FOREWORD

The road network in Ethiopia provides the dominant mode of freight and passenger transport and thus plays a vital role in the economy of the country. The network comprises a huge national asset that requires adherence to appropriate standards for design, construction and maintenance in order to provide a high level of service. As the length of the road network is increasing, appropriate choice of methods to preserve this investment becomes increasingly important.

In 2002, the Ethiopian Roads Authority (ERA) first brought out road design manuals to provide a standardized approach for the design, construction and maintenance of roads in the country. Due to technological development and change, these manuals require periodic updating. This current version of the manual has particular reference to the prevailing conditions in Ethiopia and reflects the experience gained through activities within the road sector during the last 10 years. Completion of the review and updating of the manuals was undertaken in close consultation with the federal and regional roads authorities and the stakeholders in the road sector including contracting and consulting industry.

Most importantly, in supporting the preparation of the documents, a series of thematic peer review panels were established that comprised local experts from the public and private sector who provided guidance and review for the project team.

This Manual supersedes the Drainage Design Manual part of the ERA 2002 series of Manuals. The standards set out shall be adhered to unless otherwise directed by ERA. However, I should emphasize that careful consideration to sound engineering practice shall be observed in the use of the manual, and under no circumstances shall the manual waive professional judgment in applied engineering. For simplification in reference this manual may be cited as ERA's *Drainage Design Manual - 2013.*

On behalf of the Ethiopian Roads Authority I would like to thank DFID, Crown Agents and the AFCAP team for their cooperation, contribution and support in the development of the manual and supporting documents for Ethiopia. I would also like to extend my gratitude and appreciation to all of the industry stakeholders and participants who contributed their time, knowledge and effort during the development of the documents. Special thanks are extended to the members of the various Peer Review Panels, whose active support and involvement guided the authors of the manual and the process.

It is my sincere hope that this manual will provide all users with a standard reference and a ready source of good practice for the geometric design of roads, and will assist in a cost effective operation, and environmentally sustainable development of our road network.

I look forward to the practices contained in this manual being quickly adopted into our operations, thereby making a sustainable contribution to the improved infrastructure of our country.

Comments and suggestions on all aspects from any concerned body, group or individual as feedback during its implementation is expected and will be highly appreciated.

Addis Ababa, 2013

Zaid Wolde Gebriel

Director General, Ethiopian Roads Authority

PREFACE

The Ethiopian Roads Authority is the custodian of the series of technical manuals, standard specifications and bidding documents that are written for the practicing engineer in Ethiopia. The series describes current and recommended practice and sets out the national standards for roads and bridges. The documents are based on national experience and international practice and are approved by the Director General of the Ethiopian Roads Authority.

The *Drainage Design Manual – 2013* forms part of the Ethiopian Roads Authority series of Road and Bridge Design documents. The complete series of documents, covering all roads and bridges in Ethiopia, is as follows:

1. Geometric Design Manual
2. Site Investigation Manual
3. Geotechnical Design Manual
4. Route Selection Manual
5. Pavement Design Manual Volume I Flexible Pavements
6. Pavement Design Manual Volume II Rigid Pavements
7. Pavement Rehabilitation and Asphalt Overlay Design Manual
8. Drainage Design Manual
9. Bridge Design Manual
10. Low Volume Roads Design Manual
11. Standard Environmental Procedures Manual
12. Standard Technical Specifications
13. Standard Detailed Drawings.
14. Best Practice Manual for Thin Bituminous Surfacings
15. Standard Bidding Documents for Road Work Contracts – A series of Bidding Documents covering the full range of projects from large scale works unlimited in value to minor works with an upper threshold of $300,000. The higher level documents have both Local Competitive Bidding and International Competitive Bidding versions.

These documents are available to registered users through the ERA website: www.era.gov.et

Manual Updates

Significant changes to criteria, procedures or any other relevant issues related to new policies or revised laws of the land or that are mandated by the relevant Federal Government Ministry or Agency should be incorporated into the manual from their date of effectiveness.

Other minor changes that will not significantly affect the whole nature of the manual may be accumulated and made periodically. When changes are made and approved, new page(s) incorporating the revision, together with the revision date, will be issued and inserted into the relevant chapter.

All suggestions to improve the manual should be made in accordance with the following procedures:

1. Users of the manual must register on the ERA website: www.era.gov.et
2. Proposed changes should be outlined on the Manual Change Form and forwarded with a covering letter of its need and purpose to the Director General of the Ethiopian Roads Authority.
3. Agreed changes will be approved by the Director General of the Ethiopian Roads Authority on recommendation from the Deputy Director General (Engineering Operations).
4. The release date will be notified to all registered users and authorities.

Addis Ababa, 2013

Zaid Wolde Gebriel

Director General, Ethiopian Roads Authority

ETHIOPIAN ROADS AUTHORITY
CHANGE CONTROL DESIGN MANUAL

MANUAL CHANGE	This area to be completed by the ERA Director of Quality Assurance
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This *Drainage Design Manual - 2013* is based on ERA's *Drainage Design Manual – 2002* but includes improvements resulting from recent research and extensions to deal with topics that were not included in the earlier manual.

From the outset, the approach to the development of the manual was to include all sectors and stakeholders in Ethiopia. The input from the international team of experts was supplemented by our own extensive local experience and expertise. Local knowledge and experience was shared through review workshops to discuss and debate the contents of the draft manual. ERA wishes to thank all the individuals who gave their time to attend the workshops and provide valuable inputs to the compilation of the manual.

In addition to the workshops, Peer Groups comprising specialists drawn from within the local industry were established to provide advice and comments in their respective areas of expertise. The contribution of the Peer Group participants is gratefully acknowledged.

Finally, ERA would like to thank Crown Agents for their overall management of the project.

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GLOSSARY OF TERMS

ADT	The total traffic volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.
ADTT	The total yearly traffic volume in both directions divided by the number of days in the year.
Absorption	The act or process of taking in water by inflow of atmospheric vapor, hydroscopic absorption, wetting, infiltration, influent seepage, and gravity flow of streams into sinkholes or other large openings.
Abstraction	That portion of rainfall which does not become runoff. It includes interception, infiltration, and storage in depression. It is affected by land use, land treatment and condition, and antecedent soil moisture.
Abutment	The support at either end of a bridge, usually classified as spill-through or vertical.
Accretion	1. The process of accumulation of silt, sand, or pebbles by flowing water; may be due to any cause and includes alluviation. 2. Gradual building up of a beach by wave action. 3. Gradual building of the channel bottom, bank, or bar due to silting or wave action.
Aggradation	General and progressive building up of the longitudinal profile of a channel by deposit of sediment.
Allowable Headwater	The depth or elevation of impounded water at the entrance to a hydraulic structure after which flooding or some other unfavorable result could occur.
Alluvial Channel	A channel wholly in alluvium, no bedrock exposed in channel at low flow or likely to be exposed by erosion during major flow.
Alluvium	Unconsolidated clay, silt, sand, or gravel deposited by a stream in a channel, flood plain, fan, or delta.
Anabranched Stream	A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars. The width of individual islands or bars is greater than three times the water width.
Annual Flood	The highest peak discharge in a water year.
Annual Series	A frequency series in which only the largest value in each year is used, such as annual floods.
Antecedent Moisture Condition (AMC)	The degree of wetness of a watershed at the beginning of a storm.

Area Rainfall	The average rainfall over an area, usually as derived from or discussed in contrast with, point rainfall.
Armor	Artificial surfacing of channel beds, banks, or embankment slopes to resist scour and lateral erosion.
Armoring	The concentration of a layer of stones on the bed of the stream that are of a size larger than the transport capability of the recently experienced flow.
Avulsion	A sudden change in the course of a channel, usually by breaching of the banks during a flood.
Aquifer	A porous, water-bearing geologic formation. Generally restricted to materials capable of yielding an appreciable supply of water.
Artesian	Pertains to groundwater that is under pressure and will rise to a higher elevation if given an opportunity to do so.
B	Barrel width, distance measured in meters.
Backwater	The increase in water-surface profile, relative to the elevation occurring under natural channel and flood-plain conditions, induced upstream from a structure, bridge, or culvert that obstructs or constricts a channel. It also applies to the water surface profile in a channel or conduit.
Baffle	A structure built on the bed of a stream to deflect or disturb the flow. Also a device used in a culvert to facilitate fish passage.
Bank	Lateral boundaries of a channel or stream, as indicated by a scarp, or on the inside of bends, by the stream ward edge of permanent vegetal growth.
Bar	An elongated deposit of alluvium, not permanently vegetated, within or along the side of a channel.
Base Flood	The 100-year flood.
Base Flow	Stream discharge derived from groundwater sources. Sometimes considered to include flows from regulated lakes or reservoirs. Fluctuates much less than storm runoff.
Basin, Drainage	The area of land drained by a watercourse.
Basin Lag	The amount of time from the centroid of the rainfall hyetograph to the hydrograph peak.
Bed (of a channel or stream)	The part of a channel not permanently vegetated or bounded by banks, over which water normally flows.

Bed Load	Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer.
Bed Material	Sediment consisting of particle sizes large enough to be found in appreciable quantities at the surface of a streambed.
Bed Shear (Tractive Force)	The force per unit area exerted by a fluid flowing past a stationary boundary
Berm	A narrow shelf or ledge; also a form of dike.
Braided Stream	A stream whose surface is divided at normal stage by small mid-channel bars or small islands. The individual width of bars and islands is less than three times the water width. A single large channel that has subordinate channels.
Bridge	A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, having a tract or passageway for carrying traffic or moving loads, and having an opening measured along the center of the roadway of more than six meters between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. May also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening. Also, a structure designed hydraulically using the principles of open channel flow to operate with a free water surface, but may be inundated under flood conditions.
Breakers	The surface discontinuities of waves as they break-up. They may take different shapes (spilling, plunging, surging). Zone of break-up is called surf zone.
Bridge Opening	The cross-sectional area beneath a bridge that is available for conveyance of water.
Bridge Waterway	The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.
Broken-Back Culvert	A culvert comprising two or more longitudinal structure profiles. Such culverts are sometimes effective in reducing outflow velocities by the energy dissipation of a hydraulic jump.
By-Pass	Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downstream. Also called carryover.
Capacity	A measure of the ability of a channel or conduit to convey water.
Catch Basin	A structure with a sump for inletting drainage from a gutter or median and discharging the water through a conduit. In common usage it is a grated inlet with or without a sump.

Catchment	The watershed (implying all physical characteristics).
Catchment Area	The area tributary to a lake, stream, or drainage system.
Channel	(1) The bed and banks that confine the surface flow of a natural or artificial stream. Braided streams have multiple subordinate channels that are within the main stream channel. Anabranched streams have more than one channel. (2) The course where a stream of water runs or the closed course or conduit through which water runs, such as a pipe.
Channel Lining	The material applied to the bottom and/or sides of a natural or manmade channel. Material may be concrete, sod, grass, rock, or any of several other types.
Channel Routing	The process whereby a peak flow and/or its associated stream flow hydrograph is mathematically transposed to another site downstream.
Check Dam	A low structure, dam, or weir across a channel for the control of water stage, velocity, or to control channel erosion.
Check Flow	A flow, larger or smaller than the design flow that is used to assess the performance of the facility.
Chute	Chutes are steep (greater than 15%) natural or man-made open channels used to convey water. They may be closed and usually require energy dissipation at their termini.
Coastal Zone	The strip of land that extends inland to the first major change in terrain (lake shore features).
Coefficient of Discharge	The coefficient used for orifice flow processes.
Combination inlet	Drainage inlet usually composed of two or more inlet types, e.g., curb opening and a grate inlet.
Conduit	An artificial or natural channel, usually a closed structure such as a pipe.
Conjugate Depth	The alternate depth of flow involved with the hydraulic jump.
Continuity Equation	Discharge equals velocity times cross-sectional area ($Q = V \times A$).
Control Section	A cross section, such as a bridge crossing, reach of channel, or dam, with limited flow capacity, and where the discharge is related to the upstream water-surface elevation.

Contraction	The effects of a channel constriction on flow. The response of a river to the change in its bed load requirement as a result of a contraction of flow. The flow contraction is due to an encroachment of either the main channel or the flood plain by a natural constriction or the highway embankment.
Conveyance	A measure, K, of the ability of a stream, channel, or conduit to convey water. In Manning's formula $K = (1/n)AR^{2/3}$ (SI units).
Corrosion	The deterioration of pipe or structure by chemical action.
Cover	The extent of soil above the crown of a pipe or culvert. The vegetation or vegetational debris, such as mulch, that exists on the soil surface. In some classification schemes fallow or bare soil is taken as the minimum cover class.
Criterion	A standard, rule, or test on which a judgment is based.
Critical Depth	The depth at which water flows over a weir; this depth being attained automatically where no backwater forces are involved. It is the depth at which the energy content of flow is a minimum.
Cross Drainage	The runoff from contributing drainage areas both inside and outside the highway right-of-way and the transmission thereof from the upstream side of the highway facility to the downstream side.
Cross-Section	The shape of a channel, stream, or valley viewed across its axis. In watershed investigations it is determined by a line approximately perpendicular to the main path of water flow, along which measurements of distance and elevation are taken to define the cross-sectional area.
Culvert	A structure that is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity. A structure used to convey surface runoff through embankments. A structure, as distinguished from bridges, that is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. Also, a structure which is six meters or less in centerline length between extreme ends of openings for multiple boxes.
Curb-Opening Inlet	Drainage inlet consisting of an opening in the roadway curb.
Cumulative Conveyance	A tabulation or graphical plot of the accumulated measures of conveyance; proceeding from one stream bank to the other.
Cutoff Wall	A wall that extends from the end of a structure to below the expected scour depth or scour-resistant material.

D	Culvert diameter or barrel depth.
D_{50}	Median size of rip rap. The particle diameter at the 50^{th} percentile point on a size weight distribution curve.
D_{15}	The particle diameter at the 15^{th} percentile point on a size weight distribution curve.
D_{85}	The particle diameter at the 85^{th} percentile point on a size weight distribution curve.
d_c	Critical depth of flow in meters.
Debris	Material transported by the stream, either floating or submerged, such as logs or brush.
Degradation	General and progressive lowering of the longitudinal profile of a channel by erosion.
Deposition	The settling of material from the stream flow onto the bottom.
Depression Storage	Rainfall that is temporarily stored in depressions within a watershed.
Depth-Area Curve	A graph showing the change in average rainfall depth as size of area changes.
Design Discharge Or Flow	The rate of flow for which a facility is designed and thus expected to accommodate without exceeding the adopted design constraints.
Design Flood Frequency	The recurrence interval that is expected to be accommodated without contravention of the adopted design constraints. The return interval (recurrence interval or reciprocal of probability) used as a basis for the design discharge.
Design Highwater Elevation	The maximum water level that a bridge opening is designed to accommodate without contravention of the adopted design constraints. The usual term used to describe the estimated water surface elevation in the stream at the project site for the design discharge.
Design Flood	A flood that does not overtop the roadway.
Design Flow	See Design Discharge
Design Storm	A given rainfall amount, areal distribution, and time distribution used to estimate runoff. The rainfall amount is either a given frequency (25-year, 50-year, etc.) or a specific large value.
Detention Basin	A basin or reservoir incorporated into the watershed whereby runoff is temporarily stored, thus attenuating the peak of the runoff hydrograph.

Detour	A temporary change in the roadway alignment. It may be localized at a structure or may be along an alternate route.
Dike	An impermeable linear structure for the control or confinement of overbank flow. River training structure used for bank protection.
Direct Runoff	The water that enters the stream channels during a storm or soon after forming a runoff hydrograph. May consist of rainfall on the stream surface, surface runoff, and seepage of infiltrated water (rapid subsurface flow).
Discharge	The rate of the volume of flow of a stream per unit of time, usually expressed in m^3/s.
Drainage Area	The area draining into a stream at a given point. The area may be of different sizes for surface runoff, subsurface flow, and base flow, but generally the surface flow area is used as the drainage area.
Drift	Debris that drifts on or near the water surface.
Drop Inlet	Drainage inlet with a horizontal or nearly horizontal opening.
Effective Duration	The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rainfall.
Effective Particle Size	The diameter of particles, spherical in shape, equal in size and arranged in a given manner, of a hypothetical sample of granular material that would have the same transmission constant as the actual material under consideration.
Emergency Spillway	A rock or vegetated earth waterway around a dam, built with its crest above the normally used principal spillway. Used to supplement the principal spillway in conveying extreme amounts of runoff safely past the dam.
End Section	A concrete or metal structure attached to the end of a culvert for purposes of retaining the embankment from spilling into the waterway, appearance, anchorage, etc.
Energy Dissipation	The phenomenon whereby energy is dissipated or used up.
Energy Grade Line	A line joining the elevation of energy heads; a line drawn above the hydraulic grade line a distance equivalent to the velocity head of the flowing water at each section along a stream, channel, or conduit.
Energy Gradient	Slope of the line joining the elevations of total energy along a conduit of flowing water.
Ephemeral Stream	A stream or reach of a stream that does not flow continuously for most of the year.

Equalizer	A culvert or opening placed where it is desirable to equalize the water head on both sides of the embankment.
Equivalent Cross-Slope	An imaginary straight cross-slope having conveyance capacity equal to that of the given compound cross-slope.
Erosion	The wearing away or scouring of material in a channel, opening, or outlet works caused by flowing water.
Evapotranspira-tion	Plant transpiration plus evaporation from the soil. Difficult to determine separately, therefore used as a unit for study.
Excess Rainfall	Direct runoff.
Exfiltration	The process where stormwater leaks or flows to the surrounding soil through openings in a conduit.
Fetch	The distance the wind blows over water in generating waves.
Filter	A device or structure for removing solid or colloidal material from stormwater or preventing migration of fine-grained soil particles as water passes through soil. The water is passed through a filtering medium; usually a granular material or finely woven or non-woven cloth.
Filtration	The process of passing water through a filtering medium consisting of either granular material of filter cloth for the removal of suspended or colloidal matter.
Flanking Inlets	Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets is to intercept debris as the slope decreases and act as relief to the inlet at the low point.
Flared Inlet	A specially fabricated pipe appurtenance or a special feature of box culverts. This type of inlet is effective in reducing the calculated headwater.
Flared Wingwalls	The part of a culvert headwall that serves as a retaining wall for the highway embankment. The walls form an angle to the centerline of the culvert.
Flood	In common usage, an event that overflows the normal banks. In technical usage, it refers to a given discharge based, typically, on a statistical analysis of an annual series of events.
Flood Frequency	The average time interval, in years, in which a given storm or amount of water in a stream will be exceeded.
Flood of Record	Reference to the maximum estimated or measured discharge that has occurred at a site.

Floodplain	The alluvial land bordering a stream, formed by stream processes, that is subject to inundation by floods.
Flood Pool	Floodwater storage elevation in a reservoir. In a floodwater retarding reservoir, the temporary storage between the crests of the principal and emergency spillways.
Flood Routing	Determining the changes in a flood hydrograph as it moves downstream through a channel or through a reservoir (called reservoir routing). Graphic or numerical methods are used.
Floodwater Retarding Structure	A dam, usually with an earthfill, having a flood pool where incoming floodwater is temporarily stored and slowly released downstream through a principal spillway. The reservoir contains a sediment pool and sometimes storage for irrigation or other purposes.
Flow-Control Structure	A structure, either within or outside a channel, which acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.
Flow Concentration	A preponderance of the streamflow.
Flow Distribution	The estimated or measured spatial distribution of the total streamflow.
Flume	An open or closed channel used to convey water.
Ford	A location where a highway crosses a river or wash and allowing flow over the highway. Often with cut-off walls and markers.
Freeboard	The vertical distance between the level of the water surface, usually corresponding to design flow and a point of interest such as a low chord of a bridge beam or specific location on the roadway grade.
Free Outlet	Those outlets whose tailwater is equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.
Frequency	In analysis of hydrologic data, the recurrence interval is simply called frequency.
Froude Number	A dimensionless number that represents the ratio of inertial forces to gravitational forces. High froude numbers are indicative of high flow velocity and high potential for scour.
Frontal Flow	The portion of flow which passes over the upstream side of a grate.
Functional Values	Characteristics of surface water and wetlands. These include terrestrial and aquatic wildlife habitat, flood control, groundwater recharge, aesthetics, shore and bank line geometry, and water quality.

G	The acceleration of gravity, 9.81m/s^2.
Gabion	A rectangular basket made of steel wire fabric or mesh that is filled with rock of suitable size. Used to construct flow-control structures, bank protection, groins, and jetties.
General Scour	Scour involving the removal of material from the bed and banks across or most of the width of a channel and is not localized at an element such as a pier, abutment, or other obstruction to flow. Termed contraction scour.
Graded Filter	An aggregate filter that is proportioned by particle size to allow water to pass through at a specified rate while preventing migration of fine-grained soil particles without clogging.
Grate Inlet	Drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale, or ditch.
Groin	A structure in the form of a barrier, placed oblique to the primary motion of water, designed to control movement of bed load. Groins are usually solid, but may be constructed with openings to control elevations of sediments.
Groundwater	Subsurface water occupying the saturation zone, that feeds wells and springs, or a source of base flow in streams. In a strict sense, the term applies only to water below the water table. Also called phreatic water.
Guide Banks	Embankments built upstream from one or both abutments of a bridge to guide the approaching flow through the waterway opening.
Gutter	That portion of the roadway section adjacent to the curb that is used to convey storm runoff water.
H	Total energy head loss, measured in meters.
H_E	Entrance head loss, measured in meters.
Head	The height of water above any datum.
Head Cutting	Channel degradation associated with abrupt changes in the bed elevation (head-cut) that migrates in an upstream direction.
Headloss	A loss of energy in a hydraulic system.
Headwall	The structural appurtenance usually applied to the end of a culvert to control an adjacent highway embankment and protect the culvert end.
Headwater, H_w	That depth of water impounded upstream of a culvert due to the influence of the culvert constriction, friction, and configuration.

H_f	The friction headloss, measured in meters.
Highwater Elevation	The water surface elevation that results from the passage of flow. It may be "observed highwater elevation" as a result of an event, or "calculated highwater elevation" as part of a design process.
Historical flood	A past flood event of known or estimated magnitude.
H_c	The height of the hydraulic grade line above the outlet invert, in meters.
Hydraulic Grade Line	A profile of the piezometric level to which the water would rise in piezometer tubes along a pipe run. In open channel flow, it is the water surface.
Hydraulic Gradient	The slope of the hydraulic grade line.
Hydraulic Head	The height of the free surface of a body of water above a given point.
Hydraulic Jump	A hydraulic phenomenon, in open channel flow, where supercritical flow is converted to subcritical flow. This can result in an abrupt rise in the water surface.
Hydraulic Radius	A measure of the boundary resistance to flow, computed as the quotient of cross-sectional area of flow divided by the wetted perimeter. For wide shallow flow, the hydraulic radius can be approximated by the average depth.
Hydraulic Roughness	A composite of the physical characteristics that influence the flow of water across the earth's surface whether natural or channelized. It affects both the time response of a watershed and drainage channel, as well as the channel storage characteristics.
Hydraulics	The characteristics of fluid mechanics involved with the flow of water in or through drainage facilities.
Hydrograph	A graph showing, for a given point on a stream or for a given point in any drainage system, the discharge, stage, velocity, or other property of water with respect to time.
Hydrologic Soil-Cover Complex	A combination of a hydrologic soil group and a type of cover.
Hydrologic Soil Group	A group of soils having the same runoff potential under similar storm and cover conditions.
Hydrology	The study of the occurrence, circulation, distribution, and properties of the waters of the earth and its atmosphere.

Hyetograph	A graphical representation of average rainfall, rainfall-excess rates, or volumes over specified areas during successive units of time during a storm.
Impermeable Strata	A stratum with a texture that water cannot move through perceptibly under pressure ordinarily found in subsurface water.
Impervious	Impermeable to the movement of water.
Improved Inlet	Flared, depressed, or tapered culvert inlets that decrease the amount of energy needed to pass the flow through the inlet and thus increase the capacity of culverts.
Infiltration	That part of rainfall that enters the soil. The passage of water through the soil surface into the ground. Used interchangeably herein with percolation.
Infiltration Rate	The rate at which water enters the soil under a given condition. The rate is usually expressed in centimeters per hour or day, or cubic meters per second.
Inflow	The rate of discharge arriving at a point (in a stream, structure, or reservoir).
Initial Abstraction (Ia)	When considering surface runoff, la is all the rainfall before runoff begins. When considering direct runoff, la consists of interception, evaporation, and the soil-water storage that must be exhausted before direct runoff may begin. Sometimes called 'initial loss."
Inlet	A structure for capturing concentrated surface flow. May be located along the roadway, in a gutter, in the highway median, or in a field.
Inlet Efficiency	The ratio of flow intercepted by an inlet to the total flow.
Inlet Time	The time required for stormwater to flow from the most distant point in a drainage area to the point at which it enters a storm drain.
Intensity	The rate of rainfall upon a watershed, usually expressed in centimeters per hour.
Interception	Precipitation retained on plant or plant residue surfaces and finally absorbed, evaporated, or sublimated. That which flows down the plant to the ground is called "streamflow" and not counted as true interception.
Invert	The flow line in a channel cross-section, pipe, or culvert.
Inverted Siphon	A structure used to convey water under a road using pressure flow. The hydraulic grade line is above the crown of the structure.
Isohyet	A line on a map, connecting points of equal rainfall amounts.

Jetty	An elongated obstruction projecting into a stream to control shoaling and scour by deflection of currents and waves. They may be permeable or impermeable.
Lag Time, TL	The differences in time between the centroid of the excess rainfall (that rainfall producing runoff) and the peak of the runoff hydrograph. Often estimated as 60 percent of the time of concentration ($T_L = 0.6T_c$)
Land Use	A land classification. Cover, such as row crops or pasture, indicates a kind of land use; roads may also be classified as a separate land use.
Levee	A linear embankment outside a channel for containment of flow.
Local Scour	Scour in a channel or on a flood plain that is localized at a pier, abutment, or other obstruction to flow. The scour is caused by the acceleration of the flow and the development of a vortex system induced by the obstruction to the flow.
Manhole	A structure used to access a drainage system.
Manning's "n"	A coefficient of roughness, used in a formula for estimating the capacity of a channel to convey water. Generally, "n" values are determined by inspection of the channel.
Mass Inflow Curve	A graph showing the total cumulative volume of stormwater runoff plotted against time for a given drainage area.
Maximum Probable Flood	The maximum probable flood is the greatest flood that may reasonably be expected, taking into collective account the most adverse flood related conditions based on geographic location, meteorology, and terrain.
Mean Daily Discharge	The average of mean discharge of a stream for one day, usually given in m^3/s.
Meanders	The changes in direction and winding of flow that are sinuous in character.
Migration, Channel	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
Natural Scour	Scour that occurs along a channel reach due to an unstable stream, no exterior causes.
Normal Stage	The water stage prevailing during the greater part of the years.
One-Dimensional Water Surface Profile	An estimated water surface profile that accommodates flow only in the upstream-downstream direction

Ordinary High Water	The line on the shore established by the fluctuations of water and indicated by physical characteristics such as clear, natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of liter and debris, or other appropriate means that consider the characteristics of the surrounding areas.
Outfall	The point location or structure where drainage discharges from a channel, conduit, or drain.
Overland Flow	Runoff that makes its way to the watershed outlet without concentrating in gullies and streams (often in the form of sheet flow).
Partial-Duration Series	A list of all events, such as floods, occurring above a selected base, without regard to the number, within a given period. In the case of floods, the selected base is usually equal to the smallest annual flood, in order to include at least one flood in each year.
Peak Discharge	Maximum discharge rate on a runoff hydrograph.
Percolation	The movement or flow of water through the interstices or the pores of a soil or other porous medium. Used interchangeably herein with infiltration.
Permeability	The property of a material that permits appreciable movement of water through it when it is saturated and movement is actuated by hydrostatic pressure of the magnitude normally encountered in natural subsurface water.
Perennial Stream	A stream or reach of a stream that flows continuously for all or most of the year.
Pervious Soil	Soil containing voids through which water will move under hydrostatic pressure.
pH	The reciprocal of the logarithm of the Hydrogen ion concentration. The concentration is the weight of hydrogen ions, in grams, per liter of solution. Neutral water has a pH value of 7.
Point Rainfall	Rainfall at a single rain gauge.
Precipitation	The process by which water in liquid or solid state falls from the atmosphere.
Principal Spillway	Conveys all ordinary discharges coming into a reservoir and all of an extreme discharge that does not pass through the emergency spillway.
RFCS	Road Functional Classification System, indicates planned class of road for Ethiopia.

Rainfall Excess	The water available to runoff after interception, depression storage, and infiltration have been satisfied.
Rainfall Intensity	Amount of rainfall occurring in a unit of time, converted to its equivalent in centimeters per hour at the same rate.
Rating Curve	A graphical plot relating stage to discharge.
Reach	A length of stream or valley, selected for purpose of study.
Recession Curve	The receding portion of a hydrograph, occurring after excess rainfall has stopped.
Recharge	Addition of water to the zone of saturation from precipitation or infiltration.
Recharge Basin	A basin excavated in the earth to receive the discharge from streams or storm drains for the purpose of replenishing groundwater supply.
Regional Analysis	A regional study of gauged watersheds that produce regression equations relating various watershed and climatological parameters to discharge. Use for design of ungauged watershed with similar characteristics.
Reservoir Routing	Flood routing of a hydrograph through a reservoir.
Retard	A structure designed to decrease velocity and induce silting or accretion. Retard type structures are permeable structures customarily constructed at and parallel to the toe of slope.
Retention Basin	A basin or reservoir where water is stored for regulating a flood, that does not have an uncontrolled outlet. The stored water is disposed through infiltration, injection (or dry) wells, or by release to the downstream drainage system after the storm event. The release may be through a gate-controlled gravity system or by pumping.
Revetment	A rigid or flexible armor placed on a bank or embankment as protection against scour and lateral erosion.
Riprap	Stones placed in a loose assemblage along the banks and bed of a channel to inhibit erosion and scour.
Roadway Cross-Slopes	Transverse slopes and/or superelevation described by the roadway section geometry. Usually provided to facilitate drainage and/or resist centrifugal force.
Roughness	The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the "n-value" coefficient used in Manning's channel flow equation.

Runoff	That part of the precipitation that runs off the surface of a drainage area after all abstractions are accounted for.
Runoff Coefficient	A factor representing the portion of runoff resulting from a unit rainfall. Dependent on terrain and topography.
Saturated Soil	Soil that has its interstices or void spaces filled with water to the point at which runoff occurs.
Scour	The result of the erosive action of running water, excavating and carrying away material from the bed and banks of streams.
Scupper	A vertical hole through a bridge deck for the purpose of deck drainage, sometimes a horizontal opening in the curb or barrier.
Sediment Pool	Reservoir storage provided for sediment, prolonging the usefulness of floodwater or irrigation pools.
Sedimentation	The deposition of soil particles that have been carried by flood waters.
Sedimentation Basin	A basin or tank in which stormwater containing settleable solids is retained for removal by gravity or filtration of a part of the suspended matter.
Skew	A measure of the angle of intersection between a line normal to the roadway centerline and the direction of the streamflow at flood stage on the lineal direction of the main channel.
Skewness	When data are plotted in a curve on log-normal paper, the curvature is skewness.
Slotted Drain Inlets	Drainage inlets composed of a continuous slot built into the top of a pipe which serves to intercept, collect, and transport the flow
Soffit	The inside top of the culvert or storm drain pipe.
Soil Porosity	The percentage of the soil (or rock) volume that is not occupied by solid particles, including all pore space filled with air and water.
Soil-Water-Storage	The amount of water the soils (including geologic formations) of a watershed will store at a given time. Amounts vary from watershed to watershed. The amount for a given watershed is continually varying as rainfall or evapotranspiration takes place
Splash-Over	That portion of frontal flow at a grate that splashes over the grate and is not intercepted.
Spread	The accumulated flow in and next to the roadway gutter. This water often represents an interruption to traffic flow during rainstorms. The lateral distance, in feet, of roadway ponding from the curb.

Spur	A structure, permeable or impermeable, projecting into a channel from the bank for the purpose of altering flow direction, inducing deposition or reducing flow velocity along the bank.
Spur Dike	A dike placed at an angle to the roadway for the purpose of shifting the erosion characteristics of stream flow away from a drainage structure. Often used at bridge abutments.
Stage	Height of water surface above a specified datum.
Stage-Discharge Relationship	A correlation between stream flow rates and corresponding water surface elevations. Sometimes referred to as the Rating Curve of a stream cross-section.
Stilling Basin	An energy dissipater placed at the outlet of a structure.
Storage-Indication Method	A flood-routing method, also often called the modified Puls method.
Storm Drain	The water conveyance elements (laterals, trunks, pipes) of a storm drainage system, that extend from inlets to outlets.
Storm Duration	The period or length of storm.
Stream Contraction/ Constriction	A narrowing of the natural stream waterway. Usually in reference to a drainage facility installed in the roadway embankment.
Stream Reach	A length of stream channel selected for use in hydraulic or other computations.
Submerged Inlets	Inlets of culverts having a headwater greater than about 1.2* D.
Submerged Outlets	Submerged outlets are those culvert outlets having a tailwater elevation greater than the soffit of the culvert.
Superflood	Flood used to evaluate the effects of a rare flow event; a flow exceeding the 100-year flood. It is recommended that the superflood be on the order of the 500-year event or a flood 1.7 times the magnitude of the 100-year flood if the magnitude of the 500-year flood is not known.
Surface Runoff	Total rainfall minus interception, evaporation, infiltration, and surface storage, and that moves across the ground surface to a stream or depression.
Surface Storage	Stormwater that is contained in surface depressions or basins.

Surface Water	Water appearing on the surface in a diffused state, with no permanent source of supply or regular course for a considerable time; as distinguished from water appearing in water courses, lakes, or ponds.
Swale	A slight depression in the ground surface where water collects.
Synthetic Hydrograph	A graph developed for an ungauged drainage area, based on known physical characteristics of the watershed basin. A hydrograph determined from empirical rules.
Tailwater, TW	The depth of flow in the stream directly downstream of a drainage facility. Often calculated for the discharge flowing in the natural stream without the highway constriction. Term is usually used in culvert design and is the depth measured from the downstream flow line of the culvert to the water surface.
Thalweg	The line connecting the lowest flow points along the bed of a channel. The line does not include local depressions.
Time of Concentration, T_c	The time it takes water from the most distant point (hydraulically) to reach a watershed outlet. T_c varies, but is often used as constant.
Tractive Force	The drag on a stream bank caused by passing water, which tends to pull soil particles along with the streamflow, expressed as force per unit area.
Trash Rack	A device used to capture debris, either floating, suspended, or rolling along the bed, before it enters a drainage facility.
Travel Time	The average time for water to flow through a reach or other stream or valley length.
Tributaries	Branches of the watershed stream system.
Uncontrolled Spillway	A facility at a reservoir where floodwater discharge is governed only by the inflow and resulting head in the reservoir. Usually the emergency spillway is uncontrolled.
Ungauged Stream Sites	Locations where no systematic records are available regarding actual stream flows.
Uniform Flow	Flow of constant cross-section and average velocity through a reach of channel during an interval of time.
Unit Hydrograph	A hydrograph of a direct runoff resulting from 1 centimeter of effective rainfall generated uniformly over the watershed area during a specified period of time or duration.
Unsteady Flow	Flow of variable cross-section and average velocity through a reach of channel during an interval of time.

Watercourse	A channel where a flow of water occurs, either continuously or intermittently, with some degree of regularity.
Watershed	The divide between catchment areas.
Water Table	The upper surface of the zone of saturation, except where that surface is formed by an impermeable body (perched water table).
Weir Flow	Free surface flow over a control surface that has a defined discharge vs. depth relationship.
Wells	Shallow to deep vertical excavations, generally with perforated or slotted pipe backfilled with selected aggregate. The bottom of the excavation terminates in pervious strata above the water table.
Wetted Perimeter	The boundary over which water flows in a channel or culvert taken normal to flow.

1 INTRODUCTION

The Ethiopian Roads Authority (ERA) published a series of Road Design Manuals, Specifications and Bidding Documents in 2002. These Manuals were in use for ten years before ERA decided to review and update the series.

Feedback from local experts during the updating process indicated that the ERA Drainage Design Manual (2002) required updating for the following reasons:

- The existing manual was not user friendly;
- The manual did not take sufficient account of relevant legislation and policies;
- No account was taken of sediment and pollution control mechanisms;
- Some of the information contained within the manual was outdated;
- Some of the chapters were generic and not specific to Ethiopia;
- No allowance was been made for future climate change scenarios;
- No financial evaluation methodology was included; and
- The manual was not complete and standalone, lacking important information.

Crown Agents of the UK commissioned ME Consulting Engineers Ltd in November 2011 to update the drainage design manual in collaboration with local road drainage experts. The project was undertaken under the DFID (UK) funded Africa Community Access Programme (AFCAP).

1.1 Purpose and Scope

1.1.1 Purpose

The intention of the review process was to update the 2002 manual with currently available data, and to identify improvements and provide desirable modifications in approach and utilise available technologies. The principal output is this Revised Drainage Design Manual, 2013.

1.1.2 Scope

The procedures for the design of road drainage presented in this manual are applicable to expressways, trunk roads, link roads, main access roads, collector roads, feeder roads and unclassified roads as defined in the ERA Geometric Design Manual.

The drainage design of roads is aimed at the protection of the road through the prevention of damage due to water to achieve a chosen level of service, without major rehabilitation, at the end of a selected design period. The design procedures take into account factors such as rainfall intensity, catchment areas, land use/land cover, topography, climate change, and run-off.

In this version of manual, social, economic and environmental issues are explored and discussed with respect to their impact on any proposed drainage strategy put forward.

The procedures provided in this manual cover a range of drainage design applications and policies currently used and implemented in Ethiopia. The use of the procedures described in this manual will contribute to uniformity in drainage design for a given set of conditions in Ethiopia.

Guidance is provided in Chapter 5 for complex hydrology and hydraulic problems that require specialized engineering knowledge and experience.

1.2 Organization of the Manual

Detailed procedures are given in the different sections of this manual to guide the designer either in the collection of data or in the design process of the features commonly found in road drainage systems.

Selected data, much of which are time-sensitive and subject to revision, such as rainfall intensity-duration-frequency curves, are updated and included in the revised manual. These data should be regularly checked and updated.

The manual is organized as follows:

Chapter 1: Introduction – Background information and overview of the material.

Chapter 2: Standards and Departures from Standards – Describes road drainage standards and when to depart from these standards when local factors govern the design process. A report template has been included as an appendix to this chapter.

Chapter 3: Policy and Planning - Considerations regarding road drainage design policies and planning procedures for ERA.

Chapter 4: Data Collection, Evaluation and Documentation - Data sources and data management during existing road drainage analysis and new drainage design.

Chapter 5: Hydrology – Methods used by ERA for discharge determination or estimation, guidelines and problem examples for development of runoff hydrographs, and discussion of design frequency requirements and considerations.

Chapter 6: Channels - Basic hydraulic concepts and guidance for open channels, including consideration of different channel types.

Chapter 7: Culverts - Basic hydraulic concepts and principles for culvert design, design guidance for various culvert operating conditions, and appurtenances such as improved inlets and erosion velocity protection and control devices are provided.

Chapter 8: Bridges - Basic hydraulic concepts and principles for bridges, hydraulics considerations, bridge scour and channel aggradation and degradation concerns and countermeasures.

Chapter 9: Energy Dissipaters - Basic hydraulic concepts and principles for energy dissipater including types and locations of dissipaters.

Chapter 10: Sub-Surface Drainage – Basic hydraulic concepts and principles for sub-surface drainage, including source of subsurface drainage.

Chapter 11: Storm Drainage Facilities - Basic hydraulic concepts and principles for storm water drainage facilities.

Chapter 12: Construction: - A review of the interaction and project management of projects in terms of designers and contractors is discussed together with the common impacts (erosion and sedimentation) and mitigation measures involved in construction.

Chapter 13: Operation, Maintenance, and Remediation – Discussion on current operation, maintenance and remediation process required to ensure a highway meets its design life.

Chapter 14: Cost Benefit Analysis - Calculation methods (NPV and BCR) to ensure the most economical scheme is chosen.

Chapter 15: Web based support software – Various software is recommended that can be used to make the design process and final output more robust and quantifiable.

2 STANDARDS AND DEPARTURES FROM STANDARDS

2.1 Introduction

The purpose of this chapter is to introduce and discuss a number of general design requirements/standards for road drainage infrastructure in Ethiopia. The requirements presented in this chapter cover a range of topics. More specific design requirements/standards are contained in the relevant chapters of the manual. The intention is that this chapter should be referenced first to establish general and some specific drainage standards/requirements for a road drainage project. Topic specific chapters, such as Chapters 3, 4, 5 and so on, should then be referenced as applicable / required.

2.2 Definitions

The term 'design requirements', encompasses all design: considerations; controls; criteria; and standards that must be included in or be part of the design process.

Design **considerations** encompass all aspects, issues, functionality, expectations, demands, constraints, risk, and cost that need to be appropriately addressed, or taken into account, in order to satisfy design criteria and determine trade-offs. Design **control*s*** are aspects of the road environment or project that cannot be changed, or are extremely difficult to change, and therefore place some restriction or control on the design.

Design **criteria** set the expected level of achievement or conformance to relevant design parameters or design inputs. The design criteria ensure that the end result can be judged and defended. An example of a design criterion with respect to road drainage would be the average recurrence interval for design of a particular project or drainage structure.

Design **standards,** however, set approved or prescribed values or limits for specific elements of design or set procedures and/or guides that must be followed. A design standard with respect to road drainage would be the use of the design flow estimation methods to determine the run-off from a catchment. Design standards are presented throughout this manual. Both design criteria and design standards set the mandatory limits designers must work within and/or achieve.

2.3 Surveys

As mentioned in Chapter 3 of the ERA *Geometric Design Manual*, hydrologic considerations can influence the selection of a road corridor. In addition, studies and investigations may be required at sensitive locations. The magnitude and complexity of these studies shall be commensurate with the importance and magnitude of the project and problems encountered locally. Typical data to be included in such surveys or studies include:

- Topographic Maps, Digital Elevation/Terrain Models (DEM/DTM), and Aerial Photographs;
- Soil Maps;
- Land Use/Land Cover Maps
- Geological maps
- Rainfall records;
- Flood Zone Maps;
- Catchment Flood Management Plans;

- Surface Water Management Plans;
- River Basin Master plans;
- Stream flow records;
- Historical high water marks;
- Historical flood discharges; and
- Locations of hydraulic features such as reservoirs, water projects, regulatory, and floodplain areas.

2.4 Flood Hazards

Hydrological analysis, hydraulic modelling, and flood hazard mapping are prerequisites in identifying flood hazard areas and determining those locations at which construction and maintenance will be expensive or hazardous.

2.5 Flood Immunity Criteria

The flood immunity criteria discussed within this manual relates to individual drainage components (such as cross or longitudinal drainage) and does not relate to the road project, section or link. Furthermore, setting the immunity criteria for various drainage components on a project does not imply that the road inherits the same immunity level(s). It is extremely difficult to assess immunity and set criteria for a road. Refer to Chapter 5 for a more detailed discussion regarding this issue.

2.6 Flood History

All hydrological analyses shall consider the flood history of the area and the effect of these historical floods on existing and proposed structures. The flood history includes the historical floods and the flood history of any existing structures. Public consultation with the local community is important.

2.7 Hydrological Design Standards

More hydrological data has been collected since the publication of the 2002 ERA Drainage Design Manual as part of the manual review and updating work. However, the hydrological data available for Ethiopia is still limited; therefore, the flow estimation procedures shall be applied with caution and engineering judgment. For standard procedures to be adopted confidently storm water run-off coefficients and procedures shall be calibrated and validated with available local data.

The following is a summary of standards that shall be followed for hydrological and hydraulic analysis:

2.7.1 Hydrological Flood Estimation Method

Many hydrological flow estimation methods are available. The methods to be used and the circumstances for their use are listed below. If possible the method shall be calibrated and validated to local conditions and tested for accuracy and reliability.

Discharge Estimation: Many Empirical Formulae have been devised for the purpose of simplifying the methods of estimating flood flows. Some of these formulae relate peak discharge to the total catchment areas while other formulae relate peak discharge to catchment area and slope. For more effective hydrological design, similar Regression Equations for estimation of Design Flood Discharge should be developed for Ethiopia.

However, if such empirical formulae are to be adopted for Ethiopia, their applicability for a particular area in Ethiopia should first be calibrated and verified with locally available data.

The hydrological methods approved by ERA and limitations on their use are as follows:

- Rational Method - only for drainage areas less than 50 hectares (0.5 square. km);
- SCS and other Unit Hydrograph Methods - for drainage areas greater than 50 and less than 65,000 hectares;
- Watershed Regression Equations - for all routine designs at sites where applicable;
- Log Pearson III Analyses - preferable for all routine designs provided there are at least 10 years of continuous or synthesized record for 10-year discharge estimates and 25 years for 100-year discharge estimates; and
- Suitable Computer Programs - such as HEC-HMS and Hydro CAD will be used to aid tedious hydrologic calculations.

Chapter 5: Hydrology contains details on the appropriate selection and use of these methods.

2.7.2 Design Frequency

The design frequency shall generally be in accordance with Table 2-1. A design frequency shall be selected commensurate with the facility cost, volume of traffic, potential flood hazard to property, expected level of service, strategic considerations, and budgetary constraints, as well as the magnitude and risk associated with damages from larger flood events. With long highway routes having no practical detour, and where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. When selecting a design frequency, potential upstream land use change which could reasonably occur over the anticipated life of the drainage facility shall be considered. The design frequencies in Table 2-1 have been updated to reflect the low maintenance practices in Ethiopia, climate change and uncertainties with future land use change.

2.7.3 Economics

Flood frequencies are used to size different drainage facilities so as to select the optimum design that considers both risk of damage and construction cost. Consideration shall be given to what frequency flood was used to design other structures along a road corridor.

Table 2-1: Design Storm Frequency (yrs) by Geometric Design Criteria

Structure Type	EW1/DC8/DC7		DC6/DC5		DC4/DC3		DC2/DC1/track	
	Design	Check	Design	Check	Design	Check	Design	Check
Gutters and Inlets*	5/5/5	10/10/10	5/5	10/10	5/2	10/5	---	---
Side Ditches	10/10/10	25/25/25	5/5	10/10	5/2	10/5		
Ford/Low-Water Bridge	---	---	----	----	---	---	5/5/5	10/10/10
Culvert, pipe (see Note) Span<2m	25/25/25	50/50/50	10/10	25/25	10/5	25/10	5/5/5	10/10/10
Culvert, 2m< span<6m	50/50/50	100/100/100	25/25	50/50	25/10	50/25	10/10/10	25/25/25
Short Span Bridges 6m< span<15m	50/50/50	100/100/100	25/25	50/50	25/10	50/25	10/10/10	25/25/25
Medium Span Bridges 15m< span<50m	100/100/100	200/200/200	50/50	100/100	50/25	100/50	50/25/25	100/50/50
Long Span Bridges spans>50m	100/100/100	200/200/200	50/50	100/100	50/25	100/50	50/25/25	100/50/50

EW1 Express Way
* See Chapter 10 – Storm Drainage Facilities for further details
Note: Span in the above table is the total clear-opening length of a structure. For example, the span for a double 1.2-meter diameter pipe is 2.4 meters, and the design storm frequency is therefore "culvert, 2m<span<6m." Similarly a double box culvert having two 4.5-meter barrels should use the applicable design storm frequency for a short span bridge and a bridge having two 10-meter spans is a medium span bridge. A 20% flow allowance for climate change should be added to the above design flows.

2.8 Design Life/Service Life

It is important to define, in general terms, the difference between design life and service life. The design life of a component or system of components is the period of time during which the item is expected, by its designers or as required by specification, to work or perform its intended function within specified design parameters / operating conditions. In other words, the design life is the life expectancy of the item under normal / specified operating conditions. With respect to road drainage, operating conditions can include:

- Environmental/atmospheric/geographic conditions;
- Foundation, bedding and support/cover conditions; and
- Traffic and loading.

For example, ERA may specify the design life for a new structural component (such as a culvert) as 50 years. Therefore, it is expected that the culvert will last 50 years before replacement or major repair is expected.

The service life of a component or system of components is the period of time over which the item actually provides adequate or satisfactory performance before repair or replacement is required. If the operating conditions over the life of the component or system remain within the original design parameters, theoretically, the design life will equal service life.

However, if the operating conditions move outside of the original design parameters, service life will be less than design life. In some situations, this reduction in time can be considerable, leading to premature failure. In relation to drainage infrastructure, drainage designers should be mindful of these two terms and ensure, where possible, that the designed drainage components or systems are appropriately selected for the anticipated operating conditions.

2.9 Road Locality

There are two major environments or zones potentially affected by drainage and these are defined as the road environment and the external environment.

Road Environment: The road environment is the zone which ERA has responsibility for and therefore is under its control. It is defined as the road corridor as defined by property boundaries (also known as road reserve). It is important to note that not all boundaries are clearly defined. In these situations, the road reserve is usually based about the existing road centreline and planners and drainage designers need to further investigate to establish applicable boundaries.

External Environment: The external environment is the zone outside of the road corridor which may include sensitive areas such as wetlands, rainforest, waterways, private properties or other infrastructure (e.g. railways). The external environment may extend for some distance from the road environment and is not the responsibility of ERA. However, ERA or its design consultants need to liaise or work with relevant stakeholders and authorities with respect to any proposed project as drainage work within the road environment may affect the external environment both upstream and downstream of the project.

2.10 Identifying Design Considerations

The construction of new or upgraded road drainage infrastructure may lead to changes in the existing road and external environments. Problems associated with erosion and sedimentation, flooding (changes in peak water levels) and water quality are of concern to ERA, adjacent land owners, road users and the local community. The occurrence of these problems, particularly after a project is completed, can be costly to remedy and may lead to reduced amenity.

Effective project planning covering both the road and external environments plays a major role in minimizing the potential for adverse impacts. The planning and design of road drainage infrastructure can be quite complicated and involves the consideration of a diverse set of data in order to develop the most appropriate drainage solution for a project.

Collaborative planning by a group of professionals with complementary skills is often a productive way to identify all aspects, issues, functional requirements, expectations, demands, constraints, risks and possible costs to be considered in a project.

Design consultants should identify a generic set of considerations that address drainage issues across Ethiopia. In order to develop the most appropriate drainage solution, the project team for each project must select applicable drainage considerations from the following categories: *Geometric; Geographic; Environmental; Crossing Type; Maintenance; and Safety*.

It should be noted that identified design considerations may present several options when being addressed. It is possible that upon further consideration or review, some design considerations may no longer be part of a project while others develop into key design controls.

2.10.1 Geometric Considerations

There are two aspects of geometry that must be considered in the drainage design of a road project. Some parts or components of these aspects may in turn become design controls. The first aspect deals with the geometry of the watercourse and the second aspect deals with the geometry of the road-watercourse crossing.

Watercourse Geometry

It is important to determine the geometry of the watercourse or flow path, in particular: watercourse longitudinal alignment; watercourse gradient; and channel shape.

Watercourse alignment refers to the natural meanders of the watercourse channel. While most watercourses have only one alignment for all flows, it is possible to have the situation where the alignment for a low flow differs from the alignment for a high flow in the same watercourse. This situation must be identified and considered when designing the road-watercourse crossing.

It is possible to alter the alignment of existing watercourses to improve the hydraulic performance of the road-watercourse crossing, however it is preferable to maintain or preserve the existing watercourse alignment as changes will affect the existing flow parameters (velocity, depth of flow and energy). Furthermore, it is important to note that licences maybe required from the Ministry of Water & Energy of Ethiopia to change the alignment of any defined watercourse. However, experience has shown that the process of obtaining relevant licence to alter the alignment of the watercourse may not be difficult in Ethiopia.

Watercourse gradient refers to the vertical alignment of the watercourse and changes to gradient will also affect flow parameters. Gradient has a significant influence on flow velocity and velocity in turn has a significant effect on sediment transport and scour potential.

Channel shape needs to be considered as it will tend to dictate the size and configuration of drainage structures. Altering the channel shape to accommodate a drainage structure will affect flow parameters and could increase the risk of erosion. It is preferable to maintain or preserve the existing channel shape as closely as possible and culvert structures should be designed to 'fit' the shape of the watercourse. Some channels may not contain all of the design storm run-off and overtopping of the banks will occur. Multiple culvert installations for the one catchment will be required and in this instance, specialist advice / design will be required.

Lastly, road drainage designers must have an understanding of stream morphology when considering stream geometrics. Streams are dynamic and can change over time. It is important for this aspect to be considered.

Road Geometry

Drainage is an integral component of road infrastructure and therefore drainage design cannot be undertaken in isolation from the geometric design of the road. In the design of the road-watercourse crossing, it is important to consider the skew angle between the road alignment and drainage structure. Keeping the skew angle as small as possible (or eliminating it altogether) reduces costs and construction difficulty and is therefore the most desirable option.

Given that it is highly recommended to preserve watercourse alignment, this consideration, however, does not imply any priority of drainage over road alignment and high skew angles may be unavoidable at times.

The design of the vertical alignment should be undertaken in conjunction with the design of the drainage system. An initial vertical alignment design would be used to undertake the initial drainage design of various structures. It may then be necessary to adjust the vertical alignment in order to achieve the most efficient and effective drainage design (considering allowable headwater levels, afflux and minimum cover requirements for structures). In this instance, the requirements for drainage may become a design control on the vertical alignment. However, the drainage designer needs to be aware that constraints placed on vertical alignment would make it a design control on the drainage system and force the design to change.

Furthermore, vertical alignment together with cross-sectional cross fall of the road also affects longitudinal drainage channels (such as table drains) and therefore must be designed considering minimum grade requirements for flows and minimising steeper grades where higher erosive velocities could result. Another important aspect related to the geometric design of roads is storm water run-off from the road surface. This aspect is critical as water flow (and depth) on the road surface can result in aquaplaning.

Surface flows are as a result of the geometric road design (combination of horizontal, vertical, cross section, cross fall and super elevation elements) and therefore any identified problems should be solved and mitigated through amended geometric road design. A drainage solution to aquaplaning should only be considered as a 'last resort' option. If a drainage solution is required, specialist advice is highly recommended in the development / assessment of design options.

Lastly, where the possibility of storm water crossing over the road exists (whether intentional or unintentional), adequate stopping sight distance must be provided and this factor could affect the vertical alignment design.

2.10.2 Geographic Considerations

Geographic conditions play a significant role in the determination of what type of drainage structure and/or controls may be adopted at a given location. Structures and controls that are appropriate in one part of Ethiopia may not be suitable in other parts. This section discusses some key issues for different situations and regions across Ethiopia.

Most of the ERA's roads are located in rural regions, so standard practices for the planning and design of road drainage should address most of the issues that will arise in these areas. However, it is important to note that these issues can also apply in urban regions. The design of drainage systems in all regions of Ethiopia should ensure that the road level and associated drainage infrastructure is adequate to provide the specified level of flood immunity. Furthermore, the drainage structures should be sized to ensure that flow velocities and afflux are acceptable.

Specific issues to be addressed include:

- Awareness of local drainage and management plans;
- Ensuring property and crops will not be affected by an increase in water levels or duration of inundation;
- Changes to flow patterns, and consideration of seasonal variations in hydraulic roughness linked to changes in vegetation cover.
- Concentration of flow on floodplains should be minimized because of the risk of scour; maintaining free drainage, and not creating ponding at low flows.

Urban regions have similar issues to rural areas, but may also present other constraints. Constraints may be present in the form of adjacent infrastructure (including businesses and housing) or a limit in available space (right-of-ways).

Because of the more intense level of development, afflux is usually of more concern in urban areas than in rural locations. In addition, regional authorities may have prepared catchment or storm water management plans, which will affect the future management of storm water and watercourses in an area.

Considerations in urban regions include:

- Provision for higher peak flows arising from uncontrolled upstream development (regional authorities may require flow increase to be mitigated or limited);
- Assessment of the requirements of any catchment management plan or storm water management plans prepared for the watercourse;
- The need for pollution control measures;
- Interaction of road drainage provisions with existing services;
- Minimization of ground disturbance during construction, as urban environments often have limited space for large control measures such as sediment basins; and consideration and control of afflux effects. There is often a requirement that negligible afflux increases be generated upstream/downstream of the proposed drainage structure.

With respect to possible change in water levels, it is important that each case is assessed fully in keeping with a risk management approach. Design of road drainage in flat terrain is often difficult for several reasons, including:

- Flows velocities in flat areas are usually low so larger structures are needed to convey the flow;
- Flow may be widespread and/or shallow and minor obstructions may divert the flow; these minor obstructions include levees and other floodplain works; and
- Even the road itself may cause major diversions.

It is often difficult to determine the catchment areas accurately because of minimal relief in terrain and the presence of minor obstructions as discussed above. Poorly defined flow paths also mean that it is sometimes difficult to place culverts in the most suitable locations.

In flat terrain, the impacts of the road on flood levels may extend for significant distances upstream of the road. Where afflux is a concern, this impact may often be critical. There is usually an increased risk of erosion at culvert outlets because flow will be concentrated by drainage structures, particularly where there are poorly defined flow paths and/or most flow occurs across the floodplain.

In mountainous or steep terrain, the most common factor influencing design is the gradient of the natural ground. Issues for consideration where topography is steep include:

- Control of velocities in roadside drains and culvert outlets;
- Collection and discharge of water from the upward side of the road to the downward side;
- Prevention of erosion at outlets onto steep areas; and
- The need for small scale drop structures, weirs or drop manholes.

Locations subject to inundation by water, such as floodplains by backwater, require careful consideration of how drainage infrastructure will operate under a range of water levels. The presence of high and low water levels requires significantly different approaches:

- When downstream water levels are high, the hydraulic capacity of a structure may be limited; and
- When downstream water levels are low, high velocities can result, thereby maximising the potential for erosion to occur.

It is therefore very important that both cases are considered during the design of drainage infrastructure. Regular inundation (i.e. change in water levels) can also accelerate the erosion process, through the saturation of banks, which may then fail as water levels drop.

2.10.3 Environmental Considerations

Drainage has the potential of causing environmental harm. Therefore it is important that environmental impacts are assessed and mitigated (as appropriate) as part of the development and operation of a road drainage system.

The risk of scour/erosion and sediment movement caused by the concentration of flows that typically occurs with drainage structures is of particular concern. Causal factors, including changes in flood flow patterns and changes in peak water levels should also be checked. In some instances, a new road embankment could lead to long term ponding of water which in turn could have adverse environmental impacts.

Environmental considerations will vary significantly from project to project, and hence it is not practical to list all potential issues in this section (for more detailed discussion, refer to Chapter 3). However, there are two types of environmental consideration for which details have been provided.

These are: the provision for fauna passage and the maintenance of water quality. In many projects, it will be important to ensure that the design of drainage infrastructure adequately caters for the existence of fauna, and also for the maintenance (or improvement) of the quality of storm water run-off. Chapter 3 describes the role of the environmental assessment (process and documentation) in obtaining and analysing data for the purposes of identifying potential environmental considerations for a project's drainage design.

Careful review of any relevant environmental assessment documentation, including any recommended management strategies, needs to be undertaken as some of these strategies may become design requirements or criteria. The recommended management strategies are generally based on the requirements of relevant legislation, policy, codes, guidelines and current best practice within Ethiopia.

2.10.4 Crossing Type

Determining the type of structure for any crossing is an important consideration and there are a number of factors that need to be addressed in this process. It may be necessary to assess several options of different crossing type and size in order to appropriately meet the design requirements and objectives. There are three main types of cross drainage structures used on roads and each has particular advantages and disadvantages. The three types are bridges, culverts and fords as shown in Figure 2.1



Figure 2-1: Primary Drainage Infrastructure Types

Factors in Selecting Type of Crossings

The relevant factors that need to be considered in selecting drainage infrastructure are grouped into hydraulic and other factors. The hydraulic factors include:

Flood discharge: Defined waterways with a large discharge are more suited to a bridge because of the larger waterway area. The large discharge will also generally occur in rivers, where a bridge is more appropriate and cost effective. Depending on location and importance of road, in flat terrain where the waterways are less defined and road embankment is typically low, a ford may be a better option.

Watercourse channel conditions and topography: Similarly, with the consideration of discharge, the shape and size of the channel and the catchment will also indicate whether a bridge, culvert or ford is most suitable. Large and well defined channels will be better suited to a bridge, while less well defined, smaller channels will be more suited to a culvert, especially where multiple openings are required (such as on floodplains). Fords also could be considered, particularly in flat terrain/low embankment situations.

Afflux constraints: The most suitable structure may be indicated by the amount of flow that can pass through/over the structure with acceptable afflux. The location and extent of afflux needs to be considered in detail and the alternatives assessed to minimise afflux.

Debris properties: Culverts will normally have a smaller waterway area and present a greater obstruction to the flow. They are therefore more prone to collection of debris. If a large amount of debris is conveyed by a watercourse, a bridge or larger culvert may be more suitable.

Scour risk: The effect of scour depends on the size and type of waterway. If a structure concentrates flow significantly, risk of scour may be increased, so structures that spread the flow may be favoured in these locations. This is especially important for drainage in floodplains where the flow paths may not be well defined.

Other relevant factors that need to be considered include:

Road alignment: Sometimes, the alignment of the road is well defined and this may not be the best arrangement for drainage. This may sometimes occur where land tenure needs to be considered and the alignment follows watercourses rather than crossing at a zero skew. In these cases, the sizing and locating of drainage structures must be carefully considered.

Level of serviceability: This includes the required flood immunity or trafficability and the type of structure that will be best for meeting this requirement.

Soil conditions: Particular soil conditions, such as mud or acid sulphate soils for example, may be a problem and this can affect the selection of drainage structures.

2.11 Bridge, Culvert or Fords

There are a number of factors and issues that need to be considered in the selection of the most suitable / appropriate structure for a particular crossing. These are listed in Table 2-2.

2.11.1 Culvert Types

Selection of culvert type is important in some applications. The choice is between the following predominate types:

- Pipes (any material type);
- Box culverts, including slab link culverts;

- Slab deck culverts (cast in-situ); and
- Multi-plate arches.

There are two issues of particular concern for selecting the type of culvert. The first relates to the waterway area at low flow depths and the second relates to the extent to which the culvert spreads the flow. Box culverts and slab deck culverts provide for a greater waterway area at shallow depths while pipes need to flow at a greater depth before the maximum flow capacity is reached. The use of pipes however does tend to spread the flow to a greater extent, which is often desirable for consideration of concentration of flow and risk of scour.

2.12 Maintenance Considerations

The provision of maintenance is an integral component in the planning and design phases of road drainage. Adequate maintenance is necessary for the proper operation of the drainage system. Lack of maintenance is one of the most common causes of failure of drainage systems (erosion and sediment controls). This may be attributed to reasons such as a significant reduction in hydraulic or storage capacity (e.g. blockage by debris or sediment).

Specific details on maintenance procedures and requirements for road drainage systems are provided in Chapter 13 of this manual. To enable maintenance to be properly and safely undertaken during road construction and operation, consideration must be given at the design stage.

2.13 Safety Considerations

An integral aspect of the detailed design of all road drainage systems is the underlying consideration of safety. Some of the safety issues that require consideration as part of the road drainage design process, excluding workplace health and safety issues are described below.

Maintenance Access: - Safe access needs to be provided to all drainage structures that require either ongoing (e.g. moving of drains) or occasional (e.g. removal of debris) maintenance. This access is required for vehicles and maintenance crews depending on the type of maintenance that will be undertaken. Safe access to erosion and sediment control devices during the construction phase should also be allowed.

Human Safety: - Where long culverts potentially provide a hazard (particularly in urban areas) to human safety, preventative measures should be considered. Safety measures include fencing, swing gates and grates at culvert inlets. Any safety device needs to ensure that it prevents both accesses to the culvert and trapping of people against the grate. The effect of any proposed human safety measure on culvert capacity and efficiency need to be checked.

Traffic Safety: - Projecting culvert ends have the potential to act as obstructions to 'out of control' vehicles. Where there are no safety barriers; culvert ends should be designed so as not to present an obstruction. If obstructions from projecting culverts or head walls are unavoidable then safety barriers should be considered.

Ford Safety: - The main issue associated with safety at fords is adequate sight distance for drivers to ensure vehicles can stop before entering the ford. Preferably, the ford longitudinal profile should be horizontal so that the same depth of water exists over the entire ford length. The ford length should be limited and be on a straight stretch of road

where possible. Adequate permanent and temporary signing must be erected. As flood water recedes, silt and debris can be left on the road surface of a ford and this can be a hazard to road users. ERA should inspect each affected ford as soon as possible after a flood event and clear the surface if required.

Energy Dissipaters: **-** Energy dissipation is necessary due to high flow velocities. Dissipation devices usually consist of large obstructions to the flow and result in a high degree of turbulence. For these reasons, energy dissipation structures should be avoided in urban areas where possible. Otherwise, access should be limited by appropriate fencing. Energy dissipaters are also very costly to build and maintain and changes to the design, such as flattening of channel to reduce high velocities, is preferred.

2.14 Culvert Design Criteria

All culverts shall be designed hydraulically except where difficult geometry dictates otherwise; the minimum size of culverts should be 1.2m. However, existing culverts of between 0.9m to 1.2m but functioning properly without any maintenance problem can be retained during the upgrading design of the road. Any culvert less than 0.9m in an upgrading project should be replaced by new one having a minimum opening of 1.2m diameter. For a primary valley in a rolling terrain, a group of culverts may be required and analyzed with the help of storage. HEC-HMS hydrological assessment software should be used to analyze this type of culvert arrangement. However, in areas like the Somalia region of Ethiopia where the road is mainly constructed on fills, the implementation of the minimum culvert size 1.2m might be difficult. Therefore, this culvert design criteria should be relaxed when a robust justification is provided by the design consultant and the implementation contractor where smaller culverts conveying the same flow are more practical from an economic and design point of view. For optimum sizing of culverts the latest software like Hydra flow Extension, which is freely available with AutoCAD Civil 3D software, should be utilised.

The overtopping flood selected shall be consistent with the design class of highway and commensurate with the risk at the site. Survey information shall include topographic features, channel characteristics, aquatic life, high water information, existing structures, and other related site specific information. Culvert location in both plan and profile shall be investigated to avoid sediment build-up in culvert barrels. Culverts shall be designed to accommodate debris or proper provisions shall be made for debris maintenance.

Material selection shall include consideration of service life which includes abrasion and corrosion factors. Culverts shall be located and designed to present a minimum hazard to traffic and people. The cost savings of multiple uses of the culvert(s) (utilities, stock and wildlife passage, land access, and fish passage) shall be weighed against the advantages of separate facilities.

The detail of documentation for each culvert site shall be commensurate with the risk and importance of the structure. Design data and calculations shall be assembled in an orderly fashion and retained for future reference as provided for in the data requirement in Chapter 4, and culverts shall be regularly inspected and maintained.

2.14.1 Design Limitations

Allowable Headwater is the depth of water that can be ponded at the upstream end of the culvert and which will be limited by one or more of the following:

- No damage must be done to upstream property; and
- Water level must be:
  - No higher than the shoulder or 300 millimetres below the edge of the shoulder;
  - Equal to an HW/D but not greater than 1.5;
  - No higher than the low point in the road grade; and/or Equal to the elevation where flow diverts around the culvert.

The **Review (Check) Headwater** is the flood depth that:

- Does not exceed 500 millimetres increase over the check flood in the vicinity of buildings or dwellings; and
- Has a level of inundation that is tolerable to upstream property and roadway for the review discharge.

Table 2-2: General Selection Factors - Structure Advantages & Disadvantages

Structure	Advantage	Disadvantage
Bridges	Waterway area generally increases with increased deck height; Provides greatest flood immunity; Large flow capacity; Fewer problems with debris; Deck widening does not affect capacity; Fewer disturbances to riparian environment about waterway.	Higher design, construction and maintenance costs; More structural maintenance required; Spill slopes can be affected by erosion (potential for costly batter protection requirements particularly for higher/exposed approach embankments); Pier and abutment can be affected by scour; Increased buoyancy, drag and impact risks; Susceptible to stream/channel migration.
Culverts	Simplest structure to design & construct; Generally most cost effective option; Can accommodate future changes to road geometry; Less structural maintenance; Can spread flows.	Generally require higher levels of general maintenance; Most susceptible to failure; Higher siltation/debris risk (blockage); Increased environmental impacts (fauna/fish passage); Potential for scour at outlet; Subject to abrasion; Future extension may reduce capacity; Potential for separation at joints; Potential for failure by piping (leading to failure of embankment).
Fords	Generally simple to design; May offer environmental advantages over culverts and bridges, since they will tend to spread flows more widely; Typically have low embankments; Risk of scour to waterway and surrounding land is reduced.	Allow water flow over road – immunity and safety issues; Increased disruption to traffic due to overtopping; Can have higher construction costs than culvert; Batter slopes can be affected by erosion / scour (particularly for higher embankments); Generally have costly batter protection requirements; Susceptible to stream / channel migration; Can have environmental impacts (fauna / fish passage); Potential for failure of embankment (depending on provided protection).

2.15 Bridge Design Criteria

The following are general criteria relating to the hydraulic analyses for the location and design of bridges. These principles identify specific areas for which quantifiable criteria can be developed:

- The final design selection shall consider the maximum backwater depth allowed by ERA (0.5 metres) unless the exceeded limit can be justified by special hydraulic conditions. Furthermore, backwater shall not significantly increase flood damage to property upstream of the crossing;
- Velocities through the structure(s) shall neither damage the highway facility nor increase damages to adjacent property. The final design shall not significantly alter the flow distribution in the flood plain. The existing flow distributions shall be

maintained to the extent practicable. The crest-vertical curve profile shall be considered as the preferred highway crossing profile when allowing for embankment overtopping. A freeboard shall be established to allow for passage of debris;

- Degradation, aggradation, contraction and local scour of a river shall be estimated. Appropriate positioning of the bridge foundation, below the total scour depth if practical, shall be included as part of the final design. Pier spacing, orientation, and abutment shall be designed to minimize flow disruption and potential scour. Design foundation and/or scour countermeasures shall be chosen to avoid failure by scour. Acceptable risks of damage or viable measures to counter the vagaries of alluvial streams shall be specified.

The following other general criteria relate to the location and design of bridges:

- Minimal disruption shall occur to ecosystems unique to the floodplain and stream;
- A traffic level of service compatible with that commonly expected for the design class of highway and compatible with projected traffic volumes shall prevail;
- Design choices shall support costs for construction, maintenance and operation, including probable repair & reconstruction and potential liability.

2.16 Design Storm/Flood - Backwater and Flow Velocity

Objective principles are necessary to develop rules and procedures for the design of drainage systems. Principles that have been used in the development of this manual are described and defined in the text, indicated in *italics,* and preceded by background discussion.

2.16.1 Design Storm/Flood

The design peak flood is the peak flow rate of the defined probability (or Average Recurrence Interval) for the required drainage works. Usually the design discharge is used to determine the size of the drainage structure and the level of the road. The design discharge is expressed as a flow rate, usually as cubic meters per second (m^3/s).

Usually the discharge rate is calculated directly by a hydrology procedure, such as the Rational Method or Snyder's Unit Hydrograph for the drainage structure. This rate is used directly.

In more complex situations, the design discharge is calculated while accounting for attenuation or flow diversions. A design frequency shall be selected in relation to the cost of a facility relative to budget constraints, amount of traffic and expected Level of Service, potential flood hazard to property located in the project area, political considerations; and the magnitude and risk associated with damage from larger flood events. In Ethiopia, long highway routes have no practical detour, and many sites are subject to independent flood events. Therefore, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. In selecting a design frequency, all potential upstream land use for the anticipated life of the drainage facility must be considered. Drainage works shall be designed for storms having a recurrence interval of at least that shown in Table 2-1.

All bridges and major culverts shall be checked for performance under a storm event less frequent than the design storm event shown in Table 2-1 as the Check/Review Flood. All other drainage structures shall be checked for the next lowest storm frequency compared

with the design storm event. For example, minor culverts designed for a 10 year storm shall be checked for adequate performance with a 25 year interval storm event.

2.16.2 Afflux and Backwater Effect

In hydrology, afflux is defined as a rise in the water level caused by and immediately upstream of a natural or artificial obstruction. Backwater is a consequence of afflux, in other words the afflux causes a 'backwater effect'. Figure 2.2 below provides an illustration of afflux and backwater.

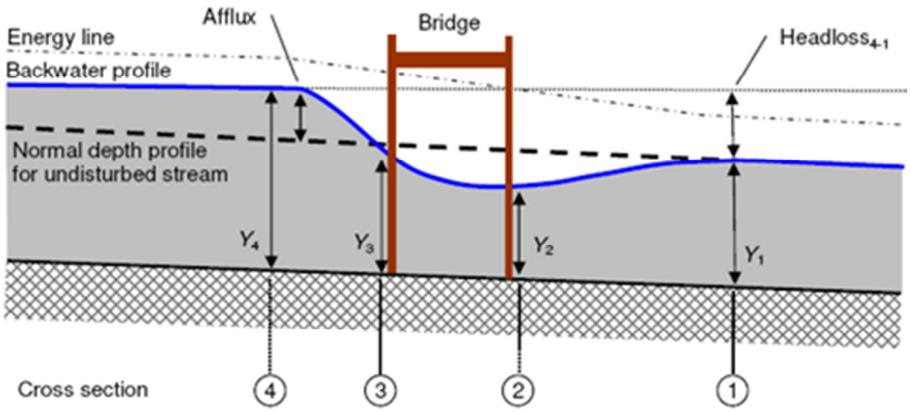


Figure 2-2: Bridge Afflux

2.16.3 Allowable Afflux

Afflux is the increase in peak water levels produced by the introduction of a culvert or bridge and is the comparison between the water levels for the existing conditions and the proposed conditions once the road has been built. Afflux is defined for a particular location and will vary across the floodplain or along the length of a channel.

The allowable afflux is often a controlling factor in the design of drainage structures and can be a serious community concern. While ERA must assess the afflux expected during the planning and design process, regional ERA authorities will often specify the requirements that they require in their region.

Afflux is usually caused by a constriction in a flow path by the construction of a culvert, bridge or ford. However in some cases, especially in flat terrain and where flow may be diverted from one catchment to another, it could be caused by a redistribution of flow. Afflux can also be negative, that is a reduction in flood level, downstream of a constriction or where flow is diverted away from a stream. The point of maximum afflux occurs immediately upstream of the road and then dissipates while moving further upstream.

There is a point where the afflux drops to zero and the influence of the bridge on flood levels disappears. In flat terrain, this point may be a considerable distance upstream, but in steep terrain with high flow velocities, the afflux may extend only a very short distance. The afflux also reaches a maximum at the point of overtopping of the road. Smaller floods will be conveyed easily through the structure, while larger floods may eventually drown out the structure. For very large floods, there may be no impact on flood levels if the structure is submerged to a significant depth.

Afflux needs to be considered in all drainage designs. During the planning phase, any properties, infrastructure or other features upstream of the crossing must be reviewed.

These structures then need to be considered in the design and the impact on flood levels at each of these must be included in the design process. If there is nothing that could be adversely impacted by an increase in flood levels, afflux consideration does not necessarily form a part of the design. In this case, the maximum permissible flow velocity through the structure is the critical factor.

The allowable afflux will vary for individual locations. In some particularly sensitive areas, no afflux may be the appropriate limit. This would be in areas where there are already flood prone properties and even a small increase in level could cause a significant increase in damage. In some locations, a small amount of afflux may be acceptable. In this instance, the afflux is often of the order of 250mm, though higher afflux may be possible in some situations.

Afflux is usually reduced by increasing the opening area of the drainage structure, but it can also be reduced by channel works or other mitigation measures. Reducing the afflux may lead to higher costs for drainage infrastructure and it may be impossible to reduce the afflux at some sensitive locations, even with extensive mitigation measures. In these cases, careful assessment of the hydraulics and potential damage is needed and this should be followed by consultation with affected property owners to develop an acceptable result.

When dwellings or other man-made structures are close to the drainage way, a limitation shall be placed on the maximum backwater effect to be tolerated for drainage structure design.

The maximum backwater effect at a drainage structure shall be 0.5 metres lower than the floor elevation of buildings or the floor level of dwellings is higher by 1.5 metres above the natural design flood elevation. Otherwise, the maximum backwater level shall be 1.0 metres lower than the floor elevation of upstream buildings or dwellings and the check flood elevation shall be 0.3 metres lower.

2.16.4 Flow Velocity

Flow velocity is a critical parameter used in design of road drainage structures. It is the velocity of water in the flow path. The flow velocity can be calculated for a particular location in a stream cross section or it can be an average over a portion or the whole of the cross section. Flow velocity can be calculated using Manning's Equation, by a hydraulic model or it can be measured during an actual flood event.

Flow velocities are usually calculated initially for the natural channel, without any drainage works (pre-development scenario). This velocity indicates the natural conditions which can be used as a basis for the consideration of the drainage works. Flow velocities can then be calculated for the post-development conditions with the addition of the proposed infrastructure.

Velocity in a flow path depends on the slope and geometry of the flow path as well as the channel roughness and the amount of flow. It often varies across a cross section and along the reach of a stream.

Water velocities within a stream are not uniform. Frictional forces decrease the water velocities along the bottom and sides of the stream channel (Figure 2.3).

The introduction of a culvert or a bridge to convey stream flow beneath a highway can cause an increase in flow velocity downstream of the structure. The increased flow velocity may be sufficient to cause erosion and degradation of the channel profile.

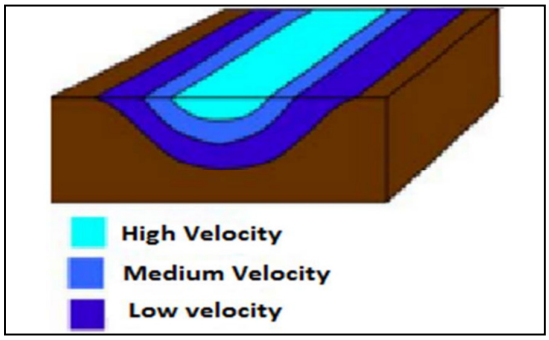


Figure 2-3: Velocity profile

This effect can be detrimental to downstream land users and to the culvert itself. If the natural stream velocity exceeds the erosive velocity, then the increased velocity at the culvert outfall will accelerate this naturally occurring process. This must be avoided to protect downstream lands and the roadway embankment.

The flow velocity at the outlet of the roadway drainage works shall not exceed the erosive velocity of the channel or the natural velocity of the channel, whichever is greater.

2.16.5 Permissible Velocities

When designing a drainage structure or channel, the flow velocity is an important input to the design process. This is because excessive flow velocities will cause scour. The risk of scour depends on the gradient (slope) and geometry of the channel, the soil conditions and the vegetation cover.

When the velocity of flow increases beyond a limit, the risk of scour will increase. In the design, the permissible flow velocities need to be defined to help in the design process.

The process used is as follows:

- The drainage structure (culvert, bridge, ford or channel) is designed, based on the best available information;
- The design flow velocity for the preliminary design is calculated;
- The maximum permissible flow velocity is compared to the calculated design velocity;
- The design may be modified to meet this limit, by increasing the opening area or reducing the slope for example; and
- If this is impossible because of constraints, appropriate mitigation measures will be needed.

The permissible velocities depend on the material of the channel bed as well as the type of soil, channel gradient & shape and vegetative cover. Permissible flow velocities are listed in Table 2-3 below and can be found in Chapter 6 in greater detail. While the permissible flow velocities are mainly set to counter the risk of scour, the permissible flow velocity may also depend on other environmental factors, such as the allowance for fish passages.

Table 2-3: Non-Erosive Velocities in Natural Streams

Stream Bed Type		Non-Erosive Velocities (m/s)
Silt		Less than 0.3
Sand	Fine Coarse	Less than 0.3 Less than 0.3
Gravel	6 mm 25 mm 100 mm	0.6 to 0.9 1.5 to 1.5 2.0 to 3.0
Clay	Soft Stiff Hard	0.3 to 0.6 1.0 to 1.2 1.5 to 2.0
Rocks	150 mm 300 mm	2.5 to 3.0 3.0 to 4.0

2.17 Cross Drainage

Aspects of cross drainage that require special consideration in drainage design include:

- Hydraulic efficiency and capacity of the culvert in its initial (short) and ultimate (long / extended) forms;
- Possible change in culvert operation (inlet control/outlet control) and subsequent outlet velocity changes;
- Potential variation in afflux and/or allowable headwater changes;
- Positioning of culvert inlets and outlets (within the stream);
- Changes to the inlet/outlet of adjacent culverts (in the same stream) where these are located within the median of a dual carriageway and where future widening will be within the median (e.g. culverts may become connected);
- Environmental considerations (e.g. scour prevention measures, fish or animal passage); resumptions (e.g. land required to accommodate future culvert inlets and outlets, allowance for maintenance access); and
- Cover over future culvert extensions due to carriageway widening (on the outside of the formation and/or in the median).

2.18 Longitudinal Drainage

Aspects of longitudinal drainage that require special consideration for drainage design include:

- Drainage of the ultimate median which must be provided;
- Height of pipes and inlets designed to fit the initial and ultimate shapes of the median and carriageway;
- Designed capacity and hydraulic operation suitable for the initial and ultimate configurations;
- Conversion of an open channel within the median to an underground piped system and the requirements for outlets;
- Road safety impacts with drainage inlets structures within the median;
- Drainage connections to bridges (including any pollutant control devices) may need to be designed for the ultimate configuration (e.g. need to cope with additional surface run-off from a widened structure);

- Resumptions (e.g. land required to accommodate catch drains, diversion drains or channels, maintenance access, sedimentation basins);
- Environmental considerations (e.g. size and location of sedimentation basins).

2.19 Surface Drainage

Aspects of surface drainage that require special consideration for drainage design include:

- Aquaplaning (e.g. pavement widening may create a problem where before there was none);
- The application of super elevation in the initial stage may need to suit the ultimate stage;
- Use of crowned multi-lane one-way carriageways to reduce aquaplaning will impact on drainage design (e.g. a third lane added to the median inside of a two-lane carriageway may be crowned and so drain towards the median); and
- Addition of kerbing / kerb and channelling in the future (e.g. channelling of unchannelled intersection, when widening a two-lane carriageway to three lanes).

2.20 Sub-Surface Drainage

Aspects of sub-surface drainage that require special consideration for drainage design include:

- Location and capacity of sub-soil drains;
- Location of outlets and cleanout points to allow for ultimate shape; and
- Changes to the water table and groundwater flows.

2.21 Medians and Obstructions

In divided roads where the ultimate median has a concrete safety barrier and the median width is at or near the absolute minimum, the ultimate median drainage system will require the use of drop inlets to an underground drainage which can be located beneath the barrier itself.

The location of obstructions or immovable features such as bridge piers and abutments must be carefully considered to enable the future stage development of the cross section of the road to be implemented without major change to these features. Preserving the required above ground horizontal and vertical clearances to these features is essential in this process as is providing underground clearances from footings or abutments to the underground storm water drainage system.

2.22 Drainage Design Controls

Design controls are aspects of the road environment or elements of the project that cannot be changed, or are extremely difficult or costly to change. These aspects and elements therefore place restrictions and constraints on the drainage design. Design controls can either place a direct restriction on a project or at least influence the development of design options, thereby becoming design considerations.

One example of a design control with respect to drainage may be the width of the road reserve. Where resumptions are undesirable, the existing right-of-way could limit the available space for drainage infrastructure and therefore control what can be done.

Another example may be the location of the horizontal alignment / centreline. While the design of the horizontal alignment should consider drainage elements, there are many

reasons why the location of the horizontal alignment may be fixed. This could then directly restrict or influence the drainage design. Where it is possible, vertical alignment should rarely be a design control over drainage design as both elements need to be developed holistically in order to achieve an appropriate design solution.

2.23 General Hydraulic Criteria

Hydraulic criteria includes the following:

- Design discharge;
- Flow velocities;
- Permissible velocities;
- Flood and stream gradient;
- Fish passage requirements;
- Erosion and sediment control;
- Permissible afflux;
- Tail water levels and backwater potential;
- Pollution control;
- Road closure periods;
- Inundation of adjacent land;
- Maintenance of flow patterns.

Establishing the hydraulic criteria requires an understanding of the hydrological and hydraulic conditions of the site or project.

2.23.1 Flood and Stream Gradient

Flood and stream gradients are considerations in drainage designs, since these affect stream discharges (hydrology) and flow velocities and flood levels (hydraulics). As discussed in Chapter 6, there are three different gradients or slopes that are relevant in road drainage design:

Energy gradient: the profile of the energy line in a flood. While this slope is not easily measured, it is the gradient used in hydraulic calculations. It is usually estimated for use in calculations;

Water surface slope: the profile of the surface of the water. This is the slope measured by observing a series of flood levels along the waterway. In open channels, the water surface slope is also the Hydraulic Grade Line (HGL);

Bed slope: the profile or slope of the bed of the channel. This slope can be measured from survey data or topographical maps. While not directly used in the hydraulic analysis, for reasonably uniform channels bed slope can be used to approximate the water surface slope and energy gradient.

Other terms used for bed slope are “ground” or “catchment” slope. The value is a representative slope for the whole catchment. Higher gradients lead to greater flow velocities, which result in lower flood levels, but increased risk of scour.

2.24 Erosion and Sediment Control

One of the most important environmental concerns for road drainage is erosion and sediment control. This should be considered in all situations and appropriate assessment and mitigation measures must be supplied. Scour at drainage structures can be a serious

environmental problem as well as providing a risk of structure failure and possible road embankment failure.

Control of scour at culverts and channels needs to consider the permissible flow velocities noted in Table 2.3, which indicates the velocity limits where scour begins to become a problem. While these are good guidelines, each individual situation needs to be considered on its own merits, since there may be a large variation for different situations. Where necessary, erosion control measures will be needed and these are described in detail in later sections of this manual.

2.25 Tailwater Levels and Backwater Potential

Tailwater is important for drainage design, as it sets the water level at the outlet of a drainage structure. It can therefore control the hydraulic performance of the structure.

Tailwater levels must be calculated as part of the hydraulic design for all drainage structures. There are a number of situations required for the calculation of tailwater which is as follows:

Normal stream depth: In this case the tailwater level is defined by the normal water level in the downstream channel, and this depends on the conditions of the stream. These conditions are the slope, channel geometry and stream roughness. The tailwater level is calculated using Manning's Equation, backwater analysis or stream rating curve.

If there is a downstream confluence (junction) with another stream, the tailwater level may be held at a higher level than would naturally be the case. In this case, the flow is at a lower velocity and the water levels are higher, which means that the culvert will not operate as efficiently as it would if the downstream water level was lower. This is especially so if the road crosses a tributary just before this tributary joins a major stream.

Two cases need to be analysed. Firstly, a major flood in the downstream catchment of the major stream may result in a higher flood level in the tributary, which may be critical for the design. Secondly, during normal to low flows in major stream, a local catchment flood in a tributary may result in lower flood levels but may also provide a critical case for the consideration of velocities through the structure;

Similarly to the tributary situation, a downstream lake or dam can affect the tailwater level. In this case, the stream flows into a lake, natural or artificial, and this body of water holds up the flood levels and thereby increases the tailwater level. This increase can occur over time, giving a dynamic tailwater. Also, another infrastructure crossing or artificial constriction downstream of proposed crossing can affect tailwater levels.

2.26 Pollution Control

While roads may make up a relatively small proportion of the catchment area, they can contribute a relatively high proportion of contaminants that are washed into streams and other receiving waters.

The contaminants include a range of materials, especially sediment, metals, oils & greases, rubber and gross pollutants. The export of these contaminants may need to be mitigated by measures provided as part of the drainage system for the roads.

The Environmental Protection Act identifies an objective to protect Ethiopia's waters while allowing for development that is ecologically sustainable. This purpose is achieved within a framework that includes:

- Identifying environmental values for Ethiopian waters (aquatic ecosystems, potable water, water supply, water for agriculture, industry and recreational use);
- Deciding and stating water quality guidelines and water quality objectives to enhance or protect environmental values.

2.27 Road Closure Periods

Consideration of times of closure is important in some situations to supplement the flood immunity assessments. The time of closure is a measure of the disruption to traffic and in some ways is a better measure of the performance of the road. This measure can be expressed at either the average annual time of submergence or closure, the average time each year when the road is affected, or as the duration of submergence or closure.

2.28 Inundation of Adjacent Land

Roads can provide a restriction to flow across a flow path or floodplain and can cause ponding upstream. This inundation must be considered carefully (extent and duration) in the planning and design of the road and any adverse impacts identified and mitigated. These impacts are important in urban areas where development or infrastructure may be affected. However there may also be concerns in rural areas, where there may be impacts on agricultural land.

Generally, the drainage systems for roads are sufficiently large enough that the duration of ponding is not increased greatly, but this may be possible in some situations. These cases need particular attention.

2.29 Maintenance of Flow Patterns

A road is a linear structure across a floodplain and therefore may divert flow across the floodplain, especially in flat areas. This diversion may have impacts on both economic and environmental factors. Any diversions should be identified and generally minimised to maintain existing flow patterns as well as possible. In some situations diversions may be worth considering, especially where there are benefits to the cost and complexity of the drainage system, but the potential impacts must be carefully assessed to determine if they are acceptable.

2.30 Cross Drainage Design Criteria

The design criteria for cross drainage in a particular project may be set either by Regional Road Authorities or by ERA strategies and may be based on any of the following conditions:

Flood immunity - This is defined as the average recurrence interval (ARI) of a flood that just reaches the height of the upstream shoulder. In other words, the road surface remains 'dry'/is immune to flooding of set ARI. Furthermore, freeboard may be required to effectively lower the water level further to keep the pavement dry and/or provide a buffer in case of error in calculation. Another definition used is the (ARI) of a flood that just reaches the point of overtopping the highest point of the road.

Trafficability: In some instances, it is desirable to allow traffic to continue to use the road while floodwater crosses its surface.

The design criteria may therefore be specified in terms of the ARI of the flood at the limit of trafficability. This limit is based on a combination of depth and velocity of flow over the road or ford and is defined as occurring when the total head (static plus velocity) at any point across the carriageway is equal to 300mm. The road is defined as closed if the flow is greater than this limit, as used below.

Time of Submergence - This is a measure of the expected time that the road is submerged in any flood but especially in a major flood such as the ARI 50 year event. Submergence is defined as the point where the road is just overtopped, even by very shallow water.

Average Annual Time of Submergence - This is a measure of the expected average time per year of submergence of the road caused by flooding. It is expressed as time per year.

Time of Closure - This is a measure of the expected time of closure of a road (road not trafficable) in any flood but especially a major flood such as an ARI 50 year event.

Average Annual Time of Closure: This is a measure of the expected time of closure of the road due to flooding, expressed as time per year.

The times of submergence and closure provide useful data to supplement the flood immunity results. They give an indication of the extent of disruption to transport that may result from flooding on the road. In some cases, low flood immunity may be acceptable if the times of closure are low and the expected disruption is relatively minor.

The impacts of these different patterns can be analysed to determine the most appropriate design for each particular crossing. The time of submergence / closure is related to catchment area and response times as well as the flood immunity. These times are calculated either from design flood events or from stream flow data, as described later in this manual.

2.31 Stream Channels Design Criteria

The following criteria apply to natural channels:

- The hydraulic effects of floodplain encroachment shall be evaluated for frequency-based peak discharges from the design frequency to the check/review recurrence intervals on any major highway facility, as deemed necessary by the designer.
- If realignment of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Stream bank stabilization shall be provided when appropriate, to minimise risk of stream disturbance, such as encroachment, and should include both upstream and downstream banks, as well as the local site.
- Features such as dikes and levees associated with natural channel modifications should have a 5 meter minimum top width with access for maintenance equipment. Turnaround points shall be provided no more than 500 meters apart and at the end of any such feature.

2.31.1 Roadside Channels Design Criteria

The following criteria apply to roadside channels:

- Channel side slopes should not exceed the angle of repose of the soil and/or lining and shall be 2:1 or flatter in the case of rock-riprap lining;
- Flexible linings shall be designed according to the method of Allowable Tractive Force;
- The design discharge for permanent roadside ditch linings should have a 10-year frequency while temporary linings shall be designed for the 2-year frequency flow. All roadside channels and/or ditches shall be hydraulically designed as per this manual; and
- Channel freeboard shall be 0.3m or two velocity heads, whichever is larger (see Chapter 6).

2.32 Longitudinal Drainage Design Criteria

The requirements for longitudinal drainage will vary from project to project. The design considerations for a site will dictate the choice between alternative longitudinal drainage options such as kerbs and channels, grassed swales, and lined or unlined table drains. It is also important that the longitudinal drainage (drain type and capacity) of the adjoining projects be considered when determining the criteria for the site being planned or designed to ensure consistency of drainage capability and to mitigate potential system failure.

In urban environments, kerbs and channels have historically been favoured for most roads, though grassed channels and swales are also common on divided roads.

The criteria below are to be considered in determining the standard for longitudinal drainage. It is important to note that the standard for longitudinal drainage should be compatible with the standard adopted for cross drainage as these two components of the drainage system typically work in combination with each other.

2.33 Shape of Side Drains

Flat-bottomed drains are the preferred type or shape of side drain. Parabolic shapes can also be used although these are difficult to construct and maintain. The use of 'V' drains is to be limited / confined to constrained sections where cross sectional width is critical. The flat-bottom of the drain is to be sloped away from the carriageway and be wide enough to allow access for maintenance machinery.

2.34 Minimum Grades

The minimum grade for unlined drains, including table drains, is 0.5% and 0.2% for lined drains however 0.3% may be regarded as the minimum practical slope for construction (allowing for construction tolerances). This is to ensure that the drain will flow and, if applicable, minimise ponding against formations and pavements.

This criterion also applies to both crest and sag vertical curves where grades fall below 0.5%. Generally, to achieve the required minimum grades, widening of the table drains is needed over the critical length (i.e the length where grade is less than that required). Widening of the table drain means that when travelling away from the vertical curve apex, the table drain invert is gradually shifted away from and then back closer to the shoulder edge, in order to deepen the drain and affect sufficient grade. However, this solution may not always work, therefore modification/adjustment of the road geometry may need to be made.

2.35 Flow Velocities

Flow velocities in longitudinal drainage should be limited to prevent erosion. Limiting flow velocities is preferred over maintaining high flow velocities and providing armouring. Acceptable velocities should be based on the soil conditions and characteristics of the site.

2.36 Flow Depths

Flow depths should be limited to prevent erosion and inundation of the pavement. An increase in the number of outflow points (e.g. turnouts or level spreaders) from the longitudinal drainage should be considered to assist in managing depth of flow.

2.37 Median Drainage

Median longitudinal drainage will usually have a concrete lined invert to assist maintenance and reduce the risk of errant vehicles rolling after hitting ruts caused by tractor mowing.

2.38 Bridge Run-off

Road run-off from bridge scuppers should be discharged into a sediment basin, gross pollutant trap or other relevant first flush containment removal device. This is particularly important where the scupper would direct bridge run-off into a base flow channel or upstream of a sensitive environment (e.g. wetland or fish habitat reserve).

2.39 Road Surface Drainage

The requirements for surface drainage primarily relate to safety (e.g. aquaplaning and ponding). The main design criterion is the allowable flow width on the road. However flow velocity also needs to be addressed, particularly when pedestrian movement is adjacent to or crosses the flow.

2.40 Immunity Criteria for Roads in Rural Catchments

For rural catchments, the generally accepted design criteria for various drainage components are specified in Table 2-4. In some situations, it might not be possible to design for this level of flood immunity without causing unacceptable impacts on existing development or because of extensive flooding that could not be managed without unacceptable cost. In such situations the ARI may be relaxed to a lower level. In this instance, assessment and use of time of closure / submergence for design criteria may be more appropriate.

This criterion also applies to rehabilitation and reconstruction projects where existing structures are assessed as hydraulically or structurally deficient and need to be completely replaced. Designers should check ERA strategies for flood immunity or trafficability requirements for specific routes and individual projects.

Table 2-4: Design ARI for Rural Road Surfaces

Location	Frequency (years)
Road surface drainage [1]	10
Bridge deck drainage	10
Road surface drainage of pavements	1
Water quality treatment devices	1

[1] Includes kerb and channel, table drains, diversion drains, batter drains, and catch drains

2.41 Immunity Criteria for Roads in Urban Catchments

The design ARI for a project in an urban area will often be influenced by the capacity or capability of the existing drainage system or network that the new work needs to connect into. For urban catchments, the generally accepted design criteria for various drainage components are specified in Table 2-5. Designers should confirm the requirements of any existing / connecting systems with the relevant authority.

Urban drainage systems are generally based on the major / minor drainage system or dual drainage system. This type of system or drainage concept has two distinct components:

- The minor drainage system is designed to fully contain and convey a design minor storm water flow of specified ARI with road flow limited in accordance with the requirements set out in Chapter 10 of this manual;
- The major drainage system conveys the floodwater beyond the capacity of the minor drainage system and up to a specified ARI.

The minor and major design storms correspond to the rainfall events for the ARI chosen for the design of the minor and major systems respectively. Designers should note that the design discharge for the major system ARI may require that the capacity of gully inlets and underground pipes be increased beyond that required by the design discharge for the minor system ARI, in order to meet the major system design criteria.

Another important design consideration is that with any proposed drainage system adjacent to sensitive areas where flood inundation will not be tolerated, the design of the major drainage system should also consider the flow conveyed in the underground minor drainage system should this system fail due to malfunction or blockage.

Table 2-5: Design ARI for Urban Road Surfaces

Location	Frequency (years)
Major system - includes all above and below ground components	50 or 100 years
Minor system components	
Cross drainage excluding fords	50
Diversion channels	50
Road surface drainage including intersections [1]	10
Bridge deck drainage	10
Sediment basins	2
Road surface drainage of pavement	1
Water quality treatment devices	1

[1] Includes kerb and channel, table drains, diversion drains, batter drains and catch drains.

2.42 Environmental Criteria

The environmental considerations and strategies for managing aspects of a project that are predicted to cause environmental harm will most likely become environmental criteria for the project. Chapter 3 deals further with the development of environmental criteria.

2.43 Water Sensitive Urban Design

Water Sensitive Urban Design (WSUD) is a particular issue for urban planning and design, but the key principles of WSUD are also applicable to road infrastructure in the rural environment. These principles are:

- Protect existing natural features and ecological processes;
- Maintain the natural hydrological behaviour of catchments;
- Protect water quality of surface and ground waters; and
- Integrate water into the landscape to enhance visual, social, cultural and ecological values.

Conventional water management has been compartmentalised with water supply, wastewater and storm water traditionally being treated as separate entities. However integrated water management needs to consider the total water cycle and this concept is increasingly being accepted and/or adopted.

Roads may represent a relatively small proportion of the total catchment, but they sometimes contribute significantly to water quality concerns. This is especially the case on roads with high traffic volumes, where a number of different contaminants may be produced. Between rainfall events, contaminates can build up and then run off at a greater rate than normal into receiving waters.

The principles that need to be considered include:

- Consider all parts of the water cycle, natural and constructed, surface and subsurface, recognising them as an integrated system;
- Consider all requirements for water, both anthropogenic (human activity) and ecological;
- Consider the local context, accounting for environmental, social, cultural and economic perspectives;

- Include all stakeholders in the process;
- Strive for sustainability, balancing environmental, social and economic needs in the short, medium and long term.

The engineer also needs to be aware of all water related issues, not only in the road reserve, but both upstream and downstream.

2.44 Extreme Rainfall Events

While the planning and design of road drainage systems is based on a determined average recurrence interval or set of average recurrence intervals, it is also a requirement to review designs for possible adverse outcomes that may occur during an extreme rainfall event.

To illustrate this, most roads are designed to an ARI 50 year standard. However, should an ARI 100 year or larger event occur, culvert velocities may become unacceptably high causing significant environmental harm; afflux may increase above the acceptable ARI 50 year limit causing excessive flooding; the road may overtop threatening the integrity of the road embankment, safety of road users; and so on.

The extent of extreme events to be analysed depends on particular circumstances, so the requirements cannot be defined exactly. Furthermore, while the risk of occurrence of these extreme events is low, the impacts of an extreme event must be assessed.

In the case of the event occurring and the adverse outcomes / risks being unacceptable, the design criteria may need to be altered and the design recalculated or appropriate mitigating measures developed and included into the project.

It is important to note that any outcomes (adverse or otherwise) resulting from an extreme rainfall event could occur within both the road and external environments, therefore identification of possible outcomes should not be limited to the road reserve and/or change limits of the project.

2.45 Erodible Soil Environments

Part of the road drainage design process is the determination of acceptable or maximum allowable velocities for storm water flows. It should be noted that these velocities are largely based on research that identified the velocity when erosion/scour started to occur in different soil/stream types. The maximum allowable velocities for a project are then used in the design of various drainage structures/devices (for example, culverts and channels) to ensure design discharge through those devices is below the set maximum allowable velocity for that location.

Some design solutions that may be adopted, equal or are set just below the maximum allowable velocity. If an extreme rainfall event occurs, the maximum allowable velocity for a given structure/device will most likely be exceeded. This in turn could result in excessive scour, erosion or environmental harm. It is therefore important that these situations are identified and assessed. If this situation is considered applicable on a project, specialist advice needs to be sought from ERA or a suitably qualified consultant as analysis methods are beyond the scope of this manual.

2.46 Excessive Flooding

Larger floods may need to be considered in locations where the impacts of the road on flood levels (based on a normal design ARI) are/will be significant/very severe. These impacts will most likely be worse in a large flood/extreme rainfall event. This issue is

particularly important where the road embankment is relatively high and the flood immunity provided by the high embankment is much greater than the usually adopted standard of ARI 50 years.

In this case, while larger floods may not overtop the road, a higher peak water level will build up on the upstream side of the road causing excessive flooding and in some cases may cause the overtopping of the catchment boundary, directing or diverting flow to an area not able to handle the increased flow. Furthermore, the higher peak water level may produce larger flow velocities through the drainage structure, which has been designed for a smaller ARI.

The higher velocity may cause scour problems or could cause the catastrophic failure of the structure itself. The above issues may be further aggravated by blockage of the drainage structure(s) (by silt and/or debris) which may lead to a greater risk to the drainage infrastructure and surrounding area, if the flow cannot overtop the road.

Therefore, where flood impacts will be significant/very severe, it is necessary (and can be specified in design/contract documentation) to consider floods up to the Probable Maximum Flood (PMF). The PMF is defined as the largest flood event that can reasonably be expected from worst climate conditions. In some situations, extreme events, though smaller than the PMF, may be more appropriate. If the scenario of excessive flooding is considered applicable on a project, specialist advice needs to be sought from ERA or a suitably qualified consultant.

2.47 'Self Cleaning' Sections

'Self cleaning' sections, for example, culverts and channels, require a reasonably regular flow of specific energy, that will pick up and transport any silt or debris within the section to a specific location beyond the section.

The required minimum velocity/energy for a 'self cleaning' flow through the section must be determined based on the anticipated sediment and/or debris (type / size / weight) that may accumulate in the section. This flow must be generated by a design storm with a suitable ARI such as ARI 1, 2 or 5 years depending on how often the channel should be 'cleaned'. The requirement for 'self cleaning' sections and the selected design interval (ARI) must be specified in the design brief/contract documents. The location that any silt or debris can be transported to (and deposited) must also be considered as it must be accessible to allow maintenance/clean out not to cause any adverse effects to the environment (for example, water quality); and not adversely affect any future flows (for example, cause ponding/increase tailwater levels).

The inclusion/presence of a 'self cleaning' section does not remove or lessen the requirement for regular/routine maintenance inspections. 'Self cleaning' sections may reduce the requirement for maintenance (cleaning) of the section.

2.48 Coordination

Since many levels of government plan, design, and construct highway and water resource projects that might have an effect on each other, interagency coordination is essential and necessary. In addition, agencies can share data and experiences within project areas to assist in the completion of accurate hydrological analysis. Coordination between ERA, Ministry of Water & Energy, Ministry of Agriculture, Mapping Agency, Local Authorities and Environmental Protection Agency is essential.

2.49 Departures from Standards

It is anticipated that there will be situations where the designer will be compelled to deviate from the standards specified in the manual. This can be, for example, financial, political, topographical etc. Where the designer departs from a standard, he/she must obtain written approval from ERA. The following information shall be submitted:

- The ID number, name, location and description of the road;
- The facet of design for which a Departure from Standard is desired;
- A description of the standard, including normal value, and the value of the Departure from Standard;
- The reason for the Departure from Standard; and
- Any mitigation to be applied in the interests of safety.

The certifying Drainage Engineer will submit all major and minor Departures from Standards to the Design and Research Division Director of ERA for evaluation. If the proposed Departure from Standard sufficiently meets the needs of the desired services to be provided, the Departure from Standard is submitted to the Director General for final approval. This review is to ensure plans for proposed roads projects provided for a facility will adequately meet the existing and probable future needs and conditions in a manner conducive to safety, durability, and economy of maintenance; and be designed and constructed in accordance with standards best suited to accomplish the foregoing objectives and to conform to the particular needs of each locality. All Departures from Standards will be documented in the design and project file. All individual comments will be submitted to the Design and Research Director of ERA for finalization.

2.50 Documentation

The design of highway drainage facilities must be adequately documented. Frequently, it is necessary to refer to plans and specifications long after the actual construction has been completed. Therefore, it is necessary to document fully the results of all hydrological analyses and hydraulic modelling results as well as the hydrological and hydraulic modelling reports and calculation sheets. It is recommended that all ERA consultants submit their work in both soft and hard copy so that the project data can be documented in ERA's central database system.

2.51 References

Ethiopian Roads Authority (ERA) 2002, Drainage Design Manual.

Australian Drainage Manual, 2010, Second Edition.

South African Roads Agency Ltd, Drainage Manual, 2007.

APPENDIX 2A – HYDRAULIC MODELING PROCEDURE AND REPORT TEMPLATE

It should be recognised that it is not always necessary to produce a hydraulic model for all channel, culvert and bridge design analysis. A decision on whether to construct a hydraulic model should be made based on the scale and nature of the potential flood risk, as well as the scale of the project and the existing information available on flood risk to the waterway crossings. In many less complex projects, simple hydrological and hydraulic analysis may be all that is required.

If there is any doubt whether a model is required, this should be discussed with ERA staff (drainage design team) at the earliest opportunity. Requirements at specific locations should always be discussed with local ERA staff to ensure that any site-specific factors are identified, which may require special treatment when carrying out the hydraulic modelling.

