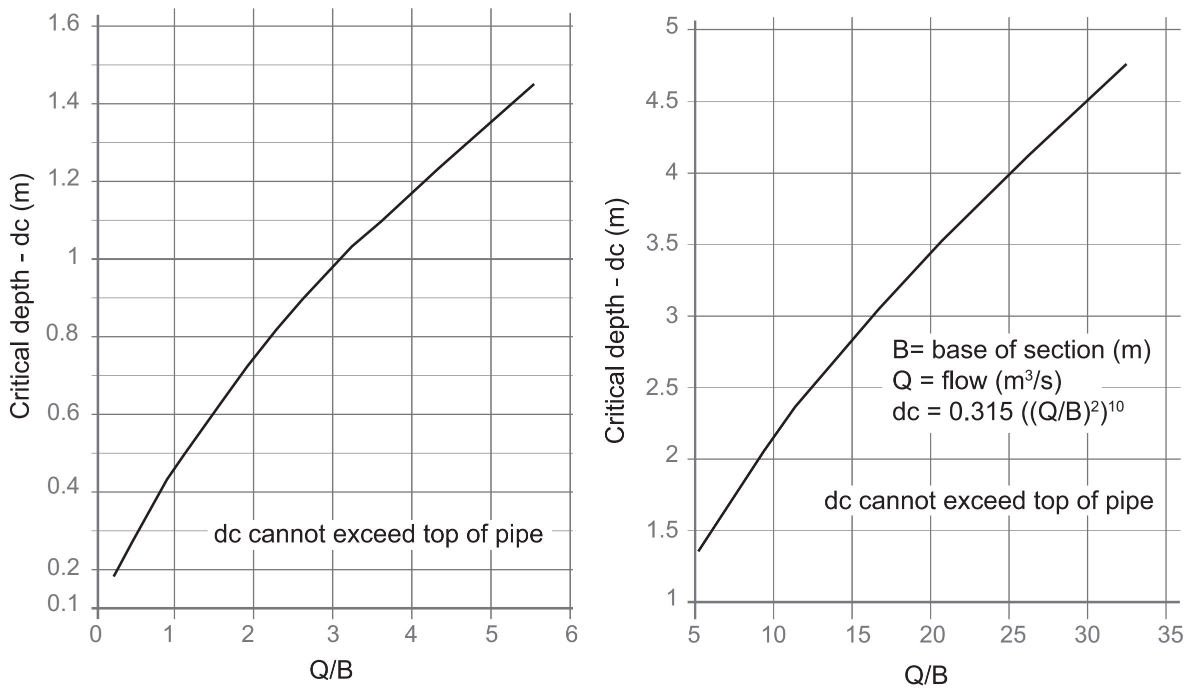


Source: Adapted from Chart 4A, FHWA, 2012

Figure C.5.10 Critical depth (d_c) for circular pipes

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

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.



SOURCE: Adapted from Chart 14A, FHWA, 2012

Figure C.5.11 Critical depth (dc) for rectangular section

Values of coefficient k_e to apply to 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.5.4.

Entrance head loss $h_e = \left(K_e \left(\frac{v^2}{2g}\right)\right)$

Table C.5.4 Entrance loss coefficients

Type of barrel and inlet	K_e
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
Mitred to conform to fill slope	0.7
End-Section conforming to fill slope (standard precast)	0.5
Bevelled edges, 33.7° or 45° bevels	0.2
Side-tapered or slope-tapered inlets	0.2
Box, Reinforced Concrete	

Type of barrel and inlet	K_e
Headwall	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or bevelled 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*
Bevelled 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 this Table.

5.3.3 DFID hydraulic analysis model for Ghana

The drainage design guidelines developed in 2006 with DFID support for DFR cover the hydraulic design of culverts, small bridges and drifts. The model is a spreadsheet developed for analysis of the sizes of different drainage structures and also rip-rap sizing, estimation of gabion mattress thickness and extent of scour protection. The approach using this tool is quite similar to the hand calculation using the nomographs. The main input data e.g. stream data, culvert data, tailwater data are all considered in the approach using the hand calculation sheet. However, the hand calculation using the nomographs does not compute the sizes of rip-rap, gabions and extent of scour checks. For further details the reader is referred to the DFID Hydrological Model. This tool is typically used by DFR for carrying out the hydraulic analysis of drainage structures.

5.3.4 Use of computer based approach - HY8

HY8 is computer-based software used for culvert analysis to help determine the size of culverts that will allow the design flows to pass without overtopping. 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 drop-down menus.

There are drop-down menus which show what information is required as the user selects each of the data inputs listed above.

After entering all data, select Analyse Crossing from the menu bar in order to obtain output results.

5.3.5 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. If an existing channel bed is altered, there will be an enhanced risk of erosion problems arising. It can sometimes be a challenge to maintain an adequate gradient over the longer length required in the case of a skewed culvert. This challenge is lessened in the case of relief culverts in sidelong ground, as there is no existing watercourse, and it is easier to ensure an adequate crossfall, even over a longer culvert length.

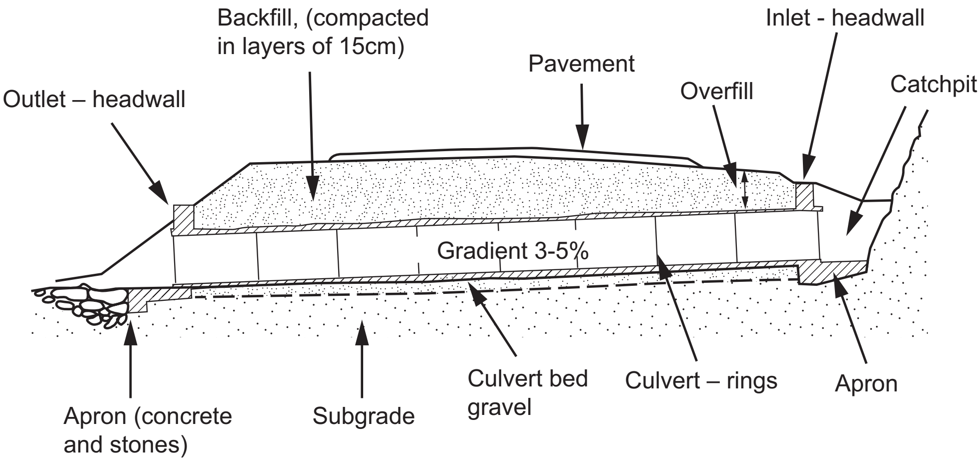


Figure C.5.12 Relief culvert

5.3.6 Worked Examples

Worked Example 1

Figure C.5.13 illustrates a pipe culvert crossing a road on an embankment at KM 5+450 on a LVR.

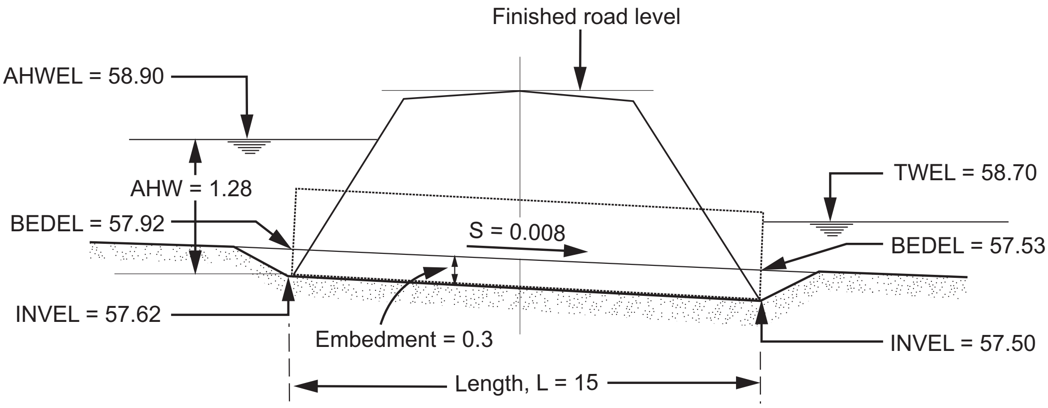


Figure C.5.13 Pipe culvert cross-section for worked example

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 m
  - No restrictions on Outlet Velocity.
- Allowable Head Water Elevation (AHWEL) = 58.90 m.
- 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 The figure in Col. 24 is larger than that in 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 Vo 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.1.

Worked Example 2

Figure C.5.14 illustrates a box culvert in inlet control located at KM 1+300 on a LVR.

Given:

- 50-year design flow i.e. Q_{50} = 28.3 m^3/s.
- Channel is nonerodible, 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.

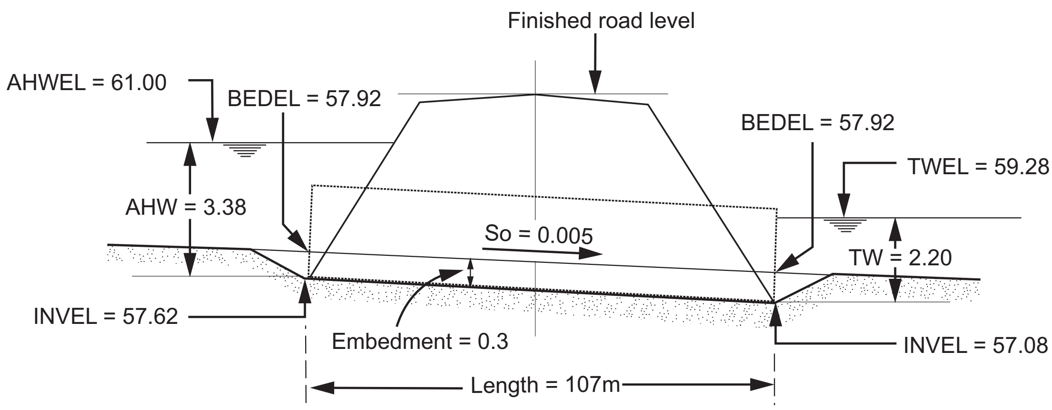


Figure C.5.14 Box culvert cross-section for worked example

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

Solution:

Step 1: Assemble and review the available data

Step 2: Culvert Selection

See the hand calculation spreadsheet in Appendix C.1. 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 m². 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 m². 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 The value in Col. 16 is larger than that in 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.5.5. 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.5.5 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, (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

NOTES:

*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 the following simplified method may be used to select the culvert size. The size of pipe is selected from Table C.5.6 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.5.6 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**

NOTES:

Q = Maximum Discharge, m^3/s

D = Internal Diameter of Pipe, m

Hw = Headwater elevation, m

It is recommended to use Hw = 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.5.7. 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.5.7 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 centre line of adjacent pipes should be at least 2 pipe diameters. Where space restrictions require a closer spacing, the factors in Table C.5.8 should be used to reduce the estimated flow rates through the pipes. Due to difficulties in ensuring adequate compaction under and between pipes, bedding of lean concrete should be used in these circumstances.

Table C.5.8 Pipe spacing and flow reduction factors

Spacing between pipe centres	Flow reduction factor
More than 2.0 pipe diameters	1.0
1.5 – 2.0 pipe diameters	0.9

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 increases from that flow. 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.5.9. The data are based on the flood return periods for the design of box culverts that are given in Table C.5.10. It is recommended to use H_w = H for box culvert design on LVRs as the cover to culverts is normally small.

Table C.5.9 Capacities of standard box culverts used on LVRs

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

NOTES:
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

Table C.5.10 Return Period (years) for standard 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

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

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. Depth of Flow (D_{tw}) = $[(Qxn)/(K \times S^{0.5})]^{0.45}$, m

a. Where,

n = Manning's Coefficient = 0.05

Coefficient K = 4.48

Q = Design Discharge

S = River Bed Slope

vii. Channel Width (W) = 5.6 x $D_{tw}^{0.26}$, m

viii. Tailwater Level (TWL) = $IL_o + D_{tw}$, m

ix. Velocity in Channel (V) = Q/ (D_{tw} x W), m/s

STEP 3: Design the drift

x. Assume length of drift (L), m. A good assumption can be made from site observations of the proposed location.

xi. Unit discharge (q) = Q/L , m^3/s/m

xii. Critical depth (D_c) = $(q^2/g)^{0.33}$

xiii. Critical Velocity (V_c) = q/D_c

V_c > 1.50 m/s, increase length of drift

xiv. Head (H_c) = $D_c + V_c^2/2g$

xv. Drift Invert Level (IL_D), to be determined on site

xvi. Maximum Depth of Water is the greater of either ($TWL - IL_D$) or H_c

5.6 Vented fords

Figure C.5.15 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 centre of the ford, and D is the diameter of pipe or the height of vent.

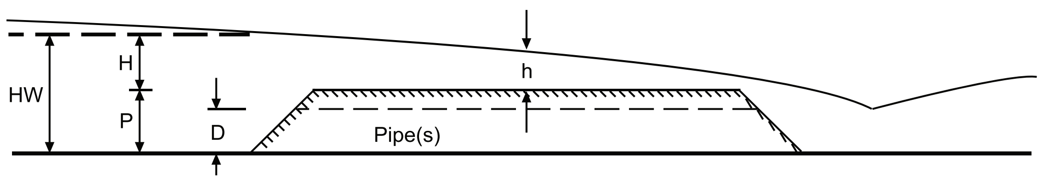


Figure C.5.15 Crossing profile of a vented ford

Design considerations for vented fords are summarized in Table C.5.11

Table C.5.11 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 3.0m/s. Exit velocity, $V_e = Q_{vent}$ /(flow area of pipe).
Pipes	Pipes should be anchored in the ground and both ends bevelled or mitred 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.

The design of a vented ford is similar to that of a culvert. Available design tools include culvert hydraulics and flow equations.

A vented ford is designed to have a flow capacity of Q_{vent}.

$$Q_{vent} = Q_e - Q_{top} \qquad \textit{Equation 1}$$

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 following equation,

$$Q_{top} = 4.83L^{0.251}H^{1.67} \qquad \textit{Equation 2}$$

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:

$$Q_{top}=1.603L^{0.0251} \qquad \textit{Equation 3}$$

After the discharge through the vent (Q_{vent}) is determined from Eqn.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: $V_e = \frac{Q_{vent}}{\frac{\pi D^2}{4}}$ *Equation 4*

5.7 Bridges

Bridges are generally the most expensive type of road structure, requiring specialist engineering advice and technically approved designs. The design of large bridges is outside the scope of this Manual. The Ministry of Transport, Standard Specification for Road and Bridge Works (2007) is a sound reference manual for design of bridge structures. The following outline guidance relates to the hydraulic analysis of minor bridge structures with a span of up to about 6 metres.