The following procedure is recommended to be followed:

Objectives of the Model Study

The objectives and the required outputs of the modelling exercise should be defined at the outset. These should be reviewed at regular intervals during the drainage structure design stage and at completion. At an early stage, the design condition should be clarified. This may, for example, include a freeboard and an allowance for climate change.

Data Collection

It is recommend that all relevant data be collected before starting to build a hydraulic model for the proposed watercourse crossing. Required data sources include channel survey, topographic survey, historic flood events, hydrometric data, existing study report, flood levels, flood extents and flows. These data sets are detailed below.

Hydrometric Data

The Ministry of Water & Energy holds existing hydrographic and river basin master plan reports, which may be of use in a flood risk assessment for the proposed crossing.

River flow, river level and rainfall data relevant to the model should be collected where these exist. An understanding of the uncertainty and confidence within this data should be sought from its owners and further developed.

Historic Information

Information on historic flooding (e.g. newspaper articles, photos, flood marks) should be collected and utilised to guide the survey extent and to aid the modelling process. Such data is particularly valuable as it can provide information on historic flooding prior to the periods covered by hydrometric data. However, the effect of any alterations and additions to the watercourse and associated structures since the date of the recorded event needs to be considered.

Previous Hydrological and Hydraulic Modelling Reports

It is advisable to contact ERA and other stakeholders for information for any previous study if existing study is available for your area of interest. Where existing studies are available, consideration should be given as to whether these could be used as part of the hydraulic analysis for the proposed crossing. Data from the ERA and other stakeholders will be supplied with any relevant data warnings or disclaimers, which must be considered if using other data. You should be aware that there may be cost, licensing and intellectual

property rights (IPR) issues associated with the use of models which will need to be resolved before any previous study is used.

If previous studies or survey data are provided by ERA or third parties it is recommended that check surveys are undertaken at key locations to ensure that the data provided is compatible with current conditions. If ERA does not own the Intellectual Property Rights to hydraulic analysis completed by third parties, ERA may not be able to release information with a license for its use.

Choice of Model Software

The modelling software chosen should be capable of producing the required output. It will generally be appropriate to choose commercial hydraulic/river modelling software that is in widespread use. However, HEC-RAS can be used as standard software for ERA projects. In certain circumstances, for example where the applicability of a model to a specific situation has not been previously demonstrated (this is a case in Ethiopia), it may be necessary for those conducting the bridge analysis to have independent benchmarking tests carried out to demonstrate model performance using standard data.

Type of Model

The choice should be made between a fully hydrodynamic one-dimensional (1D) or two-dimensional (2D) model or a steady-state backwater model, flood routing model or combination of methods. A full hydrodynamic model must be used if the study area contains either structure whose operation varies with time (e.g. pumps, sluices etc.). This should be employed in complex fluvial situations and where the watercourse is subject to rapid increases and decreases in flow. If there is significant floodplain storage and complex flow routes on the floodplain then 2D modelling of the floodplain may be more representative. In other cases, either a steady-state or hydrodynamic model may be chosen. It should be noted that a steady-state model is unlikely to give a reasonable estimation of water levels where storage is present.

Hydrological Assessment

A hydrological assessment of the design flood flows should be made using the methodology described in Chapter 5.

Hydraulic Model Building

The hydraulic model should be built to represent the key flood flow routes, flood storage and structures in the study area. The defined study area should be sufficient to demonstrate the effects of any development on locations upstream and downstream from the site of the proposed waterway crossing. Bridge and culvert blockage scenarios should be considered if appropriate.

Upstream Boundary (Inflows)

The upstream boundary or boundaries should be developed under the hydrological assessment described in Chapter 5. For some models, one single upstream inflow per flood event may be sufficient, whilst for others; many upstream boundaries may be needed if a number of tributaries or other inflows are present. The choice of location of the upstream boundaries should be based on hydraulic considerations, not on the upstream limit of the crossing site. The upstream boundary should be far enough upstream to allow the full impact of the hydraulic structure on upstream water levels to be identified.

Downstream Boundary (Levels)

The downstream model boundary should be at a location where the relationship between water level and flow is well defined, e.g. a weir. Where this is not possible, it should be sufficiently downstream of the area of interest so that any errors in the model boundary will not significantly affect predicted water levels at the bridge site. For a typical fluvial river, a rule of thumb is that a backwater effect extends a length L (m) =0.7D/s, where D (m) = bank-full depth and s = river slope. Hence, if the downstream boundary is greater than L from the site it is likely that any errors in the rating curve at the boundary will not affect flood levels at the bridge/culvert site.

Hydraulic Coefficients

The coefficients used in the model (e.g. channel roughness, weir coefficients) should be determined with guidance from standard textbooks. These texts should be referenced in the modelling report. Research is required in order to produce hydraulic roughness guidance relevant to Ethiopia, but in the meantime, standard works such as Chow and Hicks & Mason can provide some guidance.

Calibration

Wherever practicable, the hydrological assessment and the hydraulic model should be calibrated against recorded flows and/or water levels from observed flood events. If calibration data is available, the model should be calibrated using at least three separate events. If no calibration data is available, a 'reality check' on the predicted levels and flows can often be carried out from photographs, historic information and anecdotal accounts of flooding.

The coefficients used in the calibration process should only be varied within the possible ranges suggested in the standard textbooks. The calibration of steady-state models should consider flow and flood levels. Calibration of hydrodynamic models should also consider the timing of the flood peak, flood volume and shape of the flood hydrograph.

Verification

If calibration is carried out, at least one separate observed event should be run through the model after the calibration to verify the adjustment of parameters.

Sensitivity Testing

The model should be tested by adjusting the key parameters within it to assess the effects on calculated flood levels. Unless otherwise agreed with the ERA, the following parameters should be tested as a minimum:

- Estimated model inflows
- Model downstream boundary condition
- Channel roughness and
- Key structure coefficients.

The range of parameters used in sensitivity tests should reflect uncertainties, possible changes due to climate change and variations in hydraulic coefficients (e.g. from seasonal changes or periodic maintenance). Sensitivity to blockage of critical structures should also be tested.

Bridge Hydraulic Model Report Requirements

A report must accompany the submission to describe the modelling method and assumptions made. The report is to enable a review of the model and results to be carried

out. In some cases, only the report will be used to evaluate the appropriateness of the model, therefore it must be thorough. It should be a self-contained report that will provide sufficient information to allow future use of the model by ERA including if necessary replicating the work undertaken. The detail of the report should be appropriate to the complexity of the modelling work at the crossing site.

Format of Reporting

The report should be in a format that is easy to transmit electronically, and must include all plans and schematics. Adobe pdf files are therefore preferred. The language should be clear and non-technical where possible.

The following plans should be included with the report:

- Location plan at an appropriate scale, with national grid coordinates and topographical base mapping, identifying geographical features, street names and all watercourses or bodies of water in the area of the site; and
- Plan and description of any structures which may influence local hydraulics.

Report Structure

For a comprehensive report, it is recommended that the following report structure, in line with the model requirements be followed:

Introduction

General Site Description:

- Larger scale plan showing location of the drainage structure in the catchment;
- What the site is used for currently;
- Size of the site;
- What hydraulic structure is proposed?
- Whether ERA has been involved with the site previously (existing design report);
- Brief Flood History of the site;
- Source of flooding on site/mechanisms of flooding;
- Location of watercourses/drainage ditches in the area;
- Location of rainfall gauge stations in the area; and
- Location of stream flow/level gauge stations in the area.

Objectives of the Model Study

Provide a justification for why the modelling exercise has been undertaken and the planned objectives of the exercise. Indicate any deviations from the original objectives or planned project outputs, and outline the reasons why these occurred.

Method Statement and Justification

The report should include a clear method statement, detailing how the modelling has been carried out to fulfil the objectives of the project.

Data Sources

List all data sources used in the model and provide these when submitting the hydrological and hydraulic analysis report. Detail methods of data capture and/or sources of data, and the processes by which the raw data were converted. Any reference to earlier work should be clearly referenced, and applications or development of existing models should be subject to the same rigorous inspection methods. State the ownership of the data collected

and the format of the data. Uncertainty in data sources should be referenced especially where data have been discounted due to low confidence.

Hydrological Model

Explain why the chosen methodology is suitable for the catchment. Report details of decisions made and justifications for these. The report must include a table of the design inflows to be used in the hydraulic model. A complete description of the catchment areas contributing to flooding at the study site must be supplied.

Hydraulic Model

A hydraulic model will need to be produced for a hydraulic analysis where the effect of flood risk to the site can not otherwise be demonstrated (existing information, hand calculations etc). It will be necessary to produce a hydraulic model where the flood risk before and after the watercourse crossing structure needs to be demonstrated, if the project involves changes to the river channel or structures, or if the structure includes flood storage.

Provide a description of the hydraulic modelling approach including a description of the watercourse being modelled. The discussion must include justification of the selected modelling software including a technical description of the model. Only a brief technical description is required if the software is well known to ERA/widely applied, such as ISIS, Mike 11, TUFLOW and HEC-RAS. Include the name and version of the software used.

Justify the decision to use fully hydrodynamic 1D or 2D model or a steady-state backwater model, flood routing model or combination of methods. Indicate any perceived advantages or disadvantages of applying the chosen tool. Supply details of existing drainage structures and how they have been represented in the model. Provide the schematic showing how individual parts of the model are connected, as an appendix.

Parameters

State and justify the derivation of the parameters (e.g. channel/overbank roughness, weir coefficients) used within both the hydrological assessment and the hydraulic model.

Calibration/Verification

Where calibration has been undertaken, the method used must be clearly illustrated and the number of independent data sets used for verification must be displayed. The model results must be presented against observed values for key locations for each verification data set, and descriptive statistics applied to describe the error band in the model.

Sensitivity Analysis

Describe the results of the sensitivity testing and discuss the potential effect these could have on the model output.

Results

Results of the hydraulic model should be indicated in a summary table showing roughness coefficients, peak flow, water surface elevation, flow velocity, Froude Number etc. at each cross section. If possible, calculated flood levels could be shown on cross section data.

Map(s) indicating the flood extents adjacent to and including the proposed crossing site must be provided for the modelled design events.

Audit Trail

The audit trail developed should be described in unambiguous detail. This should detail the model build stages, changes made and the file names of all modelling/model support files produced. Documentation should also be included within the model data files to clearly set out the conditions applied.

Limitations

Highlight and discuss any limitations of the model or modelling technique. The impact of such limitations on the present or future use should be clearly stated. Data given to multiple decimal places gives the impression of high confidence in the accuracy. Avoid doing this unless you are able to state the accuracy and confidence in the data.

Conclusions

The report must include concluding remarks, which highlight key issues from other sections and draw attention to the critical locations and/or structures within the model.

The same key items in reporting will apply to both modelling and hydrology. The conclusion should comment on the current flood risk to the crossing site and the level of risk post construction of the crossing structure. It should also comment on the existing flood risk to locations upstream and downstream of the site and any changes to the level of risk to these areas following the road project.

Appendices

Additional items to include as appendices:

ERA and other stakeholder data used in the analysis;

Copy of the data license: Include a copy of the license/copyright which accompanies the data provided by the ERA and other data providers;

If an ERA previous study has been used/adapted as part of the analysis, include the study disclaimer, which was provided with the data. This is to ensure any data warnings have been regarded.

Appropriate Drainage Staff Involved

Include a description of experience/CV of drainage expert staff involved with the analysis. This is to demonstrate to ERA that suitably qualified and experienced personnel have carried out the work described in this document.

Quality Assurance and Audit Trail

Throughout the study, a well-defined audit trail should be defined and reported. This should include all relevant documentation and should link with the appropriate quality assurance procedures of the organisation carrying out the study. Provision should be made to make the relevant documentation available to others who may use the study in future.

3 POLICY AND PLANNING

3.1 Policy

3.1.1 Introduction

This chapter provides guidance on the assessment and management of the impacts that road projects may have on the water environment. These include possible impacts on the quality of water bodies and on the existing hydrology of the catchments through which roads pass. Where appropriate, the Standard may be applied to existing roads.

Flooding from rivers and ditch systems is a natural process that plays an important role in shaping the natural environment. However, flooding threatens life and causes substantial damage to infrastructure (roads, highway etc.) and property. The effects of weather events can be increased in severity both as a consequence of previous decisions about the location, design and nature of settlement and land use, and as a result of future climate change.

Although flooding cannot be wholly prevented, its impacts can be avoided and reduced through good planning and management. Climate change over the next few decades is likely to mean increased wetter and dryer seasons within the various regions of Ethiopia. These factors will lead to increased and new risks of flooding within the lifetime of planned schemes.

All forms of flooding and their impact on the natural and built environment are all planning considerations. Planning should facilitate and promote sustainable patterns of development, avoiding flood risk and when unavoidable (river crossings) manage the risk accommodating the impacts of climate change.

While water is vital for all living plants and animals it is crucial importance for industry and Agriculture. The Government is committed to maintaining and, where justified, improving the quality of water bodies (surface waters and groundwater). It also attaches great importance to the management of flood risk in the planning process, and taking account of climate change. To achieve these aims, the Government sets standards for protection of the water environment and passed laws to prevent its degradation.

Roads are designed to drain freely to prevent build-up of standing water on the carriageway whilst avoiding flooding. Contaminants deposited on the road surface are quickly washed off during rainfall (first flush). Where traffic levels are high the level of contamination increases and therefore, the potential for unacceptable harm being caused to the receiving water also increases. Although there are many circumstances in which runoff from roads is likely to have no discernible effect, a precautionary and best practice approach indicates the need for the assessment of the possible impact of discharges from proposed roads.

This chapter provides guidance on the governance, legislative and policy contents associated with new road construction together with the impacts on the water environment. These include possible impacts on the quality of water bodies and on the existing hydrology of the catchments through which roads pass. The issues to be considered for any new road scheme are as follows:

- Assess the impact of flood risk (surface water/overland flow, fluvial, and groundwater flooding);
- Erosion and sediment load; and
- Pollution impacts from spillages.

Planning should facilitate and promote sustainable route alignments while addressing the impacts of climate change. This requires all members of the planning and design teams to be present when key decisions are to be made which will ensure an informed, clear and transparent decisions making process. This will also ensure that all risks are identified at an early stage by the various disciplines in the planning and route alignment stage and can be managed in a sustainable manner.

3.1.2 Governance, Legislation and Government Policy

Governance Structure

Ethiopia's current constitution ratified in August 1995 established a federal structure based of nine regional states which gives them rights to govern themselves for the most part (Article 39 of the constitution1). The governance structure of a Region is comprised of, Zone, Woreda and the Kebele levels. The Regions, Zones, Woreda and Municipalities have to varying degrees constitutional powers and duties, however the Kebeles do not. The purpose of the Kebeles is to provide a point of contact for the citizens of Ethiopian when it comes to public engagement and getting their individual or collective voices heard. Refer to Figure 3.1 Ethiopian Governance Structure below.

[1] Proclamation No. 1/1995 - Proclamation of the Constitution of the Federal Democratic Republic of Ethiopia

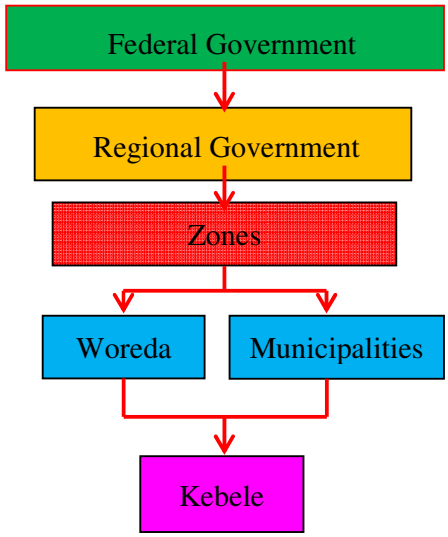


Figure 3-1: Ethiopia Governance Structure

Legislation and Government Policy

The overall aim of the legislative and policy criteria of Ethiopia is to improve and enhance the health and quality of life of all Ethiopians and to promote sustainable social and economic development. This is to be achieved in a sustainable manner so as to meet the needs of the present generation without compromising the ability of future generations to meet their own needs.

The concept of sustainable development and associated environmental rights in Ethiopia are outlined in articles 43, 44 and 92 of the constitution of the Federal Democratic Republic of Ethiopia[1] dated August 1995. The aforementioned articles state the following:

"The Peoples of Ethiopia as a whole, and each Nation, Nationality and People in Ethiopia in particular have the right to…."

Article 43: The Right to Development

- Improved living standards and to sustainable development;
- Participate in national development and, in particular, to be consulted with respect to policies and projects affecting their community;
- All international agreements and relations concluded, established or conducted by the State shall protect and ensure Ethiopia's right to sustainable development; and
- The enhancement of their capacities for development and to meet their basic needs are boldly recognized.

Article 44: Environmental Rights

- A clean and healthy environment;
- Compensation or alternative means of compensation, including relocation with adequate state assistance.

Article 92: Environmental Objectives

- Government shall ensure that all Ethiopians live in a clean and healthy environment;

- Programmes and projects of development shall not damage or destroy the environment;
- People have the right to full consultation and to the expression of views-in the planning and implementation of environmental policies and projects that affect them directly; and
- Government and citizens have the duty to protect the environment.

To provide the legal framework by which the above sustainable development and environmental objectives are adhered to, a number of proclamations have been passed which place a duty on all to ensure sustainable and environmentally-friendly road construction. The relevant Proclamations are identified below:

"Environmental Protection organs Establishment proclamation (proc.no.295/2002)[2]" stipulated the need to establish a system that enables to foster coordinated but differentiated responsibilities among environmental protection agencies at federal and regional levels. The proclamation also required the establishment of Sectoral and Regional Environmental, Units and Agencies, respectively. This shows that institutionalizing and mainstreaming environmental concerns has a legal foundation.

The **"Environmental Impact Assessment Proclamation (Proc. no. 299/2002)[3]"** made the Environmental Impact Assessment (EIA) a mandatory legal prerequisite for the implementation of major development projects, programs and plans. This proclamation is a proactive tool and a backbone to harmonizing and integrating environmental, economic, cultural, and social considerations into a decision making process in a manner that promotes sustainable development.

The "**Environmental Pollution Control Proclamation (Proc. no. 300/2002)[4]"** is incorporated within Ethiopian law with the aim to eliminate or, when not possible to mitigate pollution as an undesirable consequence of social and economic development activities. This proclamation is one of the basic legal documents, which need to be observed when undertaking an EIA and monitoring discharge of conditions associated with any authorisation.

The **"Definition of Powers and Duties of the Executive Organs of the Federal Democratic Republic of Ethiopia (Proc. no. 471/2005)[5]"** provides the definition of powers and duties of the executive organs of the federal democratic republic of Ethiopia. The proclamation also established various ministries. While all ministries are important the ones of particular relevance to roads are as follows:

- The Ministry of Transport;
- The Ministry of Water and Energy;
- The Ministry of Agriculture and Rural Development; and
- The Ministry of Mines.

[2] Proclamation No. 295/2002 - Environmental Protection Organs Establishment Proclamation.

[3] Proclamation No. 299/2002 - Environmental Impact Assessment Proclamation.

[4] Proclamation No. 300/2002 Environmental Pollution Control Proclamation.

[5] Proclamation No.471 /2005 - Definition of Powers and Duties of the Executive Organs of the Federal Democratic Republic of Ethiopia.

The **"River Basin Councils and Authorities (Proc. No. 534/2007)[6]"** looks to protect the 12 river basins as the country's economic growth causes an increase in water use. It is envisaged that river basin councils and authorities will be one of the main instruments to implement integrated water resources management, which is a pillar of the policy. Integrated water resources management process requires that the stakeholders of a river basin shall have to act in a coordinated manner in spite of their differences of approaches, interests and perceptions of the effects of their decisions, plans and activities on the hydrological cycle and on other users.

The **"Solid waste Management (Proc. No. 513/2007)[7]"** aims to prevent the adverse impacts of waste while ensuring that social and economic benefits can be generated by the waste where possible.

The "**Environmental Policy of Ethiopia (EPE, 1997)[8]"** provides a number of guiding principles that indicate and require a strong adherence to sustainable development. In particular EIA policies of the EPE include, among other things, the need to ensure that EIAs:

- Consider impacts on human and natural environments;
- Provide for an early consideration of environmental impacts in projects and programme design;
- Recognize public consultation;
- Include mitigation plans and contingency plans; and
- Provide for auditing and monitoring as legally binding requirements.

3.1.3 Roles and Responsibilities

Environmental Protection Authority

The Environmental Protection Authority (EPA) is the government regulatory authority responsible for environmental protection. The aim of the EPA is to formulate policies, strategies, laws and standards, which foster social and economic development in a manner that enhance the welfare of humans and the safety of the environment and ensure they are implemented. The Authority shall have the powers and duties to coordinate measures to ensure that the environmental objectives provided under the Constitution and the basic principles set out in the environmental Policy of Ethiopia are realised.

The EPA will prepare, review and update, or as necessary, cause the preparation of environmental policies strategies and laws in consultation with the competent agencies, other concerned organs and the public at large and upon approval, monitor and enforce their implementation; where projects are subject to federal licensing, execution or supervision or where they are likely to entail inter- regional impacts, review environmental impact study reports of such projects and notify its decision to the concerned licensing agency and, as may be appropriate, audit and regulate their implementation in accordance with the conditions set out during authorisation.

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6 Proclamation No. 534/2007 - River Basin Councils and Authorities Proclamation.

7 Proclamation No. 513/2007 - Solid Waste Management Proclamation

8 Environmental Policy of Ethiopia - Environmental Protection Authority (1997)

Ministry of Water and Energy

The Ministry of Water and Energy (MoW&E) in Ethiopia, established in 1995, has a number of overarching powers and duties as spelt out in proclamation No.471 /2005[5]. These include initiation of policies and laws, preparation of plans and budgets, and upon approval implementation of the same.

More specifically the MoW&E is required to undertake basin studies and determine the country's ground and surface water resource potential in terms of volume and quality. In addition its duties involve the issue of permits and the regulation of the construction of any works relating to water bodies. For any works not undertaken in accordance with agreed proposals, the MoW&E will ensure the enforcement of federal laws.

Regional Level Organizations

Regional agencies in Ethiopia have been established with similar designations and responsibilities as the federal ministries described above. The major regional water sector offices have the responsibility to manage resources on behalf of MoW&E. They are also mandated to administer resources under their geographical jurisdiction, i.e. non-transboundary and non-trans-regional water bodies. Their roles and responsibilities, in relation to land and water management, include: develop region-wide polices, strategic plans, directives, standards and manuals concerning the management of water resources in line with the federal water policies and laws; issue permits etc.

3.1.4 Approval Process for Road Works Impacting on Water Bodies

In order to ensure sustainable development, it is essential to integrate environmental concerns into development activities, programmes, policies, etc. Environmental Impact Assessments are one of environmental management tools which facilitate the inclusion of principles of sustainable development aspiration well in advance.

The EA procedural guideline series aims in particular towards:

- Ensuring the implementation of the Environmental Policy of Ethiopia (EPE -1997) and compliance of Environmental Assessment (EA) related to legal and technical requirements;
- Providing a consistent and good practice approach to EA administration in Ethiopia;
- Assisting proponents and consultants in carrying out their Environmental Assessment (EA) related tasks;
- Assisting interested and affected parties, especially communities in realising their environmental rights and roles;
- Assisting Environmental Protection Organs, Competent and Licensing agencies in discharging their roles and responsibilities; and
- Establishing partnership and networking among and between key stakeholders in EA administration.

Proclamation No. 299/2002 requires an EA process for any planned development project or public policy which is likely to have a negative impact on the environment. With regard to development projects, the proclamation stipulates that no person shall commence implementation of a proposed project identified by directive as requiring EIA without first passing through environmental impact assessment process and obtaining authorization from the competent environmental agency (Art. 3(1)). In line with this, project proponents must undertake EIA and submit the report to the concerned environmental body and, when

implementing the project, fulfil the terms and conditions of the EIA authorization given to them (Art. 7).

An environmental impact study report shall contain sufficient information to enable the Authority or the relevant regional environmental agency to determine whether and under what conditions the project shall proceed (Art. 8).

The "Environmental Impact Assessment Procedural Guidance[9]" provides a list of projects that require a full EIA (Schedule 1), preliminary environmental impact study (Schedule 2), and a Lists of projects that may not require environmental impact assessment (Schedule 3). For a full list of these projects refer to Annex III – Schedule of Activities of the aforementioned document.

Whether projects require a full/partial EIA or no EIA, the impact of a road project on the environment must be assess and cover the following as a minimum:

- The impact of flood risk (surface water/overland flow, fluvial, and groundwater flooding);
- Erosion and sediment load; and
- Pollution impacts from spillages.

The assessment undertaken will need to be proportionate to the size of the project involved. To ensure sustainable development, economic growth, social development and environmental protection the projects impact must be proportionately considered. In general, a significant amount of effort is put into economic growth than dealing with environmental issues. However a balance is required to achieve the sustainability objectives.

3.1.5 Rights to Discharge to Water Bodies

One way the Environmental Protection Authority the MoW&E and its regional agencies manage and regulate the construction and operation of water works relating to the impacts on water bodies is by means of work permits as described in Proclamation No's. 299/2002 and 471/2005 "Environmental Impact Assessment Proclamation" and "Proclamation to Provide for the Definition of Powers and Duties of the Executive Organs of the Federal Democratic Republic of Ethiopia" respectively.

At present the Ethiopian Roads Authority is exempt from the need to attenuate discharges from new or improved roads to existing water bodies (i.e. water courses and ground). If pollution is occurring, the Environmental Protection Authority or the relevant regional environmental agency can under Proclamation No.300/2002 "Environmental Pollution Control Proclamation" may take an administrative or legal steps against a person who, in violation of law, release pollutant knowing or otherwise to the environment.

The responsibility for ensuring that highway discharges comply with pollution legislation rests with the Ethiopian Road Authority (ERA) or proponents, advised by their agents, consultants and contractors.

[9] Environmental Impact Assessment Procedural Guideline - Series 1 (2003)

Permits are required if any work (e.g. a new outfall, bridge repairs) is proposed that would physically affect a waterbody.

In some situations, more stringent requirements may apply to specific water bodies. For example, those areas designated and identified as environmental sensitive areas as outlined in the Guideline Series Documents for Reviewing Environmental Impact Study Reports. These environmentally sensitive areas should be treated as equivalent to Schedule 1 activities irrespective of the nature of the project as identified in the "Environmental Impact Assessment Procedural Guideline Series 1[9]" (Nov 2003).

Where a body of surface or groundwater supports more than one use, the overall requirements will derive from a combination of the most stringent criteria for any of the uses concerned. No discharge, which could cause any of the overall requirements to be breached, will be acceptable. Hence, the assessment of new roads or road improvements should include consideration of all of the uses of a receiving water body. A surface water body should be assessed not only downstream of any discharge or river crossing, but also upstream where interests are potentially present. During the planning and consultation process, the EPA, MoW&E or Regional Agencies will advise on any uses as well as any physical constraints.

3.1.6 Impact of New and Improved Road Schemes on the Water Environment

This section describes possible impacts on the water environment that may arise from a road project. These include the potential impact with respect to the risk of flooding within the catchment and the potential impact to the quality of receiving water bodies, from either routine runoff or spillages. The water bodies may be either surface waterbodies or groundwaters. The possible impact on any existing amenity or economic value of affected water bodies may also need to be considered.

There is a potential for the diffuse pollution of the water environment arising from the construction, operation and maintenance of roads. The type of pollution and consequences depend on the particular activity and local circumstances as well as the design and operational usage for any given road.

Surface Water Runoff

When considering surface water runoff from a road, it should be a prerequisite that there is not an increase in flood risk or a deterioration in the status of the receiving surface water body as determined by the EPA or relevant River Basin Plan up or downstream of the point of discharge.

At present there are no guidelines or requirement to reduce the risk of flooding up and downstream post construction (by attenuating post construction discharges at pre development rates). Currently and depending on the standard of road, the surface water drainage system is designed to cater for a 1 in 2 year up to a 1 in 25 year rainfall event with no allowance for climate change (Refer to Chapter 10 of the Drainage Design Manual - Table 10-2 Design Frequency and Spread). The main objective is ensuring that for a particular standard of road, flooding does not occur.

Road runoff is an intermittent discharge and any breach of the annual average concentrations of pollutants is only likely to persist for a short duration (minutes/hours). This may go unnoticed by standard monitoring regimes for chemical parameters but may have environmental impacts nonetheless.

Overland Flow Flooding

Overland flow is water flowing over the ground surface that has not entered a natural drainage channel or artificial drainage system (another commonly used term for this phenomenon is surface water runoff flooding). Typically, overland flow can cause localised flooding in natural valley bottoms as normally dry areas become covered in flowing water, and in natural low spots where the water may pond. This flooding mechanism can occur almost anywhere, but is likely to be of particular concern in urban areas any topographical low spot, or where the pathway for runoff is restricted by terrain or man-made obstructions.

Fluvial Flood Risk

Roads that are located within a watercourse and or within a known floodplain must be designed and constructed to accommodate fluvial events of between 1 in 2 years (50%) to 1 in 100 year (1%) events with a check for the 200 year (0.5%) event (Refer to Chapter 2 of the Drainage Design Manual - Table 2-1 Design Storm Frequency). Currently no specific allowance has been made for climate change. At present there are no guidelines or requirements on compensatory floodplain storage other than ensuring the drainage infrastructure can cater for events of between 1 in 2 years (50%) to 1 in 100 year (1%).

It should be a prerequisite that for any road works occurring within a known floodplain, compensatory flood storage works should be provided where road alignments results in a reduction of available volume of flood storage. If possible compensatory flood storage should become effective at the same point in a flood event as the lost storage would have done.

Therefore road works undertaken with a floodplain should be designed with the following in mind:

- Remain operational and safe for users in times of flood;
- Result in no net loss of floodplain storage;
- Not impede water flows;
- Not increase flood risk elsewhere; and
- Provide an allowance for climate change.

Groundwater

Where surface water runoff from a road scheme is proposed to discharges to a groundwater body, the scheme must achieve the following:

- Prevent the introduction of hazardous substances and limit the introduction of pollutants into groundwater particular water bodies that are utilised for human consumption;
- Not compromise the existing groundwater classification (where this exists);
- Not lead to sustained downward trends in the quality of the receiving groundwater; and
- Not increase the risk of groundwater flooding.

A balance needs to be struck when considering whether road runoff should be discharged to surface waters or to ground. In some cases the effect on receiving surface waters could be such that discharge to ground may be appropriate. This could apply where the discharge would aggravate an existing flooding risk, or where it could have a potentially disproportionate effect on pollution within the receiving waters.

Assessing Potential Erosion and Sediment Control Issues During Construction

At the planning stage, Environmental Assessments for construction projects should include an erosion prevention and sediment control plan. The first aim of the erosion prevention and sediment control plan should be to minimise erosion by reducing disturbance and stabilising exposed materials. The plan should then consider control measures to minimise the release of mobilised sediment which results despite the erosion control measures. This is a particular problem in Ethiopia where even after construction of projects stock piles of material are dispersed around the project site (this is a waste management issue).

Measures to prevent erosion are more effective than controlling sediment once mobilised. The potential risk from erosion and sediment control issues should be identified and reported in the Environmental Impact Statement (EIS) where construction impacts are considered.

Operation – Pollution

A broad range of potential pollutants is associated with routine runoff from operational roads. These are combustion products of hydrocarbons, fuel and fuel additives, catalytic converter materials, metal from friction and corrosion of vehicle parts and lubricants.

Particulate contaminants originating from vehicles and vehicle related activities include carbon, rubber, plastics, grit, rust and metal filings. Most organic compounds have very low solubility in water. Other materials may be deposited on road surfaces such as wind blown soils from adjacent land. Studies show that routine road runoff contains both dissolved and particulate contaminants.

A large number of studies have investigated the concentrations of contaminants in road runoff. These studies have investigated a variety of road types in a number of countries. Research into the concentrations of contaminants in road runoff shows a large variation in concentrations of those contaminants detected.

Maintenance Works

A broad range of potential pollutants are also associated with maintenance works which may range from routine cleaning of gully pots and similar entrapment structures to carriageway maintenance work. The flushing-out of gully pots has been identified as a potential source of pollutants, which may be as damaging as some spillage impacts. In addition the use of herbicides for the control of plant growth along road verges and central reservations may also lead to contamination of road runoff.

New Construction, Improvement Works

During the construction of new or improved roads or maintenance of existing roads, pollution from mobilised suspended solids is generally the prime concern, but spillage of fuels, lubricants, hydraulic fluids and cement from construction plant may lead to incidents, especially where there are inadequate pollution mitigation measures.

Management of Spillages

When considering the risk of spillages on a road and the potential pollution to the receiving environment, the following factors must be considered early in the planning and design stages:

- Identify High risk areas on road network;
- Size of pollution prevention facilities;
- The pollution prevention facilities not to flood in a 1 in 100 year event; and

- Pollution prevention facilities not to flood in a 1 in 200 year event where spillage could affect: protected areas for conservation (such as those listed In the "Environmental Impact Assessment Procedural Guideline - Series 19" (2003).

When considering the impacts on water bodies from road runoff, acute pollution is most commonly associated with spillages of vehicle fuel and substances carried on roads. It can also occur on construction sites.

3.1.7 Climate Change

A significant amount of scientific work has been undertaken within the last decade and a large body of evidence gathered to conclude that climate change is occurring within Ethiopia.

The nature of climate change at a regional level will vary, and specifically within the 12 river basins. Further work is required over the next decade to establish a baseline for the individual river basins and project trends in climate change.

The climate science community has developed a suite of models to inform decision makers on future climate. GCMs (Global Climate Models), RCMs (Regional Climate Models), downscaling techniques (both empirical and statistical), and several comprehensive reviews are available on the subject. GCMs however are our primary source of information about future climate change. The climate change projections reported in this manual uses the profile developed as part of a United Nations Development Project (UNDP), carried out by McSweeney *et al.* (2008)[10].

It should be noted that all projections are stated with reference to a 1970-99 baseline. The study uses a collection of 15 General Circulation Model (GCM) runs to produce projections of climate change for three emissions scenarios. The three emissions scenarios used in the study were A2, A1B and B1, which can be broadly described as High, Medium and Low respectively.

The figures quoted here refer to the 'central estimates' (i.e. the median results - A1B) from the 15 GCMs across the 3 emissions scenarios. Where maximum and minimum figures are quoted, they refer to the High (A2) and Low (B1) scenario model results.

General Climate Observations

Ethiopia's climate is typically tropical in the south eastern and north eastern lowland regions, but much cooler in the large central highland regions of the country. Mean annual temperatures are around 15 - 20°C in the large central highland regions (high altitude regions), whilst 25 - 30°C in the north east and south east lowlands.

Seasonal rainfall in Ethiopia is driven mainly by the migration of the Inter Tropical Convergence Zone (ITCZ). The exact position of the ITCZ changes over the course of the year, oscillating across the equator from its northern most position over northern part of Ethiopia between July and August, to its southern most position located over southern Kenya between January and February.

Unlike most of the tropics where two seasons are common (one wet season and one dry season), three seasons are known in Ethiopia, namely Bega (dry season) which extends

[10] United Nations Development Programme - Climate Change Country Profiles Ethiopia C. McSweeney, M. New and G. Lizcano

from October-January, Belg (short rain season) which extends from (February-May), and Kiremt (long rain season) which extends from June-September. In terms of rainfall regions, Ethiopia can broadly be broken down in three regions, the northern and central, southern and eastern regions.

Most of Ethiopia experiences one main wet season ('Kiremt') from mid-June to mid-September (up to 350mm per month in the wettest regions) when the ITCZ is at its most northern position. Parts of northern and central Ethiopia also have a secondary wet season of sporadic, and considerably lesser, rainfall from February to May (called the 'Belg'). The southern regions of Ethiopia experience two distinct wet seasons which occur as the ITCZ passes through this more southern position. The March to May 'Belg' season is the main rainfall season yielding 100-200mm per month, followed by a lesser rainfall season in October to December called 'Bega' (around 100mm per month). The eastern most corner of Ethiopia receives very little rainfall at any time of year.

The movements of the ITCZ are sensitive to variations in Indian Ocean sea surface temperatures and vary from year to year, hence the onset and duration of the rainfall seasons vary considerably annually, causing frequent drought. The most well documented cause of this variability is the El Niño Southern Oscillation (ENSO). Warm phases of ENSO (El Niño) have been associated with reduced rainfall in the main wet season, (July August September), in north and central Ethiopia causing severe drought and famine, but also with enhanced rainfalls in the earlier February to April rainfall season which mainly affects southern Ethiopia.

Climate Change Projections

The future climate change profile for Ethiopia reported in this manual is based on the United Nations Development Project (UNDP), carried out by McSweeney *et al.* (2008)[10] and the "Climate Change Profile – Ethiopia", carried out by McSweeney *et al*. (2010)[11].

Temperature

The central estimates of the mean annual temperature shows an increase of between 1.8 and 2.7°C by the 2060's and of 2.3 to 4.2°C by the 2090's. The maximum increases in mean temperature are projected to be between 3.1°C and 5.1°C for the 2060's and 2090's respectively.

Precipitation

The projections from the various climate models are broadly consistent in indicating an increase in annual rainfall in Ethiopia. These increases are largely a result of increasing rainfall in the 'short' rainfall season (October-November-December) in southern Ethiopia.

The central estimates of annual changes in precipitation show increases of 3 to 9 percent by the 2090's for Ethiopia as a whole. The upper end of this projection shows this increase could be as much as 42 percent.

Projections of change in the rainy seasons (February to May and mid-June to mid-September), which affect the larger portions of Ethiopia (northern/central and southern

[11] Tearfund - Climate Change Profile – Ethiopia (2010) - Robert McSweeney, Mike Wiggins and Liu Liu

regions) are more mixed; but they tend towards slight increases in the south west and decreases in the north east.

The central estimates for rainfall in the 'short' rainfall season (October-November-December) season show increases of between 17 to 36 percent by the 2090's, but up to 70 percent at the upper end of the projections. Percentage increases in the 'short' rainfall season in the eastern parts of Ethiopia are also significant.

Climate Change Allowances – Rainfall Intensities and River Flows

With the variation in precipitation nationally and no significant information on the responsiveness of the increased flows within the 12 river basins, an allowance for climate change poses a significant challenge to the country's vulnerable institutions**.** Flash floods occur regularly throughout the country, particularly after a long dry spell. More recently, in the years 1988, 1993, 1994, 1995, 1996, and 2006, major floods inflicted significant losses in terms of human life as well as on the local and national economy. Floods are occurring with greater frequency and intensity across the country due to vulnerabilities imposed by high rates of deforestation, land degradation, increasing climate variability, and settlement patterns. Large scale floods occur mostly in the lowland areas, while flash floods resulting from intense rainfall events destroy settlements in the Highlands

In making an assessment of the impacts of climate change on flooding from the land and rivers as part of a flood risk assessment, the sensitivity ranges in Table 3.1 below may provide an appropriate precautionary response to the uncertainty about climate change impacts on rainfall intensities and river flow. It is acknowledged that there is not a linearly correlation between rainfall and flood events (a 100 year rainfall event will not result in a 100 year flood event). However until more research is undertaken on the individual river basins a precautionary approach is advised.

Table 3-1: Recommended national precautionary sensitivity ranges for peak rainfall intensities and peak river flows

Parameter	1999 to 2030	2030 to 2060	2060 to 2090
Peak rainfall intensity*	10%	20%	20%
Peak river flow	10%	20%	20%

*Peak rainfall intensity based on the medium emissions scenarios A1B and median % change in time period obtained from Data Summary table within McSweeney *et al.* (2008)[10].

An allowance for peak flows, suggests that changes in the extent of flood plain are negligible in steep catchments, but can be dramatic in very flat areas.

Impact of Climate Change

In 2010 the World Bank in association with the Department for International Development UK (DFID), the governments of the Netherlands and Switzerland, and the Trust Fund for environmentally and Socially Sustainable Development (TFESSD), commissioned a report

entitled "Economics of Adaptation to Climate Change – Ethiopia[12] ". The report had two objectives which were to:

- Develop a global estimate of adaptation costs for informing international climate negotiations; and
- Help decision makers in developing countries assess the risks posed by climate change and design a national strategy for adapting to it.

The impacts of climate change, and the merits of adaptation strategies, depend on future climate outcomes. These are typically derived from global circulation models (GCMs) and are uncertain, both because the processes are inherently stochastic and because the GCM models differ in how they represent those processes. To capture these uncertainties, this study utilizes the two "extreme" GCMs used in the global track of the EACC (labelled Wet1 and Dry1), as well as two additional models that are better suited to represent climate model uncertainty in the specific case of Ethiopia (labelled here Wet2 and Dry2). The Wet1 and Dry1 are used to ensure consistency with the EACC global track; but the Ethiopia Dry (Dry2) and the Ethiopia Wet (Wet2) capture more adequately the range of variation of climate outcomes specific to Ethiopia.

The analysis focuses on three main sectors of climatic vulnerability that already affect the Ethiopian economy and are likely to be of major significance under the climate of the future. These sectors are (1) agriculture, which accounted for 47 percent of Ethiopian GDP in 2006 and is highly sensitive to seasonal variations in temperature and moisture; (2) roads, the backbone of the country's transport system, which are often hit by large floods, causing serious infrastructure damage and disruptions to supply chains; and (3) dams, which provide hydropower and irrigation and are affected by large precipitation swings.

The transport sector is impacted by climate change in two areas; standard maintenance and flood-induced maintenance. The former represents costs that are incurred due to precipitation and/or temperature changes that occur during the life span of the road. These changes represent differences in the average climate conditions that exist for the road and thus change the conditions under which the road is intended to perform on an everyday basis. The latter represents changes in extreme events and the costs associated with repairing the roads from those extreme events. Ethiopia's strategy for the road sector stated that the total road length in the country was 56,113 km as of April 2006. Unpaved roads represent about 85 percent of the total road length (47,612), while paved roads represent the remaining 15 percent.

Improvement to and maintenance of transport links between urban centres, to and from ports of export and import, and in particular to rural areas are a prerequisite for economic development. However transport links, both paved and unpaved roads, are highly vulnerable to the increases in rainfall and temperature which are projected for Ethiopia. The projected increases in rainfall high temperatures and flood damage to road indicate that adaptation to climate change is necessary.

It is clear from the outputs of the World Bank report "Economics of Adaptation to Climate Change – Ethiopia" (2010)[12] Aziz Bouzaher *et al* that climate change will increase the maintenance costs of the country's road due to the fact that for each climate scenario assessed, climate change impacts will increase. The longer adaptation is delayed, the

[12] World Bank - Economics of Adaptation to Climate Change – Ethiopia (2010) Aziz Bouzaher *et al*

greater the expense that must be incurred doing reactive maintenance. These costs will be reduced and transport links maintained if road drainage and bridge designs adopt expected climatic conditions.

The IPCC also reports that while, some climate models indicate increases and some decreases in terms of annual precipitation in Ethiopia, all models suggest increases in precipitation over the longer period. This implies more flooding even in scenarios that suggest more drought. Both increased flooding and increased drought are projected by the same scenarios.

What this means for example is that the frequency of more extreme flood events will occur more frequently; for example, what originally was a 70-year flood may occur more frequently, such as a 50-year flood. This will translate to damage becoming more severe on a more frequent basis[12].

The policy and legal context for this vision and the EPA's role as Ethiopia's lead agency on climate change are drawn from the National Environmental Policy and the Environmental Protection Organs Establishment Proclamation No. 295/2002. Although the environmental policy and laws set out the basis for dealing with climate change, it is essential to recognize that the implications of climate change and the steps required for an effective response go well beyond environmental management.

Indirect impacts of climate change on land use and land management may change future flood risk. For example, changes in crop type, methods of cultivation and harvesting, deforestation and increased urban expanse will affect the porosity and surface of the ground and hence the volume, speed and direction of storm run-off. Adaptation to climate change requires an integrated approach across different sectors including land use, water resources and transport.

3.2 Planning

3.2.1 Introduction

Highway drainage structures are an essential component in the design of a highway. It is desirable that they be designed economically and provide an adequate level of service.

Factors such as initial cost, design life, climate change and the risk of loss of use of the roadway for a time due to runoff exceeding the capacity of the drainage structure, need to be considered in the design. Accordingly, the maximum design storm frequency shall be taken as specified in Table 2-1.

3.2.2 Construction Considerations

Many serious construction problems arise because important drainage and water-related factors were overlooked or neglected in the planning and location phases of the project. With proper planning, many factors can be avoided or cost effective solutions developed to prevent extended damages. Such factors include:

- Soil erosion;
- Sediment deposition;
- Drainage and landslide;
- Timing of project stages;
- Protection of irrigation systems and continued use during construction;

- Protection of streams, lakes, and rivers; and
- Protection of wetlands.

Analysis of available data, scheduling of work, and other aspects involved in the early planning and location studies can alleviate many problems encountered in the construction of drainage structures.

3.2.3 Maintenance Considerations

Planning and location studies should consider potential erosion and sedimentation problems. If a particular location will require frequent and expensive maintenance due to drainage, alternate locations shall be considered, unless these maintenance costs can be reduced by special design. Local experience is the best indicator of maintenance problems and interviews with maintenance personnel and local residents are extremely helpful in identifying potential drainage problems. Reference to highway maintenance, flood reports, and damage surveys is also valuable in evaluating potential maintenance problems.

Channel changes, drainage modifications, and revisions affecting irrigation systems usually result in certain maintenance responsibilities by the agency constructing the highway. Potential damage from erosion and degradation of stream channels and problems caused by debris can be of considerable significance from a maintenance standpoint.

3.2.4 Coordination between Agencies

Coordination between concerned agencies during the project-planning phase will help produce a design that is satisfactory to all. Substantial cost savings and other benefits can be realized frequently for highway and water resource projects through coordinated planning among the various regional and local agencies that are engaged in water-related activities (flood control and water resources planning, etc.). Interagency cooperation through, for instance, the Ministry of Agriculture, Ministry of Water & Energy, and regional and local administrations, is an essential element in serving the public interest.

3.2.5 Legal Aspects

A goal in highway drainage design shall be to perpetuate natural drainage, insofar as practicable. The courts may look with disfavour upon inflicting damage that could have been reasonably avoided, even where some alteration in flow is legally permissible. Whenever drainage problems exist or can be identified, drainage and flood easements or other means of avoiding future litigation shall be considered, especially in locations where a problem could be caused or aggravated by the construction of a highway. It is advisable to document the history and existing conditions or problems, and supplement the record by photographs and descriptions of field conditions.

3.2.6 Preliminary Data Gathering

Drainage Surveys

Since hydraulic considerations can influence the selection of a highway corridor and the alternate routes within the corridor, the type and amount of data needed for planning studies varies widely. These studies depend on such elements as environmental considerations, class of the proposed highway, state of land-use development, and individual site conditions.

Topographic maps, aerial photographs, and streamflow records provide helpful preliminary drainage data, but historical high-water elevations and flood discharges are of particular

interest in establishing waterway requirements. Comprehensive hydraulic investigations may be required when route election involves important hydraulic features, such as water-supply wells and reservoirs, flood-control dams, water resource projects, and encroachment on flood plains of major streams.

Special studies and investigations, including consideration of the environmental and ecological impact, shall be commensurate with the importance and magnitude of the project and the complexity of the problems encountered.

Data Collection

As part of planning and location studies several categories of data shall be obtained and evaluated, including:

- Physical characteristics of drainage basins;
- Maps and topographic data including channel surveys and cross sections;
- Runoff quantity data (hydrologic and precipitation data);
- Channel and flood plain delineation and related studies;
- Flood history and problem inventory;
- Existing storm water management structure characteristics;
- Development of alternative plan concepts;
- Hydrologic and hydraulic analysis of alternative concepts;
- Consideration of multipurpose opportunities and constraints, benefit/cost analysis and evaluation; and
- Runoff quality data.

Stream Crossings

Additional factors to be considered in locating a stream crossing that involves encroachment within a flood plain are:

- River type (straight or meandering);
- River characteristics (stable or unstable);
- River geometry and alignment;
- Hydrology;
- Hydraulics;
- Flood plain flow;
- Needs of the area; and
- Economic and environmental concerns.

A detailed evaluation of these factors is part of the location hydraulics study. When a suitable crossing location has been selected, specific crossing components can be determined. These include:

- The geometry and length of the approaches to the crossing;
- Probable type and approximate location of the abutments;
- Probable number and approximate location of the piers;
- Estimated depth to the footing supporting the piers (to protect against local scour);
- The location of the longitudinal encroachment in the flood plain;
- The amount of allowable longitudinal encroachment into the main channel; and
- The required river training works to ensure that river flows approach the crossing or the encroachment in a complementary way.

Exact information on these components is not usually developed until the final stage. For location criteria, refer to the *ERA Geometric Design Manual*.

Types of Data

Details associated with data collection, data needed, and where to obtain data, are outlined in the Hydrographical Survey[13] Chapter of this manual. The following is a brief description of the types of data needed for planning and location studies.

i) Topographic

Topographic data shall be acquired at sites requiring hydraulic studies. These data are needed to analyse existing flow conditions, and those created by various design alternatives. Significant physical and cultural features near the project shall be located and documented in order to obtain their elevation. Features such as residences, commercial buildings, schools, churches, mosque, farms, other roadways and bridges, and utilities can affect, as well as be affected by, the design of any new hydraulic structure. Often, recent topographic surveys will not be available at this early stage of project development. Aerial photographs, photogrammetric maps, Ethiopian Mapping Authority topographic maps, and even old highway plans may be utilized during the planning and location phases. When better survey data become available, usually during the design phase, these early estimates will need to be revised to correspond with the most recent field information.

ii) Channel Characteristics

In order to perform an accurate hydraulic analysis, the profile, horizontal alignment and cross sections of the stream shall be obtained. Data to this detail usually are not available during the planning and location phases. The designer, therefore, must make a preliminary analysis based on data such as aerial photographs, topographic maps, and old plans.

One method that can be useful in determining channel characteristics, such as material in the streambeds and banks, type and coverage of vegetal material, and evidence of drift or debris, is the taking of photographs. Field visits made early in the project life can include photographing the channel, upstream and downstream, and the adjoining flood plain. The photos can be valuable aids, especially when taken in colour, for not only preliminary studies, but also for documentation of existing conditions.

During these early phases of project development, the designer should determine the detail of field survey required at the site. This should include the upstream and downstream limits of the survey, the number of and distance between cross sections, and how far to either side of the channel the sections should extend. The minimum number of cross sections will vary with the study requirements and the particular stream characteristics. For some projects, the accuracy achieved by aerial photogrammetry will be sufficient for the level of hydraulic study needed, while other sites will require a different level of accuracy. The level of accuracy of the survey required shall be a consideration when determining the degree of hydraulic analysis needed.

[13] The U.S. Army Corps of Engineers Hydrologic Engineering Center has made a detailed study of survey requirements. The results of this study are available in Accuracy of Computer Water Surface Profiles by M. W. Burnham and D. W. Davis, Technical Paper No. 114, 1986.

For further information on survey requirements, see the ERA Geometric Design Manual.

iii) Hydrologic Data

Information required by the designer for analysis and design include the physical characteristics of the land and channel, as well as all the features that can effect the magnitude and frequency of the flood flow. These data may include climatological characteristics, land runoff characteristics, stream gauging records, high water marks, and the sizes and past performances of existing structures in the vicinity. The exact data required will depend upon the methods used to estimate flood discharges, frequencies, and stages. It shall be noted that much of the hydrologic data would not be used during the planning and location phase. However, it is important to determine the need for the data early in the project because of the time needed to collect and evaluate such data. By starting this process during planning and location, delays during the design stage shall be minimized.

iv) Catchment Characteristics

The hydrologic characteristics of the catchment of the stream under study are needed for any predictive methods used to forecast flood flows. Although many of these characteristics can be found from office studies, some are better found by a field survey of the basin. The size and configuration of the catchment, the geometry of the stream network, storage volumes of ponds, lakes, reservoirs, and flood plains, and the general geology and soils of the basin can all be found from maps. Land use and vegetal cover may be also be determined from maps, but with rapidly changing land uses a more accurate survey will probably be achieved from aerial photographs and field visits.

Having determined these catchment characteristics, runoff times, infiltration values, storage values, and runoff coefficients can be found and used in calculating flood flow values.

v) Precipitation

A precipitation survey normally consists of the collection of rainfall records for the rainfall stations near the study site. Unlike the survey of stream flow records or basin characteristics, however, rainfall records from outside the watershed can be utilized.

Ideally, these records will contain several years of events, for every month and season and will include duration values for various length rainstorms.

This manual contains guidelines for general rainfall amounts that can be used for various duration storms. If adequate rainfall records are available from the Ministry of Water Resources for the project location, more accurate runoff volumes can be established for design of drainage structures.

vi) Flood Data

The collection of flood data is a basic survey task in performing any hydraulic analysis. This data can be collected both in the office and in the field. The office acquisition includes the collection of past flood records, stream gauging records, and newspaper accounts. The field collection will consist mainly of interviews with residents, maintenance personnel, and local officials who may have recollections or photos of past flood events in the area. If there is a stream gauging station on the stream being studied, it is close to the crossing site, and has many years of measurements, then, in some cases, this may be the

only hydrologic data needed. This data shall be analyzed to ensure that stream flows have not changed over the time of measurement. Such changes in flow may be due to watershed alteration such as the construction of a large storage structure, diversion of flow to another watershed, addition of flow from another watershed, or development that has significantly altered the runoff characteristics of the watershed.

vii) High-Water Information

Sometimes high-water marks are the only data of past floods available. When collected, these should include the date and elevation of the flood event when possible. The cause of the high-water mark should also be noted. Often unusual debris rather than an inadequate structure cause the mark, therefore, designing roadway or structure grades to such an elevation could lead to an unrealistic, uneconomical design.

High-water marks can be identified in several ways. Small debris, such as grass or twigs caught in tree branches, hay or crops matted down, mud lines on buildings or bridges, are all high-water indicators. However, grass, bushes, and tree branches bend over during flood flows and spring up after the flow has passed, and this may give a false reading of the high water elevation.

viii) Existing Structures

The size, location, type, and condition of existing structures on the stream under study can be a valuable indicator when selecting the size and type for any new structure. Data to be obtained on existing structures includes size, type, age, existing flow line elevation, and condition, particularly in regards to the channel. Scour holes, erosion around the abutments upstream or down, or abrupt changes in material gradation or type can all indicate a structure too small for the site. With knowledge of flood history, the age, and overall substructure condition may also aid in determining if the structure is too small.

ix) Vegetation

During the field visit, it may not be possible to survey the entire watershed, and a sample area may have to be studied. It is important to set out the exact field needs before the trip is made to ensure all information needed is collected and all important areas visited. See Chapter 4 for specifics on the field trip.

x) Water Quality

Water quality data can be the most expensive and most time-consuming information to collect. Sometimes water quality records are available at or near the site under study but even then, the information most often required for highway studies may not have been gathered. Sample collection is expensive because of the equipment and laboratory facilities needed. The cost of having samples taken and analyzed may need to be considered.

Sample collection can be time consuming because one sample or several taken at the same time is not usually satisfactory. Water quality can reflect seasonal, monthly, or even daily variations depending on the weather, flow rate, traffic, etc. Therefore, a sampling program shall be extended for a year, if possible.

3.2.7 Hydraulic Report

The Hydraulic Report shall be as complete as possible but must be tailored to satisfy the requirements of the specific location and size of the project. The report should list all

significant watersheds with a unique number and approximate chainage for the crossing. Data and information shall be reduced to meaningful information. Coordination with all ERA sections requiring survey data before the initial fieldwork has begun will help insure that survey data is sufficient but not excessive.

All data used in reaching conclusions and recommendations during the preliminary study shall be included in a report. This should include hydrologic and hydraulic data, pertinent field information, photographs, calculations, and structure sizes and location. At this stage of the study, several structure sizes and types can usually be suggested, as the designer only needs generalities in order to obtain a rough estimate of needs and costs.

Often, specifics cannot be provided until an accurate topographic survey of the area has been made and precise hydraulic computations performed. Sometimes, however, the report will require detailed design studies in order to justify the extent of mitigation required. In general, the more environmentally sensitive and/or highly urbanized areas will necessitate more detail at earlier stages. All this information serves as documentation for decisions made at this time, as well as excellent reference material when the later, more detailed studies are performed. Therefore, it is important that this material be collected, prepared, referenced, and put into an easily understood report folder as carefully as possible.

The hydraulic report for all projects should include:

- Statement of design storm frequencies;
- Runoff formulas to be used for computing flow rates with basin size limits;
- Methods for computing time of concentration or time to peak;
- Anticipated future land use changes that may affect runoff rates and volumes;
- Sources of rainfall intensity, depth, duration, and frequency curves;
- Other information needed by the designer for determination of flow rates for ditches and culverts; and
- Source maps for determining drainage areas.

(Include additional requirements for different types of projects: new, renovation, urban, rural, highway class, as appropriate.)

3.3 References

Ethiopian Roads Authority (ERA) 2002, Drainage Design Manual

Australian Drainage Manual, 2010, Second Edition

South African Roads Agency Ltd, Drainage Manual, 2007.

4 DATA COLLECTION, EVALUATION AND DOCUMENTATION

4.1 Introduction

It is necessary to identify the types of data that will be required prior to conducting the design analysis. The effort necessary for data collection and compilation shall be tailored to the importance of the road drainage project. Not all of the data discussed in this chapter will be needed for every road project. However, a well planned data collection program leads to a more orderly and effective analysis and design that is commensurate with:

- Project scope;
- Project cost;
- The complexity of the site hydraulics; and
- Federal and regional regulatory requirements.

Data collection for a specific project must be tailored to:

- Site conditions;
- Scope of the design analysis;
- Social, economic and environmental requirements;
- Unique project requirements; and
- Federal and regional regulatory requirements.

Uniform or standardized survey requirements for all projects and in all regions may prove uneconomical or data deficient for a specific project. Special instructions outlining data requirements may have to be provided to the surveying contractor by the hydraulic designer for unique sites.

4.1.1 Data Requirements

The purpose of this chapter is to outline the types of data that are generally required for drainage analysis and design, possible sources, and other aspects of data collection, review and compilation. The following topics are presented in this chapter.

- Sources of Data;
- Types of Data;
- Survey Information;
- Field Reviews;
- Data Evaluation; and
- Channel and Floodplain Survey Specification

4.1.2 Survey Methods/Computational Accuracy

The publication "Accuracy of Computed Water Surface Profiles," U.S. Army Corps of Engineers, Dec. 1986, focuses on determining relationships between:

- Survey technology and accuracy employed for determining stream cross-sectional geometry;
- Degree of confidence in selecting Manning's roughness coefficients; and
- The resulting accuracy of hydraulic computations.

4.2 Sources and Types of Data

4.2.1 Objectives

Objectives of this chapter are summarized as:

- Identify possible sources of data;
- Rely on ERA experience as to which sources will most likely yield desired data;
- Utilize the guides in this chapter for data sources; and
- Acquaint the designer with available data and ERA procedures for acquiring the required information.

4.2.2 Source

Much of the data and information necessary for the design of highway drainage facilities may be obtained from some combination of the sources listed in Form 4-1 at the end of this chapter. The following information is given for each data source on the same list:

- Type of data;
- Contact details of source; and
- Comments on data.

4.3 Type of Data Required

The drainage designer must compile the data that are specific to the subject site.

The following are the major types of data that may be required:

- Digital Elevation Model (DEM) data from Shuttle Radar Topography Mission (SRTM) which are available from URL: http://strm.usgs.gov and ASTER freely available from URL: http://asterweb.jpl.nasa.gov/data.asp. These data can effectively be used in GIS platforms for hydrological analysis of watersheds (delineation of catchments, stream slope analysis etc.) for major river crossings;
- During site investigation, it is required to collect data in such a way that preliminary assessment of bed/bank material (to assess scour/sedimentation potential at bridge crossings) and hydraulic parameters like Manning's "n" may be suitably assumed during initial stages of the drainage design;
- For delineation of catchments for culverts in rolling/hilly areas, images from the freely available Google Earth may be effectively used;
- Catchment characteristics;
- Stream reach data (especially in the vicinity of the drainage structure);
- Other physical data in the general vicinity of the structure such as utilities or easements;
- Hydrological and meteorological data (stream flow and rainfall data related to maximum or historical peaks as well as low flow discharges and hydrographs applicable to the site);
- Existing and proposed land use data in the subject drainage area and in the general vicinity of the facility;
- Soil data;
- Anticipated changes in land use and/or watershed characteristics; and
- Flood plain and environmental regulations.

Watershed, stream reach and site characteristic data, as well as data on other physical characteristics, can be obtained from a field reconnaissance of the site. Examination of available maps and aerial photographs of the watershed is also an excellent means of defining physical characteristics of the watershed.

4.3.1 Drainage Surveys

A complete field or aerial drainage survey of the site and its contributing catchment should always be undertaken as part of the hydraulic analysis and design. Survey requirements for small drainage structures such as 0.9 meter diameter culverts are less extensive than those for major structures such as bridges. However, the purpose of each survey is to provide an accurate picture of the conditions within the zone of hydraulic influence of the facility. Forms 4-1 and 4-2 at the end of this chapter contain instructions for minor and major drainage surveys.

The following are data that can possibly be obtained or verified:

- Contributing drainage area characteristics;
- Stream reach data (cross sections and thalweg profile);
- Existing structures;
- Location and survey for development, existing structures etc., that may affect the determination of allowable flood levels, capacity of proposed drainage structures, or acceptable outlet velocities;
- Drift/debris characteristics;
- General ecological information about the drainage area and adjacent lands; and
- High water marks, including the date of occurrence.

Much of this data must be obtained from an on-site inspection. It is often much easier to interpret published sources of data after an on-site inspection. Only after a thorough study of the area and a complete collection of all required information should the designer proceed with the design of the hydraulic facility. All pertinent data and facts gathered through the survey are to be documented. Forms 4-1 and Figure 4-2 at the end of this chapter contain examples of how the field or aerial survey data discussed in this chapter shall be documented.

4.3.2 Catchment Area Characteristics

The following text is a brief description of the major data topics that relate to drainage facility analysis and design.

Physical Characteristics

Contributing Size - The size of the contributing catchment area expressed in hectares or square kilometres, is determined from some or all of the following:

- Direct field surveys with conventional surveying instruments;
- Any changes in the contributing catchment area that may be caused by:
  - Terraces;
  - Lakes;
  - Sinks;
  - Debris or mud flow barriers;
  - Reclamation/flood control structures;
  - Irrigation diversions.

- Topographic maps that are available for many areas of Ethiopia from the Ethiopian Mapping Authority; and
- Aerial maps or aerial photographs.

In determining the size of the contributing catchment area, any subterranean flow or areas outside the physical boundaries of the drainage study area that have run-off diverted into it shall be included in the total contributing catchment area. In addition, the designer must determine if floodwaters can be diverted out of the basin before reaching the site.

List of Catchment Delineation Software

The following are some of the commonly used software employed to delineate catchment areas:

- Arc Hydro Tools;
- Urban 4.0;
- HEC-GeoHMS;
- MapWindows;
- Quantum GIS;
- Global Mapper;
- Arc GIS etc.

4.3.3 Catchment Area Slopes - Characteristics

The slope of the stream, the average slope of the catchment, and other important terrain characteristics shall be determined. Hydrological and hydraulic procedures in other chapters of this manual are dependent on catchment slopes and these other physical characteristics.

4.3.4 Catchment Land Use

The present and expected future land use, particularly the location, degree of anticipated urbanization, and data source shall be defined and documented. Information on existing use and future trends may be obtained from:

- Aerial photographs (conventional and infrared);
- Land use maps;
- Topographic and other maps;
- Municipal planning agencies; and
- Landsat (satellite) images (See ERA Geometric Design Manual).

Specific information about particular tracts of land can often be obtained from owners, developers, and local residents. Care shall be exercised in using data from these sources since their reliability may be questionable and these sources may not be aware of future development within the catchment area that might affect specific land uses.

Existing land use data for small catchments can be determined or verified best from a field survey. Field surveys should also be used to update information on maps and aerial photographs, especially in catchment basins that have experienced changes in development since the maps or photos were prepared. Infrared aerial photographs may be particularly useful in identifying types of urbanization at a point in time.

4.4 Data on Streams, Rivers, Ponds, Lakes, and Wetlands

At all streams, rivers, ponds, lakes, and wetlands that will or may be affected by the proposed structure or construction, the following data shall be secured. These data are essential in determining the expected hydrology:

- Outline boundary (perimeter) of the water body for the ordinary high water;
- Elevation of normal as well as high water for various frequencies;
- Detailed description of any natural or manmade spillway or outlet works including dimensions, elevations, and operational characteristics;
- Detailed description of any emergency spillway works including dimensions and elevations;
- Description of adjustable gates, and soil and water control devices;
- Profile along the top of any dam and a typical cross-section of the dam;
- Determine the use of the water resource (stock water, fish, recreation, power, irrigation, municipal or industrial water supply etc.); and
- Note the existing conditions of the stream, river, pond, lake, or wetlands for turbidity and silt.

4.4.1 Environmental Considerations

Environmental considerations are an important component of drainage design and drainage structure silting. There is a need to investigate and mitigate possible impacts due to specific design configurations on the environment. Information to be assessed is as follows:

- Information necessary to define the environmental sensitivity of the facility's site relative to impacted surface waters, e.g. water use, water quality and standards, aquatic and riparian wildlife biology, and wetlands information;
- Physical, chemical and biological data for some streams may also be available from the Environmental Protection Agency, the Ministry of Water & Energy and from municipalities and industries that use surface waters as a source of water supply. In unique instances, data collection program possibly lasting several years and tailored to the site may be required;
- Wetlands are unique and data needs can be identified through coordination with the Ministry of Water & Energy; and
- For additional information on environmental issues concerning drainage structures, the designer should consult the ERA Standard Environmental Methodologies and Procedures Manual.

4.4.2 Site Characteristics

A complete understanding of the physical nature of the natural channel or stream reach is of prime importance to good hydraulic design - particularly at the site of interest. Any work being performed, proposed or completed, that changes the hydraulic efficiency of a stream reach, must be studied to determine its effect on the stream flow. The designer should be aware of plans for channel modifications, and any other changes that might affect the facility design.

The stream may be classified as:

- Rural or urban, improved or unimproved;
- Narrow or wide;

- Shallow or deep;
- Rapid or sluggish;
- Stable, transitional, or unstable;
- Sinuous, straight, braided, alluvial, or incised; and
- Perennial or intermittent flow.

4.4.3 Geo-morphological Data

Geo-morphological data are important in the analysis of channel stability and scour.

Types of data needed are:

- Sediment transport and related data;
- Stability of form over time (braided, meandering, etc.);
- Scour history/evidence of scour; and
- Bed and bank material identification.

4.4.4 Roughness Coefficients

Roughness coefficients, ordinarily in the form of Manning's "n" values, shall be estimated for the entire flood limits of the stream. A tabulation of Manning's "n" values with descriptions of their applications can be found in Chapter 6, Table 6-1.

4.4.5 Stream Bed Profile

Stream bed profile data must be obtained and these data should extend upstream and downstream sufficiently far enough to determine the average slope and to encompass any proposed construction or aberrations. Identification of "headcuts" that could migrate to the site under consideration is particularly important. Profile data on live streams may be obtained from the water surface. Where there is a stream gauge relatively close, the discharge, date, and hour of the reading shall be obtained. The stream bed profile should extend upstream and downstream for a distance of at least 200 meters or preferably to:

$$L = 100 \log(A)$$

Where: L = distance in meters
A = Area in km^2

4.4.6 Stream Cross-Section

Stream cross-section data that represents the typical conditions at the structure site need to be obtained, as well as at other locations where stage-discharge and related calculations will be necessary. Stream cross sections shall be taken at 200 metre intervals upstream and downstream for at least one half the distance indicated as "L" above (refer to the Channel and Floodplain Survey Specification section of this chapter).

4.4.7 Existing Structures

The location, size, description, condition, observed flood stages, and channel section relative to existing structures on the stream reach and near the site must be secured in order to determine their capacity and effect on the stream flow. Any structure, downstream or upstream, that may cause backwater or retard stream flow is to be investigated. Also, the manner in which existing structures have been functioning with regard to scour, overtopping, debris passage, fish passage, etc. shall be noted. For bridges, this data should include span lengths, type of piers, and substructure orientation which can usually be

obtained from existing structure plans. The necessary culvert data includes parameters such as size, inlet and outlet geometry, slope, end treatment, culvert material, and flow line profile. Photographs and high water profiles or marks of flood events at the structure and past flood scour data can be valuable in assessing the hydraulic performance of the existing facility.

4.4.8 Acceptable Flood Levels

Improvements, property use, and other developments adjacent to the proposed site both upstream and downstream may determine acceptable flood levels. Incipient inundation elevations of these improvements or fixtures shall be noted. In the absence of upstream development, acceptable flood levels may be based on freeboard requirements to the highway itself. In these instances, the presence of downstream development becomes particularly important as it relates to potential overflow points along the road grade.

4.4.9 Flood History

The history of past floods and their effect on existing structures is of exceptional value in making flood hazard evaluation studies, and for sizing structures. Information may be obtained from newspaper accounts, local residents, flood marks, or other positive evidence of the height of historical floods. Changes in channel and catchment conditions since the occurrence of the flood shall be evaluated in relating historical floods to present conditions. Recorded flood data may be available from agencies such as the Ministry of Water Resources and local government offices.

4.4.10 Debris Characteristics

The quantity and size of debris carried or available for transport by a stream during flood events must be investigated and such data used in the design of structures. In addition, the times of occurrence of debris in relation to the occurrence of flood peaks shall be determined; and the effect of backwater from debris on recorded flood heights shall be considered in using stream flow records.

4.4.11 Scour Potential

Scour potential is an important consideration relative to the stability of the structure over time. Scour potential is determined by a combination of the stability of the natural materials at the facility site, tractive shear force exerted by the stream and sediment transport characteristics of the stream.

Data on natural materials can be obtained from in-situ testing and materials sampling. Bed and bank material samples sufficient for classifying channel type, stability, and gradations, as well as a geotechnical study to determine the substrata if scour studies needed, will be required. The various alluvial river computer model data needs will help clarify what data are needed. In addition, these data are needed to determine the presence of bed forms so a reliable Manning's "n" as well as bed form scour can be estimated.

4.4.12 Controls Affecting Design Criteria

Many controls will affect the criteria applied to the final design of drainage structures, including allowable headwater and flood level, velocities, resulting scour, and other site-specific considerations. Site investigations need to determine what natural or manufactured controls need to be considered in the design and these downstream and upstream controls shall be documented.

Downstream Control - Any ponds or reservoirs, along with their spillway elevations and design levels of operation, shall be noted as their effect on backwater and/or stream bed aggradation may directly influence the proposed structure. In addition, any downstream confluence of two or more streams must be studied to determine the effects of backwater or stream bed change resulting from that confluence.

Upstream Control - Upstream control of run-off in the catchment must be noted. Conservation and/or flood control reservoirs in the catchment may effectively reduce peak discharges at the site and may retain some of the catchment run-off. Capacities and operation designs for these features shall be obtained from the Ministry of Water and Energy or other operating authority or agency (e.g. EEPCO).

The redirection of floodwaters can significantly affect the hydraulic performance of a site. Some actions that redirect flows are irrigation structures, debris jams, mudflows, and highways or railroads.

4.5 Survey Information

Complete and accurate survey information is necessary to develop a design that will best serve the requirements of a site. The amount of survey data gathered shall be commensurate with the importance and cost of the proposed structure and the expected flood hazard as discussed in Section 4.3 (Drainage Surveys) and as determined using Forms 4-1, and Form 4-2.

At some sites, photogrammetry is an excellent method of securing the topographical components of drainage surveys where adequate topographic mapping is not available. Planimetric and topographic data covering a wide area are easily and cost effectively obtained in many geographic areas. A supplemental field survey is often required to provide data in areas obscured on the aerial photos (underwater, under trees, etc.).

Data collection shall be as complete as possible during the initial survey in order to avoid repeat visits. Thus, data needs must be identified and tailored to satisfy the requirements of the specific location and size of the project early in the project design phase. Coordination by the Project Manager with the Hydraulics Engineer before the initial field work is conducted will help ensure the acquisition of sufficient, but not excessive, survey data. Example forms and checklists for hydraulic surveys are presented on Forms 4-1 and Form 4-2.

The available aerial photos for Ethiopia are outdated. Cross-checking of existing aerial photos with other data sources (e.g. Google Erath, ASTER, or Landsat) is necessary before using the existing maps for road drainage design purposes.

4.6 Data Collection

4.6.1 Digital and Satellite Data Models

Several methods to use electronic data for hydraulic and hydrological studies are available. Design of drainage systems can be accomplished using GIS/CAD software and electronic surface data. Hydrological and hydraulic models can be developed using this data.

The types of data normally used by digital models are:

- Elevation data;
- Features (e.g., streams and roadways);
- Land use; and
- Soils and infiltration.

Some of the electronic data is readily available, though not always with the desired resolution. Elevation data is available from Digital Elevation Model (DEM) data from Shuttle Radar Topography Mission (SRTM) which are available from URL: http://strm.usgs.gov and ASTER freely available from URL: http:/ http://asterweb.jpl.nasa.gov/data.asp. The data is normally available in UTM coordinates from 5m to 90m resolution, depending on the location. The Ministry of Agriculture (MoA) and the Ministry of Water & Energy (MoW&E) also maintain soil and land use databases in GIS formats in certain areas. Detailed hydraulic and hydrological studies may require higher resolution elevation data than is normally available through the above sources. Higher resolution data is available for the city of Addis Ababa.

Satellite imagery is available through commercial vendors. However, high-resolution elevation data is not normally available through these sources, and the technology to extract it is not yet available. Satellite imagery can be used to determine land uses. Due to the scarcity or obsolescence of elevation data, the normal approach is to develop topographic surveys for a project. There are two basic methods to develop topographic surveys:

- Aerial photogrammetry; and
- Field data collection.

4.6.2 Aerial Photogrammetry

Under this method, topographic mapping is developed using pictures of the ground taken from an aircraft or satellite. Ground controls are established using field survey methods and contours are developed.

Aircraft used for taking photographs can be fixed wing (airplane) or helicopter. Fixed wing is still the most economical method; however, helicopter based surveys offer low altitude flights, resulting in much higher accuracy. The pictures taken can also be used as data for hydraulic investigations and studies.

High-resolution satellite and multi-spectral imagery is available and may be substituted for other methods if necessary. Because satellite data is stored for a period of time, multi-spectral satellite imagery can also be used to investigate flooding after an event has occurred. Potentially, the technology can be used to develop “before and after” images and topography to investigate a flood event or other significant change in an area of interest.

A new method of aerial topographic generation is using laser or radar beams from an aircraft carrying differential GPS. The laser based method is called Light Detection and Ranging (LiDAR). LiDAR or radar generated data have the advantage of being inexpensive when compared to traditional photogrammetry. However, the accuracy is highly dependent on the technology available to the vendor in aerial equipment and available software to filter trees and other covered land areas.

4.6.3 Field Data Collection

Field data collection is normally accomplished using electronic survey equipment such as Total Station and Global Positioning System (GPS).

Using Total Station as a data collection tool, the engineer can develop topographic mapping directly from fieldwork, with little additional processing. This information can be directly used in certain highway or hydraulics software, saving time and resources in the tedious process of survey decoding and data entry. Digital Elevation Models (DEM) or Digital Surface Models (DSM) can be developed using the data collected using this method. Other feature data (e.g., flood limits, bank-full indicators, vegetation markers, point bars, flow boundaries) can also be located by a surveyor and automatically decoded along with the elevation data. The accuracy of this method can be very high but is dependent on the experience of field personnel.

GPS based surveying is still less accurate because it depends on many factors such as location of the survey reach and time of day. Hand-held GPS units that have sub-metre horizontal precision are available and can be used to collect field data.

Vertical precision to collect elevation data is not sufficiently accurate for many design functions. However, this method makes a one-person survey crew possible with minimal training. GPS data can be obtained by a hydraulics engineer during a field visit. This facilitates rapid development of field data, especially location data, and quick office evaluations.

4.6.4 Channel and Floodplain Topographical Survey Specification

Open Channel Cross-Sections

Hydraulic modelling exercises should be undertaken for the critical floodplains where the proposed road crosses major watercourses. To facilitate this work, channel and floodplain topographical surveys are required. The purpose of these is to gather data to set up a hydraulic model of the existing condition and to assess the impact of the proposed road crossings on surrounding areas as well as flood risk to the road crossing structure itself, and to provide details of the structures present in the vicinity of the proposed road route. In order to undertake channel and floodplain topographical survey work, all ERA design consultants should follow this specification.

The location of the watercourses where the road crossings are proposed should be shown on the location plan.

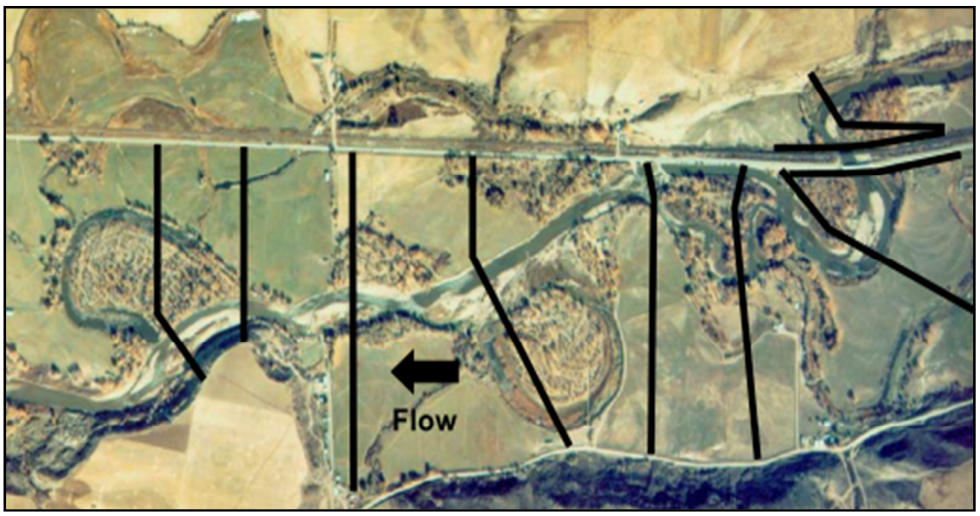


Figure 4-1: Sample cross section spacing

Channel cross-sections should be surveyed normal to the centre line of the channel at the intervals to be shown on the plan. Existing structures, if any, other than those identified on the ocation plan (any existing hydraulic structures should be marked on the plan), not falling at the specified interval should be surveyed unless stated otherwise.

Additional cross-sections should be surveyed where the channel significantly changes width or elevation (e.g. waterfalls). Where it is not practical to survey a section at the prescribed position or interval, the position of the section may be moved. However, the interval between two adjacent sections shall not exceed the prescribed interval.

Cross-sections should be surveyed viewed downstream and the origin or zero chainage of the channel cross-section must be established on the left bank (LB) of the channel viewed downstream. However, where a section is only required through the right bank, the origin or zero chainage shall be located on the waterside of the bank, i.e. in the channel.

Sufficient levels must be taken across the cross-section for the channel shape and geometry to be easily identifiable (a plan should be prepared for an indication of where levels should be taken). A description of the material lining the channel (e.g. silt, grass, pebbles, concrete etc.) should be provided at regular intervals with photographs being provided in support. Location of photographs should be identified by the label attached to the closest cross-section.

If upstream views are required, e.g. downstream elevation of bridges and weirs, this will be noted in the Survey Brief. The origin or zero chainage of the upstream view shall be established on the left bank (LB) of the channel. The section shall be plotted as viewed upstream i.e. the 'Range' values below the section plot will be negative.

Each individual structure cross-section will be given a relevant title included in the section header. Where a cross-section is of an upstream view, this must be clearly noted in the title. Open channel sections should not normally have a title.

In addition to cross-sections through the channel, cross-sections should be extended from the channel to the true land level on each side and at least 20m beyond the bank crest (where possible) unless mentioned otherwise in the Survey Brief. Where trees or bushes/shrubs line the channel the section shall extend to 5m beyond the vegetation, but no more than 50m from the channel. Beyond the extent of the cross-section, a general

indication of the ground form should be given as a label e.g. “flat”, “rises steeply”. The point used for the longitudinal section bank line shall be indicated on the plotted cross-section.

Note: Where a river bank is raised above the surrounding ground (floodplain), the crest is defined as the point on the top of the bank over which water will spill from the river onto the surrounding ground. Where there is no raised bank, the crest is the point marking the change of gradient from surrounding ground to the channel.

Points along the cross-section should be surveyed at an interval that accurately depicts the shape of the channel. For open channel sections, the drawn line of the cross-section shall be correct to better than +/- 0.1m in height allowing for up to 0.2m movement along the section line. For structure details, the drawn line of the cross section shall be correct to better than +/- 0.02m in height allowing for up to 0.02m movement along the section line.

Bushes, trees, fences and buildings adjacent to the channel cross-section should be shown as symbols – not true to scale.

If there are buildings along the proposed road route, their floors or damp-proof course level should be indicated. Where they cannot be determined the threshold level shall be recorded. Buildings will be labelled with name and/or number, type and whether a damp-proof course exists.

Any water body including lakes or ponds should be surveyed. This includes maximum water levels at the time of the survey and top of bank levels. Lake bed level bathymetry should be taken with echo sounding equipment. Fences will be labelled with their type and height. Road crossings will be labelled with name and/or number.

Presentation and Format of Data

The data to be supplied by the Surveyor should be in a specific format for loading into the hydraulic modelling suite of programs *(e.g. HEC-RAS, consult ERA for sample format).*

Data will also be supplied in x, y, z format as an Excel Spreadsheet with the following column headers.

- Section No;
- Point Eastings;
- Point Northings;
- Point Altitude.

This will allow channel survey data to be merged with topographic and photogrammetric surveys.

All longitudinal and cross-section plots should be produced on A1 sized sheets and hard copy plots shall have a 15mm border outside the frame. Left Bank and Right Bank are defined as viewed downstream.

When congested data would cause over-writing of the co-ordinates under plotted sections, the descenders should be cranked to allow the values to be plotted without over printing.

i) Altitudes

For all GPS observations using the static/rapid-static technique, dual frequency survey quality GPS receivers shall be used to measure altitudes. GPS stations shall be located with a substantially clear sky-view and not close to buildings or other structures that might introduce multipath effects. A minimum of five satellites must be observed for the full observation period, with a minimum elevation mask of 13°. PDOP, HDOP and GDOP

values must not exceed the equipment manufacturer's recommendations. These values will be tabulated in the baseline computation log file. For static and rapid-static baselines a 15 second observation interval shall be used unless otherwise stated in the survey brief.

ii) Bed Levels

Bed levels should be measured directly whenever and wherever possible. Where direct measurement is impossible, where, for instance, the water depth is too great or other causes make it impractical, then it will be sufficient to read the depth of water against a staff or to use echo sounding and to relate these readings to a measured water level.

Where silt occurs, both the hard bed and the silt top will be measured at the same point. The hard bed should be shown as a solid line. The silt top should be shown as a dashed line and shall be labelled "S" in the digital data listing.

The nature of the bed material should be recorded and plotted on the section in simplified form, e.g. 'Gravel'. Surfaces outside the water area should also be labelled.

iii) National Grid Reference and Cross-Section Orientation

The full Ethiopian National Grid reference of the cross-section chainage zero-point and the grid bearing of the section line will be added to each cross-section header in the survey data file in the appropriate fields and quoted to 3 decimal places.

Channel surveys may be merged with photogrammetric or LiDAR surveys of the floodplains and therefore positional accuracy must be of the same order. The Ethiopian National Grid Co-ordinates of the Section Zero Point will be observed to E4 standard by GPS. The orientation of sections will also be determined by GPS. The section data should also be plotted against the available topographical map background to give the true position of the section.

iv) Cross-section Reference Numbers

Cross-sections should be numbered to reflect chainage along each watercourse.

v) Scale

Cross- sections should be to appropriate scales to be plotted to A3 size. The long sections for the watercourses should be appropriately scaled to plot to A1 sized sheets.

vi) Merging Data from Previous Surveys

Any requirement for merging new survey data with data from a previous survey should be noted in the Survey Brief. Data shall be merged so that the correct sequence of chainage across the section and along the channel is maintained. A note of this shall be added to the cross-sectional plot. Cross-sections from a previous survey shall be updated if there is a significant change (e.g. a new structure).

vii) Floodplain Sections

If floodplain cross-sections are required, this should be noted together with the interval in the Survey Brief. Sections should be plotted at the scales defined in the Survey Brief.

A floodplain section should be taken normal to the centre line of the valley and not necessarily at right angles to the centre line of the channel. Because of this, flood plain sections may appear 'dog-legged' on the key plan. These sections may be defined on the contract mapping.

viii) Structures

Unless otherwise stated in the survey brief sections shall be surveyed at the **upstream** and **downstream** side of each structure which significantly affects the river flow at bank-full flow condition.

Where the structures are below roads and / or footpaths spot levels should be taken along the high point of the road (i.e. kerb height or road crest) every 10m for a distance of 100m either side of the structure. Where a parapet forms part of the structure a level should be taken on top of the parapet and the width of the parapet should be identified on the cross-section.

Structures include bridges, culverts, weirs, pipe crossings and impounding structures of any kind. Natural features which act as structures, such as rock outcrops, shall also be included. Structures that are not to be surveyed shall be photographed. The photographs and NG co-ordinates of the position of the structure shall be included as an appendix to the Survey Brief. If there is any doubt, the Surveyor should consult the Engineer to confirm whether a section is required.

All pipe crossings, including those too small to require a cross-section to be taken, shall be shown on the longitudinal section, along with critical levels and dimension.

Overhead power and telephone cable crossings should be noted and their position and their clearance height over the centreline of the channel plotted on the longitudinal section. Underground crossings (water, telephone, power etc.), where evident on site, should also to be noted and their position plotted on the longitudinal section.

Bridges and Culverts

A bridge is defined as a permanent structure spanning a channel. Cross sections of temporary and ad hoc crossings are not required unless indicated on the attached plan. Such crossings shall be shown on the longitudinal section.

A complete elevation of the upstream side of the structure is to be taken with particular attention paid to the measurement of the bridge openings and flood culverts Details of any bridge piers should also be shown. Soffit, invert and springing levels should be added as labels.

The downstream elevation should be taken viewed looking **upstream** when specifically requested **or where it is different from the upstream side**. Even when a downstream elevation is not required, the downstream soffit, top of parapet, invert, bed level and bank crests are to be measured and added to the longitudinal section.

The length of the bridge tunnel is to be measured parallel to the watercourse and this, together with hard inverts on aprons and their extent, added as labels on the cross-section plot.

Where a bridge changes section within its length and that change is significant, then an additional section shall be surveyed at the change.

When a channel changes section through a bridge, an additional section should be taken 5 to 15 metres upstream and downstream of the bridge where the channel returns to its normal size. Unless specified in the Survey Brief, the downstream section should only be measured when it differs markedly from the upstream section.

Where a structure is not normal to the channel but is skewed, the skew span should be measured together with the approximate angle of skew, this being the angle between the

bridge face and a line normal to the channel. The length of the bridge tunnel will then be the channel length through the bridge parallel to the watercourse, not the distance at right angles to the road.

Where a structure extends 10m beyond the top of the bank, then the complete elevation will be surveyed with its cross-section. Where a bridge spans the floodplain, then all relevant flood arches must be included in the cross-section. If the cross-section is excessive then a plot of the immediate channel will be drawn to the specified scale. The complete cross-section will be plotted at a reduced scale, provided on a separate sheet and cross-referenced to the channel plot.

When a culvert is longer than the section interval defined in the Survey Brief a cross-section will be taken at the entrance and exit.

Under no circumstances shall the Surveyor enter a confined space which has not been notified to him/her in the Brief and for which no proper procedures have been adopted.

Weirs and Drop Structures

A weir is defined as a permanent or temporary structure that impounds a head of water at normal summer levels greater than the height defined in the Survey Brief. A drop structure is defined as a natural or man-made step in the channel bed that will be surveyed, as defined in the Survey Brief.

A cross-section should be taken across the crest of the weir, viewed downstream with structure details incorporated as shown in the Survey Brief. Additional cross-sections should be taken immediately upstream and downstream of the weir crest, viewed downstream and normal to the centreline of the channel as shown in the Survey Brief. Levels across the weir crest or on aprons shall not be taken as soundings.

A longitudinal section through the centre line of the weir (but NOT through a drop structure) should be produced in cross-section format showing all structure details, such as positions of culvert and bridge crossings, extending both upstream and downstream to the natural riverbed. This should be plotted viewed from upstream to downstream.

Longitudinal sections through weirs should be numbered with the same section number as the downstream elevation, suffixed with an alpha character (e.g. N.NNNA).

The longitudinal section should show the following information:

- Upstream water level;
- Upstream bed level;
- Weir crests and any bridge structures;
- Upstream and downstream extent of any apron;
- Downstream water level;
- Downstream bed level, including maximum depth of scour hole where it is safe to obtain levels; and
- Water and bed levels at the tail of any weir pool

An additional cross-section should be taken both upstream and downstream of the weir where the channel returns to its normal cross-section and is free from the influence of deposition and scour.

Sluices

Sluice structures are not common in Ethiopia. However, a sluice is a useful flow controlling device and should be considered where flood control is necessary. Upstream and downstream cross sections should be taken along with opening dimensions (height and width) and descriptions of the sluice control mechanism. A level should be taken on the sluice crest. If more than one sluice exists the above measurements should be taken on each sluice, if different, and the number of sluices noted.

Waterfalls

Cross-sections should be taken at the top and bottom of the waterfall and midway through the waterfall if it extends for over 5m. Chainage of the waterfall is to be provided in a long section.

ix) Natural Constraining Features

Features such as rock formations, which cause gradient changes or affect water levels, should be treated as weirs. Changes in water level gradient over shoals and aprons, and sudden changes in bed level should be measured and added to the longitudinal section.

x) Chainage

Each cross-section shall be provided with a chainage. This is the distance along the centre line of the channel from the downstream extent of the survey. The centre-line shall be digitised from a 1:2,500 / 1:1,250 topographical map. It shall be supplied as a polyline in a separate layer and presented on the Key Plan. The cross-sections shall be plotted on the Key Plan from actual surveyed section points, and their centreline chainage deduced by measurement along the centreline of the mapped watercourse. Zero chainage will be at the downstream extent of the watercourse unless otherwise specified in the Survey Brief.

Running chainages along the watercourse shall be noted on the levelling sheets, with the start point and direction of work clearly defined. Chainages shall be noted at boundaries, ditches, drainage pipes and other identifiable features, indicating on which bank these features appear. Cross-section chainages should also be noted and clearly referenced.

xi) Key Plan

A key plan based upon a 1:2,500 or 1:1,250 map data will be produced for each longitudinal section to show the cross-section positions and watercourse centre-line. Whenever possible, this plan should be incorporated into the same sheet as the longitudinal section. When so incorporated, it will be aligned to match the longitudinal section in AutoCAD paper space mode. It is acceptable for the plan to be inverted. It should be provided with north point and grid co-ordinates.

In addition, the river centre-line shall be presented as a digital polyline created in a format suitable for input to GIS software (e.g. MAPINFO, ArcGIS etc.). It should be provided with the following attributes:

- Field Name Field Type/Width Remarks;
- Polyline_ID String max 9 characters nnnnn_nnn;
- Data Source “max 30 “” eg. ‘Survey’;
- Surveyor “max 30 “” Company Name;
- Consultant_Ref “max 30 “” Surveyor’s reference;

- Client_Ref “max 30 “” Company Job Number;
- Date “max 30 “” ` Date of survey; and
- Channel “max 30 “” Watercourse name.

Surveyors without access to GIS Software may render the polyline in AutoCAD as a file named CLXXXXXX.dwg where XXXXXX is the job name.

Content/Presentation of Longitudinal Section

A longitudinal section of the survey area should be produced from the recorded data at the scales shown in the Survey Brief. It should show the following:

- The deepest bed level at each section, both hard bed (solid) and silt line (dashed);
- The water level at each section;
- The bank crest levels derived from crest point levels shown on the cross sections, the left bank as a dashed line and the right bank as a bold line;
- The extent and level of any concrete sill or apron together with appropriate label ; and
- The section number and chainage of each section and the altitudes of each of the plotted points. The chainage shall be quoted to the nearest metre except when the scale of the survey makes it appropriate to quote the chainage to decimetres.

Each bridge, overhead crossing, weir, etc. should be shown on the longitudinal section with its critical levels (soffit, invert, deck, crest etc.) indicated. Where soffit and invert levels have been surveyed at both upstream and downstream elevations both will be labelled on the longitudinal section.

The water line for each day should be labelled at its limits with the appropriate date. Tributary channels are to be measured and depicted where they cross the bank crest line. Three points are usually adequate to describe a ditch, but more should be taken where the tributary is large.

Where the feature takes the form of a controlling structure such as a weir, sluice or overfall, then a complete cross-section should be measured. The tributary name should be added as a label.

Field drains and other infall structures 250mm diameter and greater should be measured with either invert or soffit surveyed. Individual diameter sizes and appropriate bank indicators shall be added as labels together with either a soffit or invert level. The existence of a flap valve shall be added as a label.

Side weirs, etc, which are not part of the main channel shall be shown with critical levels as variations to the bank crest.

Where changes in the levels of bank, bed or water level occur between cross-sections, these changes should be measured and added to the longitudinal section. The longitudinal section should represent an accurate and complete profile of the channel to ensure that low spot and level changes are identified.

To aid clarity insets shall be used at locations where detail is dense.

Photography

Digital photographs should be taken for each structure and should include a levelling staff to indicate scale. Both the upstream and the downstream faces of bridges should be photographed. Photographs should be taken from an appropriate distance to allow the structure to be viewed in context with its close surroundings. Photographs will be provided

with labels quoting the name of the bridge and road number, if one exists, plus the chainage to the face photographed.

Sufficient levels must be taken along the bank crest and any walls or embankments along the channel for the bank geometry and flood defence to be easily identifiable. A description of the material of the bank, natural or man-made embankment (e.g. earth, brick wall, fence, etc.) should be provided at regular intervals with photographs being provided in support. Location of photographs should be identified by the label attached to the closest survey point.

Presentation and Format of Data

The data to be supplied by the Surveyor should be in a specific format for loading into ERA's hydraulic modelling and GIS suite of programs (data format for the suitable software is provided by ERA).

Channel survey data should also be supplied in x, y, z format as an excel spreadsheet and geographically referenced.

GPS Datum Levels

Channel cross-sections and longitudinal profiles shall be supplied in 3-D AutoCad Version xxx6 (.dwg) digital format surveyed to GPS datum.

4.6.5 Data Merging

Merging of electronic surface data is common during highway design. Better data is usually collected within the highway area, while the data for the area outside the expected cut/fill lines is less precise.

Because watershed limits fall well outside the highway cut/fill lines, hydraulic engineers must negotiate with the data that has multiple resolutions. Electronic data is available in various forms differentiated by software products, type of data structure (DEMs and TINs), coordinate systems (UTM, State Plane, Latitude-Longitude), units (metres), resolution and datum. When merging data in different forms, care must be taken to ensure proper conversion prior to merging.

Standardizing all data to the most current format is the best way to ensure compatibility. There are tools available to accomplish the data "translation."

A more serious issue in data merging is caused by differences in data resolution. For example, a digital surface model developed using a photogrammetric method is typically of a lower resolution compared to a surface model developed using a field data collection survey. When merging the data, elevation differences at the boundaries of the different data areas must be carefully reconciled.

There is often a problem with artificial pits (sinks) and peaks due to the creation of DEMs and TINs. The engineer must evaluate the data and correct these inconsistencies.

Accuracy of Data

In any engineering computations, it is important to understand the limitations of accuracy of the computations based on the accuracy of the input data. In step-backwater computations utilizing HEC-RAS, ISIS or Mike 11, several factors have significant effects on the accuracy of the results (eg. accuracy of the survey data, spacing between cross sections, correct establishment of upstream and downstream study limits, and selection of roughness coefficients etc.).

Most field surveys of channel and floodplain cross sections are recorded to an accuracy of 0.031m. If the survey truly represents the cross-sections of the reach of the stream being studied to a 0.031m accuracy, the greatest accuracy that would result from a step-backwater computation could be no more than 0.031m. Any results expressed more precisely than 0.031m are simply due to the mathematics.

The accuracy of aerial survey technology for generating cross-sectional coordinate data is governed by mapping industry standards. Cross sections obtained from contours of topographic maps developed by photogrammetric methods are generally not as accurate as those generated from field data collection methods. Aerial photography can supplement field survey cross-sections. The use of aerial elevation survey technology permits additional coordinate points and cross-sections to be obtained at small incremental cost, and the coordinate points may be formatted for direct input into commonly used water surface profile computer programs such as HEC-RAS, ISIS and Mike 11.

For further information on determining the relationships between the following parameters, refer to the US Army Corps of Engineers' publication Technical Paper No. 114:

- Survey technology and accuracy employed for determining stream cross-sectional geometry;
- Degree of confidence in selecting Manning's roughness coefficients; and
- The resulting accuracy of hydraulic computations.

This publication also presents methods of determining the upstream and downstream limits of data collection for a hydraulic study requiring a specified degree of accuracy.

Upstream and Downstream Study Limits

Establishment of the upstream and downstream study boundaries for water profile calculations are required to define limits of data collection and subsequent analysis. Calculations must be initiated sufficiently far enough downstream to ensure accurate results at the structure, and continued sufficiently upstream to accurately determine the impact of the structure on upstream water surface profile. Underestimation of the upstream and downstream study lengths may produce less than desired accuracy of results and may eventually require additional survey data at higher costs than applied to initial surveys. On the other hand, significant overestimation of the required study length can result in greater survey, data processing, and analysis costs than necessary.

The downstream study length is governed by the effect of errors in the starting water surface elevation on the computed water surface elevations at the structure (see Figure 4.1). When possible, the analysis should start at a location where there is either a known (historically recorded) water surface elevation or a downstream control where the profile passes through critical depth.

Observed downstream high water marks are relatively common for calibration of models to historical events, but are unlikely to be available for evaluations of hypothetical events such as the 1% chance event. Alterative starting elevations are needed for stream conditions where high water marks and control locations are nonexistence or are too far downstream to be applicable. Two commonly applied starting criteria are critical depth and normal depth. The starting location should be far enough downstream so that the computer water surface profile converges to the base (existing condition) water surface profile prior to the bridge/culvert location.

The upstream study length is the distance to where the profile resulting from a structure-created head loss converges with the profile for the undisturbed condition. The magnitude

of the water surface profile change and the upstream extent of the structure-induced disturbance are two of the primary criteria used to evaluate the impacts of modified or new structures.

Regression analyses were performed by the Hydrological Engineering Centre to develop prediction equations for determining study limits in 1986. HEC-2 model base datasets were run for a variety of starting conditions and structure head loss values. The resulting equations and associated monographs provide the capability for determining the extent for required survey and mapping and other hydraulic parameter data collection.

The adopted regression equations are:

$L_{dc} = 6600*HD/S$

$L_{dn} = 8000*HD^{.8}/S$

$L_{u} = 10{,}000*HD^{.6}*HL^{.5}/S$

Where:

L_{dc} = downstream study length (along the main channel) in metres for critical depth starting conditions.

L_{dn} = downstream study length (along the main channel) in metres for normal depth starting conditions.

HD = average reach hydraulic depth 1% chance flow area divided by cross-section top width) in metres.

S = average reach slope in m/km.

HL = head loss between 0.1524, and 1.524 metres at the channel crossing structure for a 1 in 100 year design flood.

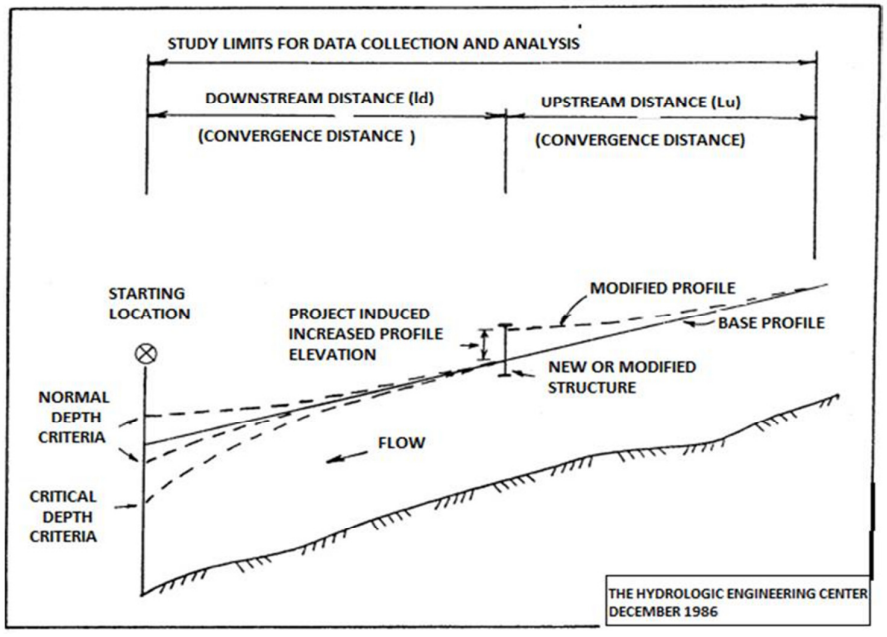


Figure 4-2: Profile study limits

4.7 Field Reviews

4.7.1 On Site Inspection

Field reviews shall be made by the Hydraulics Engineer in order for him/her to become familiar with the site. The most complete survey data cannot adequately depict all site conditions or be substituted for personal inspection by someone experienced in drainage design. Factors that most often need to be confirmed by field inspection are:

- Selection of roughness coefficients;
- Evaluation of apparent flow direction and diversions;
- Flow concentration;
- Observation of land use and related flood hazards;
- Geomorphic relationships;
- High water marks or profiles and related frequencies;
- Existing structure size and type;
- Bank erosion;
- Debris problems;
- Scour; and
- Existence of wetlands.

A visit to the site where the project will be constructed shall be made before any detailed hydraulic design is undertaken. This may be combined with a visit by others, such as the highway and structural designers and local road personnel. The hydraulic designer may visit the site separately, however, because of interests that are different from the others and the time required obtaining the required data.

Before making the field visit, the designer should determine if the magnitude of the project warrants an inspection or if the same information can be obtained from maps, aerial photos, or by telephone calls. The designer needs to consider the kind of equipment that will be needed, and most importantly, critical items at the site.

The drainage field visits can include the taking of photographs. These can consist of views looking upstream and downstream from the site, as well as along the contemplated highway centreline in both directions. If details of the streambed and banks are not clear, additional photographs along with structures in the vicinity both upstream and downstream shall be taken. Close up photographs complete with a scale or grid may be taken to facilitate estimates of the stream bed gradation.

It is important to seek local testimony regarding high water marks during the site inspection. A consensus opinion of a group shall be considered reliable testimony as to the high water mark. This is particularly valuable in corroborating other field observations.

The forms and figures to be used for identifying and cataloguing field information are illustrated on Forms 4-1 and sample Form 4-2.

4.8 Data Evaluation

4.8.1 Objectives

Once the required data have been collected, the next step is to compile it into a usable format. The drainage designer must ascertain whether the data contains inconsistencies or other unexplained anomalies that might lead to erroneous calculations or results. The analyst must draw all of the various pieces of collected information together, and fit them

into a comprehensive and accurate representation of the hydrological and hydraulic characteristics of a particular site.

4.8.2 Evaluation

Experience, knowledge, and judgment are important parts of data evaluation. It is in this phase that reliable data can be separated from less reliable data, and historical data combined with data obtained from measurements. The designer, for consistency, shall evaluate the data and identify any changes from established patterns. Reviews shall be made of previous studies, old plans, etc., for types and sources of data, how the data were used, and indications of accuracy and reliability. Historical data must be reviewed to determine whether significant changes have occurred in the catchment and whether these data can be used. The designer, for purposes of accuracy and reliability, should always subject data to careful study.

Basic data, such as stream flow data derived from non-published sources, shall be evaluated and summarized before use. Maps, aerial photographs, Landsat images, and land use studies shall be compared with one another and with the results of a field survey and any inconsistencies resolved. To help define the hydrological character of the site or region under study and to aid in the analysis and evaluation of data, general references that may be available shall be consulted and compared with the criteria specified in *Chapter 5: Hydrology*.

4.8.3 Sensitivity

Often, sensitivity studies can be used to evaluate data and the importance of specific data items to the final design. Sensitivity studies consist of conducting a design with a range of values for specific data items. The effect on the final design can then be established. This is useful in determining what specific data items have major effects on the final design and the importance of possible data errors. Time and effort should then be spent on the more sensitive data items making sure these data are as accurate as possible. This does not mean that inaccurate data are accepted for less sensitive data items, but it allows prioritization of the data collection process given a limited budget and time allocation.

The data evaluation shall result in as reliable a description of the site as possible that can be made within the allotted time and with the resources committed to this effort. The effort of data collection and evaluation shall be commensurate with the importance and extent of the project and/or facility.

4.9 Documentation

An important part of the design or analysis of any hydraulic facility is the accompanying documentation. Appropriate documentation of the design of any hydraulic structure is essential because of:

- Justification of expenditure of public funds;
- Future reference by engineers (when improvements, changes, or rehabilitations are made to the highway facilities);
- Information leading to the development of defence in matters of litigation; and
- Public information.

Frequently, it is necessary to refer to plans, specifications and analysis long after the actual construction has been completed. Documentation permits evaluation of the performance of structures after flood events to determine if the structures performed as anticipated or to

establish the cause of unexpected behaviour, if such is the case. In the event of a failure, it is essential that contributing factors be identified to avoid recurring damage.

Table 4-1: Sources of Data

Principal Hydrology Data	Sources of Data
Meteorological Data	National Meteorological Service Agency Addis Ababa Ethiopia
Regional and Local Flood Studies, River Basin Master Plan, stream flow records	Ministry of water and Energy Addis Ababa Ethiopia
Surveyed High Water Marks	Site Visit
Topographic Maps and Aerial Photos	Ethiopian Mapping Authority Addis Ababa, Ethiopia
Geological Maps	Ministry of Mines Addis Ababa, Ethiopia
Soils and Land Use Maps	Ministry of Agriculture Addis Ababa, Ethiopia

Form 4-1: Field investigation Form

Project Name: _____ Date:_____

Consultant: _____

Client Name: _____

River name: _____

Station Name: _____ Easting: _____ Northing: _____ Elevation _____

Span Width:	_____	Abutment Condition:	_____
No. of Span:	_____	Pier Condition (if):	_____
Opening height:	_____	Foundation Condition:	_____
Width:	_____	River Bed Material:	_____
Direction of flow:	_____	River Bank material:	_____
Land Use:	_____	River Bed slope:	_____
Catchment characteristic:	_____	Highest water mark:	_____
Inlet type and condition:	_____	Outlet type and condition:	_____
Flow obstruction Restriction (if any):	_____	Over flooding length (if any):	_____

Other site specific findings : _____

Sketch

Add channel geometry sketch below

Add cross sectional profile of the river U/s and D/s sketch below

Bridge Photo no _____ U/s Photo no_____ D/s Photo no_____

Summary and Remarks:

_____

_____

_____

Form4-2: Example Field investigation Form

Project Name:	*Billa - Gutine Road*			Date	*21/09/12*
Consultant:	*ME Consulting Plc.*				
Client:	*Ethiopian Road Authority*				
River name:	*Anger River*				
Station No:	*84+650*	Easting	*243859*	Northing *1059998*	Elevation *1310.18*
Span:	*26m*		Abutment condition	*Fair*	
No. of Span:	*Single*		Pier Condition (if any)	*No pier*	
Opening height:	*7.4 m*		Foundation Condition	*Invisible*	
Width:	*8.32m*		River Bed Material	*Invisible*	
Direction of flow	*Right to Left*		River Bank material	*Soil, Vegetated*	
Land Use	*Cultivated land and Cultivated scattered trees*		River Bed slope	*Gentle slope*	
Catchment characteristic	_____		Highest water mark	*0.6m below the deck level*	
Inlet type and condition	*Fair , Vegetations on banks*		Outlet type and condition	*N/A*	
Low obstruction Restriction (if)	*Straight*		Over flooding length (if any)	*Over floods the banks ,more than 200m both sides at D/s.*	
Other site specific findings	*The channel was constricted at the crossing location which results over flooding at the D/S of the bridge for more than 200m. There is a gauging station at the U/s*				

Bridge View

U/s and D/s Cross Sectional view of the river



Summary and Remarks

The Existing Anger river bridge was in affair condition before the flood. However, based on the site visit findings the free board at the time of the peak flood was less than the design recommendation, the consultant will carry out Hydrological and Hydraulic analysis to check the adequacy of the existing structure and will give recommendation based on the analysis finding

4.10 References

1. Accuracy of Computed Water Surface Profiles, U. S. Army Corps of Engineers, Dec 1986.
2. HY-11, Survey Accuracy, McTrans Center.
3. AASHTO Drainage Guidelines, Chapter 2.
4. HEC 19.
5. CDOT Drainage Design Manual, Chapter 6.
6. U.S. Army Corps of Engineers. Accuracy of Computer Water Surface Profiles. Technical Paper No. 114. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, California, 1986.

APPENDIX 4A - SAMPLE DATA

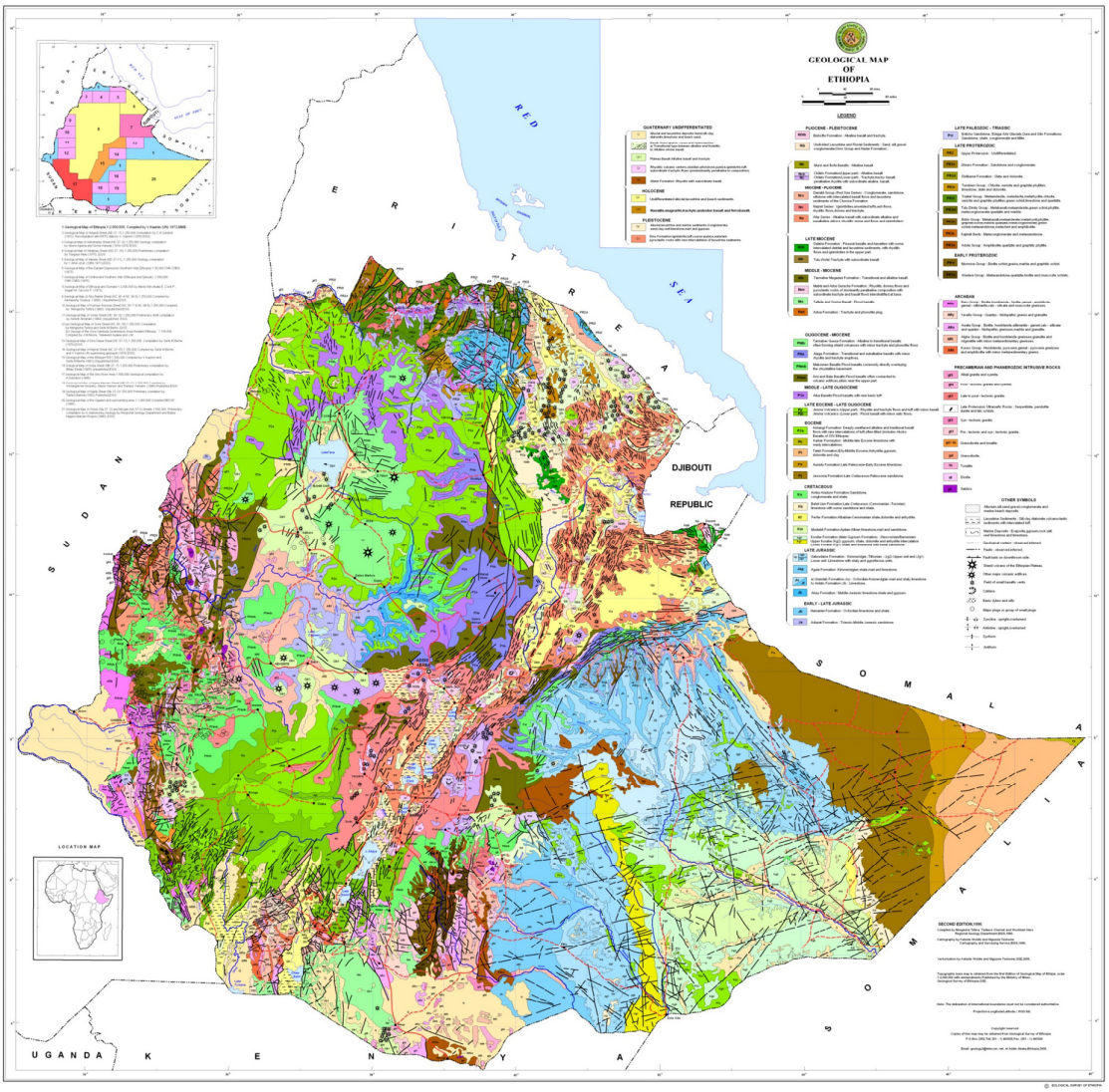


Figure 4A1: Geological Map of Ethiopia, 2nd edition

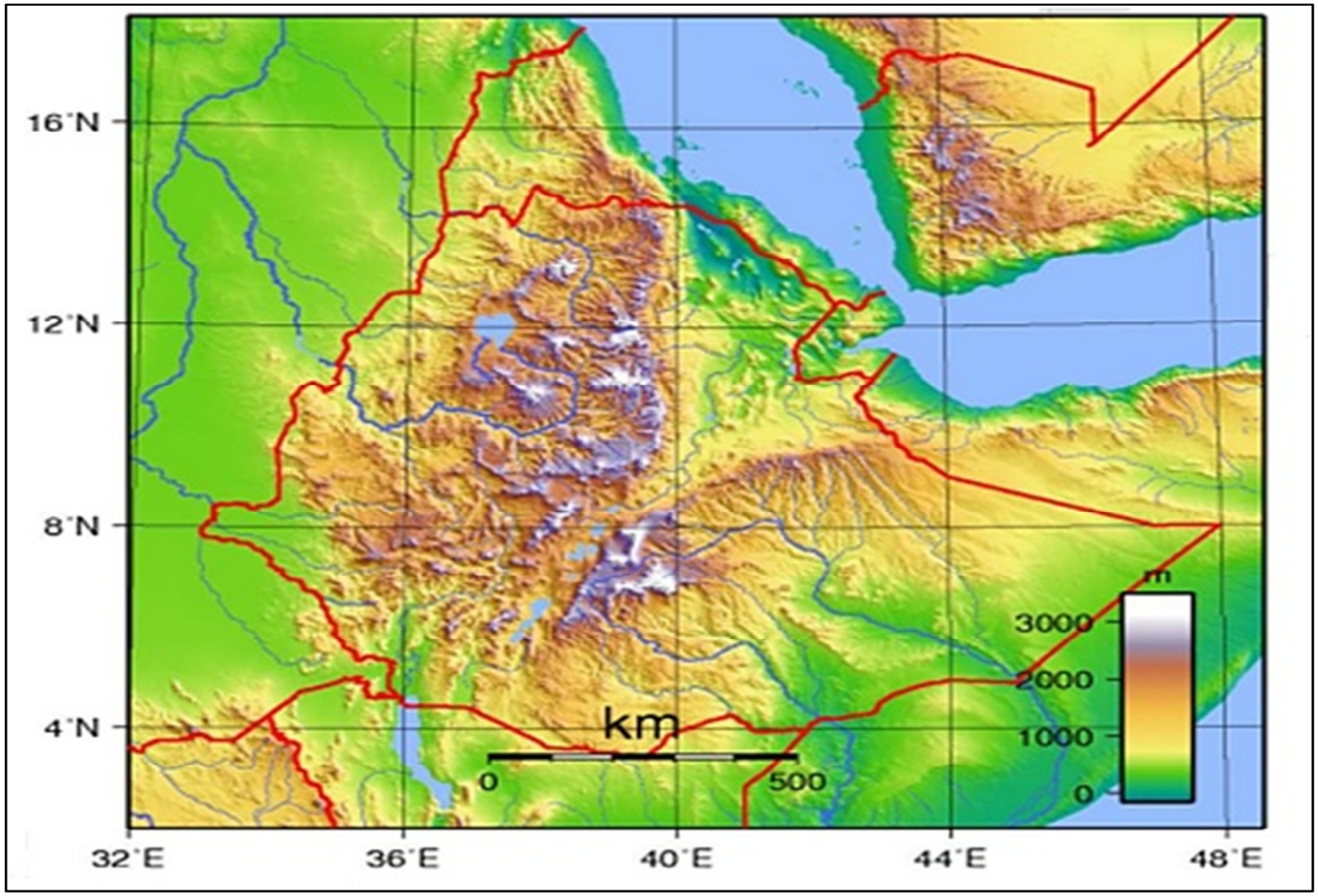


Figure 4A2: Topography of Ethiopia

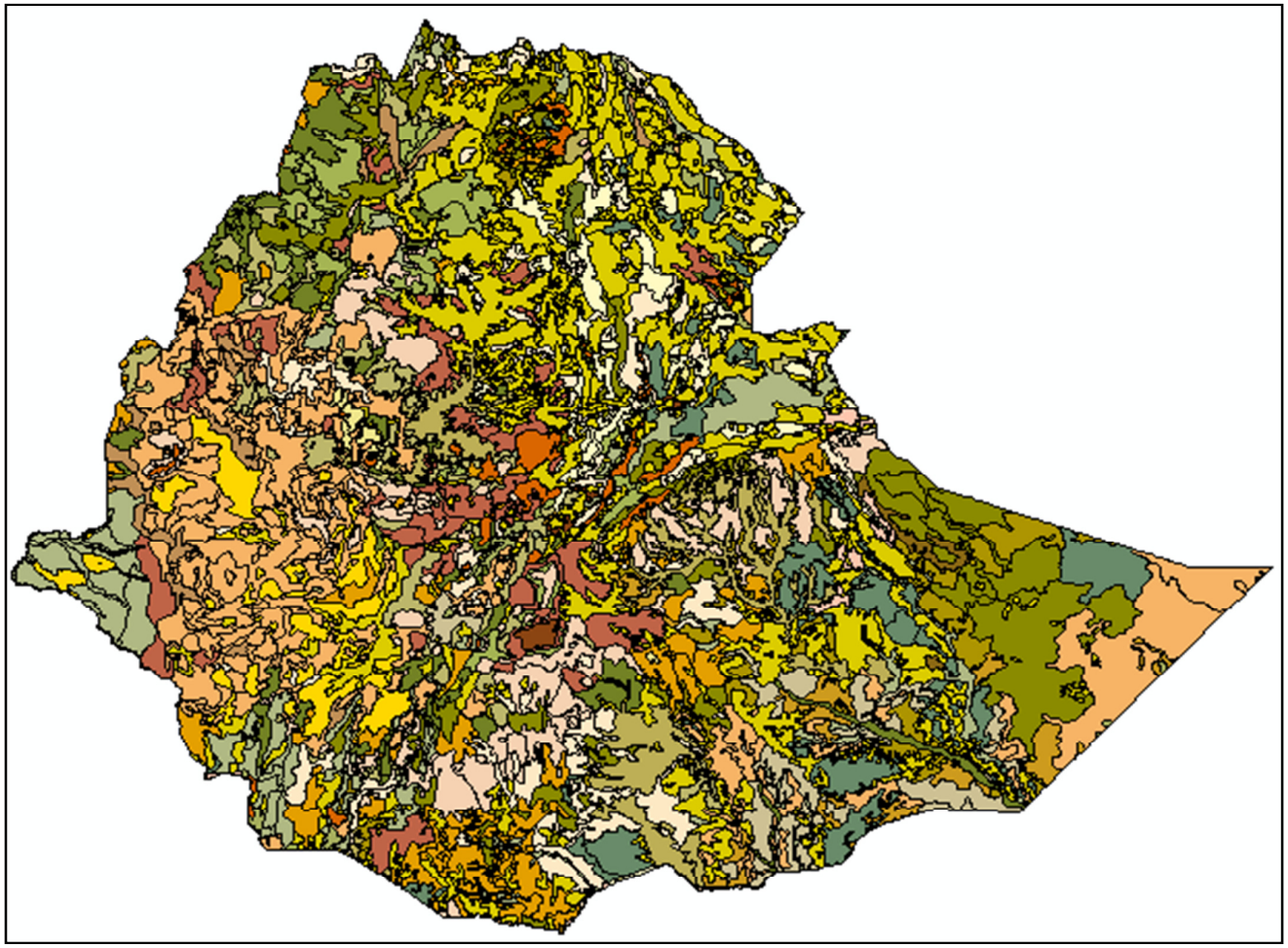


Figure 4A3: Soil Map of Ethiopia

5 HYDROLOGY

5.1 Introduction

Hydrology is the study of the properties, distribution, and effects of water on the earth's surface, and in the soils, underlying rocks, and atmosphere. For the purpose of this manual, hydrology will deal with estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic meters per second (m^3/s) and hydrographs as discharge per time. For structures that are designed to control volume of runoff, like detention storage facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest.



Photo 5-1: Drainage Basin along a Highway

(Source: ERA Website).

The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of highway drainage structures. Errors in the estimates will result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary. On the other hand, it is important to realise that any hydrologic analysis is only estimation. Although some hydraulics analysis is necessary for all highway drainage structure design, the extent of such studies should be commensurate with the hazards associated with the hydraulics structures and with other concerns, including economic, engineering, social, and environmental factors. Because hydrology is not an exact science, different hydrologic flow estimation methods developed for determining flood runoff may produce different results for a particular situation. Therefore, the engineer should exercise sound engineering judgment to select the proper flow estimation method or methods in estimation design flows. While performing the hydrological and hydraulics analysis for the design of highway drainage systems, the hydraulic engineer should recognize and evaluate potential environmental problems that would impact the specific design of a drainage structure early in the design process.

This chapter will discuss basic hydrologic concepts and provide recommendations for further research on data collection and establishment of national flow estimation techniques for Ethiopia. However, internationally recognized flow estimation methods are recommended for use until a national flow estimation method is established. The users of this manual should be aware that these methods are developed for specific regions based on specific data and they need to be calibrated with local data in Ethiopia before the result

can be used in confidence for project design work. When local data is not available, advice should be sought from an experienced hydrologist with sound hydrological knowledge and experience to undertake the hydrological analysis. The internationally available flow estimation methods are discussed and referenced here but original publications should be utilized to ensure the correct application of computational procedures.

5.2 Definition and Symbols

The following are concepts that are important in a hydrologic analysis. These concepts will be used throughout the remainder of this chapter in dealing with different aspects of hydrologic studies:

Antecedent Moisture Conditions	The soil moisture conditions of the catchment area at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably they affect the peak discharge only in the lower range of flood magnitudes – approx. below the 15-year event threshold. As floods become rarer, antecedent moisture has a rapidly decreasing influence on runoff.
Depression Storage	The natural depressions within a catchment area that store runoff. Generally after the depression storage is filled, runoff will begin.
Frequency	The number of times a flood of a given magnitude can be expected to occur on average over a long period of time. Frequency analysis is the estimation of peak discharges for various recurrence intervals. Another way to express frequency is with probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in any year.
Hydraulic Roughness	A composite of the physical characteristics that influence the flow of water across the earth's surface, whether natural or channelized. It affects both the time response of a catchment area and drainage channel, as well as the channel storage characteristics.
Hydrograph	A graph of the time distribution of runoff from a catchment area.
Hyetograph	A graph of the time distribution of rainfall over a catchment area.
Infiltration	A complex process of allowing runoff to penetrate the ground surface and flow through the upper soil surface. The infiltration curve is a graph of the time distribution at which this occurs.
Interception	The storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.
Lag Time	The time from the centroid of the excess rainfall to the peak of the hydrograph.
Peak Discharge	Sometimes called peak flow. The maximum rate of flow of water passing a given point during or after a rainfall event.
Rainfall Excess	The water available to runoff after interception, depression storage and infiltration has been satisfied.
Stage	The elevation of the water surface above some elevation datum.
Time of Concentration	The time it takes a drop of water falling on the most remote point hydraulically in the catchment area to travel through the catchment area to the outlet.
Unit Hydrograph	The direct runoff hydrograph resulting from 1mm effective rainfall event that has a specific temporal and spatial distribution and which lasts for a unit duration of time. The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one millimeter of runoff from the catchment area.

To provide consistency within this chapter, as well as throughout this manual, the following symbols will be used. These symbols were selected in the existing manual because of their wide use in hydrologic publications.

Table 5-1: Symbols

Symbol	Definition	Units
A	Catchment area	hec, sq.km.
BDF	Basin development factor	%
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
C_t, C_p	Physiographic coefficients	-
d	Time interval	hours
DH	Difference in elevation	m
I	Rainfall intensity	mm/hr
IA	Percentage of impervious area	%
I_a	Initial abstraction from total rainfall	mm
K	Frequency factor for a particular return period and skew	-
L	Lag hours	
l	Length of mainstream to furthest divide	m
L_{ca}	Length along main channel to a point opposite the Catchment area centroid	km
M	Rank of a flood within a long record	-
n	Manning roughness coefficient	-
N	Number of years of flood record	years
P	Accumulated rainfall	mm
Q	Rate of runoff	m^3/s
q	Storm runoff during a time interval	mm
R	Hydraulic radius	m
RC	Regression constant	-
RQ	Equivalent rural peak runoff rate	m^3/s
S or Y	Ground slope	m/m, m/km or %
S	Potential maximum retention storage	mm
SCS	Soil Conservation Service	-
SL	Main channel slope	m/m
S_L	Standard deviation of logarithms of peak annual floods	-
ST	Basin storage factor	%
T_B	Time base of unit hydrograph	hours
t_c or T_c	Time of concentration	min or hours
T_L	Lag time	hours
T_r	Snyder's duration of excess rainfall	hours
UQ`	Urban peak runoff rate	m^3/s
V	Velocity	m/s
X	Logarithm of the annual peak	-

5.3 Hydrologic Design Principles

Chapter 2: "Standards and Departures from Standards" defines the general principles for hydrological and drainage design in accordance with this manual. The hydrological data available for Ethiopia is generally limited so the procedures that can be applied are

consequently imprecise. No specific standards or definitive criteria for hydrological analysis are suitable for recommendation at this time. For standard procedures to be adopted confidently, storm water runoff coefficients, design parameters and procedures shall be calibrated.

5.3.1 Peak Discharge versus Frequency Relations

Highway drainage systems are designed to convey predetermined discharges in order to avoid significant flood hazards. Provisions are also made to convey floods in excess of the predetermined discharges in a manner that minimizes flood hazards. Flood discharges are often referred to as peak discharges as they occur at the peak of the stream's flood hydrograph (discharge over time). Peak discharge magnitudes are a function of their expected frequency of occurrence, which in turn relates to the magnitude of the potential damage and hazard. (*All the methods described in this manual allow determination of peak discharge*.)

The highway designer's main interest in hydrology rests in estimating runoff and peak discharges for the design of highway drainage facilities. The highway drainage designer is particularly interested in the development of a flood versus frequency relation, a tabulation of peak discharges versus the probability of occurrence or exceedance.

The flood frequency relation is usually represented by a flood frequency curve. A typical flood frequency curve is presented in Figure 5-1. In this example, the discharge is plotted on the ordinate and the probability of occurrence or exceedance is expressed in terms of return interval and plotted on the abscissa.

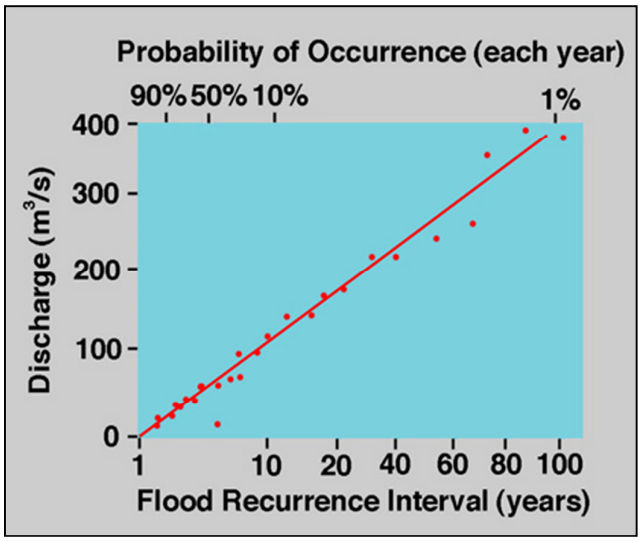


Figure 5-1: Typical Flood Frequency Curve

Also of interest is the performance of highway drainage structures during the frequently occurring low flood flow periods. Because low flood flows do occur frequently, the potential exists for lesser amounts of flood damage to occur more frequently. It is entirely possible to design a drainage facility to convey a large, infrequently occurring flood with an acceptable amount of floodplain damage only to find that the accumulation of damage from frequently occurring floods is intolerable. Adequate analysis and provision for frequently occurring floods (low flows) should be provided.

5.3.2 Flood Hydrographs

In addition to peak discharges, the hydraulics engineer is sometimes interested in the flood volume and time distribution of runoff. Sometimes, there might be a requirement to use flood hydrographs to route floods through culverts, bridges, flood storage structures, and other highway facilities. This is especially important when an embankment is built across a floodplain and flood compensation storage area is required to be provided in order to compensate for the lost natural floodplain storage area due to the construction of the road embankment and to mitigate the flood risk to the upstream areas.

By accounting for the stored flood volume, the hydraulics engineer can often expect lower flood peak discharges and smaller required drainage facilities than would be expected without considering storage volume. ***This should be considered in the Afar and Somali region of Ethiopia where flash flood occurs for a limited period***. Instead of designing large culverts or bridges with a required opening to pass the flash flood, which occurs for a limited period, it will be more economical to provide storage areas upstream of the proposed crossing to temporary hold the flood and release it at controlled manner.

However, this depends on the availability of land and geomorphology of the site. In this situation, hydrographs are required instead of peak discharges. Flood hydrographs can also be used for estimating inundation times of flow over roadways and possible closure time as well as pollutant and sediment transport analyses (see Figure 5-2 for typical hydrograph).

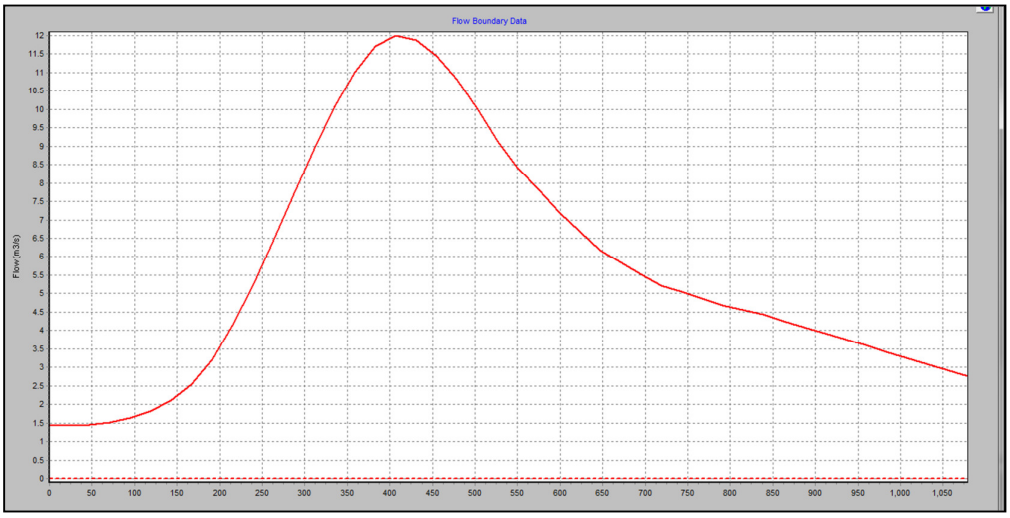


Figure 5-2: Sample Flood Hydrograph

5.3.3 Unit Hydrographs

A unit hydrograph represents the response of a watershed to a rainfall excess of unit depth and specific duration. The unit is 1mm that is, the volume associated with an excess rainfall of 1mm distributed over the entire contributing area (see Figure 5-3 for sample unit hydrograph).

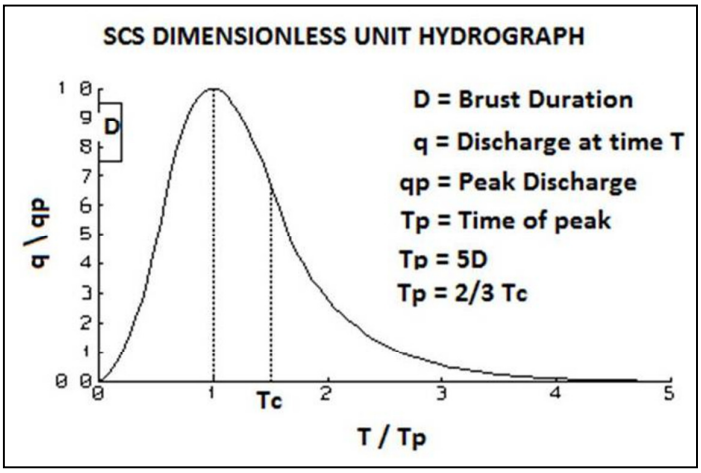


Figure 5-3: Sample SCS Dimensionless Unit hydrograph

The response of a watershed to rainfall is considered to be a linear process. This has two implications that are useful to the drainage designer: The concepts of proportionality and superposition. For example, the runoff hydrograph resulting from a two-unit pulse of rainfall of a specific duration would have ordinates that are twice as large as those resulting from a one-unit pulse of rainfall of the same duration. In addition, the hydrograph resulting from the sequence of two one-unit pulses of rainfall can be found by the superposition of two one-unit hydrographs. Thus, by determining a unit hydrograph for a watershed, it is possible to determine the flood hydrograph resulting from any measured or design rainfall using these two principles.

5.3.4 Surveys

Since hydrologic considerations can influence the selection of a highway corridor and the alternate routes within the corridor, site specific studies and investigations shall be undertaken at the Planning Stage (see *Chapter 3: Planning*) and refer to the ERA Route Selection Manual. In addition, special studies and investigations may be required at sensitive locations (example wetlands and important archaeological sites). The magnitude and complexity of these studies shall be commensurate with the importance and magnitude of the project and problems encountered. Typical data to be included in such surveys or studies are: topographic maps; aerial photographs; stream flow records; historical high water marks; flood discharges; and locations of hydraulic features such as reservoirs, wetlands, water projects, and designated or regulatory floodplain areas (for more detailed data requirement, refer to Chapter 4).

5.3.5 Flood History

All hydrologic analysis shall consider the flood history of the area and the effect of these historical floods on existing and proposed structures. The flood history includes the historical floods and the flood history of any existing structures near the proposed project area.

Interagency Coordination

A hydrologic analysis is prerequisite to identifying flood hazard areas and determining those locations where construction and maintenance will be unusually expensive or hazardous. Since many levels of government plan, design, and construction of highway and water resource projects might affect each other, interagency coordination is desirable

and often necessary. In addition, agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analysis. Interagency coordination between the Ministries of Transport, Water & Energy, Agriculture and Telecommunication etc. is very important.

Documentation

Experience indicates that the design of highway drainage structures should be documented adequately. Frequently, it is necessary to refer to plans and specifications long after the actual construction has been completed. Thus, it is necessary to fully document the collected raw hydrological data and the results of all hydrologic analysis including the hydrological modeling reports and model results.

5.3.6 Sources of Hydrological Information

Hydrological information can be obtained from the Ministry of Water and Energy. The type and source of information available for hydrologic analysis will vary from site to site and it is the responsibility of the designer to determine what information is available and applicable to a particular analysis. A comprehensive list of data sources is included in *Chapter 4: Data Collection, Evaluation and Documentation.* However, hydrological data is scarce in Ethiopia. The available rainfall and flow data is also of poor quality.

Factors Affecting Flood Runoff

Runoff: Two main factors influence runoff from a watershed: precipitation and abstractions. Precipitation in Ethiopia is represented as rainfall. Rainfall rate distributions within a watershed vary both temporally and spatially. For most determinations of peak flows for use in road drainage design and analysis efforts, it is commonly assumed that rainfall rates not to vary within the watershed during the rainfall event. However, this assumption only holds true for small and medium size catchments.

Generally, the entire volume of rainfall occurring on a watershed does not appear as runoff. Losses, known as abstractions, tend to reduce the volume of water appearing as runoff. Abstractions of precipitation in its evolution into runoff are numerous. However, for the typical highway drainage design problem, only six abstractions are commonly considered. These are discussed below in the order of their significance to the rainfall runoff.

- Infiltration: Infiltration is the amount of precipitation that percolates into the ground within the watershed. This abstraction is a function of soil type and characteristics, terrain slopes, and ground cover. In order to assess infiltration, detailed topographical, soil data and land cover/use map is required. However, availability and quality of this data is very limited in Ethiopia. In areas where these data are scarce, the engineer should use alternative methods.
- Depression storage: Depression storage is the precipitation stored permanently in inescapable depressions within the watershed. It is a function of land use, ground cover, and general topography (see comments above).
- Detention storage: Detention storage is the precipitation stored temporarily in streams, channels, and reservoirs within the watershed. It is a function of the general drainage network of streams, channels, ponds, etc. within the watershed.
- Interception: Interception is the precipitation that serves to first “wet” the physical features of the watershed (e.g., leaves, rooftops, pavements). It is a function of most watershed characteristics.

- Evaporation: Evaporation is the precipitation that returns to the atmosphere as water vapor by the process of evaporation from water concentrations. It is mostly a function of climate factors, but it is associated with exposed areas of water surface.
- Transpiration: Transpiration is the natural process of vegetation foliage which generates this. It is a function of ground cover and density of vegetation.

The specific consideration of each of these abstractions is not usually explicit in the many hydrologic methods available. An extensive data set is also required in order to make a full assessment of the above abstractions, which is currently not available. ***It is recommended that primary data collection technique to be developed and primary data acquired to assess the above abstractions.***

Watershed Area Information: Most runoff estimation techniques use the size of the contributing watershed as a principal factor. Generally, runoff rates and volumes increase with increasing drainage area. The size of a watershed will not usually change over the service life of the road drainage facility. However, agricultural activity and land development may cause the watershed area to change over time. Flow diversions and catchment area changes due to urbanization and other development inevitably will also occur at some point in the future. The drainage designer should try to identify or otherwise anticipate such changes. Urbanization, deforestation, infrastructure, development including roads, railways, and water resources projects (dams and reservoirs) will be anticipated to occur in the future in Ethiopia. This should be taken into consideration and consultation with the federal and regional government offices is required in order to establish and identify areas allocated for future development.

On the other hand, the watershed shape will also affect rainfall runoff rates. *For example, a long, narrow watershed is likely to experience lower runoff rates than a short, wide watershed of the same size and other characteristics*. Some hydrologic methods accommodate watershed shape explicitly or implicitly; others may not. If a drainage area is unusually irregular extremely narrow, the designer should consider using a hydrologic method that explicitly accommodates this watershed shape.

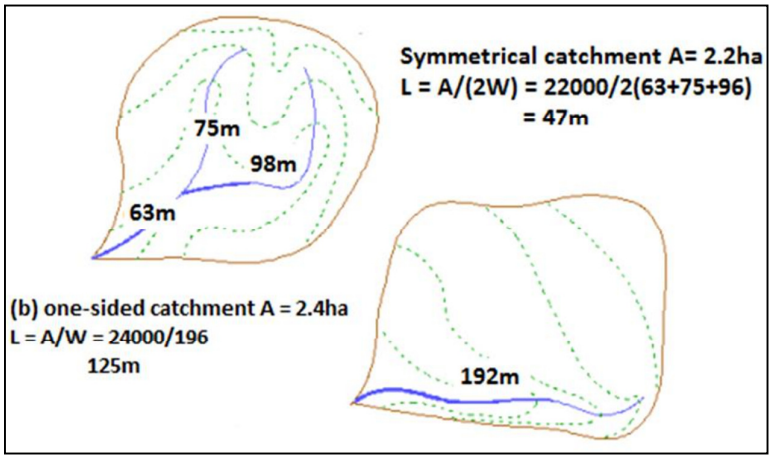


Figure 5-4: Catchment shape

(Courtesy of Alan A. Smith Inc)

The response of a watershed to rainfall runoff may vary with respect to the direction in which a storm event passes. Generally, for design purposes, the orientation of the

watershed may be ignored because it is common to assume uniform rainfall distribution over the watershed.

Geographic Location: The geographic location of the watershed within Ethiopia is a significant factor for the drainage designer. Rainfall intensities and distributions, empirical hydrologic relations, and hydrologic method applications vary because of geographic location. In Ethiopia, the rainfall distribution is mainly affected by topography. The designer should use hydrologic methods and parameters that are appropriate for the specific location in the country.

Land Use: Land use significantly affects the parameters of a runoff event. Land use and human activity within most watersheds vary with respect to time. For example, a rural watershed can be developed into a commercial area in a short period. Factors subject to change with general variations in land use include the following:

- Permeable and impermeable areas;
- Vegetation;
- Minor topographic features; and
- Drainage systems.

All of these factors usually affect the rate and volume of runoff that may be expected from a watershed. *Therefore, it is important to consider current land use and future potential land use change in the development of the parameters of any runoff hydrograph.*

Soil Type: Soil type can have considerable effect on the discharge rates of the runoff hydrograph; the soil type directly affects the permeability of the soil and thus the rate of rainfall infiltration. The Natural Resources Conservation Service (NRCS) is a good repository for information about soils but the soil parameters specified in NRCS should be calibrated and validated with site-specific local data within Ethiopia before it can be used in hydrological analysis.

Topography: Topography mostly affects the rate at which runoff occurs. The rate of runoff increases with increasing slope. Furthermore, rates of runoff decrease with increasing depression storage and detention storage volumes. Many methods incorporate a watershed slope factor, but fewer methods allow the designer to consider the effects of storage on runoff. The drainage designer should take this limitation of the chosen method into consideration.

Vegetation: In general, runoff decreases with increasing density of vegetation; vegetation helps to reduce antecedent soil moisture conditions and increases interception such as to increase initial rainfall abstractions. Vegetative characteristics can vary significantly with the land use; therefore, consider this in the assessment of potential future land use changes of the watershed.

Detention Storage Systems: Detention storage systems are common in urban areas mostly aimed at controlling increased runoff from developed areas. The ERA drainage designer should identify any detention storage systems that might exist within the subject watershed. A detention storage facility can attenuate the runoff hydrograph, thus reducing the peak discharge. The ERA may design facilities that involve detained storage to conform to Ethiopian environmental regulations, to cooperate with local regulations, or where flood attenuation is deemed necessary.

Flow Diversions: Flow diversions within a watershed can change the runoff travel times and subsequent peak discharge rates. They can decrease discharge at some locations and increase discharge elsewhere. Flow diversions may redirect flow away from a location

during light rainfall but overflow during heavy rainfall. Assess the likely effect of diversions that exist within the watershed. Also, ensure that the potential impact of necessary diversions resulting from the highway project is minimized.

Channelization: Channelization in an urban area includes the following:

- Improved open channels;
- Curb and gutter street sections;
- Inverted crown street sections; and
- Storm drain systems.

Any of these channelization types serve to make drainage more efficient. This means that flows in areas with urban channelization can be greater, and peak discharges occur much more quickly than where no significant channelization exists.

Future Conditions: Changes in watershed characteristics and climate directly affect runoff rates. A reasonable service life of a designed system is expected. Therefore, base the estimate of design flood upon runoff influences within the time of the anticipated service life of the facility.

Prediction Information: In general, consider estimates for future land use and watershed character within some future range. It is difficult to predict the future, but the designer should make an effort at such a prediction, especially with regard to watershed characteristics. Local and federal officials and planners can often provide information on potential future characteristics of the watershed. In estimating future characteristics of the watershed, consider changes in vegetative cover, surface permeability, and controlled drainage systems. Climatic changes usually occur over extremely long periods of time however, it is reasonable to consider potential climatic changes during the anticipated life span of the facility.



Photo 5-2: Vegetation and land use significantly affect watershed hydrology



Photo 5-3 Deforestation alters the runoff patterns and volumes in watersheds



Photo 5-4: Debris flows interrupt the traffic and cause extensive damage.

5.4 Design and Check Frequency

As with other natural phenomena, occurrence of flooding is governed by chance. The chance of flooding is described by a statistical analysis of flooding history in the subject watershed or in similar watersheds. Because it is not economically feasible to design a structure for the maximum possible runoff from a watershed, the designer must choose a design frequency appropriate for the structure.

All proposed structures are sized using the specified design frequency as provided in Table 2-1 of Chapter2. The following text further develops the choices presented in this table.

A design frequency shall be selected to match the facility's cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints, considering the magnitude and risk associated with damages from larger flood events. With long highway routes having no practical detour, where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. In selecting a design frequency, potential upstream land use changes that could reasonably occur over the anticipated life of the drainage facility shall be considered.

Detailed evaluation of the above factors for every site to select a design frequency is generally lengthy and very tedious. The time required may not be justified on every project. The frequencies in Chapter 2, Table 2.1 shall be used unless site conditions

warrant an economic analysis to determine a new frequency. However, no interstate highway or expressway shall be designed for less than a 50-year flood frequency.

Storm drain systems in urban areas have two separate drainage systems. One of the systems is the minor system to handle the frequently recurring storms. The minor system consists of underground piping, natural waterways and required appurtenances to protect against the average storms. The second system is the major system to handle the large infrequent flows. The major system includes street flow and other overflow provisions to pass the infrequent, large flows and protect against excessive property damage and ponding depth (see Figure 5.5 below).

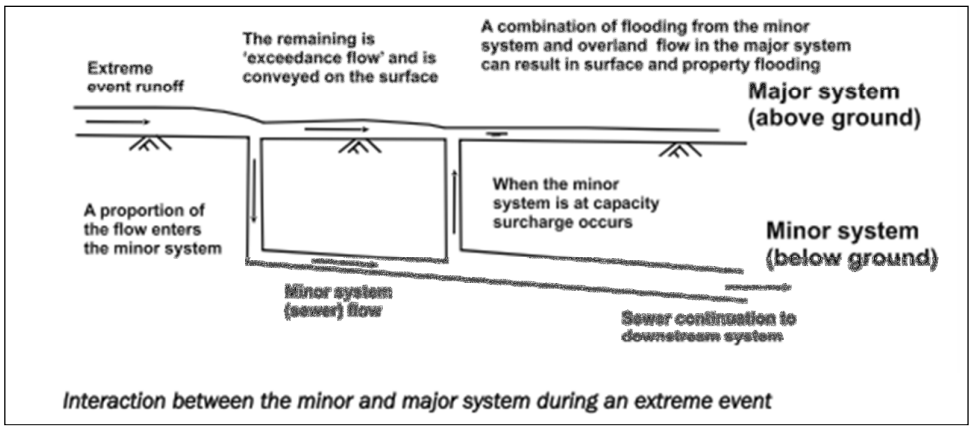


Figure 5-5: Urban Storm Drainage Systems

(Source: Designing for exceedance in urban drainage- good practice guide, CIRIA C635, 2006)

Where the standards of another agency, such as a city, exceed the values given in chapter 2, the design shall meet the higher standards unless they are demonstrably inappropriate. Appropriate local city and regional drainage criteria shall be used on off-system projects where improvements are being proposed.

Bridges and culverts may need to be rehabilitated before they are structurally deficient and it is possible that some of these existing structures do not meet current design criteria or frequency provided in this manual. All drainage structures proposed for rehabilitation should be evaluated to determine design frequency under current design criteria and any hazards related to undersized structures. If the existing structure is undersized, the engineer should consider the relative risk and cost associated with rehabilitation or replacement of the drainage structure to determine the most appropriate action.

The frequency with which a given flood can be expected to occur is the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. If a flood has a 20 percent chance of being equaled or exceeded each year, over a long period of time, the flood will be equaled or exceeded on an average of once every five years. This is called the **return period or recurrence interval** (RI). Thus, the exceedance probability equals 100/RI.

The designer should note that the 5-year flood is not one that will necessarily be equaled or exceeded every five years. There is a 20 percent chance that the flood will be equaled or exceeded in any year; therefore, the 5-year flood could conceivably occur in several consecutive years. The same reasoning applies to floods with other return periods.

Mathematically, the probability of non-occurrence is:

$$P_n = (1 - 1/f)^n \quad (5\text{-}1)$$

Where f = year flood; and n = return period.

The probability of occurrence is:

$$P = 1 - P_n \; or \; 1 - (1 - 1/f)^n \quad (5\text{-}2)$$

For example, the chance of a 50-year flood, f, or greater event occurring in any 10 year period, n, is: $P = 1- (1-1/50)^{10} = 0.18$

This is an 18% chance of occurrence and an 82% chance of non-occurrence. The equation can be applied to floods with other return periods.

Table 5-2: Flood Probabilities

Flood of years frequency	Percentage probability that a flood of frequency "f" or larger will occur at least once during the period of "n" years			
	1 year	10 year	25 year	50 year
2 years	50%	99%	99%	99%
10 years	10%	64%	93%	99%
25 years	4%	34%	64%	87%
50 years	2%	18%	40%	64%
100 years	1%	10%	22%	30%

(Source: Texas Department of Transportation Hydraulic Design Manual, 2006)

The values in this column are often used to define the probability of flood occurrence; e.g., a 100 year flood is also defined as a 1% flood.

Cross Drainage: A cross drainage facility shall be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design shall be such that the backwater (the headwater) caused by the structure for the design storm does not:

Increase the flood hazard significantly for upstream properties;

Overtop the highway; or

Exceed a certain depth on the highway embankment.

Based on these design criteria, a design-involving roadway overtopping of short duration for floods larger than the design event is an acceptable practice. Usually, if overtopping is allowed, the structure may be designed to accommodate a flood of some lower frequency with overtopping.

Storm Drains: A storm drain shall be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design shall be such that the storm runoff does not:

- Increase the flood hazard significantly for local properties;
- Encroach on to the street or highway so as to cause a significant traffic hazard; or
- Limit traffic, emerging vehicle, or pedestrian movement to an unreasonable extent.

Based on these design criteria, a design involving a street or road inundation of short duration for floods larger than the design event is an acceptable practice.

5.4.1 Design Discharge Estimation

Estimating peak discharges for various recurrence intervals is one of the most common engineering challenges faced by drainage structure designers. This is the main challenge in Ethiopia where there is no adequate primary data to base the analysis. During the manual revision work and the site visit, it was noted that many of the bridges either are overtopped or washed away by floods by being unable to accommodate the flow generated by the catchment upstream of the crossing. Therefore, flow estimation methods should be calibrated with locally collected data. Discharge determination can be divided into two general categories:

- **Gauged sites** - the site is at or near a gauging station and the stream flow record is of sufficient length, then statistical analysis should be used to estimate peak flows.
- **Ungauged sites** - the site is not near a gauging station and no stream flow record is available.

Hydrologic procedures that can be used for both categories stated above will be discussed in the next sections.

5.4.2 Rainfall vs. Frequency

Certain hydrologic procedures use rainfall and rainfall frequency as the basic input instead of flood frequency. It is also commonly assumed that the 10-year rainfall will produce the 10-year flood. Depending on antecedent soil moisture conditions, and other hydrologic parameters, there may not be a direct relationship between rainfall and flood frequency. For projects on small basins (under 25.9 sq. km) it is usually not practicable to distinguish between rainfall frequency and runoff frequency due to lack of available data. However, based on research in UK, 140 years rainfall is expected to generate a 100 year flood.

5.4.3 Check Flood Frequencies

From the standpoint of drainage structure utilization, design a structure that will operate in the following manner:

- Efficiently for lesser floods;
- Adequately for the design flood; and
- Acceptably for greater floods.

For these reasons, it is often important to consider floods of other magnitudes. To define the peak flows for frequencies other than the design frequency, use the approach of developing a general flood-frequency relation for the subject site.

The review (check) flood shall be at least as provided in *Chapter 2*, Table 2-1. In some cases, a flood event larger than the specified review flood might be used for analysis to ensure the safety of the drainage structure and downstream communities. In some cases, evaluate a flood event larger than the 100-year flood (super-flood) to ensure the safety of the drainage structure and downstream communities.

A 500-year flood analysis is required for checking the design of bridge foundations against potential scour failure.

If a catastrophic failure of a bridge or culvert can release a flood wave that would result in loss of life, disruption of essential services, or excessive economic damage, the bridge or culvert design should be evaluated in terms of a probable maximum flood or PMF. For example, a culvert under normal flood operation will act like a dam. PMF considers the

conditions under which the culvert/dam may fail. The PMF is not related to an event frequency but is a specialized analysis.

5.5 Hydrologic Analysis Method

Stream flow measurements for determining a flood frequency relationship at or near a site are usually unavailable. In such cases, it is an accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. In general, results from using several methods should be compared, not averaged. The discharge that best reflects local project conditions, with the reasons documented, should be used.

In general, follow the following guidelines to select Flow Estimation Methods (FEMs):

- Compare results from several methods;
- Use the discharge that appears to best reflect local project conditions. Averaging of results of several methods is not recommended; and
- Document reasons supporting the selection of the results.

The peak discharge is adequate for design of conveyance systems such as storm drains, open channels, culverts, and bridges. However, if the design necessitates flood routing through areas such as storage basins and complex conveyance networks, a flood hydrograph is required.

Chapter 2 defines the general principles for hydrological and drainage design in accordance with this manual. The hydrological data available for Ethiopia is generally limited so the procedures that can be applied are consequently imprecise and they should be used with caution. No specific standards or definitive criteria for hydrological analysis are suitable for recommendation at this time as the available data is not sufficient to warrant establishing specific criteria for hydrological analysis for Ethiopia. Although there is more data with long records since the publication of the 2002 ERA Drainage Manual, the quality of the available data is still inadequate. For standard procedures to be adopted storm water runoff coefficients and other parameters shall be calibrated and validated with locally available data.

Many hydrologic methods are available for estimating peak discharges and runoff hydrographs. The omission of other flow estimation methods from this manual does not necessarily preclude their use. The methods to be used and the circumstances for their use are listed below in Table 5.3. Each method has a range of application and limitations, which the engineer should clearly understand prior to using them. Basin size, hydrologic and geographic region, dominant precipitation type, elevation, and level of development are all important factors. The engineer must ensure that the selected hydrologic method is appropriate for the basin conditions and that sufficient data is available to perform the required calculations. If possible, the method should be calibrated to local conditions and flood history. Several methods will be appropriate for predicting peak flood rates and volumes at most sites. Comparison of hydrologic prediction methods on recurrence interval curves should be performed in selection of peak flow rates for a drainage design.

The following methods and sources can be used in determining peak flood magnitudes for design of road drainage structures in Ethiopia.

The following are some of the most widely used flow estimation methods:

- Rational Method;
- NRCS Runoff Curve Number Methods;
- Statistical analysis of stream data; and
- Regional regression equations.

Rational Method: The Rational Method provides estimates of peak runoff rates for small urban and rural watersheds of less than 50 hectares (0.5 square km) and in which natural or man-made storage is small. It is best suited to the design of urban storm drain systems, small side ditches and median ditches, and driveway pipes. It shall be used with caution if the time of concentration exceeds 30 minutes. Rainfall is a necessary input for this method of flow estimation. See Section 5.6 for more information on the Rational Method.

NRCS Runoff Curve Number Methods: The Natural Resources Conservation Service (formerly Soil Conservation Service) developed the runoff curve number method as a means of estimating the amount of rainfall appearing as runoff. Technical Release 20 (TR 20) employs the Runoff Curve Number Method and a dimensionless unit hydrograph to provide estimation of peak discharges and runoff hydrographs from complex watersheds. The procedure allows the designer to estimate the effect of urbanization, channel storage, flood control storage, and multiple tributaries. TR 20 can be applied to the design of culverts, bridges, detention ponds, channel modification, and analysis of flood control reservoirs. Technical Release 55 (TR 55) is a simplified form of TR 20 for use in estimating peak discharges for small watersheds (urban and rural) whose time of concentration does not exceed 10 hours. TR 55 includes a hydrograph development procedure; however, where hydrograph determination is necessary, use TR 20 or another hydrograph procedure. See Section 5.7 for more information on the NRCS Runoff Curve Number Methods. The unit hydrograph used by the SCS method is based upon an analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions in USA. The rainfall depth to be used as input is presented in Table 5-19 at the end of this chapter.

However, the SCS Curve Number method is applicable to small catchments (maximum area 6,500 ha) with a time of concentration for any sub-area of 0.1 – 10 hours (NRCS, 2002).

As detailed in Highway Hydrology - Hydraulic Design Series -2, The SCS method should be used on watersheds that are homogeneous in CN; where parts of the watershed have CNs that differ by 5, the watershed should be subdivided and analyzed using a hydrograph method, such as TR-20 (SCS, 1984).

The SCS method should be used only when the CN is 50 or greater and the tc is greater than 0.1 hour and less than 10 hours. The computed value of Ia/P should be between 0.1 and 0.5.

The method should be used only when the watershed has one main channel or when there are two main channels that have nearly equal times of concentration; otherwise, a hydrograph method should be used.

Other methods should also be used when channel or reservoir routing is required, or where watershed storage is either greater than 5 percent or located on the flow path used to compute the etc.

Statistical Analysis of Stream Gauge Data: Where stream gauge data are available, stream gauge data can be used to develop peak discharges. The Ministry of Water & Energy keeps annual stream gauge data. The method commonly used for estimating the peak discharges is usually Log-Pearson Type III distribution. However, as the record length is increased, a Log-Normal distribution or General Extreme Value (GEV) distribution could also be used. The recent data analysis demonstrated that GEV can be used to estimate the peak flow in Ethiopia. See Section 5.9 for more information on statistical analysis of stream gauge data. It is recommended that the distribution method, which gives a best fit to the record data, should be used.

Regional Regression Equations: Regional regression equations provide estimates of peak discharge for watersheds in specific geographic regions. See Section 5.10 for more information on regional regression methods and equations.

Of these possible hydrologic methods based on the available data, it should be noted that, at the present time, only the Rational and SCS methods are applicable to the whole country. Regression equations and derivations from stream gauging (Gumbel, Log Pearson, General Extreme Value) are often preferred but rely on data not available. For this reason, only the Rational Method and the SCS method are given in this chapter.

Table 5-3: Application and limitation of flood estimation methods

Method	Input data	Recommended maximum area(km^2)	Return period of flood that could be determined (years)
Rational Method	Catchment area, watercourse length, average slope, catchment characteristics, rainfall intensity	<0.5	2 – 200, PMF
SCS Method	Catchment area, watercourse length, length to catchment centroid (centre), mean annual rainfall, veg. type ,soil cover and synthetic regional unit hydrograph	0.5 to 65	2 – 200, PMF
Synthetic Hydrograph Method	Catchment area, watercourse length, length to catchment centroid (centre), mean annual rainfall, veg. type and synthetic regional unit hydrograph	0.5 to 5000	2 -200
Empirical Methods	Catchment area, watercourse length, distance to catchment centroid (centre), mean annual rainfall	No limitation large areas	2 – 200, PMF
Statistical Method	Historical flood peak records	No limitation, large areas	2 – 200 (depending on the record length)

5.5.1 Hydrologic Analysis Procedure Flowchart

The hydrologic analysis procedure flowchart Figure 5-6 shows the steps needed for the hydrologic analysis and the designs that will use the hydrologic estimates.

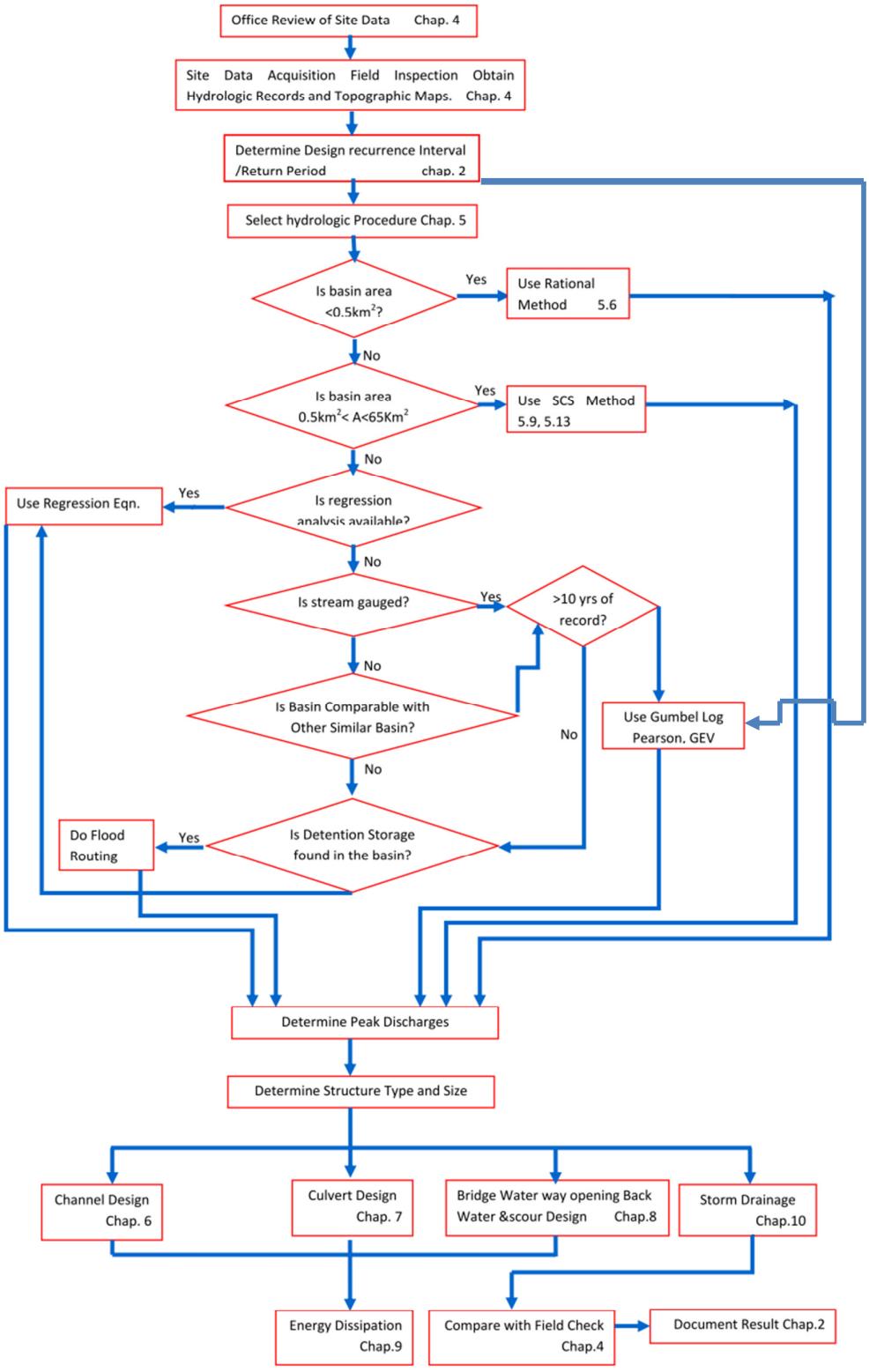


Figure 5-6: Hydrologic Analysis Procedure Flowchart

5.6 Time of Concentration

5.6.1 Description

Hydrologic methods require an estimation of the time of concentration. This section provides guidance on ways to estimate time of concentration and covers the following topics: description, flow components, and procedure to estimate time of concentration.

The time of concentration (Tc) is used in the Rational Method to determine the critical rainfall duration, which can then be combined with an appropriate rainfall intensity duration frequency (IDF) relation to establish the required design rainfall intensity. The Tc is the time required for water to flow from the most remote point of the basin to the location being analyzed.

A storm equal to this duration will permit direct runoff to arrive from all points in the watershed concentrating at the outlet. This time measure is taken to be the critical time by many flood-estimating approaches, in that it is assumed that the use of any other time would result in a lower flood estimate. A shorter time, although resulting in higher rainfall intensity, will not permit the entire basin to contribute flow simultaneously. A longer duration allows the entire basin to contribute, but with a lower intensity.

Many different Tc formulas are available, and it is through the selection and use of these formulas result in greatest error that typically occurs in applying the Rational Formula to non-urban watersheds. While some error occurs in assigning runoff coefficients, as there is a large range of possible values for each surface condition, the realistic determination of the response time of the basin is the greatest challenge, particularly for natural (rural) basins. Bondelid et al (1982) found that upwards of 75% of errors in peak flow estimates can be attributed to errors in the time of concentration.

Examples of popular Tc equations can be found in most hydrology texts books, such as those by Chow et al (1988), Maidment (1993), and McCuen (1989), and the different equations generally give a wide range of estimates for any particular set of basin parameters. This situation reflects differences in the data sets used in deriving the equations, and illustrates that most equations are generally unreliable when applied to areas that are different from those used for their development.

The Kirpich and Hathaway equations are widely used to estimate time of concentration. However, the designer should be aware that both equations have been developed from limited data for a specific site. The Kirpich equation was developed from data for six agricultural watersheds in Tennessee, USA (ranging in size from 0.4 ha to 45 ha), with well defined channels and slopes ranging from 3% to 10% (Viessman and Lewis 1996). The Hathaway formula was developed on the basis of data from very small watersheds (<1.8ha), where the slopes were less than 1% and storm runoff was dominated by surface flow (MaCuen 1989).

Drainage design practitioners in Ethiopia should be aware of the limitation of any formula used to calculate time of concentration before applying it to estimate time of concentration to be used in the Rational Method.

5.6.2 Flow Path Consideration

There may be a number of possible paths to consider in determining the longest travel time. Identify the flow path along which the longest travel time is likely to occur. This is a trial and error process.

Generally, it is reasonable to consider the three following components of flow that can characterize the progression of runoff along a travel path: overland flow (sheet flow), shallow concentrated flow, and conduit and open channel flow (or concentrated channel flow).

For each drainage area, determine the distance (L) from the outlet of the drainage area to the most remote point. Determine the average slope (S) for the same distance.

For a specific drainage basin, the time of concentration consists of an inlet time plus the time of flow in a closed conduit or open channel to the design point. Inlet time is the time required for runoff to flow over the surface to the nearest inlet and is primarily a function of the length of overland flow, the slope of the drainage basin, and surface cover. Pipe or open channel flow time can be estimated from the hydraulic properties of the conduit or channel.

For each catchment area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Manning's Equation can be used to determine velocity (see Chapter 6: Open Channels).

Pipe or open channel flow time can be estimated from the hydraulic properties of the conduit or channel. Generally, for ERA application, it is reasonable to assume uniform flow and employ Manning's Equation for Uniform Flow for open channel and conduit flow considerations.

For open channel flow, consider the uniform flow velocity based on bank-full flow conditions. That is, the main channel is flowing full without flow in the overbanks. This assumption avoids the significant iteration associated with other methods that employ rainfall intensity or discharges (because rainfall intensity and discharge are dependent on time of concentration).

For conduit flow, in a proposed storm drain system, compute the velocity at uniform depth based on the computed discharge at the upstream. Otherwise, if the conduit is in existence, determine full capacity flow in the conduit, and determine the velocity at capacity flow. It is necessary to compare this velocity later with the velocity calculated during conduit analysis. If there is a significant difference and the conduit is a relatively large component of the total travel path, re-compute the time of concentration using the latter velocity estimate.

5.6.3 Peak Discharge Adjustments

In some cases, runoff from a portion of the drainage area that is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, it is possible to adjust the drainage area and time of concentration by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is necessary to estimate several different contributing areas and associated times of concentration to determine the design flow that is critical for a particular application.

5.6.4 Overland Flow Path Selection

In drainage system design, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often, the land will be graded and swales and streets will intercept the flow that reduces the time of concentration. Care should be exercised in selecting overland flow paths in excess of 60 m in urban areas and 120 m in rural areas.

(i) Calculation of the Time of Concentration for Overland Flow

Overland flow is the type of flow that occurs in small, flat or in upper reaches of catchments, where there is no clearly defined watercourse. Run-off, then, is in the form of thin layers of water flowing slowly over the fairly uneven ground surface. The kerby formula is recommended for the calculation of Tc in this case. It is only applicable to parts where the slope is fairly even.

$$T_c = 0.604\left(\frac{rL}{S^{0.5}}\right)^{0.467} \quad (5\text{-}3)$$

Where:

T_c = time of concentration (hours)

r = roughness coefficient obtained from Table 5-6

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)

S = Slope of the catchment $S = \frac{H}{1000L}$ (m/m) See figure 5-7

H = height of most remote point above outlet of catchment (m)

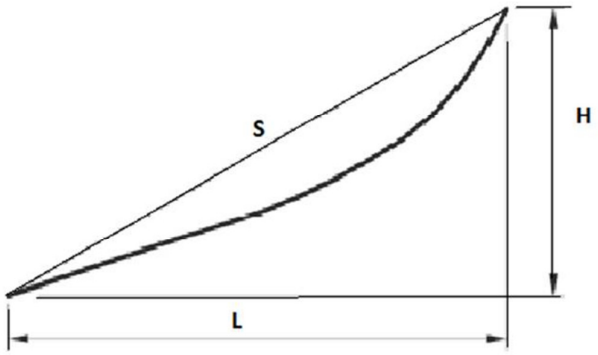


Figure 5-7: Slope definition for overland flow

(ii) Calculation of Time of Concentration for Defined Watercourses

In a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels was developed by the US Soil Conservation Service.

$$T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385} \quad (5\text{-}4)$$

Where:

T_c = time of concentration (hours).

L = hydraulic length of catchments measured along flow path from the catchment boundary to the point where the flood needs to be determined (km).

Sav = average slope (m/m).

The average slope may be determined graphically in two ways. The first procedure is based on the balance of areas obtained by balancing the areas above and below the line of the average slope as shown Figure 5.8. Alternatively, the formula developed by the US Geological Survey, and referred to as the 1085-slope method could be used Figure 5.9.

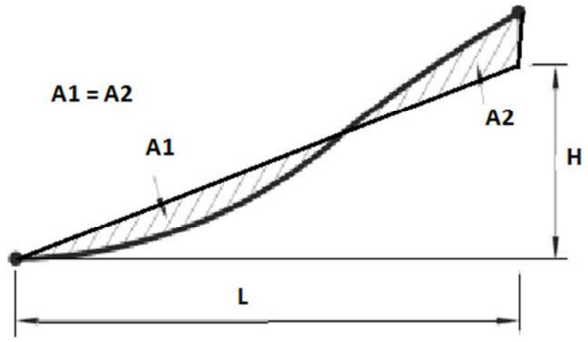


Figure 5-8: Slope according to weighted area method

In most cases, the longest water path includes both overland and channel flow. In large catchments, the channel flow is usually dominant, but in small catchments, it may be necessary to determine Tc as the sum of flow times, for overland and channel flow. To obtain a broad indication, it may be accepted that a defined watercourse exits when the average slope of the catchment is greater than 5 percent, and the catchment itself is larger than 5km^2.

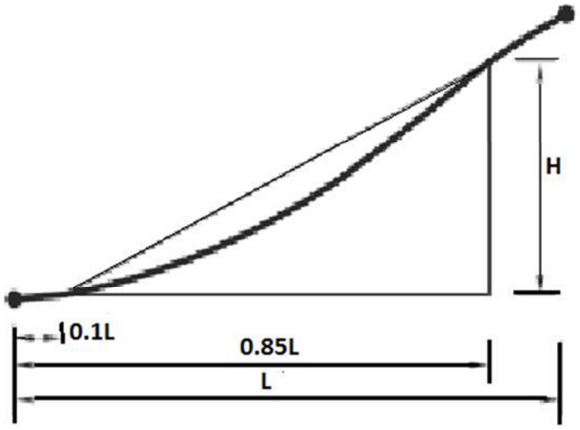


Figure 5-9: 1085-slope according to "US Geological survey"

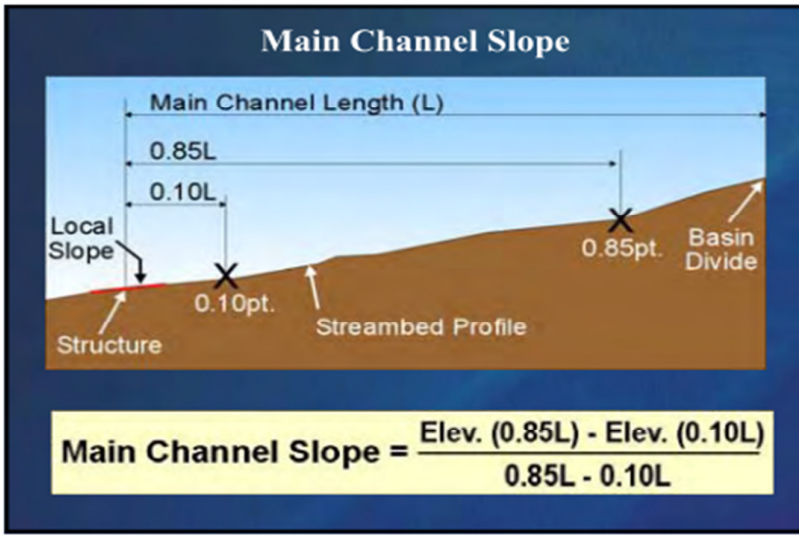


Figure 5-10: Calculation of main channel slope

The formula for determining the slope according to the 10 85-slope methods reads:

$$S_{av} = \frac{H_{0.85L} - H_{0.10L}}{(1000)(0.75L)} \; or \; S_{av} = \frac{H}{(1000)(0.75L)} \qquad (5\text{-}5)$$

Where:

Sav = average slope (m/m)
$H_{0.10L}$ = elevation height at 10% of the length of the watercourse (m)
$H_{0.805L}$ = elevation height at 85% of the length of the watercourse (m)
L = length of watercourse (km)
H = $H_{0.805L} - H_{0.10L}$ (m)

The height of waterfalls and high rapids are subtracted from the gross H value.

iii Calculation of the Time of Concentration for Urban Areas

In urban areas, the time of concentration should be determined, where applicable, by means of the flow velocities according to the Chezy or Manning's equation for uniform flow through representative cross-sections with representative slopes.

In road drainage, the volume of water that runs off as a result of a storm of less than 15 minute duration is usually not large; much of this runoff is absorbed in filling of watercourses. Times of concentration of less than 15 mins are thus generally not significant.

It is sound practice to calculate the average flow velocity (v = L/Tc) after determining Tc in order to ensure that it falls within realistic times. Typical value of the flow velocity ranges from 0.1 to 4m/s, depending on the natural conditions.

5.6.5 Common Errors

Three common errors should be avoided when calculating Tc. First, application of simplified general equations such as Kirpich for determining Tc can result in too short a time of concentration, particularly when the average basin slope varies significantly from the mean channel slope as in steep mountainous areas. Neglecting the overland flow time can also dramatically shorten the time of concentration thus increasing the design peak runoff**.**

5.6.6 The Status of the Ethiopian Hydrometric Register

The primary objective of the Ethiopian Hydrometric Register is to catalogue the national hydrometric monitoring networks and provide a range of reference and statistical information to enable the basic hydrometric data to be used more effectively by a rapidly expanding community of data users in Ethiopia. However, the Register is not structured to allow the selection of appropriate datasets for particular projects, and to assist in the interpretation of analyses based on nationally archived hydrological data.

Hydrometric data provide the foundation for both water science and water management. Skilful management and manipulation of rainfall, river flow and groundwater level data underpin the development of improved engineering design procedures. However, the development of multivariate relationships between flow statistics and catchment characteristics has to be based on catchments with good quality data and stations with long record length of data.

The Ethiopia Hydrometric Register comprises two major components cataloguing the Ethiopian surface water and, less comprehensively, groundwater monitoring networks. The Gauging Station Register provides details of around 1500 Ethiopian gauging stations, and the Well Register provides information relating to more than a 1000 index wells and boreholes.

5.6.7 Lack of detailed information of gauged data

Gauged flows are generally calculated by the conversion of the record of stage, or water level, using a stage-discharge relation, often referred to as the rating or calibration. However, no information was available how data is transmitted from the gauging station to the processing centre. There was no information what type of gauging instrument is used to record the data.

An example extracted from Scotland, UK, has been included here for demonstration purposes to show what information is required to be included with the record data to make an informed decision regarding the quality of the data and the station.

Example: 13002 Luther Water at Luther Bridge SEPA East.

Station: *Velocity-area station with cableway; 10m wide. Situation not ideal due to bend u/s and island d/s, but stage-discharge relation is regularly reviewed using routine gaugings. Stable bedrock control at low flows.*

Catchment: *Upper third of catchment is fairly steep (Grampian Mountains),the rest has moderate slopes. Lower 80% is on ORS, the remainder is metamorphic. Almost the entire catchment is covered by superficial deposits. Land use is forest (approx. half) and rough grazing at higher levels with arable and cattle elsewhere."*

It is recommended that a similar data recording procedure is adopted in Ethiopia.

5.6.8 Sources of Information

Responsibility for the collection and initial processing of hydrometric data in Ethiopia rests principally with the National Meteorological Service Agency of Ethiopia.

River flows in Ethiopia are often difficult to measure precisely – particularly in flood or drought conditions – and can be substantially affected by the geological and land- use characteristics of individual catchments, and by the net impact of water use patterns above each gauging station. Such artificial influences range from a large diminution in flows caused by a major abstraction for irrigation immediately upstream of the gauging station to the, normally, more subtle impact of land use change on river flow patterns. Groundwater levels may also be heavily influenced by man's activities – abstraction rates in particular. An appreciation of these effects is necessary to exploit the archived data most effectively. For this exercise, important material relating to the impact of changing patterns of water utilisation on river flow regimes and groundwater level behaviour was not available from The Ministry of Water & Energy.

5.6.9 The Ethiopian Gauging Station Network

More than 1500 rainfall-gauging stations have been identified across Ethiopia. However only around 32 gauging stations have been found to be suitable for inclusion in updating the existing data in the 2002 manual (refer table 5.4).

Identifying, quantifying and attributing hydrological change is an essential pre-requisite for the design of scientifically-based mitigation strategies to moderate the impact of future floods, and to minimise the flood risk to the infrastructure. However, the current network of gauging stations is mainly concentrated in the central part of Ethiopia (refer to figure 5-11). Most of the gauging stations outside the central region are not functional and the record length for some of them is too small to be used for statistical analysis. Hence, these data have not been utilised to update the manual.

In addition to the rainfall and flow data, borehole data to assess the ground water levels has also been collected from the Ministry of Water & Energy. River flows and groundwater levels in Ethiopia reflect more than just the intensity and distribution of rainfall and the magnitude of evaporation losses. Geology and land use influence river runoff and aquifer recharge patterns, and the natural variations of each are often substantially disturbed by the complex and evolving pattern of water utilisation for hydropower and irrigation. However the quality of the borehole data is poor and has not been utilised.

5.6.10 Rainfall Analysis

The daily highest rainfall data obtained from the National Meteorological Agency of Ethiopia has been statistically analysed using three methods of distribution analysis namely Generalized Extreme Value, Log Pearson-3 and Gumbel's Methods. Commercially available statistical analysis software, EasyFit 5.5 Professional has been utilised.

The reliability of the distributions is checked by the goodness of fit tests. The goodness of fit (GOF) tests measures the compatibility of a random sample with a theoretical probability distribution function. In other words, these tests show how well the distribution you selected fits to your data. The Anderson-Darling (AD), the Kolmogorov-Smirnov (KS), and the Chi-Squared tests are used for the goodness of fit test. The selection of the best fit method is based on the ranks given by the three fitness methods.

5.6.11 Rainfall Data Screening Process

Rainfall data obtained to update the data included in the previous manual has been evaluated and screened to make sure that the available data is fit for purpose. On some of the stations, some month's data were missing while on other stations, the whole year data is totally missing. There was no reason given by the Meteorological Service Agency why the data was missing. Various methods have been employed (refer to Section 5.9) to infill the missing data. However, given the uncertainty of the data and the low density of the gauging stations, the recorded data with the data gap has been rejected.

Outlier Identification, Retention, Modification, Deletion

Check on outliers has been undertaken on the recoded rainfall and flow data to identify any low or high outliers. Outliers are data points, which depart significantly from the trend of the remaining data. The retention, modification, deletion of these outliers can significantly affect the statistical parameters computed from the data, especially for small samples. All procedures for treating outliers ultimately require judgement involving both mathematical and hydrologic considerations. The procedure followed for detection and treatment of high and low outliers for this project are summarised below:

- When the station skew is greater than +0.4, test for high outliers should be considered first;
- When the station skew is less than -0.4 tests for low outliers should be considered first;
- Where the station skew is between ±0.4, test for both high and low outlier should be applied before eliminating any outliers from the data set. For detail procedure of outlier adjustment, refer to Bulletin#17B (ref 10) and ref v.chow etal.

Low and high outliers have been detected in some of the station data obtained. There is a total lack of information about the stations history and data validation. There is no clear information regarding the reliability of the data and the status of the station. In light of this, data with low and high outliers have been rejected instead of being adjusted. As the number of gauging stations and number of years with both low and high outliers are too small, the rejection of them will not make any difference on the outcome of the analysis.

5.6.12 Rainfall Curves

The results of the rainfall statistical analysis indicate that the country can be divided into several hydrological regions, which display similar rainfall patterns, as indicated on the map in Figure 5-13 at the end of this chapter. Using the statistical analyses described in the

previous sections, rainfall intensity-duration curves have been developed for commonly used design frequencies. Figures 5-16 through 5-23 at the end of this chapter show the curves prepared from presently available data.

5.6.13 Notes on Rainfall Analysis

The rainfall data available is too sparse to develop highly accurate intensity-duration-frequency curves. The 24-hour rainfall depth records were generally adequate to project the frequency of 24-hour rainfall depths. Based on the monthly rainfall depths and patterns, the country was divided into regions and sub-regions in 1989 by Dr Admassu Gebeyehu. It is recommended that the engineer to undertake further research on the rainfall region classification and revise the data.

Based on the statistical analysis undertaken as part of the current manual review work, the country could be further divided into more than 8 rainfall pattern regions. This task is outside the current scope of works and the current regional classification has been retained but this should be revised when the manual is updated in the future. The methodology adopted in the 2002 manual to generate the IDF curves has also been adapted to update the IDF curves with the latest rainfall data.

The amount of data available for shorter duration storms was too sparse for the development of intensity-duration-frequency curves, and was insufficient to do a frequency distribution plot for each rainfall period. In order to develop intensity-duration-frequency curves for each rainfall region, the ratios of the short duration data available were compared to the 24-hour data. Based on this comparison and making similar comparisons for published rainfall data from other country, principally the United States, it seemed that reasonable estimations of rainfall depths occurring in shorter periods could be expressed as a fraction of the 24-hour rainfall depth.

Many recommendations for depth-duration-frequency curves in the technical literature suggest a "broken-leg" approach such that the depth duration frequency equation for shorter duration rainfalls, less than one hour, is different from that derived for longer duration rainfalls. Because of the scarcity of data this approach was not taken and one curve was developed. The amount of rainfall data obtained for peak rainfall intensities of periods shorter than one-half hour was too limited to be useful. The curves presented are satisfactory for rainfall durations of one-half hour or more. Intensities for periods shorter than 15 minutes appear to be overestimated by the curves presented.

It is recommended in this manual that for the design of most drainage structures, the minimum time of concentration is taken as 15 minutes. The design of gutters and inlets may be based on shorter rainfall durations, but this isn't serious conservatism. The overall drainage system - drainage conduit - will usually be designed for storm duration of nearly 15 minutes or more, thus the most expensive part of the drainage system will not be unnecessarily over-designed.

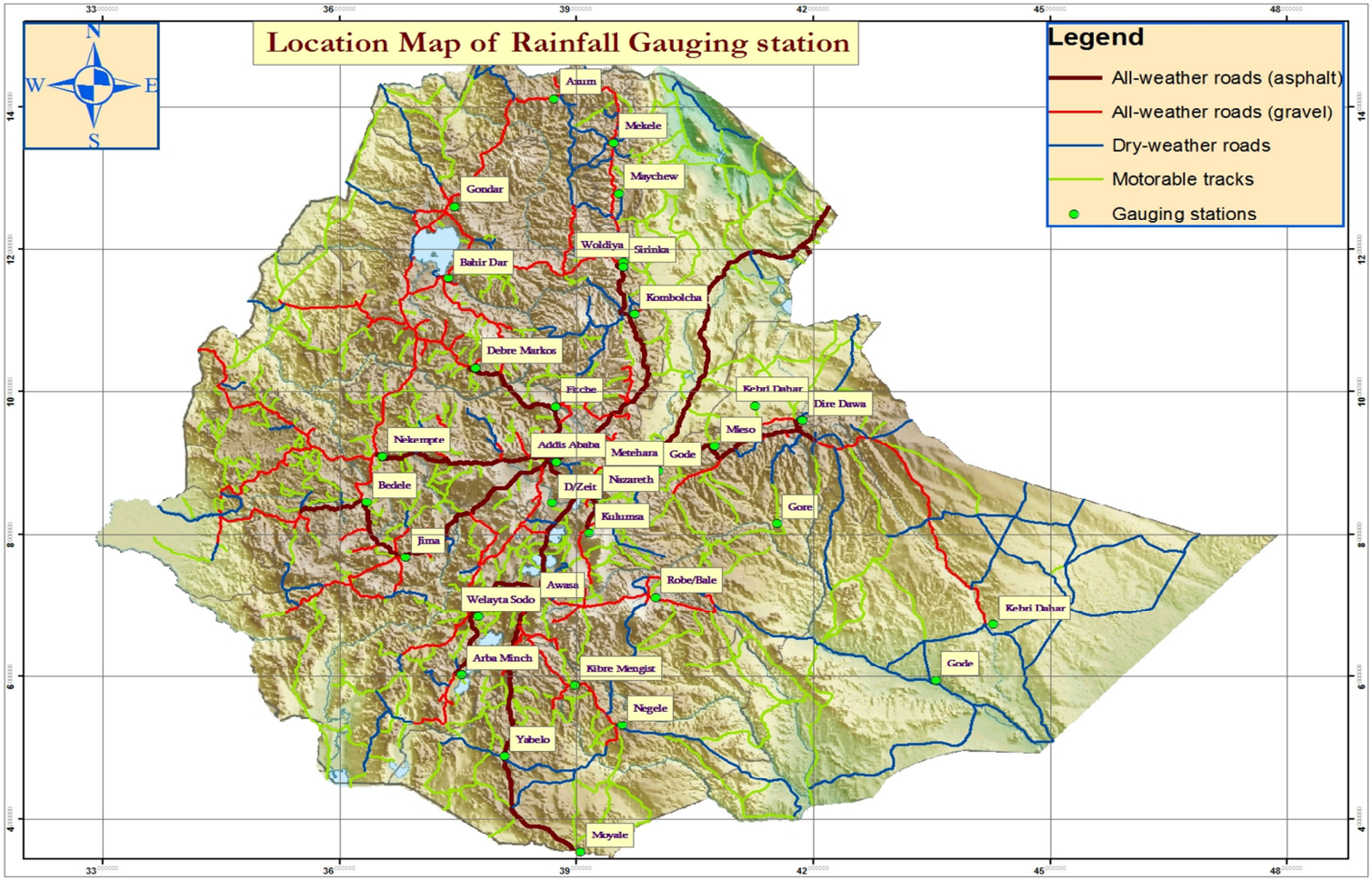


Figure 5-11: Location Map of Rainfall Gauging Stations

Table 5-4: Meteorology Stations (years of record through 2010)

Meteorological Region	Station	Years of Record	Meteorological Region	Station	Years of Record
A1	Axum	17	B	Bedele	39
	Mekele	46		Gore	56
	Maychew	32		Nekempte	40
A2	Gondar	52		Jima	54
	Debre Tabor	15		Arba Minch	23
	Bahir Dar	45		Sodo	49
	Debre Markos	55		Awasa	36
	Fitche	44	C	Kombolcha	57
	Addis Ababa	57		Woldiya	29
	Debre Zeit	55		Sirinka	27
A3	Nazareth	46	D1	Gode	33*
	Kulumsa	43		Kebri Dihar	40
	Robe/Bale	29	D2	Kibre Mengist	33
A4	Metehara	24		Negele	51
	Dire Dawa	58		Moyale	29
	Mieso	42		Yabelo	34

* Max. 24 hour rainfall not given

5.7 Rational Method

The Rational Method is most accurate for estimating design storm peak runoff for areas up to 50 hectares (0.5 square km). This method, while first introduced in 1889, is still widely used. Even though it has come under frequent criticism for its simplistic approach, no other drainage design method has achieved such widespread use.

5.7.1 Application

Some precautions shall be considered when applying the Rational Method:

- The first step in applying the Rational Method is to obtain a good topographic map and define the boundaries of the catchment area in question. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.
- In determining the runoff coefficient C value for the catchment area, thought shall be given to future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system. Also, the effects of upstream detention structures must be taken into account.

- Restrictions to the natural flow such as highway crossings and dams that exist in the catchment area shall be investigated to see how they affect the design flows.
- The charts, graphs, and tables included in this section are not intended to replace reasonable and prudent engineering judgment that should permeate each step in the design process.

5.7.2 Characteristics

Characteristics of the Rational Method that generally limit its use to 50 hectares include:

(1) The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long as or longer than the time of concentration. That is, the entire catchment area does not contribute to the peak discharge until the time of concentration has elapsed.

This assumption limits the size of the drainage basin that can be evaluated by the Rational Method. For large catchment areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows. Further, in semi-arid and arid regions, storm cells are relatively small with extreme intensity variations thus making the Rational Method inappropriate for catchment areas greater than 50 hectares.

(2) The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the catchment area, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control. For catchment areas with few impervious surfaces (little urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

(3) The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume.

This assumption is only reasonable for impervious areas, such as streets, rooftops, and parking lots. For pervious areas, the fraction of runoff does vary with rainfall intensity and the accumulated volume of rainfall. Thus, the application of the Rational Method requires the selection of a coefficient that is appropriate for the storm, soil, and land use conditions. Many guidelines and tables have been established, but seldom, if ever, have they been supported with empirical evidence.

(4) The peak rate of runoff is sufficient information for the design.

Modern drainage practice includes detention of urban storm runoff to reduce the peak rate of runoff downstream. Using only the peak rate of runoff, the Rational Method severely limits the evaluation of design alternatives available in urban and in some instances, rural drainage design.

5.7.3 The Rational Method Equation

The rational formula estimates the peak rate of runoff at any location in a catchment area as a function of the catchment area; runoff coefficient; and mean rainfall intensity, for a duration equal to the time of concentration.The rational formula is expressed as:

$$Q = 0.00278\, CIA \qquad (5\text{-}6)$$

where:

Q = maximum rate of runoff, m^3/s
C = runoff coefficient representing a ratio of runoff to rainfall (see Tables 5-5 through 5-9)
I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/hr
A = catchment area tributary to the design location, ha

5.7.4 Infrequent Storm

The coefficients given in Tables 5-5 through 5-9 are applicable for storms of 5-yr to 10-yr frequencies. Less frequent, higher intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (11). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the rational formula by a frequency factor Cf. The rational formula now becomes:

$$Q = 0.00278\, CC_f IA \qquad (5\text{-}7)$$

C_f values are listed below table 5.8. The product of C_f times C shall not exceed 1.0.

Table 5-5: Recommended Runoff Coefficient C for Pervious Surfaces by Selected Hydrologic Soil Groupings and Slope Ranges

Terrain Type	Soil Type			
	A	B	C	D
Flat, <2%	0.04-0.09	0.07-0.12	0.11-0.16	0.15-0.20
Rolling, 2-6%	0.09-0.14	0.12-0.17	0.16-0.21	0.20-0.25
Mountain, 6-15%	0.13-0.18	0.18-0.24	0.23-0.31	0.28-0.38
Escarpment, >15%	0.18-0.22	0.24-0.30	0.30-0.40	0.38-0.48

Table 5-6: Recommended Runoff Coefficient C for Various Land Uses

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Residential: Multi units, detached	0.40-0.60
Residential: Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Industrial: Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

(Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984)

Table 5-7: Coefficients for Composite Runoff Analysis

Surface	Runoff Coefficients
Street : Asphalt	0.70-0.95
Concrete	0.80-0.95
Drives and walks	0.75-0.85
Roofs	0.75-0.95

(Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984)

Table 5-8: Frequency Factors for Rational Formula Cf

Recurrence interval (years)	C_f
5	1.00
10	1.00
25	1.10
50	1.20
100	1.25

Table 5-9: Recommended Runoff Coefficient C for rural catchment

Factor		Description	Runoff Coefficient
Cs	Average slope of catchment	< 3.5% Flat	0.05
		3.5% - 10% Soft to moderate	0.1
		10% - 25% Rolling	0.15
		25% - 45% Hilly	0.2
		> 45% Mountainous	0.25
Cp	Permeability of soil	Well drained soil e.g. sand and gravel	0.05
		Fair drained soil e.g. sand and gravel with fines	0.1
		Poorly drained soil e.g. silt	0.15
		Impervious soil e.g. clay, organic silts and clay	0.25
		Water-logged black cotton soil	0.5
		Rock	0.4
Cv	Vegetation	Dense forest/thick bush	0.05
		Sparse forest/dense grass	0.1
		Grassland/scrub	0.15
		Cultivation	0.2
		Space grassland	0.25
		Barren	0.3
C = Cs + Cp + Cv			

The results of using the Rational Formula to estimate peak discharges is very sensitive to the parameters that are used. The designer must use good engineering judgment in estimating values that are used in the method.

5.7.5 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in mm/hr for duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. Rainfall-Intensity-Duration curves for use in Ethiopia are given in Figures 5-14 through 5-21 at the end of this chapter.

For drainage areas in Ethiopia, you may compute the rainfall intensity at any required time using the 24hr rainfall depth, which is known as a rainfall intensity-duration-frequency (IDF) relationship.

$$R_{Rt} = \frac{t}{24}\frac{(b+24)^n}{(b+t)^n} \quad (5\text{-}8)$$

Where:

R_{Rt} = Rainfall depth Ratio R_t: R_{24}

R_t = Rainfall depth in a given duration't'

R_{24}= 24 hr rainfall depth

b and n = coefficients b=0.3 and n=(0.78-1.09).

The general shape of a rainfall intensity-duration-frequency curve is shown in Figure 5-11. As rain-fall duration tends towards zero, the rainfall intensity tends towards infinity. Because the rainfall intensity/duration relationship is accessed by assuming that the duration is equal to the time of concentration, small areas with exceedingly short times of concentration could result in design rainfall intensities that are unrealistically high. To minimize this likelihood, use a minimum time of concentration of 10 minutes when using the coefficients presented in the Hydrology document. As the duration tends to infinity, the design rainfall tends towards zero. Usually, the area limitation of 50 hectares should result in design rainfall intensities that are not unrealistically low. However, if the estimated time of concentration is extremely long, such as may occur in extremely flat areas, it may be necessary to consider an upper threshold of time or use a different hydrologic method.

In some instances alternate methods of determining rainfall intensity may be desired, especially for coordination with other agencies. Ensure that any alternate methods are applicable.

5.7.6 Runoff Coefficient

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the designer. A typical coefficient represents the integrated effects of many drainage basin parameters. The following discussion considers the effects of soil groups, land use, and average land slope.

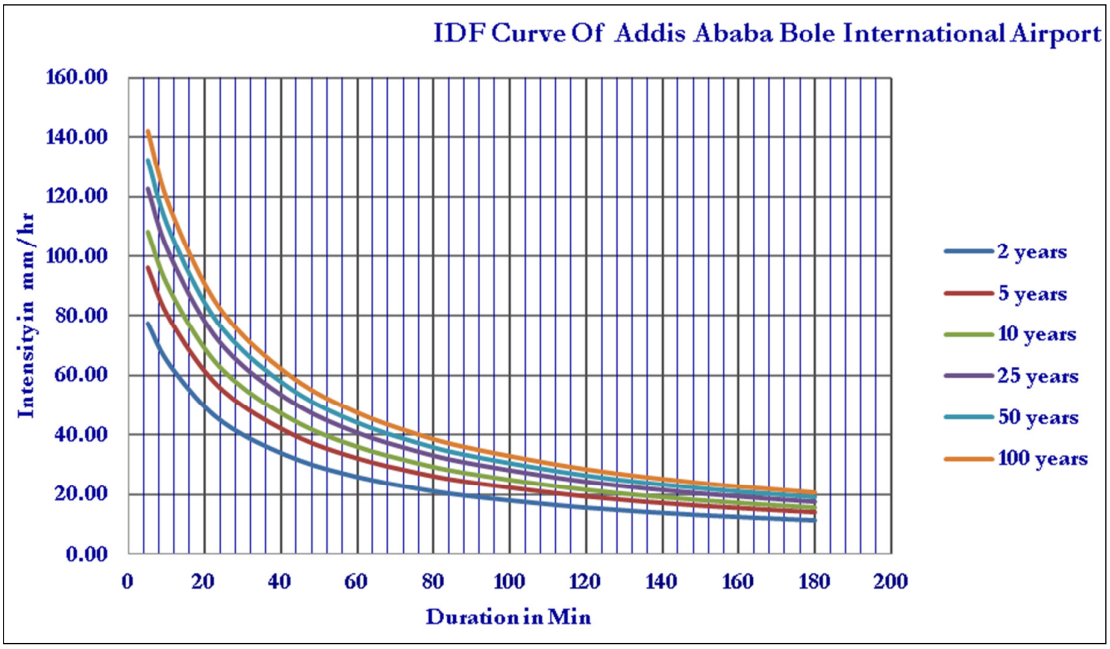


Figure 5-12: Typical Rainfall Intensity Duration Frequency Curve

Three methods for determining the runoff coefficient are presented based on soil groups and land slope (Table 5-5), land use (Table 5-6), and a composite coefficient for complex catchment areas (Table 5-7).

Table 5-9 gives the recommended runoff coefficient (C) for pervious surfaces by selected hydrologic soil groupings and slope ranges. From this table the C values for non-urban areas such as forest land, agricultural land, and open space can be determined.

5.7.7 Hydrological Soil Groups for Ethiopia

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Permeability and infiltration are the principal data required to classify soils into Hydrologic Soils Groups (HSG). Based on infiltration rates, the Soil Conservation Service (SCS) has divided soils into four hydrologic soil groups as follows:

Group A: Sand, loamy sand or sandy loam. Soils having a low runoff potential due to high infiltration rates. These soils primarily consist of deep, well-drained sands and gravels.

Group B: Silt loam, or loam. Soils having a moderately low runoff potential due to moderate infiltration rates. These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C: Sandy clay loam. Soils having a moderately high runoff potential due to slow infiltration rates. These soils primarily consist of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D: Clay loam, silty clay loam, sandy clay, silty clay or clay. Soils having a high runoff potential due to very slow infiltration rates. These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

Data from direct field measurements on soil permeability and infiltration rates for Ethiopian soils are very limited. Data is generally available only for soil types located near major irrigation projects and agricultural research stations. The hydrological soils groups presented in Table 5-10 are based on limited field measurements and from profile morphology and physical characteristics, and are subject to further review and refinement.

Table 5-10: Typical Hydrologic Soils Groups for Ethiopia

	Soil Types	Hydrologic Soil Group
Ao	Orthic Acrisols	B
Bc	Chromic Cambisols	B
Bd	Dystric Cambisols	B
Be	Eutric Cambisols	B
Bh	Humic Cambisols	C
Bk	Calcic Cambisols	B
Bv	Vertic Cambisols	B
Ck	Calcic Chernozems	B
E	Rendzinas	D
Hh	Haplic Phaeozems	C
Hl	Luvic Phaeozems	C
I	Lithosols	D
Jc	Calcaric Fluvisols	B
Je	Eutric Fluvisols	B
Lc	Chromic Luvisols	B
Lo	Orthic Luvisols	B
Lv	Vertic Luvisols	C
Nd	Dystric Nitosols	B
Ne	Eutric Nitosols	B
Od	Dystric Histosols	D
Oe	Eutric Histosols	D
Qc	Cambric Arenosols	A
Rc	Calcaric Regosols	A
Re	Eutric Regosols	A
Th	Humic Andosols	B
Tm	Mollic Andosols	B
Tv	Vitric Andosols	B
Vc	Chromic Vertisols	D
Vp	Pellic Vertisols	D
Xh	Haplic Xerosols	B
Xk	Caloic Xerosols	B
Xl	Luvic Xerosols	C
Yy	Gypsic Yermosols	B
Zg	Gleyic Solonchaks	D
Zo	Orthic Solonchaks	B

(Source: Ministry of Agriculture)

As the slope of the drainage basin increases, the selected runoff coefficient C should also increase. This is caused by the fact that as the slope of the catchment area increases, the velocity of overland and channel flow will increase allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the catchment area.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the catchment area. Composites can be made with Tables 5-5 and 5-6. At a more detailed level composites can be made with Table 5-7 and the coefficients with respect to surface type given in Table 5-9. The composite procedure can be applied to an entire catchment area or to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area.

5.7.8 Steps to Peak Plood Estimation using the Rational Method

The following procedure outlines the rational method for estimating peak discharge:

- Determine the watershed area in hectares (km2);
- Determine the time of concentration, with consideration for future characteristics of the watershed;
- Assure consistency with the assumptions and limitations for application of the Rational Method;
- Determine the rainfall IDF coefficients. Extract the Rainfall Intensity-Duration Frequency Coefficients b, and n values from the list in Hydrology according to the locality in Ethiopia and the design frequency;
- Use Equation 5- 8 to calculate the rainfall intensity in mm/hr or use developed IDF curves figure 5-16 to 5-21;
- Select or develop appropriate runoff coefficients for the watershed. Where the watershed comprises more than one characteristic, you must estimate C values for each area segment individually. You may then estimate a weighted C value; and
- Calculate the peak discharge for the watershed for the desired frequency using Equation 5-7.

Refer to worked example 5.1

5.8 SCS Unit Hydrograph

Techniques developed by the U. S. Soil Conservation Service (12) for calculating rates of runoff require the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm.

With the SCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess (14).

However, the SCS method is most suited for computing flood peaks and run of volumes for catchments smaller than 65km^2, with slopes of less than 30% and a time of concentration (Tc) less than 10 hours.

5.8.1 Catchment Area

A catchment area is determined from topographic maps; DEM data's and field surveys. For large catchment areas it might be necessary to divide the area into sub-catchment areas to account for major land use changes, obtain analysis results at different points within the catchment area, or locate storm water drainage structures and assess their effects on the flood flows. A field inspection of existing or proposed drainage systems shall be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the sub catchment areas.

5.8.2 Rainfall

The SCS method is based on a 24-hour storm event which has a Type II time distribution. The Type II storm distribution is a 'typical" time distribution which the SCS has prepared from rainfall records. It is applicable for interior rather than the coastal regions and appropriate for Ethiopia. The Type II rainfall distribution will usually give a higher runoff than a Type I distribution. Figure 5-13 shows this distribution. To use this distribution it is necessary for the user to obtain 1) the 24-hour rainfall value (from Table 5-19) for the frequency of the design storm desired, and then 2) multiply this value by 24 to obtain the total 24-hour storm volume in millimeters.

5.8.3 Rainfall-Runoff Equation

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous hydrologic and vegetative cover conditions. Data for land-treatment measures, such as contouring and terracing, from experimental catchment areas were included. The equation was developed mainly for small catchment areas for which daily rainfall and catchment area data are ordinarily available. It was developed from recorded storm data that included total amount of rainfall in a calendar day but not its distribution with respect to time. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$Q = \frac{(P-I_a)^2}{(P-I_a)+S} \qquad (5\text{-}9)$$

Where:

Q = accumulated direct runoff, mm

P = accumulated rainfall (potential maximum runoff), mm

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff, mm

S = potential maximum retention, mm.

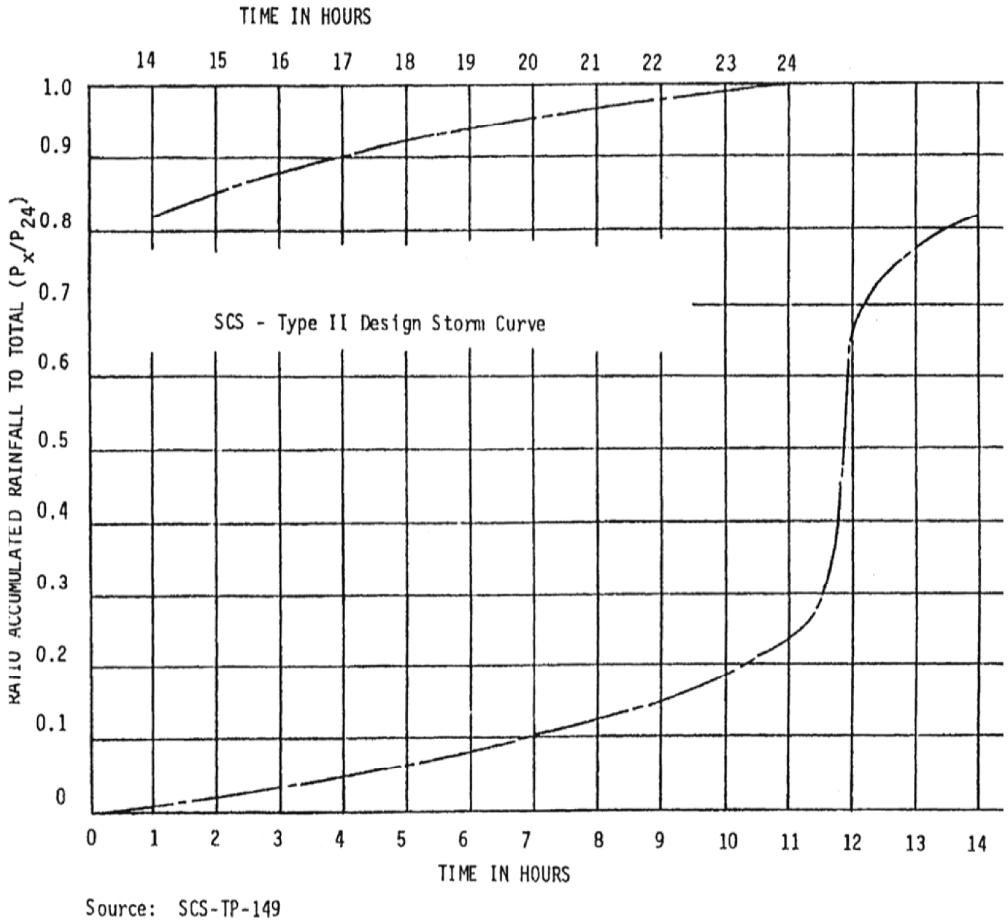


Figure 5-13: Type II Design Storm Curve

The relationship between I_a and S was developed from experimental catchment area data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the SCS runoff equation is:

$$I_a = 0.2S \tag{5-10}$$

Substituting 0.2S for I_a in equation 5.9, the SCS rainfall-runoff equation becomes:

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \tag{5-11}$$

S is related to the soil and cover conditions of the catchment area through the CN. CN has a range of 0 to 100, and S is related to CN by:

$$S = \frac{25400}{CN-254} \tag{5-12}$$

Runoff Factors

Runoff is rainfall excess or effective rainfall - the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rainwater. The principal physical catchment area characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope.

Land use is the catchment area cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc. are all part of the land use. **Land treatment** applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops. The SCS uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential.

5.8.4 Hydrologic Soil Groups

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D). These groups were previously described for the Rational Formula (see Section 5.6, Table 5-9).

Consideration shall be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes shall be made in the soil group selected. Also runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm. A five-day period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period.

5.8.5 Runoff Curve Numbers

The following pages give a series of tables related to runoff factors. The first tables (Tables 5-11 through 5-14) give curve numbers for various land uses. These tables are based on an average antecedent moisture condition, i.e., soils that are neither very wet nor very dry when the design storm begins. Curve numbers shall be selected only after a field inspection of the catchment area and a review of cover type and soil maps. Table 5-15 gives conversion factors to convert average curve numbers to wet and dry curve numbers. Table 5-16 gives the antecedent conditions for the three classifications.

Care shall be taken in the selection of curve numbers (CN's). Use a representative average curve number, CN, for the catchment area.

Selection of overly conservative CN’s will result in the estimation of excessively high runoff and consequently excessively costly drainage structures. Selection of conservatively high values for all runoff variables results in compounding the runoff estimation.

It is better to use average values and design for a longer storm frequency. Often the runoff computed using conservative CN's for a ten year storm will greatly exceed the computed runoff for average CN's for a 25 or even 50 year storm. The hydrologic designer could consider doing both in making the most appropriate selection of design discharge.

For antecedent moisture conditions (AMC) in Ethiopia, use dry for Region D1, wet for Region B1, and average AMC for all other regions. The portion of Region A2 in the vicinity of Bahir Dar should also be treated as wet. When wet AMC is used, it is unlikely that the vegetation density will also be poor to sparse.

Table 5-11: Runoff Curve Numbers- Urban Areas[1]

Cover description		Curve numbers for hydrologic soil groups			
Cover type and Hydrologic condition	**Average % impervious area[2]**	**A**	**B**	**C**	**D**
Open space (lawns, parks, cemeteries, etc.)[3]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50 % to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Desert urban areas:					
Natural desert cover		63	77	85	88
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
0.05 hectare or less	65	77	85	90	92
0.1 hectare	38	61	75	83	87
0.135 hectare	30	57	72	81	86
0.2 hectare	25	54	70	80	85
0.4 hectare	20	51	68	79	84
0.8 hectare	12	46	65	77	82
Developing urban areas Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Table 5-12: Cultivated Agricultural Land[1]

Cover description			Curve numbers for Hydrologic soil group			
Cover Type	**Treatment[2]**	**Hydrologic condtion[3]**	**A**	**B**	**C**	**D**
Fallow	**Bare soil**	-	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	**Straight row (SR)**	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C & T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
	Small grain SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
	Close-seeded SR or broadcast	Poor	66	77	85	89
		Good	58	72	81	85
	Legumes or C Rotation	Poor	64	75	83	85
		Good	55	69	78	83
	Meadow C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] Average runoff condition, and $I_a = 0.2S$.
[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
[3] Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good > 20%), and (e) degree of roughness.

Poor : Factors impair infiltration and tend to increase runoff.
Good : Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 5-13: Other Agricultural Lands[1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	**Hydrologic condition**	**A**	**B**	**C**	**D**
Pasture, grassland, or range-continuous forage for grazing[2]	Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 80
Meadow-continuous grass, protected from grazing	--	35	59	72	79
Brush-weed-grass mixture with brush the major element[3]	Poor Fair Good	48 35 30[4]	67 56 48	77 70 65	83 77 73
Woods-grass combination[5]	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Woods[6]	Poor Fair Good	45 36 30[4]	66 60 55	77 73 70	83 79 77
Farms—buildings, lanes, driveways, and surrounding lots	--	59	74	82	86

[1] Average runoff condition, and $I_a = 0.2S$
[2] Poor: < 50% ground cover or heavily grazed with no mulch
Fair: 50 to 75% ground cover and not heavily grazed
Good: > 75% ground cover and lightly or only occasionally grazed
[3] Poor: < 50% ground cover
Fair: 50 to 75% ground cover
Good: > 75% ground cover
[4] Actual curve number is less than 30; use CN = 30 for runoff computations.
[5] CNs shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pasture.
[6] Poor : Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair : Woods grazed but not burned, and some forest litter covers the soil.
Good : Woods protected from grazing, litter and brush adequately cover soil.

Table 5-14: Arid and Semi-arid Rangelands

Cover type	Hydrologic condition[2]	A[3]	B	C	D
Mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Mountain brush mixture of small trees and brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Small trees with grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Brush with grass understory	Poor	63	67	80	85
	Fair	55	51	63	70
	Good	49	35	47	55
Desert shrub brush	Poor		77	85	88
	Fair		72	81	86
	Good		68	79	84

[1] Average runoff condition, and $I_a = 0.2S$
[2] Poor : < 30 % ground cover (litter, grass, and brush overstory)
Fair : 30 to 70 % ground cover Good: > 70 % ground cover
[3] Curve numbers for Group A have been developed only for desert shrub.

Table 5-15: Conversion from Average Antecedent Moisture Conditions to Dry and Wet Conditions

CN For Average Conditions	Corresponding CN's For	
	Dry	Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Ethiopian Rainfall Region D1 (< 100 mm) Source: Ref. 15E
Ethiopian Rainfall Region A2 & B1 (mean monthly Peak > 300 mm)

Table 5-16: Rainfall Groups for Antecedent Soil Moisture Conditions during Growing and Dormant Seasons

Antecedent Condition	Conditions Description	Growing Season Five-Day Antecedent Rainfall	Dormant Season Five-Day Antecedent Rainfall
Dry	An optimum Condition of catchment area soils, where soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place	Less than 36 mm	Less than 13 mm
Average	The average case for annual floods	36 to 53 mm	13 to 28 mm
Wet	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 53 mm	Over 28 mm

(Source: Soil Conservation Service)

Time of Concentration (T_c)

The time of concentration is calculated as specified in section 5.5.