5.7.1 Bridge hydraulics

Flood Estimation

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

- The 25 - 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 analysed 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 into account 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 assessments 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.5.12 provides a guide for vertical clearance for bridges. These clearance measurements should be increased on rivers with a history of large floating items.

Table C.5.12 Vertical clearance from Design Flood Level

Discharge (m^3/sec)	Vertical clearance (mm)
< 0.3	150
0.3 to 3.0	450
3.0 to 30.0	600
30 to 300	900
> 300	1200

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 minimise 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 Guidance on 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. Minimising or eliminating the effects of scour should therefore receive the most attention in their design. 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 a localised increase in water velocity. It is important that structures are designed to minimise 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 generally require a detailed knowledge of the river and bed characteristics. They are also based on empirical data and can give rise to 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 taken into account.

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 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 m	Min. 0.6 m
Bridges	3 m	Not applicable

Rule 2 – Minimise constriction of the water flow

The amount of scour experienced at a structure is proportional to the degree to which the normal water flow is constrained. 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 constricted flow width;
- the 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 three 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, or
- Provide a piled 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 C.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 Regraded gradation curve data

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: Gradation Design of Sand and Gravel Filters, Part 633 National Engineering Handbook, October 1994, Page 26-3, United States Department of Agriculture

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 metre of fill	Safety factor
Concrete/gravel	25kN	1.5
Earth	20kN	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/m^2**

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. Unless the rock is too hard, the preferred option which should be adopted 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 are illustrated in Figure C.6.1.

The depth of the cut-off walls will depend 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 the previous section on scour. 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.

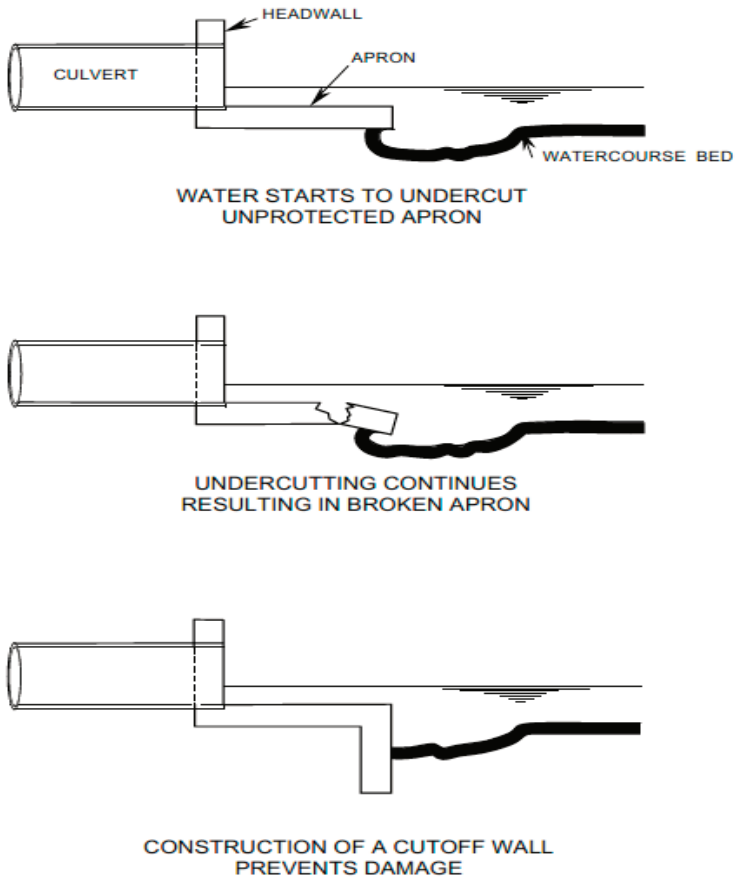


Figure C.6.1 Benefits of cut-off walls

6.5 Pipe culverts

Watercourse, relief and access culverts on LVRs are typically formed of concrete pipes. Steel and timber pipe culverts are rarely used as culverts on LVRs in Ghana. Cast ***in situ*** reinforced concrete pipes are mostly used as watercourse and relief culverts.

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

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 while reducing the risk of environmental damage, can in some circumstances be cost-effective in the long run. The minimum recommended size is generally a 900 mm diameter pipe. A 600 mm diameter pipe may be used for access road culverts and in cases where the invert depth is limited and adequate flow rates can reliably be maintained, particularly for relatively narrow very low volume roads. However, U-culverts could be used in preference to smaller diameter pipe culverts in such circumstances. The practice of achieving the required cover by raising the vertical profile of the road through humps at culverts locations is unsafe, unnecessary and 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 C.5. Standard drawings for culverts are included in a separate volume as Appendix C.2.

6.6 Headwalls and wingwalls

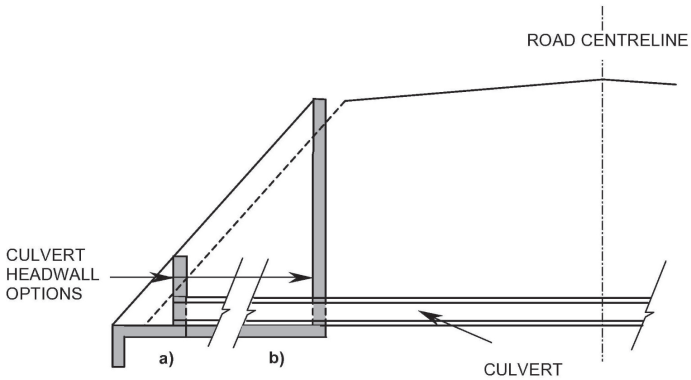
6.6.1 General

Headwalls and wingwalls are required at each end of a culvert. They serve a number of different purposes, in:

- directing the water in or out of the culvert;
- retaining the soil around the culvert openings; and
- preventing 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.



a) LONGER CULVERT, LOW HEADWALL COSTS
b) SHORTER CULVERT, HIGH HEADWALL COSTS

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 smaller. 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 metre 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).

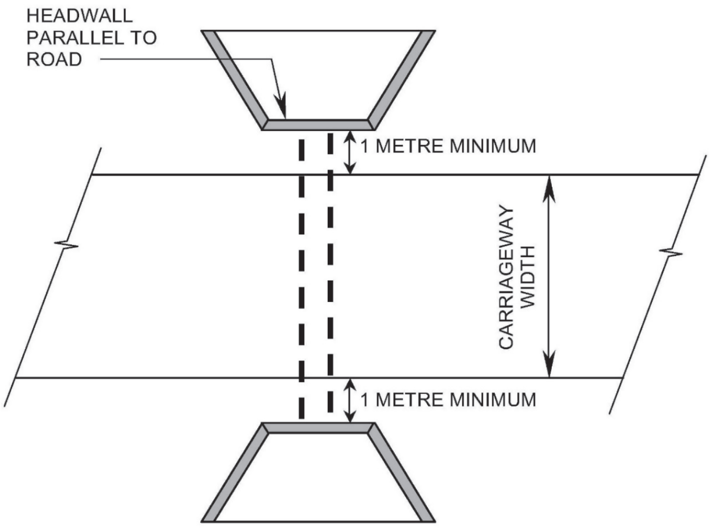


Figure C.6.4 Position of culvert headwalls

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 Wingwalls

Wingwalls are used to retain the soil behind the culvert and abutments of bridges to help guide flows 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 Features of alternative 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 raw materials to construct the wingwalls will determine the most appropriate arrangement. In general, to achieve the cheapest option, the design should ensure the smallest wingwalls are chosen for the structure and its particular 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. The two main factors affecting the overall design of a wingwall are the construction material and the bearing capacity of the soil.

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 C.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 utilises 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 also 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 is likely to depend on the type of material used for construction of the culvert. It may be constructed from gabion baskets, cemented masonry or concrete.

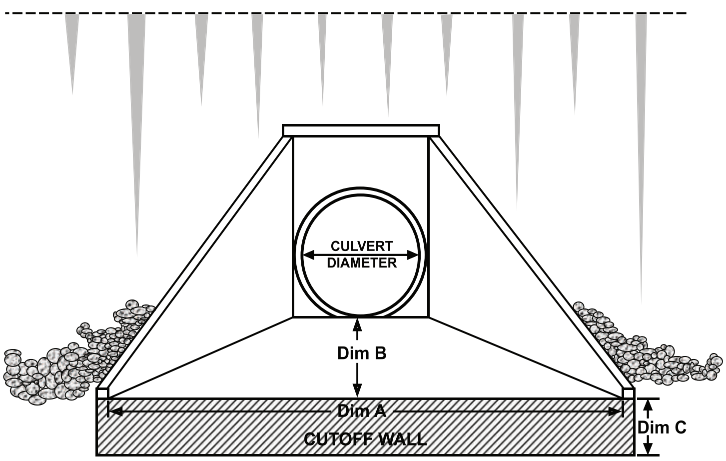


Figure C.6.6 Culvert apron

6.6.5 Approach ramps

The approaches to drifts, vented fords, large bore culverts and bridges must allow vehicles to cross the structure without losing traction or getting stuck on the crossing. Ideally crossings should not have approaches steeper than 10%. However, steeper approaches can be provided if governed by the local terrain. Approaches steeper than 10% will 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 also 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 walls (most appropriate for higher walls);
- gabion baskets;
- concrete walls (for low walls up to 500 mm); or
- timber logs (high maintenance required).
- 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 & 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 labour. 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. 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. This 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 C.6.6.4 describes the use of aprons downstream of a structure to prevent erosion and undercutting of the structure itself. 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 at downstream of culvert

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

- 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 will depend 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 is the name given to stones placed in the river bed to resist erosion. In order to be effective, the stones should be large or heavy enough that they will not be washed away during floods. Although rip-rap may appear to consist of random rocks it should be well graded and placed as tightly as possible to improve its resistance to erosion. The rocks used should also be strong and not likely to crumble. Angular rocks, in general, have the best performance, due to the interlock that is formed between 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 also be avoided as they can be easily dislodged by the water flow. 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.

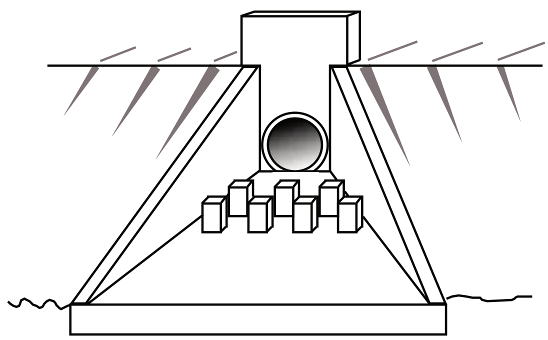


Figure C.6.8 Energy dissipating apron

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 put in the cages. The disadvantage of gabions compared with rip-rap is that they have the additional 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 river bed. If the bottom of the watercourse requires protection it is possible to make a gabion that is only 200 or 500 mm thick to form a mattress over the watercourse bed. Figure C.6.9 shows the use of gabions and mattresses for protecting the watercourse.

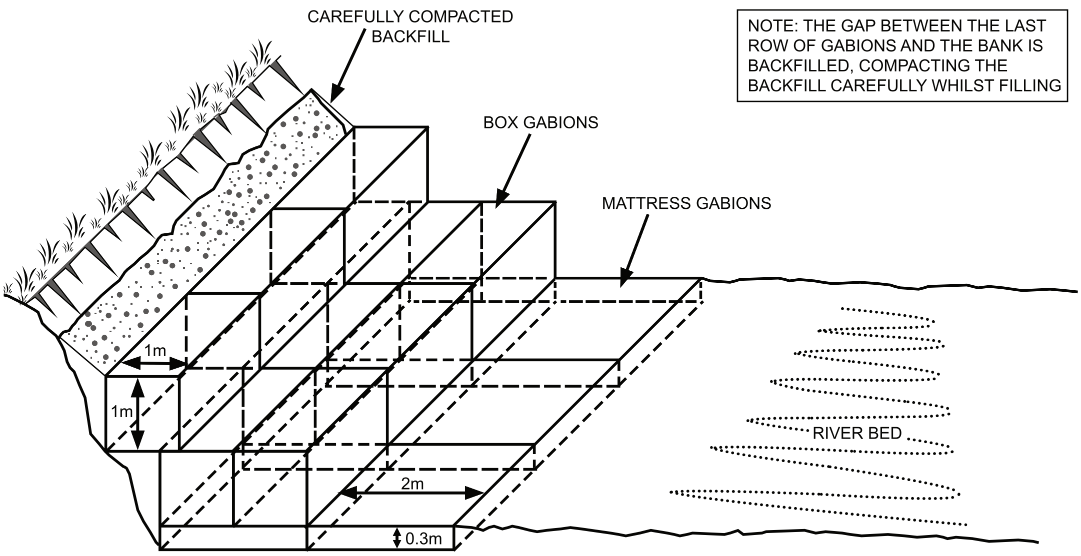


Figure C.6.9 Gabion protection on steep banks

The size of the gabions will depend on the velocity of the water flow. For all flow velocities the smallest gabion used is 0.5 x 0.5 x 1 m. 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 can also be 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 stabilisation 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 road side 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 fibre matting may also help to establish plant growth. The timing of the planting is dependent on the rainy season. Plants need to become 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 centre of the channel.

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

6.7.6 Steep channels

In areas where water is flowing down steep hillsides 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 dissipated if erosion is to be prevented. The most appropriate method in these cases is to construct a step waterfall or cascade. This 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 shows an example of gabions being used for erosion control of a steep watercourse. This type of gabion structure could be required downstream of a culvert in a seasonal watercourse, or to control flow down a steep slope, as shown in Figure C.6.11.

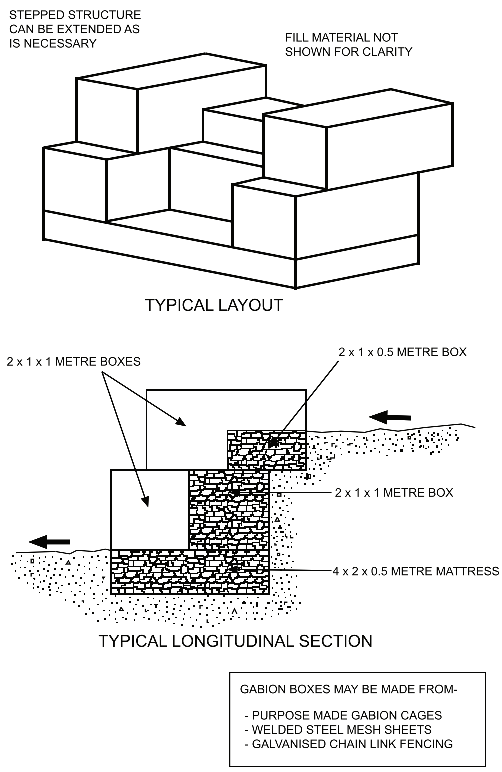


Figure C.6.10 Gabions for steep watercourse

Figure C.6.11 Gabion cascade

Any mattresses 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 very important that gabion boxes or 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.

6.8 Drift design

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 downstream erosion, requiring frequent maintenance.

NOTE: There is one situation where it may be permissible to raise the finished level of the drift above the river bed. If the site selected for the drift appears to be prone to 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.

Figure C.6.12 Drift with flow in stream

Figure C.6.13 Low water level at ford

6.8.2 Drift slab construction

There are four possible solutions for constructing the drift slab, in decreasing order of 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 labour.

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 Gabions 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 stabilise the face of the gabion and the retained material is to insert natural fibre matting in the top and face of the gabion. This also encourages vegetation growth for improved stabilisation.

6.9 Bridge design

6.9.1 General

A bridge provides clear passage over a river, road, valley or other obstacle. It is not possible to provide a standard design for a bridge since each is unique to the specific site. Bridge design is a specialised area and therefore requires the expertise of a Bridge Engineer. A bridge is basically an extension of a road, although a more sophisticated and expensive part. At a cost of typically 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 taken 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 crossing rivers. 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. Outline guidance on the hydraulic analysis of bridges is provided in Section C.5.7.

6.9.2 Location of bridges

The following guidelines relate to 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 line 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 river bed level is desirable.
3. The design should avoid locations that are undergoing change.

4. The crossing should if possible be at the narrowest section of the river and the smallest width of a flood plain.
5. The axis of the bridge should be perpendicular to the river flow in the channel and flood plain.
6. Conditions should be avoided that require excessive underwater construction.
7. Locations should be avoided where expensive river training works are needed.
8. Conditions should be avoided that require sharp curves in the approaches.

6.9.3 Channel protection

Scour

Scouring (the removal of bed or embankment materials by water currents) is one of the major characteristics of waterways, whether for permanent streams or runoff. It may occur naturally as a result of channel construction or where changes occur in the 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. A scour analysis should be undertaken and applied in the 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” (TRL, 2000).

6.9.4 Bridge structural design considerations

Load specifications

For major bridge projects on higher standard LVRs, where the commercial traffic level is relatively high, and the bridge is part of a permanent structure, appropriate 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 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 design engineer 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.6.10.

Table C.6.10 Typical values of unit mass

Material	kg/m^3
Steel	7850
Concrete (plain or reinforced)	2400
Loose sand	1600
Macadam	2240
Guard rails, fastening	3200
Timber	560 - 960
Asphalt paving	2240

2. **Live load**: Weight of vehicles passing on the road. The design engineer shall apply appropriate live load specifications depending on the road and nature of traffic.The engineer shall consult with the Chief Engineer.
3. **Sidewalk loading**: Slabs, beam and girders must be designed for side walk live loads ranging from 140 – 290 kg/m^2 .
4. **Impact load:** The dynamic loads resulting from vehicles braking or swerving are considered as impact loads in bridge engineering because of their relatively short duration. Expressed as a % of the live load, their magnitude is primarily a function of the bridge span and the structural stiffness of the bridge, but is also affected by the surface roughness. For bridges with a span L in the range 3 m – 45 m on LVRs, the % is calculated by the following formulae, depending on the bridge type, up to a maximum of 25%.
   - 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 (100 mph). 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 of equivalent 0.5 m earth added can be considered where there is no rigid approach pavement provided. It is recommended and good practice 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:** There is a Medium risk of a significant seismic event across most of Ghana, meaning that there is a 10% probability of a potentially damaging earthquake occurring in the next 50 years. In Western Region, the risk is Low while, in Tamale, Upper West and Upper East it is very low. Further information can be obtained from the National Disaster Management Organisation. 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 to respond to lateral stresses caused by other factors such as temperature changes.
9. **Temperature stresses**: The design engineer shall ensure provisions are made for movements resulting from temperature changes. This is done by providing expansion gaps.

Further considerations for the design of a bridge

The bridge designer should pay attention to:

- 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;
- cost considerations;
- aesthetic considerations;
- future requirements for maintenance and repair; and
- the overall technical and economic feasibility of the project.

Steps in the 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. Select the bridge type and prepare a proposed scheme, 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 plus dead load).

6. Calculate shear force, shear stress, and the maximum moment on the slab.
7. Calculate shear force, shear stress and the 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 a specialised field and the design should only be undertaken by individuals with the required qualifications and experience. If there are any doubts about the design of a bridge the engineer should seek guidance from the GHA or DFR. Further guidance can be found in Overseas Road Note 9 (TRL, 2000) and the Standard Specification for Road and Bridge Works (MoT, 2007).

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 cannot be reliably 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 ways 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 taking 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) serve to reduce the speed of water in a drain, remove suspended material and thus help prevent erosion of 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 can give rise to 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.

The distance between the scour checks depends on the road gradient and the erosion potential of the soils. Table C.6.11 provides a guide for scour check spacing. In very weak soils, or areas of very heavy rainfall, it may be necessary to increase this basic provision.

Table C.6.11 Scour check spacing

Drain gradient, S (%)	Scour check spacing (m)
< 5	not required
5	15
7	10
10	7
12	5

SOURCE: DFR Site Supervision Pocketbook V2, Page 16

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

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

An example of a scour check is shown 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

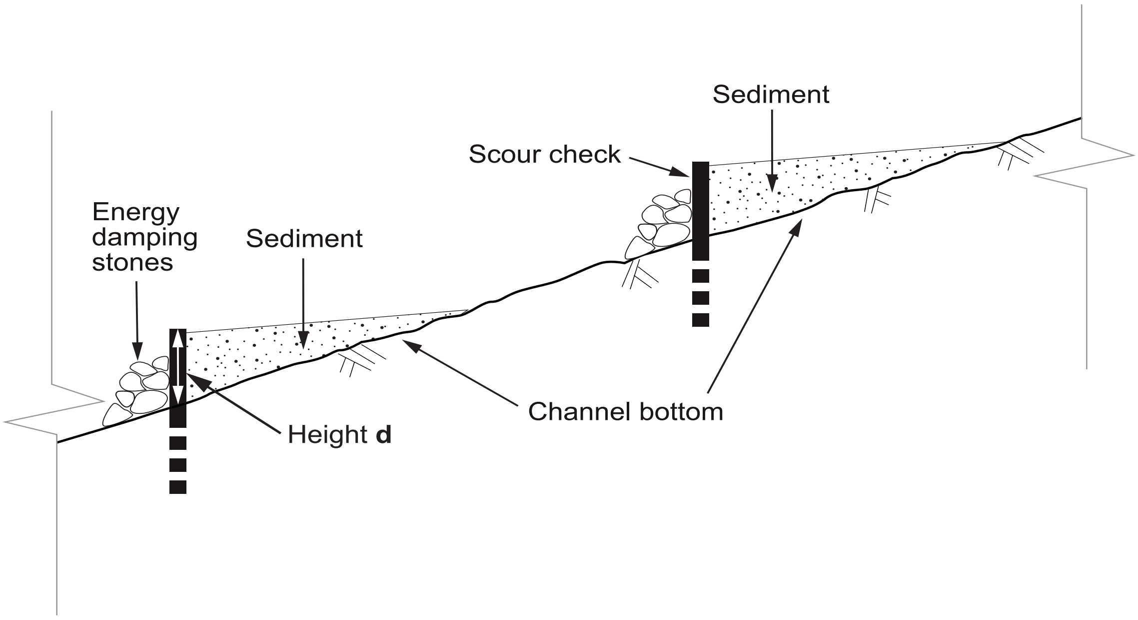


Figure C.6.15 Typical section through a channel with scour checks

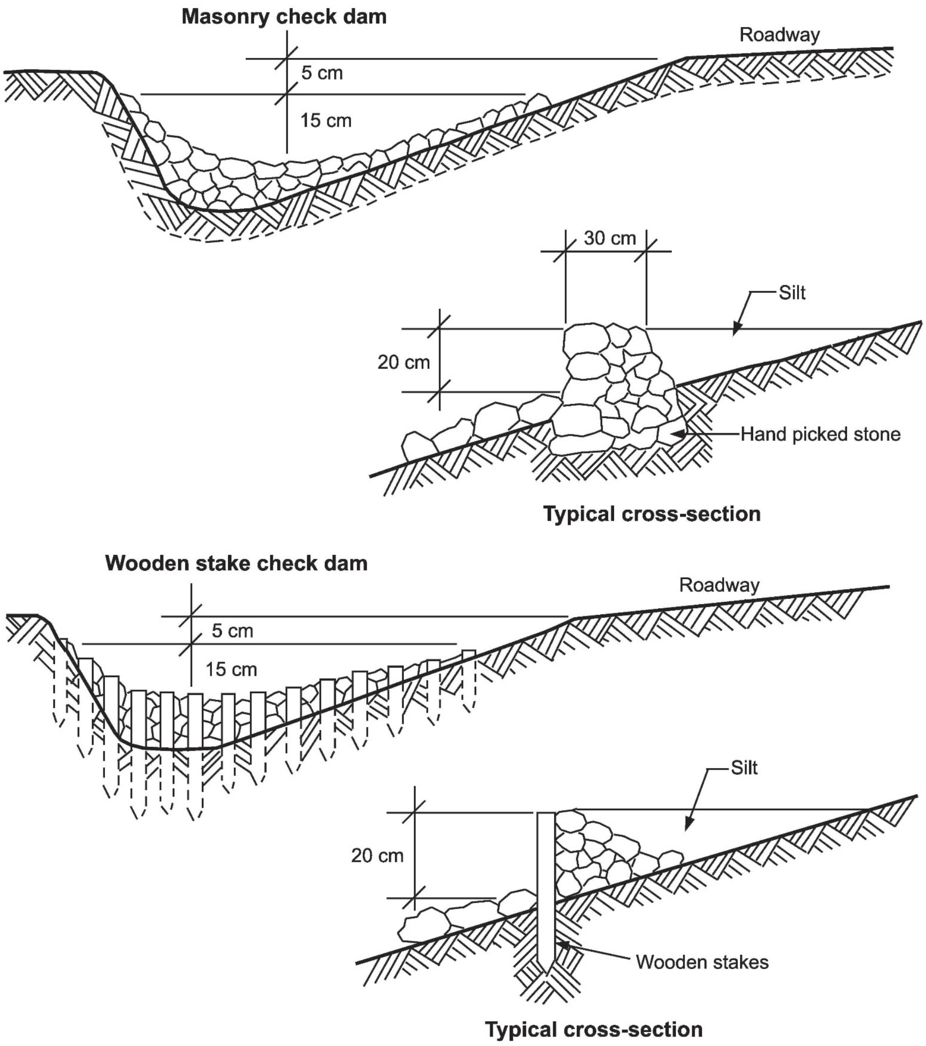


Figure C.6.16 Typical design of scour checks

6.10.3 Replanting

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

The engineering functions of vegetation in erosion protection measures include:

- retaining material from moving over the soil surface;
- armouring the surface against erosion and abrasion;
- supporting the slope by stabilising it from the base;
- 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 stabilising slopes requires good engineering design of the slope form and drainage. Chapter C.7 on roadside slope stabilisation 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 serve to minimise the risk of siltation of drainage structures:

- Design for 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 culverts are set at the right level: too low will cause silting to occur;
- Where the minimum velocity is not attainable, introduce silt traps to minimise the amount of silt entering your drainage structure. A typical silt trap is a catch-pit with a sump at least 150 mm deep;
- Ensure regular desilting; and
- Avoiding sharp turns and areas of "dead" water: silting is common where drop inlets are provided for culverts

7. ROADSIDE SLOPE STABILISATION

7.1 Introduction

Failures of natural slopes, cut slopes and embankments in hilly and mountainous areas often 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 (see Figure C.7.1 and Figure C.7.2), or where fill material is either 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. Once slope failures are initiated, they can expand rapidly, causing even greater traffic holdups and maintenance costs. Slope failures predominate in the weathered horizons, and especially in the soils formed from the weathering of metamorphic rocks in Ghana (Tsidzi, 1997).

Erosion can also take place on unprotected cut and fill slopes and in river channels, especially downstream of culverts and side drain turnouts. In addition, uncontrolled runoff can erode roadside drains, road pavements and the road shoulder. Sediment derived from this erosion not only impacts on the road, but also on the wider environment.

Large landslides can also contribute significant volumes of debris to watercourses, with major downstream environmental effects. These problems are potentially of greatest significance in the hilly and mountainous north, southwest and east of the country.

Figure C.7.1 Adverse jointing in rock slope prone to sliding along smooth joint surfaces

Figure C.7.2 Rock slide debris into road reserve due to sliding along adverse joints

Slope instability affecting the Right of Way usually occurs either above or below the road, as illustrated in Figure C.7.3. In Ghana the vast majority of slope problems relate to shallow failures and erosion above the road, involving the cut slope and sometimes the natural hillslope above it. Instability below the road can affect fill slopes and, less frequently, 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.

Further consideration is now given separately to slope instability issues that may arise above, and below the road.