I_a/p Parameter

I_a/p is a parameter that is necessary to estimate peak discharge rates. Ia denotes the initial abstraction and p is the 24 hour rainfall depth for a selected return period. The 24 rainfall depth is taken from the frequency analysis result or from the ERA DDM rainfall region rainfall depth recommendations. For a given 24 hour rainfall distribution I_a/P represents the fraction of rainfall that must occur before runoff begins.

Peak Discharge Estimation

The following equation were used for the estimation of the peak discharge in SCS method

$$q_p = q_u A Q \quad (5\text{-}13)$$

Where

q_p = peak discharge, m^3/s
q_u = unit peak discharge, $m^3/s/km^2/mm$
A = drainage area, Km^2
Q = depth of runoff, mm

The unit peak discharge is obtained from the following equation, which requires the time of concentration (t_c) in hours and the initial abstraction rainfall (I_a/p) ration as input:

$$q_u = \propto 10^{C_o + C_1 log t_c + C_2 (log t_c)^2} \quad (5\text{-}14)$$

Where C_0, C_1 and C_2 = regression coefficients given in table 5-17 for various I_a/p ratios: $\propto$ = unit conversion factor equal to 0.000431 in SI unit.

Table 5-17: Coefficients for SCS Peak Discharge Method

Rainfall Type	Ia/P	C_0	C_1	C_2
I	0.1	2.3055	-0.5143	-0.1175
	0.2	2.23537	-0.5039	-0.0893
	0.25	2.18219	-0.4849	-0.0659
	0.3	2.10624	-0.4570	-0.0284
	0.35	2.00303	-0.4077	0.01983
	0.4	1.87733	-0.3227	0.05754
	0.45	1.76312	-0.1564	0.00453
	0.5	1.67889	-0.0693	0.00000
IA	0.1	2.03250	-0.3158	-0.1375
	0.2	1.91978	-0.2822	-0.0702
	0.25	1.83842	-0.2554	-0.0260
	0.3	1.72657	-0.1983	0.02633
	0.5	1.63417	-0.0910	0.0000
II	**0.1**	**2.55323**	**-0.6151**	**-0.1640**
	0.3	**2.46532**	**-0.6226**	**-0.1166**
	0.35	**2.41896**	**-0.6159**	**-0.0882**
	0.4	**2.36409**	**-0.5986**	**-0.0562**
	0.45	**2.29238**	**-0.5701**	**-0.0228**
	0.5	**2.20282**	**-0.5160**	**-0.0126**
III	0.1	2.47317	-0.5185	-0.1708
	0.3	2.39628	-0.512	-0.1325
	0.35	2.35477	-0.4974	-0.1199
	0.4	2.30726	-0.4654	-0.1109
	0.45	2.24876	-0.4131	-0.1151
	0.5	2.17772	-0.3680	-0.0953

5.8.6 Steps to Peak Flood Estimation using SCS Method

The following procedure outlines the SCS method for estimating peak discharge.

- Determine the watershed area in hectares (km2);
- Determine the time of concentration, with consideration for future characteristics of the watershed;
- Determine the soil type, soil group, and land use and curve number of the watershed area; determine the hydrologic region, check the AMC and convert the Cn value if required to wet or dry condition;
- Determine the 24hr rainfall depth and calculate the Ia/p ratio;
- Use Equation 5-14 to calculate the unit peak flood;

- Calculate the peak discharge for the watershed for the desired frequency using Equation 5-13.

Refer to worked example 5.2 in Appendix A

5.9 Flood Hydrograph Routing Methods

This section presents two ways of routing flood hydrographs: storage (or reservoir) routing and channel routing:

- Use storage routing to account for inflow and outflow rates and significant water storage characteristics associated with reservoirs and detention;
- Use channel routing when known hydrographic data are located somewhere other than the point of interest or the channel profile or plan is changed to alter the natural velocity or channel storage characteristics.

5.9.1 Storage Routing

As a flood hydrograph approaches and passes through a reservoir or detention facility, the characteristics of unsteady flow become significant. It is necessary to make an accounting of inflow and outflow rates and water storage characteristics by routing a flood hydrograph through the storage facility.

Reservoir or detention pond storage routing also applies when outflow depends only upon the volume of flood storage. Use storage routing techniques to do the following:

- Determine peak discharges from watersheds containing reservoir flood water detention basins and other flow retardation structures;
- Specify overtopping flood magnitudes; and
- Evaluate traffic interruption due to roadway overtopping and the associated economic losses.

5.9.2 Hydrograph Storage Routing Method Components

Several analytical and graphical methods route flood hydrographs through reservoirs or other detention facilities. All of the methods require reliable descriptions of the following three items:

- An inflow runoff hydrograph for the subject flood;
- The storage capacity versus water elevation within the facility; and
- The performance characteristics of outlet facilities associated with the operation of the facility.

By definition, when inflow and outflow from a reservoir (or any type of storage facility) are equal, a steady-state condition exists. If the inflow exceeds the outflow, the additional discharge is stored in the system. Conversely, when the out flow exceeds the inflow, water is taken from storage.

5.9.3 Channel Routing

Routing of flood hydrographs by means of channel routing procedures is useful in instances where known hydrographic data are at a point other than the point of interest. This is also true in those instances where the channel profile or plan is changed in such a way as to alter the natural velocity or channel storage characteristics. Routing analysis estimates the effect of a channel reach on an inflow hydrograph.

5.10 Statistical Analysis of Stream Gauge Data

5.10.1 Stream Gauge Data

Some sites exist where a series of stream flow observations have been made and stream gauge data obtained. It is necessary to use this data, with certain qualifications, to develop a peak discharge versus frequency relation for peak runoff from the watershed.

Peak Stream Flow Frequency Relation. Stream gauging stations recording annual peak discharges have been established at 127 stream flow-gauging stations within the 12 River Basins of Ethiopia. If the gauging record covers a sufficient period, it is possible to develop a peak stream-flow frequency relation by statistical analysis of the series of recorded annual maximum flows. It is possible to use such relationships productively in several different ways:

- If the road drainage site is near the gauging station on the same stream and watershed, the discharge can be used directly for a specific frequency (T-year discharge) from the peak stream flow frequency relationship;
- If the drainage structure site is within the same basin but not proximate to the gauging station, transposition of gauge analysis results is possible;
- If the structure site is not within a gauged basin, it is possible to develop the peak-flow flood-frequency from data from a group of several gauging stations based on either a hydrologic region (e.g., regional regression equations), or similar hydrologic characteristics.

Curve Development Stipulations. It is possible to develop a peak stream flow versus frequency curve for a site by statistical means provided the following stipulations are met:

- **Sufficient peak discharge sample** - A sufficient statistical sample of annual peak discharges must be available. This usually means a minimum of eight years of data. Some statisticians prefer a sample of 20 or more years. However, 20 years usually is not realistic for available observation periods, and fewer observations are often used as a basis for an analysis.
- **No significant change in channel/basin** - No significant changes in the channel or basin should have taken place during the period of recording. If significant changes did occur, the resulting peak-stream flow frequency relation could be flawed. The urbanization character of the watershed must not be likely to change enough to affect significantly the characteristics of peak flows within the total time of observed annual peaks and anticipated service life of the highway drainage structure. No means of accommodating future changed characteristics of a watershed within the statistical methods are used in highway hydrology.

- **No physical flow regulations existing** - A series of observed data from a watershed within which there have been, are, or will be physical flow regulations is not a sound basis for a hydrologic analysis.
- **Data representative of watershed** - The measured data must be representative of the subject watershed, either directly or by inference.

Stream Gauge Record Sources - Generally, the drainage designer will need to acquire a record of the annual peak flows for the appropriate gauging station. Some of the gauging stations in Ethiopia have only staff gauges without authomatic data recording device, and thus data is only recorded one or twice a day. This may lead to missing of the flood peaks.

The Ministry of Water & Energy (MoW&E) provides a source of stream gauge records.

Applicability and Limitations. For highway drainage purposes, a statistical analysis of stream gauge data is typically applied only in those instances where there is adequate data from stream gauging stations.

Table 5-18: Recommended Minimum Stream Gauge Record Lengths

Design Frequency (Years)	Minimum Record Length (Years)
10	8
25	10
50	20
100	25

If adequate data are not available, the design peak discharge should be based on analyses of data from several stream flow-gauging stations. In some cases, a site requiring a design peak discharge is on the same stream and near an active or discontinued stream flow-gauging station with an adequate length of record (see the "Recommended Minimum Stream Gauge Record Lengths" Table 5-18).

Having determined that a suitable stream gauge record exists, it is necessary to determine if any structures or urbanization may be affecting the peak discharges at the design site.

Consider the following guidelines:

- **Period of record similar to design site** - The period of record for the gauging station's annual peak discharges should represent the same or similar basin conditions as that of the design site. Therefore, exclude from the analysis any gauged peak discharges not representing the basin conditions for the design site.
- **Factors affecting peak discharge** - The most typical factors affecting peak discharges are regulation by urbanization and reservoirs. Densities of impervious cover less than 10 percent of the watershed area generally do not affect peak discharges. The existence in the watershed of a major reservoir or many smaller reservoirs or flood control structures can greatly affect the runoff characteristics.
- **Length of record** - The length of record should be adjusted to include only those records that have been collected subsequent to the impoundment of water by reservoirs and subsequent to any major urbanization. If the resulting records then become too short, do not use the procedures in this section.

5.10.2 Infilling Gaps in Annual Maximum Flood Series

Missing observations in stream flow records at gauging locations are very common and one of the elementary steps in any hydrological data analysis is to make decisions about dealing with these missing data points. Missing records in the annual maximum flood series were in-filled where the extra data points can be estimated with sufficient accuracy to contribute additional information rather than 'noise'. One of the following methods can be applied, as documented in Rahman (1997) and Haddad, Rahman and Weinmann (2008b).

Method 1:

Comparison of the monthly instantaneous maximum (IM) data with monthly maximum mean (MMD) data at the same station for years with data gaps should be identified. If a missing month of instantaneous maximum flow corresponds to a month of very low maximum mean daily flow, then that should be taken to indicate that the annual maximum does not occur during that missing month.

Method 2:

Method 2 involves a linear regression of the annual maximum mean flow series against the annual instantaneous maximum series of the same station. Regression equations developed should be used for filling gaps in the IM record, but not to extend for the overall period of record of instantaneous flow data. For in-filling the gaps, Method 1 is preferrable over Method 2, as it is more directly based on observed data for the missing month and involves fewer assumptions.

5.11 Regional Regression Methods

Regional regression equations are the most commonly accepted method for establishing peak flows at larger ungauged sites (or sites with insufficient data for a statistical derivation of the flood versus frequency relation). Regression equations have been developed to relate peak flow at a specified return period to the physiography, hydrology, and meteorology of the watershed in 1989 by Dr Admassu Gebeyehu and have been retained in the revised manual. However, these regression equations should be revised based on further studies.

5.11.1 Regression Methods and Equations

Regression analyses use stream gauge data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel, and meteorological characteristics; they are often termed hydrologically homogeneous geographic areas.

It is difficult to choose the proper set of regression equations when the design site lies on or near the hydrologic boundaries of relevant studies.

Another problem occurs when the watershed is partly or totally within an area subject to mixed population floods.

Care should be exercised using regression equations in these instances:

- Conduct a field visit to assess the watershed characteristics for comparison with other watersheds;
- Collect all available historical flood data; and
- Use the gathered data to interpret any discharge values.

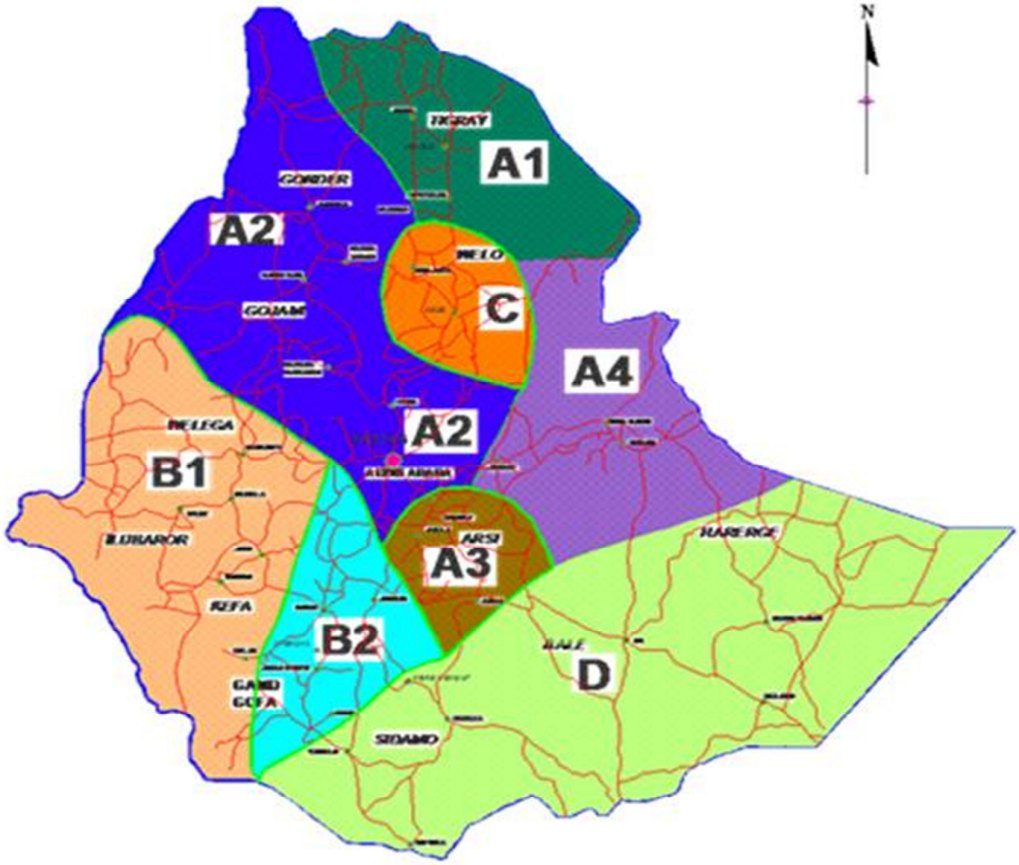


Figure 5-14: Rainfall Regions

Note: Rainfall data used in the preparation of this figure have been collected from meteorological service agency meteorology stations (see Table 5-4). In the course of the preparation of this manual, they have been subjected to statistical techniques. The results indicate that the country can be divided into the above hydrological regions displaying similar rainfall patterns. The information is reviewed with the current available data up to 2010, and future data may indicate the need for a further refinement in both values and regional boundaries.

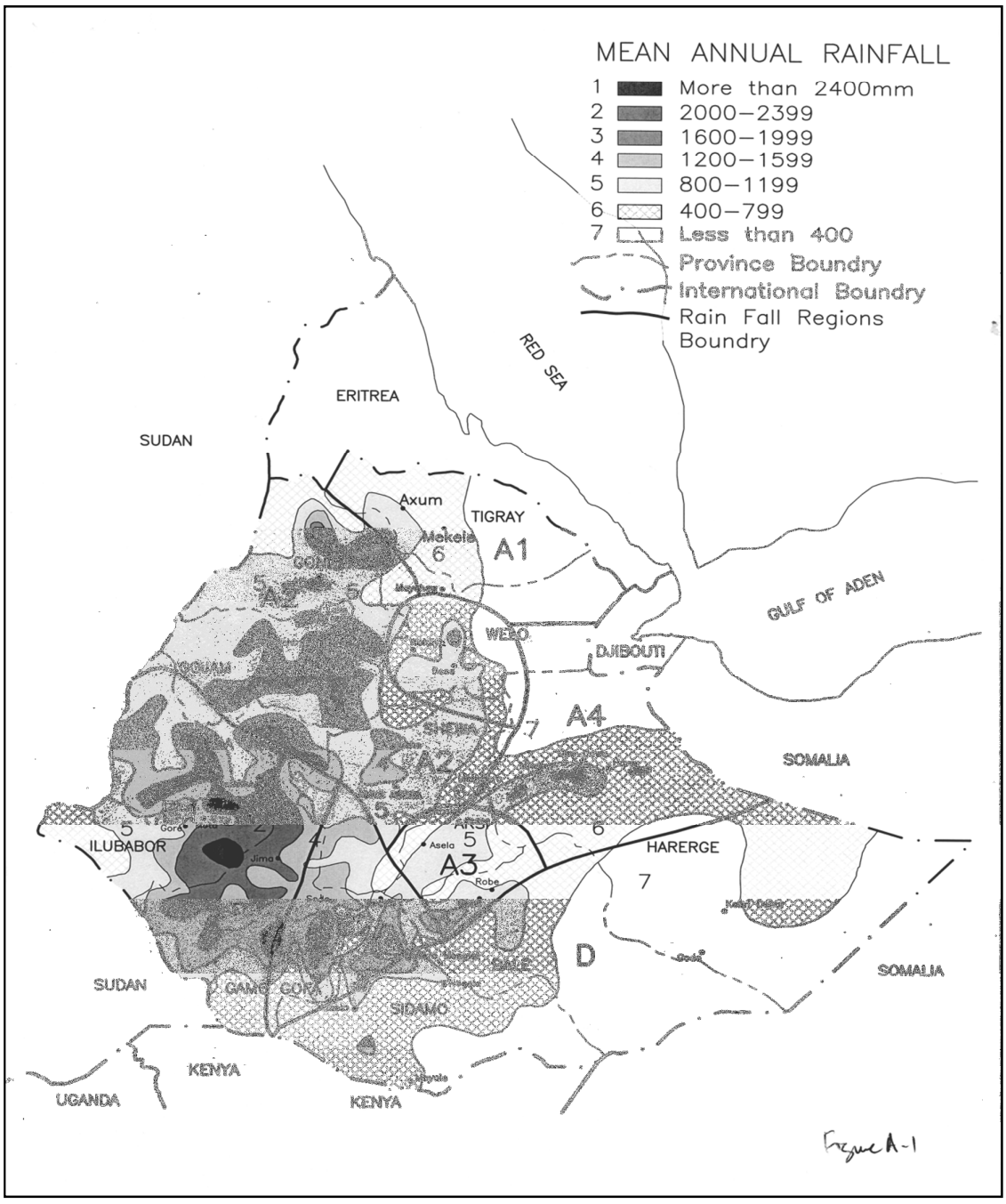


Figure 5-15: Mean Annual Rainfall for Ethiopia

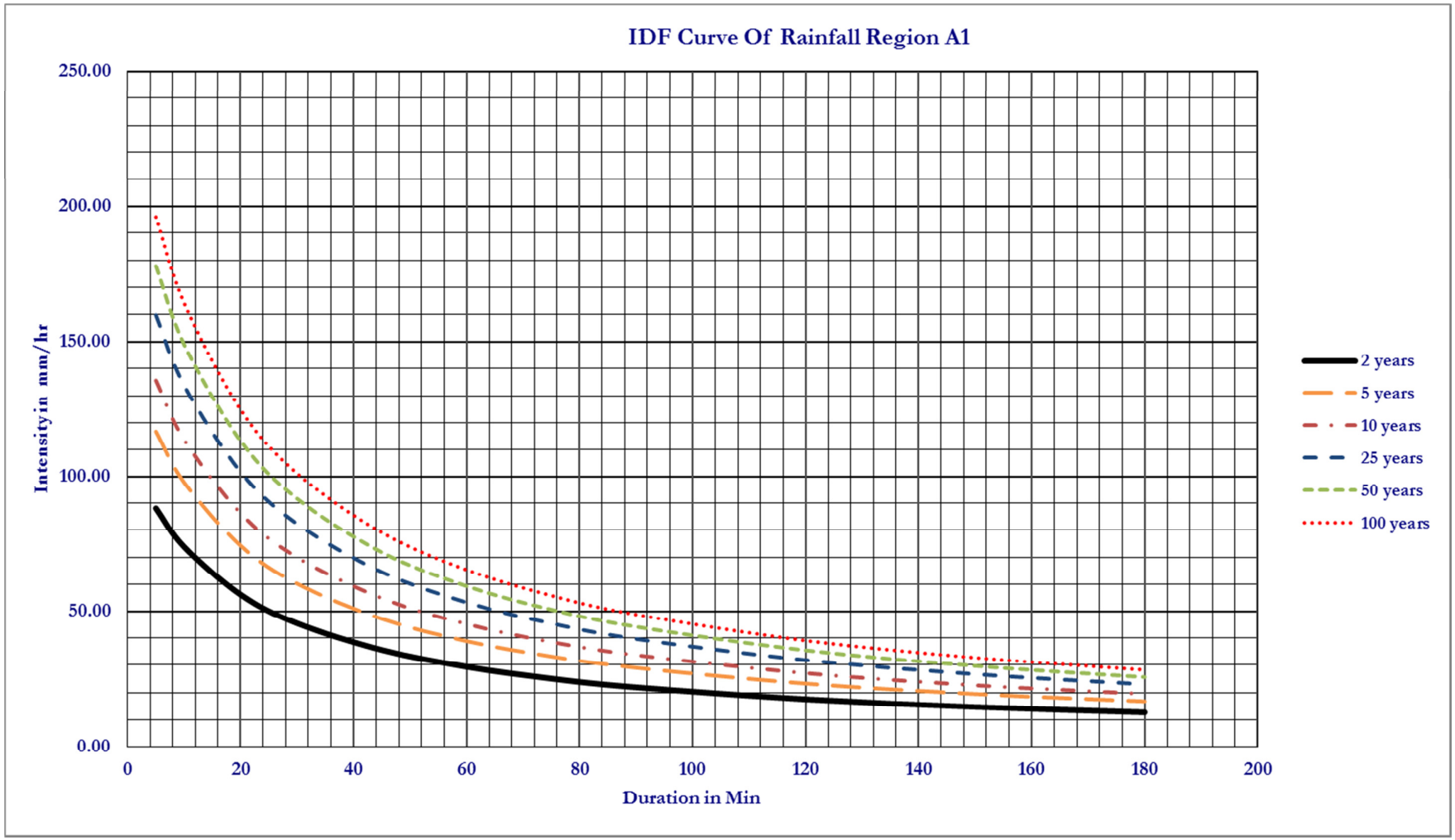


Figure 5-16: IDF Curve of Rainfall Region A1

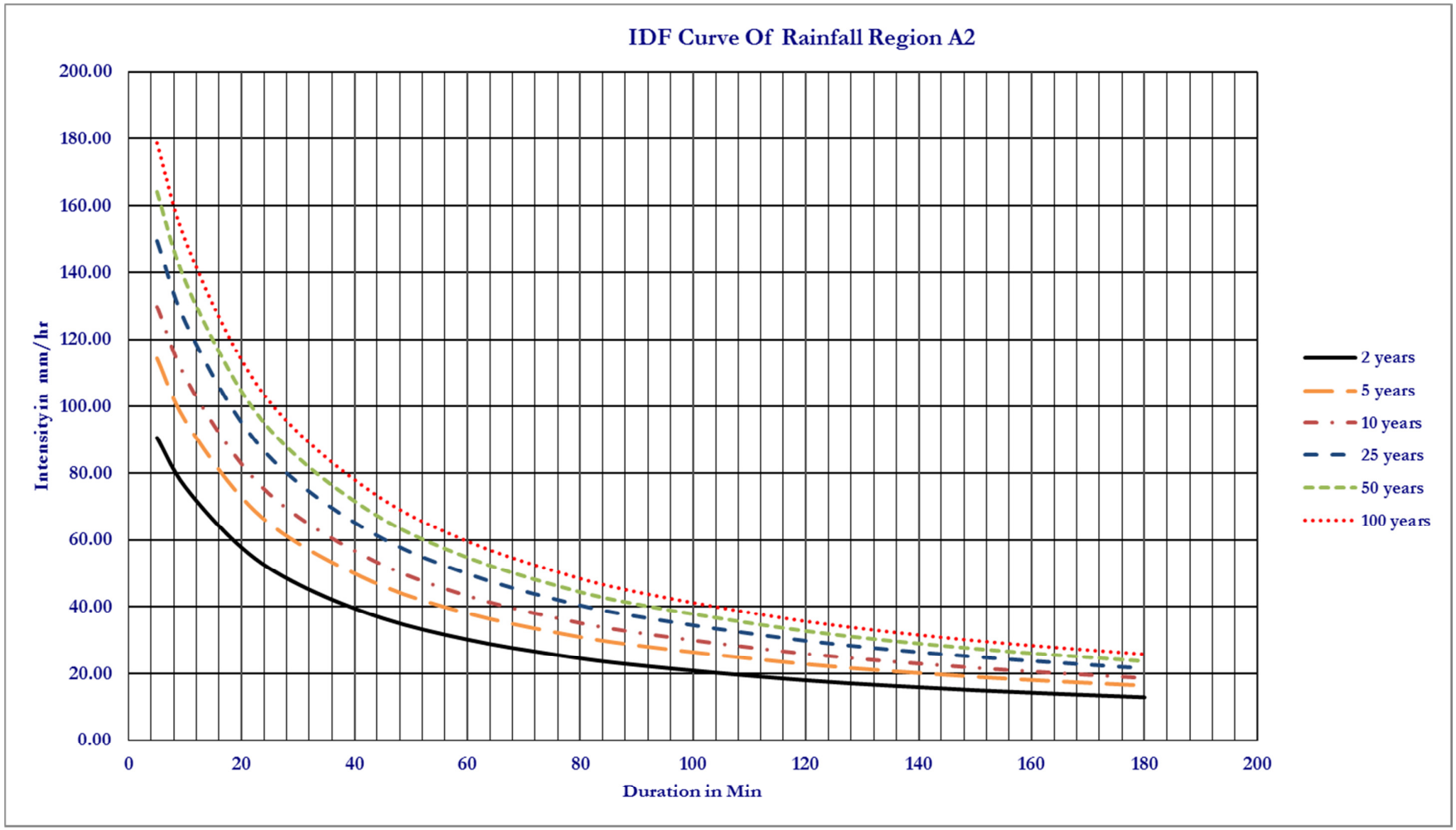


Figure 5-17: IDF Curve of Rainfall Region A2

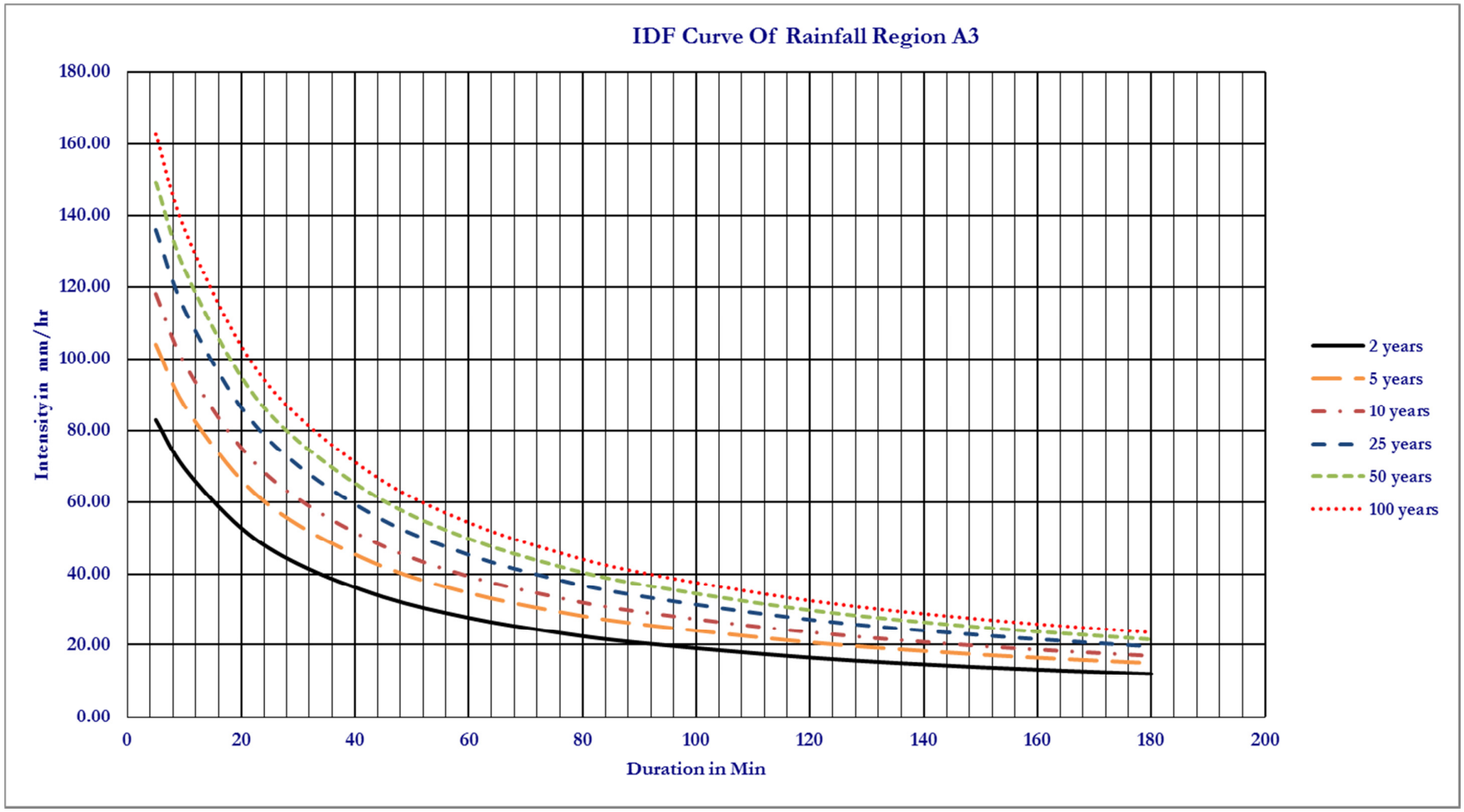


Figure 5-18: IDF Curve of Rainfall Region A3

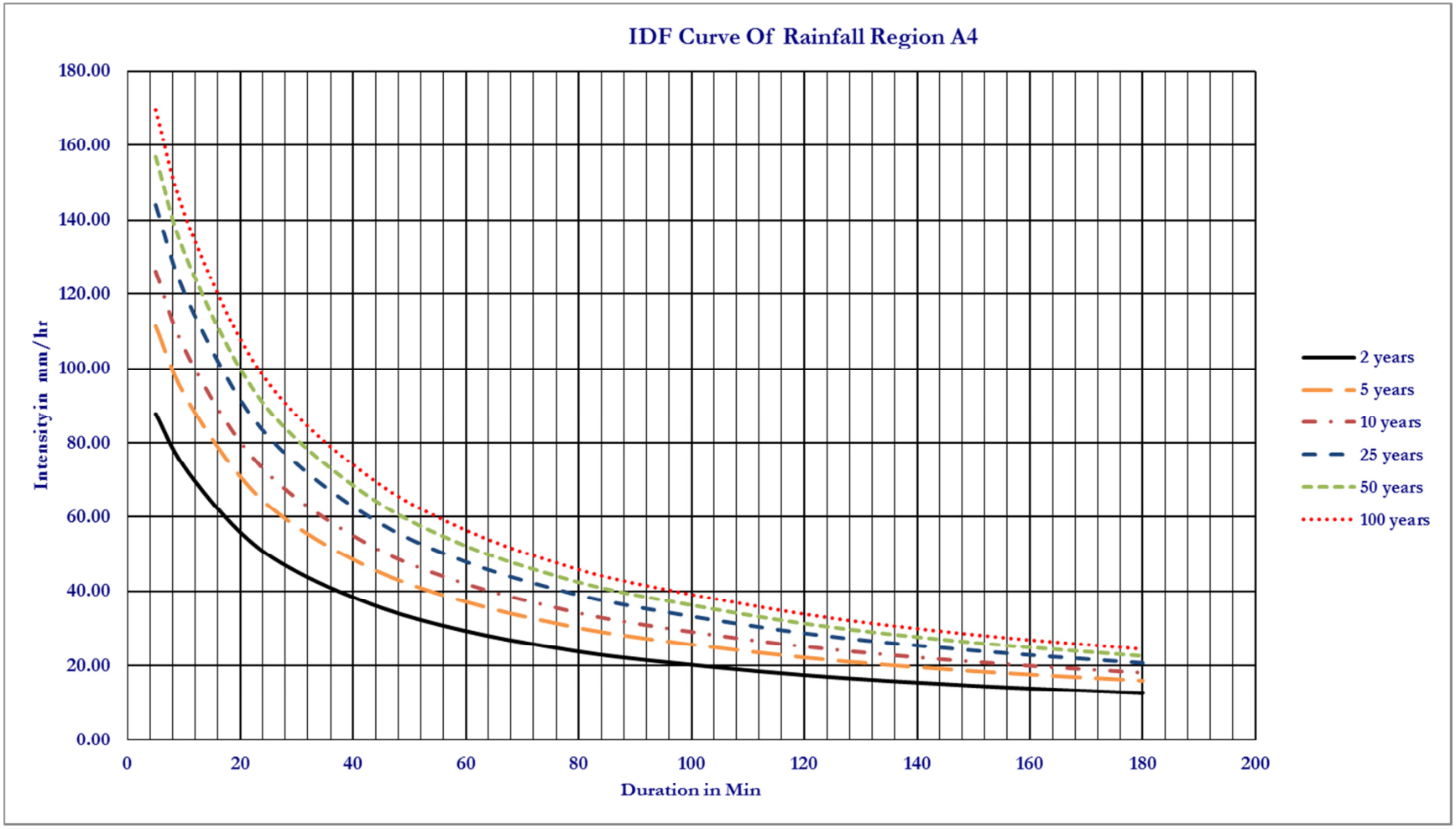


Figure 5-19: IDF Curve of Rainfall Region A4

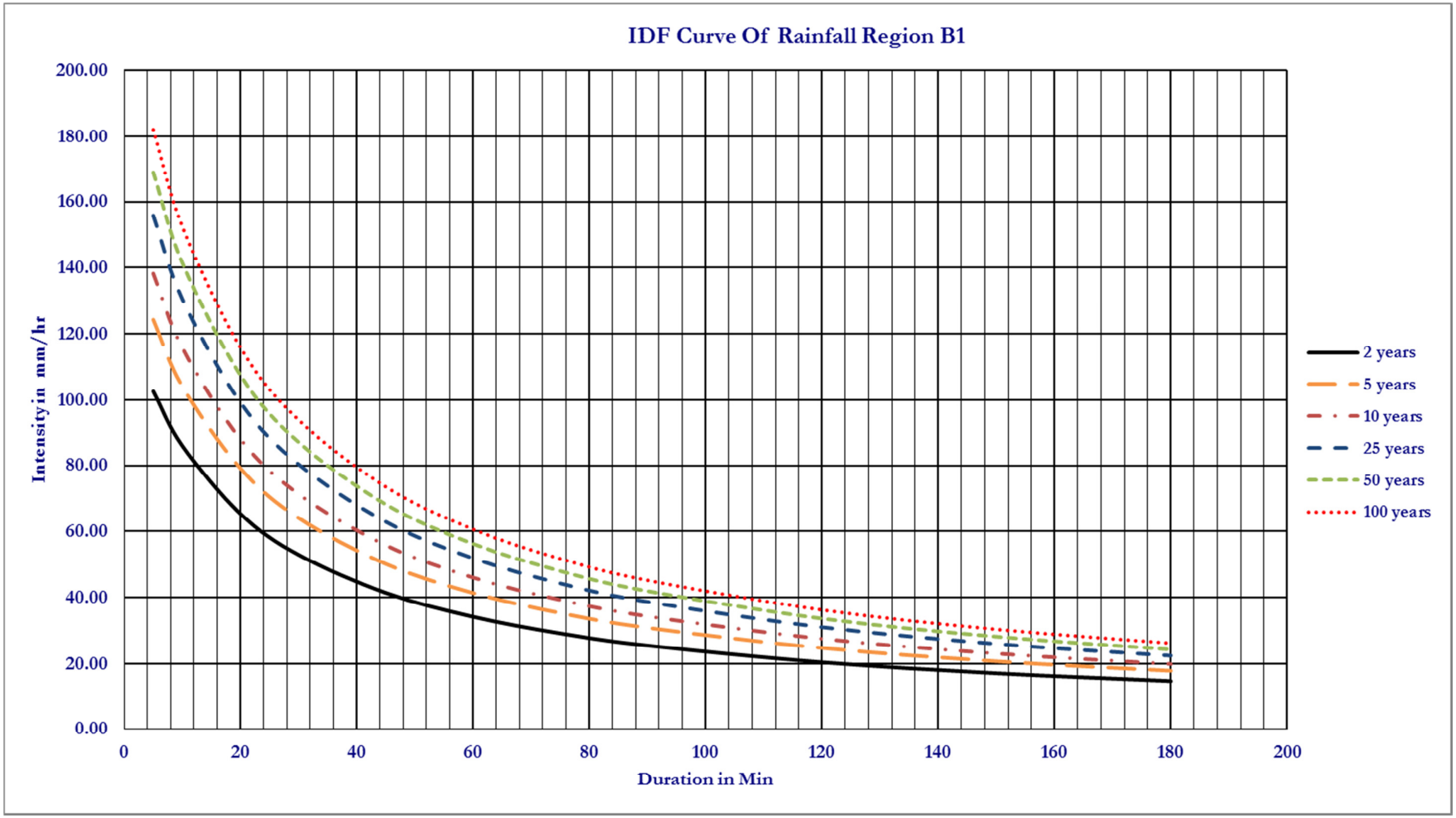


Figure 5-20: IDF Curve of Rainfall Region B1

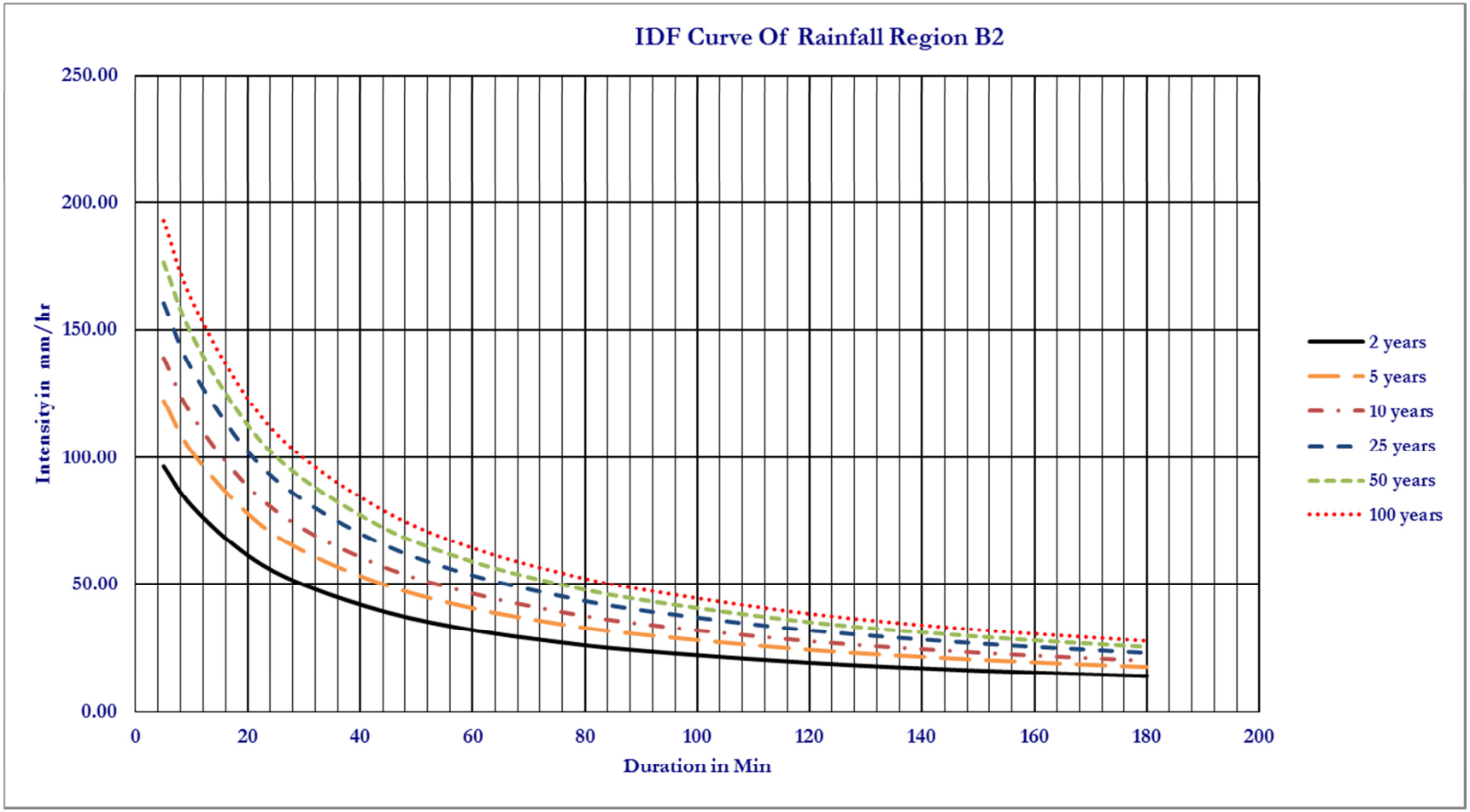


Figure 5-21: IDF Curve of Rainfall Region B2

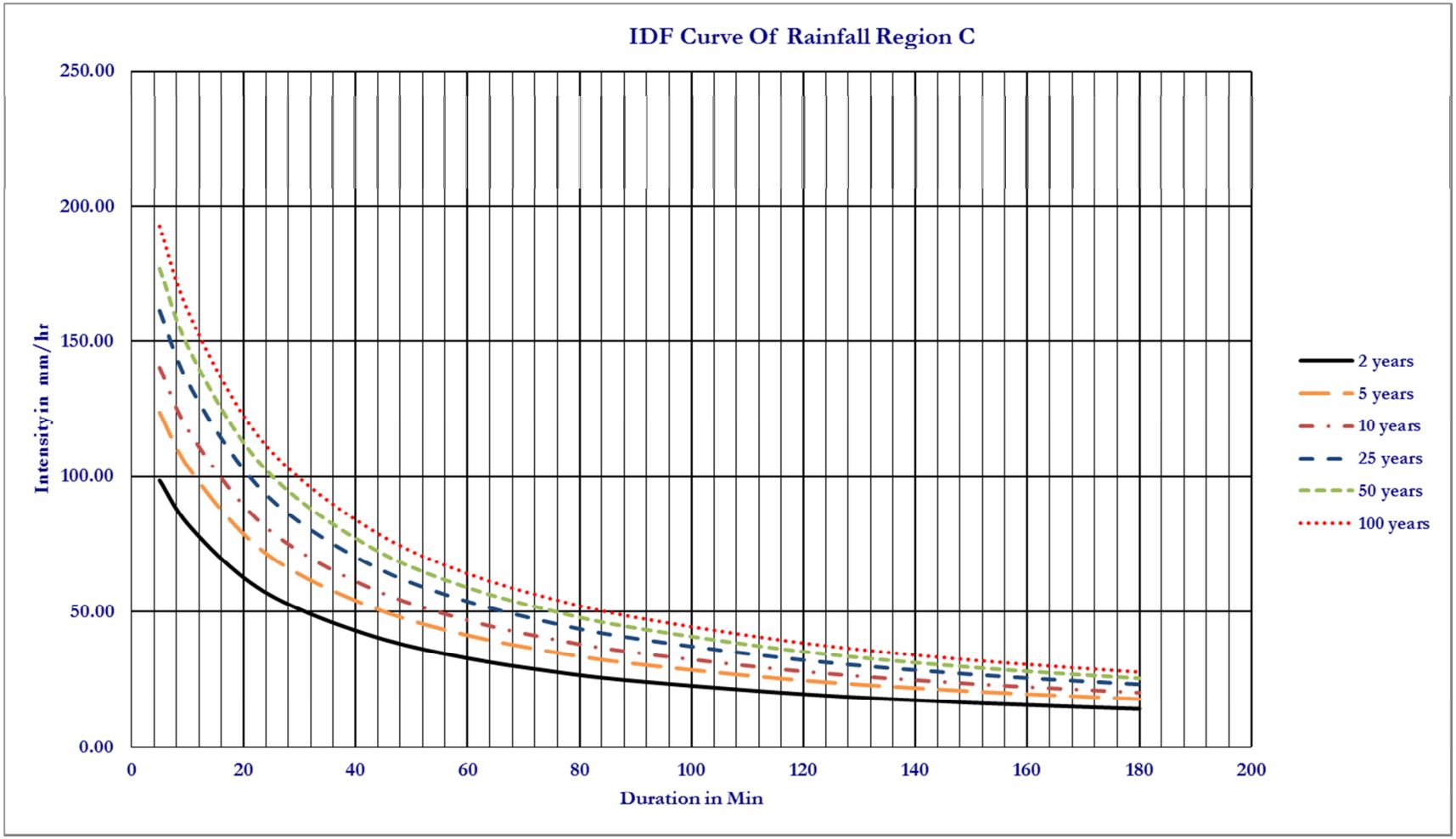


Figure 5-22: IDF Curve of Rainfall Region C

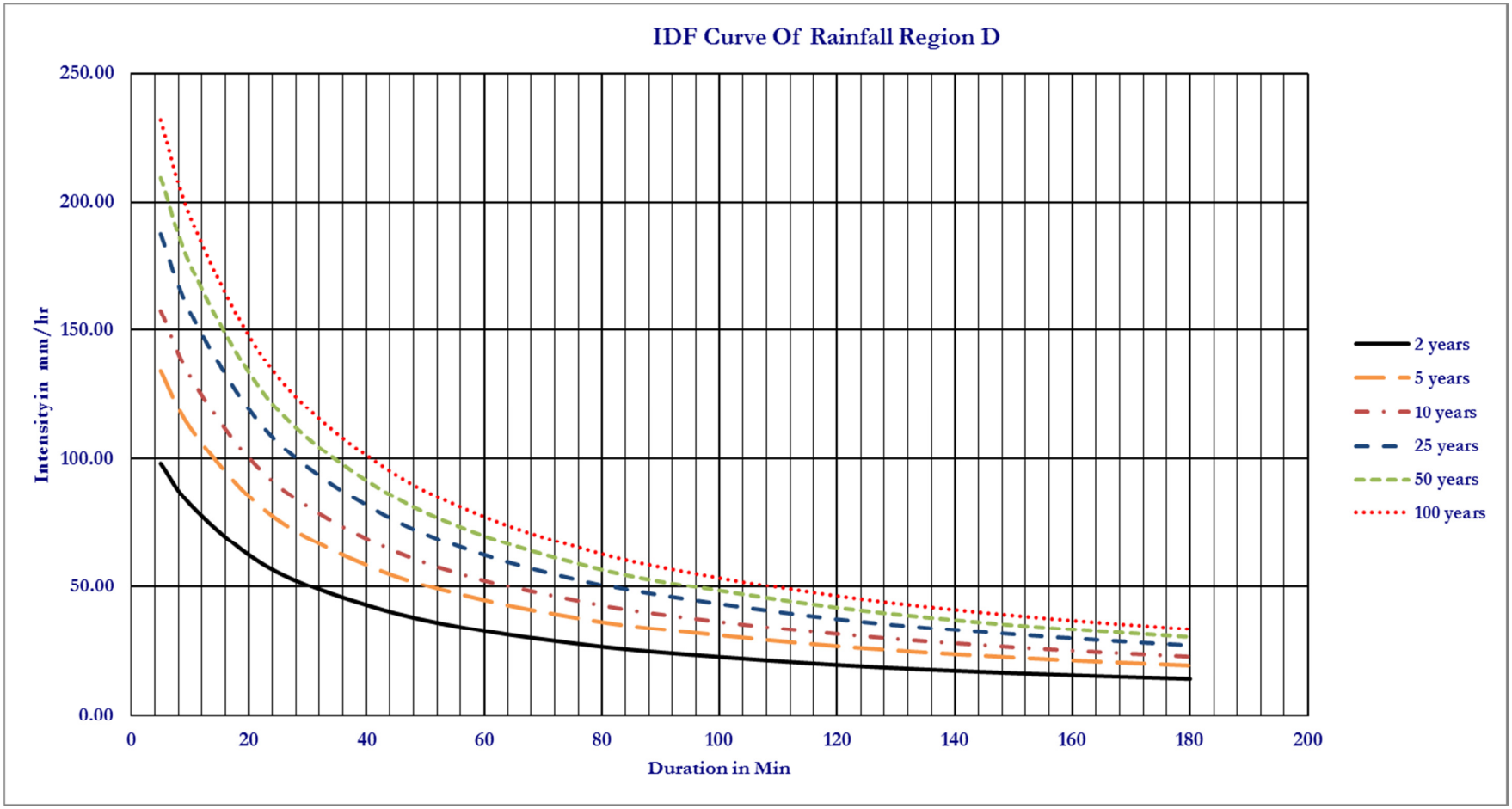


Figure 5-23: IDF Curve of Rainfall Region D

Table 5-19: 24hr Rainfall Depth Vs Frequency

24 hr Rainfall Depth (mm) vs Frequency (yr)								
Return Period Years	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**500**
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06	117.48	130.00
RR-A2	51.92	65.52	74.45	85.70	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.50	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146.00	163.44

Note: RR- Rainfall Region

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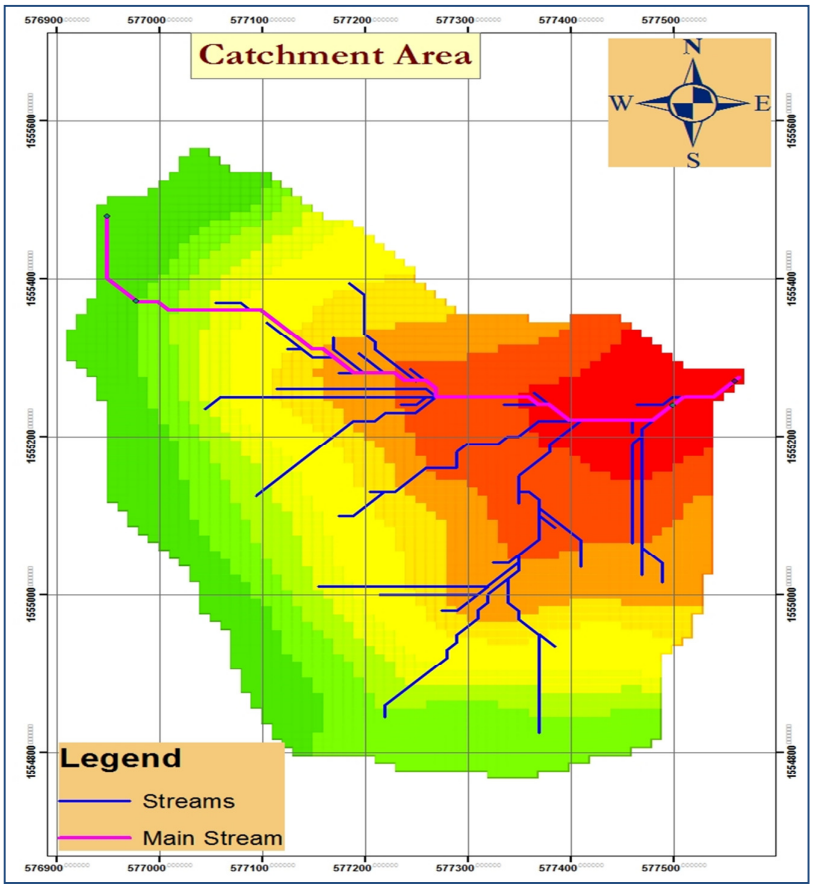
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APPENDIX 5A - EXAMPLE PROBLEMS

Example 1: Rural Runoff Example using Rational Method

The example describes the process to determine the peak runoff from a simple rural catchment area around Wukro.

Step 1: Determine Catchment Area

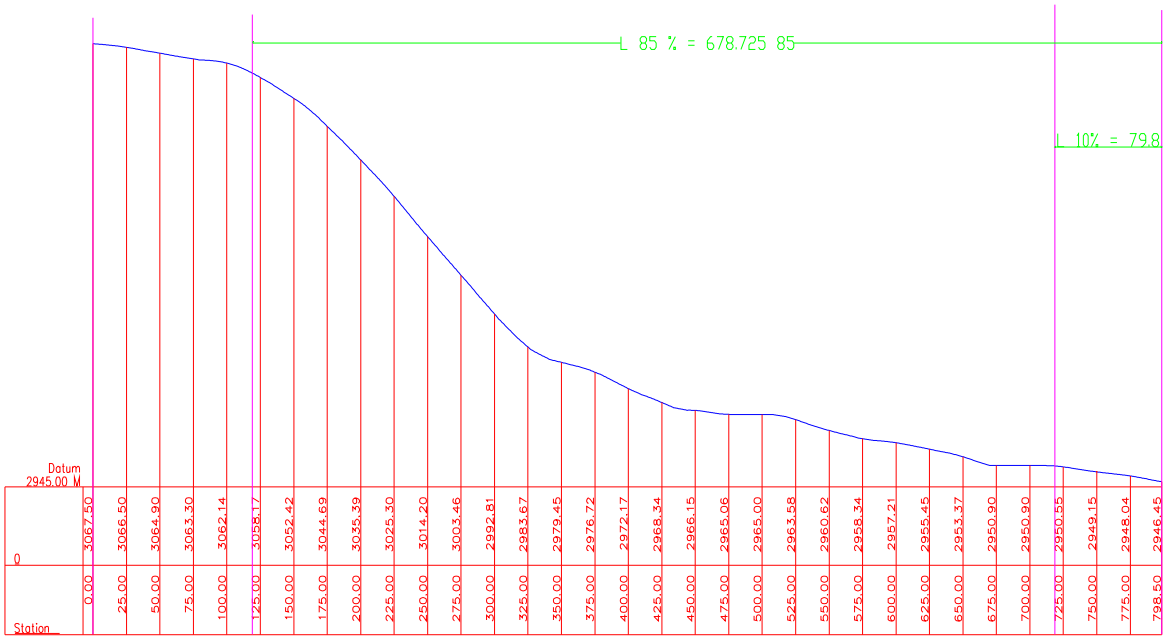


Step 2: Determine longest flow path and elevations.

Name	Catchment Area Km^2	Stream length (m)	Elevation @ U/S (m)	Elevation @ D/S (m)	Elevation @ 10% (m)	Elevation @ 85% (m)
C1	0.3252	798.20	3067.50	2946.90	2950.80	3058.70

Step 3: Determine Catchment Property

Land Cover	Soil Type	Hydrologic Soil Group	Rainfall Region	AMC
Cultivated	Lithosols	D	Rainfall Region A1	Normal



Step 4: Calculate Time of concentration

The time of concentration in rural area divide in to two sections as specified in sec 5.5.

1) Time of concentration for overland flow

$$T_c = 0.604 \left(\frac{CvL}{S^{0.5}}\right)^{0.467}$$

Cv = roughness coefficient of land use Table 5…. = **0.2**

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) = **0.12km**

H = height of most remote point above outlet of catchment (m)

S =Slope of the catchment or $S = \frac{H}{1000L}$ (m/m) = $\frac{(3067.5-3059.172)}{(1000*0.12)}$ = **0.0694**

Tc = time of concentration (hours) = $0.604 \left(\frac{0.2*0.12}{(0.0694)^{0.5}}\right)^{0.467}$ = **0.1973** hr

2) Time of concentration for defined water course

$$T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$

L = hydraulic length of catchments, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) = **0.7982km**

S_{av} = average slope (m/m)

$$S_{av} = \frac{H_{0.85L} - H_{0.10L}}{(1000)(0.75L)}$$

$H_{0.10L}$ = elevation height at 10% of the length of the watercourse (m) = **2950.80**

$H_{0.85L}$ = elevation height at 85% of the length of the watercourse (m) = **3058.70**

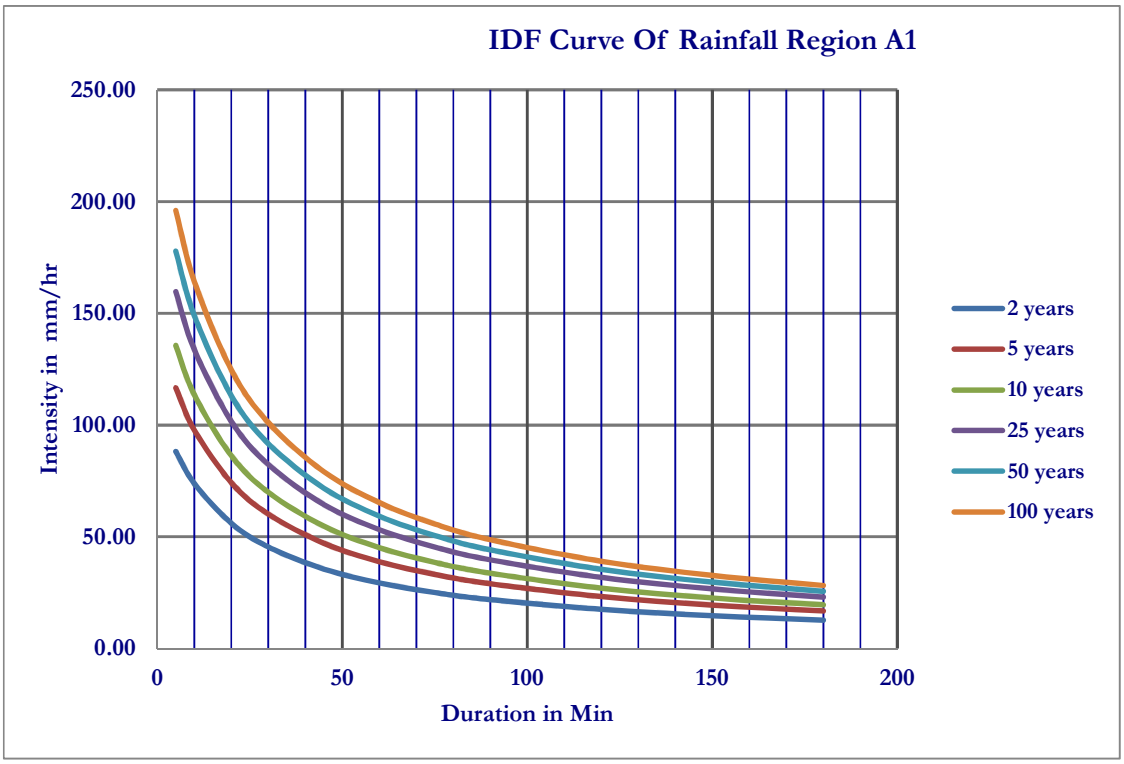
L = length of watercourse (km) = **0. 7982**

$$\boldsymbol{S_{av}} = \frac{3058.7 - 2950.8}{1000 * 0.75 * 0.7982} = \mathbf{0.21204}$$

T_c = time of concentration (hours) = $\left(\frac{0.87*0.7982^2}{1000*0.21204}\right)^{0.385} = \mathbf{0.1014}$

T_{ct}= T_{C1}+T_{C2} = 0.1973+0.1014= **0.2986 hr = 17.92min**

Step 5: Determine rainfall intensity



The catchment area were found in rainfall region A1, use the IDF curve of rainfall region A1 (or use project specific IDF curve) and find the rainfall intensity for different return periods.

I_2=59.5mm/hr ; I_5=78.6mm/hr; I_{10}=90mm/hr; I_{25}=107.3mm/hr; I_{50}=119.4mm/hr; I_{100}=130mm/hr

Step 6: Determine runoff coefficients

The runoff coefficient were depends on the catchment slope, permeability of the soil and vegetation cover.

$$C = C_s + C_p + C_v = 0.1 + 0.15 + 0.2 = 0.45$$

Cs= catchment slope coefficient = **0.10**

C_p= Soil permeability coefficient = **0.15**

C_V= Land cover coefficient = **0.20**

Step 7: Calculate the Peak flood

$$Q = 0.278\, CC_f IA$$

Q = maximum rate of runoff, m^3/s

C = runoff coefficient

I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/hr

A = catchment area tributary to the design location, Km^2

Return period	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
I =mm/hr	59.5	78.6	90	107.3	119.4	130
Cf	1.0	1.0	1.0	1.1	1.2	1.3
Q =m^3/s	8.71	11.50	13.17	17.27	20.97	23.78

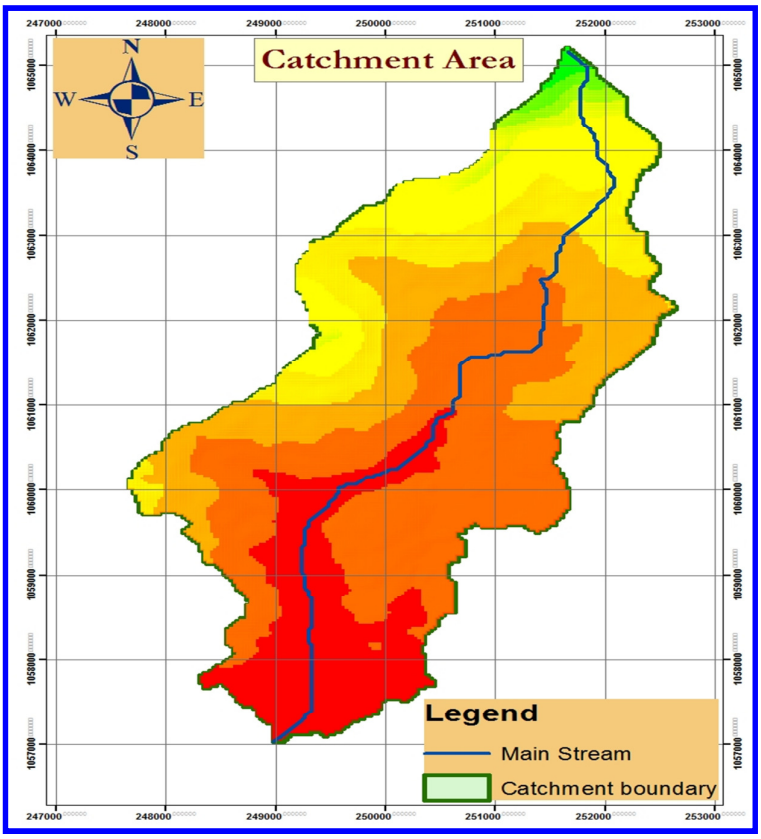
Example 2: Rural Runoff Example using SCS Method



Photo showing crossing location

This example describes the process to determine the peak runoff from a simple rural catchment area around Nekempte.

Step 1: Determine Catchment Area

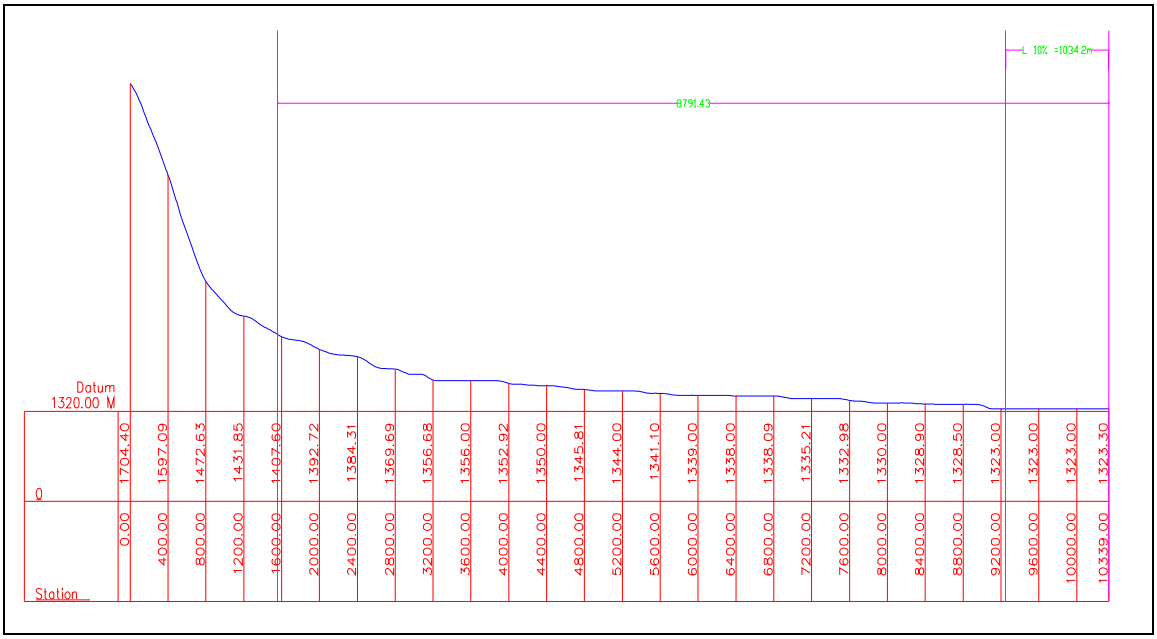


Step 2: Determine longest flow path and elevation

Name	Catchment Area Km^2	Stream length (m)	Elevation @ U/S (m)	Elevation @ D/S (m)	Elevation @ 10% (m)	Elevation @ 85% (m)
C2	19.077	10342.87	1704.4	1320.3	1323	1411.099

Step 3: Determine Catchment area Property

Land Cover	Soil Type	Hydrologic Soil Group Soil Type	Rainfall Region	AMC	Normal Curve No	Wet Region Curve no.
Grass land	Dystric Cambisols	B	Rainfall Region B1	Wet	69	85
Grass land	Orthic Acrisols	B	Rainfall Region B1	Wet	69	85



Step 4: Calculate Time of concentration

The time of concentration in rural area divide in to two sections as specified in sec 5.5.

- **time of concentration for overland flow**

$$T_c = 0.604\left(\frac{C_v L}{S^{0.5}}\right)^{0.467}$$

C_v = roughness coefficient of land use Table 5…. = **0.15**

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) = **0.20km**

H = height of most remote point above outlet of catchment (m)

S = Slope of the catchment where $S = \frac{H}{1000L} = \frac{1704.4-1653.769}{1000*0.2} = \mathbf{0.25316}$ (m/m)

T_c = time of concentration (hours) = $T_c = 0.604\left(\frac{0.15*0.2}{(0.25316)^{0.5}}\right)^{0.467} = \mathbf{0.16187}\ hr$

- **Time of concentration for defined water course**

$$T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$

L = hydraulic length of catchments, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) = 1**0.339km**

S_{av} = average slope (m/m)

$$S_{av} = \frac{H_{0.85L} - H_{0.10L}}{(1000)(0.75L)}$$

$H_{0.10L}$ = elevation height at 10% of the length of the watercourse (m) = **1323.00**

$H_{0.805L}$ = elevation height at 85% of the length of the watercourse (m) = **1411.099**

L = length of watercourse (km) = **10.399**

$$S_{av} = \frac{1411.099 - 1323.00}{1000 * 0.75 * 10.399} = \mathbf{0.01130}$$

Tc = time of concentration (hours) = $\left(\frac{0.87*10.399^2}{1000*0.0.01130}\right)^{0.385} = 2.2613$

$T_{ct} = T_{C1}+T_{C2}$ = 0.16187+2.2613 = **2.42319 hr = 145.392min**

Step 5: Calculate peak flood

The peak flood estimation was done by using the SCS unit hydrograph flood estimation formula.

1) Rainfall Runoff equation

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous hydrologic and vegetative cover conditions.

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S}$$

Where: Q = accumulated direct runoff, mm

P = accumulated rainfall (potential maximum runoff), mm

I_a= initial abstraction including surface storage, interception, and infiltration prior to runoff, mm I_a=0.2*S

$$S = \frac{25400}{CN} - 254$$

S = potential maximum retention, mm

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff, mm I_a = 0.2*S

CN= Curve Number , in the above catchment characteristics table the area were located in WET region of the country the normal curve number changed to wet region as per the manual. = **88**

The catchment area located in rainfall region B1 use the 24hr rainfall depth of Nekempte meteorological station.

Return period	2 Years	5Years	10 Years	25 Years	50 Years	100 Years
P	68.78	83.45	92.34	102.72	109.86	116.49
Q	34.193	46.502	54.224	63.43	69.86	75.89

2) Peak Discharge estimation

The following equation were used for the estimation of the peak discharge in SCS method

$qp = qu * A * Q$

Where qp = peak discharge, m^3/s

qu = unit peak discharge, $m^3/s/km^2/mm$

$qu = \propto * 10^{Co+C1logtc+c2(logtc)^2}$

Where Co, C1 and C2 = regression coefficients given in table 5… for various Ia/p ratios: $\propto$ = unit conversion factor equal to 0.000431 in SI unit.

A = drainage area, Km^2

Q = depth of runoff, mm

Return period	2 Years	5Years	10 Years	25 Years	50 Years	100 Years
Ia/p	0.130	0.107	0.097	0.087	0.0816	0.0769
Qu	0.0821	0.0837	0.0845	0.0853	0.0861	0.0865
qp (m^3/s)	**53.568**	**74.271**	**87.444**	**103.213**	**114.701**	**125.163**

Example 3: Flood Routing Method

Large Catchments with Multiple Watersheds

Subdivision of the area of a large basin may be necessary because of the size and complexity of the physical system. A basin with streams and/or a diverse topography may be divided into smaller components. In this section, the model input preparation for a subdivided basin is presented. This watershed may be divided into sub basins. The watershed parameter for the sub basins are given in Table 4. These parameters have been described in step 1. For this exercise one precipitation distribution is assumed for both sub basins. The base flow will be defined here as the flow which results from releases of water from subsurface storage.

The input parameters starting base flow discharge (ST), threshold discharge (QR), and rate of recession index (RT) are used to model the base flow in the HEC-HMS model. ST represents the initial flow in the river, RT is equal to the ratio of the recession limb flow to the recession limb flow occurring one hour earlier and QR indicates the flow at which an exponential recession begins on the receding limb of the computed hydrograph.

The base flow record is used to input the base flow into the HEC-HMS model. The parameters selected as an input are 5 m^3/s and $8m^3/s$ for the values of ST for the north and south sub basins, respectively, 0.05 for QR (i.e. 0.05 times the peak runoff, recommended in the HEC-1 manual) and 0.9 for RT (a typical recession ratio) for both sub basins.

A computer program, "Hydrologic Modelling Systems" (HEC-HMS), developed by the U.S. Army Corps of Engineers is used in this example. This program is used in engineering practice to determine the drainage characteristics of both rural and urban watersheds.

Flood movement through river reaches and reservoirs is simulated by flood routing. Most of the flood-routing methods available in HEC-HMS are based on the continuity equation and some relationship between flow and storage (or stage).

HEC-HMS Input Parameters

To compute an SCS synthetic hydrograph for this exercise, the following parameters will be determined.

Watershed Area

A topographic map is necessary to delineate a watershed for the study area and calculate its enclosed area. Most of the time, a detailed topographic map of the site does not exist. However, the available topographical from the Ethiopian Mapping Agency should be obtained.

Hydrologic Soil Characteristics

The SCS curve number loss rate is used to determine the hydrologic soil characteristics for the watershed. This book may be obtained from the SCS office in the state or county of interest. A hydrologic classification of each soil may be determined from reference TR 55 (1986).

Antecedent moisture condition (AMC) II was assumed for this analysis. AMC II should be selected unless rain records of the site are available. The curve numbers in Table 2 apply for AMC II. For dry conditions (AMC I) or wet conditions (AMC III), equivalent curve numbers can be computed (Veissman, Lewis and Knapp (1989)).

Precipitation

There are six methods of precipitation models available. The gage data is not available for the watersheds and will not be used for the example given. The frequency-based storm will be used in this illustrative example. This hypothetical storm will be automatically distributed according to the specified depth/duration data. A triangular precipitation distribution is constructed such that the depth specified for any duration occurs during the central part of the storm.

The required inputs are:

Exceed Probability;
Storm frequency; the 100-year storm is used for the example or 1% probability
Storm Area; storm size will be the same as the watershed area.
Series Type; annual or partial.
Duration of Max Intensity; smallest input duration
Storm Duration; maximum input duration

Unit Hydrograph Method

The SCS dimensionless unit hydrograph is used in the illustrative example. The only parameter necessary to obtain this unit hydrograph is the Time of Concentration (TC), which is determined by techniques given in TR-55 (1986). For the example watershed, there are three components of TC: sheet flow, shallow concentrated flow, and open channel flow. See Table 3 for details in the determination of these values.

Flood Hydrograph Determination

As outlined above the frequency based storm will be used to determine the precipitation data, SCS curve number for the loss rate, and the SCS unit hydrograph. The determinations of these parameters are given in Tables 1, 2 and 3.

Table 1: Soil Types and Curve Numbers Computation Sheet

Soil	**Description**	**Area***	**CN**	**Weighted CN (Area x CN)**
Water	New Pond	28	100	2800
RsC	Rockaway Rock Outcrop	71.5	75	5362.5
RvF	Rock Outcrop	31.8	90	2862
HbC	Stony Loam	27.5	70	1925
RsD	Rock Outcrop	23.25	89	2069.25
RgA	Ridgebury Very Stony Loam	18.5	80	1480
RpC	Rock Very Stony Sandy Loam	17.25	72	1242
RiB	Extremely Stony Loam	12	81	972
RrD	Extremely Stony Sandy Loam	9.5	76	722
Cm	Carisle Muck	7.75	82	635.5
Ad	Adrian Muck	3	80	240
PeC	Park Extremely Stony Sandy Loam	2	70	140
Wm	Whitman, Stony Loam	1.5	80	120
RsC	Rockaway, Rock outcrop	1.5	83	124.5
RpC	Rock Very Stony Sandy Loam	0.75	75	54
Total		**255.8**		**20748.75**

* Note: Area in arbitrary units (Total Weighted CN)/(Total Area) **= CN = 81.11**

Table 2: Runoff Curve Numbers

Cover Type	Hydrologic Condition	Curve Numbers For Hydrologic Soil Group			
		A	B	C	D
Pasture, grassland, or range— continuous forage					
	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow, continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush--brush-weed-grass mixture with brush the major element.					
	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30	48	65	73
Woods--grass combination (orchard or tree farm).					
	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads--buildings, lanes, driveways and surrounding lots		59	74	82	86

Because of the varied shape and topography of the Stickle Pond watershed, three paths of flow were selected to determine TC, see Figure 2. These three paths are located in the top, middle and bottom of the watershed. The TR-55 reference 1 is used to determine TC.

There are three components of TC:

1. Sheet flow $TC_1 = \frac{0.007\,(nL)^{0.8}}{(P_2)^{0.5}\,S^{0.4}}$

Where, n = 0.4 woods,
L = 150 (L maximum of 200 m, a smaller value is recommended),
P_2 = 3.25 mm (2 year, 24 hour rain),
S = slope of watershed at the divide.

2. Shallow concentrated flow $TC_2 = \frac{L}{3600v}$

Where, v = $16.134s^{1/2}$
s = water course slope.

3. Open channel flow $TC_3 = \frac{L}{3600v}$

where, v = $(1.49r^{2/3} s^{1/2})/n$
and n = 0.03 from site visit, TR55 recommends 0.05;
s = channel slope;
r = hydraulic radius = 0.4 (TR55).

4. Total TC = TC1 + TC2 + TC3

Table 3: Computation Sheet For Tc, Time of Concentration

Path	S	TC1	L (m)	DH (m)	S	TC2	L (m)	DH (m)	S	TC3	Total TC
Top	.060	.317	1600	80	.050	.123	4400	78.3	.0178	.339	.779
Middle	.0313	.411	4900	125	.026	.523	2000	68.3	.0342	.111	1.045
Bottom	.224	.187	3400	272	.080	.207	3200	19.3	.0060	.425	.819

The largest value of the time of travel is usually selected for TC. But because of the number of assumptions made in selecting the parameters, an average TC will be used.

TC = 0.88 hours and, TLAG = 0.6 x TC = **0.53 hours**

Table 4: Sub Basin Characteristics

Sub-Basin	Tc(Hours)	Area (Square km)	CN	100 yr Precip (mm)
North	0.77	.771	85.2	7.5
South	0.9245	1.859	79.4	7.5

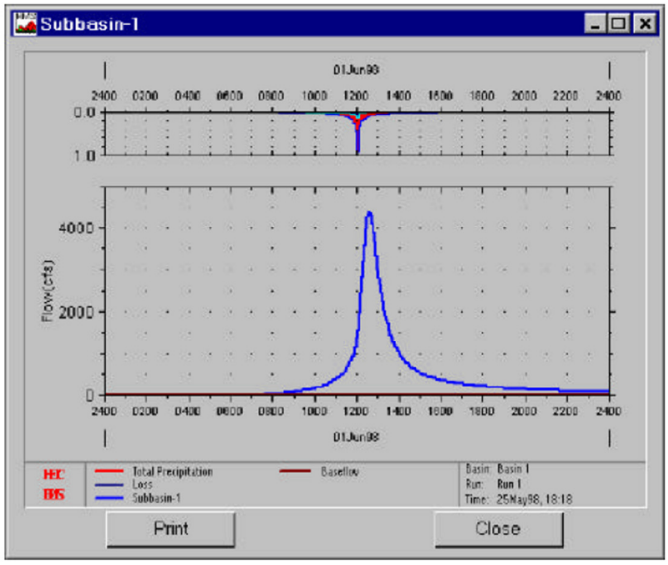


Figure 1: Output for a sub-basin Hydrograph

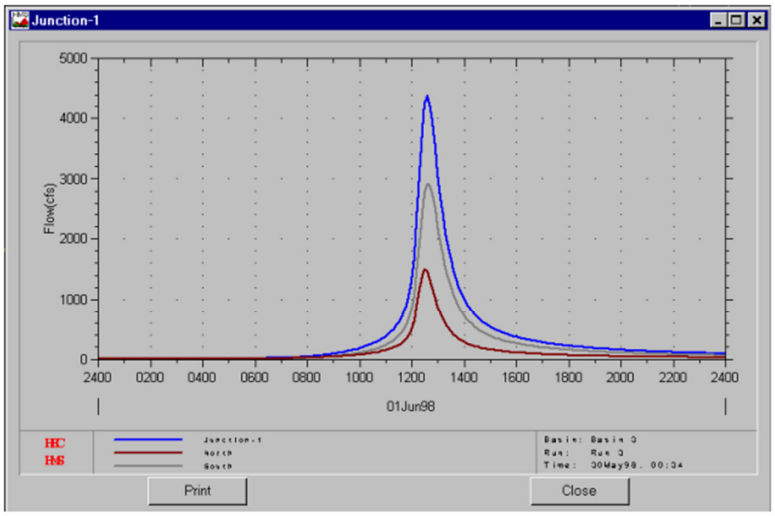
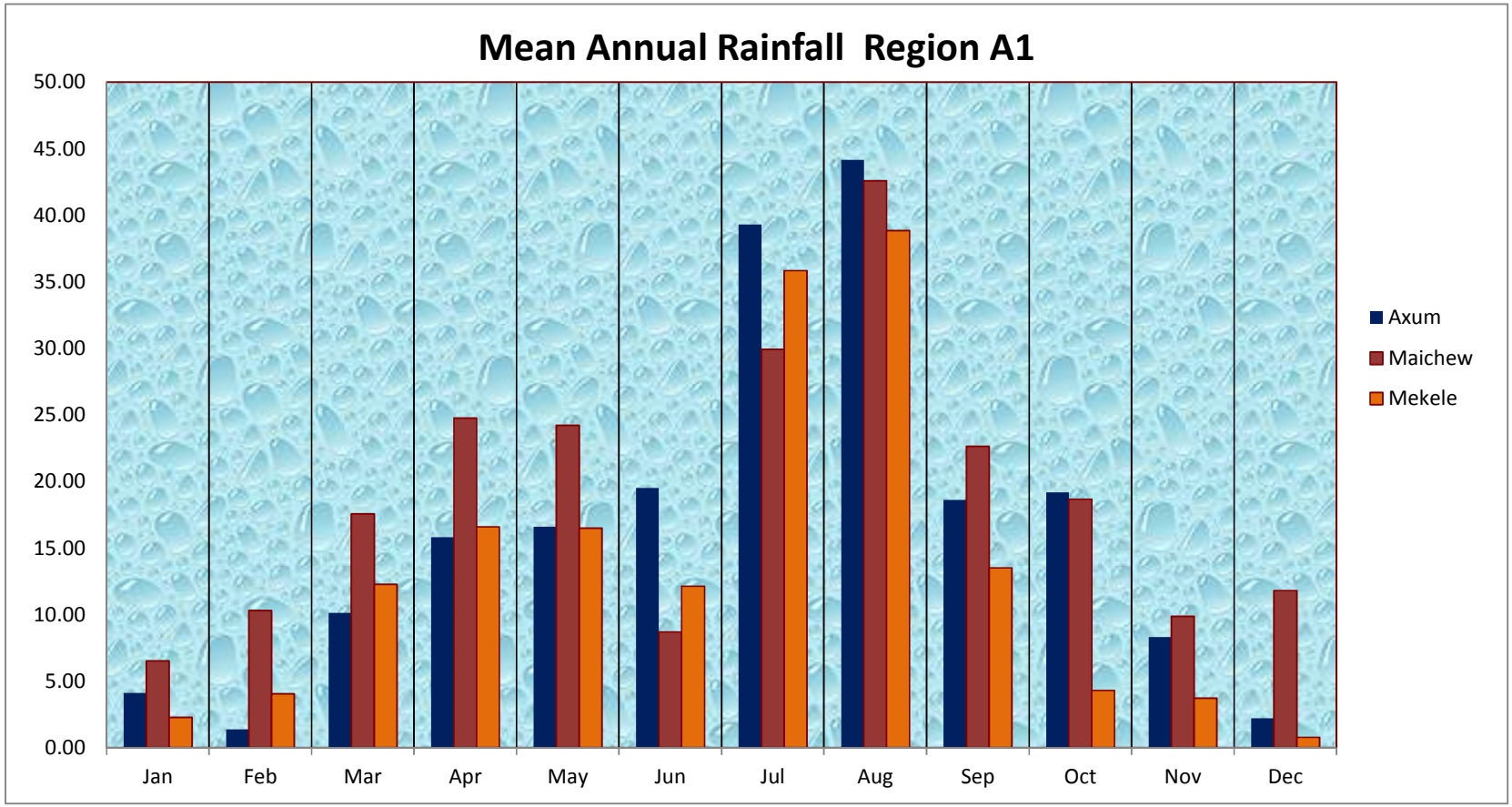
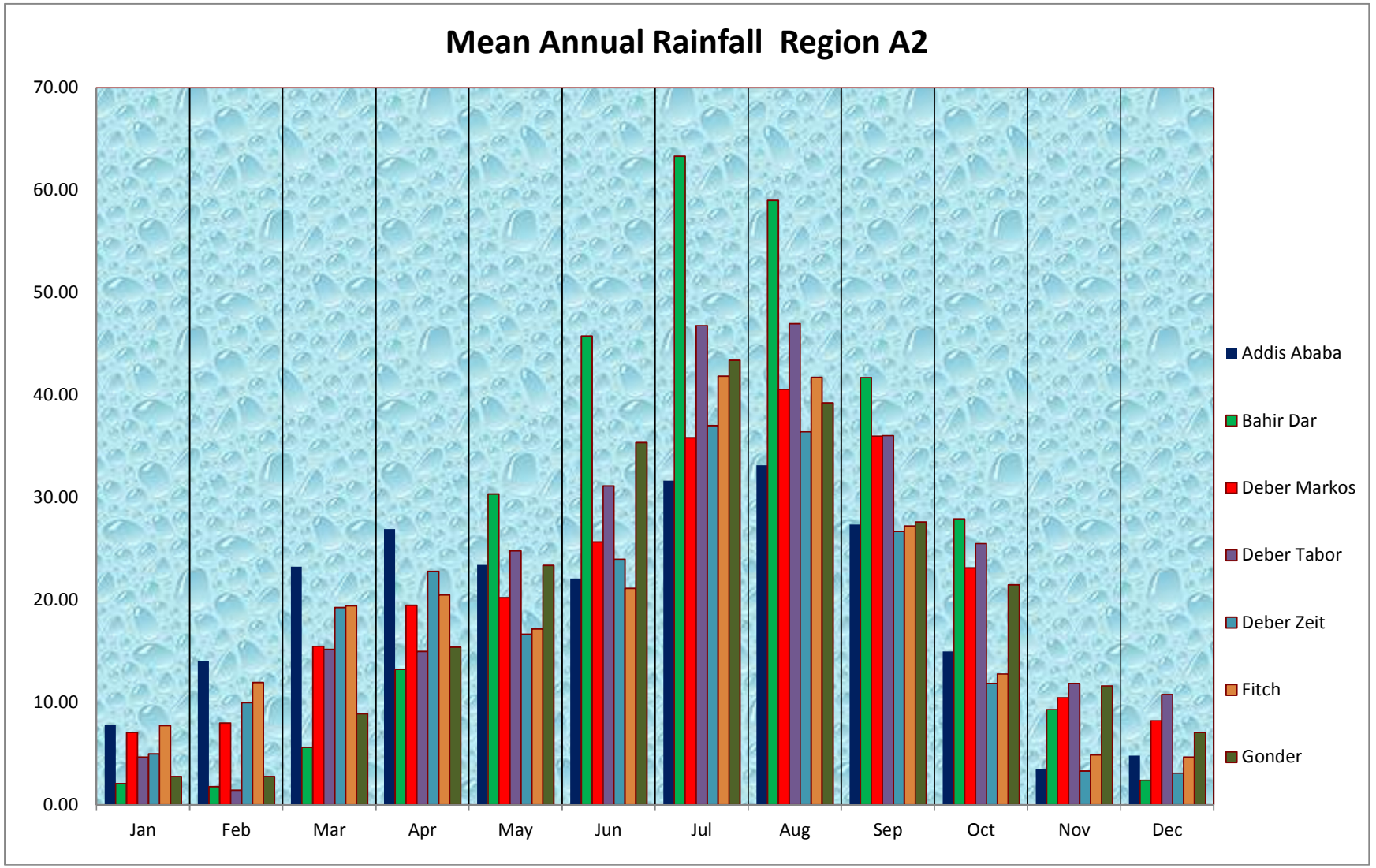
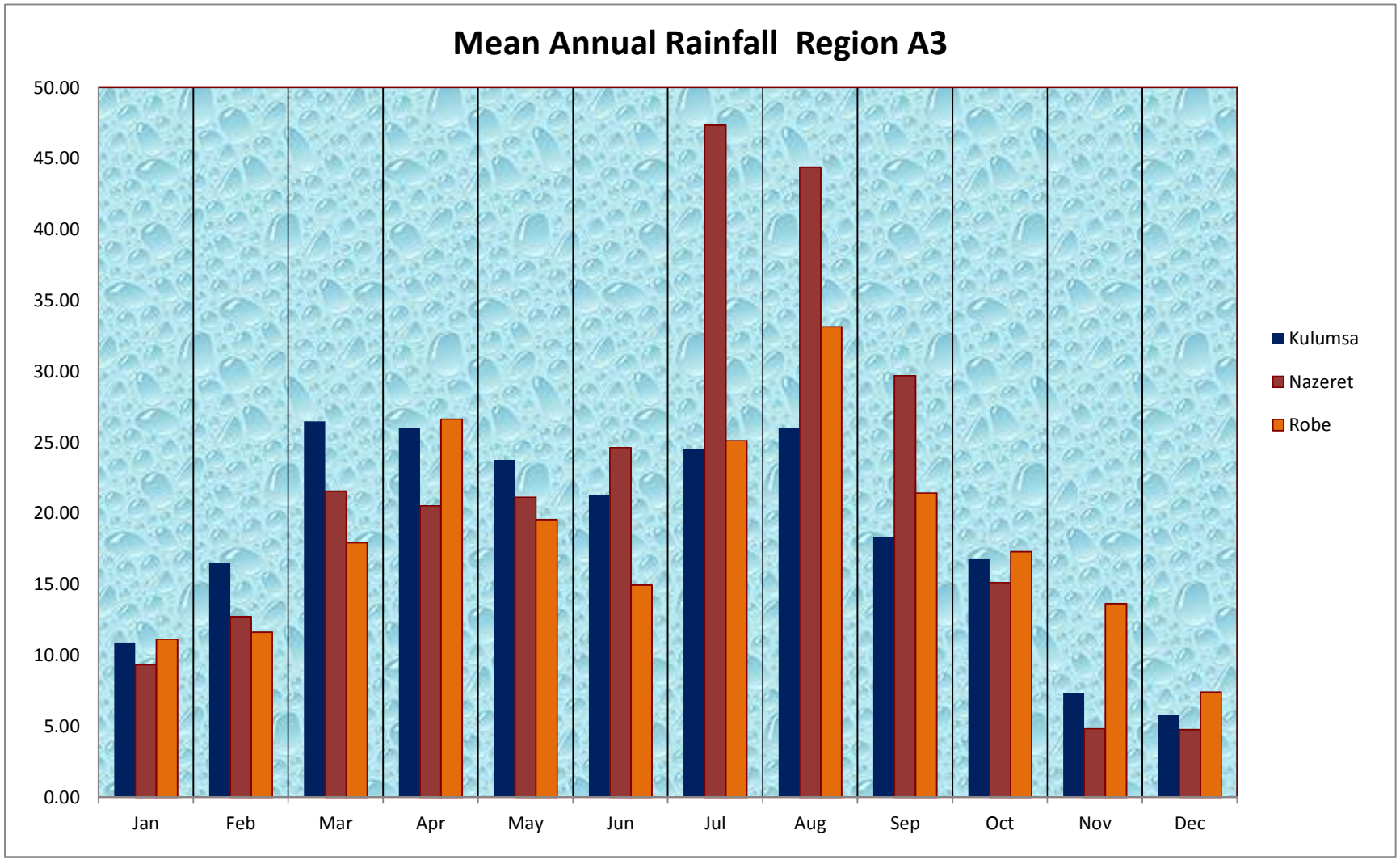


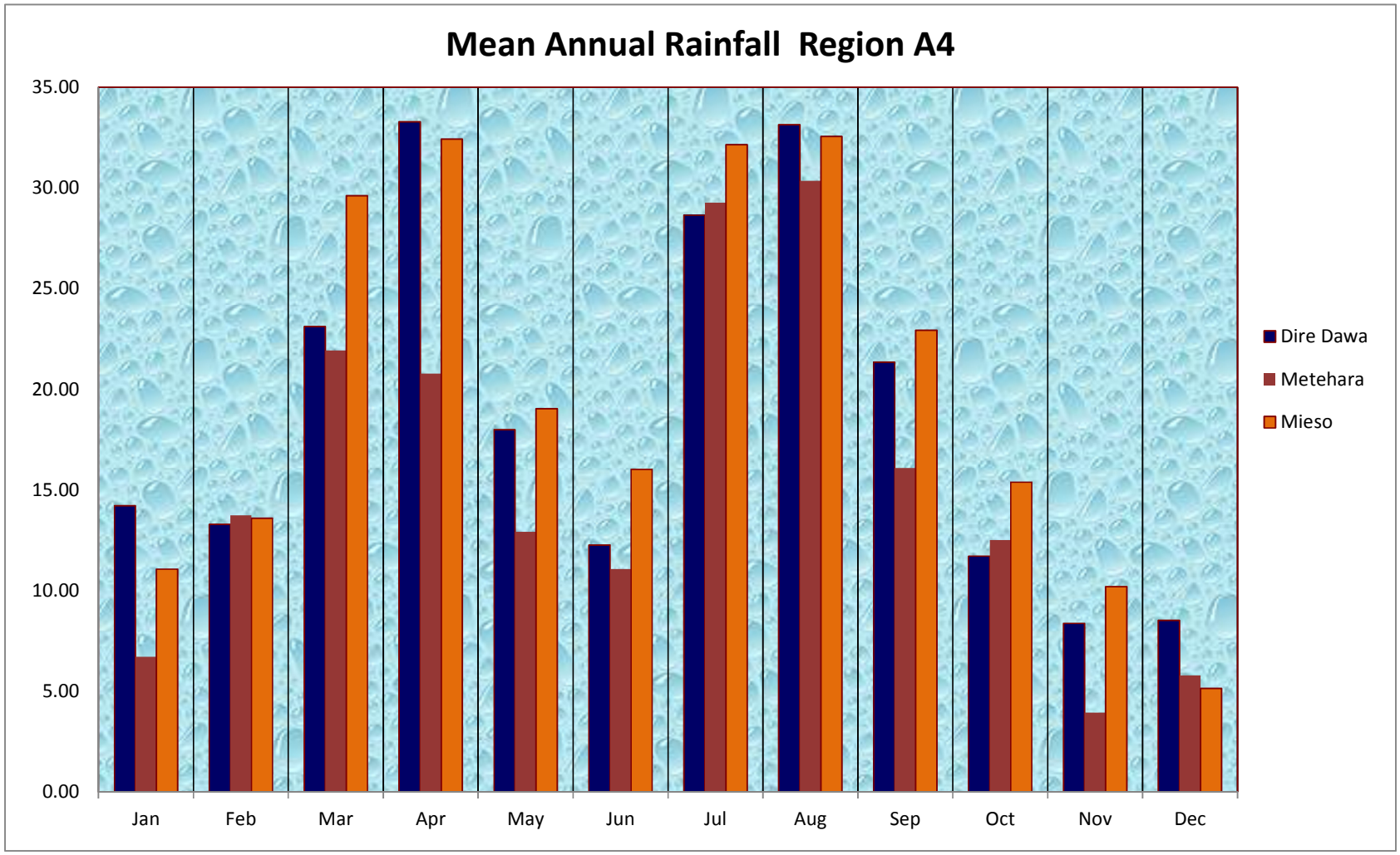
Figure 2: Combined output hydrograph

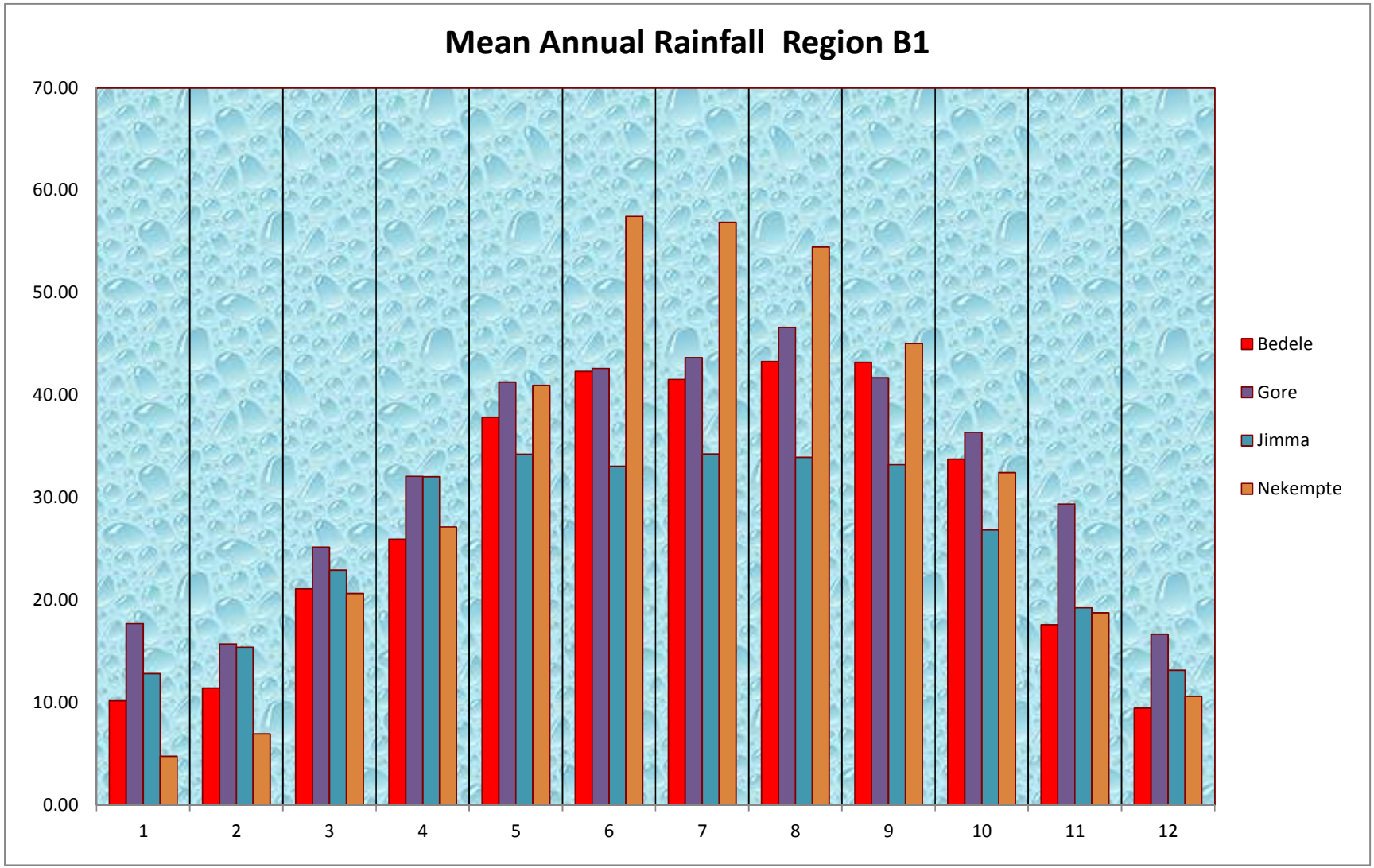
APPENDIX 5B - MEAN ANNUAL RAINFALL

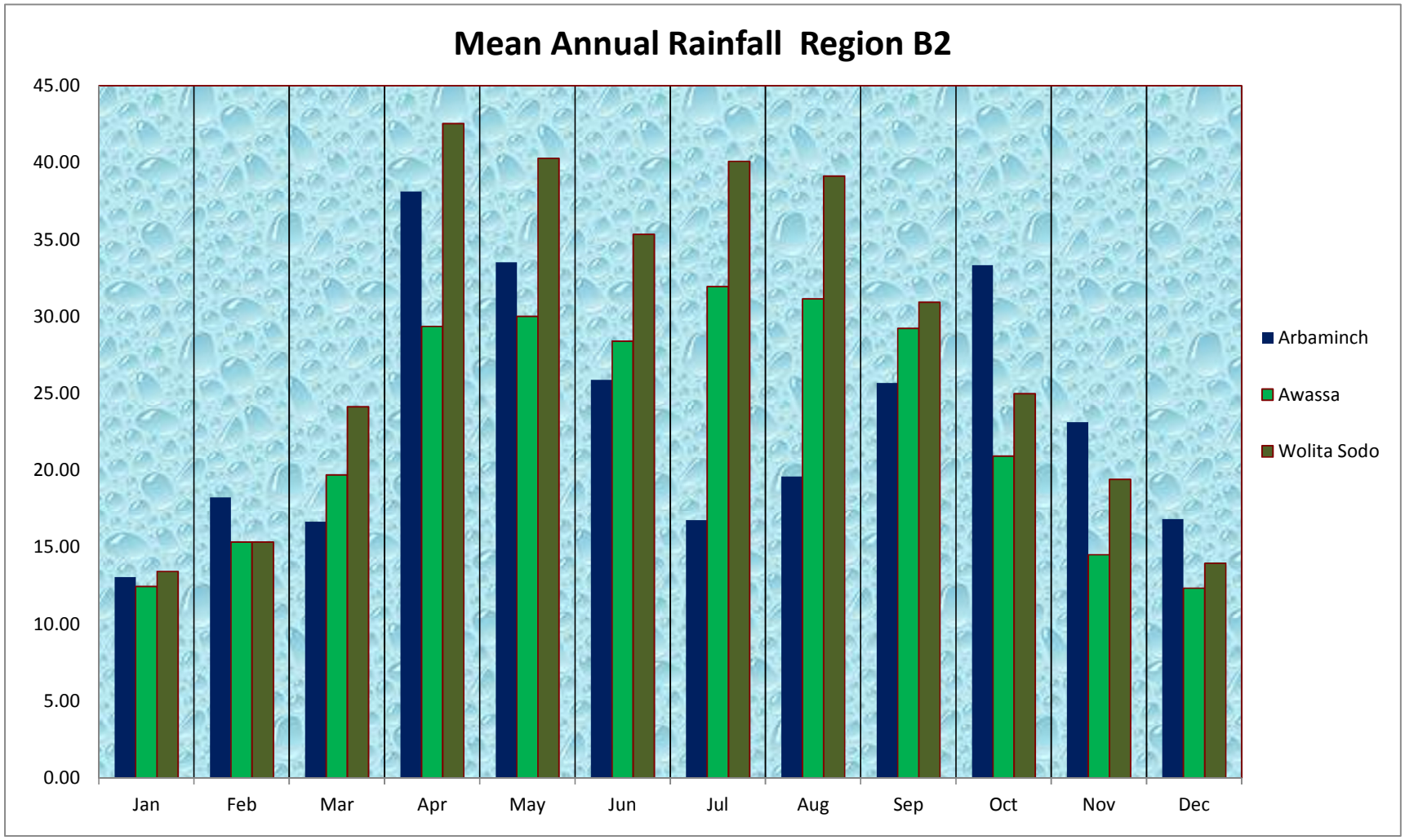


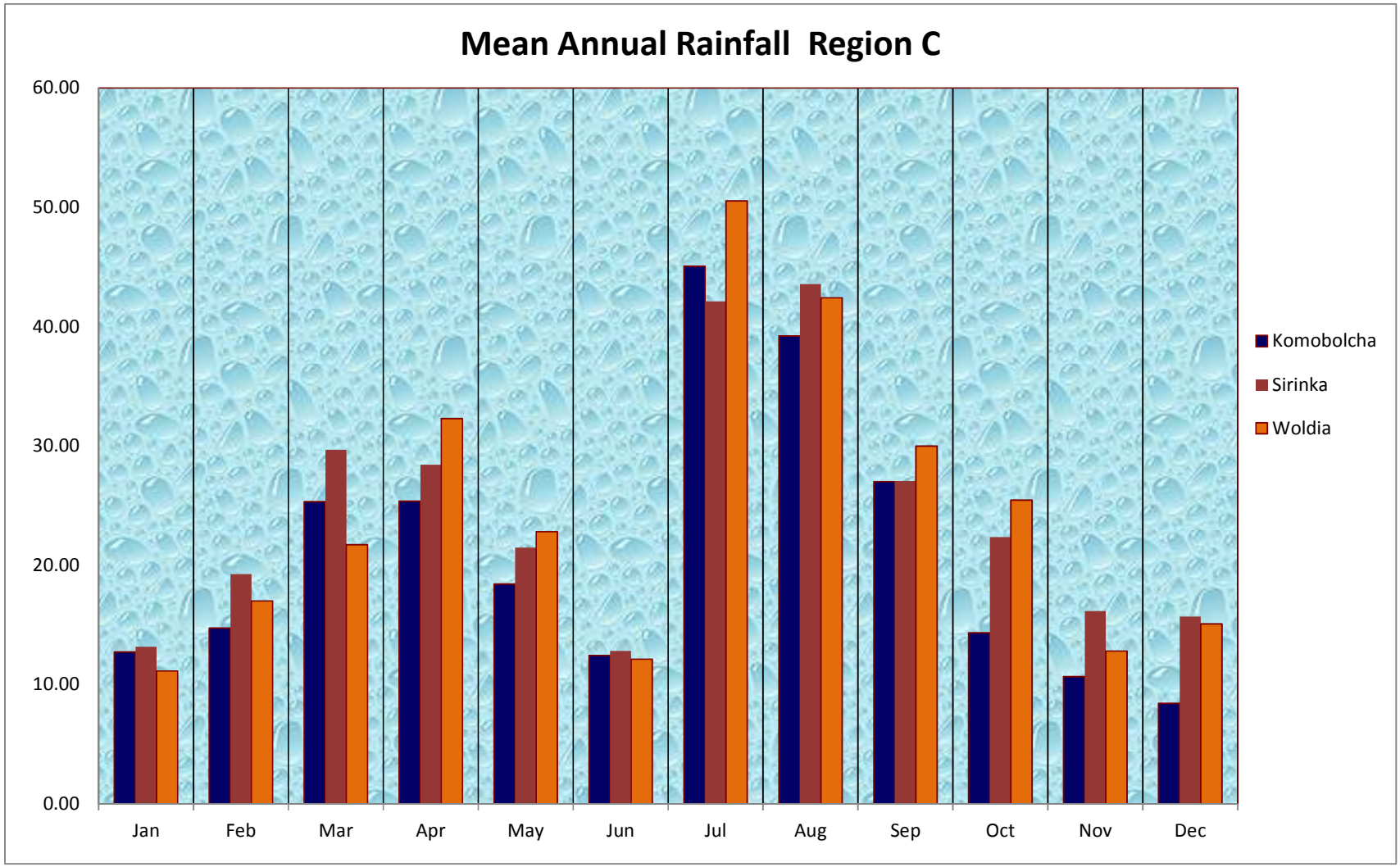

Mean Annual Rainfall Region A2
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
Jan
Feb
Mar
Apr
May
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Jul
Aug
Sep
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Nov
Dec
Addis Ababa
Bahir Dar
Deber Markos
Deber Tabor
Deber Zeit
Fitch
Gonder

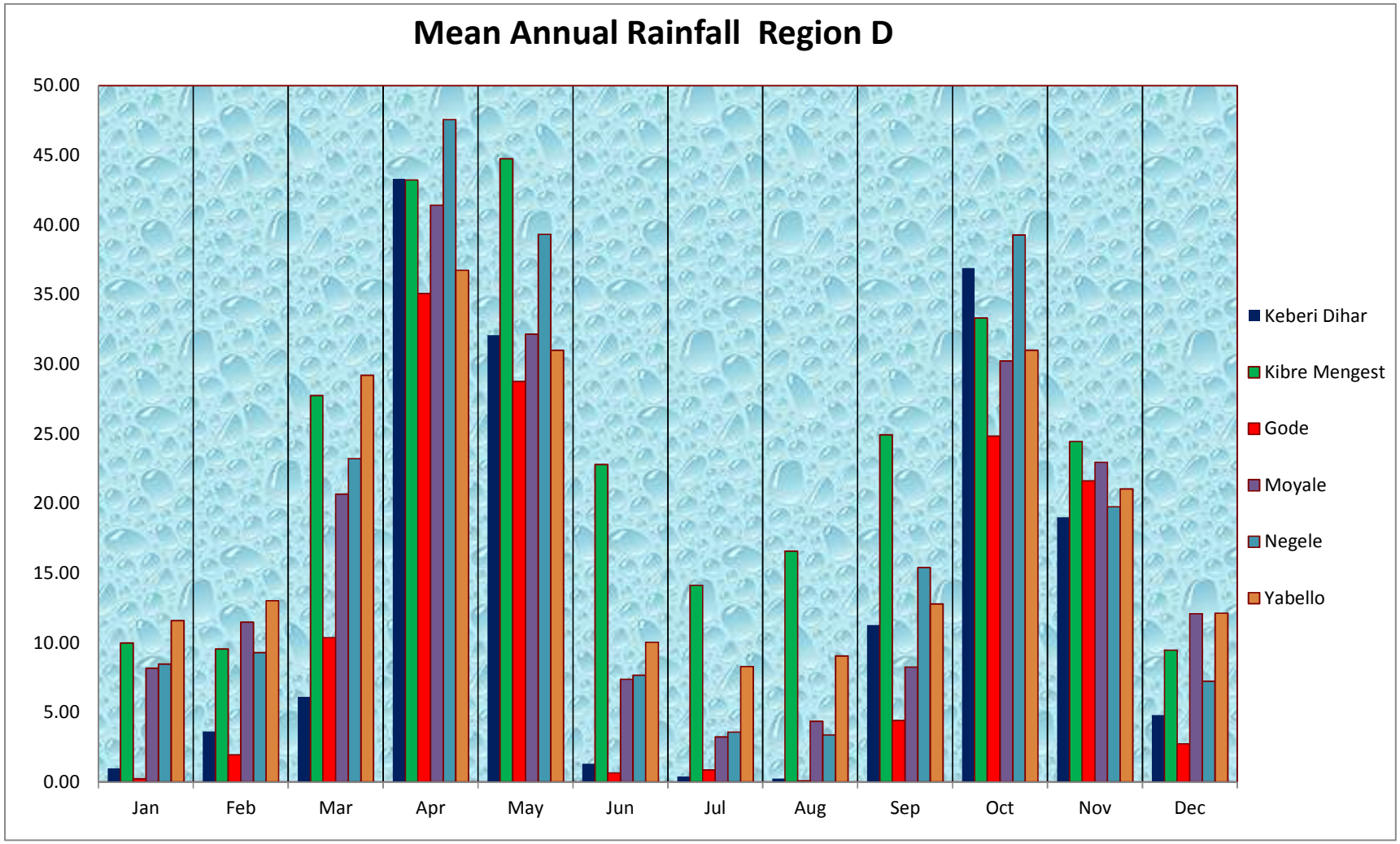

Mean Annual Rainfall Region A3
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Kulumsa
Nazeret
Robe


Mean Annual Rainfall Region A4
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Dire Dawa
Metehara
Mieso


Mean Annual Rainfall Region B1
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
1
2
3
4
5
6
7
8
9
10
11
12
Bedele
Gore
Jimma
Nekempte


Mean Annual Rainfall Region B2
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Jan
Feb
Mar
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May
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Jul
Aug
Sep
Oct
Nov
Dec
Arbaminch
Awassa
Wolita Sodo


Mean Annual Rainfall Region C
60.00
50.00
40.00
30.00
20.00
10.00
0.00
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Komobolcha
Sirinka
Woldia


Mean Annual Rainfall Region D
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
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Nov
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Keberi Dihar
Kibre Mengest
Gode
Moyale
Negele
Yabello

6 HYDRAULIC DESIGN OF OPEN CHANNELS

6.1 Introduction

An open channel is a conduit in which water is conveyed with a free surface. Although closed conduits such as culverts and storm drains are open channels when flowing partially full, the term is generally applied to natural and improved watercourses, gutters, ditches, and channels. While the hydraulic principles discussed in this section are valid for all drainage structures, the primary consideration is given to channels along, across, approaching and leaving the highway.