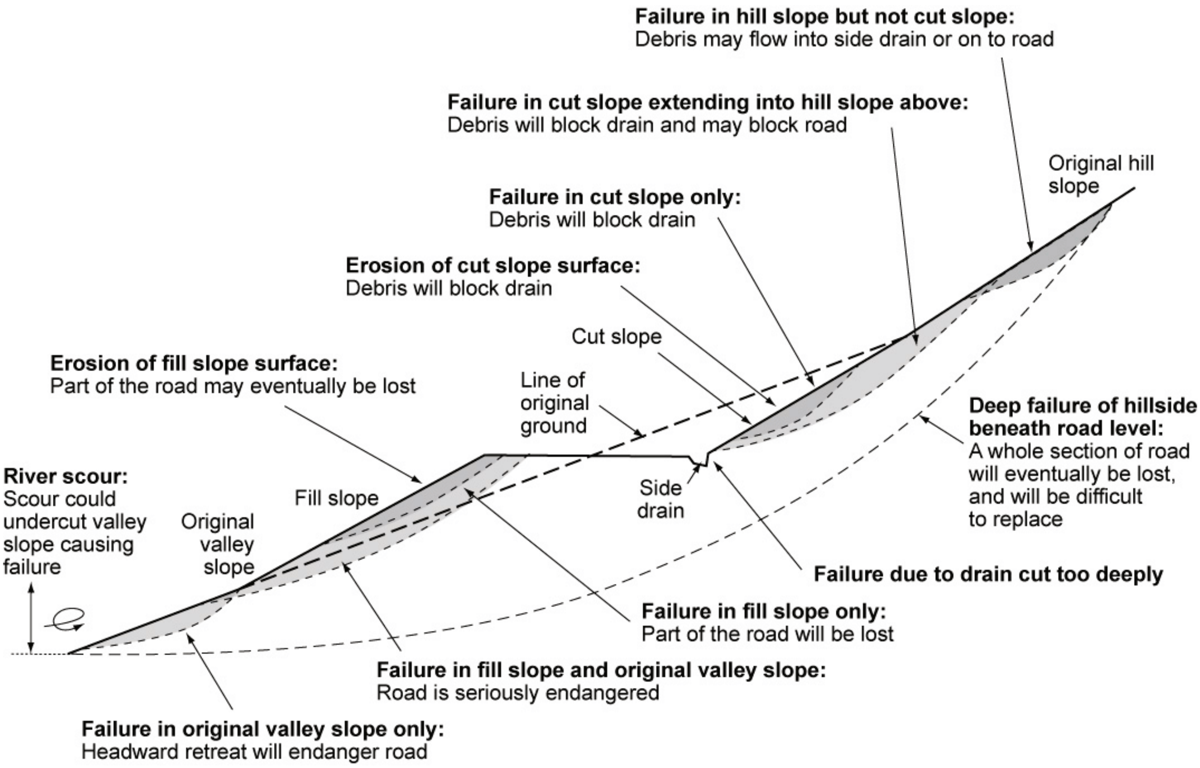


Figure C.7.3 Typical slope instability affecting roads in hilly and mountainous terrain

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 development 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 Ghana 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 on Ghanaian 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. The majority of failures affecting the natural hillside above roads are caused by the removal of support in the road cutting, combined with the effects of groundwater rise during heavy or prolonged rainfall.

Notwithstanding the above, it is also common to find that slope excavations expose an upper weathered mantle of perhaps 1-3 metres 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 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 (see Section C.7.2.2).

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 metres. 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 metres 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 metres 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 C.7.4). 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 metres in height) are generally designed in a prescriptive manner based on past experience with similar soil and rock materials. Cut slopes much greater than 3 metres may, depending on the complexity of the ground conditions, require an engineering geological assessment. This would include an assessment of the strength of the soil and the orientation of joints in the rock, if exposed.

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 (Figure C.7.5), and cuts in formations susceptible to landslides. In Ghana, the latter might include shales and mudstones with adverse bedding.

Figure C.7.5 Colluvium/landslide material, vulnerable to failure if cut too steeply

For LVRs it is imperative to minimise earthworks and retaining wall costs. Table C.7.1 shows the range of recommended cut slopes for weathered rock, residual soils and transported soils (e.g. gravelly taluvium) commonly encountered in excavations. The recommended cutting angles are based on weathering grade 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 will need to be assessed on a case-by-case basis to ensure that rock masses do not become destabilised along adversely-orientated joints. The Design Standards for DFR show cut slopes of 1:1 on bench faces.

Table C.7.1 Recommended cutting angle for rock, residual soil and transported soils

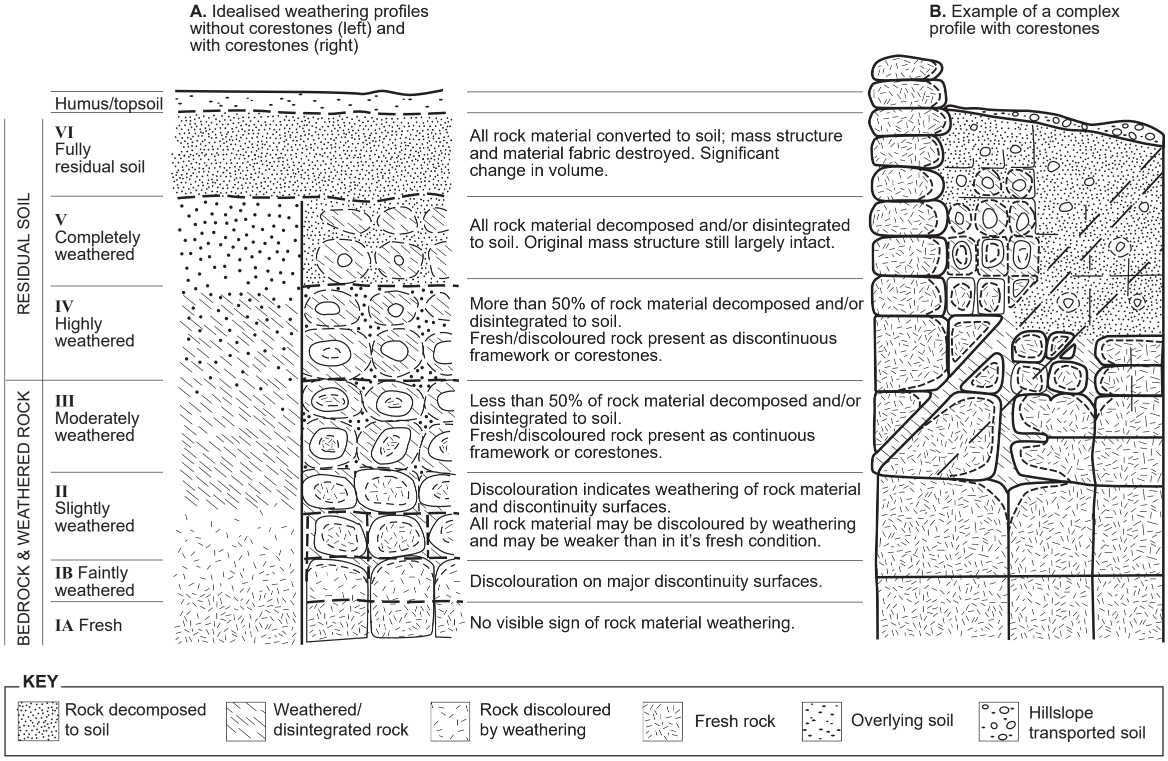
Material type	Weathering Grade	Slope face angle (degrees)	Slope face gradient (vertical:horizontal)
Competent rock	I - II	80 - 85	6:1 to 10:1
Weathered rock	III - IV[1]	60 - 75	2:1 to 4:1
Coarse-grained residual soil	V - VI	45	1:1
Fine-grained residual soil	V - VI	35[2]	1:1.5 to 1:2[2]
Coarse-grained 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]

SOURCE: Hearn,2011
NOTES:
1. weathering grade IV is borderline soil/rock
2. depends on clay content and moisture content

Ferralitic residual soils are found extensively in Ghana. They are usually formed under good drainage conditions from the tropical weathering of sesquioxide-rich rocks (e.g. Gidigasu 1972; 1974) and contain iron and aluminium oxides and hydroxides that impart a red-brown colouration. Further information about the residual soil types of Ghana can be found in Section B.3. Their engineering properties often depend more on their inherited structure than on grading and mineralogy. They are usually fine-grained and may contain inter-particle bonding, nodules and concretions that develop in the matrix caused by the deposition of iron and aluminium oxides and hydroxides as a result of chemical weathering. If surface water or groundwater are allowed to infiltrate these soils, then they may cause them to fail. If the soils are reworked, either naturally or as part of construction earthworks, they will lose their strength. This is why these materials can usually only be used on low-angle slopes as fill.

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

This is due principally to the additional strength imparted by residual rock fabric, inter-particle bonding, the effects of negative pore pressures (suctions), and case-hardening of the cut slope surface through oxidation. For example, a study was carried out of cut slopes along the Accra-Kumasi road by Liautaud in 1970 (reported in AID 1971). Soils were ferralitic and found to be derived from either weathered granite or weathered schist/phyllite (metamorphic rock). Table C.7.2 summarises the data collected for this study.



SOURCE: (Modified from Fookes 1997, and reproduced from Hearn 2011)

Figure C.7.6 Weathering grade classification

Table C.7.2 Soil properties & slope geometry of mature cut slopes on Accra-Kumasi Rd

Material	% gravel	% sand	% silt	% clay	Liquid limit %	Plastic limit %	Plasticity Index %	Moisture content %	Slope angle (V:H)	Height (m)
Granite soils	15	45	25	15	50	32	18	9	1:1 to 6:1	2-10
Phyllite/ schist soils	5	15	50	30	65	38	27	21	2:1 to 9:1	2-8

The examples given in Table C.7.2, where soil slopes have apparently remained stable at up to 9V:1H (i.e. almost vertically), are extreme cases, and it is not recommended that slopes are cut to these angles, except over low height ranges (< 2 metres), and where the slope will remain largely dry throughout the year. If there is an obvious source of surface water that will penetrate the soil, or if water can be seen to be seeping out of the slope during the wet season, then it would be unwise to cut slopes to such steep angles. Also, the excavation of steep soil cut slopes in situations where failure would pose a potential risk to neighbouring buildings and the public should be avoided.

Another study of earthworks slopes in Ghana was undertaken by Tsidzi (1997). Based on slope inventories made along the Awaso-Nobekaw and Elubo-Asemkrom roads, this author produced the following guidelines for cut slopes of up to 10m in height:

1. Competent rock and laterite: 6V:1H
2. Moderately to completely weathered (WG III-V) granite, gneiss, quartzite, sandstone: 3V:1H
3. Moderately to completely weathered (WG III-V) slate, phyllite, schist, mudstone: 1V:1H

Of these, 1) above is consistent with Table C.7.1, whereas 2) may be too steep for some of the weaker soils in this category. In the case of 3) weathered mudstone would probably fail at 1:1, depending on groundwater conditions, and the stability of excavations in slate, phyllite and schist will depend mostly on the orientation of foliation and other joint sets.

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

For deep excavations in rock (deeper than 5-6 metres), it is common to bench the cut slope. Figure C.7.7 illustrates an example of a benched cut slope in rock. Table C.7.3 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 Design Standards for the Department of Feeder Roads (MRH 2009) show typical cross-sections for paved and gravel roads with 4 m bench heights and 2 m bench widths with a catchwater drain on each bench.

Figure C.7.7 Benched cut slope in rock

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) will be 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 localised 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.7.8) as long as 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.

Table C.7.3 Advantages and disadvantages with 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 localised 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: 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 and they remain stable for considerable periods of time. This is due principally to the additional strength imparted by residual rock fabric, the effects of negative pore pressures (suctions) on slope stability and case-hardening of the cut slope surface through oxidation. Red-brown residual soils are found extensively in Ghana. They are usually formed under good drainage conditions and they are formed by the development of iron and aluminium oxides and hydroxides. The clay mineralogy of these soils usually includes kaolinite and they are dominated by the ferruginous, ferrisol and ferralitic residual soil groups. Their engineering properties often depend more on their inherited structure than on grading and mineralogy and they usually behave as if they are bonded. They are usually fine-grained and may contain nodules or concretions that develop in the matrix when there are high concentrations of oxide precipitation. If surface or groundwater is allowed to infiltrate these soils, then they may fail. If the soils are re-worked, they will lose their strength which is why these materials can only be used on low-angle slopes as fill.

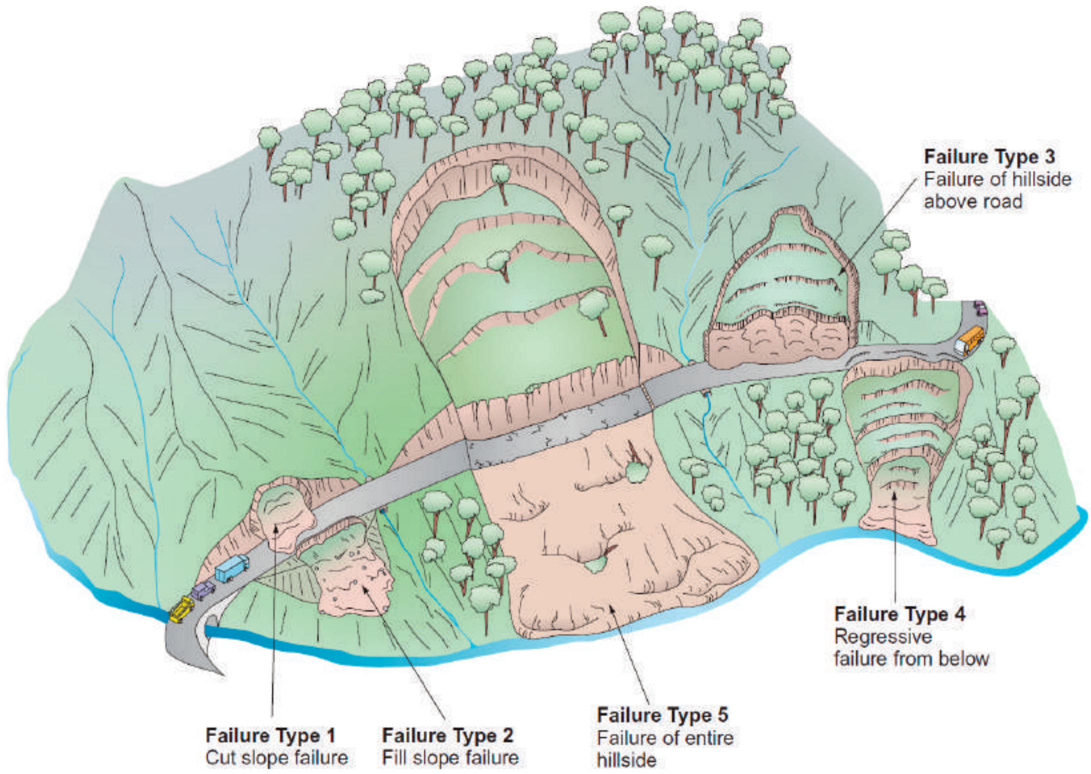
Figure C.7.8 Benched cut slope in residual soils

7.2.3 Slope stabilisation above the road

Soil slopes

If a cut slope or natural hillside above the road fails, there are a number of 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 possibly 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 considered to be one of the principal causes of slope instability.

Figure C.7.9 illustrates some of the common and less common forms of slope instability affecting mountain roads. Most slope stability problems relate to shallow landslides in cut slopes (failure types 1 and occasionally 2). Table C.7.4 then outlines some typical slope management responses to these hazards. For existing roads, the three right-hand columns apply (stabilisation, protection, or accept). Techniques of stabilisation and protection are summarised in Table C.7.5. For LVRs, the priority will be to find low-cost solutions, maximising the use of surface drainage and earthworks and locally-available materials, for example through the construction of gabion and masonry walls (Section C.7.5), and the use of bio-engineering measures (Section C.7.6). 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 stabilisation or protection is too prohibitive compared to the risk posed by the hazard. For example, a slow-moving landslide on a LVR may be easily manageable by refilling and regrading compared to the cost of attempting to stabilise it.