In addition to performing its hydraulic function, the drainage channel should be economical to construct and maintain. Open channels should be reasonably safe for vehicles accidentally leaving the traveled way, pleasing in appearance, convey collected water without damage to the highway or adjacent property and minimize the environmental impacts.

These considerations are usually so interrelated that optimum conditions cannot be met for one without compromising one or more of the others. The objective is to achieve a reasonable balance, but the importance of traffic safety must not be underrated.

There are various types of open channels encountered by the road drainage designer of highway facilities including:

- Stream channels;
- Chutes;
- Roadside channels or ditches;
- Irrigation channels; and
- Drainage ditches.

The principles of open channel flow hydraulics are applicable to all drainage systems including culverts.

Stream channels are usually:

- Natural channels with their size and shape determined by natural forces (morphology);
- Compound in cross section with a main channel for conveying low flows and a floodplain to transport extreme flood flows; and
- Shaped geomorphologically by the long-term history of the sediment load and water discharge that they have experienced.

Artificial channels include roadside channels, irrigation channels, and drainage ditches that are man-made with regular geometric cross sections, and unlined, or lined with artificial or natural material to protect against erosion.

While the principles of open channel flow are the same regardless of the channel type, stream channels and artificial channels (primarily roadside channels) are treated separately in this chapter.

6.2 Hydraulic Considerations

An evaluation of hydraulic considerations for channel design alternatives should be made early in the project development process. The extent of the hydrologic and hydraulic analysis should be commensurate with the type of highway, complexity of the drainage facility, and associated costs, risks, and environmental impacts.

The hydraulic design of an open channel consists of developing a channel section to carry the design discharge under the controlling conditions, adding freeboard as needed and determining the type of channel protection required to prevent erosion. In addition to erosion protection, channel linings can be used to increase the hydraulic capacity of the channel by reducing the channel roughness.

The hydraulic capacity of a drainage channel is dependent on the size, shape, slope and roughness of the channel section. For a given channel, the hydraulic capacity becomes greater as the grade or depth of flow increases. The channel capacity decreases as the channel surface becomes rougher.

A rough channel can sometimes be an advantage on steep slopes where it is desirable to keep flow velocities from becoming excessively high. A good open channel design minimizes the effect on existing water surface profiles.

Open channel designs, which lower the water surface elevation, can result in excessive flow velocities and cause erosion problems. A planned rise in water surface elevation can cause:

- Objectionable flooding of the road bed and adjacent properties; and
- An environmental and maintenance problem with sedimentation due to reduced flow velocities.

Additional hydraulic considerations include those of channel and floodwater characteristics such as movable beds, heavy bed loads and bulking during flood discharges.

6.3 Safety Consideration

An important aspect of highway drainage design is that of traffic safety. The shape of a roadside channel section should minimize vehicular impact and provide a traversable section for vehicles leaving the traveled way. The ideal channel section, from a safety standpoint, will have flattened side slopes and a curved transition to the channel bottom.



Figure 6-1: Errant Vehicles

(Source: TRL,Highway Design Note 1/01)

6.4 Maintenance Consideration

The design of open channels and roadside ditches should recognize that periodic maintenance inspection and repair is required. Provisions should be incorporated into the design for access to a channel by maintenance personnel and equipment.

When assessing the need for permanent or temporary access easements, entrance ramps and gates through the right of way fences, consideration should be given to the size and type of maintenance equipment required. Damaged channels can be expensive to repair and interfere with the safe and orderly movement of traffic. Minor erosion damage within the right of way should be repaired immediately after it occurs and action taken to prevent the recurrence.

Conditions, which require extensive repair or frequently recurring maintenance, may require a complete redesign rather than repetitive or extensive reconstruction. The advice of an Expert Drainage Engineer should be sought when evaluating the need for major restoration.

The growth of weeds, brush, and trees in a drainage channel can effectively reduce its hydraulic efficiency. The result being that a portion of the design flow may overflow the channel banks causing flooding and possible erosion.

Channel work on some projects may be completed several months before total project completion. During this interim period, the contractor must provide interim protection measures and possibly advance the planned erosion control program to assure that minor erosion will not develop into major damage.



Figure 6-2: Damaged side ditch along Assossa Kumruk Road

6.5 Economics

Economical drainage design is achieved by selecting the design alternative which best satisfies the established design criteria at the lowest cost.

The economic evaluation of design alternatives should be commensurate with the complexity and importance of the facility. Analysis of the channel location, shape, size, and materials involved may reveal possibilities for reducing construction costs, flood damage potential, maintenance problems and environmental impacts.

6.6 Coordination with Other Agencies

There are many government agencies and private entities engaged in water related planning, construction and regulation activities whose interests can affect the design of highway drainage channels. Such agencies may request the channel design satisfy additional and perhaps governing design criteria. Early coordination with these agencies may help avoid delays in the project development process. Early coordination may also reveal opportunities for cooperative projects, which may benefit ERA, Ministry of Water & Energy, Ministry of Agriculture and private irrigation farm owners.

6.7 Environmental Considerations

Many of the same principles involved in sound highway construction and maintenance of open channels require environmental considerations. Erosion, sedimentation, water quality, and aesthetics should be of prime concern to the highway design engineer.

6.8 Alignment and Grade

The highway drainage channel must be located where it will best serve its intended purpose, using the grade and alignment obtainable at the site. However, abrupt changes in alignment and grade should be avoided as much as possible. A sharp change in alignment presents a point of attack for flowing water, and abrupt changes in grade can result in possible scour when the grade is steepened or deposition of transported material when the grade is flattened.



Figure 6-3: Erosion at a channel bends (Wollega Region)

Changes in alignment should be made as gradually as possible to introduce the least amount of unstable flow. When possible, locate horizontal curves where the profile is flattest. Open channels should be graded to a desirable minimum gradient of 0.005 m/m so that some slight settlement will not create large areas of standing water. Maximum slopes should reflect the type of soil and linings to be used especially for grass-lined channels.



Figure 6-4: Typical grass-lined channel

In principle, a drainage channel should have a flow velocity that neither erodes nor cause deposition in the channel (refer to Chapter 2, Section 2.17.3 Table 2. 3. This optimum velocity is dependent on the size and slope of channel, the quantity of flowing water, the material used to line the channel, the nature of the bedding soil and the sediment being transported by the flow.



Figure 6-5: Points of discharge

The point of discharge into a natural watercourse requires special attention. Water entering a natural watercourse from a highway drainage channel should not cause eddies with attendant scour of the natural watercourse. In erodible embankment soils, if the flow line of the drainage channel is appreciably higher than that of the watercourse at the point of discharge, then the use of a spillway is advisable to prevent erosion of the channel.

6.9 Channel Section

6.9.1 Natural Channels

Natural channels are water conveying sections such as streams, rivers, and swales which have been formed by natural forces. Good drainage design involving natural channels will ensure that the existing flow characteristics are retained such as size and shape of channel, flow velocities, and flow distributions.

Channel cross sections at roadside locations should be determined by hydraulic and erosion considerations to achieve a channel that has adequate capacity for the flow quantity to be

handled, will require little or no maintenance, will fit in the space available and will be economically acceptable to construct.

6.9.2 Triangular V-Ditch

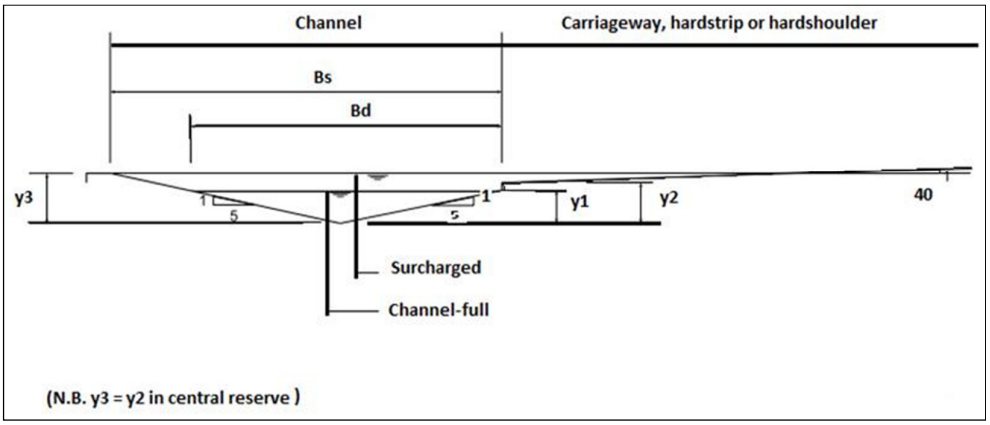


Figure 6-6: Cross-sectional shape of triangular channel



Figure 6-7: A non-traversable drainage V-ditch such as this is a safety hazard

The shape of a channel section is generally determined by considering the intended purpose, terrain, flow velocity and quantity of flow to be conveyed. The Triangular V-ditch is intended primarily for low flow conditions such as in median and roadside ditches. V-shaped ditches are susceptible to erosion and will require lining when flow velocities exceed the permissible flow velocities in Chapter 2, Table 2.3

6.9.3 Trapezoidal Ditch

The most common channel shape for large flows is the trapezoidal section. Trapezoidal channels are easily constructed by machinery and are often the most economical to construct. When a wide trapezoidal section is proposed, both traffic safety and aesthetics can be improved by rounding all angles of the channel cross section with vertical curves.



Figure 6-8: Triangular drain at Abay Valley

(Addis Ababa –Debre Markos Road, left and Wollega region right)

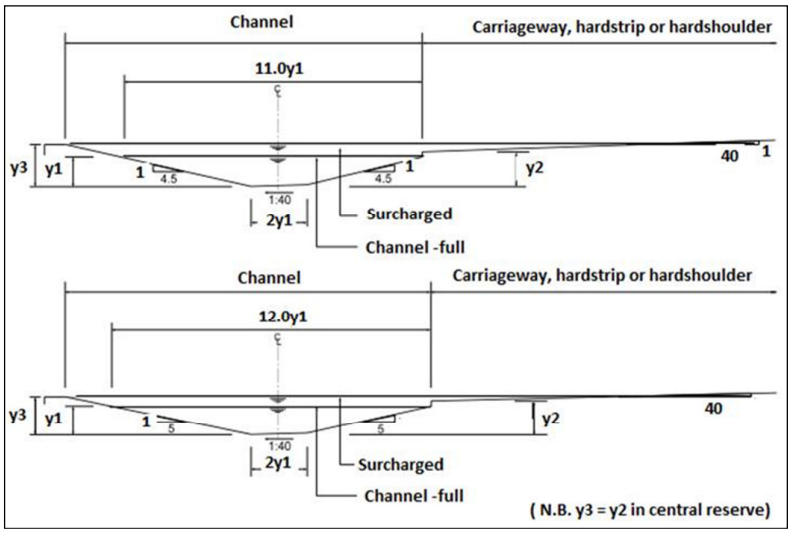


Figure 6-9: Cross-sectional shape of trapezoidal channels

6.9.4 Rectangular Ditch

Rectangular channels are often used to convey large flows in areas with limited right of way and at town sections. At some locations, guardrail or other types of positive traffic barrier may be necessary between the traveled way and the channel.



Figure 6-10: Typical rectangular ditch north of Addis Ababa

Although rectangular channels are relatively expensive to construct, since the walls must be designed as earth retaining structures, the construction costs can be somewhat offset by the reduced costs associated with right of way, materials, and channel excavation.

6.10 Channel Design

Hydraulic design associated with natural channels and roadway ditches is a process that selects and evaluates alternatives according to established criteria. These criteria are the standards established to insure that a highway drainage system meets its intended purpose without endangering the structural integrity of the facility itself and without undue adverse effects on the environment or the public welfare.

Side drains are essential for the performance of the road and they should be properly designed. Any savings in design cost will be far outweighed by increased maintenance costs over the life of the road.

The design of side drains should consider:

- Whether the drain serves the whole width of the road or just half the width;
- Does the drain serve just the road or does it also provide drainage to the adjoining areas (refer to Figure 6.11);
- The gradient of the road; and
- The nature of the materials the road is crossing: are they easily eroded like silts and sands or erosion resistant like stiff clays or rock

The designer controls a range of variables to fit the side drains to their environment:

- The channel shape and size;
- Whether or not the channel is lined;
- Whether scour checks are provided and if so their spacing; and
- The spacing of turnouts or side drain relief culverts.



Figure 6-11: Roadside ditch collecting lateral flows

(At CMC Roundabout, North of Addis Ababa)

6.10.1 Erosion Control

The need for scour checks should be assessed according to the gradient, soil type, velocity and volume of water in the side drain.

Look for evidence of erosion to assess the severity of the problem and the frequency of scour checks required. In extreme cases (steep slopes where there is no possibility of using relief culverts to evacuate water) the side drain will have to be lined in either masonry or concrete to prevent erosion. Where gullies outlets are proposed, the discharge can be controlled by using check dams (see figure 6.12) built from rock just downstream of the gully outlet. Gabion baskets should be used for stability in deeper gullies.



Figure 6-12: Check dams in Tigray Region on the left and in Gojam on the right

6.10.2 Turnouts

Although it is more of a construction rather than a design issue, it is best practice to leave a solid bund of earth to dam the drain downstream of the turnout. If drains are being built by hand then a short section of drain can be left un-excavated to provide the bund (refer to Figure 13).

The frequency of turnouts is controlled by various factors. On flat ground, the capacity of the side drain is critical: turnouts are required to prevent the side drain from overflowing. On sloping ground concentration of flows is critical: as the ground and hence the side drain becomes steeper more turnouts are required to prevent the flow concentration leading to

erosion at the end of the turnout.



Figure 6-13: Photo of a Turnout (in Wollega)

6.10.3 Relief Culverts

Relief culverts are different from watercourse culverts. Their purpose is to relieve the flow from long stretches of side drain ditches located on the uphill side of a road where miter drains cannot be provided. These culverts should be used where possible instead of using a long side ditch without relief culverts.

6.10.4 Catch Pit Inlets

Side drain relief culverts are normally provided with a catch pit inlet structure. This is designed to capture the water flowing along the side drain and turn that flow through 90^{o}. It then flows under the road and is discharged through an outfall channel.



Figure 6-14: Typical photo of catch pit inlet structure (Gojam)

6.10.5 Town Section Drainage Channels

The provision of access across drainage channels is often overlooked in Ethiopia. If access across drainage channels is not provided, people will block the drain if it presents an obstruction, causing water to flood the road. Therefore, access needs to be considered carefully at a design stage (refer to figures 6.15, 6.16 and 6.17).

In rural areas a small number of accesses may be required in villages – for a market or shop for instance. Lined drains with cover slabs are a typical solution (see figure 6.16).



Figure 6-15: Typical town section drainage channels

(Addis Ababa left and Assossa town right)

Junctions with other roads may also need access culverts to maintain continuity of drainage. The same covered drain detail can be used to avoid raising the road locally to accommodate larger structures.

6.10.6 Access Type Culverts at a Junction

The purpose of an access culvert is to allow water in side drains to flow under a minor road where it branches off from the main road at a junction.

- An access culvert will be required if the minor road slopes towards the main road;
- But if the minor road slopes away from the main road the side drains can be connected.

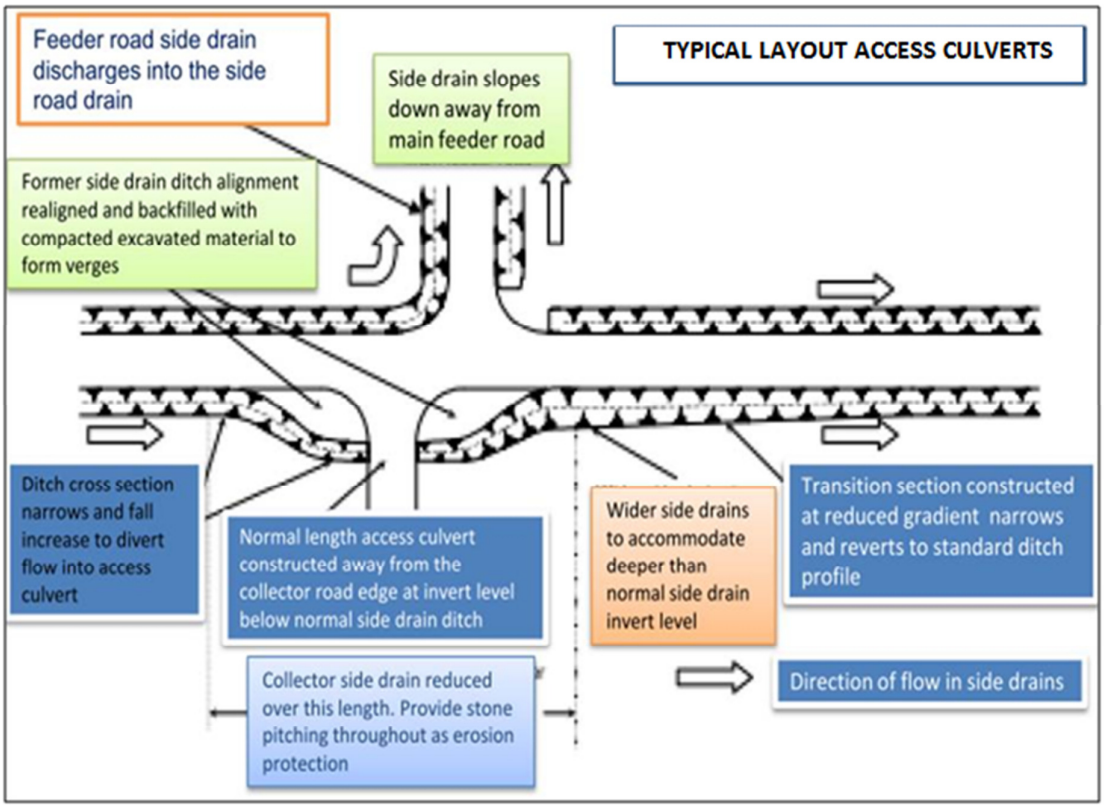


Figure 6-16: Typical layout of junction drain ditch design

An access culvert will normally be required at junctions between a feeder road and a collector road. Crossroads junctions will typically require one or two access culverts unless located on a natural ridge or saddle. Access to plots or property is usually provided in the form of access slabs over lined drains.



Figure 6-17: Sample photos of access slabs

6.10.7 Design Considerations

The invert level of an access culvert should be the same as (or slightly lower) than that of the uphill side drain that needs to be passed under the road.

It is common to provide a bell mouth to allow longer turning vehicles to negotiate the junction. For reasons of economy, the access culvert is often sited away from the collector road edge before the bell mouth where the feeder road is of normal width. The collector

road side drain should be diverted to flow through the access culvert. Such diversions are frequently lined in stone pitching and may have a slightly greater fall than the side drain to encourage the change in flow direction.

Access culverts should be of just sufficient capacity to accommodate the maximum side drain flow.

6.11 Design Criteria of Channels

Design criteria establish the standards by which an open channel shall be constructed. They form the basis for the selection of the final design configuration. Listed below are examples of design criteria that shall be considered for channel design.

6.11.1 Stream Channels

The following design criteria apply to natural channels:

- The hydraulic effects of flood plain encroachments shall be evaluated over a full range of frequency-based peak discharges from the design frequency to the check/review recurrence intervals on any major highway facility as deemed necessary by the designer;
- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions as much as practicable. Some form of energy dissipation may be necessary when existing conditions cannot be duplicated;
- Stream bank stabilization shall be provided, when appropriate, to any stream disturbance such as encroachment and should include both upstream and downstream banks and the local site; and
- Features, such as dikes and levees, associated with natural channel modifications should have a 5-meter minimum top width with access for maintenance equipment. Vehicle turning points shall be provided no further than 500 meters apart and at the end of any such feature.



Figure 6-18: Sample photo of stream channel

6.11.2 Roadside Channels

Side drains Keep water off the surface of the road and keep the foundations of the road dry. Effective side drains will reduce the need for maintenance by preventing deterioration of the surface and will provide a drier and hence safer road.

If the side drains are missing or not working then, water running along or across the road may lead to gully erosion. The foundations may get wet and soft leading to rutting.

A common reason that side drains stop working is that people crossing the drain block them: either vehicles driving across the drain and damaging it or pedestrians trying to make walking over it easier.

Water Disposal

Side drains collect runoff water. That water then has to be discharged from the drain. This can be either:

- To the adjoining ground by means of a turnout; or
- Across the road to the side drain on the downstream side of the road via a side drain relief culvert.

The position and number of turnouts should be indicated on the design drawings. The final location should be determined by site inspection so they are provided where they will work.

Erosion Control

Side drains channel water and concentrate flows, especially where water is discharged via turnouts. Scour of the side drains, if not controlled, can lead to the formation of gullies that eventually can become so deep that the road may have to be abandoned. The construction of simple scour checks and check dams in the side drains will reduce velocity, remove silt, and allow vegetation to become established thus controlling erosion.

Erosion downstream of turnouts can affect not only the road but also the adjoining land. To prevent erosion provide sufficient turnouts to disperse the flow and provide erosion protection where necessary.

Channel Location and Type

Assuming adequate functional design, the next most important design consideration is channel location. Locations that avoid poorly drained areas, unstable soil conditions, and frequently flooded areas can greatly reduce drainage related problems.

Often drainage and open channel considerations are not considered the primary decision factors in the roadway location; however, they are factors, which will often directly or indirectly affect many other considerations. Often minor alignment adjustments can avoid serious drainage problems.

If a channel can be located far enough away from the highway, the concerns of traffic safety and aesthetics can be somewhat mitigated. The cost of additional right of way may be offset somewhat by the reduced cost of erosion control, traffic protection, and landscaping.

Ditches should be located where they can fully intercept the flow from the natural catchments adjacent to the road. The location of ditches is mainly dependent on the space available. Possible locations are:

- Along the edge of the road;
- Along the top of cuttings; or
- At the toe of embankments.

In cuttings, ditches should preferably be positioned at the top of the cuttings to avoid potential erosion of the slope by surface water. Large sized ditches may create stability problems in the cutting slope and, therefore, appropriate measures should be taken.



Figure 6-19: Typical Road Side Ditch Locations

Where ditches are located alongside the road, they may be designed to convey the runoff from the carriageway as well as that of the natural catchment. Ditches should preferably consist of earth channels lined with a native grass species (or combination of species), in order to provide adequate resistance to flow erosion. However, this depends on the availability of water throughout the year in order for the native grass to grow.

The following design criteria apply to roadside channels:

- Channel side slopes shall not exceed the angle of repose of the soil and/or lining and shall be 2:1 or flatter in the case of rock-riprap lining. Stone pitching or grouted riprap must be used for channel side slopes steeper than 2:1;
- Flexible linings shall be calculated using the method of allowable tractive force;
- The design discharge frequency for permanent, roadside, ditch linings and for temporary linings shall be per Table 2.1, Chapter 2; and
- Channel freeboard shall be 0.3 meters.

6.12 Open Channel Flow

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow. The basic principles of fluid mechanics continuity, momentum, and energy can be applied to open channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principle problems of open channel flow analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than artificial channels. The drainage designer is expected to posses the knowledge of theoretical principles of open channel flow; hence most of the theoretical equation are not included in this chapter. The designer should refer to open channel flow text books for theoretical background information.

6.12.1 Flow Classification

The classification of open channel flow is summarized as follows:

Steady Flow

- Uniform Flow;
- Non uniform Flow;
  - Gradually Varied Flow;
  - Rapidly Varied Flow.

Unsteady Flow

- Unsteady Uniform Flow (rare);
- Unsteady Non uniform Flow;
  - Gradually Varied Unsteady Flow;
  - Rapidly Varied Unsteady Flow.

The steady uniform flow case and the steady non uniform flow case are the most fundamental types of flow treated in highway engineering hydraulics.

6.12.2 Equations

The following equations are those most commonly used to analyze open channel flow and are included here.

Continuity Equation – The continuity equation is the statement of mass in fluid mechanics. For the special case of one dimensional, steady flow of an incompressible fluid, it assumes the form:

$$Q = A_1V_1 = A_2V_2 \quad (6\text{-}1)$$

Where

Q = discharge, m^3/s

A = cross-sectional area of flow, m^2

V = mean cross-sectional velocity, m/s (which is perpendicular to the cross section). Subscripts 1 and 2 refer to successive cross sections along the flow path.

Manning's Equation –For a given depth of flow in a channel with a steady, uniform flow, the mean velocity, V, can be computed with Manning's equation:

$$V = \left(\frac{1}{n}\right) R^{2/3} S^{1/2} \quad (6\text{-}2)$$

Where

V = velocity, m/s

n = Manning's roughness coefficient

R = hydraulic radius = A/P, m

P = wetted perimeter, m

S = slope of the energy gradeline, m/m (note: for steady uniform flow,

S = channel slope, m/m)

The selection of Manning's 'n' is generally based on observation; however, considerable experience is essential in selecting appropriate 'n' values. The range of 'n' values for various types of channels and floodplains is given in Table 6-1. A further guide to the selection of 'n' values is the series of channel Photographs 6.1- 6.8 following Table 6.1.

For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in steady uniform flow. It is called the normal depth. The normal depth is used to design artificial channels in steady, uniform flow and is computed from Manning's Equation:

$$Q = \left(\frac{1}{n}\right) AR^{2/3} S^{1/2} \quad (6\text{-}3)$$

Where:

Q = discharge, m^3/s
n = Manning's roughness coefficient
A = cross-sectional area of flow, m^2
R = hydraulic radius = A/P, m
P = wetted perimeter, m
S = channel slope, m/m

If the normal depth computed from Manning's Equation is greater than critical depth, the slope is classified as a mild slope. Conversely, if the normal depth is less than critical depth, the slope is a steep slope. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

6.12.3 Channel Conveyance

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance K:

$$K = \frac{AR^{2/3}}{n} \quad (6\text{-}4)$$

and then Manning's Equation can be written as:

$$Q = KS^{1/2} \quad (6\text{-}5)$$

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and is independent of the streambed slope.

The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the flow distribution through the opening in a proposed stream crossing. Manning's Equation should not be used for determining high-water elevations in a bridge opening.

Energy Equation - The energy equation expresses conservation of energy in open channel flow as energy per unit weight of fluid that has a dimension of head and it therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities that give the total energy head at any cross section when added. In comparing an upstream open channel cross section designated 1 and a downstream cross section designated 2, the energy equation is:

$$h_1 + \propto_2 \left(\frac{{V_1}^2}{2g}\right) = h_2 + \propto_2 \left(\frac{{V_2}^2}{2g}\right) + h_L \qquad (6\text{-}6)$$

Where:

h_1 and h_2 = the upstream and downstream stages, respectively, m

α = kinetic energy correction coefficient

V = mean velocity, m/s

h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, m

The stage 'h' is the sum of the elevation head 'z' at the channel bottom and the pressure head, or depth of flow 'y,' i.e. h = z + y. The terms in the energy equation are illustrated graphically in Figure 6-2. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.

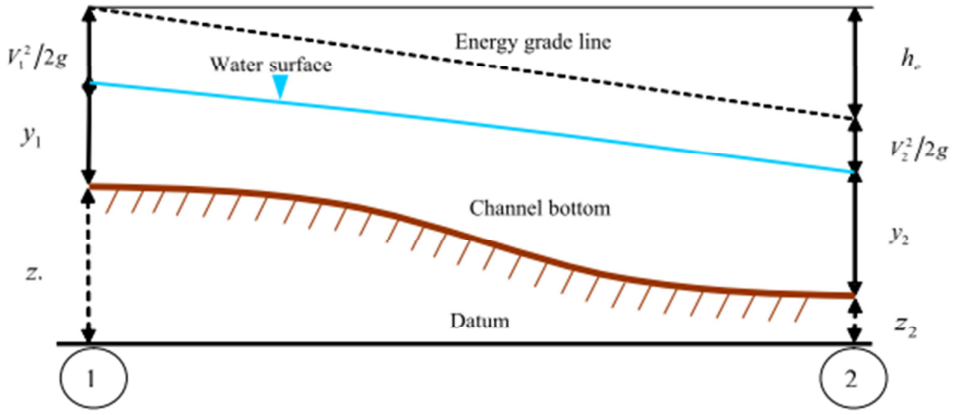


Figure 6-20: Terms in the Energy Equation

(Source: R A Crowder FDG2 – Chapter 7, 2009)

6.13 Hydraulic Analysis

Hydraulic design is about ensuring that drainage structures are selected to avoid flooding, that there is sufficient space for the water to flow at times of flood. It is also about avoiding potential problems such as erosion and siltation.

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness, and slope. The depth and velocity of flow are necessary for the design or analysis of channel linings and highway drainage structures.

6.13.1 Hydraulic Analysis Methods

Two methods are commonly used in hydraulic analysis of open channels. The *Single-Section* method is a simple application of Manning's Equation to determine tailwater rating curves for culverts, or to analyze other situations in which uniform or nearly uniform flow conditions exist. The *Step-Backwater* method is used to compute the complete water surface profile in a stream reach to evaluate the unrestricted water surface elevations for bridge hydraulic design, or to analyze other gradually varied flow problems in streams.

The single-section method will generally yield less reliable results because it requires more judgment and assumption than the step-backwater method. In many designs, however, the single-section method is all that is justified, for example, for a standard roadway ditch, culvert, storm drain outfall, etc.

6.13.2 Computer Programs

A computer program best performs hydraulic analysis of open channel problems. The recommended personal computer program is the US Army Corps of Engineers HEC-RAS River Analysis System. The Program was specifically developed for analysis of Highway Bridge and culvert backwater analysis. HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. HEC-RAS is capable of importing GIS/CAD data. For more information on computer programs refer to Chapter 15.

Another open channel modelling software is ISIS. ISIS is a full hydrodynamic simulator for modelling flows and levels in open channels. ISIS is able to model complex looped and branched networks, and is designed to provide a comprehensive range of methods for simulating floodplain flows. ISIS incorporates both unsteady and steady flow solvers, with options that include simple backwaters, flow routing and full unsteady simulation. The simulation engine provides a direct steady-state solver and adaptive time-stepping methods to optimize run-time and enhance model stability.

ERA should adapt standard software program to be used in all its drainage design works. HEC-RAS should be used in all routing design except minor ditches. Based on our assessment, only a minority of drainage designers use software for hydraulic analysis. The current drainage design practice in Ethiopia is based on manual calculation in the form of spreadsheet. It is recommended that drainage engineers to adopt hydraulic analysis software in the design of road drainage systems.

The mathematical equations provided in this manual are embedded in the software program; however, some of these equations have been retained in the manual as the majority of drainage designers in Ethiopia use tradition manual calculation methods. Once the skill in the use of software programs is developed in the country, these equations should be used for information only.

6.13.3 Manning's 'n' Value Selection

Manning's 'n' is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and flood plains for which the discharge has been measured and Manning's 'n' has been calculated are very useful (see Ref. 3 and 4). Once the Manning's 'n' values have been selected, it is highly recommended that they be verified with historical high water marks and/or gauged stream flow data.

Manning's 'n' values for artificial channels are more easily defined than for natural stream channels. See Table 6-1 for typical 'n' values for both artificial and natural stream channels. Photos 6.1-6.8 following table 6-1 also illustrate various types of channels and their corresponding Manning's n.

Table 6-1: Values of Roughness Coefficient n (Uniform Flow)

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense Weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Backhoe-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2 Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
1 Minor streams (top width at flood stage < 30 m)			
a. Streams on Plain			
1. Clean, straight, full stage, no rims or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8 Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravel, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
2 Flood Plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050

Type of Channel and Description	Minimum	Normal	Maximum
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of spouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3 Major Streams (top width at flood stage > 30 m). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	--	0.060
b. Irregular and rough section	0.035	--	0.100
4 Various Open Channel Surfaces			
a. Concrete	0.012-	0.020	
b. Gravel bottom with:			
Concrete	0.020		
Mortared stone	0.023		
Riprap	0.033		
c. Natural Stream Channels			
Clean, straight stream	0.030		
Clean, winding stream	0.040		
Winding with weeds and pools	0.050		
With heavy brush and timber	0.100		
d. Flood Plains			
Pasture	0.035		
Field Crops	0.040		
Light Brush and Weeds	0.050		
Dense Brush	0.070		
Dense Trees	0.100		



Photo 6-1: n = 0.045



Photo 6-2: n = 0.035



Photo 6-3: n = 0.08



Photo 6-4: n = 0.05



Photo 6-5: n = 0.1



Photo 6-6: n = 0.07



Photo 6-7: n = 0.08



Photo 6-8: n = 0.15

6.13.4 Calibration

Equations can be calibrated to ensure that they accurately represent local channel conditions. However, the calibration process requires a large amount of data, including cross-sections, recorded water levels and flow rates. It should be considered if the failure of a facility would increase risk to life or property.

The calibration process involves varying input parameters until a good agreement exists between measured and simulated values. Hydraulic parameters which are varied include roughness coefficients and expansion and contraction coefficients. The parameter with the greatest influence on water levels is the Manning roughness coefficient.

6.13.5 Switchback Phenomenon

If the cross section is improperly subdivided, the mathematics of the Manning's Equation causes a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area which causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed which is lower

than the discharge based upon the lower water depth. More subdivisions within such cross-sections should be used in order to avoid the switchback.

This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross-section being used in a step backwater program. For this reason, the cross-section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value, itself, may be the same in adjacent subsections (refer to example, calculations for further details).

6.13.6 Single-Section Analysis Method (Slope-Area Method)

The single-section analysis method (slope-area method) is simply a solution of Manning's Equation for the normal depth of flow given the discharge and cross-section properties including geometry, slope, and roughness. It implicitly assumes the existence of steady, uniform flow, however, uniform flow rarely exists in either artificial or stream channels. Nevertheless, the single-section method is often used to design artificial channels for uniform flow as a first approximation, and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outlet.

A stage-discharge curve is a graphical relationship between stream-flow depth or elevation and discharge at a specific point on a stream. This relationship should cover a range of discharges up to at least the base (100-year) flood. The stage-discharge curve can be determined as follows:

- Select the typical cross-section at or near the location where the stage-discharge curve is needed;
- Assign a roughness coefficient (Manning's n-value) as described above;
- Estimate water-surface slope: since uniform flow is assumed, the average slope of the streambed can usually be used;
- Apply a range of incremental water surface elevations to the cross-section;
- Calculate the discharge using Manning's Equation for each incremental elevation. Total discharge at each elevation is the sum of the discharges from each subsection at that elevation; in determining hydraulic radius, the wetted perimeter shall be measured only along the solid boundary of the cross-section and not along the vertical water interface between subsections; and
- After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge shall be made, this plot is the stage-discharge curve and it can be used to determine the water-surface elevation corresponding to the design discharge or other discharge of interest.

Channels can also be sized using the charts listed as Charts 6-1 through 6-3 at the end of this chapter, for differing shapes, linings, widths, side slopes, linings, and channel slopes.

In stream channels the transverse variation of velocity in any cross section is a function of subsection geometry and roughness and may vary considerably from one stage and discharge to another. It is important to know this variation when designing erosion control measures and locating relief openings in highway fills, for example. The best method of establishing transverse velocity variations is by current meter measurements. If this is not possible, the single section method can be used, whereby the cross section is divided into subsections of relatively uniform roughness and geometry. It is assumed that the energy grade line slope is the same across the cross section so that the total conveyance K_T of the cross section is the sum of the subsection conveyances:

$$K_T = K_1 + K_2 + \cdots + K_n \quad (6\text{-}7)$$

The total discharge is then $K_t S^{1/2}$ and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation,

V = Q/A.

Alluvial channels present a more difficult problem in establishing stage-discharge relations by the single-section method because the bed itself is deformable and may generate bed forms such as ripples and dunes in lower regime flows. These bed forms are highly variable with the addition of form resistance, and selection of a value of Manning's 'n' is not straightforward. Instead, several methods (Ref. 35) have been developed for this case and shall be followed unless it is possible to obtain a measured stage-discharge relation.

There may be locations where a stage-discharge relationship has already been measured in a channel. These could exist at gauging stations on streams monitored by the Ministry of Water and Energy. Measured stage-discharge curves will generally yield more accurate estimates of water surface elevation and should take precedence over the analytical methods described above.

6.13.7 Step-Backwater Analysis

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned, and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, a computer program such as U.S. Army Corps of Engineers HEC-RAS can be used to assist with the equations.

The method requires definition of the geometry and roughness of each cross section. Manning's n-values can vary both horizontally across the section as well as vertically. Expansion and contraction head loss coefficients, variable main channel and overbank flow lengths, and the method of averaging the slope of the energy grade line can all be specified.

To develop the methodology, the energy equation is repeated from Section 6.4:

$$h_1 + \propto_1 \left(\frac{{V_1}^2}{2g}\right) = h_2 + \propto_2 \left(\frac{{V_2}^2}{2g}\right) + h_L \quad (6\text{-}8)$$

Where:

h_1, h_2 = the upstream and downstream stages, respectively, m

α = velocity distribution coefficient

V = mean velocity, m/s

h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, m

The stage 'h' is the sum of the elevation head 'z' at the channel bottom and the pressure head, or depth of flow y, i.e., h = z+y. The energy equation is solved between successive stream reaches with nearly uniform roughness, slope, and cross-sectional properties.

The total head loss is calculated from:

$$H_L = K_m \left[\frac{(\propto_1 V_1{}^2)}{2g} - \frac{(\propto_2 V_2{}^2)}{2g}\right] + S_f L \qquad (6\text{-}9)$$

Where:

K_m = expansion or contraction loss coefficient.

S_f = mean slope of the energy grade line evaluated from Manning's equation and a selected averaging technique m/m

L = discharge-weighted or conveyance-weighted reach length, m

These equations are solved numerically in a systematic procedure called the Standard Step Method from one cross section to the next.

Water surface profile computation requires a beginning value for elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section, but in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can be found either by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the profiles do not converge, then the stream reach may need to be extended downstream, a shorter cross-section interval shall be used, or the range of starting water surface elevations shall be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure ***6.21).***

Given a sufficiently long stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the structure site, and continued a sufficient distance upstream to determine accurately the impact of the structure on upstream water surface profiles(refer to chapter 4 how to define limits of data collection). Refer to Figure 6-21.

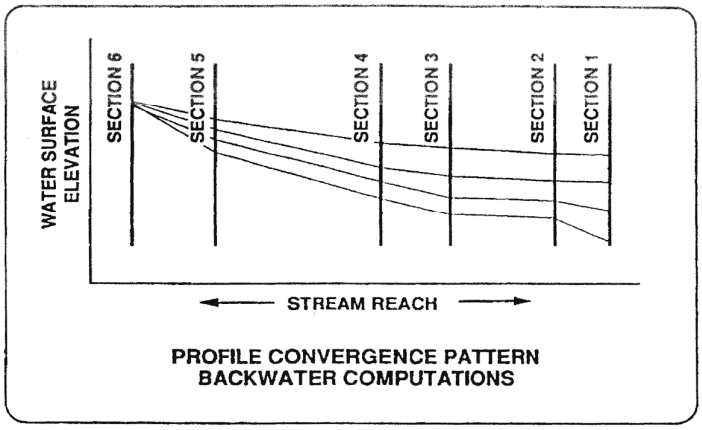


Figure 6-21: Profile Convergence Pattern Backwater Computation

6.13.8 Water and Sediment Routing

Water and sediment routing methods should be employed where possible scour and/or sediment are of concern. It is generally not used at stable stream locations. Various computer models can be employed to investigate water and sediment routing. The latest version of HEC-RAS has a sediment transport-modeling module. The scour or deposits in each stream tube, determined by sediment routing, will give the variation of channel geometry in the vertical direction.

6.14 Channel Design Procedure

The design procedures for all types of channels have many similar elements; however each type of channel will require unique inputs. This section will:

- Outline a process for assessing a natural stream channel: and
- Offer a more specific design procedure for roadside channels.

6.14.1 Stream Channels

The analysis of a stream channel, in most cases, takes place in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway in such a manner that will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of studies necessary shall be commensurate with the risk associated with the action and the environmental sensitivity of the stream and adjoining flood plain.

Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a process that, in general, will be applicable.

Step 1 Assemble Site Data and Project File (Consult Chapter 4 for more details)

A. Data Collection
- Topographic, site, and location maps
- Roadway profile
- Photographs
- Field reviews
- Design data at nearby structures
- Gauging records.

B. Studies by other agencies
- Ministry of Water and Energy studies
- Ministry of Agriculture studies
- Floodplain studies
- Catchment area studies
- River Basin Master Plan studies.

C. Environmental constraints
- Floodplain encroachment
- Floodway designation
- Fish habitat
- Commitments in review documents.

D. Design criteria
- See Section 6.11.

Step 2 Determine the Project Scope

A. Determine level of assessment
- Stability of existing channel
- Potential for damage
- Sensitivity of the stream.

B. Determine type of hydraulic analysis
- Qualitative assessment
- Single-section analysis
- Step-backwater analysis.

C. Determine additional survey information
- Extent of streambed profiles
- Locations of cross sections
- Elevations of flood-prone property (threshold level)
- Details of existing structures
- Properties of bed and bank materials.

Step 3 Evaluate Hydrologic Variables

A. Compute discharges for selected frequencies
B. Consult Hydrology, Chapter 5.

Step 4 Perform Hydraulic Analysis

A. Single-section analysis (6.14.6)
- Select representative cross section (6.14.6)
- Select appropriate n values (Table 6-1)
- Compute stage-discharge relationship.

B. Step-backwater analysis (6.14.6)
C. Calibrate with known high water marks.

Step 5 Perform Stability Analysis

A. Geomorphic factors
B. Hydraulic factors
C. Stream response to change.

Step 6 Design Countermeasures

A. Criteria for selection
- Erosion mechanism
- Stream characteristics
- Construction and maintenance requirements
- Vandalism considerations
- Cost.

B. Types of countermeasures
- Meander migration countermeasures
- Bank stabilization
- Bend control countermeasures
- Channel braiding countermeasures
- Degradation countermeasures
- Aggradation countermeasures.

C. For additional information
- HEC-20 Stream Stability
- Highways in the River Environment
- See Reference List.

Step 7 Documentation

- Prepare report and file with background information.

6.14.2 Roadside Channels

Each project is unique, but the following six basic design steps are normally applicable:

Step 1 Establish a Roadside Plan

A. Collect available site data
B. Obtain or prepare existing and proposed plan-profile layout including highway, culverts, bridges, etc.
C. Determine and plot on the plan the locations of natural basin divides and roadside channel outlets. An example of a roadside channel plan/profile is shown in Figure 6.9; and
D. Perform the layout of the proposed roadside channels to minimize diversion flow lengths.

Step 2 Obtain or Establish Cross Section Data

A. Provide channel depth adequate to drain the sub-base
B. Choose channel side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access;
C. Establish cross sectional area required and determine appropriate ditch shape and size;
D. Identify features that may restrict cross section design:
- Right-of-way limits, trees or environmentally-sensitive areas;
- Utilities, and existing drainage facilities.

Step 3 Determine initial channel grades

A. Plot initial grades on plan-profile layout (Slopes in roadside ditch in cuts are usually controlled by highway grades)
B. Provide minimum grade of 0.3 percent. Note that this gradient does not necessarily need to be equal to the roadway gradient.
C. Consider influence of type of lining on grade
D. Where possible, avoid features that may influence or restrict grade, such as utility locations

Step 4 Check flow capacities and adjust as necessary

A. Compute the design discharge at the downstream end of a channel segment (see Hydrology, Chapter 5)
B. Set preliminary values of channel size, roughness coefficient, and slope
C. Determine maximum allowable depth of channel including freeboard
D. Check flow capacity using Manning's Equation and single section analysis
E. If capacity is inadequate, possible adjustments are as follows:
   - Increase bottom width;
   - Make channel side slopes flatter;
   - Make channel slope steeper;
   - Provide smoother channel lining; and
   - Install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity.
F. Provide smooth transitions at changes in channel cross sections
G. Provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge.

Step 5 Determine channel lining/protection needed (HEC-15)

A. Select a lining and determine the permissible shear stress τ_p in Pascals (N/m^2) from Table 6-2 and/or Table 6-3
B. Estimate the flow depth and choose an initial Manning's n value from Table 6-4
C. Calculate normal flow depth yo (m) at design discharge using Manning's Equation and compare with the estimated depth. If they do not agree, repeat Steps 5B and 5C
D. Compute maximum shear stress at normal depth as:
   τ_d (Pa) = 2990 y_o S where S = channel slope, m/m
E. If $\tau_d < \tau_p$ then lining is acceptable, otherwise consider the following options:
   - Choose a more resistant lining,
   - Use concrete, gabions, or other more rigid lining either as full lining or composite,
   - Decrease channel slope,
   - Decrease slope in combination with drop structures, and/or
   - Increase channel width and/or flatten side slopes.

Step 6 Analyze outlet points and downstream effects

A. Identify any adverse impacts to downstream properties that may result from one of the following at the channel outlet:
   - Increase or decrease in discharge,
   - Increase in velocity of flow,
   - Confinement of sheet flow,
   - Change in outlet water quality, or
   - Diversion of flow from another catchment area

B. Mitigate any adverse impacts identified in 6A, possibilities include:

- Enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel,
- Install velocity control structures,
- Increase capacity and/or improve lining of downstream channel,
- Install sedimentation/infiltration basins,
- Install sophisticated weirs or other outlet devices to redistribute concentrated channel flow, and
- Eliminate diversions that result in downstream damage and which cannot be mitigated in a less expensive manner.

6.15 Stream Morphology

The three-dimensional form assumed by a natural stream is a function of many variables for which cause-and-effect relationships are difficult to establish. The stream may discharge the same amount of sediment that it receives although there may be short-term adjustments in its bed forms in response to flood flows. On the other hand, the stream reach may be aggrading or degrading as a result of deposits or scour in the reach. The plan-form of the stream may be straight, braided, or meandering. The complexities of stream morphology can be assessed by inspecting aerial photographs and topographic maps for changes in slope, width, depth, meander form, and bank erosion with time.

A qualitative assessment of the river response to proposed highway facilities is possible through a thorough knowledge of river mechanics and accumulation of engineering experience.

Table 6-2: Classification of Vegetal Covers as to Degrees of Retardancy

Retardance	Cover	Condition
A	Native grass	Excellent stand, tall >750 mm
B	Native grass	Good stand, tall (average 300 – 600 mm)
C	Native grass	Good stand, uncut 150 – 300 mm
D	Native grass	Good stand, uncut 50 – 150 mm
E	Native grass	Good stand, cut to 40 mm, Stubble

Table 6-3: Summary of Shear Stress for Various Protection Measures

Protective Cover	Category	τ_p (Pa)
Vegetation	Class A Class B Class C Class D Class E	177 101 48 29 17
Temporary	Woven Paper Jute Net Straw W/Net Curled Wood Mat Synthetic Mat	7 22 69 74 96
Gravel: D_{50} = 25 mm D_{50} = 50 mm		19 38
Rock: D_{50} = 150 mm D_{50} = 300 mm		120 239
Gabions		1676
Geoweb		479
Soil Cement	(8%)	>2155
Concrete construction blocks, granular filter under layer		>958
Wedge-shaped blocks with drainage slot		>1197

Table 6-4: Manning's Roughness Coefficients (HEC-15)

		n – value : Depth Ranges		
Lining Category	**Lining Type**	**0-0.15m**	**0.15–0.06m**	**> 0.6m**
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	25 mm D_{50}	0.044	0.033	0.030
	50 mm D_{50}	0.066	0.041	0.034
Rock Riprap	150 mm D_{50}	0.104	0.069	0.035
	300 mm D_{50}	--	0.078	0.040

Note: Values listed are representative values for the respective coefficients, n, vary with the flow depth.

*Some "temporary" linings become permanent when buried.

The natural stream channel will assume a geomorphologic form that will be compatible with the sediment load and discharge history that it has experienced over time. To the extent that a highway structure disturbs this delicate balance by encroaching on the natural channel, the consequences of flooding, erosion, and deposits can be significant and widespread. The hydraulic analysis of a proposed highway structure should include a consideration of the extent of these consequences.

6.15.1 Stream Response to Change

The major complicating factors in river mechanics are: 1) the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a stream system; and 2) the continual evolution of stream channel patterns, channel geometry, bars, and forms of bed roughness with changing water and sediment discharge. In order to understand better the responses of streams to the actions of man and nature, a few simple hydraulic and geomorphic concepts are presented.

Any natural or artificial change that alters channel slope can result in modifications to the existing stream pattern. For example, a cutoff of a meander loop decreases channel sinuosity and increases channel slope. Conversely, it is possible that a slight decrease in slope could change an unstable braided stream into a meandering one.

The different channel dimensions, shapes, and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, a change in hydrology may cause changes in stream sinuosity, meander wavelength, and channel width and depth. A long period of channel instability with considerable bank erosion and lateral

shifting of the channel may be required for the stream to compensate for the hydrologic change.

6.15.2 Countermeasure

A countermeasure is defined as a measure incorporated into the crossing of a stream to control, inhibit, change, delay, or minimize stream and bride stability problems. Countermeasures may be installed at the time of highway construction or retrofitted to resolve stability problems at existing crossings.

The practice of retrofitting makes good economic sense and in many locations it is also good engineering practice. This is because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop.

The selection of an appropriate countermeasure for a specific bank erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs. Below is a brief discussion of possible countermeasures for some common river stability problems.

Note: The reader is encouraged to consult with the references listed at the end of this chapter for detailed information on the design and construction of countermeasures.

The best countermeasure against meander migration is a crossing location on a relatively straight reach of stream between bends. Other countermeasures include the protection of an existing bank line, the establishment of a new flow line or alignment, and the control and constriction of channel flow. Countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardant structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations. Measures may be used individually or in combination to combat meander migration at a site (Refs. 30 and 21).

Countermeasures used at a braided stream are usually intended to confine the multiple channels to one channel. This tends to increase sediment transport capacity in the principal channel and encourage deposits in secondary channels.

The measures usually consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. These include spur dikes at bridge ends used in combination with revetments on highway fill slopes; rip rap only on highway fill slopes; and spurs arranged in the stream channels to constrict flow to one channel.

6.15.3 Degradation and Aggradation

Degradation in streams can cause the loss of bridge piers in stream channels, as well as piers and abutments in caving banks. A check dam, which is a mini dam or weir constructed across a channel, is one of the most successful techniques for halting degradation on small to medium streams.

Longitudinal stone dikes placed at the toe of channel banks can be effective countermeasures for bank caving in degrading streams. Precautions to prevent outflanking such as tiebacks to the banks, may be necessary where installations are limited to the vicinity of the highway stream crossing. In general, channel lining alone is not a successful countermeasure against degradation problems (Ref. 21).

Current measures in use to alleviate aggradation problems at highways include channelization, bridge modification, continued maintenance, or any combination of these. Channelization may include excavating and cleaning channels, constructing cutoffs to

increase the local slope, constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradation.

6.16 Design of Outfalls for Surface Water Channels

Surface water channels for drainage of run-off from highways can be a suitable alternative to conventional kerbs and gullies or filter drains. Amongst other advantages, such as providing separate systems for drainage of surface and sub-surface water, they allow greater distances between outlets when compared with conventional gully systems.

This section describes suitable layouts for outlets from triangular and trapezoidal surface water channels and provides methods of designing each type according to the flow rate in the channel.

The design methods enable the performance of the outlets to be assessed for channel-full conditions and for surcharging conditions when the flow may extend to the edge of the carriageway. The channel-full conditions are normally specified to correspond to storms with a return period of 1 year whereas the surcharged situation typically refers to storms with a return period of 5 years. It should be noted that surcharging is not allowed for channels built in the central reserve.

The design methods apply to symmetrical triangular channels with cross-falls of 1:5 and also channels with a trapezoidal cross-section and cross-falls of 1:4.5 or 1:5. High capacity channels are required for drainage of wide roads and long lengths with flat gradients. In such situations, trapezoidal cross-sections provide higher capabilities than triangular channels of the same depth and surface width. The trapezoidal channels considered have a base width equal to twice the channel-full depth. In order to promote self- cleansing conditions, the base of the channel has a cross-fall of 1:40 towards the verge (or central reserve).

The channel shape can be modified at the outlet to accommodate gratings in the invert by steepening the sides of the channel locally to slopes not exceeding the allowable limit of 1:4. Figures B_1 and B_2 in Appendix A show the cross-sectional shapes of the recommended channels. As shown in these figures, y1 is the depth of the channel from the lower edge of the carriageway, y_2 is the depth of the channel from the upper edge of the carriageway, and y3 is the overall depth of the surcharged channel.

Three alternative geometries of outlet are recommended. One is an in-line outlet, where the water is essentially collected symmetrically either side of the channel invert. Another type is an off-line outlet, where the channel is widened away from the carriageway and the outlet is off-set from the centreline of the channel.

A third type of outlet, a weir outlet, is recommended for steep slopes (typically >1:5) where the water is made to curve towards a side-weir.

6.16.1 Flow Conditions Approaching Outlet

The flow rate to use in the design of the outlets should be calculated using Manning's resistance equation:

$$Q = \frac{A\,R^{2/3}\,S^{1/2}}{n}$$

where Q is the flow rate (m^3/s); A is the cross-sectional area of the flow (m^2), S is the longitudinal gradient of the channel (m/m); and n is the Manning roughness coefficient.

The hydraulic radius R is defined by:

$$R = \frac{A}{P}$$

where P is the wetted perimeter, ie the perimeter of the channel in contact with the water flow. If the longitudinal gradient of the channel is not uniform along its length, an equivalent value of the slope, S, should be used in the calculation of the flow rate.

When checking for surcharged conditions, the flow rate, Q_s , to use in the design of outlets can be estimated from Figure B_3 for triangular channels and Figure B4 for trapezoidal channels. In these Figures B_d and Q_d are respectively the surface width of the flow and the discharge corresponding to the design capacity of the channel. Q_d is equal to the value of Q given by Manning's Equation when A and R corresponds to the design depth of flow, y_1, in the channel (measured from the invert centerline to the lower edge of the carriageway). The curves in Figures B_3 and B_4 in Appendix 6A are based on 1m width of surcharging of the carriageway at cross-falls of 1:30, 1:40 and 1:60. The value of Q_s /Q_d can be read off the curves and, with Q_s calculated using Manning's Equation, the value of Q_d can then be determined.

6.16.2 Types of Outlets

Channels outlets can be defined as intermediate or terminal according to their position along a channel. Terminal outlets are located at low points along a length of channel and should be designed to collect all the flow carried by the channel. Intermediate outlets are located at points part-way along a length of channel where the flow rate of water from the road reaches the carrying capacity of the channel.

The design methods in this section are based on a minimum value of the waterway area (defined as the total area of openings) in relation to the plan area of the grating. If G is the width of the grating, the minimum waterway area needed to produce the required hydraulic performance is $0.44G^2$. The efficiencies of outlets comprising grating with bigger waterway areas and similar bar patterns will not be less that given by these methods.

Alternative designs of in-line and off-line outlets are recommended for each of the two types of channels (triangular and rectangular). For triangular channels the inline outlet recommeneded is generally more efficient than the off-line outlet but resason for chusing between them will mainly depend on construction aspects.

Other aspects being equal, in-line outlets are poreferable to off-line outlets since they require a smaller land take. However, in-line and off-line outlets are not suitable for steep channels where the high kinetic energy of the flow renders gratings less effcetive. In such situations the flow should be collected by curving it towards an off-line wier.

Triangular Channels

The in-line outlet geometry recommended for this type of cahnnel consisits of pairs of gratings positioned on the side slopes of the channel (see Figure B5). The number of pairs of gratings reqiured will depend on the amount of flow in the channel. More than three pairs of gratings are likely to be uneconomical, and other measures should be taken to cope with higher flows.

The spacing between pairs of gratings should not be less than 1.7G, where G is the width of the gratings (see Figure B4, Appendix 6A). The size of the required gratings should be chosen so that the ratio of the width G over the depth of the channel y_1, is within the following limits:

$$1.5 \frac{G}{y_1} \leq 5.1$$

The lower limit corresponds to the minimum width of grating necessary to achieve the performance specified. The upper limit corresponds to the widest grating that can be installed in the channel. The required length H of each grating is given by:

$$H \geq G$$

The lower edge of each grating should be set as close as possible to the invert of the channel in order to maximize flow interception, ie distance in Figure B5 should be minimized. A design of in-line outlet with gratings set flat in the channel invert is not included because the limit with maximum cross fails of 1:4 would allow the use of only small gratings with inefficient flow capacity.

The recommended geometry for off-line outlets is shown in Figure B6. The number of gratings may vary from one to three depending on the amount of flow approaching the outlet. However, outlets formed by a single grating may have the disadvantage of being easily blocked by debris, particularly when the outlets are widely spaced than 1.25G where G is the width of the gratings. The size of the gratings is determined by:

$$\frac{G}{y_1} \geq 4.5$$

Trapezoidal Channels

The in-line outlet geometries recommended for trapezoidal channels are shown in Figures B7 and B9, Appendix 6A. The width of the gratings is determined by:

$$\frac{G}{y_1} = 3.0$$

The off-line geometries recommended are shown in Figure B8 and B10, Appendix 6A. The width of the gratings is determined by:

$$\frac{G}{y_1} \geq 4.0$$

Terminal Outlets

The requirement that surface water channels should not have any sides steeper than 1:4 applies also to the geometry of terminal outlets. When not protected by a safety barrier, surface water channels must therefore terminate with a smooth transition, without abrupt changes in level or width. Examples of recommended terminal outlets are shown in dashed lines in Figures B5 to B10, Appendix 6A. The terminal ramps should be built at a certain

minimum distance from the grating furthest downstream. This reduces the probability of blockage of the gratings by debris since some of the debris will tend to accumulate in the area between the gratings and the terminal ramp. For in-line and off-line outlets in triangular channels, this distance should equal the grating width. For in-line and off-line outlets in trapezoidal channels, the recommended distances are given in terms of the grating width, G, and are shown in Figures B7 to B10.

6.16.3 General Recommendations on Design of Outfall Structures

An outfall conveys water from one or more outlets in surface water channel to a suitable discharge point. The design of an outfall may vary considerably depending on the general topography and nature of the ground, the layout of the road scheme and weather the water is discharged to a watercourse or below –ground pipe system. A chamber or gully pot should be located below or immediately adjacent to each outlet to collect sediment carried with the flow from the surface water channel. Standard circular gully pots have a limited hydraulic capacity and it is recommended that they should not be used for flow rates exceeding 5 l/s unless their suitability has been determined by test. The plan shape of the chamber will be determined by the layout of the gratings forming the outlet. The invert of the outgoing pipe from the chamber should be set a minimum of 300mm above the bottom of the chamber to retain an adequate volume of sediment.

The invert level of the outgoing pipe should be chosen so that the water level in the chamber does not raise high enough to prevent flow discharging freely from the surface water channel into the outlet. For design, it is recommended that the water level in the chamber should be at least 150mm below the underside of the gratings when the outlet is receiving flow from the channel under surcharged condition. The height Z (in m), of the water surface in the chamber above the invert of the outgoing pipe can be estimated from the equation:

$$Z = \frac{D}{2} + 0.23\,\frac{Q^2}{D^4}$$

where D is the diameter of the pipe (in m) and Q is the flow rate (in m^3/s) in the chamber corresponding to surcharge conditions in the surface water channel. The gradient and diameter of the outgoing pipe should be determined from standard flow tables or resistance equation so that the pipe is just flowing full under surcharged conditions.

Provided the chamber below the outlet is designed to trap sediment, the outgoing pipe from the chamber may be connected directly to a collector pipe by means of a 45^o Y junction without the need for a manhole at the junction. If weir outlet is used, the collecting channel into which flow drops from the weir should be deep enough to allow the outlet to discharge freely when the surface water channel is flowing under surcharged conditions. The design flow depth, J (in m), can be estimated from the equation:

$$J = 4.82\left(\frac{E^4 Q^4}{A^5}\right)$$

Where E is the top width of flow (in m) and Q is design rate of flow (in m^3/s). The overall depth of the channel is obtained by adding 0.15m to the value of J[14]. The top width of the channel should not be less than 0.5m. It is recommended that the collecting channel below a weir outlet should discharge into a chamber with a removable cover in order to sill the flow and allow sediment to be collected.

6.17 References

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APPENDIX 6A - TYPICAL CHANNEL DETAILS

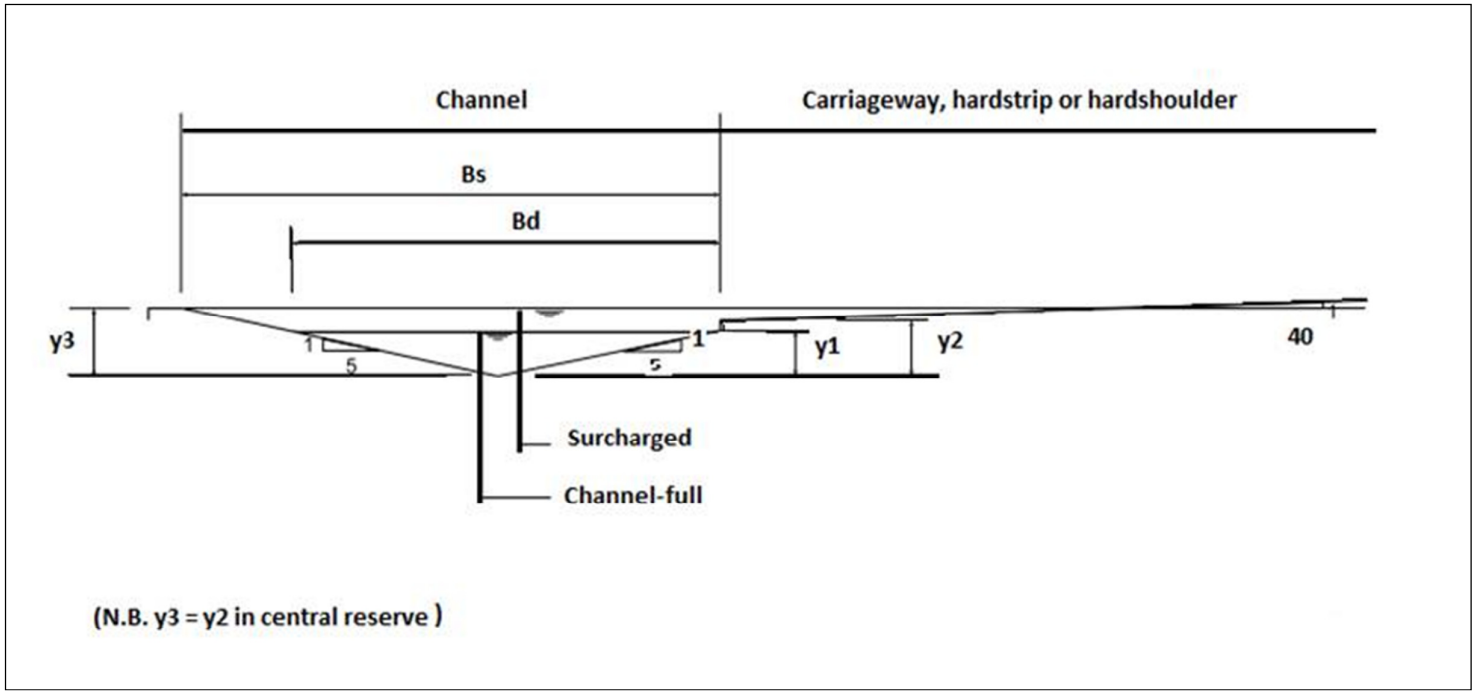


Figure: 6A-1 Cross-sectional shape of triangular channel

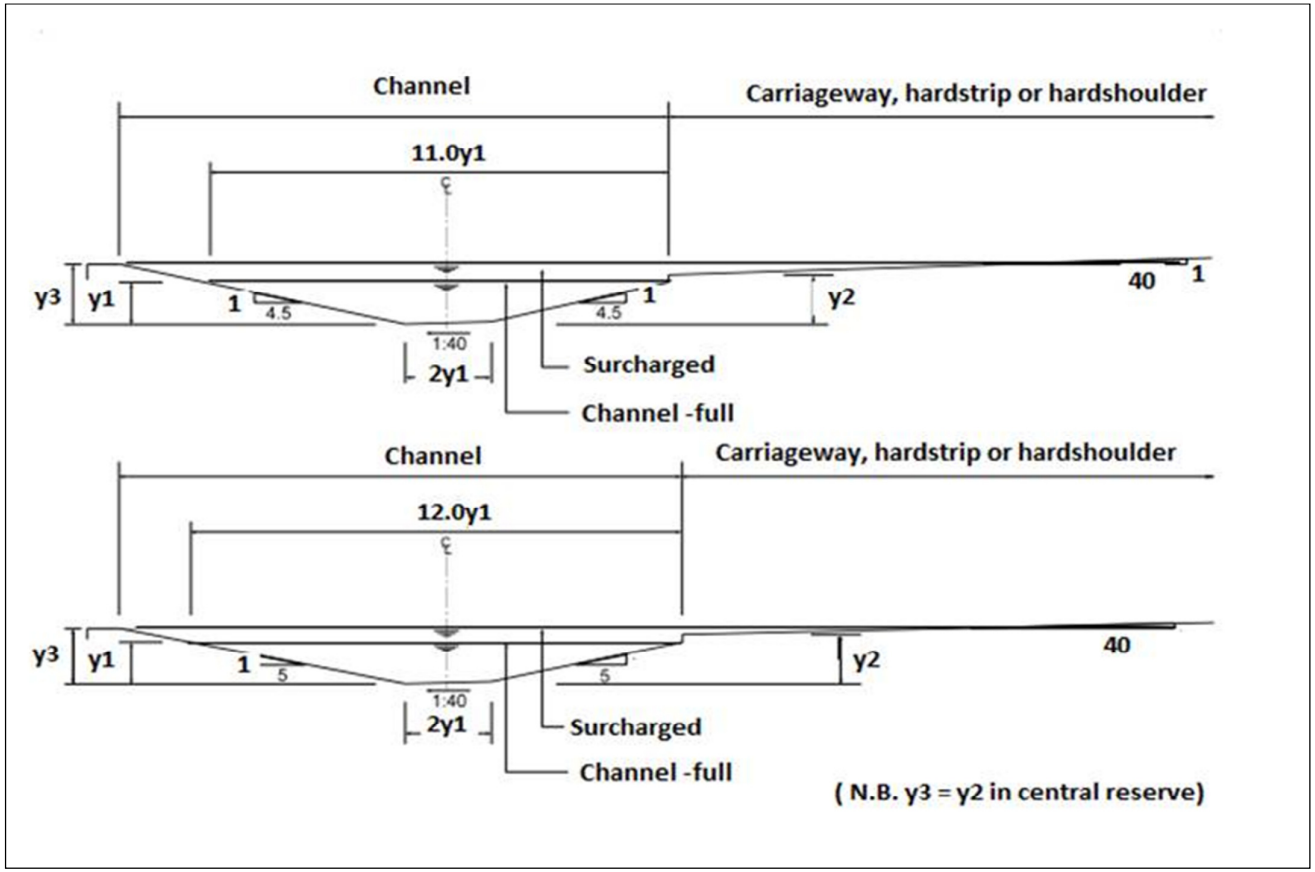


Figure 6A-2: Cross-sectional shape of trapezoidal channels

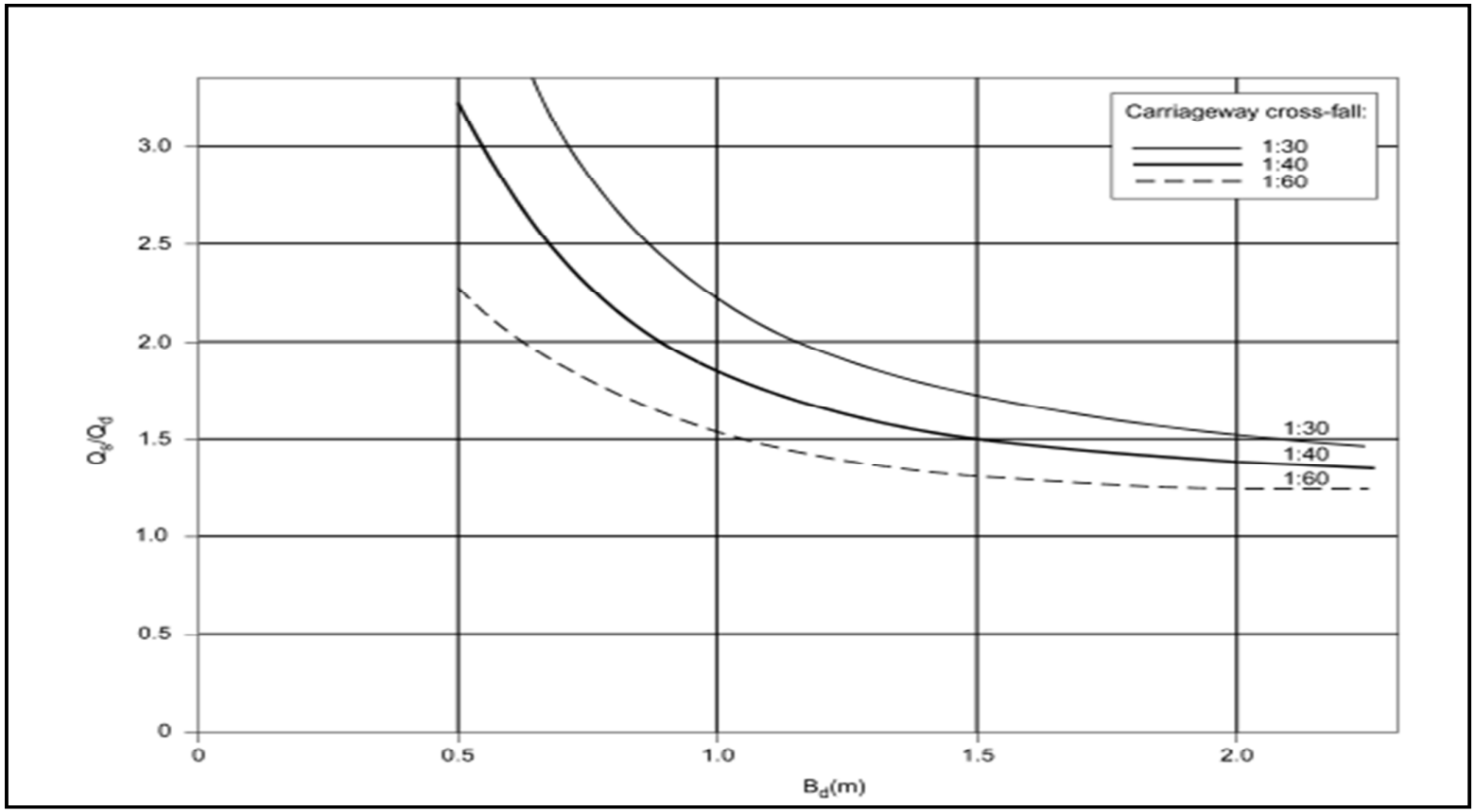


Figure 6A-3: Relationship between surcharged and channel-full flows: Trapezoidal channels

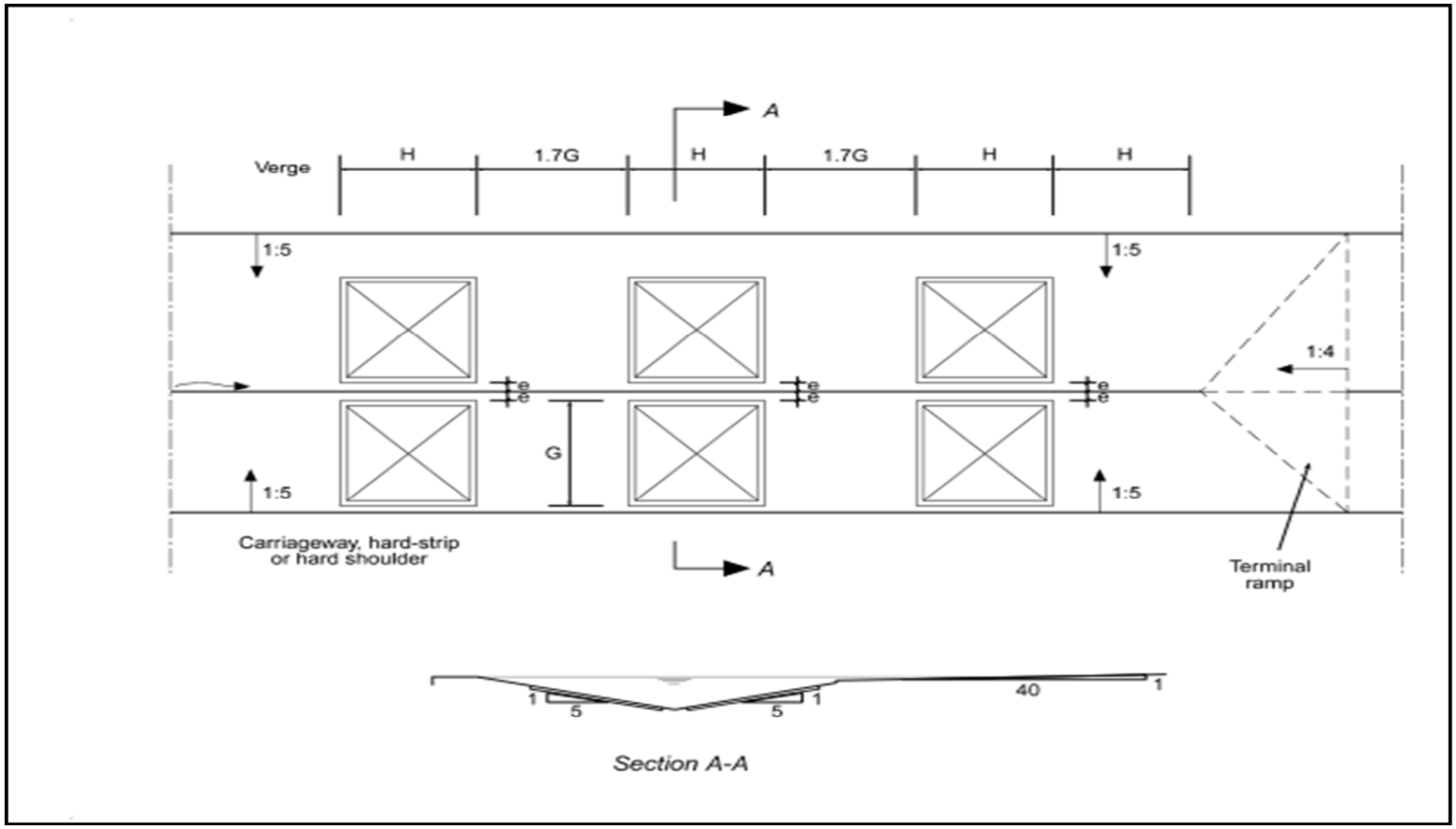


Figure 6A-4: Triangular channel In-line outlet

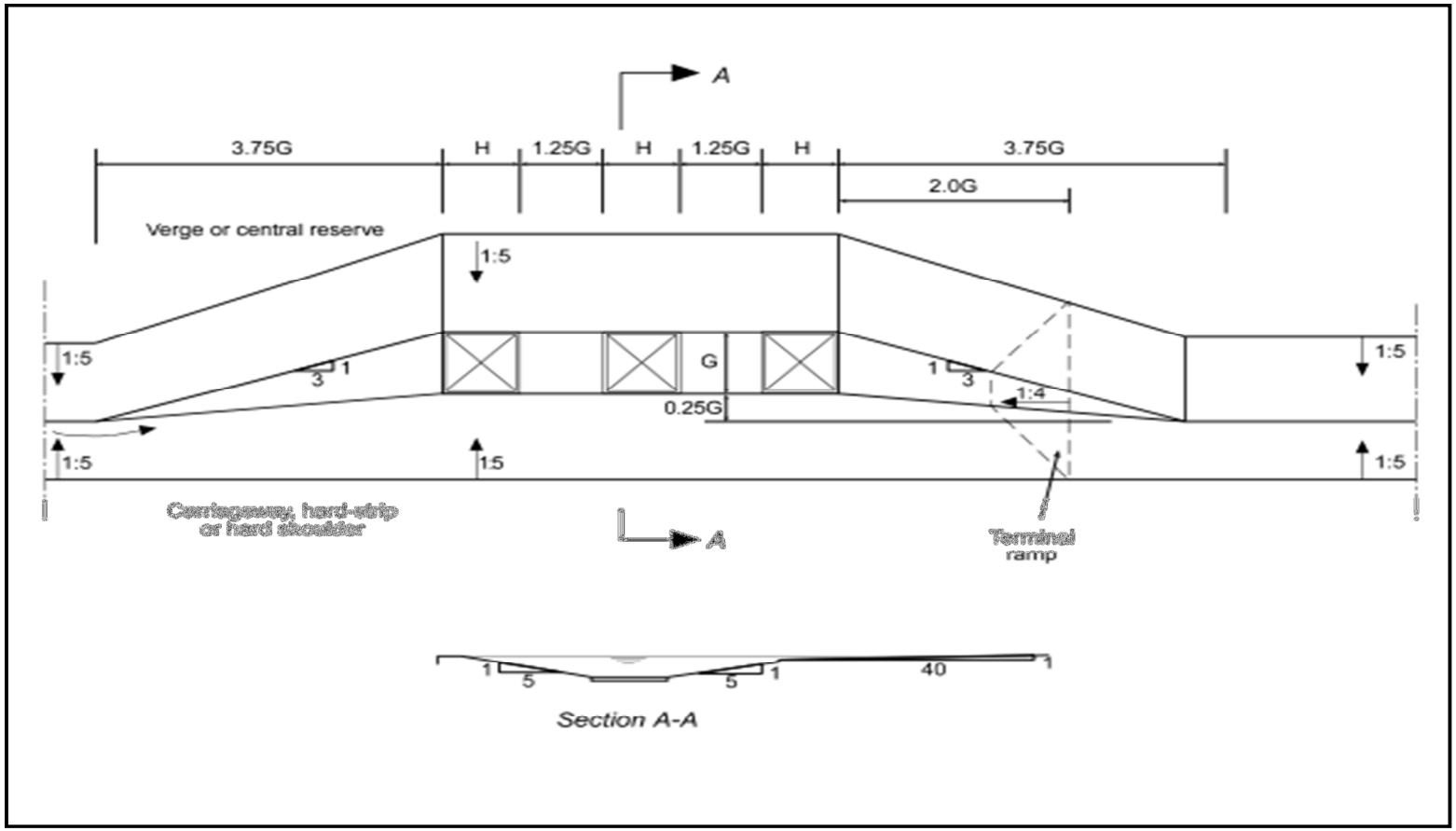


Figure 6A-5: Trpezoidal Channel Off-line outlets

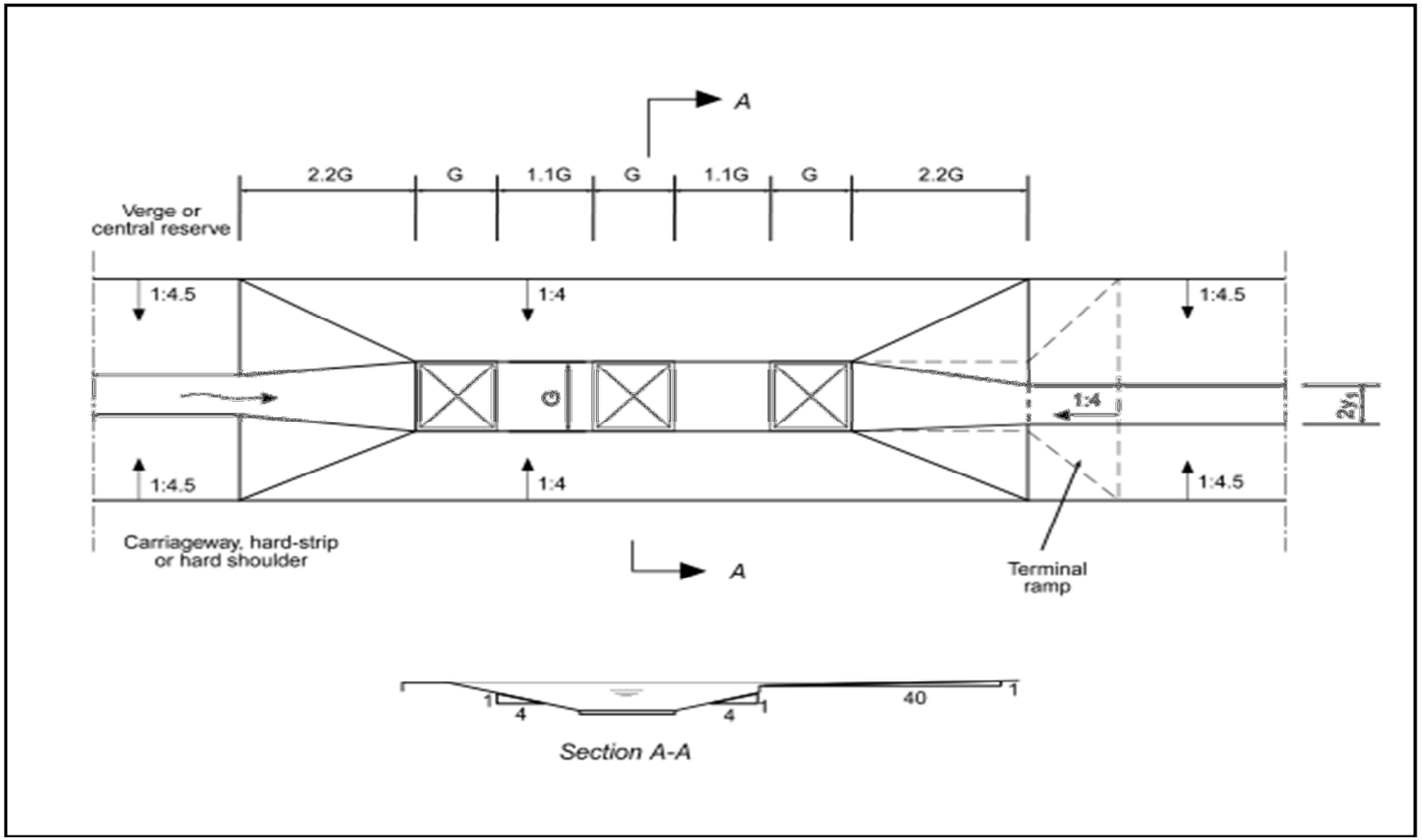


Figure 6A-6: Trapezoidal channel with cross-falls of 1:4.5 in-line outlet

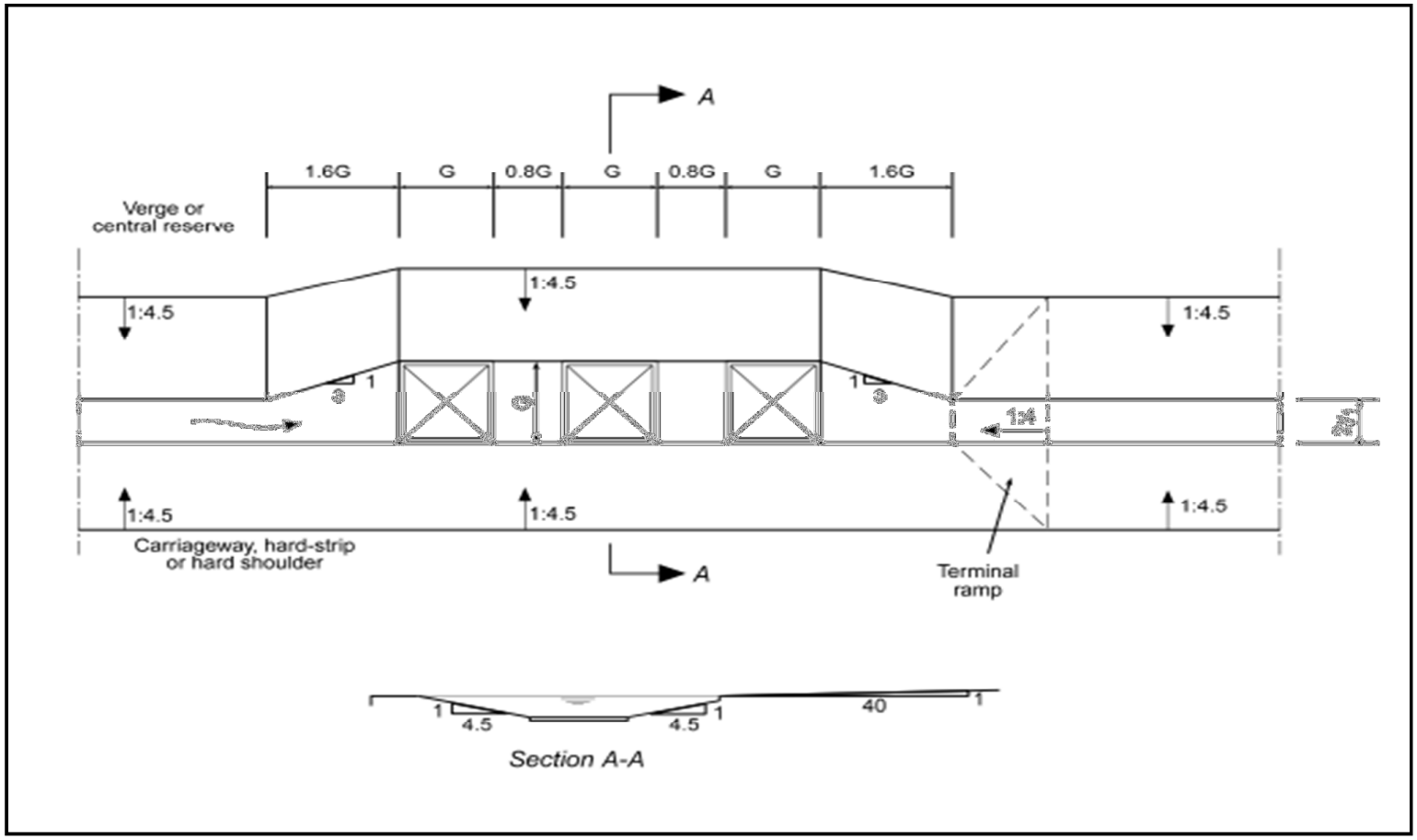


Figure 6A-7: Trapezoidal channel with cross-falls of 1:4.5 - off-line outlet

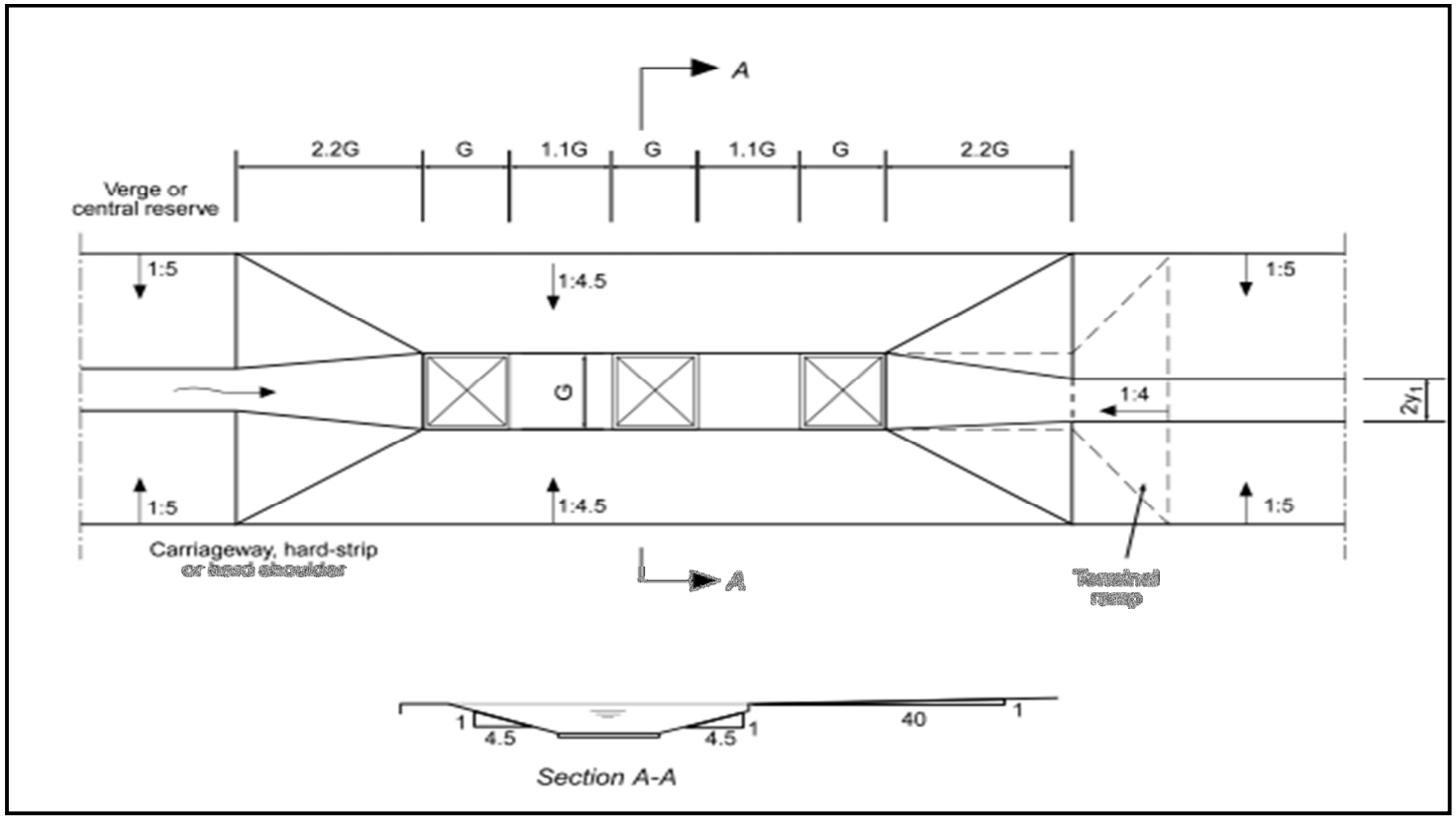


Figure 6A-8: Trapezoidal channel with cross-falls of 1:5 in-line outlet

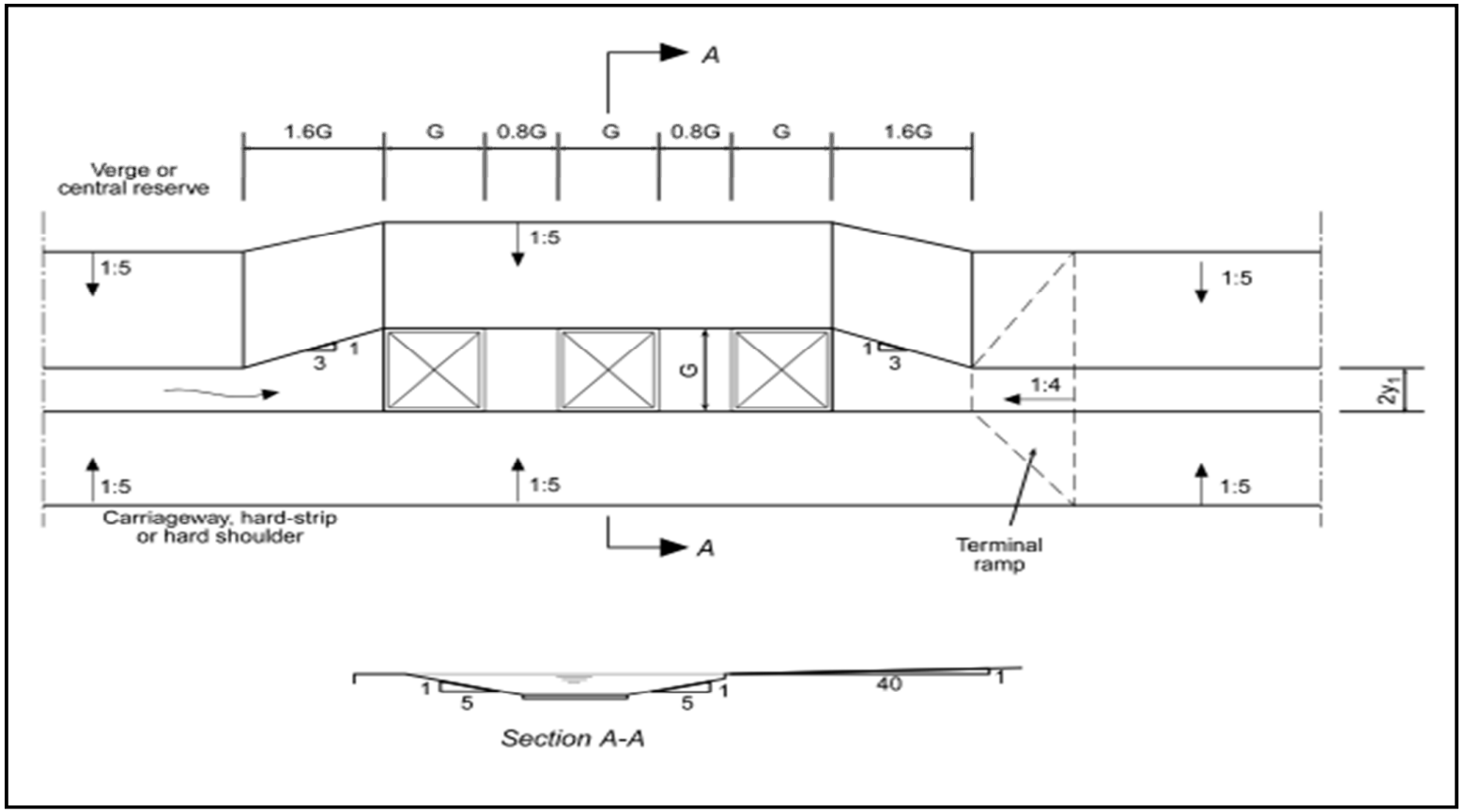
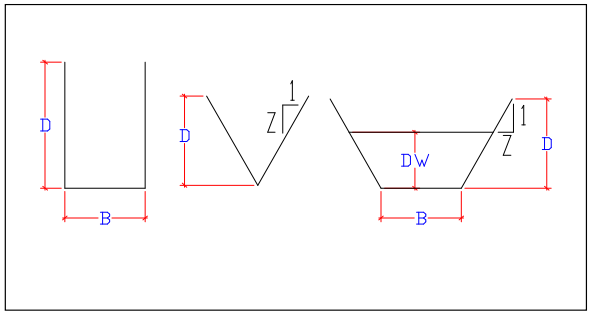


Figure 6A-9: Trapezoidal channel with cross-falls of 1:5 - off-line outlet

CURVE	URBAN DITCH FLOW CAPACITY							
			SECTION DIMENSIONS					
#	LINING	n	shape	B	Z	D	K	Scrit
	grass							
U1	rip rap	0,030	triangular	0	1	0,45	0,9	0,0253
U2			rect	0,5	0	0,45	1,5	0,0396
U3				0,5	0	0,75	2,9	0,0514
U4				1	0	0,75	3,9	0,0252
U5	concrete	0,015	rect	0,5	0	0,75	5,8	0,0129
U6		0,015		1	0	0,75	7,7	0,0063



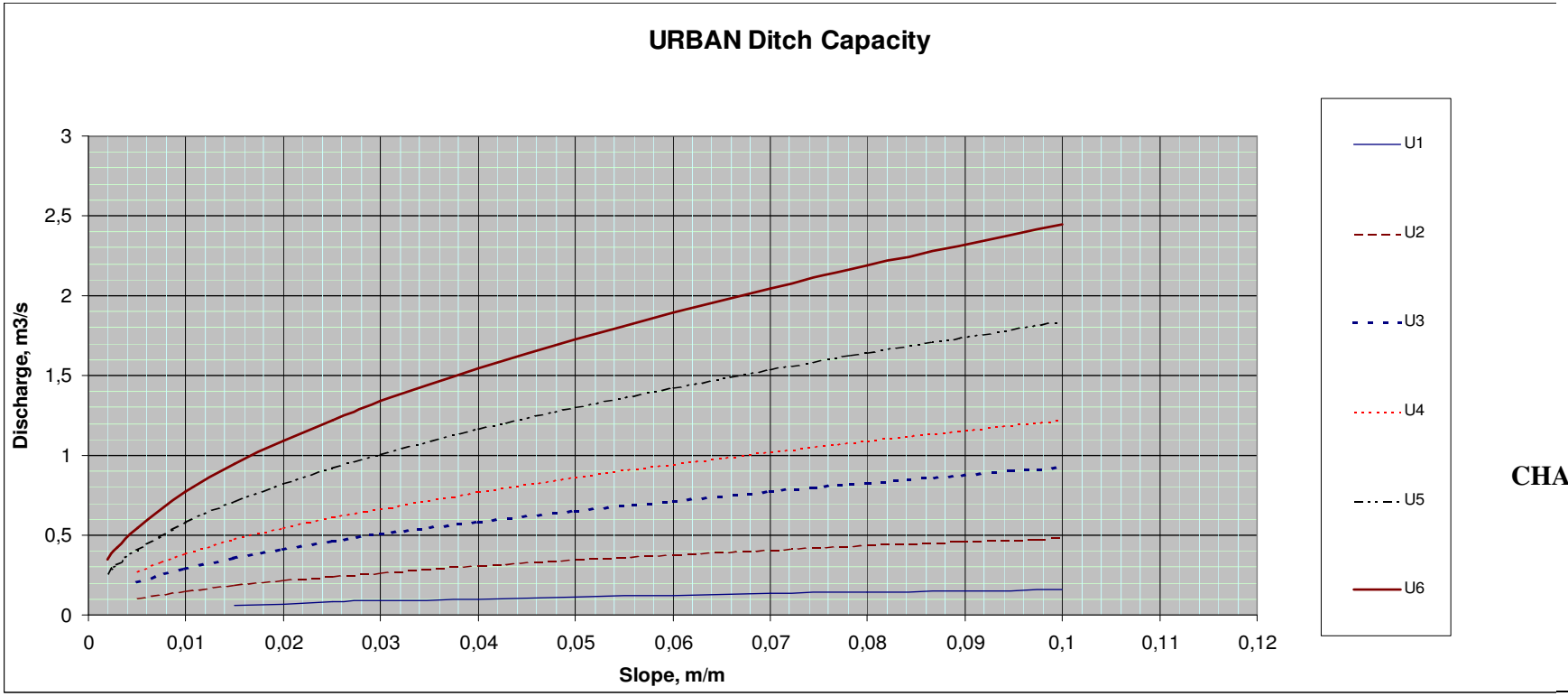
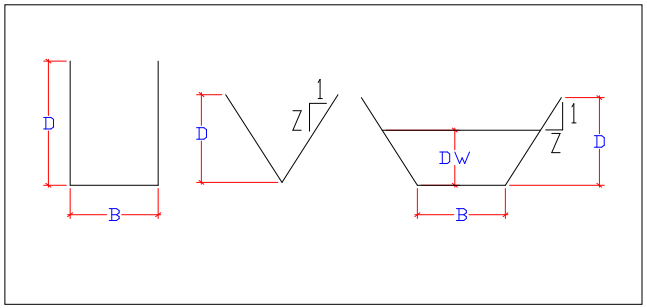


CHART 6-1

DITCH FLOW CAPACITY								
curve		RURAL						
#	LINING	n	Shape	B	Z	D	K	Scrit
R1	rip rap	0,030	triangular	0,00	1,5	0,75	6,0	0,0172
R2		0,030		0,00	1,5	0,90	9,8	0,0162
R3		0,030		0,00	1,5	0,90	13,7	0,0147
R4		0,030	trapezoidal	0,50	1,5	0,90	16,8	0,0146
R5		0,030		0,50	2,0	0,90	20,9	0,0137
R6		0,030		1,00	1,5	0,90	24,4	0,0136
R7		0,030		1,00	2,0	0,90	28,5	0,0131
C1	concrete	0,015	trapezoidal	0,50	1,5	0,90	33,7	0,0036
C2		0,015		0,50	2,0	0,90	41,7	0,0034
C3		0,015		1,00	1,5	0,90	48,8	0,0034
C4		0,015		1,00	2,0	0,90	56,9	0,0033



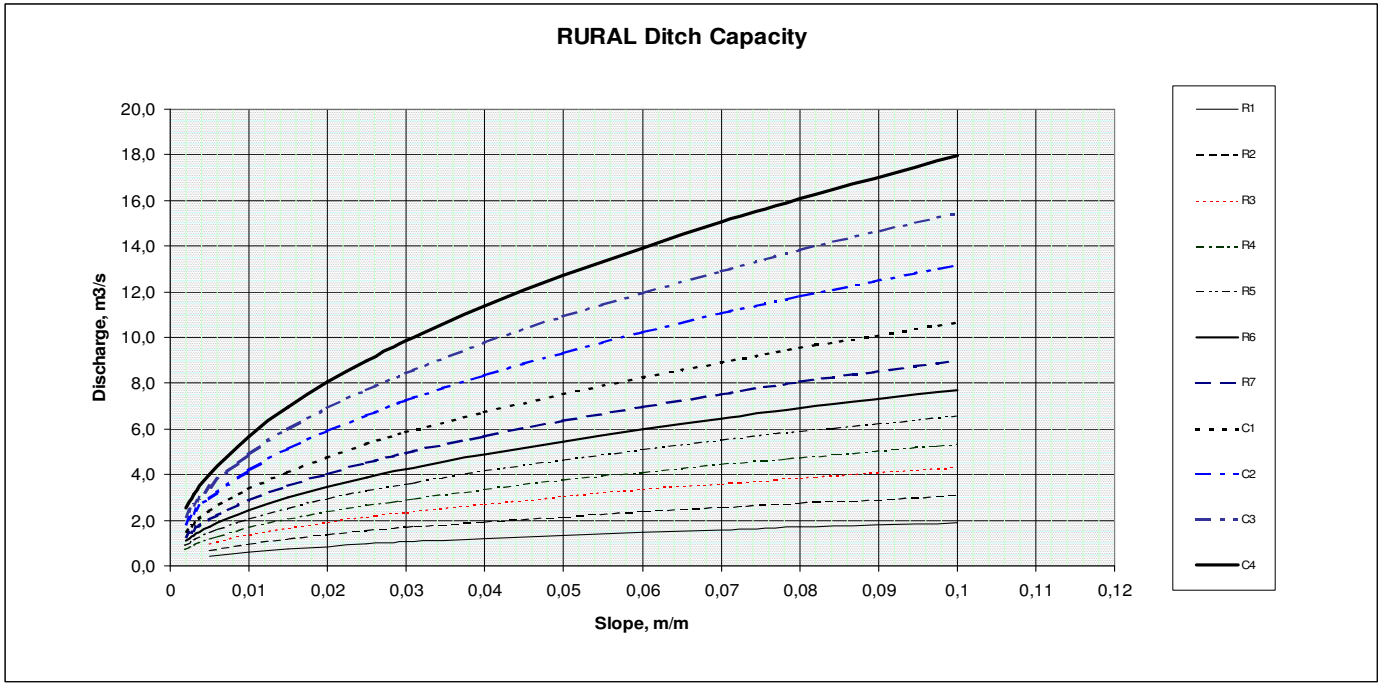
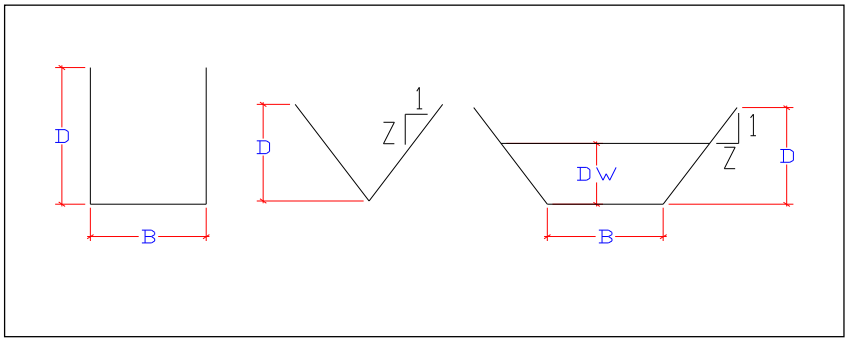
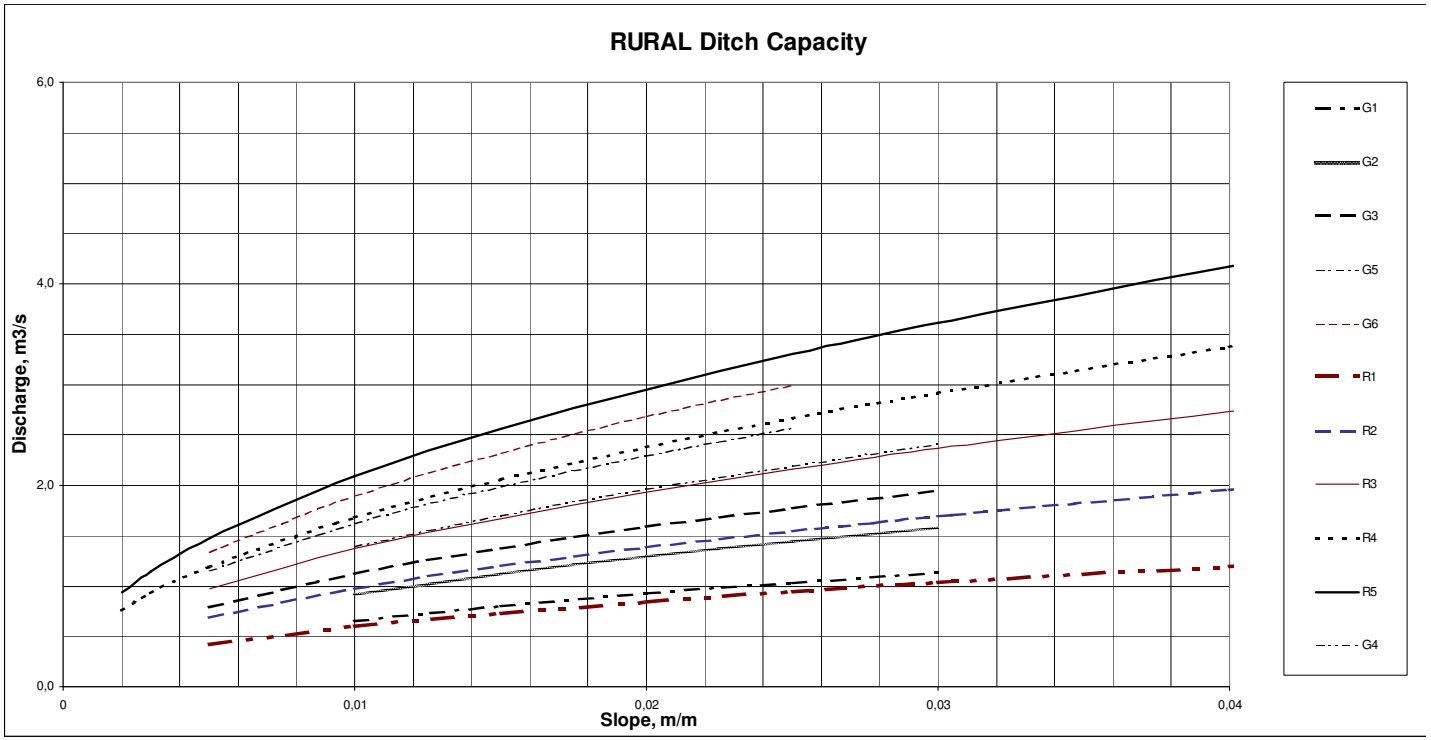


CHART 6-2

DITCH FLOW CAPACITY								
CURVE		RURAL						
#	LINING	n	Shape	B	Z	D	K	Scrit
G1	grass	0,045	triangular	0,00	1,5	0,90	6,5	0,0364
G2		0,045		0,00	2	0,90	9,1	0,0331
G3		0,045	trapezoida	0,50	1,5	0,90	11,2	0,0328
G4		0,045		0,50	2	0,90	13,9	0,0308
G5		0,045		1,00	1,5	0,90	16,3	0,0307
G6		0,045		1,00	2	0,90	19,0	0,0294
R1	rip rap	0,030	triangular	0,00	1,5	0,75	6,0	0,0172
R2		0,030		0,00	1,5	0,90	9,8	0,0162
R3		0,030		0,00	1,5	0,90	13,7	0,0147
R4		0,030	trapezoida	0,50	1,5	0,90	16,8	0,0146
R5		0,030		0,50	2,0	0,90	20,9	0,0137

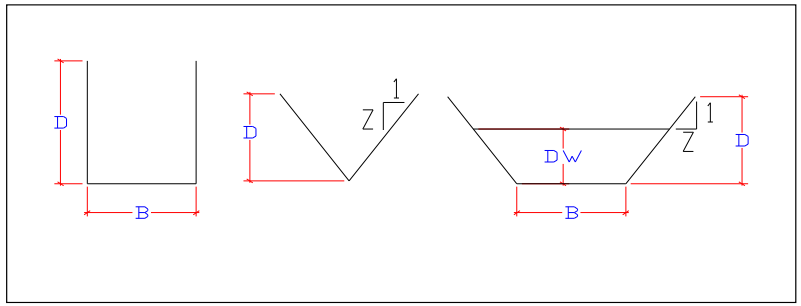


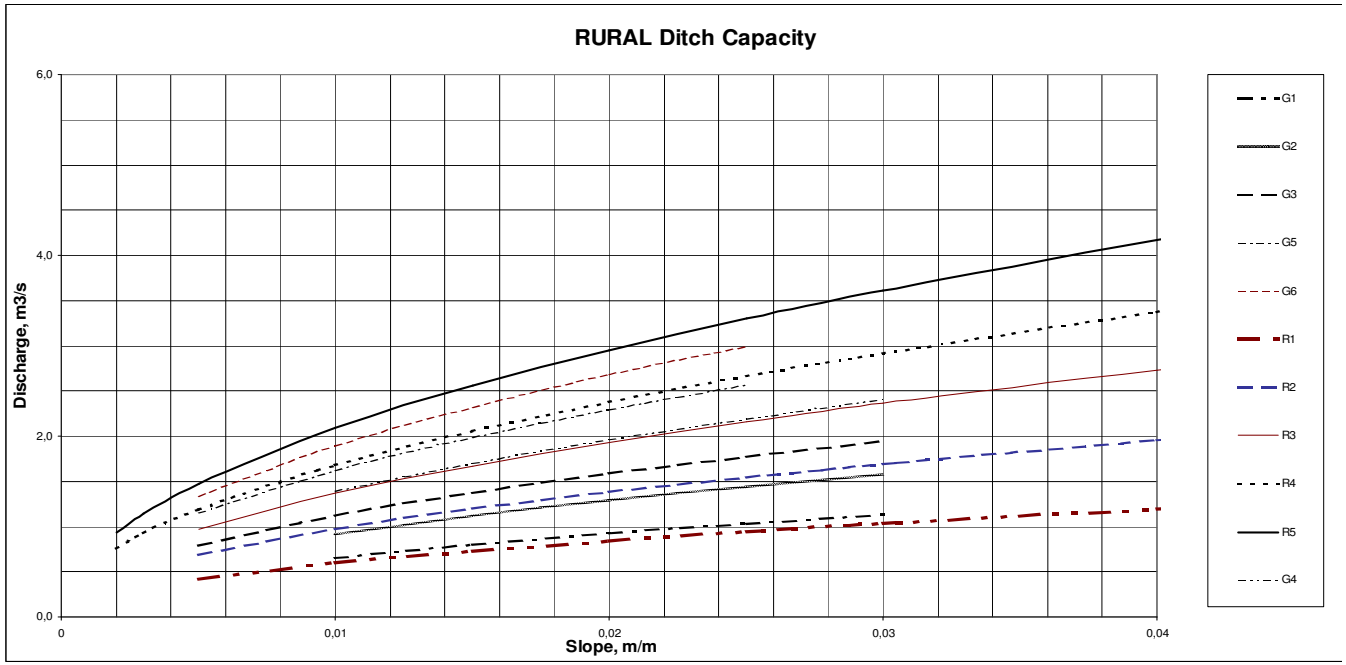


NOTE : DITCH CAPACITY ONLY APPLICABLE OVER RANGE PLOTTED ON CHART. DITHCH CAPACITIES SHOWN ARE LIMITED BY BOTH MINIMUM AND MAXIMUM VELOCITIES

CHART 6-3

DITCH FLOW CAPACITY								
CURVE		RURAL						
#	LINING	n	Shape	B	Z	D	K	Scrit
G1	grass	0,045	triangular	0,00	1,5	0,90	6,5	0,0364
G2		0,045		0,00	2	0,90	9,1	0,0331
G3		0,045	trapezoida	0,50	1,5	0,90	11,2	0,0328
G4		0,045		0,50	2	0,90	13,9	0,0308
G5		0,045		1,00	1,5	0,90	16,3	0,0307
G6		0,045		1,00	2	0,90	19,0	0,0294
R1	rip rap	0,030	triangular	0,00	1,5	0,75	6,0	0,0172
R2		0,030		0,00	1,5	0,90	9,8	0,0162
R3		0,030		0,00	1,5	0,90	13,7	0,0147
R4		0,030	trapezoida	0,50	1,5	0,90	16,8	0,0146
R5		0,030		0,50	2,0	0,90	20,9	0,0137





NOTE : DITCH CAPACITY ONLY APPLICABLE OVER RANGE PLOTTED ON CHART. DITHCH CAPACITIES SHOWN ARE LIMITED BY BOTH MINIMUM AND MAXIMUM VELOCITIES

CHART 6-4

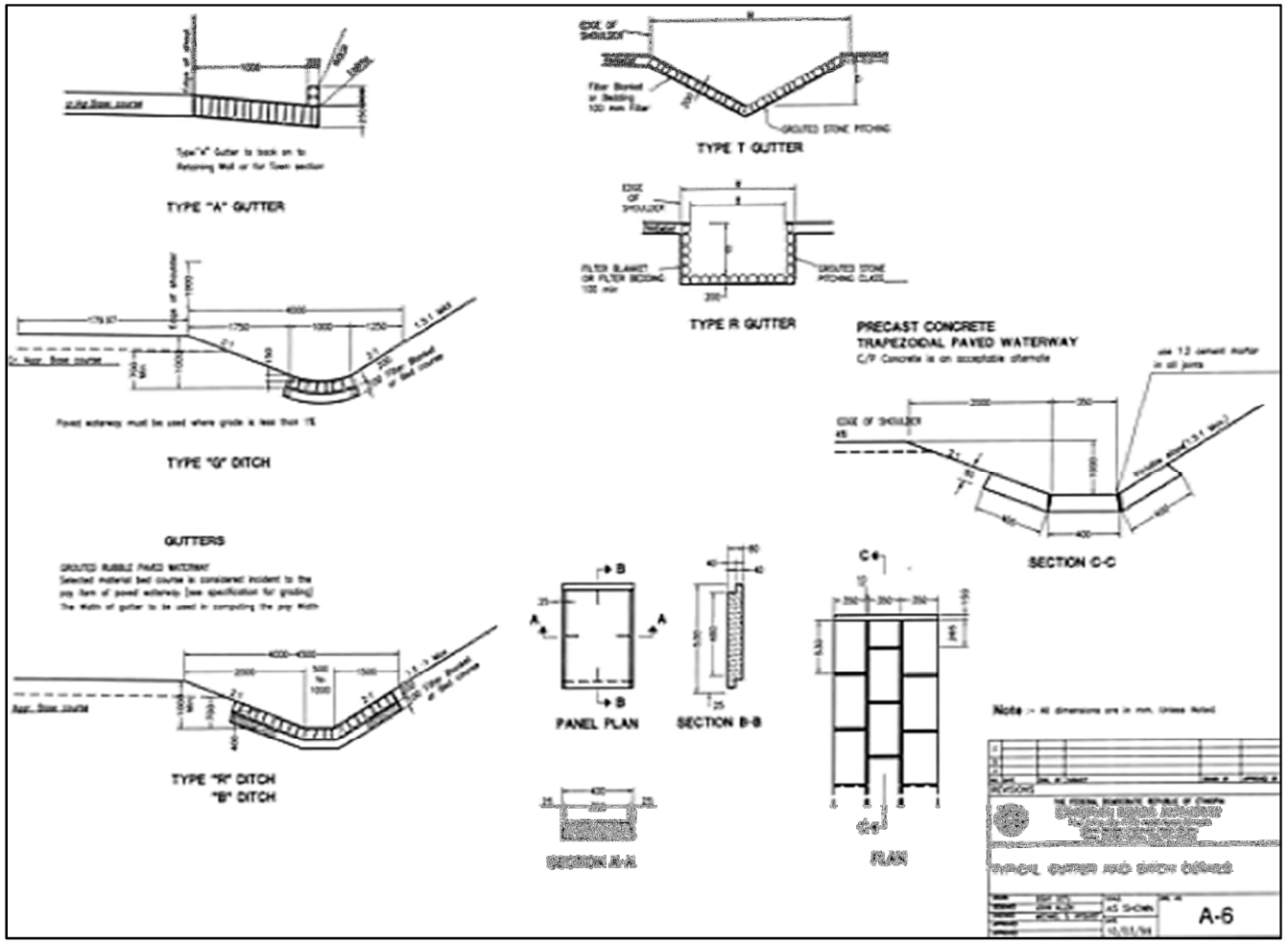


Figure 6A-10: Typical Gutter and Ditch Details

APPENDIX 6B - WORKED EXAMPLES

Example 6-1

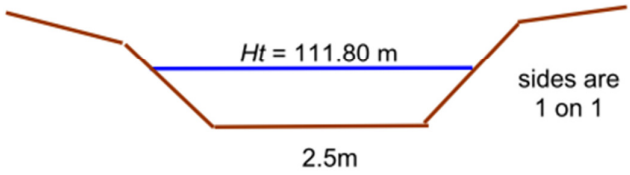
This example describes the process to determine the flow rate, the velocity of a flow and the state of flow in a stream. The example commences after the stream data (such as cross section, terrain, condition of channel and stream profile to determine site bed slope) has been gathered (refer Chapter 4). The task for this example is, given the stream data and height of flow (see diagram below), determine the velocity of flow in the channel, the flow rate and state of flow (subcritical / critical / supercritical flow).

Stream Data

Bed slope about the site is 0.8%

Ht of channel bed is 110.60 m

Channel is regular and considered a little rough with a lot of trees and weeds along the banks.



Solution

We need to determine the velocity of flow using Manning's formula first, then the flow rate using the fundamental equation and finally, determine Froude's number to describe the state of flow.

Manning's equation is:

$$V = \frac{R^{0.667}\, x\, S^{0.5}}{n}$$

Where R is the hydraulic radius, determines as cross sectional area of flow (A) divided by the wetted perimeter (P). Also, S is the slope of the energy line which we don't have, therefore we can use the bed slope (S_o) to approximate S.

Step 1. Calculate the cross sectional area of the flow.

$$A = 1.2 * 1.2 + 1.2 * 2.5 = 4.44\ m^2$$

Step 2. Calculate the wetted perimeter.

$$P = \left(\sqrt{(1.2 * 1.2 + 1.2 * 1.2)}\right) * 2 + 2.5 = 5.89\ m$$

Step 3. Calculate the hydraulic radius.

$$R = \frac{A}{P} = \frac{4.44}{5.89} = 0.75\ m$$

Step 4. Now determine an appropriate Manning's roughness coefficient.

Using Table 6.1, Manning's n Values for Natural Channel for this example is assumed trees and weeds with " n" values ranging between 0.06 and 0.08. Now, with the bed being a little rough, a value n = 0.07 (in the middle of range) is considered appropriate.

Step 5. All variables have now been determined, therefore calculate velocity.

$$V = \frac{0.75^{0.667} * 0.008^{0.5}}{0.07}$$

$$V = 1.06\ m/s$$

Step 6. Using fundamental equation Q = V.A, we can now determine the flow rate in the channel.

$$Q = 1.06 * 4.44 = 4.71\ m^3/s$$

Step 7. To determine the 'state of flow', we calculate Froude's number.

$$F_r = Q\sqrt{\frac{B}{gA^3}}$$

We have determined that Q = 4.71 m^3/s, A = 4.44 m^2 and g is acceleration due to gravity (taken as 9.81 m/s^2), therefore we need to calculate B, the width of flow across the surface.

$$B = 1.2 + 2.5 + 1.2 = 4.9\ m$$

Therefore:

$$F_r = 4.71\sqrt{\frac{4.9}{9.81 * 4.44^3}}$$

$$F_r = 0.36$$

Froude's number is below 1.0, therefore the flow is subcritical.

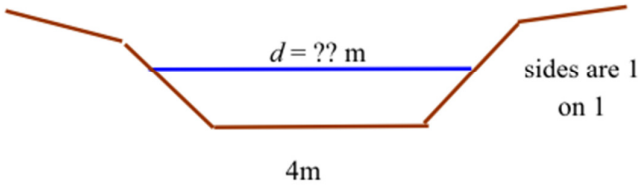
End of Example

Example 6-2

This example describes the process to determine the depth and velocity of flow based on a known discharge / flow rate in a stream, based on discussion in Section 8.4.
The example commences after the stream data (such as cross section, terrain, condition of channel and stream profile to determine site bed slope) has been gathered (refer Chapter 4) and the flow rate (as determine using Rational Method) has been determined (refer Chapter 5). The task for this example is, given the stream data and flow rate (see diagram below), determine the depth and velocity of flow in the channel.

Stream Data

- Discharge/flow rate = 17.86 m^3/s
- Bed slope about the site is 1.2%
- Ht of channel bed is 65.10 m
- Max depth of flow is 2.0 m
- Manning's n = 0.06.



Solution

To solve for d, we need to use Manning's formula and develop a Stage-Discharge curve.

$$V = \frac{R^{0.667} * S^{0.5}}{n}$$

A Stage-Discharge curve plots discharge against depth of flow. Therefore several iterations using Manning's formula are required for several depths of flow.

Step 1. Using the maximum channel depth of 2.0m, calculate stream velocity and flow rate. Calculate the cross sectional area of the flow, wetted perimeter and hydraulic radius:

A = 12.00 m^2, P = 9.66 m therefore R = 1.24 m

Now,

$$V = \frac{1.24^{0.667} * 0.012^{0.5}}{0.06} = 2.11\ m/s$$

Using Q/VA, Q = 25.33 m^3/s

This flow is greater than the known discharge therefore we know that the channel can easily carry the flow.

Step 2. Now, using the same method, re-calculate stream velocity and flow rate for several lesser depths (suggest using even increments).

Depth	*A*	*P*	*R*	Velocity	Discharge
1.50	8.25	8.24	1.00	1.83	15.07
1.00	5.00	6.83	0.73	1.48	7.42
0.50	2.25	5.41	0.42	1.02	2.29

Step 3. Now draw the Stage-Discharge curve for this site/channel (refer next page).

Step 4. From the curve, we can now read of the flow depth for our design flow of 17.86 m^3/s.

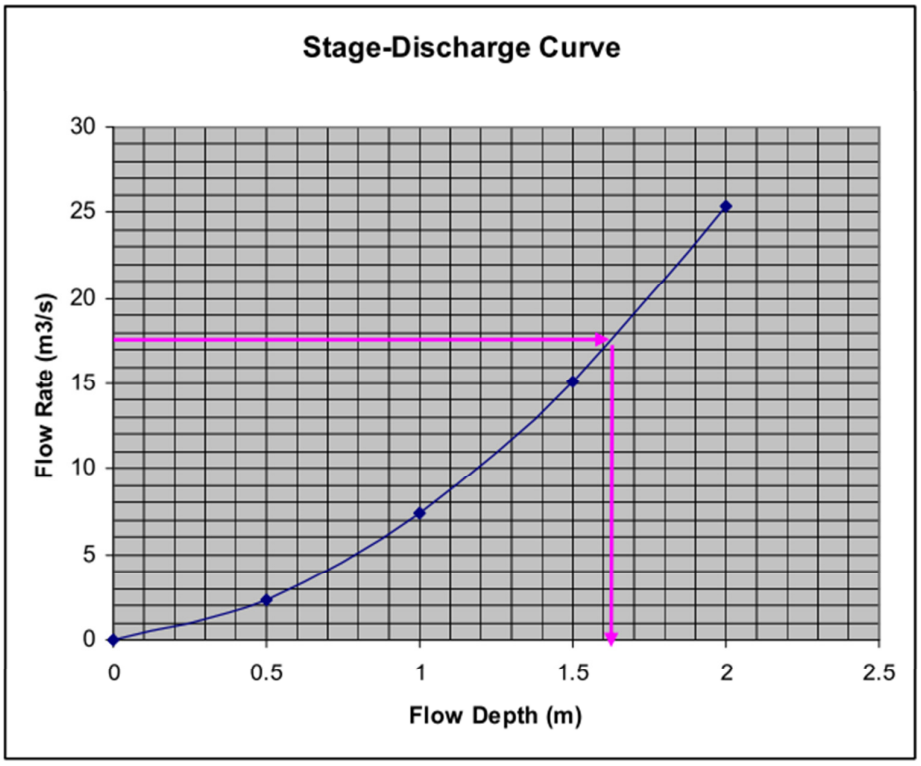
Q = 17.86 m^3/s, therefore d = 1.62 m

Step 5. Now we can use the depth to calculate flow area, then Q = V.A to determine the average flow velocity.

A = 1.62 2 + 1.62 x 4 = 9.10 m^2

17.86 m 3/s = V x 9.10 m^2

V = 1.96 m/s



End of Exercise

Example 6- 3

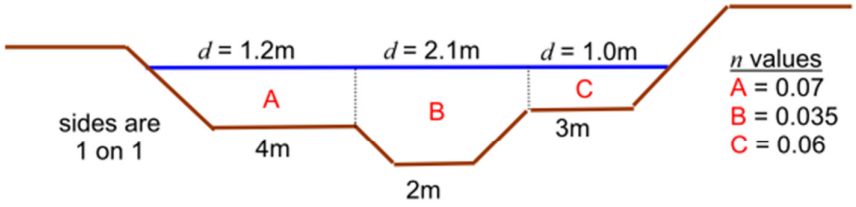
This example describes the process to determine the flow rate, the average velocity of a flow in a compound stream.

The example commences after the stream data (such as cross section, terrain, Manning's n and stream profile to determine site bed slope) has been gathered (refer Chapter 4).

The task for this example is, given the stream data and height of flow (see diagram below), determine the average velocity of flow in the channel and the flow rate.

Stream Data

- Bed slope about the site is 0.8%



Solution

To solve for Q, we need to use Manning's formula for each sub section of stream:

$$V = \frac{R^{0.667} * S^{0.5}}{n}$$

After calculating V for each sub-section, use $Q\ total = (V_A * A_A) + (V_B * A_B) + (V_C * A_C)$ to determine total flow rate.

Step 1. For sub-section A, calculate V_A using Manning's equation.

Calculate the cross sectional area of the flow, wetted perimeter and hydraulic radius for sub-section A are:

$$A_A = \left(\frac{1.2^2}{2}\right) + 1.2 * 4.0 = 5.52\ m^2$$

$$P_A = \sqrt{(1.2^2 + 1.2^2)} + 2.5 = 5.70\ m$$

It is important to remember that that water - water boundary between sub-sections A & B does not contribute any length to the wetted perimeter.

$$R_A = \frac{A}{P} = \frac{5.52}{5.70} = 0.968\ m$$

Now,

$$V_A = \frac{0.97^{0.667} * 0.008^{0.5}}{0.07} = 1.25 m/s$$

Using $\frac{Q}{VA}, QA = 6.90 m^3/s$

Step 2. For sub-section B, calculate V_B using Manning's equation.

$$A_B = 7.39 m^2$$

$$P_B = 4.83 m$$

$$R_B = 1.53 m$$

Now,

$$V_B = \frac{1.531^{0.667} * 0.008^{0.5}}{0.035} = 3.40 m/s$$

Using $\frac{Q}{VA}, Q_B = 25.09 m^3/s$

Step 3. For sub-section C, calculate V_C using Manning's equation.

$$A_C = 3.50 m^2$$

$$P_C = 4.41 m$$

$$R_C = 0.79 m$$

Now,

$$V_C = \frac{0.793^{0.667} * 0.008^{0.5}}{0.06} = 1.28 m/s$$

Using $\frac{Q}{VA}, Q_C = 4.47 m^3/s$

Step 4. Now we can calculate:

$$Q_{total} = 6.90 + 25.09 + 4.47 = 36.46 m^3/s$$

And

$$A_{total} = 5.52 + 7.39 + 3.50 = 16.41 m^2$$

Therefore

$$V_{avg} = \frac{36.46}{16.41} = 2.22 m/s$$

End of Exercise

Example 6.4

Given: A roadside drainage channel is trapezoidal with a bottom width of 1.2 m and 1V:3H side slopes. The bed slope is 0.005 m/m and the design flow rate is 0.6 m^3/s.

Find: Calculate the required diameter (D_{50}) of a gravel riprap that is to be used as a permanent channel lining, and the design depth of flow.

Solution: The solution follows the procedure outlined in HEC-15, which is based on the tractive force method.

(1) Choose a rounded gravel with D_{50} = 25 mm

Then τ_p = 19 Pa (Table 6-3)

(2) Estimate n = 0.033 from Table 6-4 for depth y = 0.15 – 0.6 m

(3) Calculate y from Manning's equation (Figure 6-4)

$$\left[\frac{1.486nQ}{(b^{8/3})(S^{1/2})}\right] = \frac{[1.486(0.033)(0.6)]}{[(1.2^{8/3})(0.005^{1/2})]} = 0.256$$

Then from Figure 6-4 with Z = 3: y/b = 0.29 and y = 0.35 m

(4) Calculate maximum bed shear stress, τ_d

τ_d = 9800 yS = 9800 x 0.35 x 0.005 = 17 Pa

Now because $\tau_d < \tau_p$, accept D_{50} of approximately 25 mm. Otherwise repeat with another riprap diameter.

(5) Side slopes will be stable because side slope is not steeper than 1V:3H. If side slopes are steeper than 1V:3H or if channel slope is steep, consult HEC-15 for additional computations.

Example 6-5

Given: A median ditch is lined with a good stand of native grasses (approximately 0.203m in height). The ditch is trapezoidal with a bottom width of 1.2m and side slopes of 1V:4H. The ditch slope is 0.010m/m.

Find: Compute the maximum discharge for which this lining will be stable and the corresponding flow depth.

Solution: From Table 6-2, the native grass has a retardance class of C and from Table 6-3, the permissible shear stress is

τ_p = 48 Pa

Then the allowable depth can be determined by assuming $\tau_p = \tau_d$:

$$y = \frac{\tau_p}{9800S} = \frac{48}{9800 * 0.01} = 0.49m$$

Now determine the flow area A and hydraulic radius R:

$$A = y(b + zy) = 0.49\big(1.2 + (4 * 0.49)\big) = 1.55m^2$$

$$P = b + 2y(1 + z^2)^{1/2} = 1.2 + (2 * 0.49)(1 + 15)^{1/2} = 5.24m$$

$$R = \frac{A}{P} = \frac{1.55}{5.24} = 0.30m$$

Finally determine the Manning's n value from Photo 21 and solve for Q from Manning's equation:

From Photo 21, n = 0.080 and

$$Q = \left(\frac{1}{n}\right) AR^{2/3}S^{1/2}$$

$$Q = \left(\frac{1}{0.080}\right)(1.55)(0.30)^{2/3}(0.01)^{1/2} = 0.86m^3/s$$

(This method is called the maximum discharge method and is useful for determining the stable channel capacity for a variety of different linings for purposes of comparison).

Example 6-6

Given: A rectangular channel on a slope of 0.001 with a width of 1.8m expands to a width of 3 m in a straight walled transition, Z = 0. The design discharge is 8.5 m^3/s and Manning's n = 0.02.

Find: Calculate the depth of flow in the upstream 1.8m wide channel if normal depth is the downstream control.

Solution: (1) Compute the downstream normal depth y_2:

$$\left[\frac{1.486nQ}{b^{8/3}S^{1/2}}\right] = \left[\frac{1.486 * 0.02 * 8.5}{3.0^{8/3} * 0.001^{1/2}}\right] = 0.427$$

Then from Figure 6-4 with z = 0: $y_2/b = 0.68$ and y_2 = 2.04 m and for a rectangular channel, $y_c = ((Q/b)^2/g)^{1/2}$ = 0.9 m
SUBCRITICAL

(2) The specific energy downstream is:

$$E_2 = \frac{(y_2 + Q^2)}{(2_g A_2{}^2)} = 2.04 + \frac{8.5^2}{(2 * 9.81)(3 * 2.04)^2} = 2.14m$$

(3) Choose a straight-walled transition with a divergence angle of 12.5 degrees which has an expansion loss coefficient of 0.5 (HEC-14). The length of the transition would be:

$$L = \frac{\left(\frac{3.0 - 1.8}{2}\right)}{\tan 12.5°} = 2.71m$$

(4) Check if subcritical flow is possible by assuming critical depth in upstream channel.

$$E_{1c} = \frac{(y_{1c} + V_{1c}{}^2)}{2g} \; and \; E_1 = E_2 + z_2 - z_1 + h_L$$

Where: z_2-z_1 = 0.001(2.71) = 0.003 ≈ 0

$y_{1c} = ((8.5/1.8)^2/9.81)^{1/3}$ = 1.31 m

V_{1c} = Q/A_{1c} = 8.5/(1.8 x 1.31) = 3.6 m/s

V_2 = Q/A_2 = 8.5/(2.04 x 3) = 1.39 m/s

$H_L = 0.5\ (3.6^2 - 1.39^2)/(2x9.81)$ = 0.281 m

then $E_{1c} = 1.31 + 3.6^2/(2x981)$ = 1.97 m

and E_1 = 2.14 + 0 + 0.281 = 2.42 m

Now since $E_1 > E_{1c}$, a **subcritical** solution exists. If this were not the case, the width of 1.8 m would have to be increased.

(5) Solve the energy equation, Eq. 6.10, by trial:

$$\frac{z_1 + y_1 + Q^2}{(2gA_1{}^2)} = z_2 + E_2 + h_L$$

$$y_1 + \frac{(1 - 0.5)(8.5)^2}{(2 * 9.81) * (1.8^2 x y_1{}^2)}$$

$$= \frac{2.14 - 0.5(8.5)^2}{(2 * 9.81) * (3 * 2.04)^2}$$

where: ..... z_1-z_2 = 0

..... $h_1 = 0.5\ (Q^2/(2gA_1{}^2) - Q^2/(2gA_2{}^2))$

..... E_2 = 2.14 m

..... A_2 = 3.0 x 2.04 m^2

with the result y_1 = 1.94 m and V_1 = 3.07 m/s

(6) Calculate the water surface profile using the Standard Step Method if boundary resistance losses are of concern.

Example 6.7

Given: A rectangular transition contracts from a width of 3.0 m to a width of 1.5 m. The approach flow rate is 8.5 m^3/s with a depth of 0.3 m.

Find: Calculate the depth in the contracted section and the angle and length of the contraction so that the transmission of standing waves downstream is minimized.

Solution: (1) ..... Calculate the approach Froude number for a rectangular channel.

$$F = \frac{V}{(gd)^{1/2}} = \frac{\frac{8.5}{(3.0 * 0.3)}}{(9.8 * 0.3)^{1/2}} = 5.5\ \boldsymbol{SUPERCRITICAL}$$

(2) Determine the contraction ratio:

$$r = \frac{b_3}{b_1} = \frac{1.5}{3.0} = 0.5$$

(3) Use Figure 6-12

$$\theta = 5° and \frac{y_3}{y_1} = 2.1\ or\ y_3 = 0.62m$$

$$\frac{F_3}{F_1} = r^{-1}\left(\frac{y_3}{y_1}\right)^{-3/2} = 0.66\ or\ F_3 = 3.6$$

$$L = \frac{\left(\frac{b_3 - b_1}{2}\right)}{\tan 5°} = 8.57m$$

This design satisfies the criterion F_3>2 and also is just to the left of curve, A which means choking is not possible (for the complete equations see HEC-14 and Storm. 1985).

7 CULVERTS

7.1 Introduction

Cross drainage involves the conveyance of surface water and stream flow across or from the highway right of way. This is accomplished by providing either a culvert or a bridge to convey the flow from one side of the roadway to the other side or past some form of flow obstruction.



Photo 7-1: Typical Culvert at Adet Quarit Road Gojjam

In addition to the hydraulic function, a culvert must carry construction and highway traffic and earth loads. Culvert design, therefore, involves both hydraulic and structural design. However, this section of the manual is concerned with the hydraulic design of culverts. Both the hydraulic and structural designs must be consistent with good engineering practice and economics.

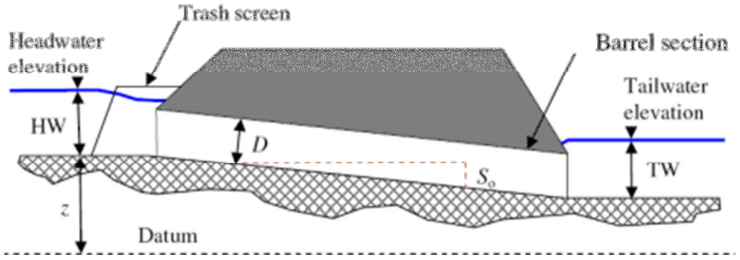


Figure 7-1: Culvert components

A culvert is a structure that is designed hydraulically to take advantage of submergence to increase hydraulic capacity. It is also a structure used to convey surface runoff through embankments. A culvert is usually covered with fill and is composed of structural material around the entire perimeter. These include steel and concrete pipe culverts and concrete box culverts. However, a culvert can also be a structure supported on spread footings with the streambed serving as the bottom of the culvert. These include some multi-plate steel structures and concrete slab culverts. In addition, a culvert can be a structure that is 6 meters or less in centreline span length, or between the extreme ends of openings for multiple boxes. Structures designed hydraulically as a culvert regardless of length are treated in this chapter.

In designing culverts, a number of issues must be considered including:

- Economy;
- Eoad immunity – the extent to which flows are passed through culverts under the road rather than allowed to overflow the road;
- The configuration of culverts including size and number; alternative culvert types and materials;
- Afflux (that is the increase in water level caused by the road and its culvert);
- The culvert's outlet velocity;
- The special needs of culverts which are to be used as fishways, for the passage of terrestrial fauna, or as cattle creeps (larger culverts will often be required for fauna or fish passage than for hydraulic reasons);
- Safety (that is catering for the needs of pedestrians, cyclists, or maintenance crews); and
- Environmental issues (minimising the potential for unacceptable environmental damage).

To provide consistency within this chapter the following symbols are used. These symbols are selected for their wide use in culvert publications.

Symbol	***Definition***	**Units**
A	Area of cross section of flow	m^2
AHW	Allowable Headwater depth	m
B	Barrel width	m
D	Culvert diameter or barrel height	mm or m
d	Depth of flow	m
d_c	Critical depth of flow	m
g	Acceleration due to gravity	m/s^2
H	Sum of $H_E + H_f + H_o$	m
H_b	Bend head loss	m
H_E	Entrance head loss	m
H_f	Friction head loss	m
H_L	Total energy losses	m
H_o	Outlet or exit head loss	m
H_v	Velocity head loss	m
h_o	Hydraulic grade line height above outlet invert	m
HW	Headwater depth (subscript indicates section)	m
K_E	Entrance loss coefficient	m
L	Length of culvert	m
n	Manning's roughness coefficient	m
P	Wetted perimeter	m
Q	Rate of discharge	m^3/s
R	Hydraulic radius (A/P)	m
S	Slope of culvert	m/m
TW	Tailwater depth above invert of culvert	m
V	Mean velocity of flow with barrel full	m/s
V_d	Mean velocity in downstream channel	m/s
V_o	Mean velocity of flow at culvert outlet	m/s
V_u	Mean velocity in upstream channel	m/s
γ	Unit weight of water	N/m
τ	Tractive force	Pa

7.2 Information Required

The catchment area should be carefully defined from maps, as well as aerial photographs, LIDAR and photogrammetry if available. It is usual to survey stream bed and adjacent land upstream and downstream of the culvert site and features such as other culverts, houses and commercial developments (and possibly their floor levels, if it appears they might be more at risk of flooding due to the new culvert). Cultivated crops and any utility services, which may influence the location and level of the culvert, should also be noted.

Important information in relation to the highest known past flood levels can be obtained by interviewing local residents and ERA road maintenance supervisors. The road drainage designer should also inspect the site thoroughly as the survey may not show all the details relevant to good design and ease of construction.

7.3 Culvert Location

In general, culverts should be located to fit natural channels in line and grade, following moderate curvature and natural changes in grade as far as is practical. A culvert placed on a different skew to that of the natural channel could cause progressive bank erosion and protection of the bank at risk could be costly.



Photo 7-2: Erosion and bank instability at Quarit Adet Road

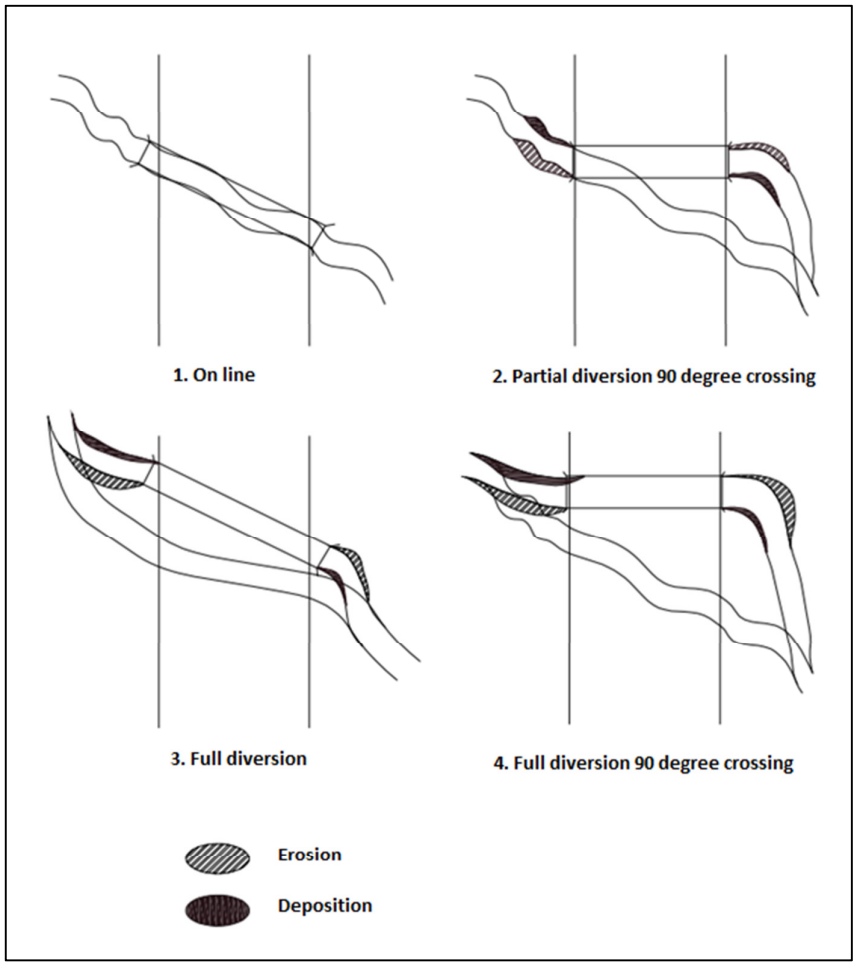


Figure 7-2: Culvert Alignment Options

The culvert should be designed to suit the outlet conditions even if inlet conditions have to be modified (e.g. a drop inlet to reduce potential scouring velocities through the culvert). In most cases, culvert locations are predetermined by the intersection of a watercourse and an existing roadway. However, where circumstances allow, culverts should be located away from:

- Erodible or meandering channel bends or banks;
- Critical or isolated aquatic habitat areas; and
- Isolated sections of remnant, valued, or protected riparian vegetation.

If at all possible, culverts should not be located on the bend of an unstable or otherwise meandering channel. Realigning short sections of an existing channel to fit the culvert alignment should also be avoided. Where roads traverse broad floodplains or otherwise interfere with overland flow patterns, regular culverts may be needed to mitigate against the adverse environmental effects of drainage shadow.



Photo 7-3: Culvert located on a channel bend Arba Minch

7.4 Outlet Velocity

High outlet velocities can cause bank erosion a significant distance downstream of an outlet. Where high outlet velocities are expected, appropriate dissipation measures will be required. Alternatively, in some limited cases, a stabilised scour hole or energy dissipator may be acceptable; however, the design of these is not covered in this chapter.



Photo 7-4: Erosion at outlet of a culvert at Sodo Sawla Road

Where possible, culverts should be designed to provide acceptable velocities without the need for additional stream bed protection. Allowable streambed velocities to avoid scour vary according to soil type and topography. Suggested maximum average culvert velocities for various stream bed materials are given in Table 7.1. Scour and preventative measures are discussed further in Section 7.18.

Table 7-1: Maximum culvert velocities

Material downstream of culvert endwall	Allowable velocity (m/s)
Rock	4.5
Stone 150 mm diameter or larger	3.5
Gravel 100 mm or grass cover	2.5
Firm loam or stiff clay	1.2 – 2.0
Sandy or silty clay	1.0 – 1.5

Note: These are target velocities at the culvert outlet

7.5 Vertical Profile

Most longitudinal culvert profiles should approximate the natural streambed. Other profiles may be chosen for either economic or hydraulic reasons. Modified culvert slopes, or slopes other than that of the natural stream, can be used to prevent stream degradation, minimise sedimentation, improve the hydraulic performance of the culvert, shorten the culvert, or reduce structural requirements. Modified slopes can also cause stream erosion and deposition. Slope alterations should, therefore, be given special attention to ensure that detrimental effects do not result from the change.

Channel changes often result in culverts being shorter and steeper than the natural channel. A modified culvert slope can be used to achieve a flatter gradient to prevent channel degradation.

7.6 Culverts in Flat Terrain

In flat terrain, drainage channels are often ill-defined or non-existent and culverts should be located and designed for least disruption of the existing flow conditions. In these locations multiple culverts can be considered to have a common headwater elevation, although this will not be precisely correct.



Photo7-5: Multiple culverts

(Sources: Left South African Manual 2006, Right Photo from Tigray Ethiopia)

In flat terrain it may be necessary to construct levee banks (Figure 7.3) to achieve the design headwater at the culvert location. Where necessary, approval of the local road authorities should be obtained prior to construction of any levee banks.

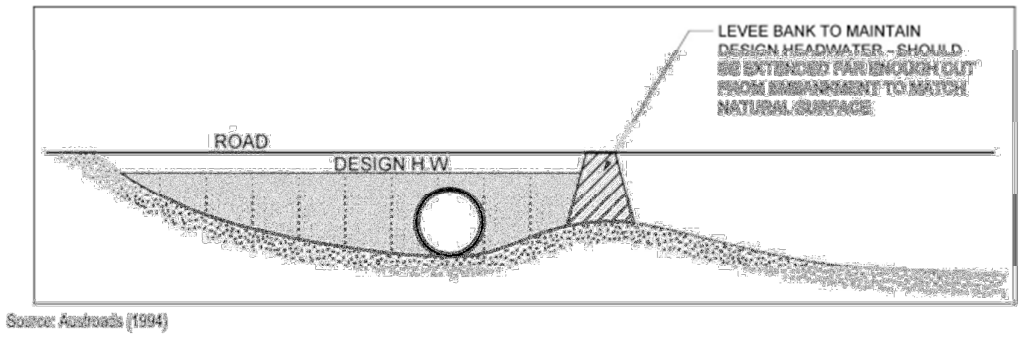


Figure 7-3: Development of headwater

7.7 Culvert Type

The selection of the most appropriate type of culvert is dependent on a range of factors including economics, site conditions, and environmental considerations. While the majority of culverts consist of concrete pipe or box culverts, corrugated metal pipes, pipe-arch or arches, may be appropriate and economic in some situations.

Box culverts are generally used where there is insufficient headroom for pipes, where the available waterway area for the culvert is at a minimum, or where fauna passage is an issue. In multi-cell construction, slab linked box culverts are often chosen for economical purposes. Metal culverts have some advantages such as lower cost, and ease of transport and installation.

However, they also have some serious disadvantages such as the potential for corrosion, damage due to poor construction or compaction, and higher cover requirements. Unless there are large financial savings, or other construction restraints, other more robust and more durable materials should generally be used.

7.8 Siltation/Blockage

The likelihood of blockage should be considered for all culverts. Blockage can occur through siltation or vegetation, although blockage by siltation is more likely to be temporary in nature. This is because during flood events, silt deposits can be removed by high velocity flows. To prevent siltation the desirable minimum velocity in the culvert should be above 0.7 m/s. A check of velocities should be undertaken as part of design process.

Where debris blockage is considered likely, larger culvert sizes may be required, in accordance with the extent of adverse impacts that could occur to the roadway or to surrounding properties. Blockage by debris is more likely to occur where the catchment contains significant woody riparian vegetation. In this case detailed assessment of the catchment is required.



Photo7-6: Culvert blocked by siltation and debris Tigray

7.9 Allowable Headwater

One or more of the following conditions will usually determine the allowable headwater for a culvert:

- The elevation of upstream ponding should not cause unacceptable damage or adverse effects to adjacent properties. The fact that floods may already enter properties even without any road embankment should be ascertained in field investigations or by design calculations;
- Where a road is designed not to be overtopped in the 1 in 50 year or 0.5% Annual probability event for example, it is desirable to provide freeboard between the design upstream floodwater surface and the upstream road shoulder edge. Where this is not economically acceptable, pavement design should make allowance for higher water levels, and the likely duration of inundation;
- In the event of both of the above conditions permitting a high headwater, the associated outlet velocity may be intolerably high. In this case, the allowable headwater may have to be reduced to limit the outlet velocity to an acceptable value, that is, one that does not cause unacceptable scouring; and
- Where practicable, it is desirable to keep the headwater in the road reserve or upstream with the landowner's permission.

7.10 Tailwater

Tailwater (TW) is the depth at the culvert outlet, measured from the water surface in the downstream channel to the invert of the culvert. Tailwater is significant for the following reasons;

- A high tailwater may cause the culvert to flow full or under pressure, so increasing the headwater necessary to pass the flow; and
- A low tailwater relative to the depth of flow in the culvert can result in erosion of the downstream channel.

If the channel is regular in shape and steady uniform flow conditions can be expected, the tailwater level can be determined using Manning's formula as follows:

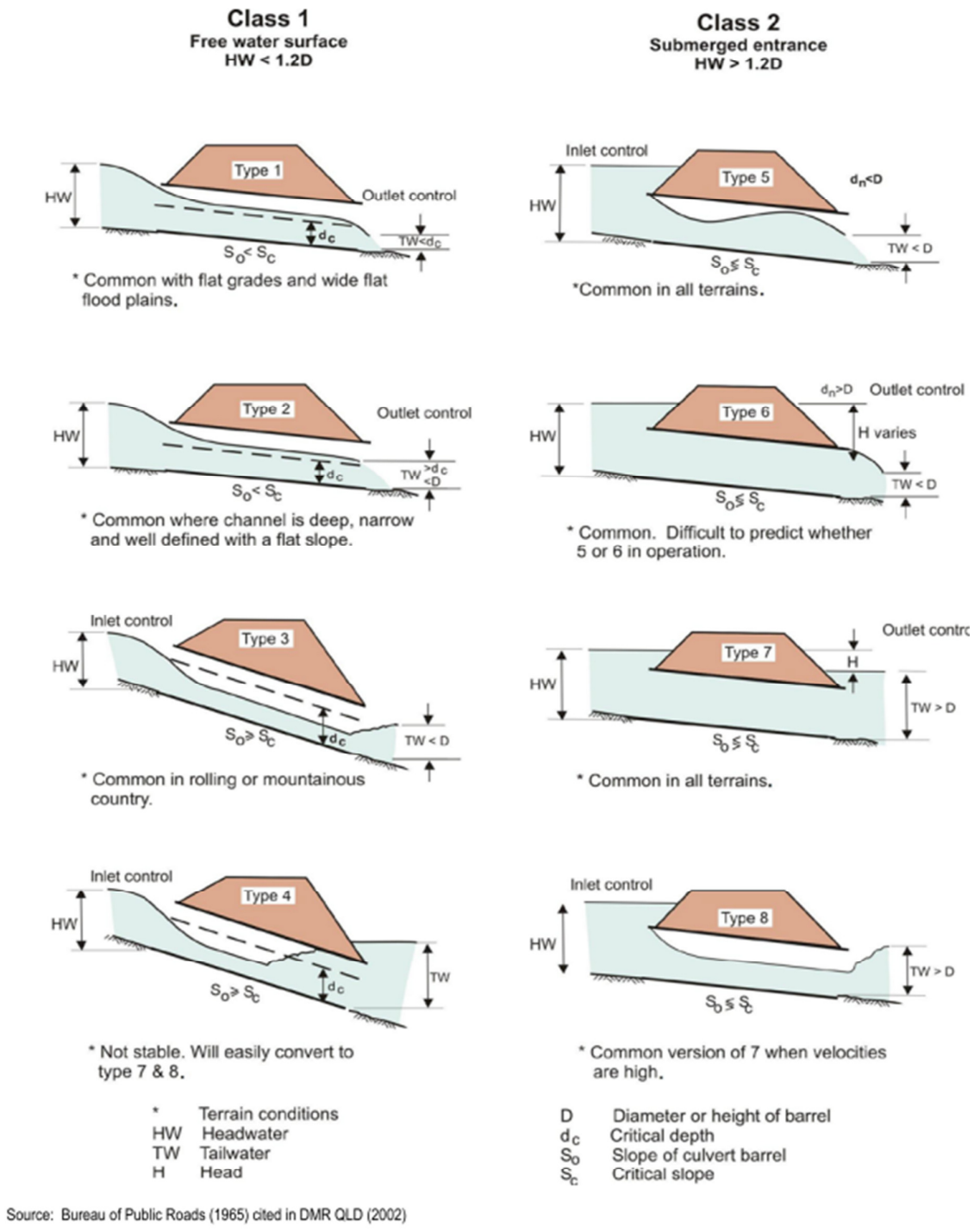
- Select a trial value of TW. This could be based on the suggested maximum velocity in Table 7.1 and A = Q/V. The closer the trial TW is to the true value, the less iteration will be required;
- Calculate the average channel velocity for this trial depth using Manning's formula, then calculate Q = AV;

- If the channel capacity exceeds the design discharge, recalculate with a reduced depth; or if the channel capacity is less than the design discharge, recalculate with an increased depth; and
- Repeat these steps until the estimated channel capacity is within 10% of the design discharge.

For complex channels, backwater models such as HY8, the Hydraulic Engineering Centres River Analysis System (HEC-RAS) MIKE, or ISIS can be used, although this would normally only be necessary for large catchments.

7.11 Hydraulic Performance of Culverts

The most important consideration in culvert hydraulics is whether the flow is subject to inlet or outlet control. Figure 7.4 shows the range of flow types commonly encountered in culverts. For inlet control two distinct regimes exist, depending on whether the inlet is submerged or not submerged. Outlet control occurs in long culverts, culverts laid on flat grades and with high tailwater depths. In designing culverts, the type of control is determined by adopting the greater of the headwater depths calculated for both inlet control and outlet control.



Source: Bureau of Public Roads (1965) cited in DMR QLD (2002)

Figure 7-4: Typical conditions under which standard culverts operate

7.12 Inlet Control

When the capacity of the culvert barrel is greater than that of the inlet, the culvert is subject to inlet control. Then the important factors are the cross-sectional area of the culvert barrel, the depth of headwater or ponding at the entrance and the entrance conditions, including the entrance type, existence and angle of headwalls and wing walls, and the projection of the culvert into the headwater pond.

For one-dimensional flow, the relationship between the discharge and the upstream energy can be computed by an iterative process.

Inlet control can occur with the inlet submerged and the outlet not submerged. Under these conditions, the flow contracts to a supercritical jet immediately downstream from the inlet. When the tailwater depth exceeds critical depth, hc and the culvert is laid on a steep grade, flow remains supercritical in the barrel and a hydraulic jump will form near the outlet. If the culvert is laid on a slope less than critical, then a hydraulic jump will form in the barrel.

When the culvert flows under inlet control, the roughness and length of the culvert barrel and the outlet conditions (including the depth of tailwater) are not factors in determining culvert capacity. An increase in the slope of the culvert reduces headwater only to a small degree, and can normally be neglected for conventional culverts flowing under inlet control. Design charts for the design of concrete culverts with inlet control are provided in Appendix 7A.

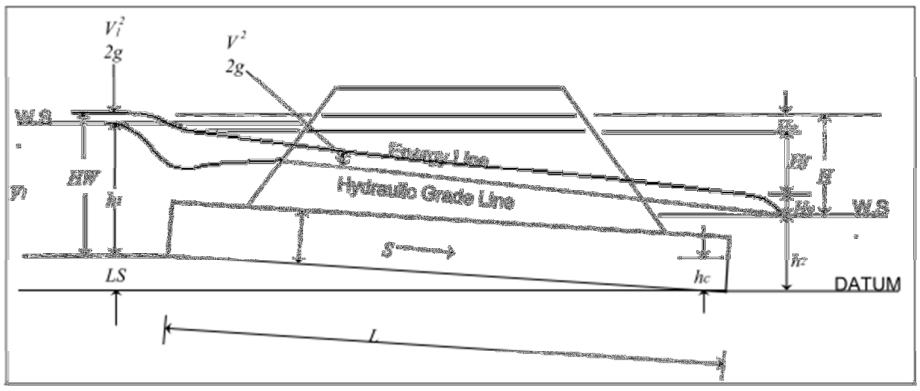
7.13 Outlet Control

With outlet control the culvert flow is restricted to the discharge which can pass through the conduit for a given level of water in the outlet channel (tailwater level). The slope, cross-sectional area, roughness and length of the culvert barrel have to be considered as these losses exceed the inlet losses. However, inlet edge geometry can still affect the capacity.

In general the control will be at the outlet if the culvert slope is less than critical. A tailwater depth equal to 80% or more of the height of the culvert barrel/cell will usually indicate outlet control, except in rolling or mountainous country with the culvert on natural surface slopes. However, a check of the design assuming inlet control is such an easy process that it forms part of standard design procedure.

Culverts flowing with outlet control can flow with the culvert barrel full or with the barrel part-full for all of the culvert length. With outlet control, and both the inlet and the outlet submerged the culvert flows full and under pressure. The culvert, also, can flow full over part of its length with part-full flow at the outlet. The point at which the water surface breaks away from the barrel soffit depends on the tailwater depth and culvert grade, and can be determined by using flow profile calculations.

If the culvert is laid at a flat grade, outlet control can occur with both inlet and outlet not submerged, and part-full flow throughout the culvert length will be flowing under sub-critical conditions. Variations of these main types can occur, depending on the relative value of critical slope, normal depth, culvert height and tailwater depth. While the potential flow conditions shown in Figure 7.4 are the most common for simple culverts, different flow conditions are possible where complex culvert structures are required and advice may be required from an expert in such cases.



Source: Austroads (1994)
Note: WS denotes water surface and S the slope of the culvert

Figure 7-5: Hydraulics of culvert flowing full under outlet control

Flow under outlet control can be calculated from the formulae below, the parameters for which are illustrated in Figure 7.5. The total head (H) required to pass water through a culvert flowing under outlet control is determined by:

$$H = H_v + H_e + H_f$$

Where:

$$\boldsymbol{H_v = Velocity\ head(m) = \frac{V^2}{2g}}$$

$$\boldsymbol{H_e = entrance\ loss(m) = K_e \frac{V^2}{2g}}$$

$$\boldsymbol{H_f = friction\ loss(m) = \frac{19.6n^2L}{R^{1.33}} * \frac{V^2}{2g}}$$

and

V = velocity of flow in the culvert barrel, (m/s)

K_e = entrance loss coefficient, for values see Table 7.2

N = Manning's friction factor, for values see Table 7.3

L = length of culvert barrel, (m)

g = acceleration due to gravity = 9.8 m/s²

$$\boldsymbol{R = hydraulic\ radius, (m) = \frac{Area\ of\ flow\ at\ outlet}{Wetted\ Perimeter}}$$

Substituting in the first equation above and simplifying:

$$\boldsymbol{H = 1 + K_e + \left(\frac{19.6n^2L}{R^{1.33}} * \frac{V^2}{2g}\right)}$$

This equation can be solved for H by the use of the full flow nomographs in Figure A 5 and Figure A 6 for concrete culverts and Figure A 7 to Figure A 10 for metal culverts (refer to Appendix 7A).

From the development of this energy equation and Figure 7.4, H is the difference between the elevation of the hydraulic grade line at the outlet and the energy line at the inlet. Since

the velocity head in the entrance pool usually is small when ponded conditions occur ($v^2/2g \approx 0$), the water surface of headwater pool elevation can be assumed to equal the elevation of the energy line.

Notes: The effect of wingwalls reduces with multi-cell culverts for 3 – 6 cell culverts, assume entrance loss for wingwalls 100 to 250 to barrel; for culverts with more than 6 cells, assume wingwalls parallel (extension of sides), regardless of actual wings.

Table 7-2: Culvert Entry Loss Coefficient

Type of Structure and design of entrance	Coefficient of Ke
Concrete pipe	
Projecting from fill, socket end (grove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls, socket end of pipe	0.2
Square edge	0.5
Rounded (radius = D/12)	0.2
Mitred to conform to fill slope	0.7
End section confirming to fill slope	0.5
Corrugated metal pipe	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls, square edge	0.5
Mitred to conform to fill slope	0.7
End section confirming to fill slope	0.5
Reinforced concrete box	
Headwall parallel to embankment (no wingwalls), square edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 cell dimensions	0.2
Wingwalls at 30º to 75º to cell, square edged at crown	0.4
Crown edge rounded to radius of 1/12 cell dimension	0.2
Wingwalls at 10º to 25º to cell, square edged at crown	0.5
Wingwalls parallel (extension of sides), square edged at crown	0.7
Note: end sections conforming to fill slope, refers to the sections available from manufacturers	

Source: Bureau of Public Road (1965)

Table7-3: Recommended Manning's n Values for Pipe

Type of Conduit	Wall Description	Manning's n
Concrete Pipe	Smooth Walls	0.010-0.013
Concrete Boxes	Smooth Walls	0.012-0.015
Corrugated Metal Pipes and Boxes,	68mm x 13mm corrugations	0.022-0.027
Annular or Helical Pipe	150mm x 25mm corrugations	0.022-0.025
	125mm x 25mm corrugations	0.025-0.026
	75mm x 25mm corrugations	0.027-0.028
	150mm x 50mm structural plate	0.033-0.035
	230mm x 64mm structural plate	0.033-0.037
Corrugated Metal Pipes, Helical	68mm x 13mm corrugations	0.012-0.024
Corrugations, Full Circular Flow		
Spiral Rib Metal	Smooth Walls	0.012-0.013

Headwater depth under outlet control is calculated according to the equation:

$$HW = H + h_o - LS_o$$

where:

h_o = tailwater depth (m)

L = length of culvert, (m)

S_o = slope of culvert barrel, (m/m).

The various components of this equation are illustrated in Figure 7.5 and Figure 7.6

The tailwater level (h_o) to be adopted when the tailwater elevation (TW) is below the soffit of the culvert at the outlet is the greater of the two values:

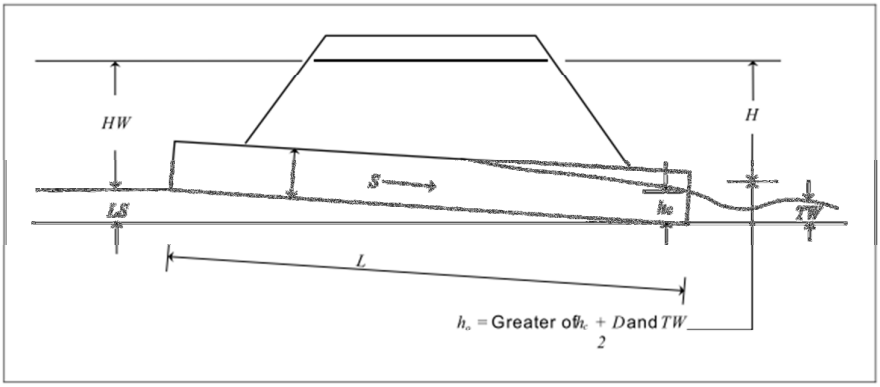
$$\boldsymbol{TW \ or \ \frac{h_c + D}{2}}$$

Where:

h_c = Tailwater depth (m)

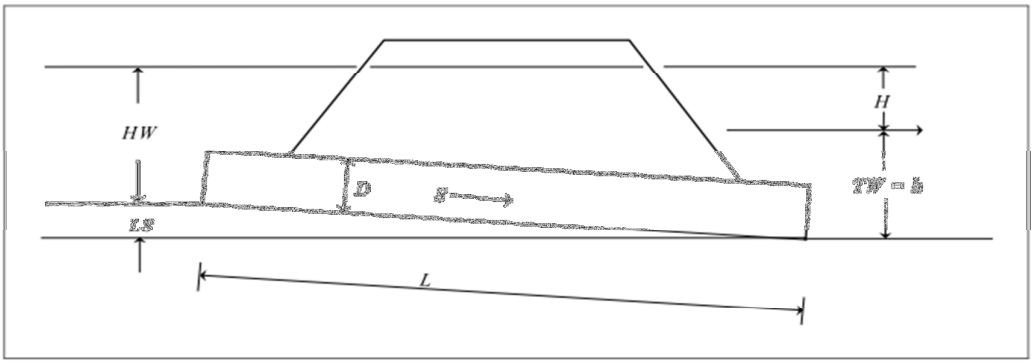
D = Diameter of culvert, (m).

For calculation of the outlet velocity, the TW may be slightly different than shown in Step 11 in Figure 7.6.



Source: Source: Austroads (1994)

Figure 7-6: Determination of h_o for Tailwater Below Top of Opening



Source: Austroads (1994)

Figure7-7: Determination of h_o for High Tailwater

7.14 Compute Outlet Velocity and Determine need for Channel Protection

The average outlet velocity will be the discharge divided by the cross-sectional area of flow at the culvert outlet. The cross-sectional area of flow depends, in turn, on the flow depth at the outlet:

If inlet control governs the headwater, the flow depth can be approximated by the normal depth of open-channel flow in the barrel, computed by Manning's equation for the discharge flow, barrel size, roughness and slope of culvert selected. For flow in pipes, Figure B 1 and Figure B 2 in Appendix 7A may be used.

If outlet control governs the headwater, the flow depth will be one of the following:

- Critical depth if the tailwater is below critical depth (Figure B 3 to Figure B 6 appendix 7B);
- Tailwater depth if the tailwater is between critical depth and the top of the barrel;
- The height of the barrel if the tailwater is above the top of the barrel.

7.15 Culvert End Treatment

Culvert end treatments, including the shape of culvert ends, end structures such as wingwalls, cut-offs and anchorages, and erosion control measures for the adjoining fill and channel, may be required to perform one or more of the following functions:

- Prevent fill from encroaching on the culvert opening;
- Prevent erosion of the fill and adjacent channel;
- Prevent undermining of culvert ends;
- Inhibit seepage and piping through the bedding and backfill;
- Meet traffic safety requirements;
- Improve the appearance of large culverts;
- Resist hydraulic uplift forces on corrugated metal pipe culverts; and
- Strengthen the ends of large flexible culverts, especially those with mitred or skewed ends.

7.16 Typical End Treatments

Headwalls and wingwalls are the most common end treatment. An apron is generally incorporated between the wingwalls to limit scour of the streambed. They are usually constructed from reinforced concrete, but can be formed from masonry, rock-filled gabions or mattresses, or concrete-filled mattresses.

Mitred ends are generally limited to corrugated metal pipe culverts, where the end of the pipe is cut parallel to the slope of the embankment. The area of embankment around the ends of the culvert is usually paved with concrete or rock.

Projecting ends are used where the ends of the culvert project from the face of the embankment. Although they are the least costly end treatment, they are not commonly used because they do not meet safety requirements and are visually objectionable.

7.17 Scour Issues

7.17.1 Estimation of Scour Depths

Flow from culverts will generally have a higher velocity than that of the natural watercourse and this can result in erosion of the bed and banks of the channels. When the depth and/or extent of the scour hole is such that it undermines the foundations of the structure or its outlet wing walls, structural damage can occur leading to collapse.

The potential for scour should be assessed for all culverts. The maximum scour depth should be determined for the natural bed material and then compared with the depth of the culvert foundation level to assess the risk of failure. The following design formulae are suggested for estimating maximum scour depths, y_s, and the extent of the scour hole, L_s, for the two distinct cases of rectangular culverts or pipes producing two dimensional (2D) jets and circular or square culverts producing three dimensional (3D) jets: It is recommended that HEC-RAS or other similar software should be used to assess scour. These formulae have been added here for the drainage engineer to have an understanding of theoretical principles. For detailed information, the designer should refer to relevant text books.

7.17.2 2D Jets Scour Depth

Use the equation due to Hoffmans (1997).

$$y_s = \left(\frac{50}{k}\right)\left(1 - \frac{U_2}{U_1}\right) y_1$$

where y_s is the depth of scour below the invert of the culvert (in m), y_1 is the vertical thickness of the jet(in m), U_1 is the depth-averaged velocity in the jet, and U_2 is the depth-averaged velocity in the receiving channel (both in m/s). k is a non-dimensional scour factor dependent on the d 90 size of the sediment (for which 90% of the material by weight is fines) in the channel bed (in mm) and is defined as:

$$k = 2.95{d_{90}}^{1/3} \; for \; 0.1mm < d_{90} < 12.5mm$$

$$k = 6.85 \; for \; d_{90} > 12.5mm$$

Scour Extent

The overall length of the scour hole can be estimated to be 5 to 7 times the scour depth.

7.17.3 3D Jets Scour Depth

Use the equation due to Ruff et al (1982)

$$y_s = 2.07D\left(\frac{Q}{\sqrt{gD^5}}\right)^{0.45}$$

where y s is the depth of scour below the invert of the outfall or culvert (in m), Q is the flow rate (in m^3/s), g is the acceleration due to gravity (m/s^2) and D is the diameter of the pipe (in m) where y s is the depth of scour below the invert of the outfall or culvert (in m), Q is the flow rate (in m^3/s), g is the acceleration due to gravity (m/s^2) and D is the diameter of the pipe (in m).

Scour Extent

The overall length of the scour hole will be approximately 7 times the scour depth. The above formulae apply to horizontal jets.

7.17.4 Design of Scour Protection

Scour protection measures reduce the vulnerability of a structure to failure by lining the bed material with a more erosion-resistant surface. Together with optimisation of the outfall layout, this is one of the most common means of avoiding or controlling scour problems. One of the most common materials used is riprap, or loose quarry stone that is placed in a controlled way to provide a blanket for scour protection.

There is a wide range of proprietary and non-proprietary scour protection materials and the choice depends on a range of factors: construction cost/ availability, environmental considerations, accessibility and construction restraints, underwater or dry construction, maintenance issues, etc. It should be noted that scour protection may need to be extended beyond the immediate vicinity of the predicted scour hole since residual turbulence can affect the stability of the bed and banks of the receiving channel (Refs 16 and 17).

Riprap

Simple guidelines for sizing riprap downstream of culverts (or outfalls) are given by Bohan (1970) ***(Ref 21):***

$$\frac{d}{D} = 0.25F_c \; for \; y_T < \frac{D}{2}$$

$$\frac{d}{D} = 0.25F_c - 0.15 \; for \; y_T \geq \frac{D}{2}$$

where d can be taken as the d_{50} size of the stone, D is the pipe diameter, y_T is tailwater depth and Fc is the Froude number of the flow discharging from the outfall or culvert:

$$F_c = \frac{U_1}{\sqrt{gD}}$$

where U_1 is the mean flow velocity at the culvert outlet and g is the acceleration due to gravity. The length of the scour protection blanket, Lp is dependent on the value of Fc:

$$for \; F_v \leq 1$$

$$\frac{L_p}{D} = 8$$

$$for \; F_c > 1$$

$$\frac{L_p}{D} = 8 + 17log_{10}F_c \; for \; y_T < \frac{D}{2}$$

$$\frac{L_p}{D} = 8 + 55log_{10}F_c \; for \; y_T \geq \frac{D}{2}$$

In order to secure the scour protection blanket in place, this should be turned downwards into the bed at its downstream end for a distance of at least one pipe diameter In some cases it may be more economical to include an energy dissipation measure downstream of the pipe or culvert to reduce the energy of the flow (see, for example, Peterka, 1978, for design guidance and layout details **-** *Ref 19*)**.**



Photo 7-7: Outlet Protection with Riprap

7.18 Managing Sediment

Sediment deposits within culverts, especially multi-cell culverts, can cause significant operational and maintenance problems.

Occasionally sediment traps (basins) are constructed upstream of culverts. In these cases, an access ramp for maintenance must be provided to allow de-silting of the trap.

In critical areas, or for long culverts where maintenance is extremely difficult, a small sediment trap/weir can be constructed at the inlet to divert low flows to just one or two culvert cells. This will allow the flow to enter the remaining cells only during high flows.

These sediment weirs should be designed to be fully drowned during major flood events so that no adverse backwater effects occur.

It is important that sediment traps are designed so that maintenance activities are able to be undertaken in a practical and economic manner. Factors that should be considered include access to the basin for the removal of sediment and debris, access for low loaders delivering plant, and the type of plant that will be necessary (e.g. the reach of a backhoe compared to an excavator).

7.19 Debris Control

The performance of culverts can be compromised by the accumulation of debris at inlets. This accumulation can cause failure of the drainage structure, possibly resulting in over-topping of the roadway by floodwaters, with ensuing damage to the embankment or to the properties upstream and downstream of the culvert. Where there is likelihood of a culvert being so affected, consideration should be given to the installation of a debris control structure.

7.20 Improved Inlets

The capacity of a culvert operating under inlet control can be significantly increased by providing a more efficient inlet, which reduces the flow concentration at the entrance and increases the flow depth in the barrel. For culverts operating under outlet control, the entrance losses are not a significant component of the total energy losses and major inlet improvements usually are not justified.

Major types of improvements, include bevelled inlets, depressed inlets and tapered inlets. The aim of these major improvements is to increase the velocity head or the effective headwater depth.

The incorporation of a sudden drop in the inlet as a grade control structure is referred to as a ‘drop inlet’. Drop inlets can result in high approach velocities upstream of the drop. These velocities can cause safety problems and bed scour. To avoid such problems, a small weir structure can be placed at the crest of the drop, or else concrete lining may be appropriate. To avoid upstream ponding, a free-draining, low-flow, V-notch can be formed in the weir. Drop inlets are not recommended in regions where fish migration is required.



Photo7-8: Drop Inlet Structure

7.21 Safety

7.21.1 Trafic Safety

An exposed culvert end (projecting from the plane of the batters) acts as an unyielding obstruction, likely to bring an out of control vehicle to an abrupt stop, causing considerable damage to the vehicle and high deceleration forces on the occupants.

Where a road safety barrier is not provided, culvert ends should be designed so that they will not present an obstruction to vehicles running off the road. This can be achieved by covering exposed sides with fill, providing headwalls or wingwalls which will not present an obstruction, or mitring culvert ends flush with the embankment surface.

The location of culvert ends placed flush with the embankment slope should be indicated by markers to reduce hazards to equipment operators and others. High culverts in populated areas should be fenced whenever possible.

The hazard presented by culverts under private and side-road entrances should be minimised by placing them as far as practicable from the roadway, and avoiding the use of headwalls.

7.21.2 Child Safety

Culverts can also be an attraction for adventurous and inquisitive children. At locations where long culverts could be a hazard, especially in urban areas, fencing, swing gates or grates at upstream ends should be considered to prevent entry. However, this may cause blockages and reduce the efficiency of the culvert.

7.22 Design Limitations

Allowable Headwater: is the depth of water that can be ponded at the upstream end of the culvert that will be limited by one or more of the following:

- Will not damage up stream property;
- Not higher than 300 mm below the edge of the shoulder;
- Equal to an HW/D not greater than 1.5;
- No higher than the low point in the road grade; and
- Equal to the elevation where flow can be diverted around the culvert.

Review (Check) Headwater: the review headwater is the flood depth that:

- Does not exceed 500 mm increase over the existing 100-year flood in the vicinity of buildings or habitations; and
- Has a level of inundation that is tolerable to upstream property and roadways for the review discharge.

Tailwater Relationship – Channel

- Evaluate the hydraulic conditions of the downstream channel to determine a tailwater depth for a range of discharges which includes the review discharge (see Chapter 6);
- Calculate backwater curves at sensitive locations or use a single cross section analysis;
- Use the critical depth and equivalent hydraulic grade line if the culvert outlet is operating with a free outfall; and
- Use the headwater elevation of any nearby, downstream culvert if it is greater than the channel depth.

Tailwater Relationship – Confluence or Large Water Body

- Use the high water elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent); and
- If statistically independent, evaluate the joint probability of flood magnitudes and use a likely combination resulting in the greater tailwater depth.

Maximum Velocity: the maximum velocity at the culvert exit shall be consistent with the velocity in the natural channel or shall be mitigated with:

- Channel stabilization (see Chapter 6); and
- Energy dissipation (see Chapter 9).

Minimum Velocity: the minimum velocity in the culvert barrel should result in a tractive force ($\tau=\gamma dS$) greater than critical τ of the transported streambed material at low flow rates.

- Use 0.8 meters per second when streambed material size is not known;
- If clogging is probable, consider installation of a sediment trap or size culvert to facilitate cleaning.

Storage (Temporary or Permanent): if storage is being assumed upstream of the culvert, consideration shall be given to:

- The total area of flooding;
- Limiting the average time that bank-full stage is exceeded for the design flood, to 48 hours in rural areas, or 6 hours in urban areas; and
- Ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

Flood Frequency: the flood frequency used to design or check the culvert shall be based on:

- The values given in Table 2-1, Chapter 2;
- An economic assessment or analysis to justify the flood frequencies greater or lesser than the minimum flood frequencies listed in Table 2-1 in Chapter 2.

7.23 Microcomputer Solution

Culvert hydraulic analysis can also be accomplished with the aid of the HY8 Culvert Analysis Software, version 7.2. Further explanations and worked example about this software are given in Chapter 15. There are several other types of software that can be used to ease the design process, e.g. ISIS, Mike 11 etc.

7.24 Flood Routing Culvert Design

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the backwater of the culvert. The findings from culvert analyses will be conservative if flood routing is not considered. If allowable headwater is selected without considering flood routing then costly over-design of both the culvert and outlet protection may result depending on the amount of temporary storage involved. There are many ramifications associated with culvert flood routing, including:

- Right-of-way of the upstream property may be required;
- There is a perceived loss of a subjective safety factor;
- There are credibility concerns, both in legal as well as in technical negotiations;
- Environmental concerns must be evaluated;
- Realistic assessments need to be made of potential flood hazards; and
- An estimate of sediment problems must be made.

Ignoring temporary storage effects by reducing the selected design flood magnitude and assuming that this provides a factor of safety is not recommended. This practice results in inferior safety factors at the culvert site, as the culvert is then dependent on the amount of temporary storage available. Further, with little or no temporary storage at a site, the factor of safety would be one, or, essentially, there would be no factor of safety. If a factor of safety is desired, it is essential that flood routing practices be used to insure consistent and defensible factors of safety are used at all culvert sites. In steep terrain, there is little storage and this effect can be neglected but in flat terrain storage upstream of a culvert can dramatically reduce the peak design discharge and should be considered in culvert design.

Improved hydrology methods or changed catchment area conditions are factors that can cause an older, existing culvert to be inadequate. A culvert analysis that relies on findings that ignore any available temporary storage may be misleading. A flood routing analysis may show that what was thought to be an inadequate existing culvert is, in fact, adequate.

Often existing culverts require replacement due to corrosion or abrasion. This can be very costly, particularly where a high fill is involved. A less costly alternative is to place a smaller culvert inside the existing culvert. A flood routing analysis may demonstrate, where there is sufficient storage, that this is acceptable and that no increase in flood hazard results.

With legal proceedings or in resolving conflicting design findings it is essential that creditable and defensible practices be used. By ignoring flood routing where significant storage occurs, findings may be discredited. With legal proceedings, claims of design negligence may result depending on the nature of the case.

With culvert flood routing, a more realistic assessment can be made where environmental concerns are important. The temporary time of upstream ponding can be easily identified. This allows environmental specialists to assess whether such ponding is beneficial or harmful to local environmental features such as fisheries, wetlands, and uplands.

Potential flood hazards increase whenever a culvert increases the natural flood stage. Some of these hazards can conservatively be assessed without flood routing. However, some damages associated with culvert backwater are time dependent and thus require an estimate of depth versus duration of inundation. Some vegetation and commercial crops can tolerate longer periods and greater depths of inundation than others can. Such considerations become even more important when litigation is involved.

Complex culvert sediment deposition ("silting") problems require the application of a sediment routing practice. This practice requires a time-flood discharge relationship, or hydrograph. This flood hydrograph must be coupled with a flood discharge-sediment discharge relationship in order to route the sediment through the culvert site.

There are situations where culvert sizes and velocities obtained through flood routing will not differ significantly from those obtained by designing to the selected peak discharge and ignoring any temporary upstream storage. This occurs when:

- There is no significant temporary pond storage available (as in deep incised channels);
- The culvert must pass the design discharge with no increase in the natural channel's flood stage; and
- Runoff hydrographs last for long periods such as with irrigation flows.

7.24.1 Routing Equations

In addition to the previous Design Equations (Section 7.5), the following routing equations shall be used.

The basic flood routing equation is:

$$\boldsymbol{I - O = \frac{S}{\Delta t}\ or} \tag{7-14}$$

$$\frac{\boldsymbol{2S_1}}{\boldsymbol{\Delta t - O_1 + I_1 + I_2}} = \frac{\boldsymbol{2S_2}}{\boldsymbol{\Delta t + O_2}} \tag{7-15}$$

For a finite interval of time, Δt, equation 7.14 can be expressed by:

$$\boldsymbol{\Delta S - Q_i \Delta t - Q_0 \Delta t} \tag{7-16}$$

From these equations:

$$\frac{\boldsymbol{I_1 + I_2}}{\boldsymbol{2}} = \frac{\boldsymbol{\Delta S}}{\boldsymbol{\Delta t}} + \frac{\boldsymbol{O_1}}{\boldsymbol{2}} + \frac{\boldsymbol{O_2}}{\boldsymbol{2}} \tag{7-17}$$

Where:

$\Delta S = S_2 - S_1$

S_1 = storage volume in the temporary pond at the beginning of the incremental time period, Δt, m^3

S_2 = storage volume in the temporary pond at the end of the incremental time period, Δt, m^3

Δt = incremental routing time interval selected to subdivide hydrograph into finite time elements, s

I = average hydrograph inflow to the temporary pond during incremental time period, Δt

I_1 = instantaneous inflow to the temporary pond at the beginning of the incremental time period Δt, m^3/s

I_2 = instantaneous inflow at the end of the time period Δt, m^3/s

O = average outflow from the temporary pond during incremental time period Δt, m^3/s

O_1 = instantaneous outflow at the beginning of the time period Δt, m^3/s

O_2 = instantaneous outflow at the end of the time period Δt, m^3/s

7.25 References

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APPENDIX 7A - CONSTRUCTION DETAILS

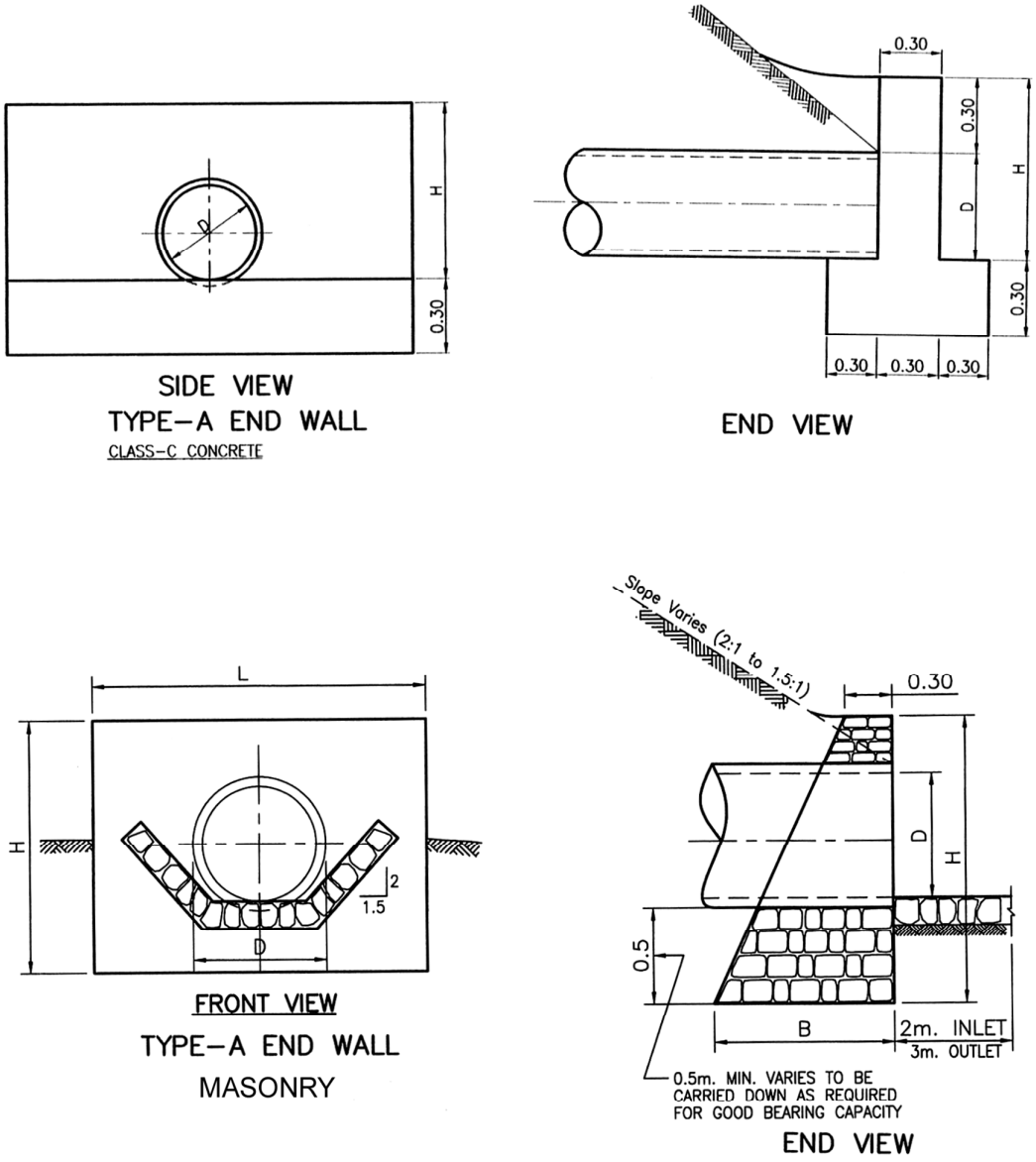


Figure 7A -1: Type A Walls Concrete or Masonry

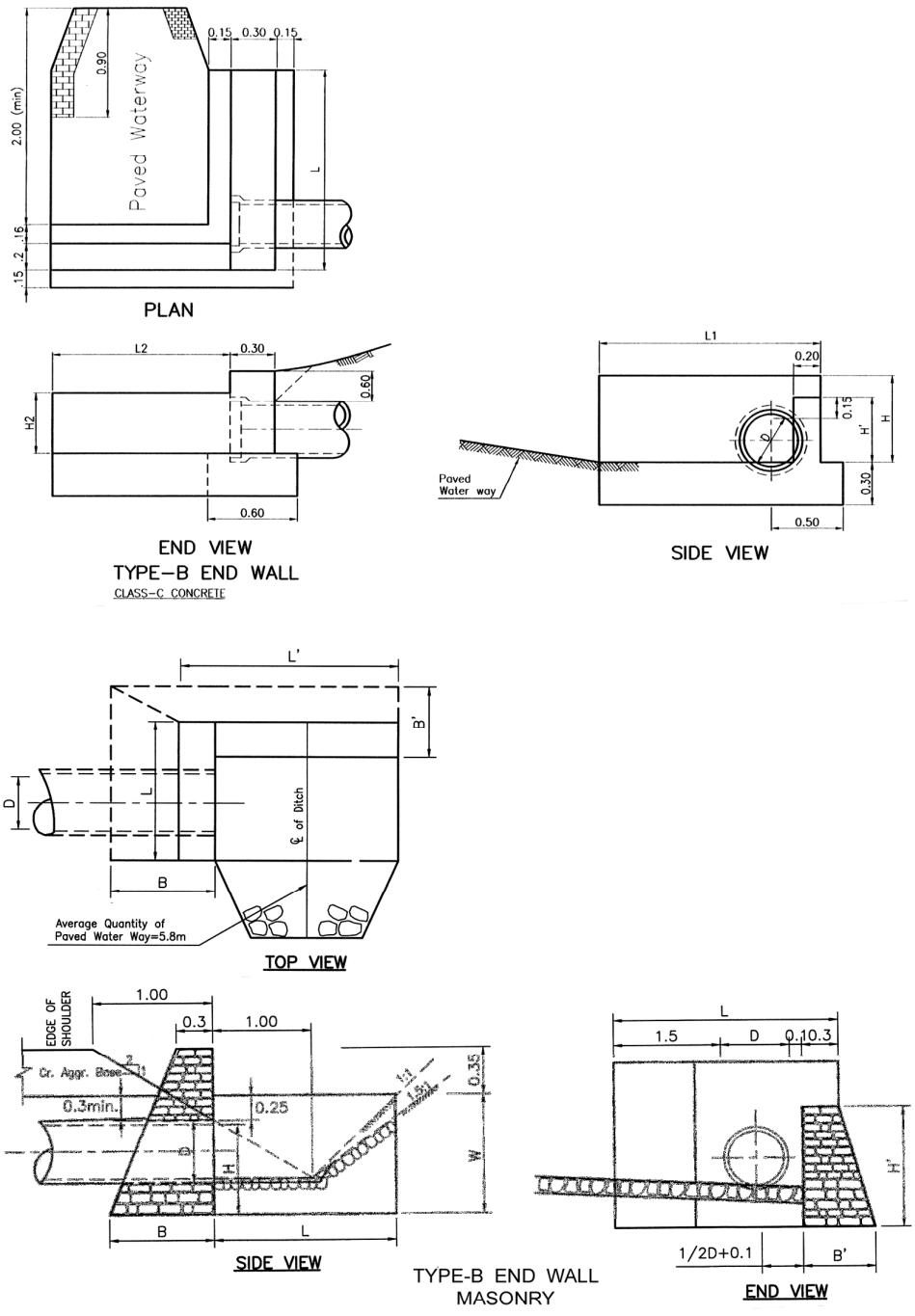


Figure 7A- 2: Type B End Walls Concrete or Masonry

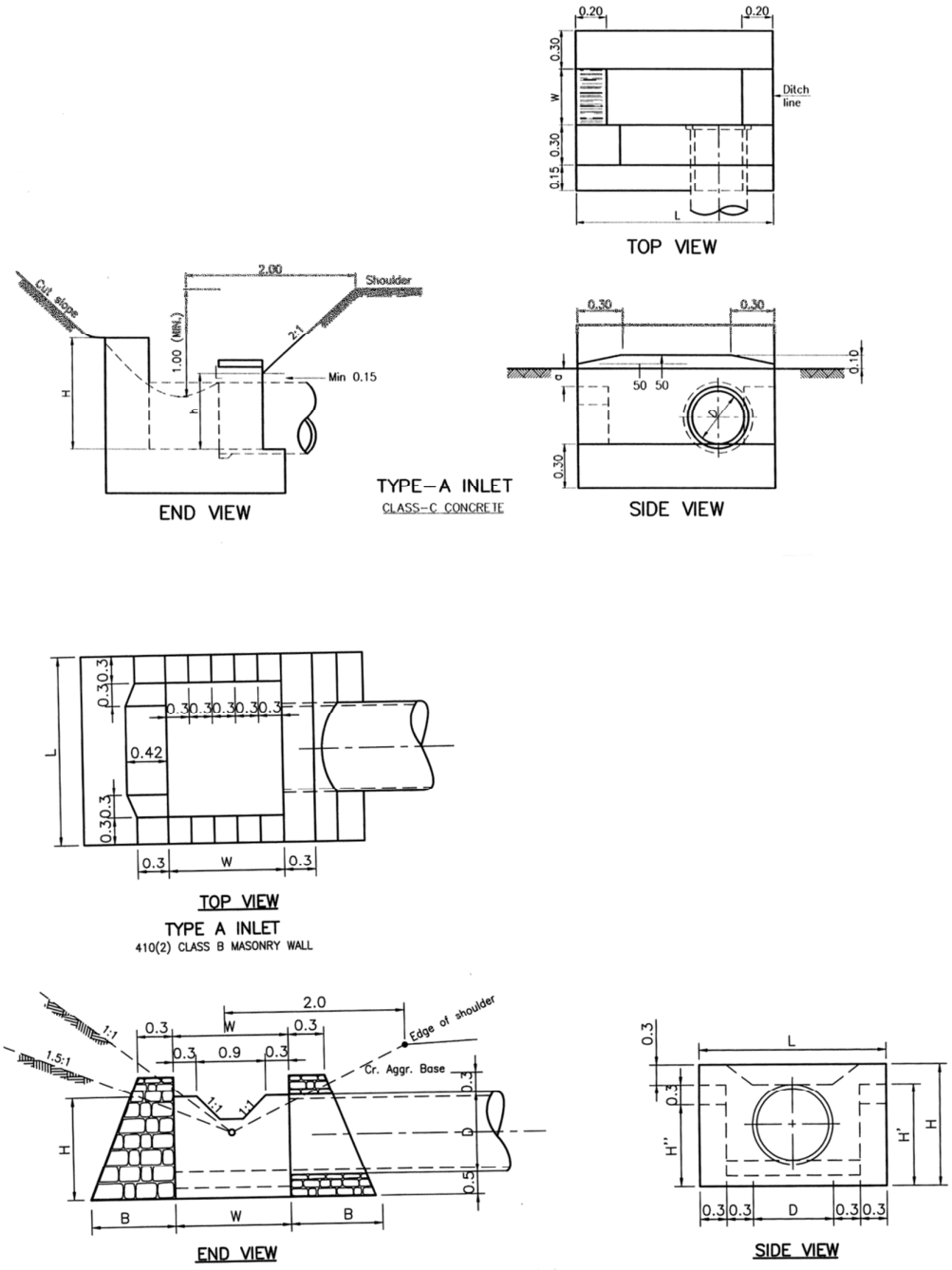
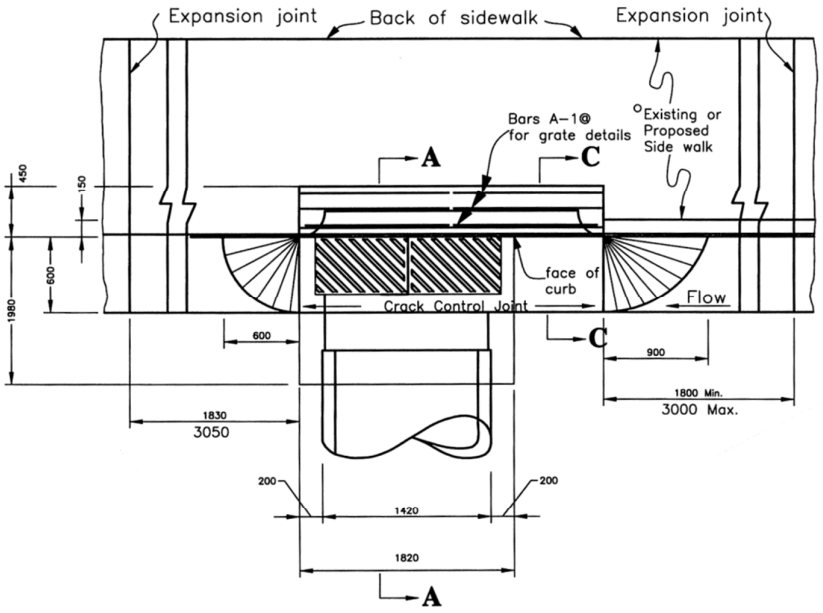
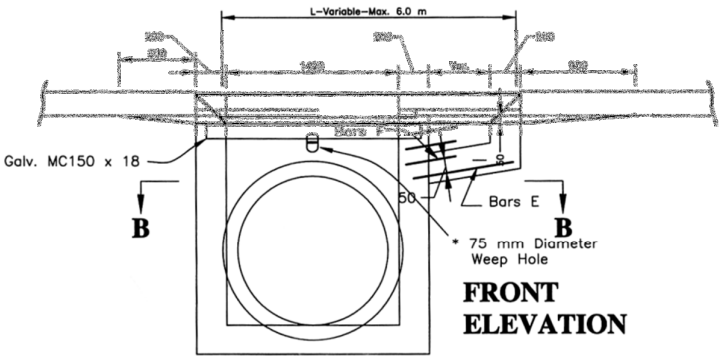


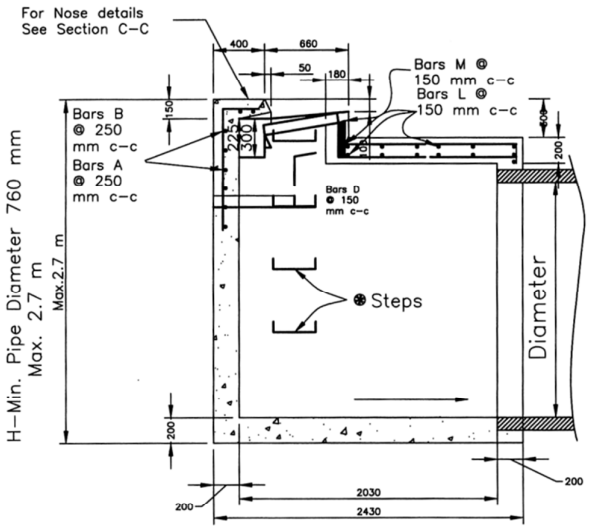
Figure 7A-3: Type A Inlet Concrete or Masonry



PLAN VIEW



FRONT ELEVATION



SECTION A-A

Figure 7A-4: Curb Drop Inlet

APPENDIX 7B - WORKED EXAMPLE AND NOMOGRAPH

HY8 - Culvert Analysis Example

Part I - Impute Data

Site Data - Culvert Crossing

Site Data Option:	Culvert Invert Data
Inlet Station:	0.00 m
Inlet Elevation:	1420.00 m
Outlet Station:	10.00 m
Outlet Elevation:	1419.83 m
Number of Barrels:	2

Tail water Channel Data - Crossing

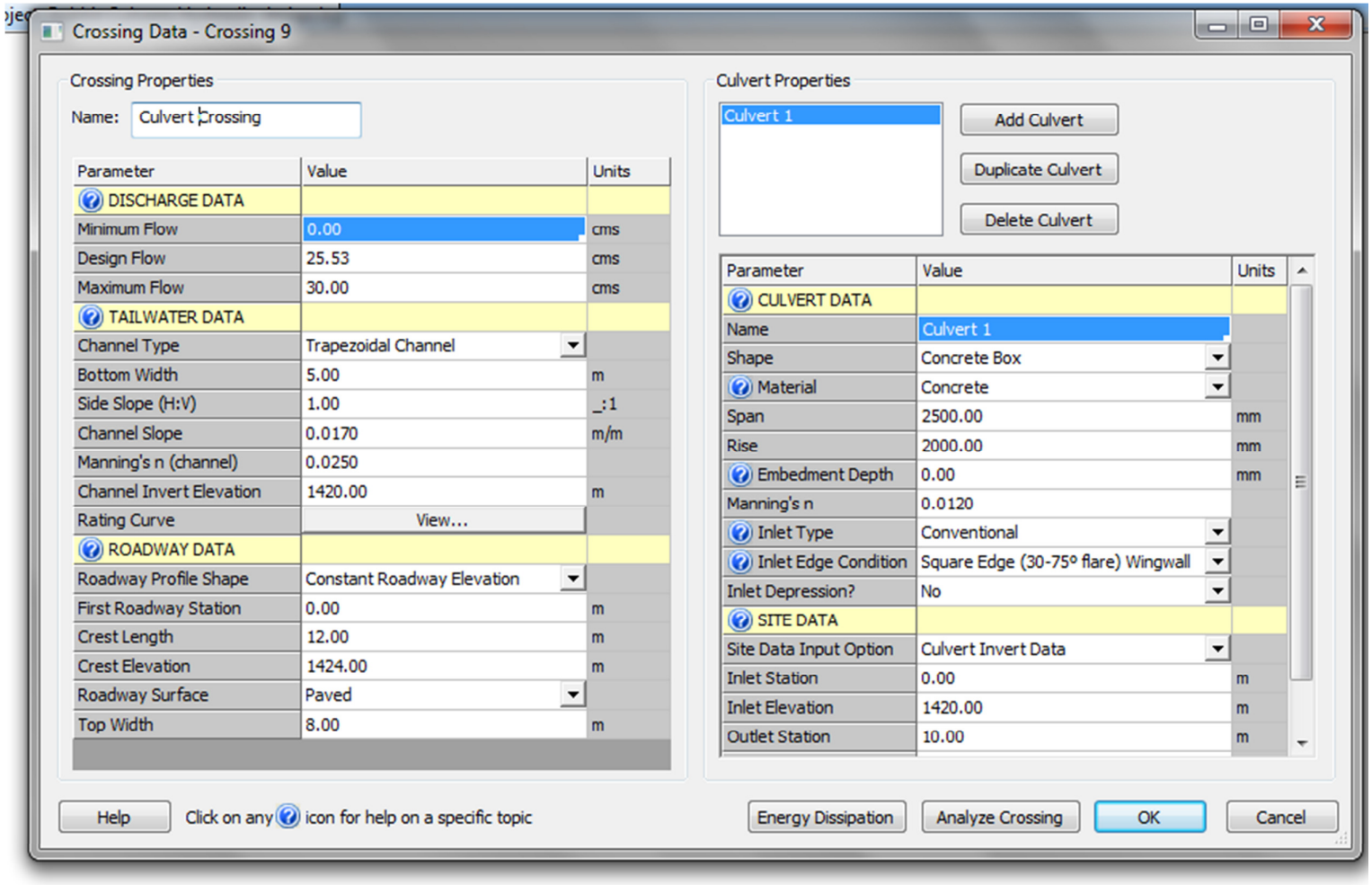
Tail water Channel Option:	Trapezoidal Channel
Bottom Width:	4.00 m
Side Slope	(H:V): 1.00
Channel Slope:	0.0170
Channel Manning's n:	0.0250
Channel Invert Elevation:	1420.00 m

Roadway Data for Crossing:

Roadway Profile Shape:	Constant Roadway Elevation
Crest Length:	12.00 m
Crest Elevation:	1424.00 m
Roadway Surface:	Paved
Roadway Top Width:	8.00 m

Culvert Data Summary - Culvert crossing

Barrel Shape:	Concrete Box
Barrel Span:	2000.00 mm
Barrel Rise:	2000.00 mm
Barrel Material:	Concrete
Embedment:	0.00 mm
Barrel Manning's n:	0.0120
Inlet Type:	Conventional
Inlet Edge Condition:	Square Edge (30-75º flare) Wingwall
Inlet Depression:	NONE


Crossing Data - Crossing 9
Crossing Properties
Name: Culvert Crossing
Parameter
Value
Units
DISCHARGE DATA
Minimum Flow
0.00
cms
Design Flow
25.53
cms
Maximum Flow
30.00
cms
TAILWATER DATA
Channel Type
Trapezoidal Channel
Bottom Width
5.00
m
Side Slope (H:V)
1.00
_:1
Channel Slope
0.0170
m/m
Manning's n (channel)
0.0250
Channel Invert Elevation
1420.00
m
Rating Curve
View...
ROADWAY DATA
Roadway Profile Shape
Constant Roadway Elevation
First Roadway Station
0.00
m
Crest Length
12.00
m
Crest Elevation
1424.00
m
Roadway Surface
Paved
Top Width
8.00
m
Culvert Properties
Culvert 1
Add Culvert
Duplicate Culvert
Delete Culvert
Parameter
Value
Units
CULVERT DATA
Name
Culvert 1
Shape
Concrete Box
Material
Concrete
Span
2500.00
mm
Rise
2000.00
mm
Embedment Depth
0.00
mm
Manning's n
0.0120
Inlet Type
Conventional
Inlet Edge Condition
Square Edge (30-75° flare) Wingwall
Inlet Depression?
No
SITE DATA
Site Data Input Option
Culvert Invert Data
Inlet Station
0.00
m
Inlet Elevation
1420.00
m
Outlet Station
10.00
m
Help
Click on any icon for help on a specific topic
Energy Dissipation
Analyze Crossing
OK
Cancel

Part II – Out put

Table: 7B-1 - Culvert Summary Table: Culvert Crossing

Total Discharge (cms)	Culvert Discharge (cms)	Head-water Elevation (m)	Inlet Control Depth (m)	Outlet Con-trol Depth (m)	Flow Type	Nor-mal Depth (m)	Critical Depth (m)	Outlet Depth (m)	Tail water Depth (m)	Outlet Velo-city (m/s)	Tail water Velocity (m/s)
0.00	0.00	1420.00	0.000	0.000	0-NF	0.000	0.000	0.000	0.000	0.000	0.000
3.00	3.00	1420.58	0.583	0.0*	1-S2n	0.215	0.386	0.262	0.314	2.862	2.216
6.00	6.00	1420.93	0.926	0.0*	1-S2n	0.337	0.613	0.444	0.475	3.382	2.822
9.00	9.00	1421.22	1.222	0.0*	1-S2n	0.448	0.804	0.602	0.605	3.736	3.232
12.00	12.00	1421.49	1.485	0.0*	1-S2n	0.547	0.974	0.747	0.717	4.014	3.548
15.00	15.00	1421.73	1.730	0.0*	1-S2n	0.642	1.130	0.883	0.818	4.247	3.808
18.00	18.00	1421.97	1.970	0.0*	1-S2n	0.732	1.276	1.012	0.910	4.446	4.031
21.00	21.00	1422.22	2.219	0.0*	5-S2n	0.820	1.414	1.135	0.995	4.626	4.226
24.00	24.00	1422.49	2.486	0.0*	5-S2n	0.904	1.546	1.253	1.075	4.789	4.400
25.37	25.37	1422.62	2.616	0.0*	5-S2n	0.942	1.604	1.305	1.110	4.860	4.474
30.00	30.00	1423.10	3.101	0.0*	5-S2n	1.069	1.793	1.478	1.222	5.074	4.701

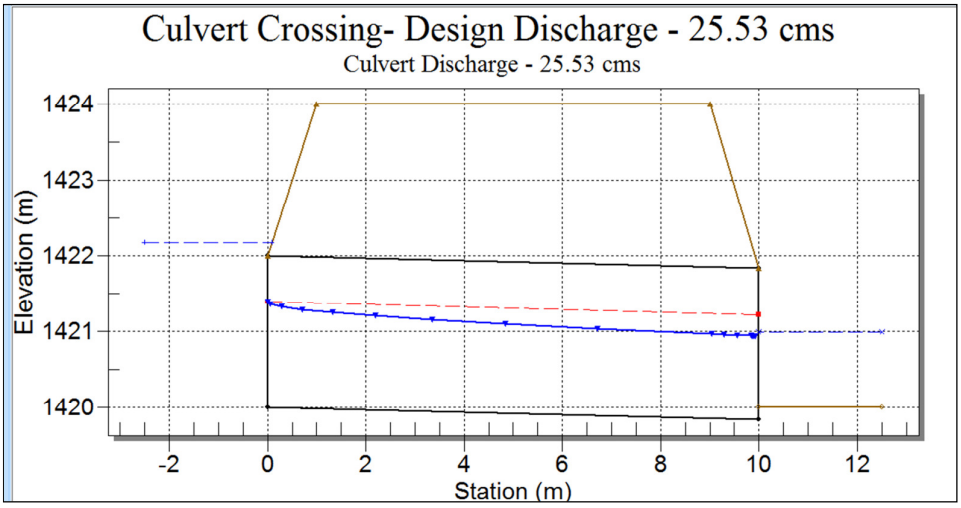


Figure 7B-1 Water Surface Profile Plot for Culvert: Culvert Crossing

* Theoretical depth is impractical. Depth reported is corrected.

Inlet Elevation (invert): 1420.00 m, Outlet Elevation (invert): 1419.83 m, Culvert Length: 10.00 m, Culvert Slope: 0.0170.

Table: 7B-2: Downstream Channel Rating Curve (Culvert -Crossing)

Flow (cms)	Water Surface Elev (m)	Depth (m)	Velocity (m/s)	Shear (Pa)	Froude Number
0.00	1420.00	0.00	0.00	0.00	0.00
3.00	1420.31	0.31	2.22	52.29	1.31
6.00	1420.48	0.48	2.82	79.18	1.37
9.00	1420.60	0.60	3.23	100.78	1.41
12.00	1420.72	0.72	3.55	119.48	1.44
15.00	1420.82	0.82	3.81	136.24	1.45
18.00	1420.91	0.91	4.03	151.57	1.47
21.00	1420.99	0.99	4.23	165.79	1.48
24.00	1421.07	1.07	4.40	179.11	1.49
25.37	**1421.11**	**1.11**	**4.47**	**184.94**	**1.50**
30.00	1421.22	1.22	4.70	203.63	1.51

Table: 7B-3: Summary of Culvert Flows at Culvert Crossing

Headwater Elevation (m)	Total Discharge (cms)	Culvert 1 Discharge (cms)	Roadway Discharge (cms)	Iterations
1420.00	0.00	0.00	0.00	1
1420.58	3.00	3.00	0.00	1
1420.93	6.00	6.00	0.00	1
1421.22	9.00	9.00	0.00	1
1421.49	12.00	12.00	0.00	1
1421.73	15.00	15.00	0.00	1
1421.97	18.00	18.00	0.00	1
1422.22	21.00	21.00	0.00	1
1422.49	24.00	24.00	0.00	1
1422.62	**25.37**	**25.37**	**0.00**	**1**
1423.10	30.00	30.00	0.00	1
1424.00	37.04	37.04	0.00	Overtopping

Nomograph Design Example

The following example problem follows the Design Procedure Steps described in Appendix 7c.