SOURCE: Hearn, 2011

Figure C.7.9 Common & less common forms of slope instability affecting mountain rds.

Table C.7.4: Slope hazard management options

Failure type	Engineering Management				
	Avoid	Remove	Stabilise	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, stabilisation or protection is too difficult or costly compared to the damage inflicted.
2	Shift road into hillside to avoid unstable fill slope below. However, this may initiate type 1 and type 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 re-compaction, 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 stabilisation 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 type 3 failures.		If the slope failure is local to the road, then stabilisation 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.		Stabilisation 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.

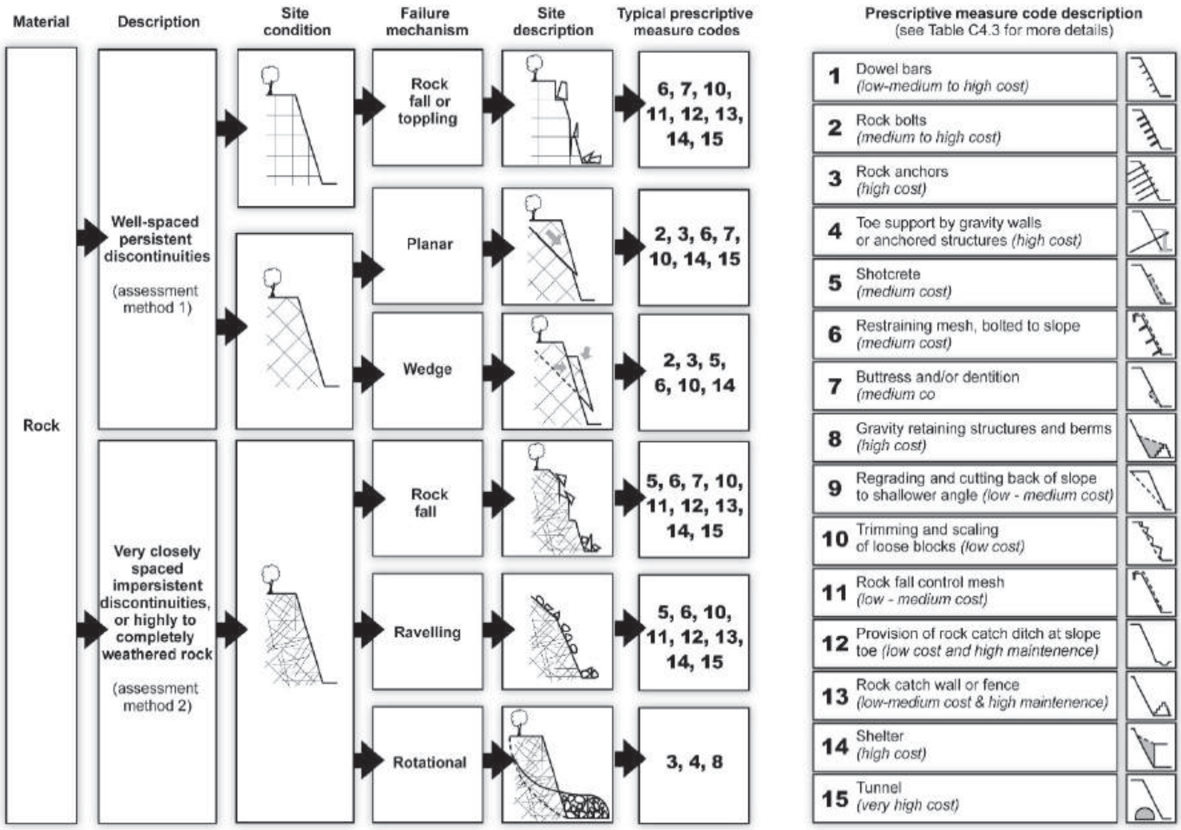
Table C.7.5: Stabilisation and protection options for slopes above the road

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

SOURCES: Table C.7.4 Hearn (2011)
Table C.7.5 Based on Hunt et al., 2008

Rock slopes

Figure C.7.10 shows a range of measures commonly used to stabilise or mitigate rock fall and rock slide hazards along mountain roads. For LVRs, prescriptive codes 12-16 (right hand column) may be the only practical options based on cost considerations.



SOURCE: From Hearn, 2011

Figure C.7.10 Rock slope materials, failure mechanisms and remedial measures

Where adversely-orientated rock joints are steep-dipping the simplest approach is 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 cause 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 might include:

Where the hazard is from falling rock:

- Realign the centre-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 and passing traffic.
- 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 sprayed or hand-applied 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 reinforce the rock mass by anchoring surface rock layers into the underlying stable rock mass.

For new roads, if at all possible, the road should be located away from hazardous slopes. 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 into the hillside.

Wire netting has been used on high volume roads in Ghana to contain rock fall debris from adversely jointed rock masses in cut slopes. It will be a matter for site assessment to determine whether such schemes should be applied to resolve individual slope instability problems on LVRs, but if the risk is considered high enough then there may be justification. Wire netting can be used to good effect with sprayed concrete (shotcrete) in high risk slope situations, though specialist engineering geological advice should be sought. Figure C.7.11 illustrates the use of wire netting on a cut slope to contain rock fall debris and encourage plant growth.

Figure C.7.11 Use of wire netting on cut slope

7.3 Slope instability below the road

7.3.1 Causes and mechanisms

Figure C.7.9 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.7.9, relate to scour below culverts and mitre drains 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 2m. Generally, embankment stability is dependent on fill type and compaction, presence of water or drainage provision, shrink and swell cycles, vegetation, slope angle and height, construction method and type of foundation. Embankment failure during or 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.

7.3.2 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.7.6. 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 1V:1.25H 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.

In general, for LVRs, fill slopes 3 metres or less in thickness, with 1V:2H or flatter 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 metres thick 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.7.6: Recommended fill slope batters

Fill material	Fill slope height	
	< 5m	5-10m
Rock fill	1V:1.5H	
Well-graded sand, gravel and sand or silt mixed with gravel	1V:1.5H	1V:2H
Poorly-graded sand	1V:2H	1V:3H
Sandy clay soils, silty clay and stiff clay soils (excluding expansive clay)	1V:3H	Not recommended
Soft clay/plastic clay (excluding expansive clay)	1V:3H	

SOURCE: Ethiopian Low Volume Roads Manual, ERA, 2017

NOTE: The use of expansive clays is not recommended; slopes may need to be modified for traffic safety reasons.

The Practitioner's Guide to Rural Roads Improvement and Maintenance (MLGRD 2012) contains further guidelines on embankment slope design and construction.

Fill slope stabilisation and protection

Table C.7.7 provides some slope stabilisation 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 on *in situ* soils where allowable bearing pressures permit.

Table C.7.7: Stabilisation and protection measures for slope instability below the road

Instability	Stabilisation 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

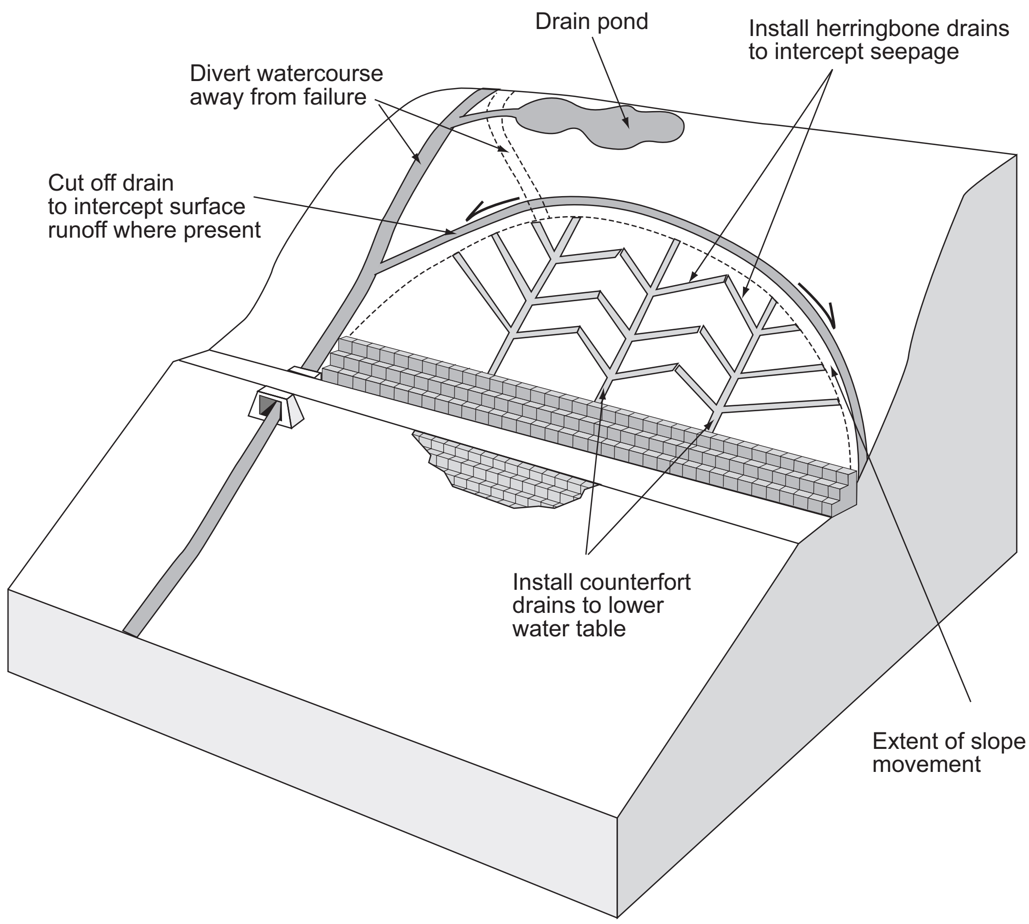
7.4 Drainage

Several surface drainage systems can be considered on earthworks slopes and natural slopes, depending on slope geometry, materials and potential failure mechanisms (Table C.7.8). 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.7.8 Common techniques of slope drainage

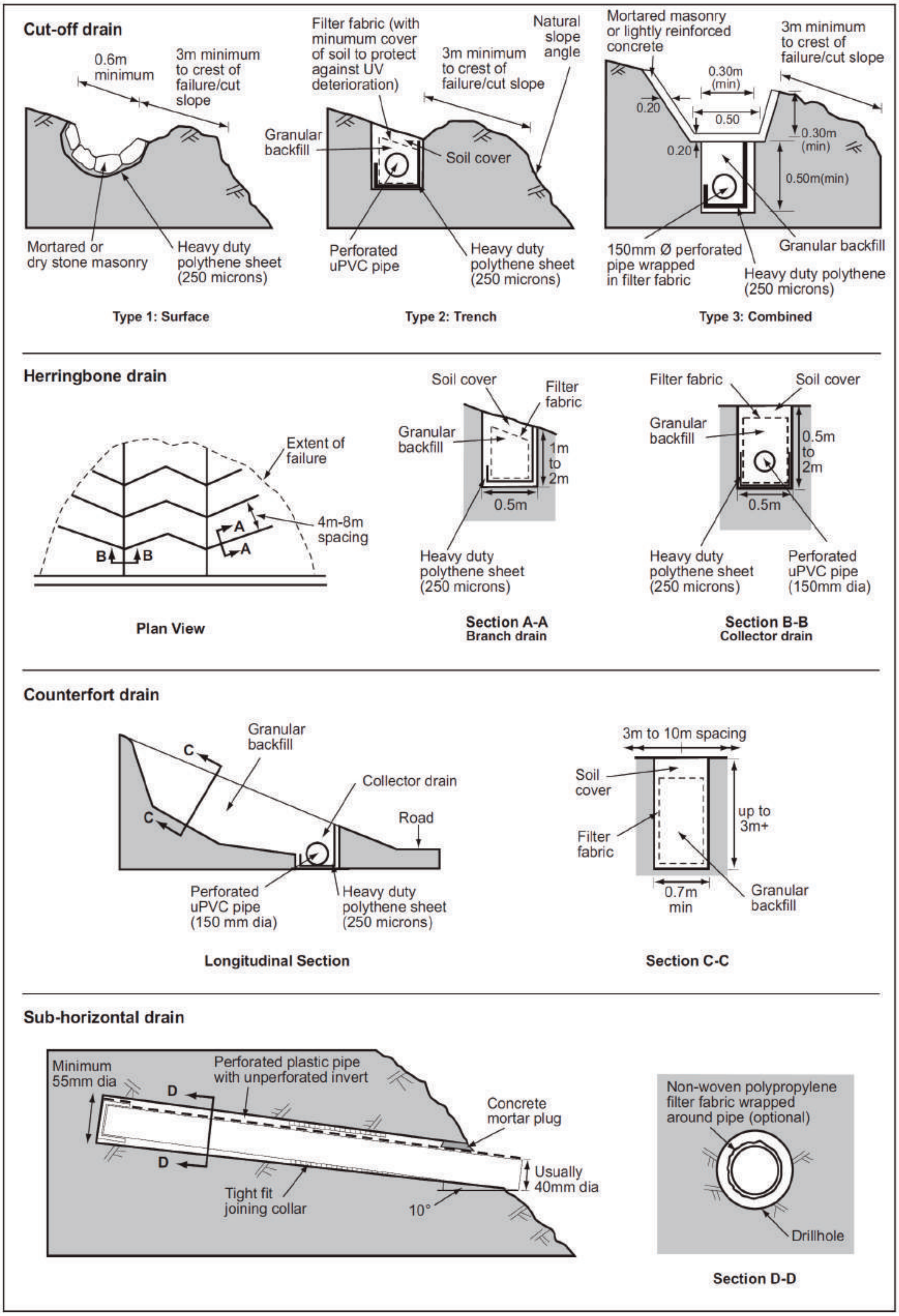
Function	Type	Advantage	Limitation
Interception of surface runoff	Unlined cut-off drain (open ditch)	Cheap, can in some instances prevent ingress of surface runoff into landslide masses	Prone to leakage and erosion; may act as incipient tension crack beyond slope crest; requires frequent inspection for 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	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
	Berm drain on fill slopes		
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 stabilise 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 minimise 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: From Hearn 2011



SOURCE: Hearn, 2011

Figure C.7.13 Typical slope drainage measures



SOURCE: From Hearn, 2011

Figure C.7.14 Typical slope drainage details.