Step 1 Assemble Site Data and Project File

a. Site survey project file contains roadway profile and embankment cross section (see Figure 7B-3)

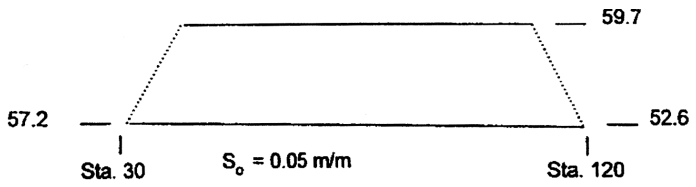


Figure 7B-3 Cross-Section

Site visit notes indicate

- No sediment or debris problems and
- No nearby structures

b. Studies by other agencies – none

c. Environmental risk assessment shows

- No buildings near floodplain
- No sensitive floodplain values and
- Convenient detours exist

d. Design criteria

- 50-year frequency for design and
- 100-year frequency for check

Step 2 Determine Hydrology

SCS equations yield

- Q50 = 11.3 m3/s
- Q100 = 14.16m3/s

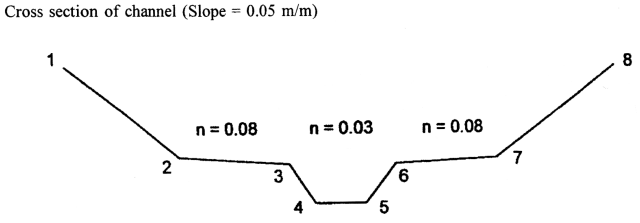


Figure 7B-4: Cross-Section of Channel

Step 3 Design Downstream Channel (see figure 7B- 4)

Point	Station, m	Elevation, m
1	3.7	54.86
2	6.7	53.34
3	9.8	53.19
4	10.4	52.58
5	11.9	52.58
6	12.5	53.19
7	15.5	53.34
8	18.6	54.86

The rating curve for the channel calculated by normal depth yields:

Q (m3/s)	TW (m)	V (m/s)
2.83	0.43	3.39
5.66	0.63	4.18
8.50	0.76	4.87
11.33	0.85	5.34
14.16	0.93	5.73

Step 4 Summarize Data on Design Form (see Figure 7B-5)

PROJECT: _____ STATION: _____ CULVERT DESIGN FORM

SHEET _____ OF _____ DESIGNER / DATE: _____ OF _____

REVIEWER / DATE: _____ OF _____

HYDROLOGICAL DATA

☐ METHOD: _____

☐ DRAINAGE AREA: _____ ☐ STREAM SLOPE: _____

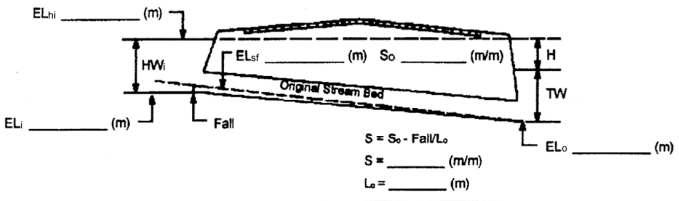
☐ CHANNEL SHAPE: _____

☐ ROUTING: _____ ☐ OTHER: _____

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (m^3/s)	TW (m)

ROADWAY ELEVATION: _____ (m)



CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (m^3/s)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	HWi	FALL (3)	ELhi (4)	OUTLET CONTROL TW (5)	dc	(dc + D)/2	ho (6)	ke	H (7)	ELho (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
RCB - 2135 x1830 mm-Bevel	*11.33*	*5.32*	*1.27*	*2.32*	*0*	*59.47*	*0.85*	*1.43*	*1.63*	*1.63*	*0.2*	*0.85*	*55.06*	*59.47*	*9.68*	*59.47 < 59.47 OK*
	14.16	*6.69*	*1.56*	*2.87*	*0*	*60.02*	*0.93*	*1.65*	*1.74*	*1.74*	*0.2*	*1.31*	*55.63*	*60.02*	*10.36*	*> 59.47 Calc Qr*
Performance Curve	*2.83*	*1.30*	*0.46*	*0.85*	*0*	*58.00*										
	5.66	*2.69*	*0.78*	*1.43*	*0*	*58.58*										
	8.50	*3.99*	*1.01*	*1.86*	*0*	*59.01*										

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW/D = HW/D OR HW/D FROM DESIGN CHARTS

(3) FALL = HW - (EL_{hw} - EL_{tw}); FALL IS ZERO FOR CULVERTS ON GRADE

(4) ELhi = HWi + ELi (INVERT OF INLET CONTROL SECTION

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h_o = TW or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = [1+k_e+(19.63\ n^2L)/R^{1.33}]\ V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUSCRIPT DEFINITIONS:
a. Approximate
f. Culvert Face
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
o. Outlet
sf. Streamed at Culvert Face
tw. Tailwater

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: _____

SHAPE: _____

MATERIAL: _____ n _____

ENTRANCE: _____

Figure 7B-5

Step 5 Select Design Alternative

Shape – box size - 2135 mm by 1830 mm

Material – concrete

Entrance - Wingwalls, 45^o bevel, rounded

Step 6 Select Design Discharge

($Q_d = Q_{50} = 11.33\ m^3/s$)

Step 7 Determine Inlet Control Headwater Depth (HW_i)

Use inlet control nomograph - Chart 7-6

a. D = 1.83 m

b. Q/B = 11.33/2.13 = 5.32

c. HW/D = 1.27 for 45^o bevel

d. HW_i = (HW/D)D = (1.27)1.83 = 2.32 m (Neglect the approach velocity)

Step 8 Determine Outlet Control Headwater Depth at Inlet (HW_{oi})

a. TW = 0.85 m for Q_{50} = 11.33 m^3/s

b. d_c = 1.43 m from Chart 7-7

c. $(d_c + D)/2 = (1.43 + 1.83)/2 = 1.63$ m

d. h_o = the larger of TW or $(d_c + D/2)$

$h_o = (d_c + D)/2 = 1.63$ m

e. K_E = 0.2 from Table 7-2

f. Determine (H) - use Chart 7-8

- K_E scale = 0.2
- culvert length (L) = 90 m
- n = 0.012 same as on chart
- area = 3.90 m^2
- H = 0.85 m

g. $HW_{oi} = H + h_o - S_oL = 0.85 + 1.63 - (0.05)90 = -2.02$ m

HW_{oi} is less than 1.2D, but control is inlet control, outlet control computations are for comparison only

Step 9 Determine Controlling Headwater (HW_c)

- $HW_c = HW_i = 2.32$ m > $HW_{oi} = -2.02$ m
- The culvert is in inlet control

Step 10 Compute Discharge over the Roadway (Q_r)

a. Calculate depth above the roadway:

$$HWr = HW_c - HW_{ov} = 2.32 - 2.59 = -0.27\text{m}$$

b. If $HW_r \leq 0$, $Q_r = 0$

Step 11 Compute Total Discharge (Q_t)

$Q_t = Q_d + Q_r = 11.33\ m^3/s + 0 = 11.33\ m^3/s$

Step 12 Calculate Outlet Velocity (V_o) and Depth (d_n)

Inlet Control

a. Calculate normal depth (d_n):

$$Q = \left(\frac{1}{n}\right) AR^{2/3}S^{1/2} = 11.33 m^3/s$$

$$= \left(\frac{1}{0.012}\right)(2.13 * d_n)\left[\frac{(2.13 * d_n)}{(2.13 + 2d_n)}\right]^{2/3}(0.05)^{0.5}$$

$$= (2.13 * d_n)\left[\frac{2.13 * d_n}{(2.13 + 2d_n)}\right]^{2/3} = 0.608$$

try d_n = 0.6 m, 0.675 > 0.608

use d_n= 0.55 m, 0.596 ≈ 0.608

b. A = (2.13)0.55 = 1.17 m^2

c. V_o = Q/A = 11.33/1.17 = 9.68 m/s

Step 13 Review Results

Compare alternative design with constraints and assumptions, if any of the following are exceeded repeat, Steps 5 through 12

- Barrel has (2.59 m – 1.83 m) = .76 m of cover
- L = 90 is OK, since inlet control
- Headwalls and wingwalls fit site
- Allowable headwater (2.59 m) > 2.32 m is ok and
- Overtopping flood frequency > 50-year

Step 14 Plot Performance Curve

Use Q_{100} for the upper limit, Steps 6 through 12 should be repeated for each discharge used to plot the performance curve, these computations are provided on the computation form, Figure 7B-6 that follows this example.

Step 15 Related Designs

Consider the following options (see Section 7.3: Design Features, and Related Designs)

a. Consider tapered inlets, culvert is in inlet control and has limited available headwater

- No flow routing, a small upstream headwater pool exists
- Consider energy dissipaters since V_o= 9.5 m/s > 6 m/s in the downstream channel
- No sediment problem

Step 16 Documentation

Report prepared and background filed

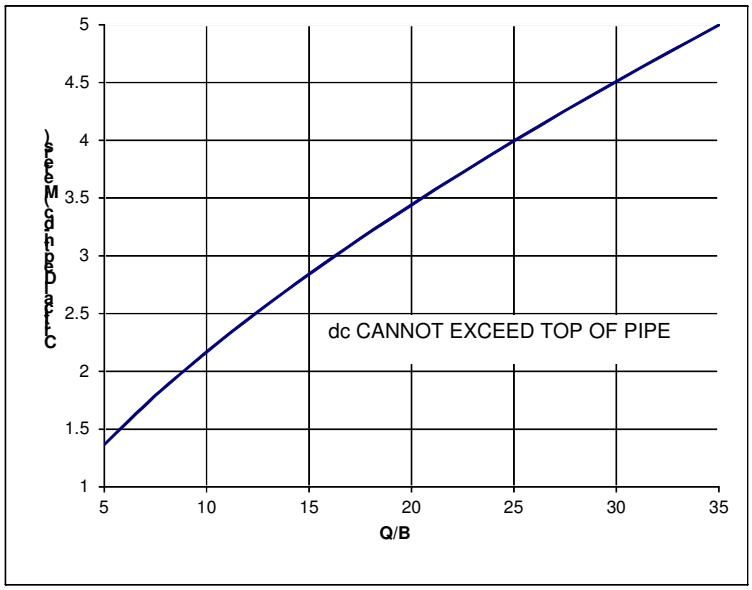


Figure 7B-6

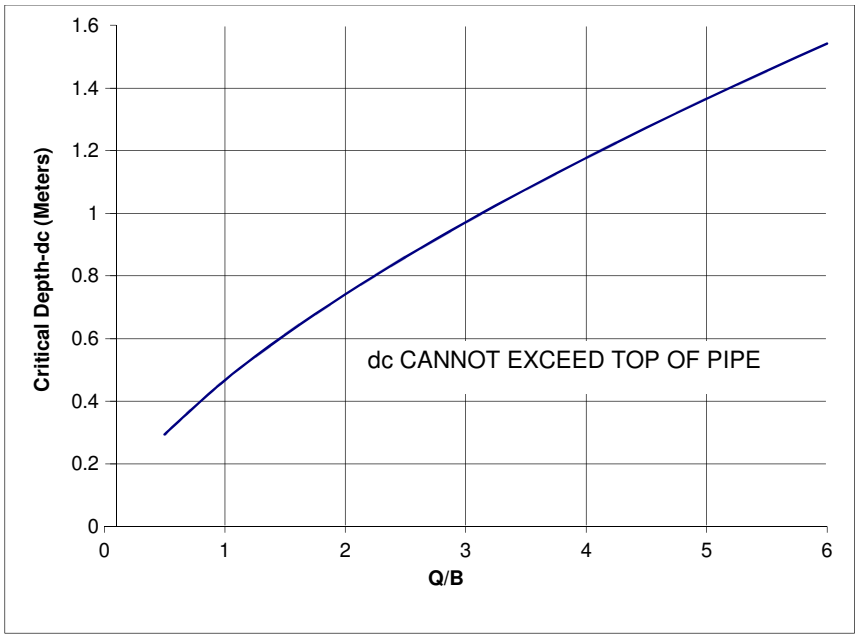
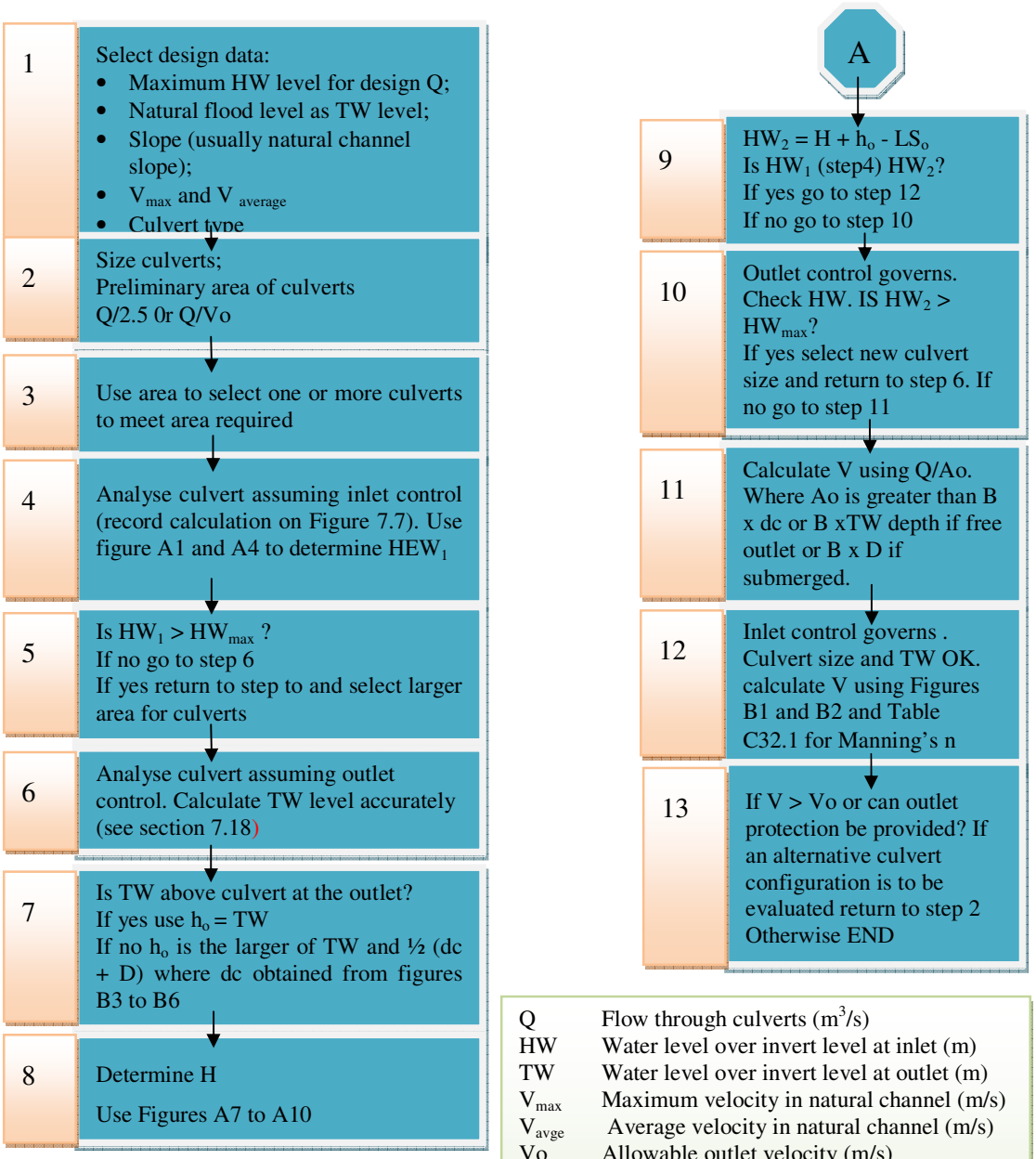


Chart 7B-1: Critical Depth – Rectangular Section

APPENDIX 7C – DESIGN PROCEDURES AND NOMOGRAMS

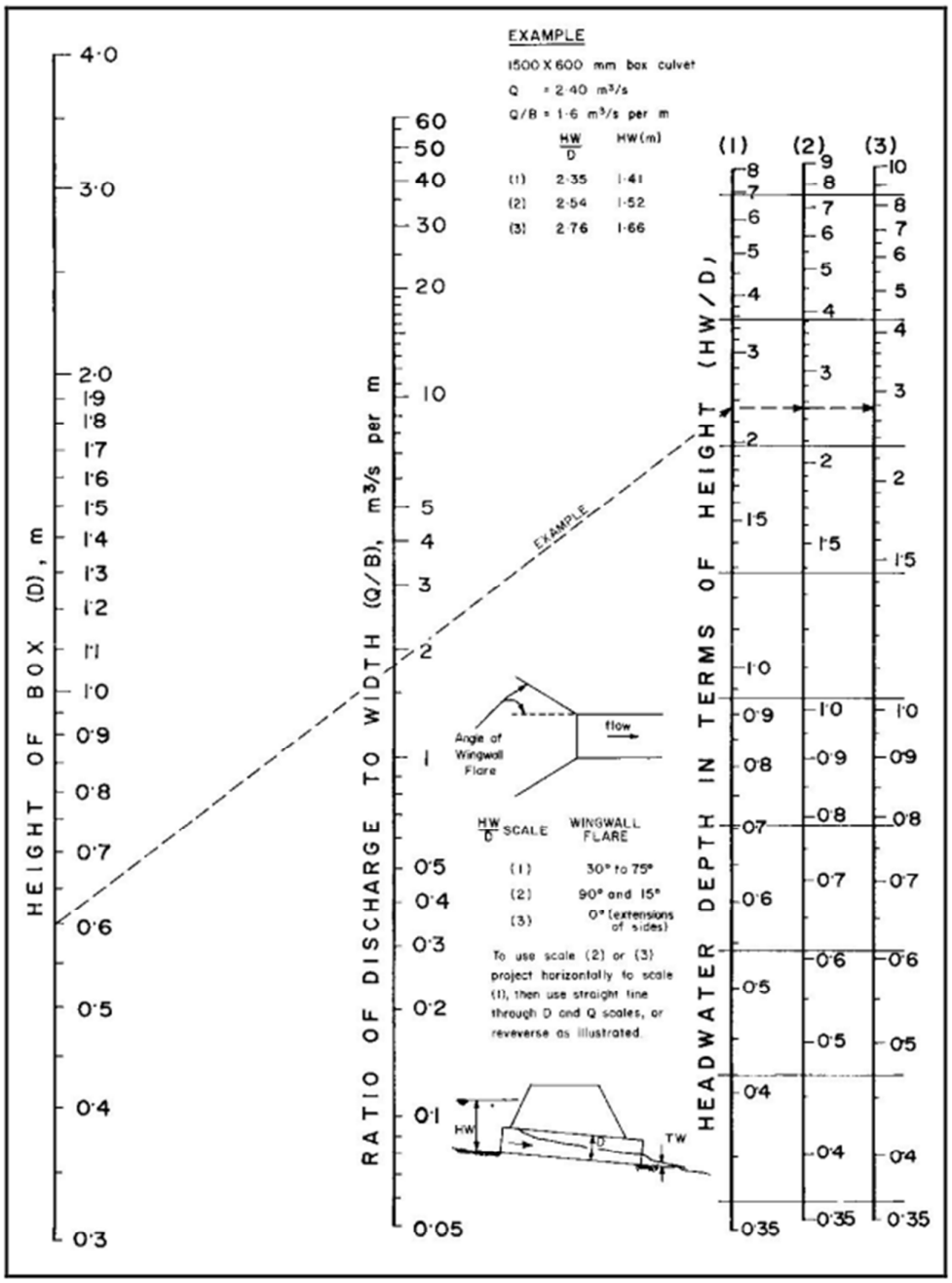
Figure 7C-1. Culvert design flow chart in steps 1 – 13



Source: DMR QLD (2002)

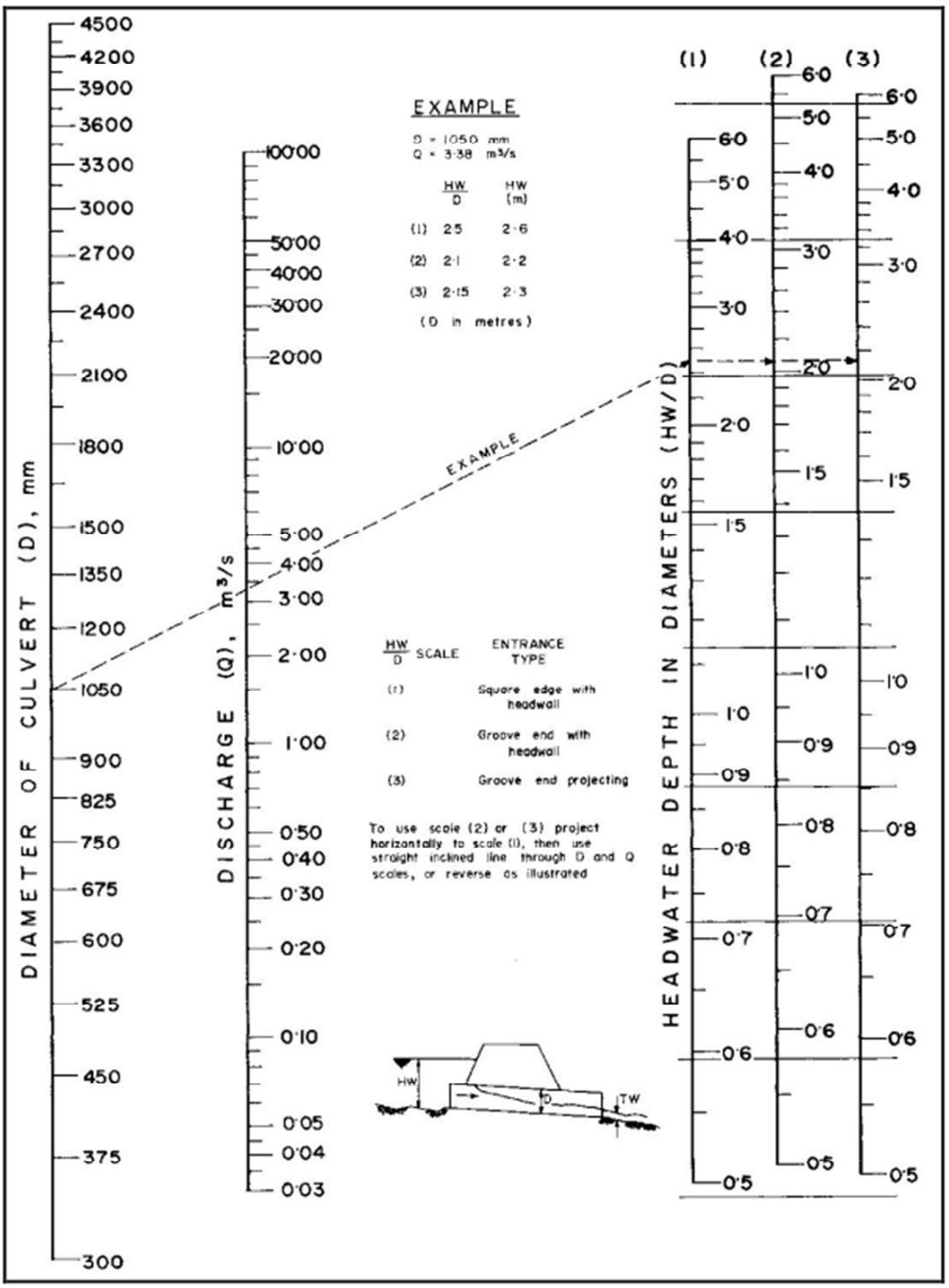
Q	Flow through culverts (m^3/s)
HW	Water level over invert level at inlet (m)
TW	Water level over invert level at outlet (m)
V_{max}	Maximum velocity in natural channel (m/s)
V_{avge}	Average velocity in natural channel (m/s)
Vo	Allowable outlet velocity (m/s)
ho	Calculated head at the outlet of the culvert over the invert (m)
B	Width of culvert (m)
D	Height depth of culvert (m)
dc	Critical depth of flow in culvert (m)
L	Length of culvert (m)
So	Slope of culvert (m/m)
V	Calculated culvert out let velocity (m/s)
H	HW - TW

A.2 Inlet Control



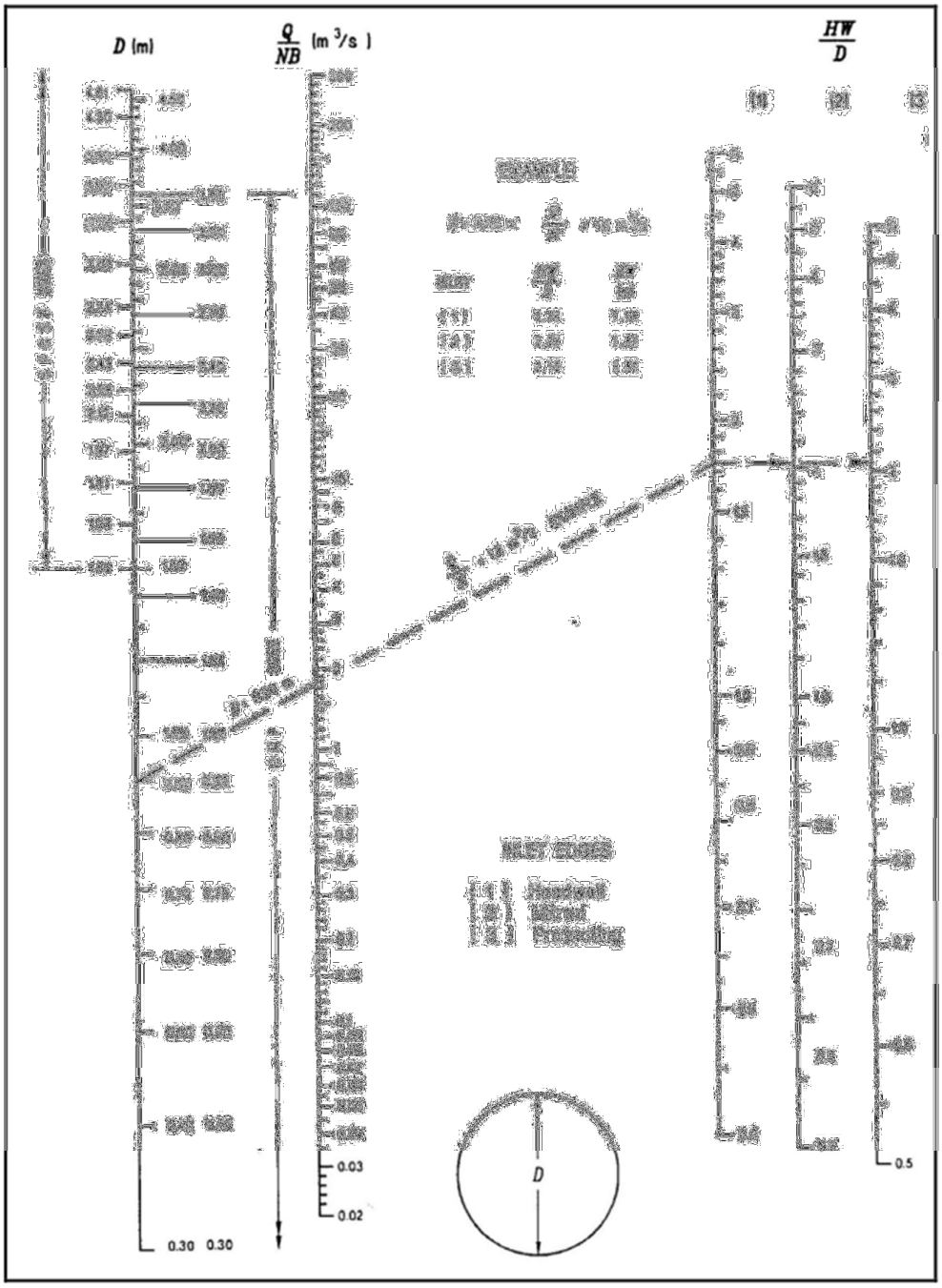
Source: DMR QLD (2002)

Figure A1: Headwater Depth for Box Culvert with Inlet Control



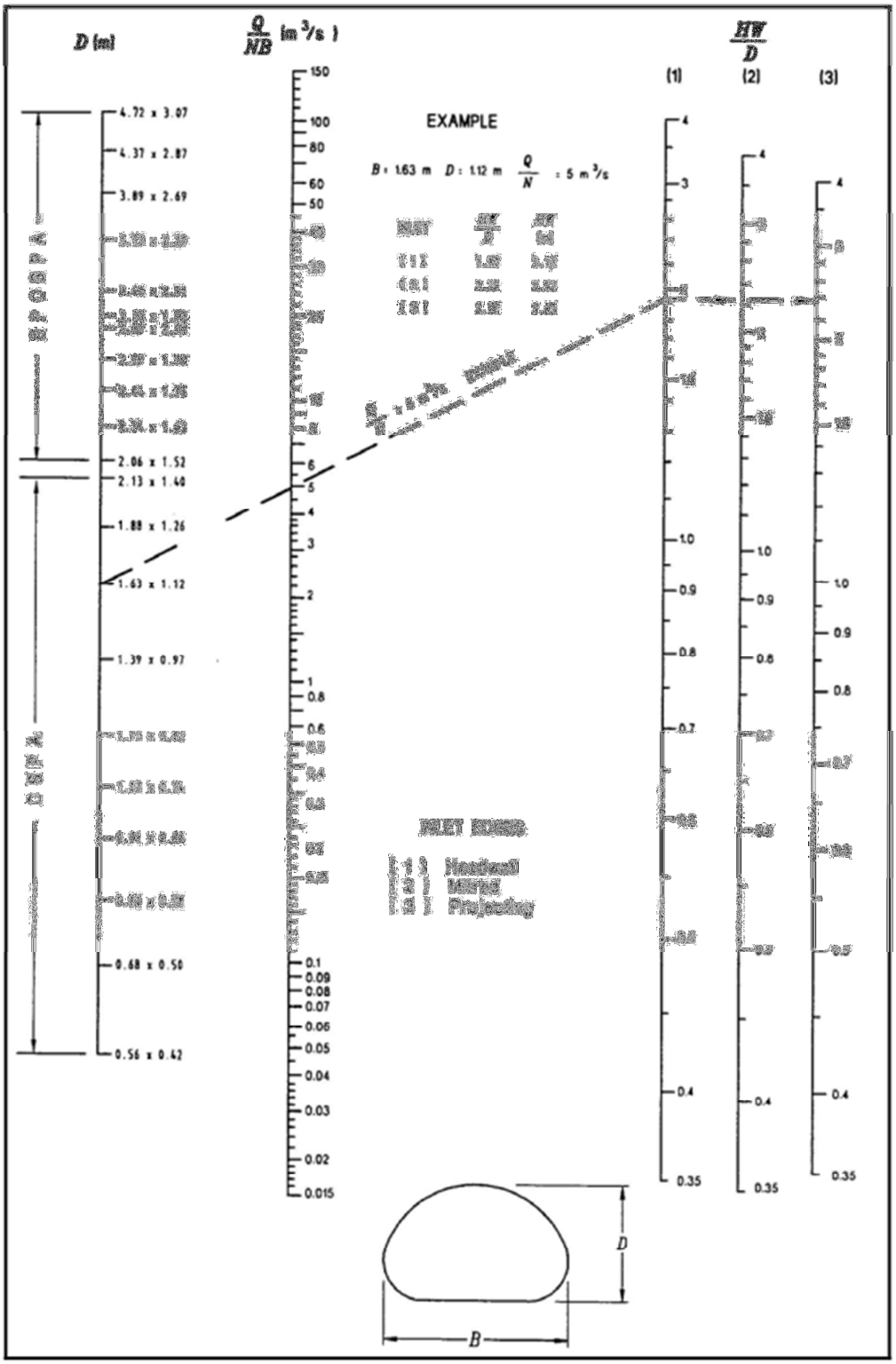
Source: DMR QLD (2002)

Figure A2: Headwater Depth for Concrete Pipe Culverts with Inlet Control



Source: DMR QLD (2002)

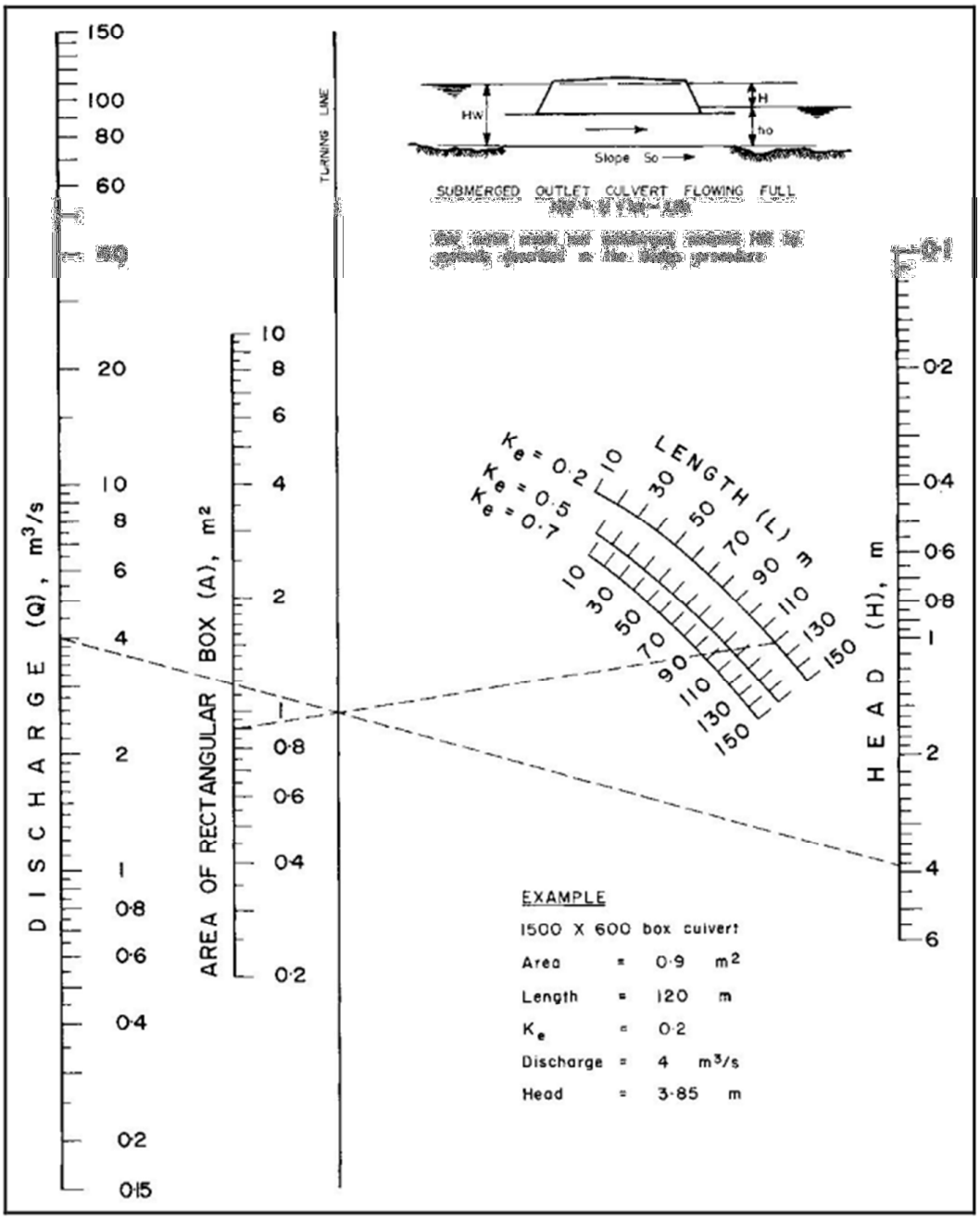
Figure A3: Inlet Control Nomograph – CMP and SPCSP Culvert



Source: DMR QLD (2002)

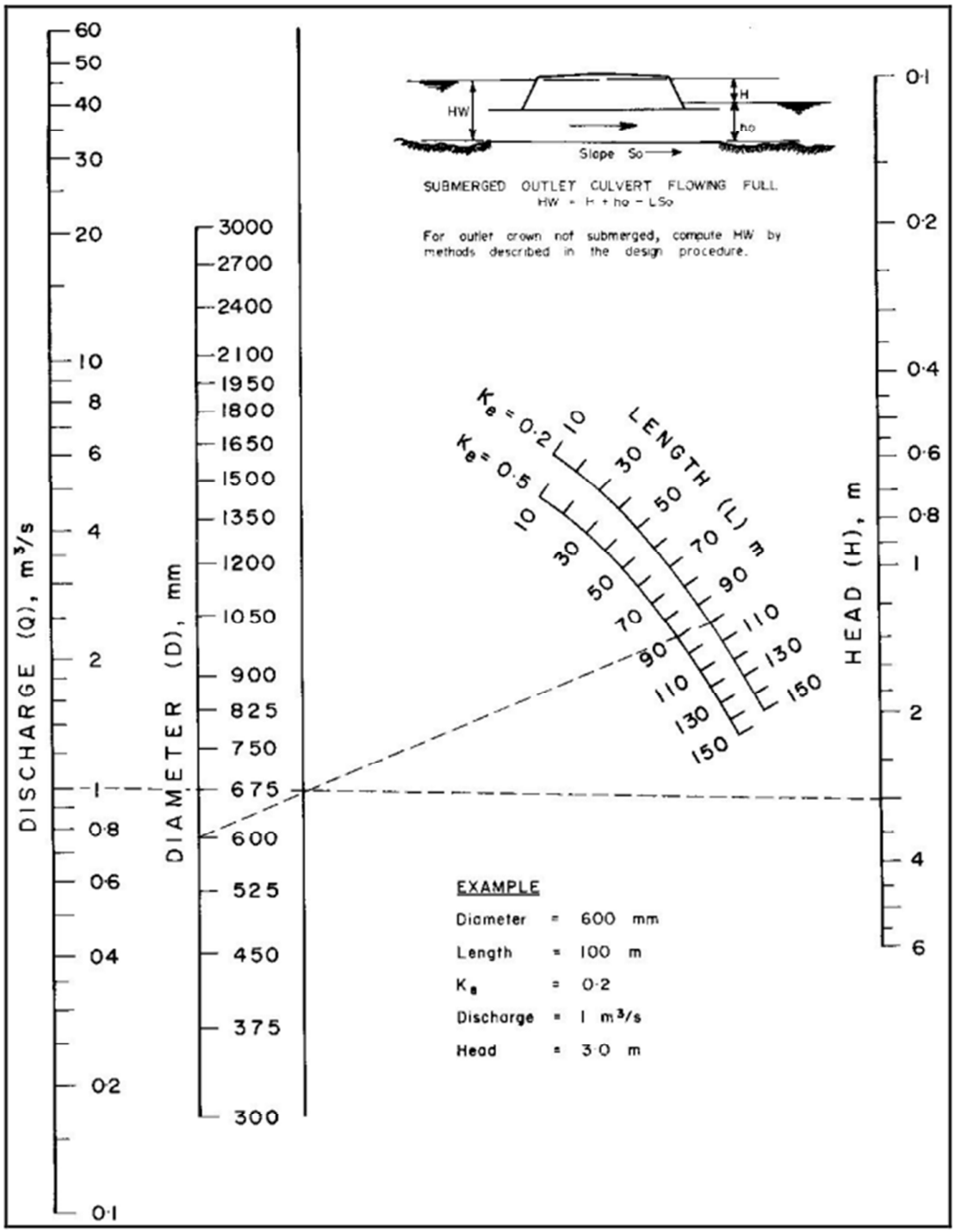
Figure A4: Intel Control Nomograph – CMPA and SPCSPA Culvert

A.3 Outlet Control



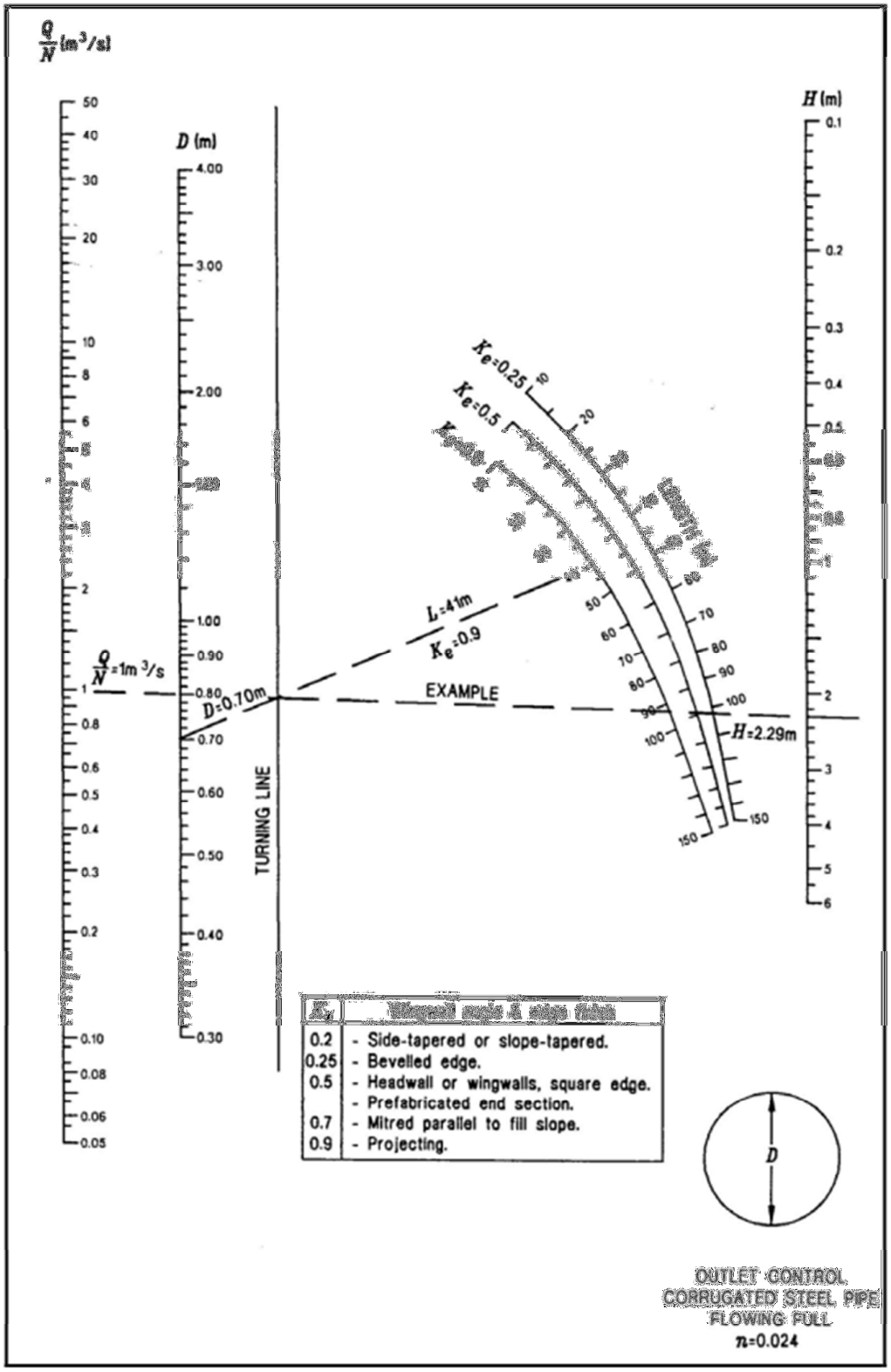
Source: DMR QLD (2002)

Figure A5: Head for Concrete Box Culverts Flowing Full with Outlet Control (n=0.012)



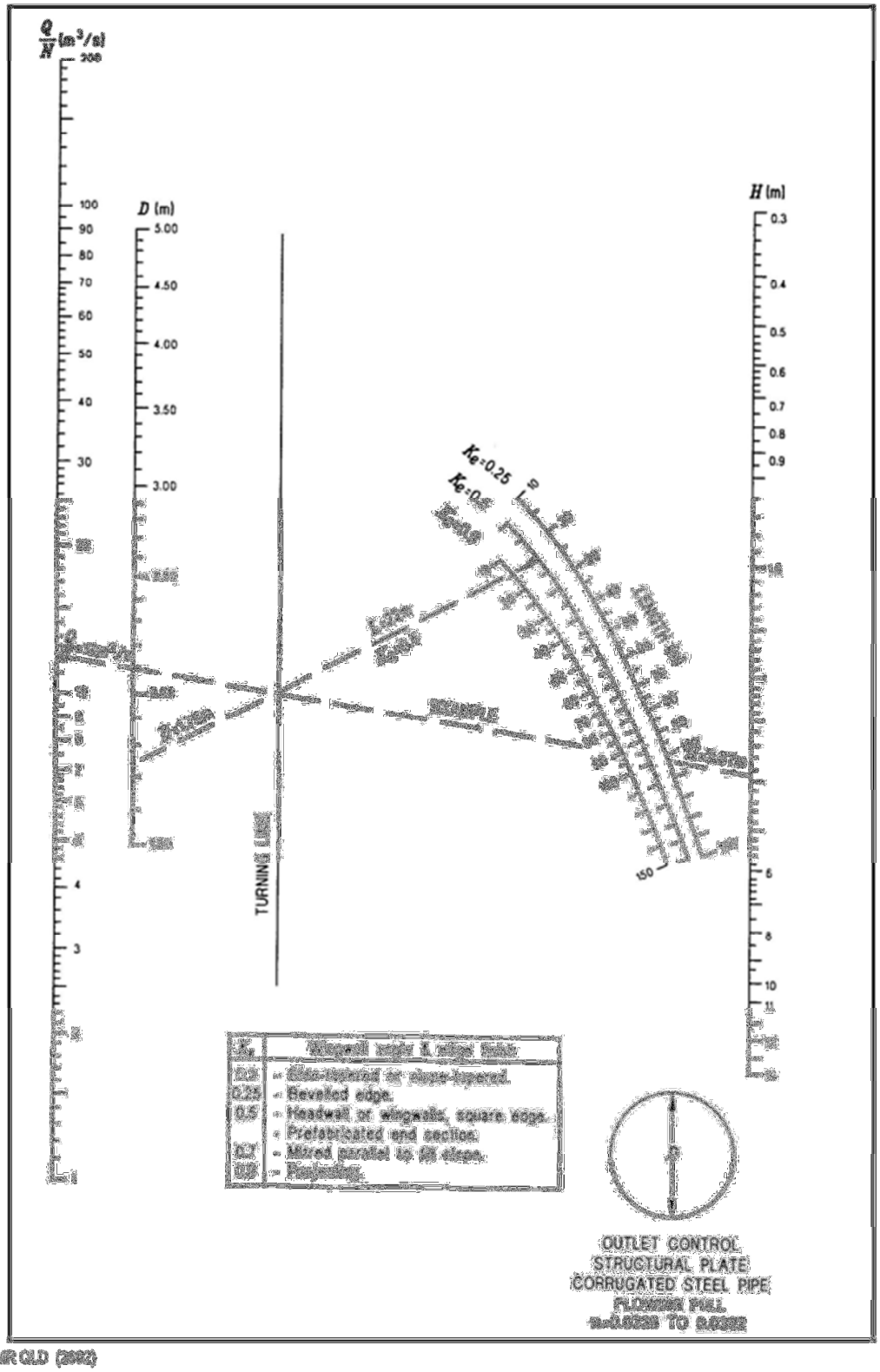
Source: DMR QLD (2002)

Figure A6: Head for Concrete Pipes Flowing Full with Outlet Control (n=0.012)



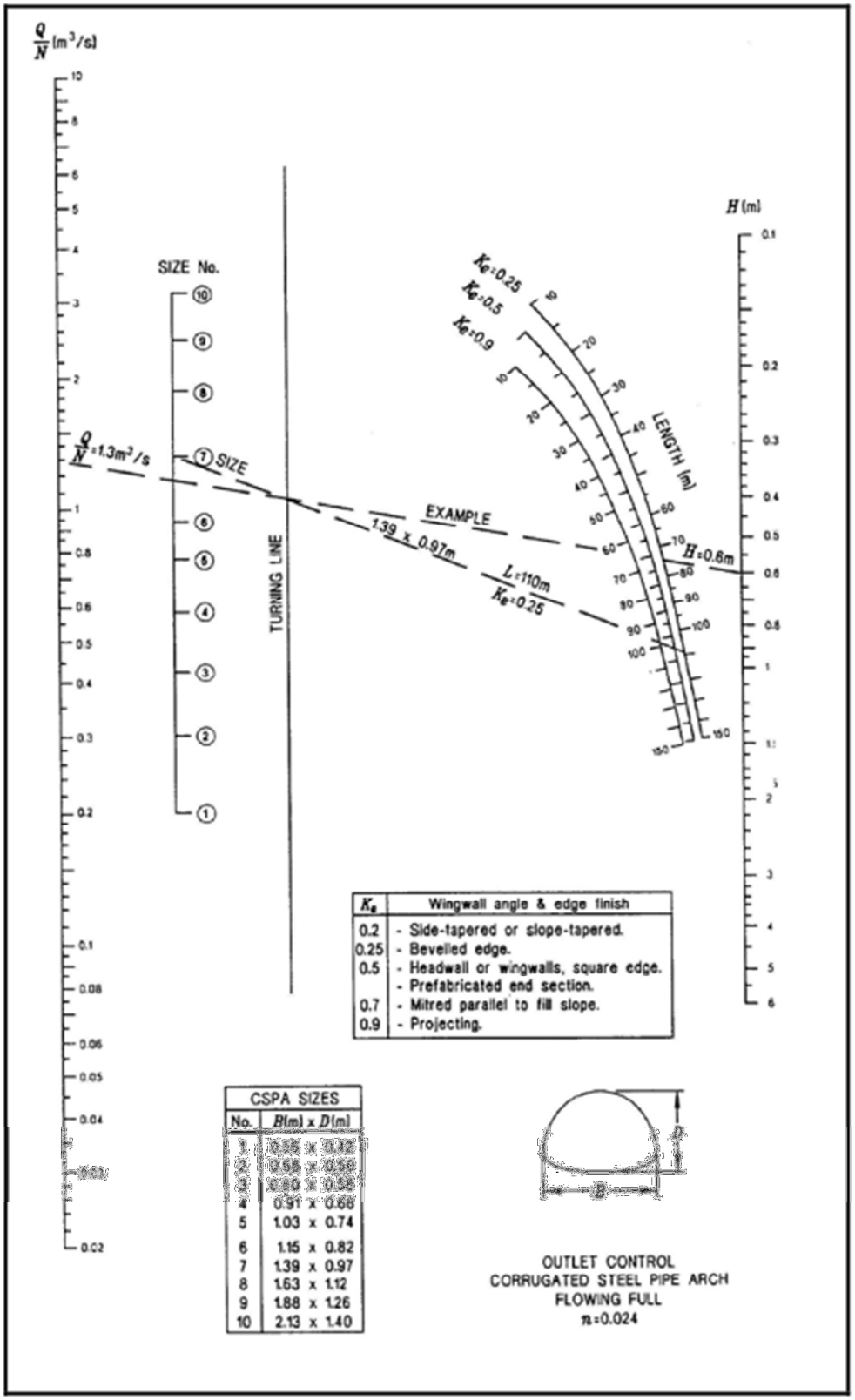
Source: DMR QLD (2002)

Figure A7: Outlet Control Nomograph – Corrugated Metal Pipe (CMP) Flowing Full (n=0.024)



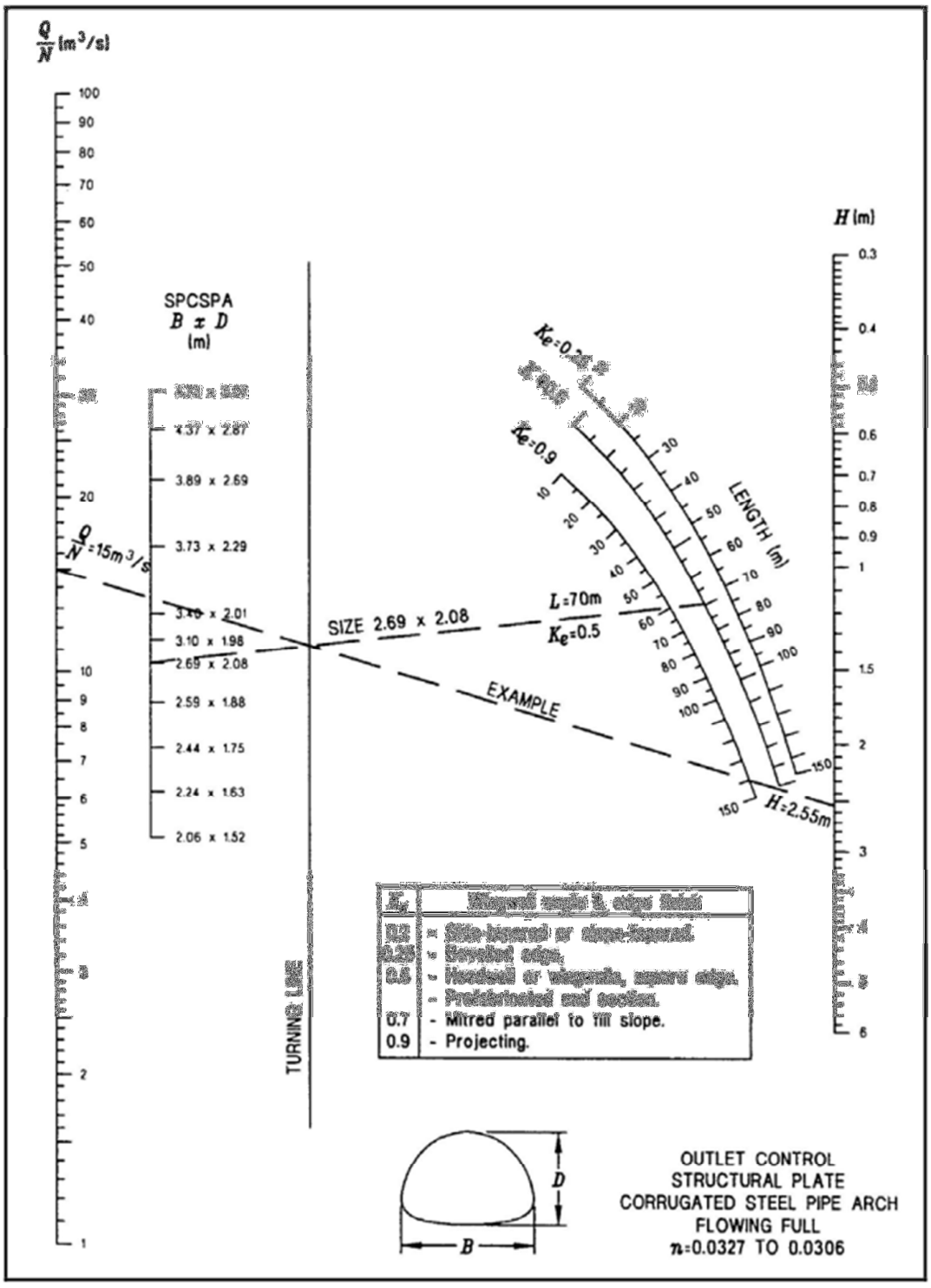
Source: DMR QLD (2002)

Figure A8: Outlet Control Nomograph – Structural Plate Corrugated Steel Pipe (SPCSP) Flowing Full (n=0.0328 to 0.030)



DMR QLD (2002)

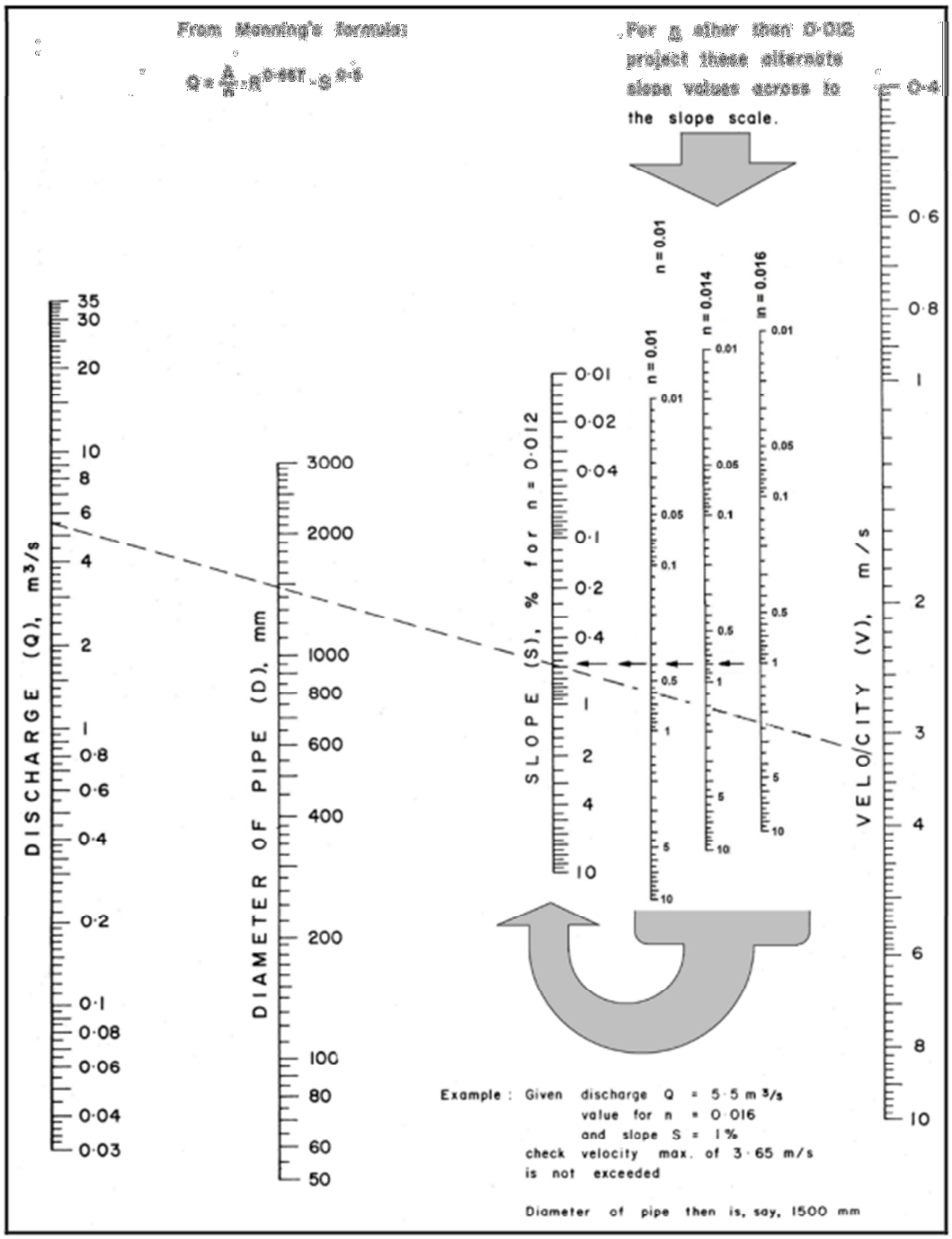
Figure A9: Outlet Control Nomograph – Corrugated Steel Pipe Arch (CSPA) Flowing Full (n=0.024)



Source: DMR QLD (2002)

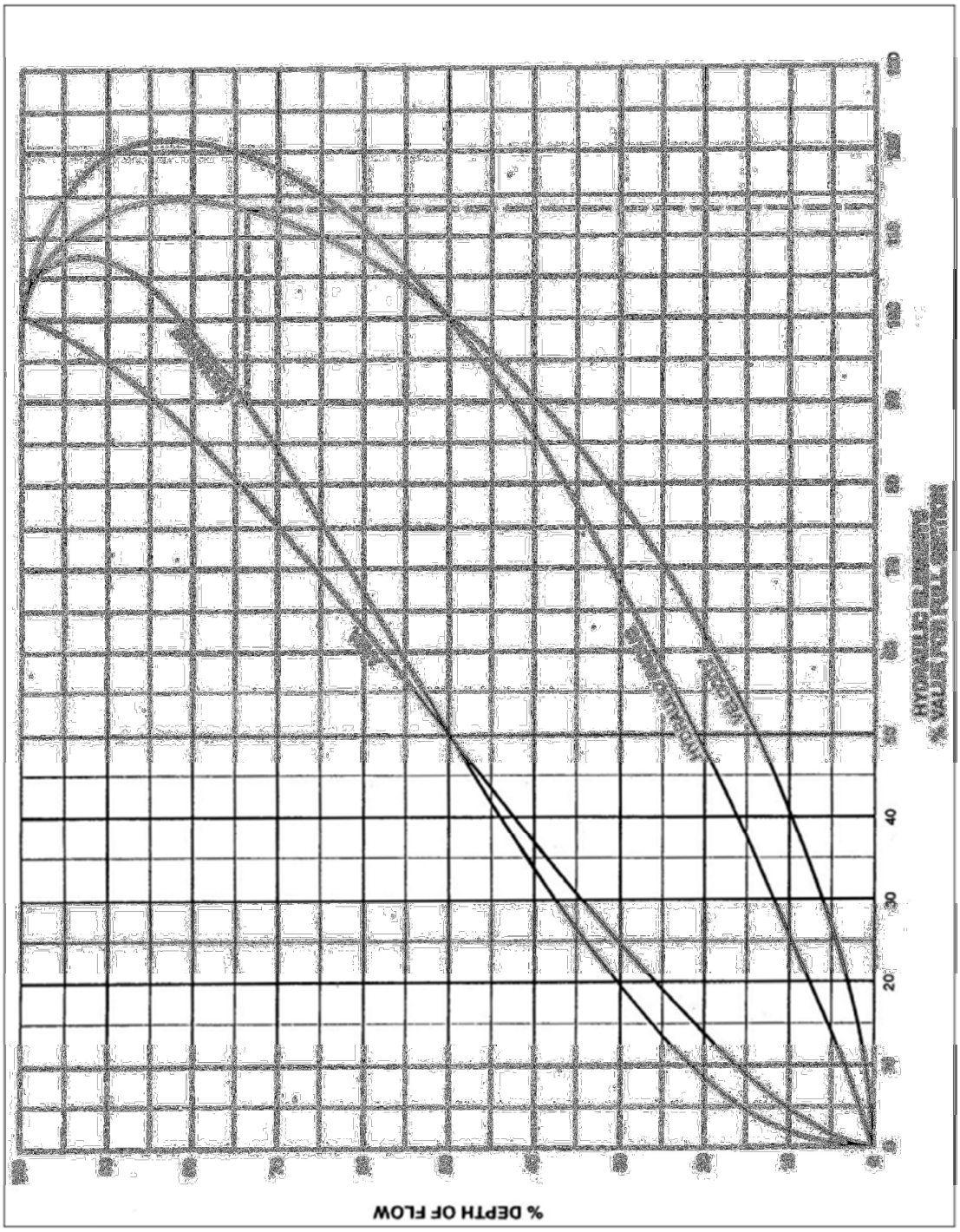
Figure A10: Outlet Control Nomograph - Structural Plate Corrugated Steel Pipe Arch (SPCSPA) Flowing Full (N=0.0327 to 0.0906)

B.2 Velocity



Source: DMR QLD (2002)

Figure B1: Discharge and Velocity in Round Pipes Flowing Full



Source: DMR QLD (2002)

Figure B2: Velocity and Discharge in Part-Full Pipes

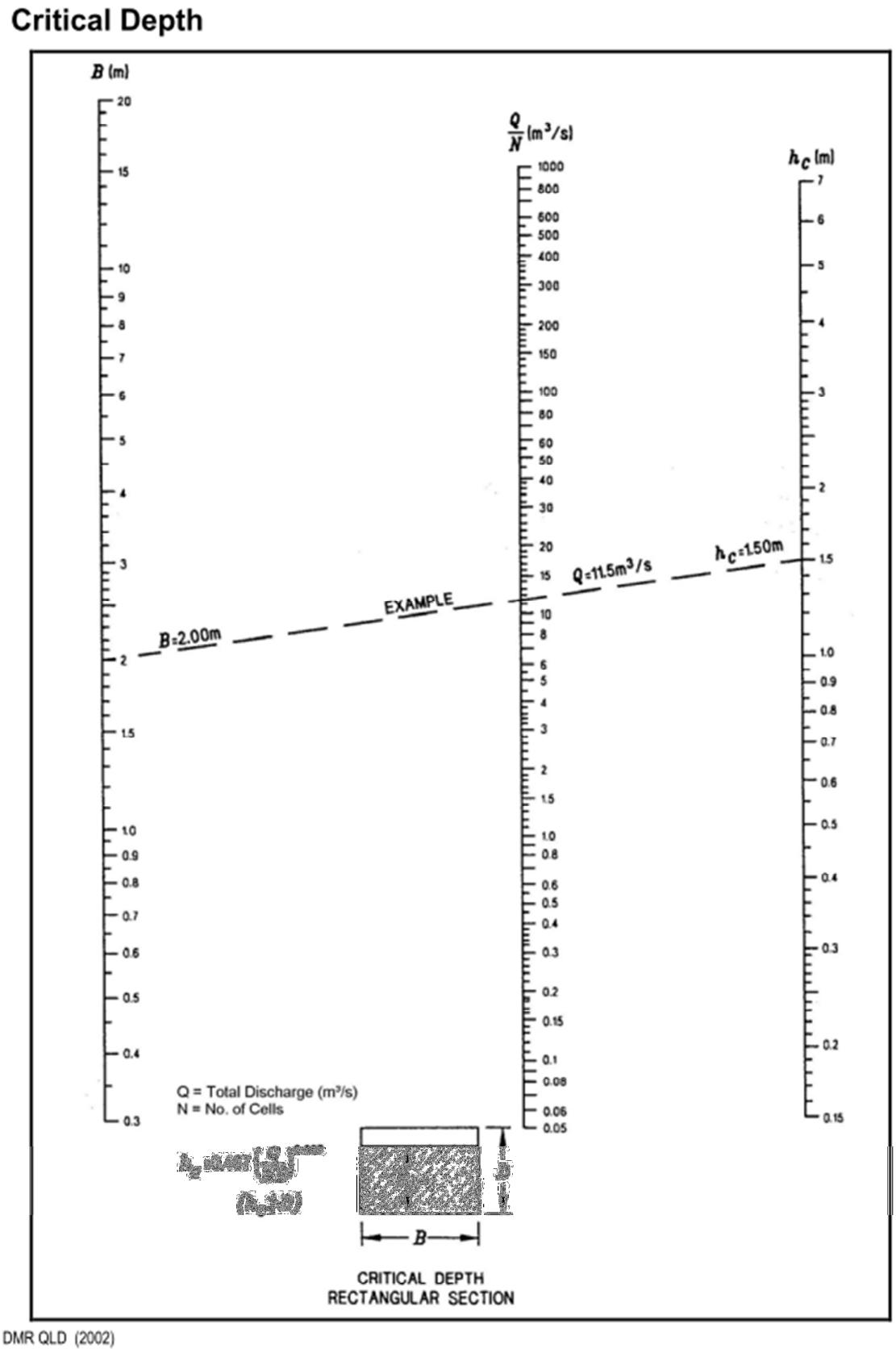
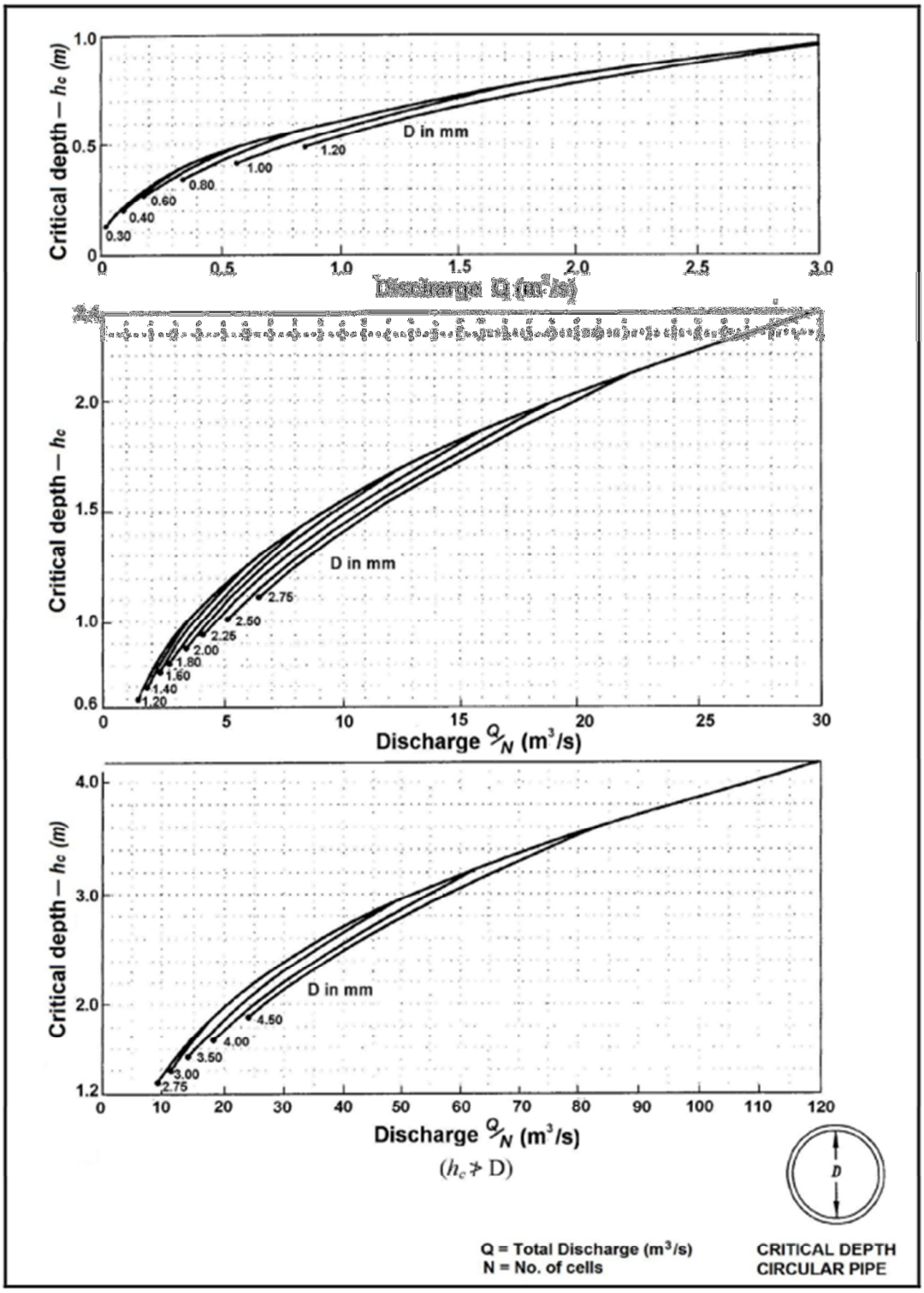
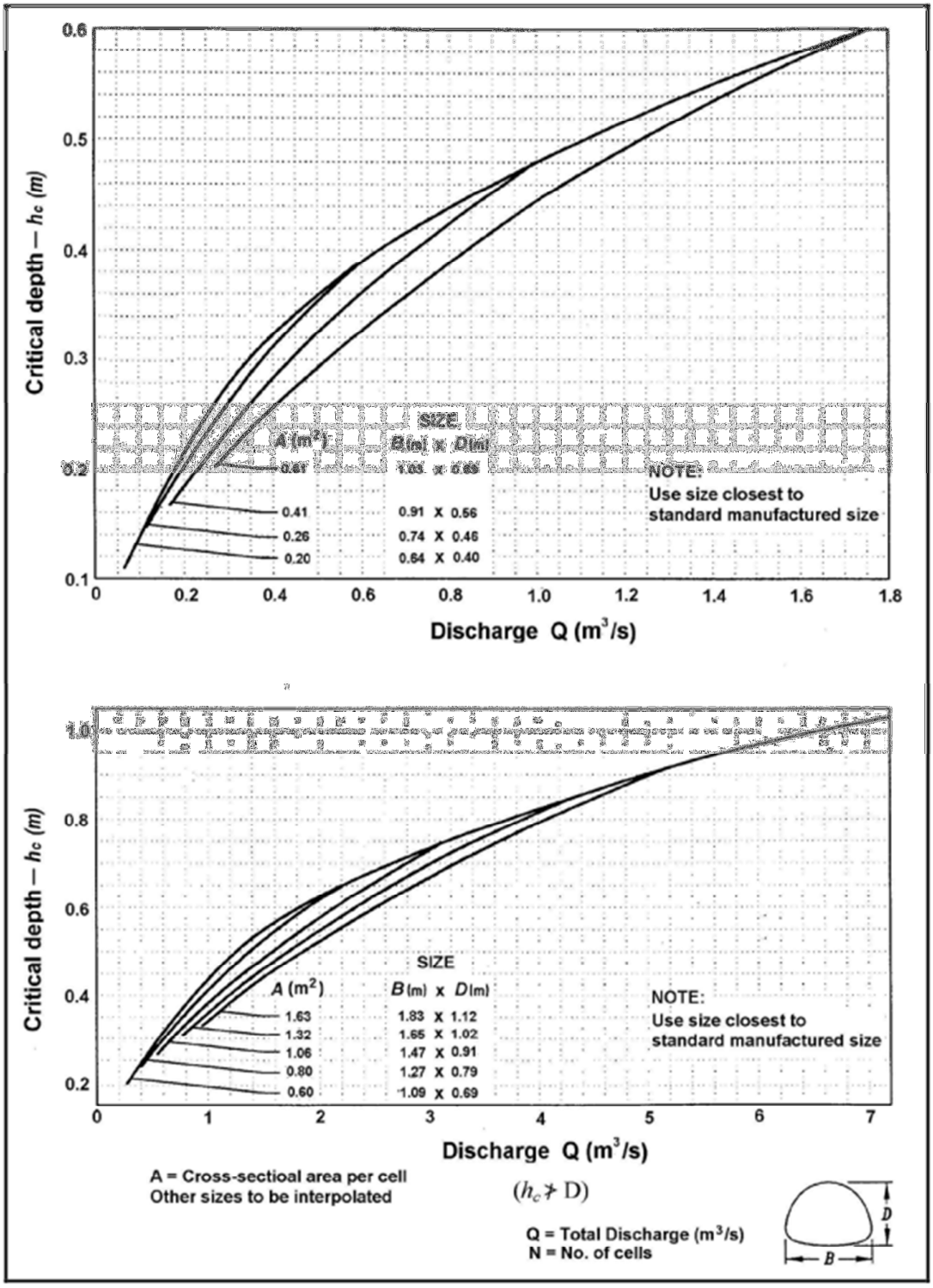


Figure B3: Critical Depth in a Rectangular Section



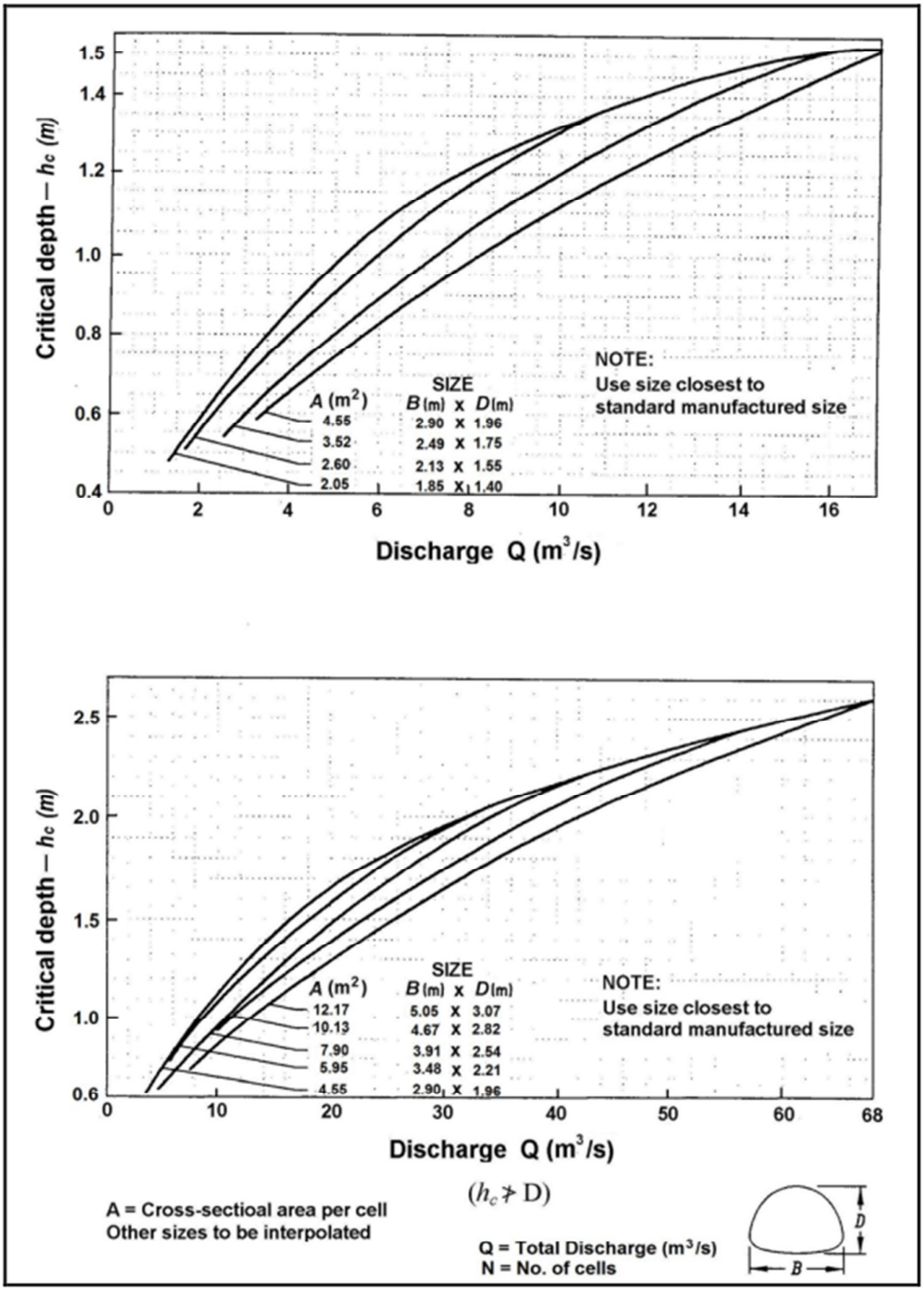
Source: DMR QLD (2002)

Figure B4: Critical Depth in a Circular Pipe



Source: DMR QLD (2002)

Figure B5: Critical Depth in a Corrugated Steel Pipe Arch (CSPA) (h_c)



Source: DMR QLD (2002)

Figure B6: Critical Depth in a Structural Plate Corrugated Steel Pipe Arch (SPCSPA) (h_c)

8 BRIDGES

8.1 Introduction

Hydraulic engineers and designers are faced with a wide variety of choices when determining the capacity or location of a new bridge or an existing bridge that is to be replaced. In addition to the choices, regarding hydrologic and hydraulic components of a bridge hydraulic analysis, there are many other factors and requirements to consider in addition to the engineering design.

One early consideration is the level of service the bridge is expected to provide. If the bridge is remote and carries a low volume of traffic, it can be designed with a lower hydraulic capacity resulting in a smaller and less expensive bridge. This means that the bridge and/or approach roadways will be overtopped more frequently and ERA and regional road authorities can expect the bridge and approach roadways to require more frequent maintenance and repair.

On the other hand, if the bridge is on an important route such that significant hardships or economic impacts would be encountered if the bridge were out of service, then it should be designed with a higher hydraulic capacity resulting in a larger and more expensive bridge and higher approach embankments. These bridges and/or approach roadways would be expected to rarely overtop and would need less frequent maintenance or repair.

8.1.1 Bridge Opening and Road Grade Design Conditions

In general, given a particular design discharge at a given crossing, the shorter a bridge the more backwaters it will create. This same smaller bridge will also have higher velocities through the bridge opening and an increased potential for scour at the bridge foundation. A longer bridge at this same crossing will generate a smaller amount of backwater and will have lower velocities and potential for scour. Policy considerations and economics require an understanding of the impacts that the bridge could have on the flow of water in the floodplain and the impacts it might have on adjacent properties.



Photo 8-1: Example Wide Bridge (Arba Minch)



Photo 8-2: Example Narrow Bridge (Arba Minch)

The bridge waterway width is directly associated with the bridge length, from abutment to abutment. Hydraulic capacity should be a primary consideration in setting the bridge length.

The bridge design must provide enough capacity to:

- Avoid excessive backwater in order to prevent adverse floodplain impacts; and
- Prevent excessive velocity and shear stress within the bridge waterway.

In order to reduce backwater and blockage by debris, adequate freeboard should be provided. Freeboard refers to the vertical distance from the water surface upstream of the bridge to the low chord of the bridge (this well be discussed in detail in the later sections). The freeboard requirement is associated with a particular design recurrence-interval event, which is usually the 50- or 100-year event. Rural, low-traffic routes often allow a lower recurrence interval for establishing hydraulic capacity and freeboard.



Photo 8-3: Example of a Bridge Without Freeboard (Rib River)

The road profile can also have a significant effect on bridge crossing hydraulics. Even if a bridge is designed to provide freeboard above a 100-year flood, the approach roadways may be overtopped by that same flood. When the overtopping occurs over a long segment of roadway, the associated weir flow is an important component of the overall hydraulic capacity of the crossing. In such a case, raising the road profile will have the potential to increase backwater unless additional capacity is provided in the bridge waterway to compensate for the lost roadway overtopping flow capacity.

The design of the piers and abutments has an effect on the bridge hydraulic capacity. Although this effect is small compared to the bridge length and road profile, it can still be important. For example, a bridge that crosses a floodplain must be shown to cause no increase in backwater over existing conditions. In such a case, the energy losses that are affected by the number of piers and their geometry can be significant. Spill-through abutments, set well back from the tops of the main channel banks, are advisable when bridge hydraulic capacity must be optimized.

Frequently the bridge waterway design includes subtle changes to the channel cross section under the bridge and for a short distance upstream and downstream of the bridge. These changes are intended to enhance channel stability and, in some cases, to improve hydraulic efficiency. Channel stability can be enhanced, for instance, by grading the channel banks to side slopes of 2H:1V or flatter, and by providing channel bank revetment. Capacity can be improved by a moderate widening of the channel bottom in the immediate vicinity of the bridge, with appropriate width transitions upstream and downstream.

There are several potential bridge opening and road grade considerations that impact hydraulic capacity and upstream flood risk, especially when a road is upgraded and the bridge is replaced. These include bridge length, deck width, abutment configuration (spill through or vertical wall), number and size of piers, low chord elevation, freeboard, and road grade. If a crossing with a 25-year level of service is improved to a 50-year level of service, the road elevation may need to be increased. To avoid increased flood risk, the replacement bridge may need to be considerably longer and higher than the existing bridge. If there is inadequate freeboard, debris may collect along the deck and reduce flow conveyance (see photo below).



Photo 8-4: Bridge in Arba Minch Completely Blocked by Debris

This chapter presents a design procedure that emphasizes hydraulic analysis using computer programs like ISIS, Mike 11 and HEC-RAS. These are the most recent programs used in bridge hydraulic analysis. Computer programs are now widely used for hydraulic analysis in many countries both developed and developing. It is recommended that the HEC-RAS software program be used as standard software for ERA bridge design analysis. Hand on training on how to use the software will be provided by ERA.

This discussion of bridge hydraulics in this chapter considers the total crossing, including approach embankments and structures on the flood plains.

8.2 Bridge Drainage Design Principles

The following general bridge design principles should be considered:

- The final bridge design selection should consider the maximum backwater allowed (0.5m) unless exceeding of the limit can be justified by special hydraulic conditions;
- The final design should not significantly alter the flow mechanism in both the main channel and the flood plain;
- The "crest-vertical curve profile" shall be considered as the preferred highway crossing profile when allowing for embankment overtopping;
- A specified clearance based on site specific conditions shall be established to allow for passage of debris if required;
- Degradation or aggradation of the river as well as contraction and local scour shall be estimated as part of the final design preferably using a two- dimensional computer model and the design should either minimize scour or provide scour protection; and
- Foundation level shall be positioned below the total scour depth whenever practical.

8.3 Bridge Drainage Design Criteria

8.3.1 General Criteria

Design criteria are the means for placing accepted policies and standards described into action and become the basis for the selection of the final design of the bridge structure. The following are the general criteria related to the hydraulic analyses that can be used as guidance for the location and design of bridges:

- Backwater will not significantly increase flood damage to property upstream of the waterway crossing;
- Velocities through the bridge structure(s) will not either damage the highway facility or increase damages to adjacent property both upstream and downstream of the crossing;
- Maintain the existing pre-construction flow mechanism to the extent practicable;
- Pier spacing, orientation, and abutment are to be designed to minimize flow disruption and potential scour; spill-through type abutments using side slopes are preferred over deep abutments to minimize scour and backwater effect;
- Select foundation design and/or scour countermeasures to avoid failure by scour;
- Allow freeboard at structure(s) designed to pass anticipated debris and minimise blockage of bridge openings;

- Acceptable risks of damage or viable measures to counter the unpredictability of alluvial streams;
- When two or more bridges are constructed in parallel over a channel, care should be taken to align the piers and to provide streamlined grading and protection for abutments. This abutment grading is to minimize expansion or contraction of flow between the two bridges;
- Commercial mining of sands and gravel in streams is common in Ethiopia because the material is clean and well graded and the stream replenishes the supply. Borrow pits, either upstream or downstream of a highway-stream crossing, can cause or aggravate scour at the bridge. This fact should be considered when calculating bridge scour, and should be estimated by sediment transport modelling using an industry standard software package;
- Minimal disruption of ecosystems and values unique to the floodplain and stream;
- Provide a level of traffic service compatible with that commonly expected for the class of highway and compatible with projected traffic volumes;
- Design choices should support costs for construction, maintenance, and operation, including probable repair and reconstruction and potential liability; and
- Adequate right-of-way shall be provided upstream and downstream of structure for maintenance operation.

8.3.2 Inundation

Inundation of the carriageway dictates the level of traffic services provided by the facility. The carriageway overtopping flood level identifies the limit of serviceability. Desired minimum levels of protection from carriageway inundation for functional classifications of roadways are presented in *Chapter 2, Standards and Departures from Standards*.

8.3.3 Risk Evaluation

The selection of hydraulic design criteria for determining the waterway opening, road grade, scour potential, riprap, and other features should consider the potential impacts to interruptions to traffic, adjacent property, the environment, and the infrastructure of the highway.

8.3.4 Backwater

Backwater and/or increases over existing condition should be decided based on site-specific conditions. It is important to eliminate or minimize backwater if possible especially in urban areas. However, if there are constraints, up to 0.5 m increase in backwater level upstream of the bridge during the passage of the 100-year flood can be allowed, if practicable.

8.3.5 Clearance

Clearance should be determined based on site-specific condition. However, a minimum clearance conforming to the requirements of the Bridge Design manual (usually 1.5 meters) shall be provided between the design approach water surface elevation and the low chord of the bridge for the final design alternative to allow for passage of debris. Other constraints may make lower free boards desirable, but the low chord must not impinge on the design high water level.

For off-system bridge replacement structures, the low chord should approximate that of the structure to be replaced unless the results of a flood risk assessment indicate a different structure is the most beneficial option.

8.3.6 Flow Distribution (Mechanism)

Any stream crossing that uses a combination of landfill and bridge within the floodplain disturbs flow distribution during floods. Preserve flow distribution to the extent practicable in order to:

- Avoid disruption of the stream-side environment;
- Preserve local drainage patterns;
- Minimize damage to property by either excessive backwater or high local velocities;
- Avoid concentrating flow areas that were not subjected to concentrated flow prior to construction of the highway facility; and
- Avoid diversions for long distances along the roadway embankment.

For many situations one-dimensional analysis techniques suffice for determining optimum bridge locations. When analyzing complex sites, such as those at a bend, and skewed crossings with one-dimensional models only, the designer need a great deal of intuition, experience, and engineering judgment to supplement the quantitative analysis. The development of two-dimensional techniques of analysis greatly enhances the capabilities of hydraulics designers to deal with these complex sites. On this situation, a HEC-RAS hydraulic modelling or something similar should be undertaken.

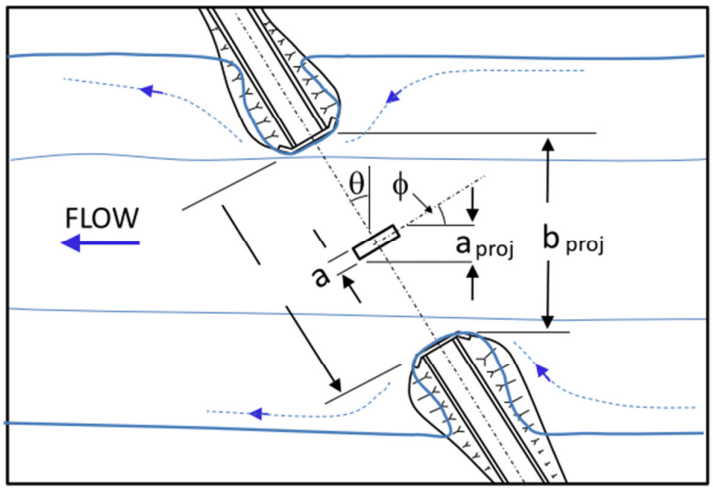


Figure 8-1: Illustration of Skew Bridge Crossing

(Source: Hydraulic Design Series Number 7, 2012)

8.3.7 Scour and Stream Stability Consideration and Guidance

Another critical component of the design and/or evaluation of a bridge opening is to design the bridge to be stable from scour at the piers, abutments, and across the contracted opening. From a hydraulic perspective, the magnitude of local scour at a pier is a function of depth and velocity of flow, alignment of the pier with flow, and pier type and location.

Depending on foundation costs and complexity it will be necessary to balance the number and size of piers, length and height, and anticipated total scour depth against increased costs of the superstructure associated with longer spans (girder type and allowable span) and foundation required to resist scour.

The magnitude of local scour at an abutment is a function of depth and velocity of flow, the skew of the embankment to the floodplain, as well as the amount of flow from the overbank that passes through the bridge opening. It is also a function of where the abutment is located in relation to the main channel. It is recommended that an abutment not be located in or close to the main channel if possible.

Recommended procedures for evaluating and designing bridges to resist scour can be found in FHWA publications HEC-20 (FHWA 2012a) and HEC-18 (FHWA 2012b).

Research on scour on selected rivers in Ethiopia is strongly recommended to calibrate scour formulae, which are developed in other countries. Scour analysis formulae are developed in laboratories under specific conditions for specific regions. Their applicability is limited to the environment under which they are developed. Their applicability to the Ethiopian rivers should be checked before they are applied.

8.4 Bridge Hydraulic Conditions

This section provides a qualitative description of the various types of flow conditions that can exist at a bridge crossing. It is assumed that the bridge drainage designer is familiar with the theory of bridge hydraulics. However, the designer is recommended to refer to hydraulics books for detailed theory of bridge hydraulics. Later sections explain the technical approaches to modelling and the different bridge hydraulic conditions.

8.4.1 Free-Surface Bridge Flow

Free-surface bridge flow refers to the range of flow conditions at a specific bridge in which the bridge lowchord is not submerged. The HEC-RAS Hydraulic Reference Manual classifies free surface bridge flow conditions as Class A, Class B or Class C, depending on flow regime in the stream reach being crossed and in the bridge waterway itself. Class A, B, and C flow can be considered roughly equivalent to Type I, II, and III flow, respectively, as described in HDS 1 (FHWA 1978).

Class A is the most commonly encountered free-surface bridge flow condition. In this class of flow the conditions are subcritical upstream of the bridge, downstream of the bridge, and throughout the bridge waterway. Class A flow generally satisfies the constraints of gradually varied flow throughout the reach of interest. HEC-RAS provides four available approaches to modelling Class A free-surface bridge flow at a bridge.

In the Class B scenario, the flow passes through critical depth within the bridge waterway, which requires that supercritical flow exist at least for a short distance downstream of the critical depth control section. The potential for Class B flow to occur inside a bridge waterway stems from two causes. First, the elevation of critical depth is often higher in the constriction than upstream or downstream. Second, the water surface within the constriction is dropping rapidly. Most commonly the flow conditions upstream and downstream of a bridge in Class B flow are subcritical, and a hydraulic jump will often exist either within the bridge waterway or a short distance downstream of the bridge. Class B flow can sometimes occur in conjunction with a supercritical stream profile. In this case the bridge waterway is a control section with subcritical flow upstream and a hydraulic jump occurring some distance upstream of the bridge.

In Class C flow, the regime is supercritical upstream and downstream of the bridge and through the bridge waterway. Class C flow is an extremely rare condition because natural channels on steep grades, such as mountain streams, rarely support uninterrupted supercritical flow over long reaches (Jarrett 1984). Class C flow, therefore, would typically be expected only in engineered flood control channels on a steep slope. Figure 8.2 illustrate Class A, B, and C flow conditions.

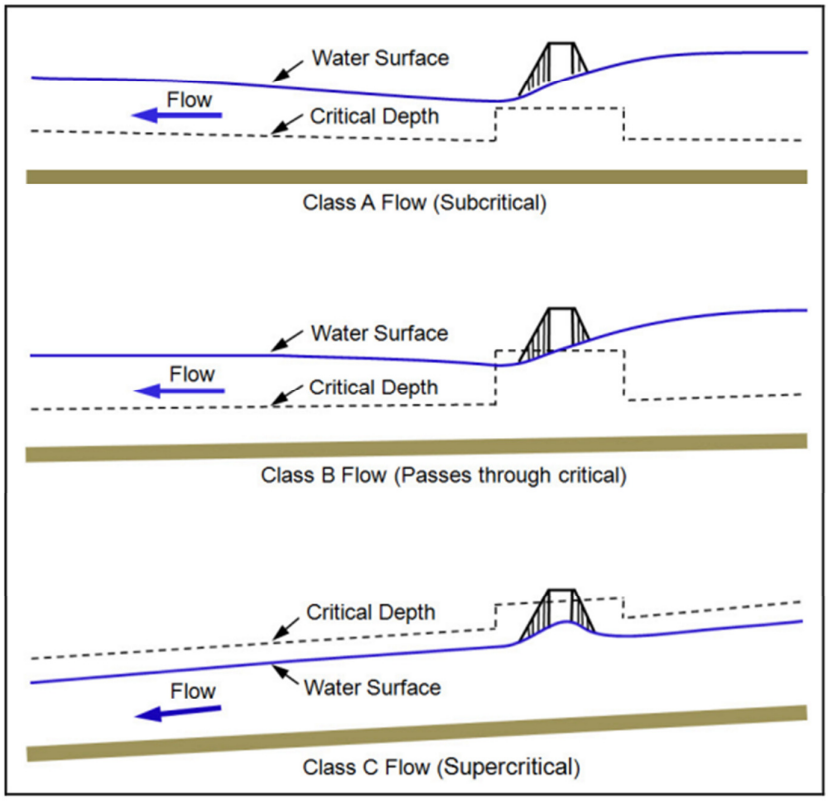


Figure 8-2: Illustration of Free-Surface Bridge Flow Classes A, B, and C

(Source: USACE 2010c).

8.4.2 Overtopping-Flow

Overtopping flow is the condition in which flow is crossing over the roadway approaches or the bridge deck itself. Overtopping flow conditions are appropriately represented by a broad-crested weir, since the road embankment is elevated above the floodplain grade, the dimension of the crest in the direction of flow (e.g. across the road) is broad and the overtopping depth is comparatively shallow. For example, in a wide floodplain with a low road profile, the quantity of flow going over the road instead of through the bridge can be considerable. With 0.3 m of overtopping depth, for instance, the weir flow could easily exceed 0.7 m^3/s for every 3 m of weir length.

Overtopping flow at bridge crossings is combined with either free-surface bridge flow or submerged-deck flow in the bridge waterway. When overtopping flow occurs, the engineer must determine how much flow is going through the bridge and how much over the bridge deck or roadway. This determination is accomplished by the principle that all flow paths from the upstream bounding section to the downstream bounding section should result in the same energy loss. Only one flow distribution between overtopping and bridge flow will result in equal energy loss.

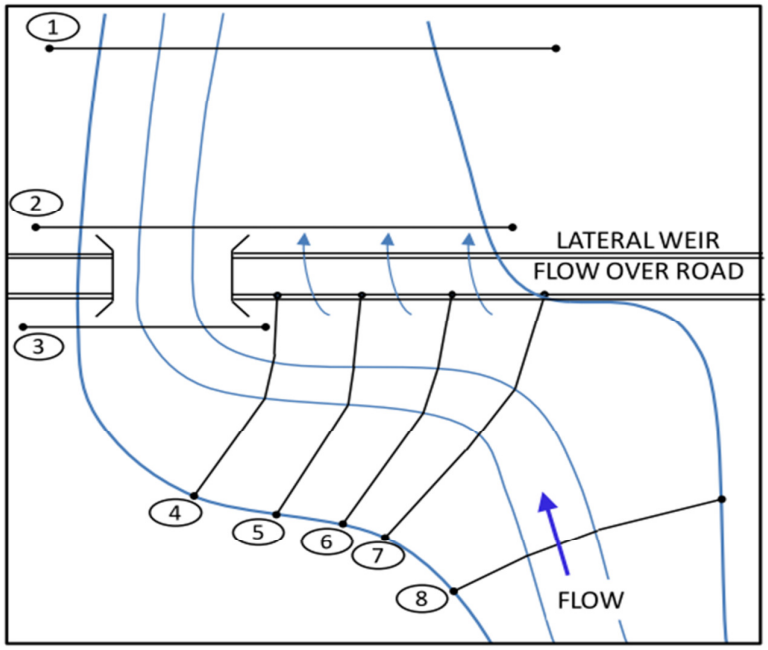


Figure 8-3: Illustration of Model in Incorporating Lateral Weir Flow

(Source: Hydraulic Design Series Number 7, 2012).

In some cases a weir does not accurately represent the flow over the roadway approaches.

This can occur either because the road is at or very near floodplain grade (in other words there is little or no embankment fill) or because the downstream water surface elevation is so high above the weir crest elevation that the weir control is drowned out.

8.4.3 Flow Submerging the Bridge Low Chord

A condition in which the water surface is above the highest point of the bridge low chord is usually representative of orifice flow. When the low chord is submerged only at the upstream edge of the superstructure, the orifice is considered free-flowing, and thus not affected by tail water. This condition is analyzed using the same approach as for an orifice (FHWA 1978) and in this manual is referred to as "orifice bridge flow." Just as the headwater upstream of an inlet-control culvert is not affected by conditions downstream of the culvert entrance, the backwater upstream of a bridge operating under orifice bridge flow is not affected by conditions downstream of the upstream edge of the superstructure.

Another type of orifice flow exists when the highest point of the low chord is submerged at both the upstream and downstream edges of the superstructure. This type of flow is analyzed using a formulation for a tail water-controlled orifice (FHWA 1978). Just as the headwater upstream of an outlet-control culvert is sensitive to conditions within and

downstream of the culvert barrel, the backwater upstream of a bridge operating under full-flowing or tail water submerged orifice conditions is affected by conditions within and downstream of the bridge waterway. For purposes of this manual, this condition is termed "submerged-orifice bridge flow".

As discussed above bridge hydraulic analysis is too complex and applying manual calculation to solve bridge hydraulics is impractical. Hence, it is recommended that hydraulic modelling using industry standard software be adapted for ERA bridge design.

8.5 Bridge Drainage Design Procedure

8.5.1 Bridge Location Selection and Orientation Guidelines

Although many factors, including non-technical ones, are used to determine the final location of a stream-crossing system, the hydraulics of the proposed location must have a high priority. Hydraulic considerations in selecting the location include floodplain width and roughness, flow distribution and direction, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium), and stream controls. The hydraulics of a proposed location also affects environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability.

For selecting and orienting bridge locations, consider the following guidelines:

- Bridges should be located and centered on the main channel portion of the entire floodplain. This may mean an eccentricity bridge in the location with respect to the entire stream cross section, but it allows for better accommodation of the low flows of the stream;
- Design the bridge waterway opening to provide a flow area sufficient to maintain the through-bridge velocity at no greater than the allowable through-bridge velocity under the circumstances of design discharge;
- Orient headers and interior bents to conform to the streamlines at flood stage;
- Accomplish this within reason, using standard skew values (15°, 30°, 45°, etc.) where feasible. Locate the toe of slope of the header away from deep channels, cuts, and high velocity areas;
- Locate and orient the bridge headers and piers to minimize the potential for excessive scour;
- If the intrusion of either or both roadway headers into the stream floodplains is more than about 240m, consider including either relief openings or guide banks; and
- Incorporate existing vegetation in the overall bridge plan. Where practicable, leave trees and shrubs intact even within the right-of-way. Minimizing vegetation removal also tends to control turbulence of the flow into, through, and out of the bridge. On the other hand, you should consider safety and maintenance aspects of retaining vegetation within the right-of-way and near the travel lanes.

For some configurations, it is necessary to incorporate roadway approaches that accommodate overflow. Such overflow approaches allow floods larger than the design flow to overtop the roadway, thereby reducing the threat to the bridge structure itself. This action interrupts the function of the roadway, and the designer should consider the potential costs associated with such interruption and associated damage to the embankment.

8.5.2 Bridge Drainage Design Processes

The following design processes shall be used as guidance. Although the scope of the project and individual site characteristics make each design a unique one, this procedure shall be used as a general guide (refer to Chapter 4 for more detailed data requirements):.

I. Data Collection

A. Survey

1. Topography
2. Geology
3. High-water marks
4. History of debris accumulation and scour at the existing structure in the vicinity of the proposed crossing;
5. Review of hydraulic performance of existing structures
6. Maps, aerial photographs
7. Rainfall and stream gauge records if available
8. Field reconnaissance visit

B. Other Relevant Information

1. River basin studies
2. Hydraulic performance of existing bridges

C. Influences on Hydraulic Performance of Site

1. Other streams, reservoirs, water intakes
2. Structures upstream or downstream
3. Natural features of stream and flood plain
4. Channel modifications upstream or downstream
5. Flood plain encroachments
6. Sediment types and bed forms

D. Environmental Impact

1. Existing bed or bank instability
2. Flood plain land use and flow distribution
3. Environmentally sensitive areas (fisheries, wetlands, etc.)

E. Site-specific Design Criteria

1. Preliminary flood risk assessment
2. Application of ERA drainage design criteria

II. Hydrologic Analysis

A. Catchment area morphology (characteristics)

1. Drainage area (attach map)
2. Catchment area and stream slope
3. Channel geometry

B. Hydrologic computations

Discharge for historical flood that complements the high water marks used for calibration (if available).

Discharges for specified frequencies (determined).

III. Hydraulic Analysis

A. Computer hydraulic model construction:
B Computer model calibration and verification (if data is available):
C Computer model sensitivity analysis (roughness, structural coefficients, future climate change and blockage scenarios);
D. Hydraulic performance for existing conditions;
E. Hydraulic performance of proposed designs;
F. Impact analysis
G. Mitigation Measures

IV. Selection of Final Design

A. Flood Risk Assessment/least-cost alternative (LTEC, or Least Total Expected Cost)
B. Measure of compliance with established ERA hydraulic criteria
C. Consideration of ERA environmental and social criteria
D. Design details such as riprap, scour abatement, and river training

V. Documentation

A. Complete project records (data, model), etc.
B. Complete correspondence and reports (survey, hydrological and hydraulic modelling reports)

8.6 Hydraulic Design of Bridges

8.6.1 Design Sequences

Prior to the project-scoping meeting, the hydraulic designer may make an initial determination of the anticipated structure, either a culvert or bridge. The basic sequence for the hydraulic analysis of a bridge is summarized in Figure 8.4 and consists of the following:

- Determine watershed hydrology as per Chapter 5, Hydrology;
- Visit the site and obtain flood history from ERA maintenance staff, bridge inspection files, and local residents. Check to see that the channel survey is adequate (see ERA Survey Manual or Channel and Floodplain Survey Specification in Chapter 4). Investigate upstream and downstream of the proposed or existing bridge crossing for conditions affecting stream stability such as man- made structures, significant hydraulic features, gravel mining activities, etc;
- Check for existing ERA studies for a road project in the vicinity of the site, and use the data from the study if possible;
- If the existing ERA data is used, the existing survey benchmark will need to be tied to the project survey data. Often the ERA data may be out of date and no longer represents the actual conditions for the proposed crossing site (e.g. land use/cover). Hence, the most updated survey should be used for sizing the bridge;
- Complete a water surface profile analysis through the bridge reach. This analysis should include the analysis of the natural situation without the proposed bridge and an analysis of the proposed bridge. The water surface profile should be determined using the program HEC-RAS, (limited) ISIS or Mike 11. The HEC-RAS program and ISIS free are freely available to download and are recommended to be used as standard software for bridge crossing analysis;

- The return period and design discharge for the profile analysis shall be computed as discussed in Chapter 5, Hydrology. Factors which contribute to the selection of the return period include the capacity and size of the highway, whether it is located in a rural or urban area, and the expected traffic levels;
- A range of bridge opening sizes smaller and larger than the existing channel should be analyzed and then compared with the existing and natural conditions to choose the optimum bridge channel width for the design flow;
- Locate the bridge within the floodplain and select a skew to best fit the alignment of the main channel and floodplain. Keep skew to a minimum to reduce construction and maintenance costs. Be aware that flow patterns can change as the discharge changes;
- Assess the impacts to the surrounding property and roadway for the overtopping and check flood 100-year flood for the various alternatives identified in step 4. Comparing the proposed impacts with the existing ERA study discussed in step 3 if available. Any increase in the floodplain should be avoided if possible. If not possible, the impacted area will require purchasing by ERA or other mitigation measure be put in place (e.g. compensation storage areas);
- Make preliminary calculations for aggradation/degradation, contraction scour and local scour; and
- Select the necessary revetment protection (i.e. riprap, guide banks, spur dikes, etc.) for the bridge and channel. Request right-of-way (ROW) if needed for the revetment protection.