French drains (Type 2 on Figure C.7.14) 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 design and installation, to ensure that the drainage system performs as an integral component within the overall slope improvement or slope stabilisation scheme.

Stepped channels are sometimes constructed to convey water down a slope. Such ‘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.

7.5 Retaining walls

7.5.1 Use of retaining walls

The use of retaining walls on LVRs in Ghana 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 minimise the need for retaining walls. However, the use of retaining walls is sometimes unavoidable, for example to:

- support road fill on steep slopes;
- retain slipped material; or
- support cut slopes that have to be cut more steeply than the exposed material can stand unsupported.

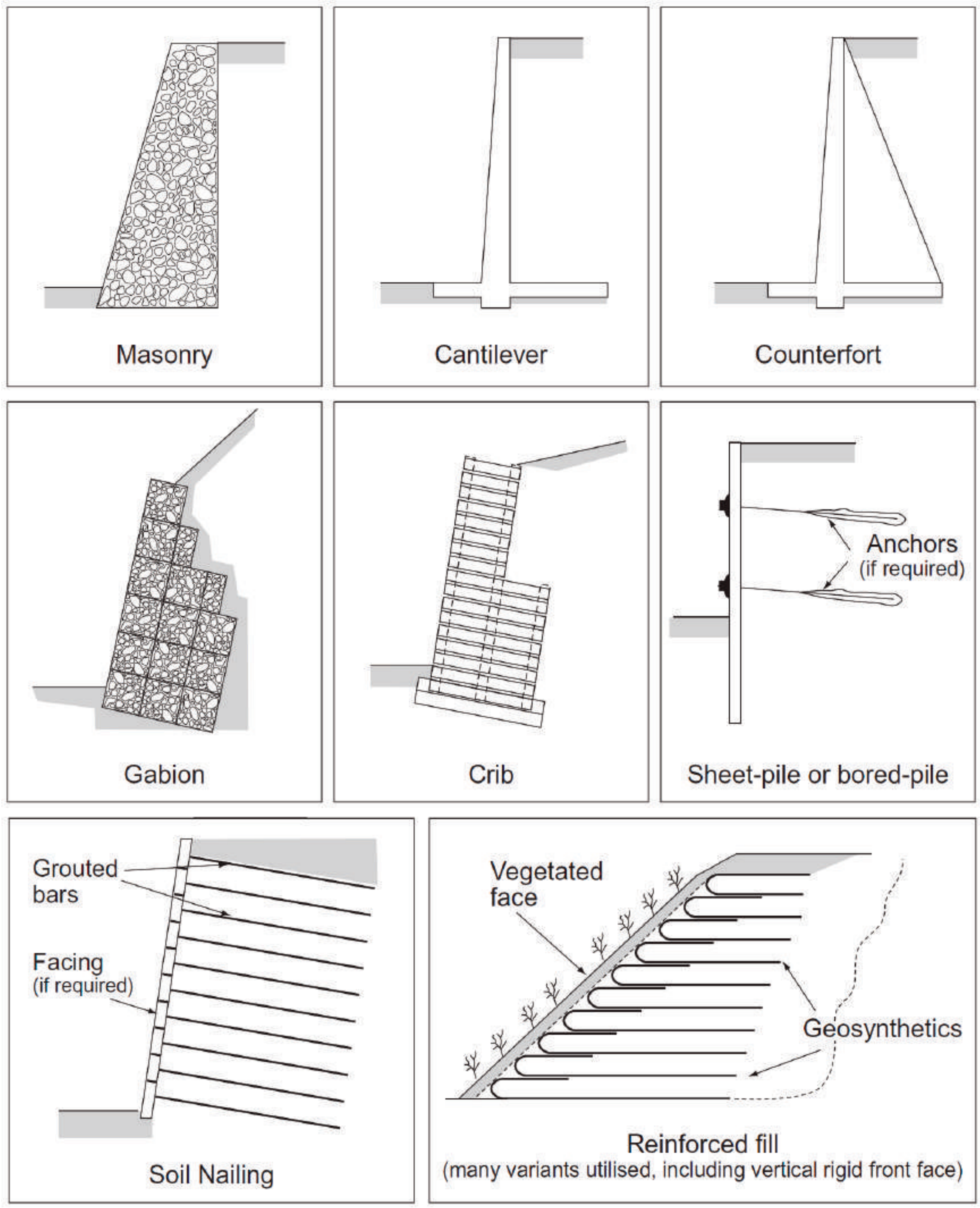
In rarer cases, retaining walls are used to support the road where it crosses unstable ground, as described in Section C.7.3.

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 the majority of slip surfaces that affect mountain roads. Geological interpretations will be 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. However, gabion, dry stone and mortared masonry are by far the most common of wall types found on LVRs, and they are briefly outlined below.

7.5.2 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.



SOURCE: From Hearn, 2011

Figure C.7.15 Typical retaining structures

Gabion walls are cost-effective because they employ mainly locally available rock and low-skilled labour. Gabion is commonly used for walls of up to 6 metres high, 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 in order 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 can give 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, in order to reduce the potential for differential settlement.

7.5.3 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 metres 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 C.7.6). Revetments of 1 to 1.5 metres 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.

7.5.4 Mortared masonry walls

Masonry walls are brittle and cannot tolerate settlement. They are especially 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 in a slope protection capacity. Mortared masonry walls tend to be more expensive than gabion walls, though there are exceptions.

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 metre centres 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

7.5.5 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 probably 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 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 likely to be 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).

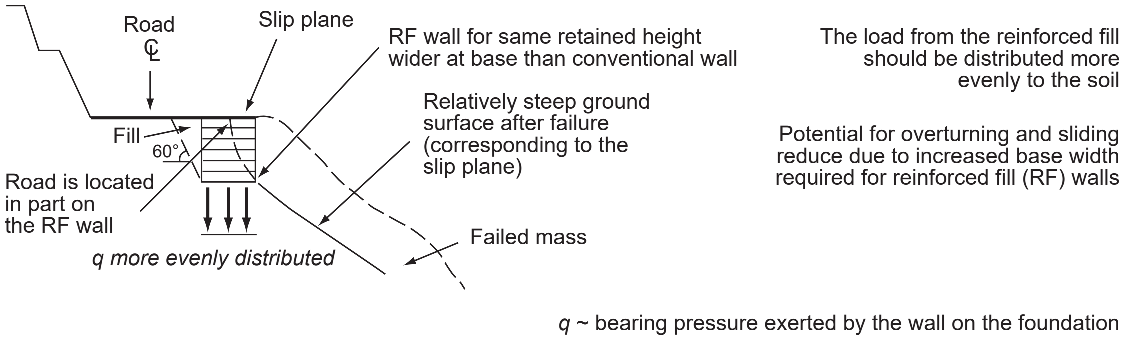


Figure C.7.17 Bearing pressures distribution below reinforced fill, & retaining, walls

7.6 Bio-engineering techniques

7.6.1 Overview

Bio-engineering entails the use of a combination of grass and shrub planting with small-scale engineering works, such as dry-stone walls, check dams, and surface drainage measures to prevent erosion and aid slope drainage. Section 8.5 of the Practitioner's Guide to Rural Roads Improvement and Maintenance (MLGRD 2012) provides some general advice for protecting embankment slopes with grassing and turfing and this guidance should be read in conjunction with the text below.

Certain types of plants, due their ground cover and dense root systems provide particularly effective 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 metre depth over which roots normally penetrate. However, some species, such as vetiver, have very deep root systems and can offer greater potential for shallow slope stabilisation. Vetiver has been used successfully as an earthworks erosion protection measure in other parts of Africa, and has already had some application in Ghana (see www.vetiver.org).

7.6.2 Selection of appropriate 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).

There are no known applications of grass planting for soil conservation purposes along highway slopes in Ghana. The use of grass planting and other techniques of bio-engineering as an erosion-control measure on earthworks slopes was applied by ArcelorMittal Liberia Ltd (AML) in the Nimba mountains of Liberia in 2009 under terrain and soil conditions not too dissimilar from parts of Ghana. Research carried out by AML (Poilecot 2015) identified a number of grass species with soil conservation potential in that humid tropical environment. 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. They may have potential application for soil conservation purposes in Ghana as well, and it is recommended that experimentation is made with the use of some of these grasses in consultation with forestry and agricultural specialists, possibly in connection with the Forestry Commission. Tsidzi (1997) describes how Chromoleana odorata (Siam weed), Panicum maximum (Guinea grass) and Andropogon gayanus (Gamba grass) were observed to provide protection against erosion on cut slopes along the Awaso-Aboferem road. These plants also have animal feedstock potential and it is recommended that their use is discussed with farming communities in different parts of Ghana to determine which plants offer the greatest multiple-use value. Siam weed, however, is regarded by many (Uyi and Igbinosa 2010) as an invasive pest, and careful selection of species is therefore required.

7.6.3 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.

The following steps are recommended:

- Trim soil and debris slopes to the final design profile. Remove excessively steep sections of slope. In particular, remove over-steep lower sections, since a small failure at the toe can destabilise the whole slope above.
- Remove all small protrusions and large unstable boulders.
- Remove depressions that make the surrounding material unstable by trimming back the whole slope around them or by filling the depression.

7.6.4 Recommended techniques

Table C.7.9 and Table C.7.10 outline the different types of bio-engineering techniques recommended for slopes and soil materials above and below the road respectively. Bio-engineering techniques commonly used for erosion control in the vicinity of the Right of Way in general are given in Table C.7.11. Some of the bio-engineering techniques referred to in these tables are illustrated in Figure C.7.18.

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

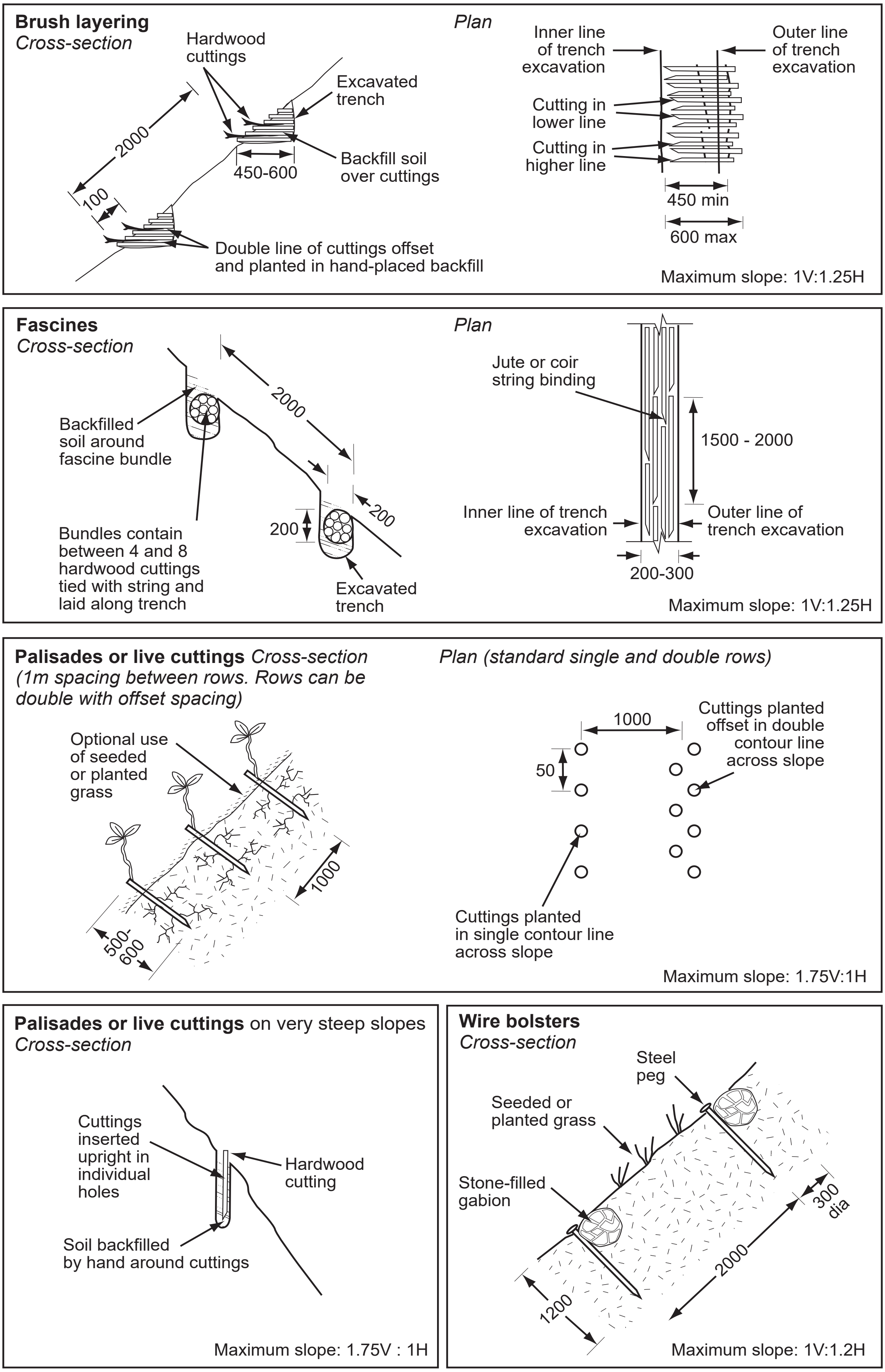
Site characteristics	Recommended techniques
Cut slope in soil, very highly to completely weathered rock or residual soil, at any grade up to 2V:1H	Grass planting in lines, using slip cuttings.
Cut slope in colluvial debris, at any grade up to 1V:1H (steeper than this would require a retaining structure)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

Table C.7.10 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 10m 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.7.11 Bio-engineering techniques for slope improvement near Right of Way

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



All dimensions in mm

SOURCE: From Hearn, 2011

Figure C.7.18 Typical bio-engineering details

7.7 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 in order 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.
- Recognise that 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.
- Recognise that bio-engineering can significantly enhance the slope environment, but is ineffective if the depth of movement or potential movement is significantly greater than about 0.5 metres.

8. CONSTRUCTION METHODS

8.1 Introduction

For drainage structures on LVRs to achieve their design life, is essential that they are well designed and properly constructed in accordance with the specifications.

This Chapter provides guidance on the construction methods used for a range of small structures such as drifts and culverts. These methods have a bearing on the preparatory work, the planning of site works, and the various site activities that culminate in satisfactory completion and eventual handover of site works. It includes aspects of programming, construction, supervision and monitoring of works, whether the structure is built by a contractor, a road authority work force or a work group set up specifically for the task. Because most aspects of drainage-related works lend themselves to labour-based construction methods, the focus is on the use of local labour that can be utilised 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.

8.2 Construction materials

A range of materials can be used for the construction drainage structures on LVRs. These include concrete (plain or reinforced), timber, block masonry, stone masonry and steel.

The topic of Construction Materials for LVRs is dealt with in detail by the Labour-Intensive Public Works Manual - Practitioners Guide to Rural Roads Improvement and Maintenance developed by the Ministry of Local Government in close consultation with DFR (MLGRD, 2014). This document is available from the Koforidua Training Centre (KTC). It is recommended that it is used as a supplementary manual.

8.3 Preparatory work

8.3.1 General

Preparatory work of varying nature, depending on the size and complexity of the small structure, is always required in terms of planning and mobilising 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.

8.3.2 Minor culverts

In the case of minor culverts, the amount of preparatory work that proves possible in practice may be limited due to resources constraints and may not in any case be justified on account of the standardised nature of most such culverts. However, some aspects of the preparatory work in the following sections for larger structures may be relevant.

8.3.3 Bridges, drifts and major culverts

The size, funding and other resources required for larger structures will usually necessitate considerable preparatory work before site works can begin.

It is assumed that structural survey and design will have been carried out in accordance with the guidelines elsewhere in this Manual and with any locally established standards. It is also assumed that cost estimates, detailed drawings and bills of quantities will have been prepared for the works.

If the works are contracted, appropriate contract documentation should be prepared in accordance with local standards and procedures. When a contractor is to be appointed, local contractor classification, tendering, selection and award procedures should also be complied with. Arrangements should be in place for independent administration of the contract and for the professional resolution of any disputes that may arise. Arrangements for management, supervision, testing, approval and audit of the works should be clearly defined. All of these issues should be clearly documented and communicated 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 that is not fit for its purpose, to an inefficient use of resources, or even to serious damage to or eventual loss of the structure.

8.3.4 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 and by a Contractor.

Table C.8.1 shows a checklist which may be used as the basis for developing a Bill of Quantities. Bills of Quantities in a national standardised format, with activity related items, will assist Employers and Contractors in pricing works and assessing value for money

Table C.8.1 Checklist of cost components for detailed costing of a drainage structure

Direct costs	Overheads
▪ Materials ▪ Unskilled labour ▪ Skilled labour ▪ Equipment purchase ▪ Equipment operating costs ▪ Equipment hire ▪ 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.	

8.4 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 remote from organisational 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 can lead to serious delays and increased costs to the project. Table C.8.2 and Table C.8.3 are checklists for developing a plan to ensure the timely and successful implementation of projects.

Table C.8.2 Checklist for planning site works

Ref	Planning activity	Check
1	List all construction and support activities and prepare a Work Breakdown Structure (WBS)	
2	Prepare a program of works or construction programme (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, labour, (permanent/casual) employment, gender and disadvantaged groups opportunities, payment, security for payment to labourers, 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 in place for (sub) contractors, and suppliers.	
14	Plan staffing identify skills locally available or required to be imported to the site area, accommodation, logistics, transport to site, recruitment and training.	
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.8.3 Checklist for preparing a construction programme

Ref	Construction Activity	Check
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 kerbs, 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. Particular attention to be paid to all compaction within 5 metres 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 following suggested output rates will be useful in estimating the resources and time required for each activity.

Table C.8.4 Recommended output rates

Ref	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
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 mm or 900 mm diameter culvert lines (including excavation and backfill)	0.8 - 1.2 linear metre 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: Ethiopian Design Manual, Part D

Productivity depends on a number of factors, including worker nutrition, fitness, experience and motivation, site organisation, 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 may also be less productive. Under all circumstances both the quality and the quantity of worker outputs is highly dependent on the manner in which they are managed and motivated. Attention to working conditions including clarity over what exactly must be achieved in order to complete a task can result in marked increases in individual and team outputs. Attention to health and safety considerations in the distribution of individual activities, and to the quality and condition of availability hand tools can improve productivity by 25% or more.

Table C.8.5 shows 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 obtained from outside the project area. Some skills may be taught through on-the-job training. This will involve costs and a short-term loss of productivity but can give rise to longer term benefits in terms of the local availability of skills for future repair and maintenance. Workers not from the area of the structure site may require temporary accommodation and give rise to costs relating to travel and allowances.

Table C.8.5 Potential skills requirements

Ref	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.8.6 shows a checklist of hand tools and equipment that may be required.

Table C.8.6 Checklist of hand tools and site equipment

Handtools		Equipment
▪ Ranging rods ▪ Spirit level /Abney level ▪ Water tube level ▪ String lines, pegs ▪ Profile boards & travellers ▪ 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 moulds and curing tank ▪ Soil density testing equipment ▪ Sandbags for water control	▪ Culvert moulds ▪ 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/moulds ▪ Traffic signs and barriers ▪ Safety helmets and equipment

8.5 Site works

8.5.1 Introduction

This section considers the 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 they are executed according to the project specification and drawings. The following site works are described:

- Setting out techniques;
- Surface drainage;

- Side drains;
- Mitre drains;
- Scour checks; and
- Culverts and drifts.

8.5.2 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 labour-based, simple setting out techniques mostly used. The example of setting out a right angle is shown in Figure C.8.1.

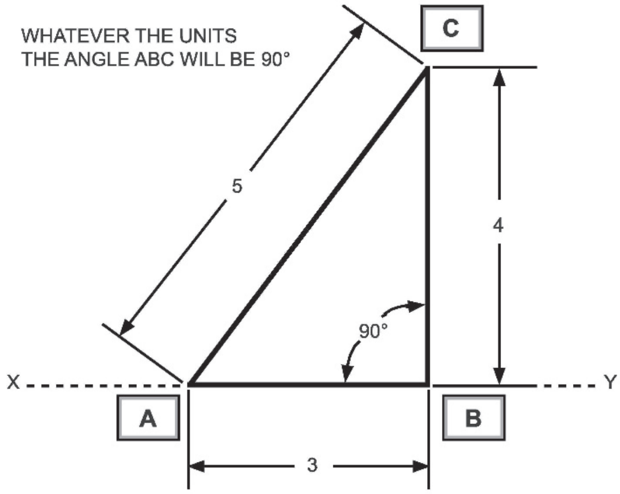


Figure C.8.1 Setting out right angle

8.5.3 Surface drainage

Adequate surface drainage is achieved by providing 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.

Figure C.8.2 Stagnant water due to poor camber

Road camber is the slope from the centre 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 (after compaction) can be formed to a gradient between 5% and 7% and for sealed roads between 2% and 4% gradient. In the cases of a standard camber and of cross-slopes towards hillside (inwards cross-slope), a side drain is required to drain off the surface water at the toe of the hill. However, where the cross-slope is away from the hillside (outwards slope), no side drain is required.

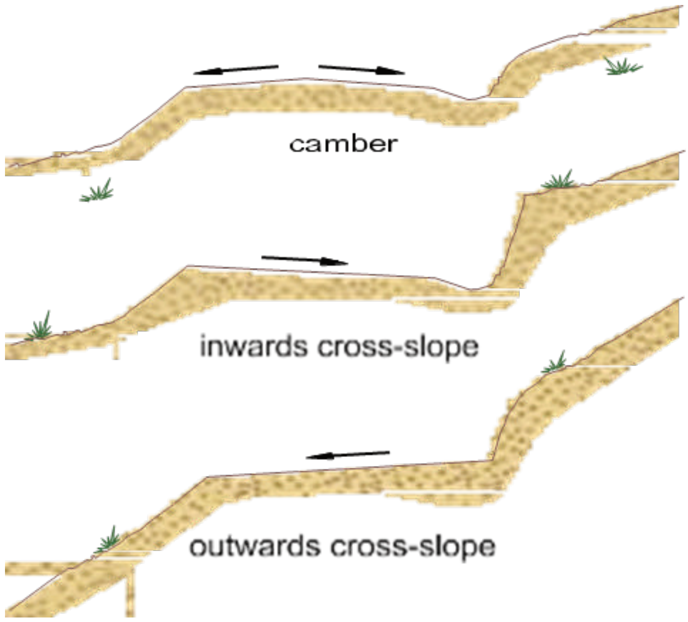


Figure C.8.3 Types of cross-slopes on LVRs

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 centre line of the road during forming of the side drains and spread towards both sides of the road to the shoulders. Sufficient quantity of soil is piled up along the centre and it is then spread loose before compaction to form the camber with the correct slope. The loose soil is spread to a slope between 7% and 10% before compaction. The anticipated slope after compaction will then be between 5% and 7%.

The levels of the shoulders and centreline 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 5-metre centres. The exact level of any point along the road can accurately be determined by stretching ropes between the pegs. The ropes serve as a useful guide for workers carrying out the levelling.

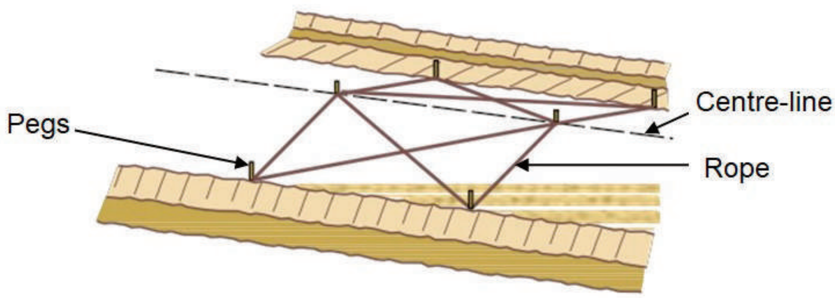


Figure C.8.4 Setting out levels for road camber

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, and specifications. If there are any irregularities in the levels, the soil should be removed and filled in again and compact properly. The checking process is then repeated to ensure works are completed to the require standards and quality.

8.5.4 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 are usually constructed by motor-grader using its blade. It is easily silted up and has a lower carrying capacity compared with 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 labour-based, excavation of ditches is done in two stages. The first stage involves the excavation of a rectangular ditch. This is followed by excavation of the side slopes of the ditch. Excavation work is set out using string line and pegs and controlled through the use of ditch templates. Excavated soil from the ditches is placed in the middle of the road before then being spread to form the camber.

Figure C.8.5 illustrates excavation of the side slope and the associated use of ditch templates.

Figure C.8.5 Checking drain shape with template

8.5.5 Mitre drains

Mitre drains divert water away from the side drains to lower areas to help control siltation and erosion of the ditches.

The angle between the mitre drain and the side drain should not be greater than 45°. 30° is an ideal angle. If it is necessary to take water off at an angle greater than 45°, it should ideally be done in two or more bends so that each bend is less than 45°.

The angle between the mitre drain can be checked by constructing a 90° then using the measurements as shown in the Figure C.8.6.

Construction of mitre drains

Construction of mitre 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 mitre drain marked on the pegs.

Excavated soils from the mitre drains should be placed in such a way that the material is not washed back into the drain after any rains during construction.

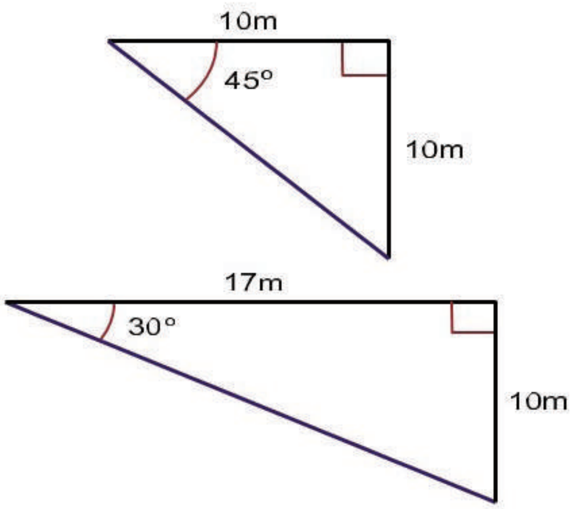


Figure C.8.6 Triangles for setting out mitre drains

8.5.6 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.8.8 to Figure C.8.10 show culvert layouts for different terrain types.

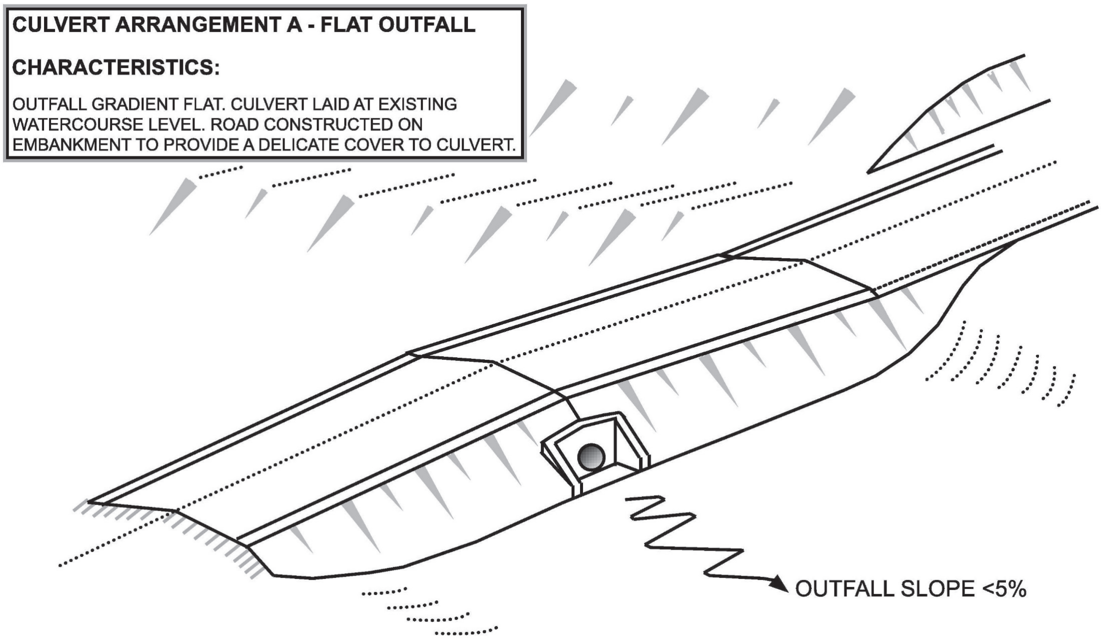


Figure C.8.7 Culvert arrangement A - flat outfall

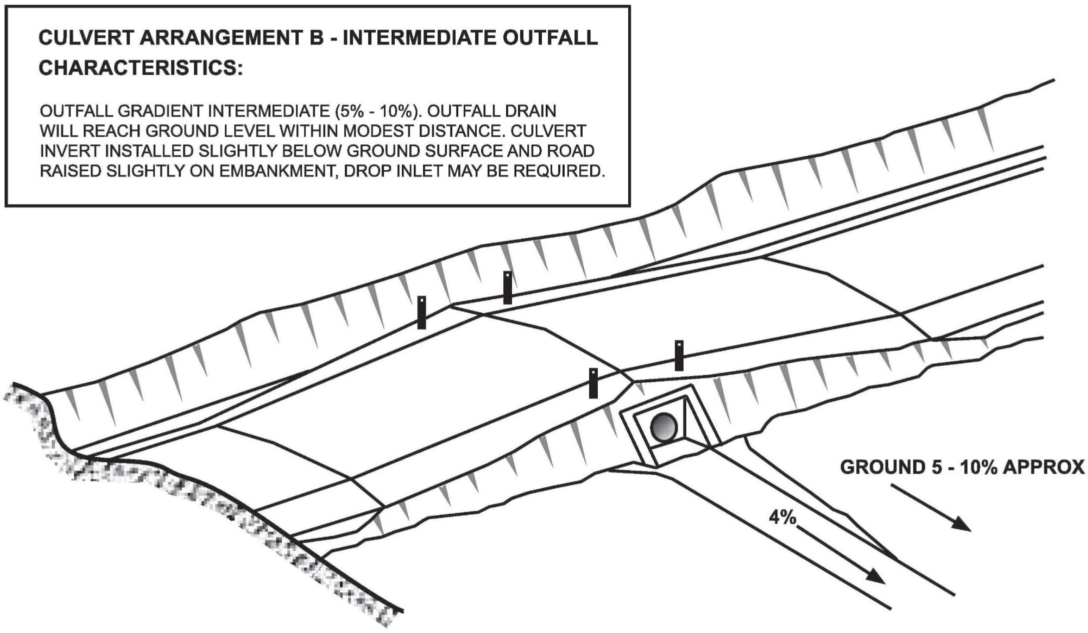


Figure C.8.8 Culvert arrangement B - intermediate outfall

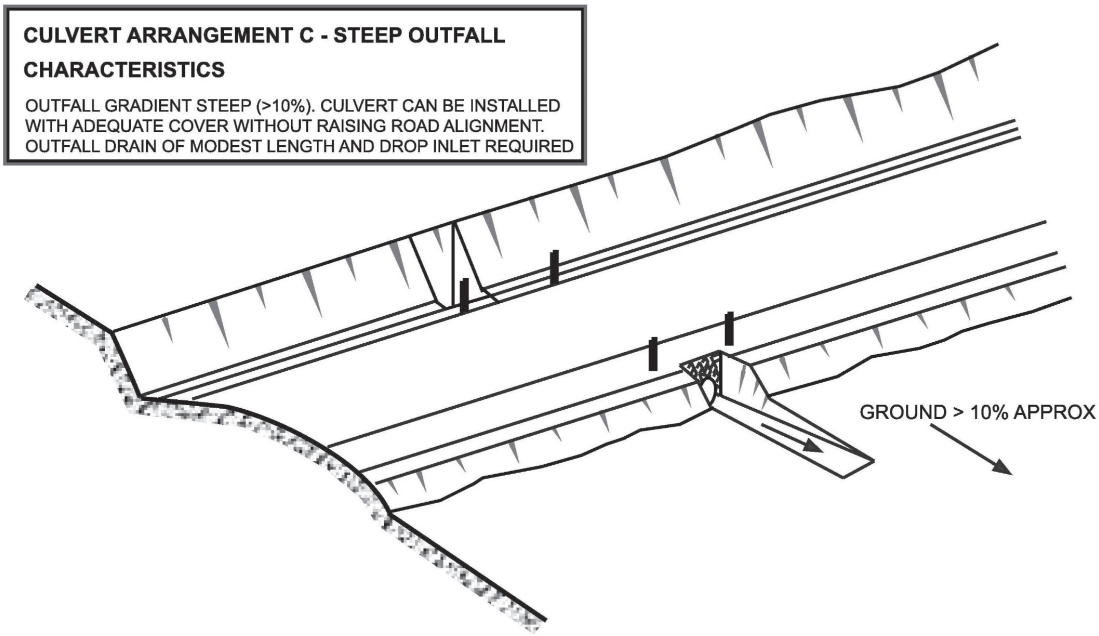


Figure C.8.9 Culvert arrangement C - steep outfall

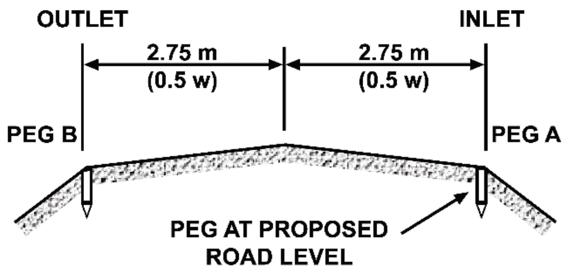
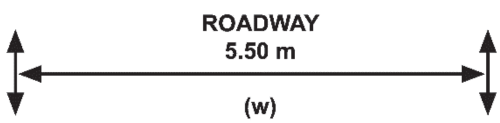
Calculating the required depth of excavation

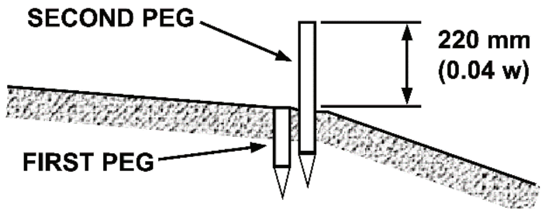
Culverts should be set out according to the design and detailed drawings. The main aspects to be set out accurately are the centre-line of the culvert barrels, the culvert extent and the inlet and outlet levels of the culvert.

If the culvert site is flat, the watercourse gradient must be checked for 20 metres 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 metre fall in 20 metres) 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 metres.

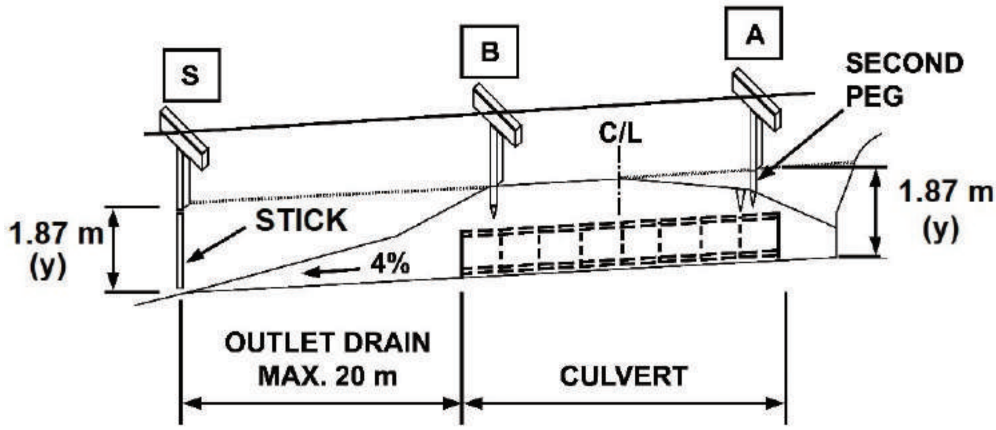
If the conditions for Arrangement A are not in place Arrangement B or C is required. Guidance on calculating the correct depth for installing a 600 mm diameter culvert under Arrangement B or C is given in Table C.8.7. The roadway width is assumed to be 5.5 metres but the dimensions in brackets are for a cross-section with roadway width of ‘w’ metres.

Table C.8.7 Calculating the depth for culvert installation

Procedure step by step	Example and explanation
STEP 1 Fix the centre-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).	
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).	**Outside diam of 900 mm culvert = 1.05 m** **Cover to culvert pipe = 0.60 m** **Total depth, d = 1.65 m**

Procedure step by step	Example and explanation
STEP 4 Calculate the difference in culvert level between Peg A and B with the chosen culvert gradient (4% in this case).	**Difference in level = 4% x 5.50 m = 0.22 m** (for road width w, difference in level = 0.04w)
STEP 5 Calculate the depth to be excavated from the proposed road level to the underside of the culvert at the outlet (Peg B).	**Road width (m)** **5.50 m** w **Inlet depth** **1.65 m** 1.62 m **Difference in level +** **0.22 m** 0.04w **Depth at outlet** **1.87 m** y = (1.62 + 0.04w)
STEP 6 Raise the level of Peg A by the same measurement calculated under Step 4, establishing a second peg (difference in level).	

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).

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 metres apart, or w for other cross sections).
This is Arrangement B.

NOTE:

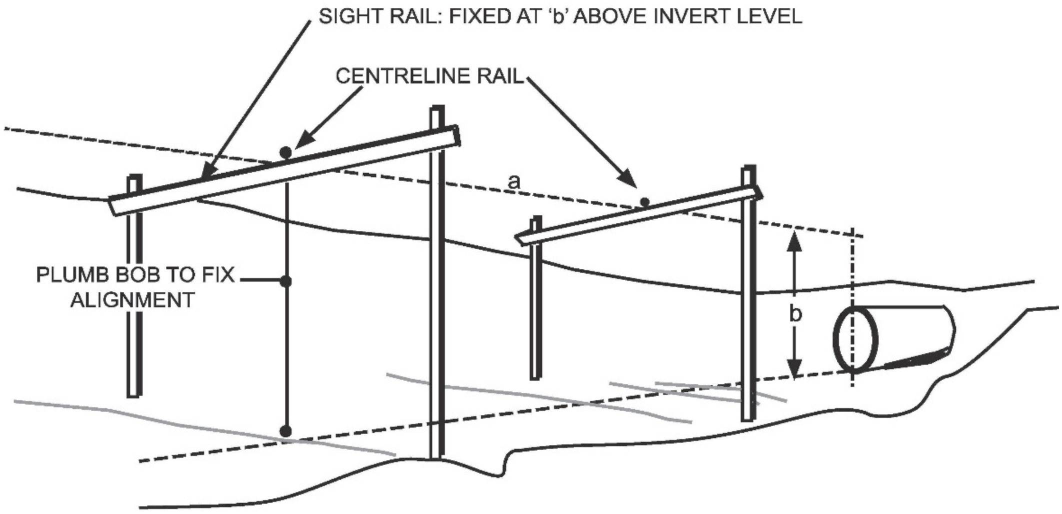
Where pipes will be added on imported material, the excavation levels must be lowered by the thickness of the bedding material.

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.8.11 and Figure C.8.12. 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 metres 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 traveller.

During excavation along the line, from inlet to outlet points, the traveller can be used to control that the correct levels have been achieved. By placing the traveller 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 traveller is below the sight line between the two fixed profile boards, the ditch has been excavated to too low a level. If the traveller 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 metres along the sight line. When sufficient slots have been dug, the workers can start excavating the trench by joining up the excavated slots. The traveller 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.



a = Centreline

b = Traveller or boning rod height above invert

Figure C.8.10 Setting out culvert profile

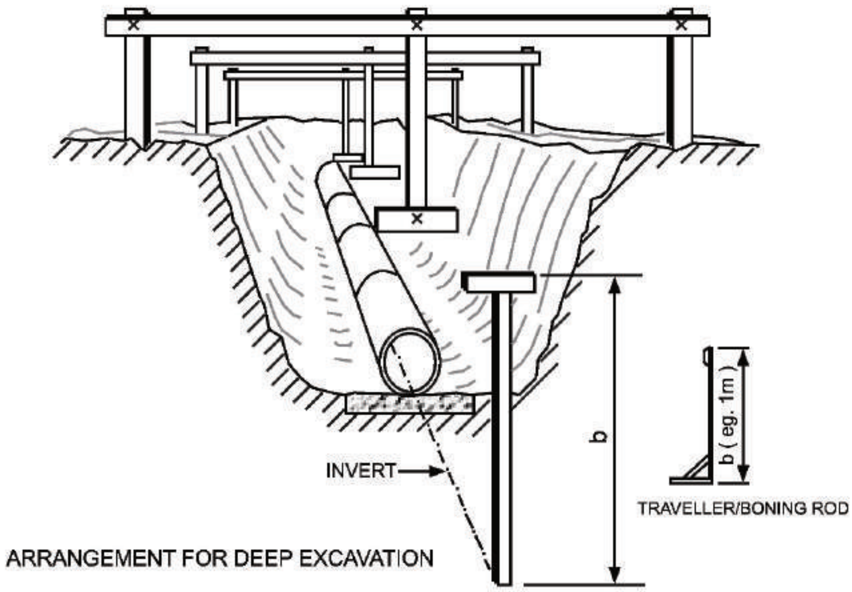


Figure C.8.11 Setting out culvert profile – deep excavation

8.6 Site administration

The activities listed in Table C.8.8 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.8.8 Checklist of site administration tasks

Ref	Task
1	Set and review individual/gang task rates
2	Daily labour and work achievement record for site labour force
3	Daily diary of works achieved; problems encountered and methods of solving them should be recorded
4	Update work programme
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 labour 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 programmes should be prepared based on the Bills of Quantities, the overall works programme and anticipated 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 any deviations 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 that should be recorded.
- 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.

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Appendix C1 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	$(d_c+D)/2m$	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
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)	Col. 11	No. of barrels	Col. 19	Determine using Figures C.4.10
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	Determine using Figures C.4. 6	23	Col. 7 x col. 8
7	Culvert Length (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	Determine from Table 4.2	25	Larger of cols. 16 &24
10	D (pipes) or B x D (rectangular)	18	Determine using Figures C.4.8	26	Outlet vel. If req'd (Subsec. 3.2.3)

WORKED EXAMPLE 2: HAND-CALCULATION OF CULVERTS

Location: KM 1+300 | Prepared by: F.O.O

Sheet No.:001 | CONVENTIONAL CULVERT DESIGN | 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.2 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:

- Unreinforced Concrete Pipe Culverts (Single);
- Unreinforced Concrete Pipe Culverts (Double);
- Unreinforced Concrete Pipe Culverts (Triple);
- Reinforced Concrete Pipe Culvert (Single);
- Reinforced Concrete Pipe Culvert (Double);
- Reinforced Concrete Box Culvert (Single);
- Reinforced Concrete Box Culvert (Double);
- Mass Concrete Relief Culverts; and
- Vented Fords.