For hydraulic crossings, in general and early in preliminary design, give the:

- Preliminary channel width;
- Elevation at excavated channel width;
- Skew, station at centerline of channel;
- Recurrence interval for design event;
- Drainage area;
- Design discharge;
- 100-year discharge;
- 500-year discharge for bridge scour (this should be discussed with local drainage experts and agreed up on);
- Minimum low girder elevation;
- Thalweg elevation;
- Ordinary high water elevation (OHW);
- Design high water elevation (DHW);
- 100-year high water elevation;
- 500-year high water elevation;
- Design velocity (V);
- 100-year velocity;
- 500-year velocity and riprap dimensions to the bridge design staff

8.6.2 Bridge Hydraulic Information Transmittal Sheet

Examples of the Bridge Hydraulic Information Transmittal sheets are shown in Figures 8.5 and 8.6. The bridge design engineer should use this information to evaluate how different bridge materials and configurations can be employed in order to best span the channel. Then, from this, the bridge design engineer should complete the General Layout Sheet.

For hydraulic crossings at most major structures, additional levels of hydraulics information are required after the preliminary design. Typically, the hydraulics engineer should draw in elevation view, the total scour envelope and differentiate the design scour depth from the 500-year flood scour depth (*the 500–year flood needs to be agreed with ERA or can by revised).* In addition, the hydraulics engineer should provide the existing and proposed water surface elevations; determine the backwater associated with the profile and waterway opening, etc. These additional items of information are to be provided to the bridge designer early in the final design. The "Hydraulics Work Flowchart for Major Structures" Figure 8.4 indicates the coordination required between Hydraulics, Geological and Bridge Engineers.

After the initial review meeting and the geology report is received, the final scour profile should be completed. The scour depth should be provided to the structural engineer and the geologist for final bridge design.

Complete all documentation, the bridge hydraulics analysis report and the bridge hydraulic information sheets for the plans. The scour depths will be shown on the bridge layout plan sheet. Figures 8.5 and 8.6 provide copies of the Bridge Hydraulic Information Transmittal Sheets for spill through and vertical-wall abutments. *These general design steps need to be followed for bridge drainage design in all regions of Ethiopia.*

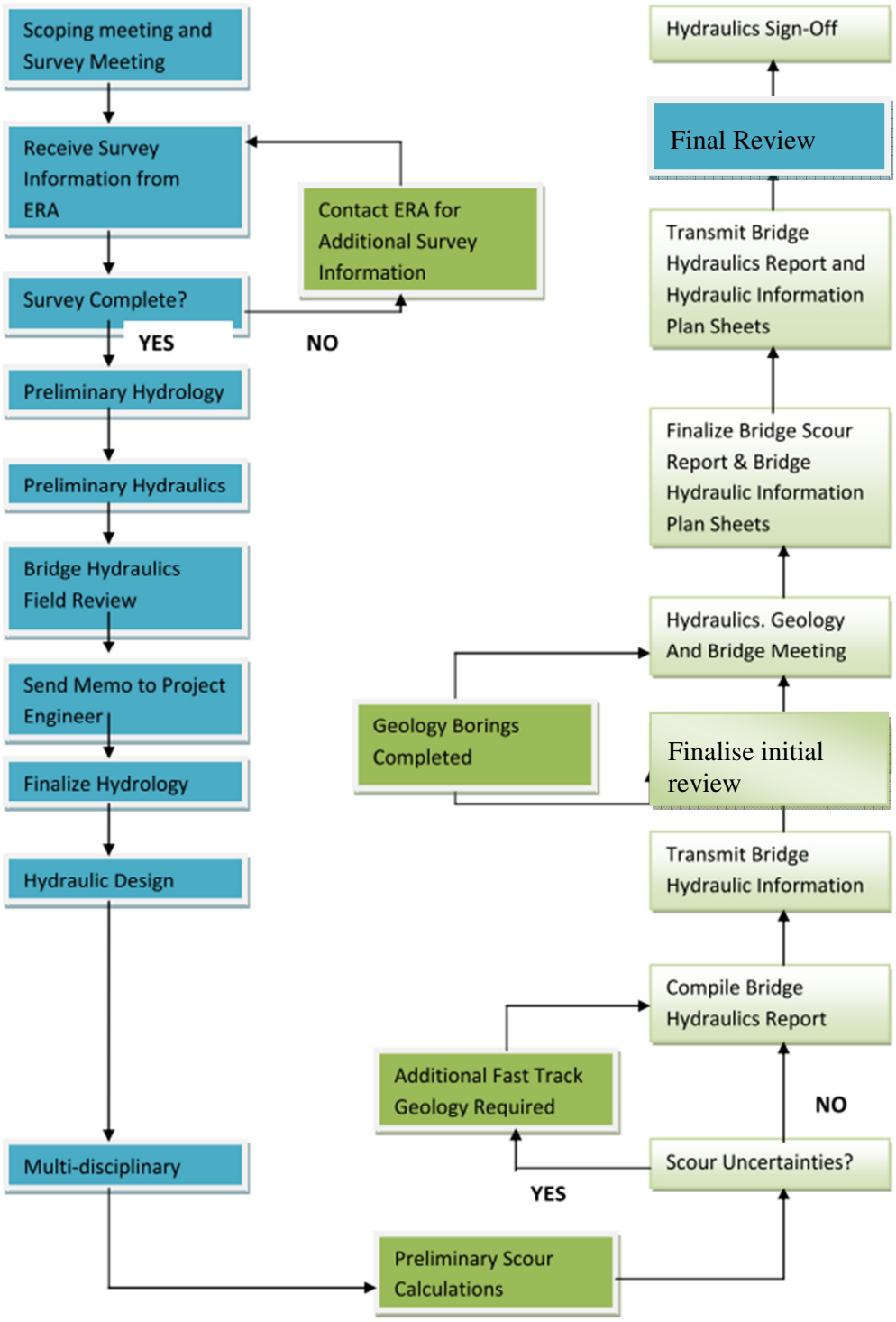


Figure 8-4: Work Plan for the Hydraulic Analysis of a Bridge.

Bridge Hydraulic Information Transmittal Sheet for Spill through Abutments

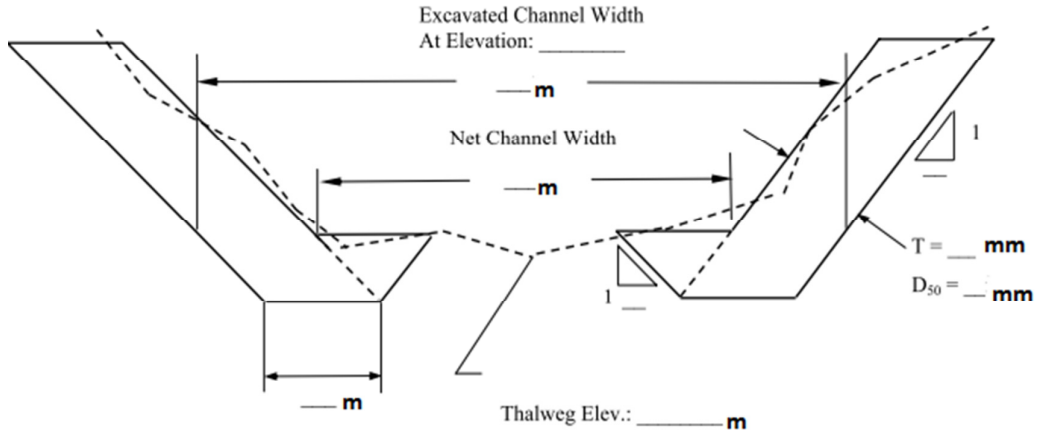
This is the structure opening and hydraulic information required for the bridge across_____ on _____at/near _____.

Project Information

Date:	Construction Project Number:
To:	Project Manager:
From:	Project Name:

Bridge Information

Existing Structure Number:	
Station at Centerline of Channel:	
Skew:	
Minimum Low Girder Elevation:	
Design Year Event:	_____ year recurrence.



Hydraulic Information

D.A. =_____sq.km	Q(Design) =_____m3/s	Q (100) =____m3/s	Q (500) =____m3/s
OHW = _____ m	DHW (Design) =_____m	DHW (100) =____m	DHW (500) =____m
	V (Design) =_____m/s	V (100) =_____m/s	V (500) =_____m/s

Please submit this sheet to ERA Staff with the information required by ERA Drainage Design Manual so they may proceed with design.

Bridge Layout requested: yes ☐ no ☐

Comments:_____

_____

_____

Figure 8-5: Transmittal of Bridge Hydraulic Information Sheet for Spill through Abutments

Bridge Hydraulic Information Transmittal Sheet for Vertical Wall Abutments

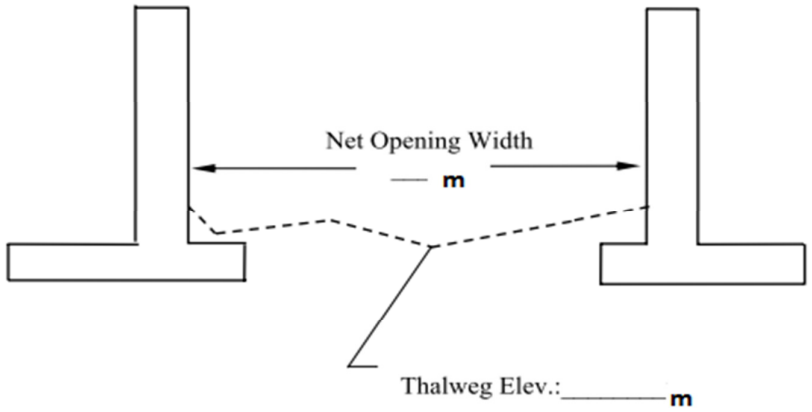
Here is the structure opening and hydraulic information required for the bridge across _____ on _____at/near _____.

Project Information

Date:	Construction Project Number:
To:	Project Manager::
From:	Project Name:

Bridge Information

Existing Structure Number:	
Station at Centerline of Channel:	
Skew:	
Minimum Low Girder Elevation:	
Design Year Event:	_____ year recurrence.



Hydraulic Information

D.A. =_____sq.km	Q(Design) =_____m3/s	Q (100) =____m3/s	Q (500) =____m3/s
OHW = _____ m	DHW (Design) =_____m	DHW (100) =____m	DHW (500) =____m
	V (Design) =_____m/s	V (100) =_____m/s	V (500) =_____m/s

Please submit this sheet to ERA Staff with the information required by ERA Drainage Design Manual so they may proceed with design.

Bridge Layout requested: yes ☐ no ☐

Comments:_____

_____

_____

Figure 8-6: Transmittal of Bridge Hydraulic Information Sheet for Vertical Wall Abutments

8.6.3 Bridge Computer Modelling Methodologies

As discussed in the previous sections, computer based hydraulic analysis is required for design of bridges. However, decisions must be made regarding the type of model computational methods, model extent, and amount of topographic data that needs to be collected. An assessment of flow resistance caused by channel and floodplain conditions needs to be made and the impacts on flow due to different seasonal conditions also needs to be evaluated. An understanding of flow type, historic flow conditions, and flooding at the site also provides valuable insight into the model approaches that need to be employed.

Once the preliminary data has been collected and an understanding of the flow complexity at the bridge site is obtained, a decision must be made regarding the type of hydraulic model that should be used at the hydraulic crossing. Some situations call for a one-dimensional gradually-varied steady-state flow model while others require the use of unsteady flow models, or two-dimensional steady or unsteady flow models to more fully understand the flow conditions at the hydraulic crossing. Some situations call for a more sophisticated modeling approach because of other factors. These can include the need for a more complete understanding of the flow conditions because of bridge scour or bank stabilization.

The purpose of this section is to provide guidance for carrying out a bridge hydraulic analysis where computer river modelling is considered.

8.6.4 Hydraulic Modeling Criteria and Selection

Any hydraulic model, whether it is numerical or physical, has assumptions and requirements. It is important for the hydraulic engineer to be aware of and understand the assumptions because they form the limitations of that approach. It is the goal of any hydraulic model study to accurately simulate the actual flow condition. Violating the assumptions and ignoring the limitations will result in a poor representation of the actual hydraulic condition.

Treating the model as a black box will often produce inaccurate results. This is not acceptable given the cost of bridges and the potential consequences of failure. Therefore, the approach should be selected based primarily on its advantages and limitations, though also considering the importance of the structure, potential project impacts, cost, and schedule.

8.6.5 One-Dimensional Versus Two-Dimensional Modeling

One-dimensional modelling requires variables (velocity, depth, etc.) change predominantly in one defined direction, for example x, along the channel. Because channels are rarely straight, the computational direction is along the channel centerline.Two-dimensional models compute the horizontal velocity components (Vx and Vy) or, alternatively, velocity vector magnitude and direction throughout the model domain.

Therefore, two-dimensional models avoid many assumptions required by one-dimensional models, especially for the natural, compound channels (free-surface bridge flow channel with floodplains) that make up the vast majority of bridge crossings over water.

The advantages of two-dimensional modelling include a significant improvement in calculating hydraulic variables at bridges. Therefore, the use of two-dimensional models is recommended over one-dimensional models for complex waterway and/or complex bridge hydraulic analyses. One-dimensional models are best suited for in-channel flows and when floodplain flows are minor. They are also frequently applicable to small streams. For

extreme flood conditions, one-dimensional models generally provide accurate results for narrow to moderate floodplain widths. They can also be used for wide floodplains when the degree of bridge constriction is small and the floodplain vegetation is not highly variable. In general, where lateral velocities are small one-dimensional models provide reasonable results. Avoiding significant lateral velocities is the reason why cross section placement and orientation are so important for one-dimensional modelling. Two-dimensional models generally provide more accurate representations of:

- Flow distribution;
- Velocity distribution;
- Water Surface Elevation;
- Backwater;
- Velocity magnitude;
- Velocity direction;
- Flow depth; and
- Shear stress.

Although this list is general, these variables are essential information for new bridge design, evaluating existing bridges for scour potential, and countermeasure design.

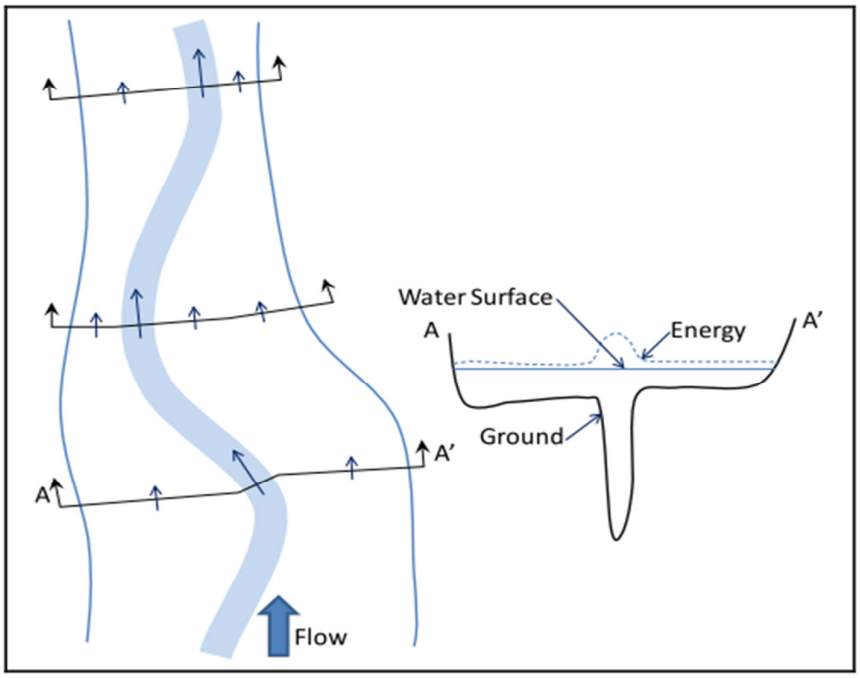


Figure 8-7: One–Dimensional Model Cross Section

Two-dimensional models should be used when flow patterns are complex and one-dimensional model assumptions are not robust.. If the hydraulic engineer has great difficulty in visualising the flow patterns and setting up a one-dimensional model that realistically represents the flow field, then two-dimensional modeling should be used.

Multiple Openings: Multiple openings along an embankment are often used on rivers with wide floodplains (see chapter 7). Instead of using a single bridge, additional floodplain bridges are included. Although one-dimensional models can be configured to analyze multiple openings, the judgment and assumptions that are made by the hydraulic engineer

in combination with the assumptions and limitations of the software result in an extreme degree of uncertainty in the results. The proportion of flow going through a particular bridge and the corresponding flow depth and velocity are important for structure design and scour analysis. Because multiple opening bridges represent a large investment, two-dimensional analysis is always warranted.

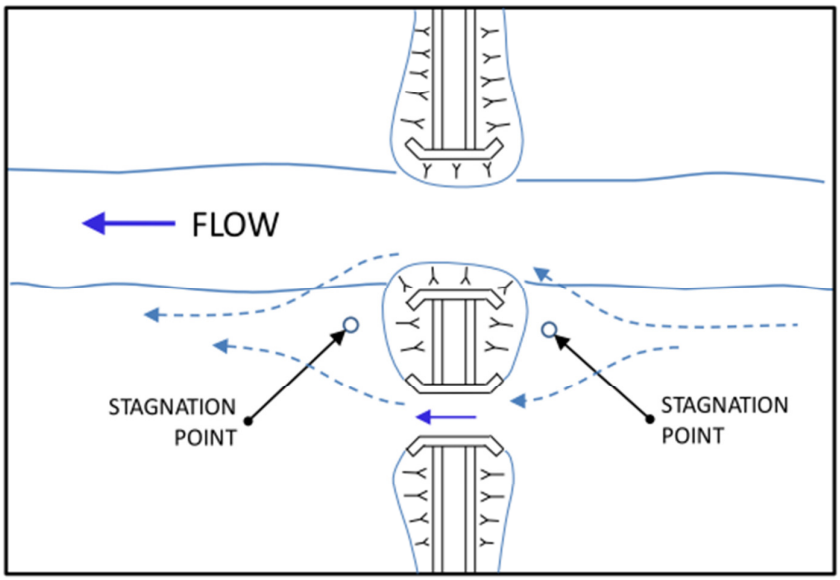


Figure 8-8: Plan View Sketch of a Multiple–Opening Bridge Crossing

Another type of multiple opening is multiple bridges in series. There are conditions when this bridge configuration should be analyzed using two-dimensional models. These include unmatched bridge openings or foundations that do not align. An upstream or downstream railroad or parallel road may significantly alter the flow conditions and warrant two-dimensional analysis.

Wide Floodplains: Floodplains often include features that significantly impact flow conveyance and flow distribution (see Figure 8.9). Historic channel alignments and changes in land use or vegetation affect floodplain flow distribution. In a one-dimensional model, two cross sections that are a short distance apart may have significantly different vegetation, such as wooded versus cleared, or may have significantly different topography due to land use activities.

If the hydraulic engineer uses these cross sections exactly as they exist, the one-dimensional model will depict a sudden change in flow distribution that is not physically possible. To better depict the flow conditions, the hydraulic engineer would need to adjust the cross section locations or alter the Manning's n values, although this is difficult to implement. The two-dimensional model avoids these difficulties because in the simulation all the flow is interconnected. Therefore, wide and complex floodplains benefit from two-dimensional analysis. *It is recommended that two-dimensional modelling be used when a bridge is constructed across wide floodplains.*



Photo 8-5: Multiple Bridge in Series (Outside Addis Ababa)

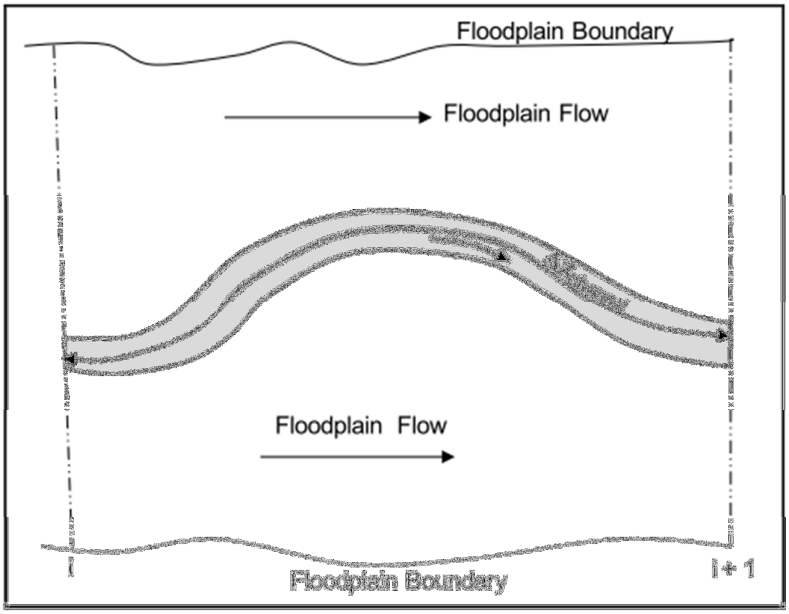


Figure 8-9: Channel and Floodplain Flows

Skewed Roadway Alignment: Roadways should be aligned perpendicular to channel and floodplain flows. FHWA (1978) indicates that skewed crossings with angles of up to 20 degrees produced no objectionable flow patterns. The HEC-RAS Reference Manual (USACE 2010c) indicates that using the projected opening is adequate for skew angles of up to 30 degrees for small flow constrictions. *Two-dimensional modelling is the recommended approach for higher skew angles or moderate amounts of skew combined with moderate to high flow contraction.*

Road Overtopping: When computing road overtopping, the HEC-RAS model (USACE 2010c) uses the total energy grade line in the cross section upstream of the bridge as the head value in the weir equation. This assumption is reasonable for many conditions. Because standard use of ineffective flow areas can trigger full floodplain flow for any amount of overtopping, USACE (2010a) recommends comparing the road overtopping

discharge to the floodplain flow and adjusting the Manning's n to better maintain flow continuity. As illustrated in Figure 8.6, for roads crossing wide floodplains or skewed crossings, two-dimensional models offer a better approach. Road overtopping is still computed using the weir equation, but nodes on either side of the embankment are connected using a weir segment. The water surface and velocity at the two connected nodes are used to determine head and submergence. The head at the upstream node is used rather than the total energy grade line of the entire upstream cross section. Therefore, better estimates of the initiation of overtopping and overtopping discharges are achieved.

Bends, Confluences and Angle of Attack: Highly sinuous rivers are, by definition, not one-dimensional, especially during floods when water in the floodplain flows more directly down valley and moves in and out of the channel. One-dimensional models must consider different channel and floodplain flow distances between cross sections and compute a discharge-weighted flow length. *Two-dimensional models do not make any simplifying assumptions related to channel versus floodplain flow distance because the two-dimensional network directly incorporates flow paths*. Flow conditions at confluences also vary depending on the proportion of flow in the main stem and tributary. With a one-dimensional model, determining the angle of attack for pier scour calculations is highly subjective in these situations and can be difficult for many other conditions. Two-dimensional models provide improved estimates of angle of attack because velocity direction is computed directly.

Multiple Channels: Branched and braided rivers have multiple channels and flow paths that complicate hydraulic calculations. The hydraulic engineer would have to decide the amount of adjacent floodplain to assign to each channel segment and may well need to allow for lateral flow between floodplain segments. Two-dimensional models, while still be a significant challenge, clearly have numerous advantages in this situation. Although many multiple channel situations are well simulated with the split-flow options in HEC-RAS, the effort in developing a two-dimensional model for these conditions may be less than an equivalent one-dimensional model.

8.6.6 Selecting Upstream and Downstream Model Extent

The minimum extent of a hydraulic model for bridge hydraulics is the location where flow is fully expanded both upstream and downstream of the flow constriction. Flow constriction is often the major contributor to backwater, so complete flow expansion and contraction must be included in the modelling. For one-dimensional models, the use of the minimum downstream extent does not detract from the results as long as the downstream water surface is known with a high degree of certainty. However, if the water surface is not known with confidence, then extending the model further downstream will decrease uncertainty at the structure. This is illustrated in Figure 8.10, which shows water surface profiles for a simple bridge model. The three profiles are all for the same discharge with the only difference being the downstream boundary condition. Each one of the profiles represents a valid solution to the equations of fluid motion. The downstream boundary is located far enough downstream so the profiles converge and the 1.2 m of initial difference is eliminated before reaching the bridge.

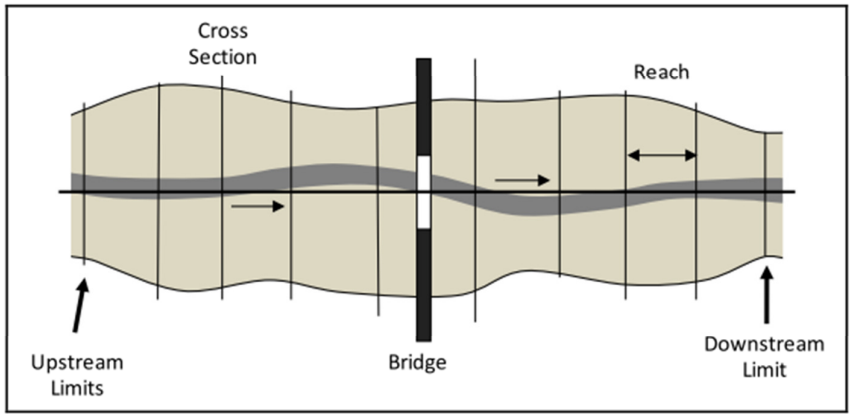


Figure 8-10: Example Model Study Limits Upstream and Downstream

(Source: Hydraulic Design Series Number 7, 2012)

Thus, an important principle of numerical modelling is that the further downstream the model extends, the smaller the influence the boundary condition will have on the location of interest. The further the boundary is from the bridge, the less uncertainty exists at the bridge because channel and floodplain geometry and roughness will dictate the results. This does not mean, however, that all uncertainty is removed. Inaccuracies or change in any of the input variables result in uncertainty in the results.

The minimum downstream extent for two-dimensional models is similar to one-dimensional models with flow fully expanded upstream and downstream. It is also desirable to select a location where flow is reasonably one-dimensional, especially at the downstream boundary. This is because the downstream boundary is usually specified as a constant water surface elevation along the boundary. One useful approach is to place the upstream and downstream boundaries at least one floodplain width upand downstream of the crossing. As with one-dimensional models, the further the boundary is located away from the crossing the less influence the boundary condition will exert of the results.

When there are other structures or hydraulic controls either upstream or downstream that will influence or can be impacted by the project, then the modelling should be extended to include these structures. Figure 8.16 shows some backwater created by the crossing. Although the extent of the model probably captures the maximum water surface increase, extending the model upstream would be required to fully assess potential upstream impacts.

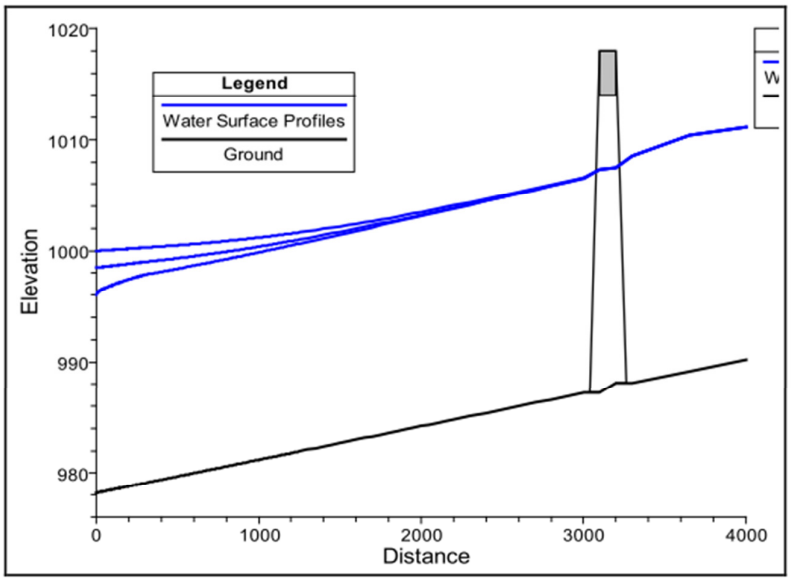


Figure 8-11: Flow Profile with Downstream Boundary Uncertainty

(Source: Hydraulic Design Series Number 7, 2012)

8.6.7 Identifying and Selecting Model Boundary Conditions

An important part of the hydraulic engineer's responsibility is to select representative boundary conditions for the hydraulic analysis. Peak discharge is one boundary condition that is commonly used for river projects and flood hydrographs are most frequently used for unsteady riverine modelling. For subcritical flow conditions, the downstream water surface must be specified or computed. For supercritical flow, the upstream condition is specified and for mixed flow conditions, the downstream and upstream condition is specified. The model extent and boundary condition should be selected based on identifiable hydraulic controls or on other reliable information. There are several types of hydraulic controls that can establish the boundary condition at a bridge. These include slope breaks where critical depth occurs (from flat to steep in the downstream direction), diversion dams, bridges, roads and other structures. The discussion below relates to a downstream boundary but also applies to upstream boundary conditions for supercritical or mixed flow models.

Water Surface

A known water surface is very commonly used in hydraulic modelling, where the hydraulic engineer specifies the elevation as the starting downstream condition.

If the water surface in the vicinity of the bridge crossing is already known, starting the model relatively close to the bridge is more efficient and the water surface can be extracted from the appropriate location on the profile. Gauge data or an observed high water mark can also be used to establish known water surface elevations as input boundary conditions. If this information is not available, the modelling model boundary should be fixed to a location where the uncertainty does not affect the result at the crossing site.

Normal Depth and Energy Slope

Normal depth occurs when the bed profile, water surface, and energy grade line are all parallel, and the flow depth and velocity do not change along the channel flow path. This occurs relatively infrequently in natural rivers, though it can be a reasonable approximation for establishing boundary conditions in many situations. The use of the channel invert (thalweg) to compute bed slope should be avoided in natural channels because the channel bed elevation can vary widely over short distances. A better approximation is to use the floodplain slope as measured from a topographic map. The channel slope can, however, be much less than the valley slope for highly sinuous meandering channels. A conveyance weighted slope can be used, but this requires an assumed water surface to compute channel and floodplain conveyance.

When energy slope or normal depth is used, the model iteratively computes a water surface that produces the desired slope. Flow conditions are unlikely to actually satisfy normal depth criteria because of longitudinal topographic and roughness variations. The model downstream extent should be extended for this situation. The variability in channel and floodplain conditions is then incorporated into the model solution and uncertainty caused by the boundary condition is reduced.

Rating Curve

A rating curve is a stage versus discharge table or curve relating stage and discharge. Gauging stations rating curve data in Ethiopia should be published r and regularly checked and updated by the Ministry of Water & Energy (MoW&E) . Gauging station data can also be used to establish new rating curves. This data only apply to the specific gauge location. Any data from MoW&E studies can also be used to develop rating curves for a specific location and used as a model boundary condition. The same uncertainties can apply to the use of energy slope, so extending the model downstream is warranted.

Critical Depth

Critical depth is a relatively well defined boundary condition when a control structure produces a sudden drop in the channel. Critical depth in natural channels is unusual except in steep, bedrock or boulder-bed channels. In HEC-RAS (USACE 2010c) critical depth is defined as the minimum total energy. In a natural channel, total energy includes the energy correction coefficient, α, so roughness and flow distribution impact the determination of critical depth. Critical depth should be confirmed as reasonable before using it as a boundary condition in natural channels.

8.6.8 Computer Programs

Example problems using the HEC-RAS model has been included in Appendix 8A for demonstration purposes. Refer to chapter 15 for detailed information for computer programs used in bridge analysis.

8.6.9 Physical Modelling

Complex hydrodynamic situations defy accurate or practicable mathematical modelling. Physical models shall be considered when:

- Hydraulic performance data is needed that cannot be reliably obtained from mathematical modelling; and
- Risk of failure or excessive over-design is unacceptable, and research is needed.

The constraints on physical modelling are:

- Size (scale);
- Cost; and
- Time.

8.7 Bridge Scour and Aggradation

Reasonable and prudent hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge vulnerability to undermining due to potential scour. Because of the extreme hazard and economic hardships posed by a rapid bridge collapse, special considerations must be given to selecting appropriate flood magnitudes for use in the analysis. Sediment transport analysis is an infant science and the existing sediment transport formulae are developed in university laboratories in specific environment for a specific location. The designer shall use the most appropriate scour forecasting methods and calibration of the method with local data is recommended. This section of the manual describes some of the fundamental theory of scour at bridges. The bridge drainage designer is encouraged to use software programs for the analysis of bridge scour.



Photo 8-6: Example of Scoured Bridge Foundation (Dechatu Bridge, Dire Dawa)

8.7.1 Concepts and Definitions

Incipient Motion

Various relationships exist that define the boundary conditions under which a stream will begin to erode materials along its bed and banks. The best known of these is probably the shields relationship, which is still in use and is based on a representative particle size for a specific material density.

Analysis of incipient movement, in terms of stream power considerations, has lead to the representation of incipient motion of cohesionless materials as shown in Figure 8.12 (Modified Liu Diagram), which expresses the boundary between sediment movement and no sediment movement in terms of a plot of a "shear Reynolds Number" against the ratio between shear velocity and settling velocity is constant.

$$\frac{V_{*c}}{V_{ss}} = 0.12 \qquad (8\text{-}1)$$

With

$$V_* = \sqrt{gDS} \quad (8\text{-}2)$$

Where:

V_* = Shear velocity (m/s)
V_{*C} = critical velocity (m/s)
g = gravitational acceleration (9.81 m/s^2)
D = flood depth (m)
V_{SS} = particle settling velocity (m/s) (Figure 8.18)

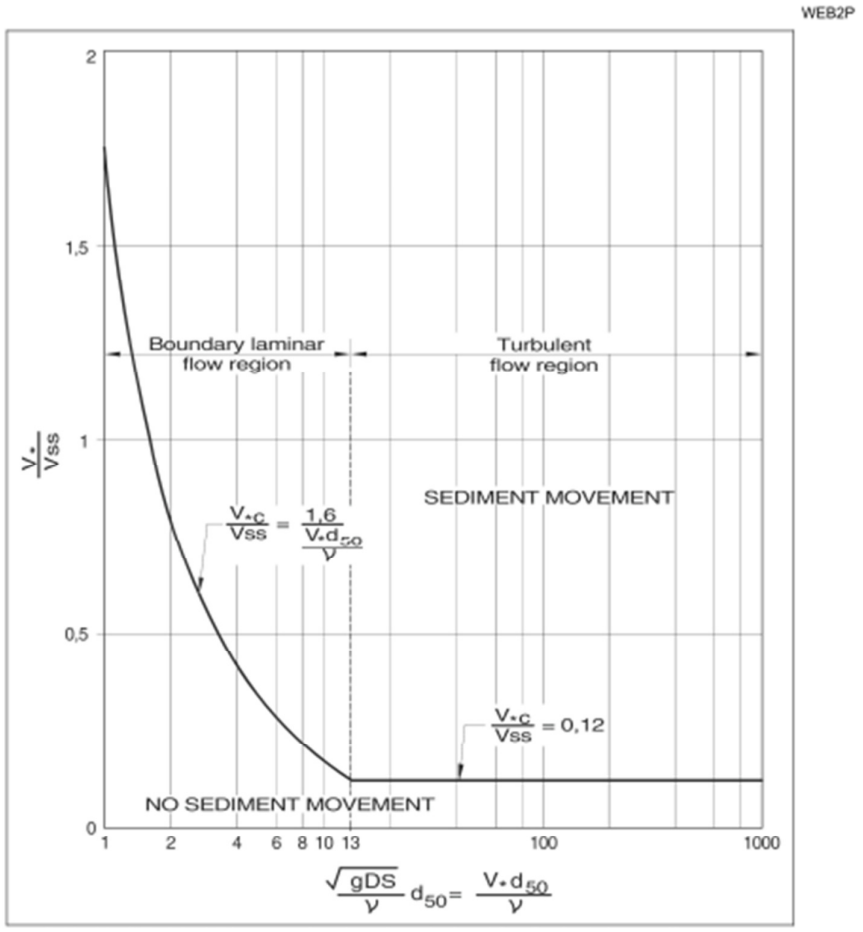


Figure 8-12: Modified Lui Diagram Showing the Relationships for Incipient Movement

(Source: South Africa Drainage Design Manual, 2006)

For laminar boundary conditions, the critical condition between sediment movement is given by:

$$\frac{V_{*c}}{V_{ss}} = \frac{\frac{1.6}{V.d_{50}}}{\nu} \quad (8\text{-}3)$$

where:

$d_{50} = average\ particle\ diameter\ (m)$

$$v = kinematic\ fluid\ viscosity\ \frac{m^2}{s}$$

The boundary between turbulent and laminar conditions is defined by:

$$\frac{V.d_{50}}{v} = 13 \qquad (8\text{-}4)$$

For values less than 13, the boundary flow condition is laminar and for values more than 13, boundary flow is turbulent. This boundary coincides with a particle size in the order of 5mm.

The relationship between settling velocity and particle diameter is shown in Figure 8.13

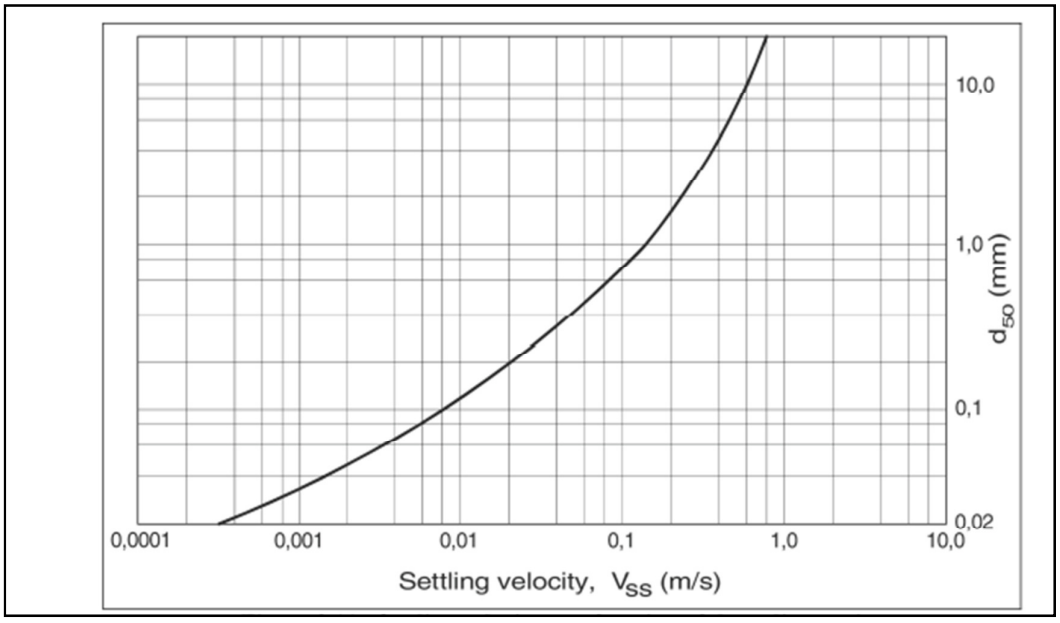


Figure 8-13: Settling Velocity as a Function of the Sediment Size

8.7.2 Sediment Loads

Ethiopian rivers generally carry a significant amount of fine sediments in suspension (see example Photo 8.8 below). The amount of transportable materials may be related to various factors such as rainfall, soil erodibility, slope and land use. These factors vary both in space and in time.

Not taking these factors into consideration at the feasibility and initial design stage of the crossing structure is found to be the major contributing factor for many of the bridge failures in Ethiopia. During the review of this manual and a site visit to the regions, it was noted that, these factors are not considered in detail during the design process. It was also noted that the bridge hydraulic analysis is mostly limited to the vicinity of the waterway crossing. It is recommended that the above factors be carefully considered and a wider catchment area sediment transport assessment is carried out as part of the bridge design process.



Photo 8-7: Gojjam Adet–Quarit Road

Users of this manual should consult HEC-18 (Ref. 9) which can be downloaded from the internet at no cost for more thorough information on scour and scour prediction methodology. A companion FHWA document to HEC-18 is HEC-20 "Stream Stability at Highway Structures (FHWA 2012b) and "HEC-23 "Bridge Scour and Stream Instability Countermeasures (FHWA 2009)".

8.7.3 Scour Types

The following are common scour types:

- Long term profile changes (aggradation/degradation);
- Plan form change (lateral channel movement);
- Contraction scour/deposition; and
- Local scour.

Long Term Profile Changes

Long-term profile changes can result from streambed profile changes that occur from aggradation and/or degradation.

Aggradation is the deposition of bed load due to a decrease in the energy gradient; and Degradation is the scouring of bed material which increased stream sediment transport capacity that results from an increase in the energy gradient.

Forms of degradation and aggradation shall be considered as imposing a permanent future change for the streambed elevation at a bridge site whenever they can be identified. It is recommended that a multi-disciplinary team should do some long-term scour predictions, as knowledge of hydraulic, geomorphology, geology, etc. is required.

Plan Form Changes

Long-term General Scour

Long-term general scour should be considered as part of the domain of specialist. Specialist will typically use procedures developed in the field of large dam engineering to determine the influence of major structures and make use of sophisticated modelling tools in order to quantify long-term river aggradation or degradation effects. In addition, it is

recommended that a multi-disciplinary team should do some long-term scour predictions, as knowledge of hydraulic, geomorphology, geology, etc. is required.

Short-Term General Scour

Short–term general scours in alluvial channels. Farady and Charlton recommended the following relationships for the equilibrium dimensions for a channel, based on the work of Blench (Ref: South Africa Drainage Manual 2006):

$$y = 0.38q^{0.67}{D_{50}}^{-0.17} \quad (8\text{-}5)$$

and

$$\boldsymbol{B = 14Q^{0.5}{D_{50}}^{0.25}{F_S}^{-0.5}} \quad \textbf{(8-6)}$$

where

B = mean channel width (m)
y = mean depth of flow (m)
Q = equivalent study discharge which would generate the channel geometry (m^3/s)
q = discharge per unit width (Q/B) ($m^3/s.m$) (Note: to estimate channel geometry conditions under flood conditions the design flood flow may be used)
D_{50} = median size of bed material (m)
F_s = side factor to describe bank resistance to scour (table 8.1)

The following side factors may be applied in the channel width equation (Equation 8.9):

Table 8-1: Side Factors

Bank type	Value of F_s
Sandy loam	0.1
Silty clay loam	0.2
Cohesive banks	0.3

(Source: South African Manual 2006)

It is necessary that to calculate the equilibrium width before the equilibrium depth can be calculated.

The maximum channel depth, y_{max}, can be determined by multiplying the calculated equilibrium depth with the factor on Table 8.1. Now the short term general scour depth, d_c, can be determined as the difference between y_{max} and the normal flow depth (y_n).

Short-Term General Scour in Cohesive Bed Channels:

For cohesive bed channels, the equilibrium depth equation becomes:

$$y = 51.4n^{0.86}q^{0.86}\tau_c^{-0.43} \quad \textbf{(8-7)}$$

where:

y = mean depth of flow (m)
n = Manning's coefficient of roughness ($s/m^{1/3}$)
q = discharge per unit width (m^3/s)
τ_c = critical tractive stress for scour to occur (N/m^2) – Refer to Table 8.2

Table 8-2: A Guide to Assess the Physical Properties of Clay

Voids ratio	2.0 – 1.2	1.2 – 0.6	0.6 – 0.3	0.3 – 0.2
Dry bulk density (kg/m3)	880 -1200	1200 - 1650	1650 – 2030	2030 - 2210
Saturated bulk density (km/m3)	1550 - 1740	1740 - 2030	2030 - 2270	2270 - 2370
Type of soil	τ_c critical tractive stress (N/m^2)			
Sandy clay	1.9	7.5	15.7	30.2
Heavy clay	1.5	6.7	14.6	27.0
Clay	1.2	5.9	13.5	25.4
Lean clay	1.0	4.6	10.2	16.8

(Source: South African Manual 2006)

The bulk density in this table assumes specific particle density = 2.64 and the relationship with voids ratio reads as follows:

$$\rho_d = \frac{\rho s}{e+1} \quad (8\text{-}8)$$

and

$$\rho_s = \frac{\rho(s+e)}{e+1} \quad (8\text{-}9)$$

where:

ρ = density of water (kg/m^3)
ρ_d = dry bulk density (kg/m^3)
ρ_s = saturated bulk density (kg/m^3)
s = specific gravity of soil particles
e = void ratio of soil mass

Detailed descriptions of the type of soil (e.g. clay, lean, lean clay.etc.) have not been provided. It is proposed that the Casagrande classification be followed where "lean clay" is clayey silts (CL), "clay" is clay of medium plasticity (CI), "heavy clay" is taken as clays of high plasticity (CH) and " sandy clay" is well graded sands with small clay content (SC).

The mean flow depth (y) calculated by means of the Equation 8.5 and 8.7, needs to be adjusted in order to calculate maximum flow depths that might result from short term general scour. The recommended factors are provided in table 8.3.

Table 8-3: Factors to Cover Mean Flow Depth (y) to Maximum Channel Depth

Description	Multiplying factor
Straight reach of channel	1.25
Moderate bend	1.50
Sever bend	1.75
Right–angled abrupt run	2.0

(Source: South African Manual 2006)

The basic assumptions for which the Blench equations are valid include:

- The flow is steady;
- Bed transport is negligible;
- Sediment transport through turbulent suspension that is sufficiently limited not to influence the calculations;
- Channels sections and slopes are uniform;
- Viscosity does not vary significantly;
- Conditions under which the equations are applied are similar to conditions for which the equations had been deduced. These conditions represent the ideal situation, which is not often found in rivers under extreme flood conditions. Nevertheless, the equations are approximately valid as long as discharge and bed transport of sediment do not vary too quickly; and
- Flow is in the rough turbulent phase.

Lateral Scour at Abutments in Cohesive Bed Channel:

In cases of cohesive scour at abutments, Faraday and Charlton recommended the use of the appropriate Blench normal depth equation for cohesive beds, with the correction factors for maximum depth given in Table 8.3. Although these factors were derived for alluvial materials, they provide a first estimate of the scour in cohesive soils.

Lateral Movement of Rivers

Lateral movement of the main channel of a river within a floodplain may have serious impacts on scour and bridge stability. These may include increased scour depths at piers and abutments, erosion of approaches to the bridge structure and changed angles of flow relative to piers and abutments. Lateral movement is influenced by factors such as the geomorphologic properties of the stream, bridge locations, and flow characteristics of the bed and bank materials.

Inspection of the river is crucial in understanding the potential for lateral movement of the bridge site. The common-sense understanding of these processes underlying the regime theory may be used as a framework to consider the potential impact of lateral stream movement at the bridge site.

In this regard, the following points may be useful:

- The equilibrium geometry of rivers is established under high, but not extreme, flow conditions;
- A constriction in channel width would lead to increased scour of the river bed;
- Straightening of the river (e.g. in the case of river training) may result in large-scale local erosion if care is not taken; and
- Fixing the channel geometry at one point in an alluvial channel (e.g. at a bridge site) would probably lead to increased channel movement at another location.

The typical scour related problem that could be encountered as part of the dynamic processes in rivers were summarised and are shown in Table 8.4



Photo 8-8: Example photo of lateral channel movement (Arba Minch)

Table 8-4: Typical scour related problems that can be encountered in rivers

River type	Stream type	Materials typically exposed	Dominant geomorphologic processes	Potential scour-related problem
Steep mountainous rivers	Boulder torrent	Bedrock/boulders	Down cutting and waterfalls	Erosion of rivers banks
	Braided gravel river	Sand, gravel, cobbles	Movement of coarse alluvium	Scour, choice of length of openings
	Alluvial fan	Sand, gravel, cobbles,	Deposition of coarse alluvium: sudden channel shifts	Control of approach channel geometry: scour
Stream with moderate slopes	Entrenched river channel	Bedrock, shale	Thin layer of materials is transported	Few
	Laterally meandering rivers	Sand, gravel, cobbles	Widening of river valley, sediment transport	Bank erosion and outflanking of bridge openings, scour,: erosion of bridge approaches
Plains and streams with flat slopes	Meandering river	Sand and silt	Migration of meanders: erosion of river banks	Erosion of river banks and outflanking of bridge openings
		Clay, silt, cobbles	Degradation, erosion of riverbanks.	Erosion of river banks
	Low velocity stream with multiple widening	Silt, sand	Relatively inactive	n/a
	Lake crossings	Silt, clay, organics	n/a	Soft foundation

It is difficult to anticipate when a change in plan form may occur. It may be gradual or the result of a single major flood event. In addition, the direction and magnitude of the movement of the stream are not easily predicted. It is difficult to evaluate properly the vulnerability of a bridge due to changes in plan form; however, it is important to incorporate potential plan form changes into the design of new bridges and design of countermeasures for existing bridges.

Contraction Scour

Rooseboom in TRH 205 indicated that the formulae for short-term general scour might consider contraction scour. The flow width that is used is set to the contraction width and by applying Equation 8.5 or 8.7 the contraction scour based on regime theory is calculated.

Where an existing bridge is evaluated which has a width less than the equilibrium flow width of the channel, or where fixed banks occur, the contraction scour depth has to be determined. In the following relationships, a distinction is made between sediment-laden and clear water flow.

To test whether sediment-laden flow occurs in the river, determine the average particle size, d_{50}, of the sediment in the river upstream of the bridge area. Use figure 8.11 to determine its settling velocity V_{ss}. Calculate the value of Equation 8.4 to determine if the flow is in the laminar of turbulent region. To calculate the shear velocity, apply equation 8.3 for laminar flow or Equation 8.4 for turbulent flow. The velocity at a boundary, between sediment moment and no sediment moment (the critical velocity), V_c, is determined from the logarithmic relationship:

$$V_c = 5.75 V_{*c} log \frac{12R}{k_s} \quad (8\text{-}20)$$

Where:

R and K_s, represent the hydraulic radius and absolute roughness value just upstream of the bridge structure.

If the approach velocity $V > V_c$, sediment-laden flow takes place, else clear water flow occurs. The later tends to occur at bridge openings on floodplains (Relief Bridge) where the velocity is lower and the resistance to scour is greater. Velocity and sediment size become important in clear water scour estimates and therefore a different set of equations is used to estimate construction scour for clear water flow conditions.

Sediment-Laden Flow

For definition of terms see Figure 8.11. The equation below is applicable to a constriction of the river and was adapted from HEC-18, based on the assumption that in Ethiopian conditions the model of bed material transport is mostly suspended bed material discharge:

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \left(\frac{B_1}{B_2}\right)^{2/3} \left(\frac{n_2}{n_1}\right)^{1/3} \quad (8\text{-}11)$$

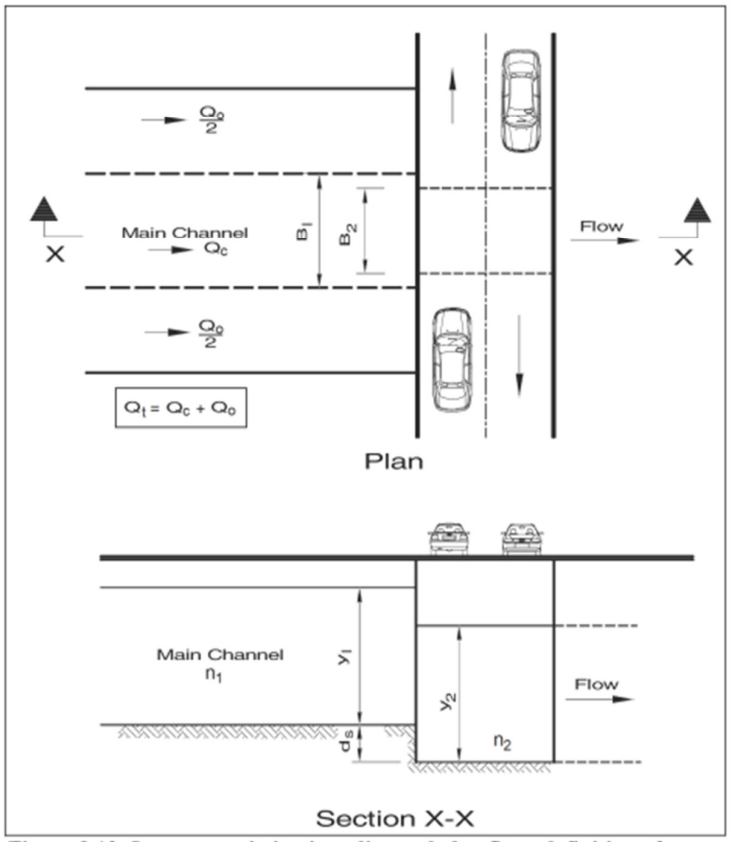


Figure 8-14: Long Constriction in Sediment–Laden Flow: Definition of Terms

(Source: South Africa Manual 2006)

The depth of scour is given by:

$$d_s = (y_2 - y_1) + (1 + K)\left(\frac{\bar{V}_2^{\,2} - \bar{V}_1^{\,2}}{2g}\right) \qquad (8\text{-}12)$$

Where K is the secondary energy loss coefficient for the constriction. If the degree of constriction is slight, it will be found that the last term in Equation 8.12 is negligible, and can be neglected.

The equations above apply to subcritical flow, with uniform flow upstream and downstream of the transition. The bed material is non-cohesive and is identical in both the wide and the constricted parts. The effect of varying sediment characteristics has been investigated and found to play only a significant part in the case of sever constrictions.

Clear Water Flow

For a definition of terms see Figure 8.15. The equation below was taken from HEC- 18

$$y_2 = \left[\frac{Q^2}{40 D_{ss}^{\,2/3} B_2^2}\right]^{3/7} \qquad (8\text{-}13)$$

and d_s is calculated from Equation 8.12.

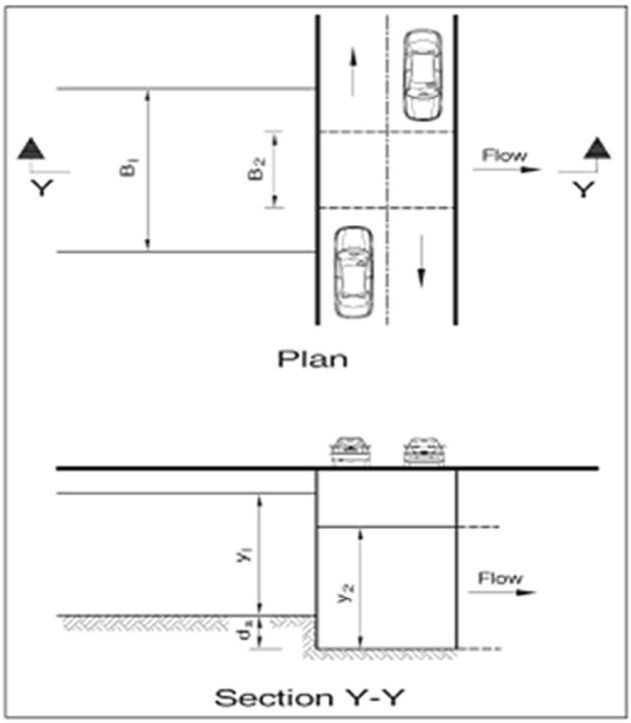


Figure 8-15: Long Constriction in Clear Water Flow: Definition of Terms

(Source: South Africa Manual 2006)

Note that Equation 8.13 is based on a Shields parameter of 0.039 and an assumption of homogeny bed material. The term D_{ss} is the effective mean bed material size and is equal to $1.25D_{54}$.

A constriction of the channel, which may be caused, in part, by bridge piers in the waterway, can result in channel contraction scour (see figures 8.16 and 8.17).

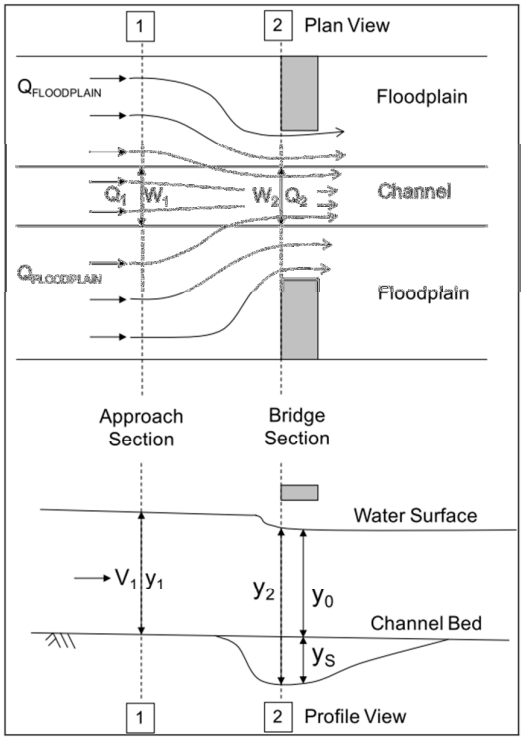


Figure 8-16: Live–Bed Contraction Scour Variable

(Source: Hydraulic Design Series Number 7, 2012).

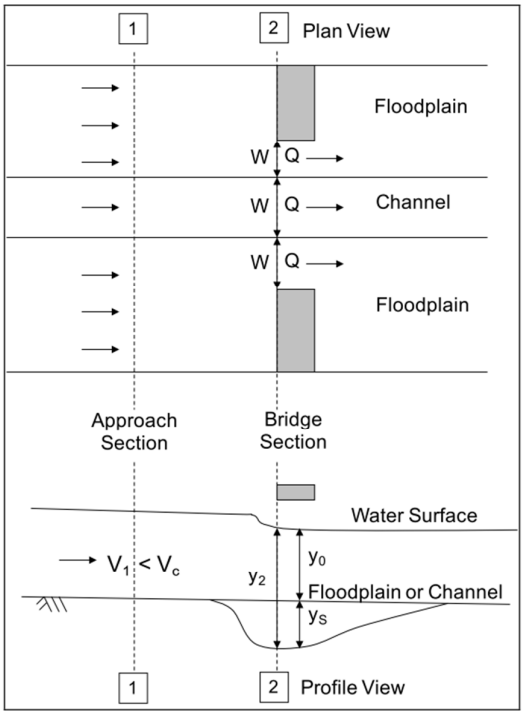


Figure 8-17: Clearwater Contraction Scour Variable

(Source: Hydraulic Design Series Number 7, 2012).

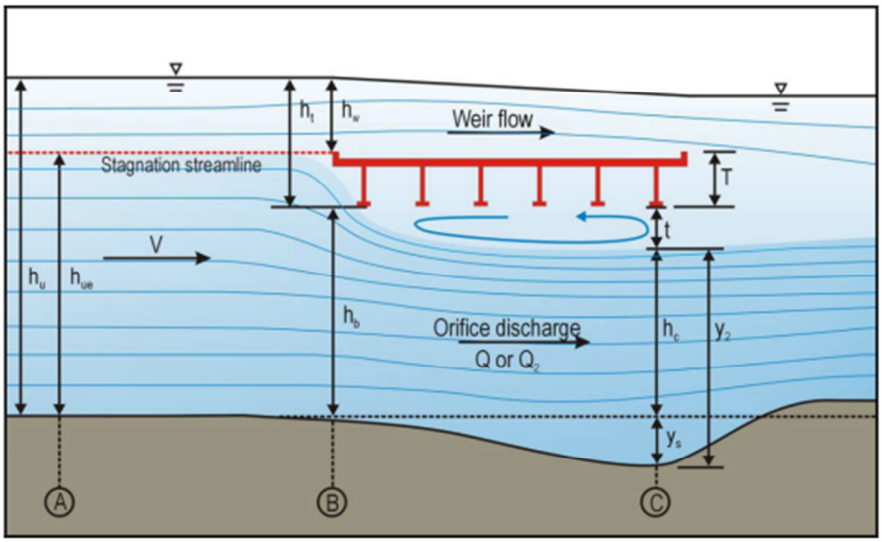


Figure 8-18: Vertical Contraction Scour

(Source: Hydraulic Design Series Number 7, 2012).

Deposition results from an expansion of the channel or the bridge site being positioned immediately downstream of a steeper reach of stream. Highways bridges and natural channel contractions are the most commonly encountered cause of constriction scour. Two practices are provided in this manual for estimating deposition or contraction scour.

- **Sediment routing practice** - This practice shall be considered should either bed armoring or aggradation from an expanding reach be expected to cause an unacceptable hazard; and
- **Empirical practice** - This practice is adapted from laboratory investigations of bridge contractions in non-armoring soils and, as such, must be used considering this qualification. This practice does not consider bed armoring and its application for aggradation may be technically weak.



Photo 8-9: Contraction Scour Adet Quarit Road, Gojjam

Local Scour

The mechanism causing local scour is complex and is characterised by a downward flow at the upstream face of the bridge pier (caused by stagnation pressure - see Figure 8.19), a horse-shoe vortex at the base of the pier (which removes sediments) and oscillating slipstream vortices at the back of the pier that further removes sediments.

Lateral Scour at Piers in Alluvial Channels (Cohesionless Material):

Blench presented the following equation for calculating local scour in cohesionless material at bridge piers:

$$d_s = 1.8{y_0}^{0.75} b^{0.25} - y_0 \quad (8\text{-}14)$$

where:

d = local scour depth at pier (m)
y_0 = depth upstream of pier (m) (calculated by means of regime equation 8.5)
b = pier width (m).

This depth is recommended for local scour at cylindrical piers. Corrections for other pier shapes should be done by multiplying the value obtained from Equation 8.13 with the correction factors in Table 8.5. To take the angle of attack into account, the correction factors in Table 8.6 have to be used.

The CSU equation should be used to compare the result obtained using the above equation.

$$\frac{ys}{b} = 2.0K_1K_2K_3K_4\left(\frac{y1}{b}\right)^{0.35} {Fr_1}^{0.43} \quad (8\text{-}15)$$

where:

y_s = scour depth (m)
y_1 = flow depth directly upstream of pier (m)
K_1 = correction factor for pier nose shape
K_2 = correction factor for angle of attack of flow
K_3 = correction factor for bed condition
K_4 = correction factor for armouring due to bed material size
b = pier width (m)
L = pier length (m)
Fr_1 = Froude number directly upstream of the pier

$$Fr_1 = \frac{\bar{V}_1}{\sqrt{gy_1}} \quad (8\text{-}16)$$

where:

$\overline{V_1}$ = mean velocity upstream of the pier (m/s)
g = gravitational acceleration (9.81m/s^2)

The correction factor is given in Table 8.5 and 8.6

Table 8-5: Correction Factor K_1, for Pier Nose Shape

Shape of pier in plan #	Length/Width ratio (L/b)	K_1
Circular	1,0	1,0
Lenticular	2,0	0,91
	3,0	0,76
	4,0	0,67-0,73
	7,0	0,41
Parabolic nose		0,8
Triangular 60°		0,75
Triangular 90°		1,25
Elliptic	2,0	0,91
	3,0	0,83
Ogival	4,0	0,86-0,92
Rectangular	2,0	1,11
	4,0	1,11 (Hec 18) - 1,40 (F&C)
	6,0	1,11

Table 8-6: Correction Factor K_2, for Angle of Attack of the Flow

Angle (skew angle of flow)	L/b = 4	L/b = 8	L/b = 12
0	1,0	1,0	1,0
15	1,5	2,0	2,5
30	2,0	2,75	3,5
45	2,3	3,3	4,3
90	2,5	3,9	5,0

Table 8-7: Correction Factor K_3, for Bed Condition

Bed condition	Dune Height (m)	K_3
Clear-water scour	Not applicable	1,1
Plane bed and anti-dune flow	Not applicable	1,1
Small dunes	0,6 m – 3 m	1,1
Medium dunes	3 m – 9 m	1,1 – 1,2
Large dunes	≥ 9 m	1,3

The value of correction factor for armouring, K_4, could be determined from the following sets of equations:

$$K_4 = [1 - 0.89(1 - v_R)^2]^{0.5} \quad (8\text{-}17)$$

where:

$$v_R = \left[\frac{v_1 - v_i}{v_{c90} - v_i}\right] \quad (8\text{-}18)$$

And

$$v_i = 0.645 \left[\frac{D_{50}}{b}\right]^{0.053} V_{c50} \quad (8\text{-}19)$$

With:

v_R = velocity ratio
v_1 = approach velocity (m/s)
V_i = approach velocity when particles at pier begin to move (m/s)
v_{c90} = critical velocity for D_{90} bed material size (m/s)
v_{c50} = critical velocity for D_{50} bed material size (m/s)
b = pier width (m)

and

$$v_c = 6.19 y^{1/6} {D_c}^{1/3} \quad (8\text{-}20)$$

Where:

D_c = critical particle size for the critical velocity vc (m)

Local Scour at Piers in Cohesive Bed Channels

The limited data were used to compile a rough guide on the expected scour depths at piers in cohesive material. Table 8.8 reflects the findings.

Table 8-8: Local Scour Depths at Piers in Cohesive Materials

Pier shape (plan)	Inclination of pier faces	Depth of local scour (b=pier width)
Circular	Vertical	1,5b
Rectangular		2,0b
Lenticular		1,2b
Rectangle with semi-circular noses	Vertical	1,5b
	Inclined inwards towards top. (Angle more than 20° to vertical)	1,0b
	Inclined outwards towards top (Angle more than 20° to vertical)	2,0b

Local Scour at Abutments in Alluvial Channels (Cohesionless Material)

Lateral scour at abutments is difficult to quantify. Approximate estimates of scour may be made by applying an appropriate factor selected from the following table (Table 8.9) to the general (short-term) average scour depth in order to obtain a maximum scour depth

Table 8-9: Factors for Estimating Scour Depth at Abutments and Training Works

Description	Factor
Nose of groynes or guidebanks	2,0 to 2,75
Flow impinging at right angles on bank	2,25
Flow parallel to bank	1,5 to 2,0

In cases where the abutment protrude into the river channel, a conservative approach is recommended in which the local scour level is taken as the lower value of the maximum scour at piers and the general scour level multiplied by a factor of 2.0

Local Scour at Abutments in a Cohesive Bed Channel

In the case of cohesive scour at abutments, Faraday and Charlton recommended the use of the appropriate Blench normal depth equation for cohesive beds, with the corresponding factors for maximum depth given in Table 8.3. Although these factors were derived for alluvial materials, they provided a first estimate of the scour in cohesive soils.

A Check Method for Total Scour Based on Applied Stream Power Principles

The equation presented below was derived from applied stream power principles.

The form of equation recommended for the calculation of the total scour in rivers is:

$$\frac{C(Y_t)(v_{ss}k_s)^{1/3}}{q\sqrt{g}} = F \qquad (8\text{-}21)$$

where:

$$C = Chézy\ coefficient$$

$$Y_t = Y_o + Y_s \qquad (8\text{-}22)$$

where:

Y_t = total maximum scour depth (m)

Y_o = maximum general scour depth (m)

Y_s = local scour depth (m)

v_{ss} = particle settling velocity (m/s)

k_s = absolute roughness of river bed (m)

q = discharge per unit width (m^3/s.m)

g = gravitational acceleration (9.81 m/s^2)

F = constant obtained from measured data



Photo 8-10: Local Scour at Bridge Pier Assossa Kumruk Road and Dire Dawa Dechatu River Bridge.

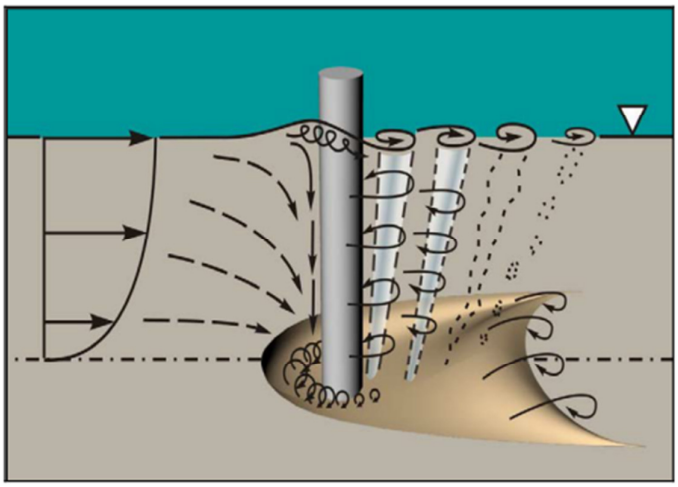


Figure 8-19: The Main Flow Features Forming the Flow Field at a Cylindrical Pier

(Source: NCHRP 2011a)

8.7.4 Naturally Occurring Scour Resistant Materials

Caution is necessary in determining the scour resistance of bed materials and the underlying strata. With smaller size material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour resistant material the maximum predicted depth of scour may not be realized during the passage of a particular flood; however, some scour resistant material may be lost. Commonly, this material is replaced with more easily scoured material. Thus, a later flood may reach the predicted scour depth. Serious scour has been observed to occur in materials commonly perceived to be scour resistant, such as consolidated soils, so-called bed rock streams, and streams with gravel and boulder beds.

8.7.5 Scour Analysis Methods

For detailed scour analysis methods refer to HEC-18 (this document is included in a DVD at the back of the final manual or can be directly downloaded from the FHWA website)

8.7.6 Scour Assessment Procedure

Seven steps that need to be undertaken to conduct an initial estimation of scour at bridges are discussed in the following paragraphs.

Step 1: Data Acquisition Phase

The following information may be relevant in estimating the effect of scour at the bridge site and should be collected:

- Borehole logs define the geology of the bridge site (obtain advice from a geotechnical expert in defining the spacing and depth of drilling required). In addition to valuable information on the properties of the bed material and strata, the logs my potentially yield clues as to previous scour depths at the bridge site;
- Geomorphology of the site;
- Erosion history of the stream where available;
- Development history of the stream, and the catchment;

- Other relevant factors;
- Aerial photographs of the area may yield valuable insights. Also it may be useful to obtaine a comparison of older photographs with more recent ones, where these are available.

From the above information, an initial qualitative assessment of the potential scour impact on the bridge should be made.

Step 2: Analyse Long-Term Change

Preferably, consult an expert, especially if there are any major dams constructed or being planned in the vicinity of a bridge. The expert may use techniques, such as extrapolation of existing trends, worst-case scenarios or sophisticated computer modelling and good engineering judgment to make recommendations on potential long-term effects.

Step 3: Determine Short-Term General and Contraction Scour Effects

The following parameters feed into the scour equations and need to be determined (refer to HEC-18 for scour equations):

- Estimate the design flood for the site as per Chapter 5, hydrology, and use this value for the further calculation;
- Estimate the contracted flow width at the bride site for determining short-term general and contraction scour effects. The unit width discharge could be calculated by dividing the design flood by the flow width;
- Make estimate of the bed roughness under flood conditions and representative sediment material sizes to be used in calculations:

For Alluvial Material

- Use Equation 8.5 to determine mean flow depth and equation 8.6 to determine mean channel width (if width is required), with side factor values from table 8.2;
- Convert mean flow depth to maximum flow depth using the factors in table 8.4;
- Calculate the scour depth which is the difference between the maximum flow depth and the normal flow depth;

For Cohesive Material

- Estimate cohesive material properties using table 8.2 and equation 8.7 and 8.9;
- Use equation 8.7 for mean flow depth;
- Covert the maximum flow depth using table 8.3;
- Calculate the scour depth as the difference between the maximum flow depth and the normal flow depth.

Contraction scour is factored into step 3 through the judicious choice of flow width at the bridge site. If an existing bridge or a channel with fixed banks is considered, apply the set of constriction scour Equations 8.10 to 8.13, the maximum scour depth from the different analyses should be used.

Step 4: Determine Local Scour Effects

For Piers in Alluvial Cohesionless Materials

- Use Equation 8.14 and 8.18 to compute local scour into different ways. Obtain the factors needed for Equation 8.14 from Table 8.4 and 8.5. Obtain the factors needed for Equation 8.15 from Tables 8.6, 8.7 and 8.8 and Equations 8.17, 8.18, 8.19 and 8.20. Compare answers obtained from Equations 8.14 and 8.15 and select a conservative answer using good engineering judgment.

For Piers in Cohesive Materials

- Use table 8.8 as a rough guide to estimate local scour at the piers.

For Abutments in Alluvial Cohesionless Materials

- Apply factors in Table 8.9 to the general short-term average scour depth obtained from Equation 8.8.

For Abutment in Cohesive Materials

- Apply the factors in Table 8.9 to the short-term average scour depth obtained from Equation 8.6. This is a preliminary indication only.

Step 5: Determine Total Scour

Total scour is the sum of the long-term general scour (where applicable), short-term general scour, (contraction scour) and local scour.

The total scour at piers in alluvial rivers, check the answers against values obtained by means of Equation 8.21 and 8.22. Select design values on the bases of good engineering judgment. Try to corroborate calculations with available published on-site evidence.

Step 6: Plot Design Values

Plot the design scour depth values using the design water level (e.g. 1:50 year return period water level, as determined by the method described in Chapter 6 and 7 for a fixed–bed configuration).

Step 7: Assess the Result Obtained

Assess the results, taking into account all available qualitative and quantitative information available. In case of significant complexity, or cases where significant financial and other risks exist, consider consulting an expert or doing a physical hydraulic model study at the bridge site.

Concluding Remarks on the Relationships for Scour at Bridges

The equations presented in this section provide bridge designers with simple methods in which an initial estimate of the potential effects of scour at bridges may be made. If serious problem are foreseen, expert advice should be acquired.

Computer Models

Computer models like HEC-RAS are recommended to be used for scour assessment at bridges for all bridge design. For details of recommended computer models to simulate sediment transport, refer to Chapter 15.

8.8 Scour Countermeasures at Bridge Crossings

Bridge scour and stream instability problems have always threatened the safety of Ethiopian's Roads. Countermeasures for these problems are defined as measures incorporated into a road-stream crossing system to monitor, control, inhibit, change, delay, or minimize stream instability and bridge scour problems. An action plan for monitoring structures during and/or after flood events can also be considered a countermeasure.



Photo 8-11: Bridge Crossing with Countermeasures Near Mekelle Town Tigray

Scour countermeasures at bridges are aimed at reducing the negative impact of shear stresses and turbulence and velocity variations near the boundary of the structure-water interface, and of micro turbulence flow processes, such as eddies and helicoidal flows around bends.

Observations provide the most positive indication of erosion potential. Observation comparison can be based on historic information or current site conditions. Aerial photographs, old maps, survey notes, bridge design files, river survey data, and inspection report however, most of this information are not often readily available in Ethiopia. Interviews of long-time residents, ERA maintenance personnel and ERA construction engineers can provide documentation of any recent and potentially current channel movement or bank instabilities.



Photo 8-12: Example Insufficient Scour Countermeasures Sodo Sawla Road

This section of the manual does not attempt to provide a comprehensive coverage of all available scour countermeasures. Instead, it provides information and guidance on some of the most used options that are available and potentially useful under local conditions in Ethiopia. Further research and development is required in this area.

Hydraulic countermeasures (mainly river training structures and revetments) are designed to either modify erosive flow characteristics or to provide resistance against hydraulic and turbulence effects.

- **River training structures:** These structures may have to be put into place to counter the effect of bank erosion, migrating meanders and other dynamic river processes at the bridge-crossing site. They are divided into transverse, longitudinal and area types, depending on their orientation relative to the flow direction. The structures that are broadly discussed in the following sections are **spurs** (transverse) and **dykes** and **berms** (or **guide banks**) (longitudinal).
- **Revetment and bed armouring:** These measures protect channel beds and banks against the erosive effects of river flow through the provision of protective layer, covering a specified area of the channel. Appropriate revetment types may include riprap, gabions, precast concrete blocks, in-situ concrete and still sheet piling. *Considerations that need to be taken into account when making a selection include the extent of protection needed, cost of construction, maintenance and environmental issues.* Structural countermeasures involve the design of structural elements and foundation of bridges to minimize scour effects. Countermeasures during the maintenance phase include monitoring and implementation of further scour countermeasures when scour problems are identified.

General considerations in respect of scour countermeasures:

- Scour countermeasures at bridges should be seen as an integrated approach aiming to reduce the risk of scour damages to vulnerable structures. The approach should be a considered one in which design and construction of scour countermeasures are complemented with actions, such as continuous monitoring and where necessary, ongoing maintenance and expansion of scour protection measures;
- The effective design and implementation of scour countermeasures should be an interdisciplinary effort, with inputs from hydraulic, geotechnical and structural engineers;

- Selection of scour protection measures should be undertaken with environmental impact, construction and maintenance implications kept in mind; and
- Most importantly, the benefit of scour protection should be measured against the cost of provision of these countermeasures. For new bridges, the most economical option tend to be the lowering of foundation depths to levels well below estimated scour depths and ensuring that abutments are placed back from eroding river banks.

It is important to keep in mind that countermeasures themselves are often damaged and that erosion process may take place in locations where this may not have been envisaged. This means that a large component of scour protection measures may often be implemented during the maintenance phase, as an economical option.

The FHWA recommends the following principles in the design of bridge scour countermeasures:

- Comparison of costs against benefits is of prime importance except in certain cases where routes are strategic importance;
- Designs should be based on channel trends and experience of similar field situation is extremely valuable;
- The environmental impacts of scour countermeasures have to be addressed;
- The designers should personally undertake a field inspection trip of the site and the river and catchment upstream and downstream of the bridge site;
- Any previous evidence of dynamic changes in the vicinity of the site (such as early photographs), is useful;
- Geotechnical and soil characteristics that may impact on the design of countermeasures should be determined and taken into account;
- Many of the countermeasures induce complex interactions with the river and its environment. This means that a physical hydraulic model study may often by justified to study the impact of these complex interactions and to determine potential unforeseen effects; and
- Often the dynamic nature of scour processes implies that not all effects may be foreseen at the stage that the bridge is initially designed and constructed. This means that an inspection and maintenance plan is usually essential in order to affect ongoing countermeasure as and when required.

In the following paragraph some of the scour countermeasures are discussed in more detail.

8.8.1 Spurs, Berms and Dykes

Spurs (Groynes) are intended to control the movement of river meanders and erosion of river banks. This may be placed either upstream of the area to be protected. Spurs should preferably be used in groups to either repel or attract flow. Spurs usually require some scour protection themselves. It is recommended that the following formula for groups of spurs be used.

$$L_s < \frac{Cy^{1.33}}{2gn^2} \qquad (8\text{-}23)$$

Where:

L_s = space between spurs (m),
C = a constant (approxumately 0.60)
y = mean depth of flow (m)
n = Manning's roughness coefficient
g = gravity acceleration (m/s^2)

On similar narrow rivers, it should be carefully considered whether spurs offer the most appropriate form of erosion protection, as the use of these on one bank may result in erosion on the opposite bank.

The interaction of factors influencing the layout and spacing of spurs is complex and a model study is recommended for most cases. An expert should preferably be involved in the design of spurs. Economic considerations should feature strongly in the final layout decision.

8.8.2 Berms (Guide Banks)

Berms may have various uses, including:

- Protecting the bridge and approaches from erosion through elimination of lateral stream movement patterns;
- Improving flow distribution through the bridge opening and in the process also reducing the effects of contraction scour; and
- Changing the angle of the approach flow.

The main design issues are orientation relative the bridge opening, shape length upstream and downstream of the bridge and crest levels.

The plan layout of berms should conform to good hydraulic principles, in addition to providing adequate protection against lateral stream movement (see Figure 8.20 below). Their height should normally allow for the design water level plus freeboard. Guide banks normally need some protection against undermining. An expert should preferably undertake or review the design of guide banks.

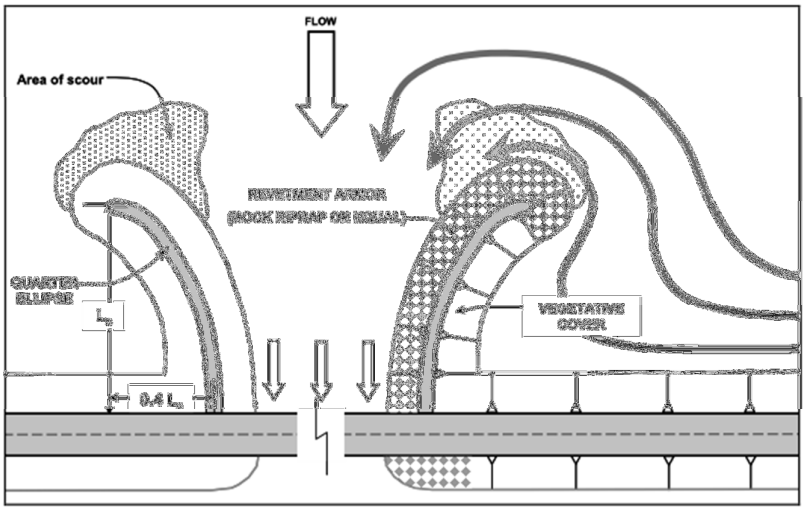


Figure 8-20: Typical Guide Bank

(Source: Modified from FHWA 1978)

8.8.3 Dykes

Dykes are linear structures that prevent or control over bank flow. These may typically be used to prevent flood flow from bypassing the bridge opening. Hydraulic model studies to optimise the layout of dykes are usually justified for major bridges.

8.8.4 Revetment: Riprap Protection

Riprap is defined as follows:

"Flexible channel or bank lining or facing consisting of a well graded mixture of rock, broken concrete, or other material, usually dumped or hand-placed, which provides protection from erosion."

Riprap protection serves its main purpose in reducing the effects of local scour. It has been popular in many countries because of the availability of materials, ease of construction and low costs.

Riprap layers fail in various ways, which should be considered during the design processes. Some of these failures modes may even cause protection to fail where the stone sizes are adequate. The failures that might be encountered with riprap layers are summarized below.

Riprap particle erosion: Various factors may contribute to this type of failure. These include inadequate stone sizing, removal of particles through various causes and steep slopes where the angle of repose of the material could be easily overcome. This failure mode is limited by sizing of particles to withstand hydraulic and turbulent forces, but factors such as slope, waves and vandalism may not be addressed in this way.

Mass failure: This failure mode happens when a large mass of the riprap slides or slumps because of the effect of gravity. This may happen because of pore water pressure, steep slopes, or loss of support (such as undermining of toe of support at abutments) that may be ascribed to erosive processes. Undermining and failure of toe support is considered to be a primary reason for revetment failure and significant attention should be paid to this aspect during design and construction.

Substrata particle erosion or base material failure: In this case, the underlying materials are displaced though erosion or fails and slumps. This may be countered through the use of filters. However, care has to be exercised in the design of the filters to ensure that water pressure does not build up due to filter blockages.

Special care should be taken with the edges of revetment, such as the head, toe and flanks, to ensure that undermining does not occur.

Riprap Stone Sizing

Much of the understanding of riprap design is based on laboratory experiments, with limited field data to verify this understanding. This should be kept in mind when using the design formulae presented for use.

A relationship for the required particle size that was derived from tractive force theory has been recommended for the determination of the riprap size. The formula was derived for straight channels with uniform flow conditions, but a coefficient in the formula allows for flow conditions that deviate from this ideal conditions. The formula is:

$$D_{50} = \frac{K_u \bar{V}_a{}^2}{d_{avg}{}^{0.5} K_1{}^{1.5}} \qquad (8\text{-}24)$$

Where:

D_{50} = median riprap size (m)
K_u = 0.0059
C = coefficient for a specific gravity and stability factors
$\overline{V_a^2}$ = average velocity in the main channel (m/s)
d_{avg} = average depth in the main channel (m)
K_1 = a factor defined by equation 8.28

$$K_1 = \left[1 - \left(\frac{sin^2\theta}{sin^2\varphi}\right)\right]^{0.5}$$

where:

θ = bank angle with the horizontal (^{0})
φ = riprap angle of repose (^{0})

The angle of repose for various riprap types may be read from Figure 8.12

The coefficient for specific gravity and stability factors may be obtained from the following equations:

$$C = \frac{1.61(SF)^{1.5}}{(S_s-1)^{1.5}} \quad (8\text{-}25)$$

Here the following values for the variable are used:

Ss = specific gravity of riprap (usually approximately 2.65)
SF = required stability factor to be applied (typically varies between 1.2 and 2.0)

The recommended values for the stability factor, SF, are shown in Table 8.10.

Table 8-10: Recommended Values for Stability Factor, SF

Flow description	Stability Factor
Uniform flow, no significant bends, little uncertainty regarding parameters involved.	xx– 1.2
Gradually varied flow, moderate bend curvature, moderate debris impact.	1.3 – 1.6
Rapidly varied flow: significant bends; high turbulence (e.g. at abutments); high parametric uncertainty.	1.7 – 2.0

Grading of Riprap and Stone Characteristics

Riprap should in general be well graded in order to ensure maximal interlocking of particles. Individual stones should have a length to width ratio of not more than 1:3. In addition, materials should not weather easily or be prone to chemical wear. The grading in Table 8.11 is recommended for riprap.

Table 8-11: Recommended Grading of Riprap

Diameter	Weight	Percentage Passing
$1.5D_{50}$ to $1.7\ D_{50}$	$3.0\ W_{50}$ - $5.0\ W_{50}$	100
$1.2\ D_{50}$ to $1.4\ D_{50}$	$2.0\ W_{50}$ - $2.75\ W_{50}$	85
$1.0\ D_{50}$ to $1.15\ D_{50}$	$1.0\ W_{50}$ - $1.5\ W_{50}$	50
$0.4\ D_{50}$ to $0.6\ d_{50}$	$0.1\ W_{50}$ - $0.2\ W_{50}$	15

It is recommended that the 85% requirement be dropped in the case where the above specification would overburden certain smaller quarries.

Riprap Layer Thickness

Guidelines stipulate that riprap layer thickness should generally not be less than the greater of the D_{100} stone diameter or 1.5 times the D_{50} stone diameter. The layer should have a minimum thickness of 300 mm for practical purposes. In case where the riprap is placed under water, layer thickness should be increased by 50% because of uncertainties involved in the placement processes.

Design of Filters and Filter Materials

Filters have a dual purpose to fulfill as part of scour protection system. Firstly, the filters have to prevent fine materials from leaching out underneath the riprap layers or other protection layers. Secondly, the filter materials have to provide for drainage to prevent build – up of pore pressure. A geotechnical specialist should preferably undertake the design or be consulted at the design stage.

As a broad guideline, the following formula may be used for the design of filter materials.

$D_{15\ (Coarser\ layer)}/D_{85\ (Finer\ layer)} < D_{15\ (Coarser\ Layer)}/D_{15\ (Finer\ layer)} < 40.$

The left side of the inequality provides for erosion prevention and the right side for sufficient permeability.

8.8.5 Revetment: Gabions and Stone Mattresses

Gabions and stone mattresses may be an option where available rock is of too small and of lesser quality than would be required for riprap. The manufacturer of gabions and rock mattresses is labour intensive, which may be considered an advantage in instances where local job creation is considered an important component of projects. This is important from the Ethiopian government policy of job creation for the youth.



Photo 8-13: Sample Photo of Gabion Wall Protection

(Source: *http://www.ieca.org/*)

However, the disadvantages of gabions and stone mattresses have to be carefully considered prior to use. This includes the higher costs of installation and maintenance relative to riprap and the decrease for flexibility of the system that is created. Whereas riprap particles may move individually to adapt to geometrical deformation, gabions and stone mattress systems are more rigid. More rigid systems generally have higher potential for catastrophic failure.

The failure mechanisms that need to be considered in the case of gabions and stone mattresses are:

- Failure of the wire mesh of the baskets; and
- Movement of stones within the basket that exposes base materials, with the potential of subsequent base material erosion and system failure.

For these reasons, gabions and stone mattresses are only recommended for small streams, preferably with no vertical stability problem.

8.8.6 Local Scour or Armoring

Sizing of riprap at piers

The standard Isbash formula, recommended by FHWA in HEC-11 is:

$$D_{50} = \frac{0.692(Kv)^2}{2g(S_1 - 1)} \qquad (8\text{-}26)$$

Where:

D_{50} = riprap size (m)
v = velocity along the pier (m/s)
S_s = specific gravity of riprap (approximately 2.65)
K = pier shape coefficient (1.5 for round and 1.7 for rectangular pier)

The velocity along the pier may be calculated by multiplying the average channel velocity by a coefficient that varies between 0.9 for a pier near the bank of a uniform reach to 1.7 for a pier in the main current at a bend in the river.

8.8.7 Riprap Mat Dimensions at Piers

The following may be considered as guidelines to the dimensions of riprap protection that are required (Table 8.12)

Table 8-12: Recommended Riprap Dimensions

Dimension	Recommended
Horizontal	Twice pier width of both sides
Thickness	At least three stone diameters (D_{50})
Maximum rock size	Not more than twice D_{50} of riprap

FHWA considers it be disadvantageous to place riprap at such a depth that the top of the riprap is buried below stream level, as this creates problems during inspections to determine the extent to which riprap has been damaged or removed. The top level of the riprap layer should, therefore, be at the invert level of the streambed. This is in contrast with the view of other references that the top level of the riprap should be below the expected general scour levels. *Riprap should, however, never be placed at levels higher than the streambed invert.*

8.8.8 Sizing of Riprap at Abutments

Two equations are recommended by HEC-11, for the sizing of riprap at abutments:

For Froude numbers less or equal to 0.80 – the following Isbash relationship is recommended:

$$\frac{D_{50}}{y} = \frac{K}{S_s - 1}\left[\frac{\bar{V}^2}{gy}\right] \qquad (8\text{-}27)$$

Where:

D50 = median store diameter (m)
$\bar{V}$ = characteristic average velocity in the contracted section
Ss = Specific Gravity of riprap
G = gravitational acceleration (m/s^2)
Y = depth of flow in the contracted bridge opening (m)
K = 0.89 for a spill – through abutment
= 1.02 for vertical wall abutment

For Froude number in excess of 0.80 – the recommended equations reads

$$\frac{D_{50}}{y} = \frac{K}{S_s - 1}\left[\frac{\bar{V}^2}{gy}\right]^{0.14} \qquad (8\text{-}28)$$

With the symbols having similar meanings as in the previous equation, and

K = 0.61 for spill- through abutments or
K = 0.69 for vertical wall abutments

The characteristics average velocity in the contracted section method of calculation is as follows:

A setback ratio (SBR) is calculated for each abutment. (This is the ratio of the distance from the near edge of the main channel to the toe of the abutment to the channel flow depth). If the SBR is less than five for both abutments, the characteristic velocity could be based on the overbank flow only. It should be assumed that the entire overbank flow passes through the bridge section in the overbank section only.

In cases where the SBR on one side is less than 5 and on the other side more than 5, the characteristic velocity for the side less than 5 should be based on a flow area bounded by the abutment and an imaginary boundary along an opposite main channel bank,

It is recommended that the HEC-11 equation described in the previous section be used as well as a control method before an engineering judgment is made. In this case, the velocity in the vicinity of the abutment should be used, instead of the main channel velocity, with a stability factor of between 1.7 and 2.0 to allow for high turbulence. It should be remembered that all these equations are mainly based on laboratory data and should be treated as such.

In complex cases or where damage may be costly, scale model studies may be an attractive option. It should be remembered that riprap is not generally the preferred method of scour protection for new structures and alternative protection measures should be considered.

8.8.9 Dimensions of Riprap Protection at Abutments

The extent to which riprap should be placed is as follows:

- The apron at the toe of the abutment should extend along the entire abutment toe around curved portions to the tangent point with the embankment slopes;
- The horizontal dimensions of the riprap into the overbank area of the river should be twice the flow depth on the overbank area in the vicinity of the abutment, but not less than1.5 m or more than 7.5m;
- For spill-through abutment, riprap layers should be placed to a height of at least 0.6m above the high water level for the design flood. Downstream placements should extend back from the abutment for twice the flow depth, or 7.5 m (to 15m), whichever, are greater; and
- Riprap layer thickness should not be less than the larger of 1.5 times D50 or D100. If the riprap is placed under water, this thickness should be increased 1.5 times. The minimum thickness should be 300 mm.

Abutments should be inspected after every major flood event for scour damage.

8.8.10 Structural Countermeasures

Adherence to certain general principles during the design of the bridge may reduce the effects of scour. These may include:

- Locating the invert levels of pier footing well below the maximum estimated scour levels and preferably on rock, where possible;
- Putting piers on piles or columns that extend to a great depth below scour levels;

- Designing slender structural elements that do not provide a significant flow obstruction and orienting piers and abutments to ensure good hydraulic flow patterns;
- Reduction of number of piers and widening of bridge opening to reduce contraction scour; and
- Designing the bridge superstructure to be above water (and debris) level at design flood levels will reduce the risk of pressure scour, which could have a sever effects,

8.8.11 Maintenance Measures

It is imperative to note that maintenance measures include not only the continuous monitoring of scour patterns at bridges, but also knowing which actions need to be taken once a problem has been identified.

Maintenance measures include inspection of scour prone bridges after flood events, the removal of debris caught on piers and the repair of any scour damage observed after major flood event.

8.9 Deck Drainage

Improperly drained bridge decks can cause numerous problems including hydroplaning. Bridge decks should drain over the side. Where curb and gutter is used deck drainage is carried to the ends, the bridge decks shall be watertight, and drains at the end of the bridge should have sufficient inlet capacity to carry all bridge drainage.

Where it is necessary to intercept deck drainage at intermediate points along the bridge, the design of the interceptors should conform to the HEC-12 procedures presented in *Chapter 10: Storm Drainage Facilities*, and to the ERA *Standard Detail Drawings.*



Photo 8-14 : Sample Photos of Inappropriate Bridge Deck Drainage Design

(Adet-Quarit road, Gojjam and Tigray on Tekeze Dam site road)

8.10 Construction/Maintenance

Construction plans shall be reviewed jointly by the Contractor and the Hydraulic Engineer to note any changes in the stream from the conditions used in the design. Temporary structures and crossings used during construction shall be designed for a specified risk of failure due to flooding during the construction period. The impacts on normal water levels, fish passage, and normal flow distribution must be considered.

All borrow areas existing within the flood plain shall be chosen so as to minimize the potential for scour and adverse environmental effects within the limits of the bridge and its approaches on the flood plain.

The stream-crossing design should incorporate measures which reduce maintenance costs whenever possible. These measures include spur dikes, retards, guide dikes, jetties, riprap protection of abutments and embankments, embankment overflow at lower elevations than the bridge deck, and alignment of piers with the flow.

8.11 Waterway Enlargement

There are situations where roadway and structural constraints dictate the vertical positioning of a bridge and result in a small vertical clearance between the low chord and the ground. Significant increases in span length provide small increases in effective waterway opening in these cases.

It is possible to increase the effective area by excavating a flood channel through the reach affecting the hydraulic performance of the bridge. However, several factors must be accommodated when this action is taken:

The flow line of the flood channel shall be set above the stage elevation of the dominant discharge;
The flood channel must extend far enough up and downstream of the bridge to establish the desired flow regime through the affected reach; and
The flood channel must be stabilized to prevent erosion and scour.

8.12 Auxiliary Openings

The need for auxiliary waterway openings, or relief openings as they are commonly termed, arises on streams with wide flood plains. The purpose of openings on the flood plain is to pass a portion of the flood flow in the flood plain when the stream reaches a certain stage. It does not provide relief for the principal waterway opening in the sense that an emergency spillway at a dam does, but has predictable capacity during flood events.

Basic objectives in choosing the location of auxiliary openings include:

- Maintenance of flow distribution and flow patterns;
- Accommodation of relatively large flow concentrations on the flood plain;
- Avoidance of flood plain flow along the roadway embankment for long distances; and
- Crossing of significant tributary channels.

The technological weakness in modelling auxiliary openings is in the use of one-dimensional models to analyze two-dimensional flow. The development of 2-D models is a major step toward more adequate analysis of complex stream-crossing systems.

The most complex factor in designing auxiliary openings is determining the division of flow between the two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event. The design of auxiliary openings should usually be generous to guard against that possibility.

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APPENDIX 8A - WORKED EXAMPLES

Worked Example 8A-1 Backwater at a bridge (adapted from the South African manual)

Problem description Example 8A-1

Determine the backwater caused by a bridge across a river. Details of the bridge are shown below in Figure 8a-1.

Design discharge	Q = 150m3/s
Average bed slope	S_0 = 0.00082m/m
Angle of skew	$\theta = 15^o$
Bridge span on skew	bs = 17.6m
Projected bridge span	b = 17.0
No of rows of piers	Np = 1
projected width of pier	Wp = 2.00m

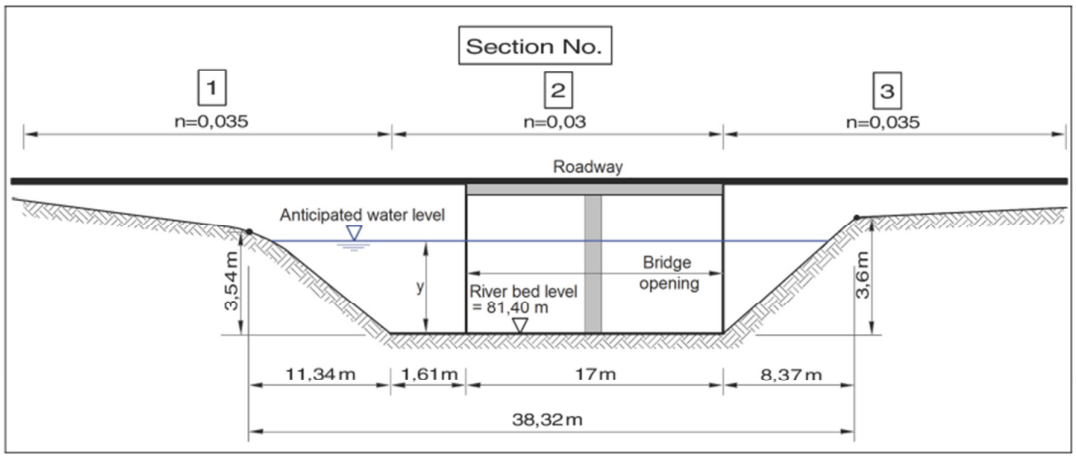


Figure 8A-1: Upstream view of the bridge

Determine:

1 Characteristics of the unconstructed flood state
2 The flow type
3 Bridge opening ration
4 Velocity head coefficient
5 Calculated backwater

Solution Example 8A-1

1 Characteristics of the unconstructed flood state

First determine the normal flow depth, y_n,

Sub-section	n_i	A_i(m)	P_i(m)
1	0.035	$1.602y^2$	$(y^2 + 10.26y^2)^{0.5}$
2	0.030	$18.61y^2$	18.61
3	0.035	$0.163y^2$	$(y^2 + 5.41y^2)^{0.5}$

Utilising the Manning's equation the normal flow depth can be calculated, y_n = 3.157

Sub-section	A_i (m^2)	P_i (m)	$R_i = \frac{A_i}{P_i}(m^{2)}$	q_i (m^3/s)	$\overline{V}_i = \frac{q_i}{A_i}(m/s)$
1	15.96	10,59	1.51	17.17	1.08
2	58.75	18.61	3.61	120.69	2.05
3	11.59	7.99	1.45	12.14	1.05
	86.30	**37.19**		**150.00**	

The result in the:

Flood satage level = 84.56m

Width at flood stage = 56.06m

2 Determine flow type

$$Fr_n = \left(\frac{Q^2 B}{g A_n{}^3}\right)^{1/2} = \left[\frac{(150)^2(36.06)}{(9.81)(86.30)^3}\right]^{1/2}$$

$$= 0.359 < 1$$

Flow is Type I or Type II.

Calculate specific energy (E_{sn}) of unconstricted normal flow:

with y_n = 3.157 m (Flood stage level – river bed level)

$$\bar{V}_n = \frac{Q}{A_n} = \frac{150}{86.30}$$

$$= 1.738 m/s$$

$$E_{sn} = y_n + \frac{\bar{V}_n{}^2}{2g} = 3.157 + \frac{(1.738)^2}{2(9.81)}$$

$$= 3.311 m$$

Calculate specific energy (E_{sc}) of constricted flow critical depth:

$$y_{2c} = \left(\frac{Q^2}{gb^2}\right)^{1/3} = \left(\frac{(150)^2}{(9.81)(17)^2}\right)^{1/3}$$

$$= 2.817 m$$

$$\bar{V}_{2c} = \frac{Q}{y_{2c} b} = \frac{150}{(1.817)(17)}$$

$$= 3.132 m/s$$

$$E_{sc} = y_{2c} + \frac{\bar{V}_{2c}{}^2}{2g} = 2.817 + \frac{(3.132)^2}{2(9.81)}$$

$$= 3.317 m > E_{sn} indicating\ Type\ II\ flow$$

Because the value of Esn and Esc are fairly close, and other losses are so far ignored, it would be prudent to check Type I and Tyope II flow:

3 Calculate bridge opening ratio

$$Q_b = (120.69)\left(\frac{17}{17+1.61}\right)$$

$$= 110.25 m^3/s$$

$$M = \frac{Q_h}{Q} = \frac{110.25}{150}$$

$$= 0.735$$

4 Calculate velocity head coefficients

$$a_1 = \frac{\sum q\bar{v}^2}{Q\bar{v}_n{}^2}$$

$$= 120$$

$a_2 = 1.15$ (from Figure 8.6)

Calculate backwater:

For Type 1 flow:

Determine secondary energy loss coefficient K^* from Figure 8.7.

Projected area of piers in flow direction and projected area below normal water level.

$$A_p = W_p y_n$$

$$= (2)(3.157)$$

$$= 6.314 m^2$$

$$A_{n2} = (b\cos\emptyset)(y_n)$$

$$= (17\cos(15))(3.157)$$

$$= 51.84 m^2$$

$$J = \frac{A_p}{A_{n2}} = \frac{6.314}{51.84}$$

$$= 0.122\ (use\ J = 0.1\ from\ \boldsymbol{Figure\ 8.7})$$

Eccentricity

$$e = 1 - \frac{Q_a}{Q_c} = 1 - \frac{12.14}{17.17 + (120.69)\left(1 - \frac{17}{17+1.61}\right)}$$

$$= 0.44$$

From Figure 8A-1 and with θ= 15^{o}

K* = 0.73

Approximate backwater (to estimate A_1 in Equation 8,2)

$$\bar{v}_{n2} = \frac{Q}{A_{n2}} = \frac{150}{51.84}$$

$$= 2.894 m/s$$

Final estimate of backwater:

$$h_1{}^{*1} = h_1{}^{*1} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right]\frac{\bar{v}_{n2}{}^2}{2g}$$

$$= (0.358) + (1.20)\left[\left(\frac{51.84}{86.30}\right)^2 - \left(\frac{51.84}{99.22}\right)^2\right]\left[\frac{(2.894)^2}{2(9.81)}\right]$$

$$= \mathbf{0.403}\boldsymbol{m}$$

For Type II flow:

$$b_c = (b - \sum W_p) = 17.0 - 2.0$$

$$= 15.0m$$

$$C_b = 0.125 \; from \; Figure \; 8.9$$

$$y_{2c} = \left(\frac{Q^2}{g b_c{}^2}\right)^{1/3} = \left[\frac{(150)^2}{(9.81)(15)^2}\right]^{1/3}$$

$$= 2.168m$$

$$\bar{y} = \frac{A_{n2}}{b} = \frac{51.84}{17}$$

$$= 3.050m$$

In 1st iteration, assume

$$\bar{v}_1 = \frac{Q}{A_n} = \frac{150}{86.30}$$

$$= 1.738 m/s$$

For $\bar{v}_{2c}$ based on the net width:

$$\bar{v}_{2c} = (g y_{2c})^{0.5} = ((9.81)(2.168))^{0.5}$$

$$= 4.612m/s$$

$$h_1^{*1} = a_2 \frac{\bar{v}_{2c}^{\,2}}{2g}(C_b + 1) - a_1 \frac{\bar{v}_1^{\,2}}{2g} + y_{2c} - \bar{y}$$

$$= \frac{(1.15)(4.612)^2(0.125+1)}{2(9.81)} - \frac{(1.20)(1.738)^2}{2(9.81)} + (2.168) - (3.050)$$

$$= 1.403 - 0.185 + 2.168 - 3.050$$

$$= 0.336m$$

Adjust result for improved value of $\bar{v}_1$:

$$A_1 = A_n + h_1^{*1}B = (86.30) + (0.336)(36.06)$$

$$= 98.43m^2$$

$$\bar{v}_1 = \frac{150}{98.43}$$

$$= 1.524m/s$$

$$h_1^{*1} = 1.403 - \frac{(120)(1.524)^2}{2(9.81)} + 2.168 - 3.050$$

$$= 0.379m$$

Although the difference in this case is negligible, to be conservative, the higher value should be used. From the calcualtion h_I^{*1} for TYPEII flow was **0.379 m** which is less than the backeater calcualted fro Type I flow, thus Type I flow previals i.e. h_I^{*1} = **0.403m.**

Note that this example was also modelled in HEC-RAS and that the highest backwater was obtained by the standard Step Energy Method. The backwater 300 mm, which is less than the value of 403 mm obtained above. However, in this model the effective flow area option had been used. The model was then re-run, with this option switched off and a higher backwater of 508 mm was obtained in the revised model.

Users of HEC-RAS should therefore carefully consider the options where the bridge approach conditions are smoothed, thereby reducing the backwater.

Worked Example 8-2- Bridge Design with HEC-RAS

Input data

Design floods

Since there is no gauged Rivers data, SCS method is applied to estimate the design flood.

Estimation of Design Floods at the Bridges

River Name	= Kelkeli (Tigray Region)
Eastings (UTM)	= 573210
Nothings (UTM)	= 1327607
Catchment area (km^2)	= 99.97
Design Flood (m^3/s)	50 year = 172.9
	100 year = 201.9

Estimation of Hydraulics parameters

Manning coefficient of the existing bridges is determined based on filed survey of the natural channels of the rivers and the longitudinal slopes of the rivers from topographic survey. The parameters of the river channels of the bridges are given below:

Channel slope	=	0.015
Manning's Coefficient	=	0.03

Bridge Size Determination

The Bridge opening data (i.e. bridge total span, no piers and other bridge parameters) obtained from the structural survey and piers coefficients were estimated:

Bridge span	= 38m
No of Spans	= 2
Pier Spacing	= 21.5m
Height	= 4.5m
Low cord Elevation	= 1471.5m AOD
High cord Elevation	= 1472m AOD
Shape of pier	= Semi-circular
Bridge Pier Coefficients	= 1.2

Determination of X-sections for hydraulic analysis

Different cross section intervals are used for the bridge, and for suitability 20m is used, which may vary according to the suitability of the river section survey data. The layout of the x-sections and profile along the river is worked out for the bridge as shown below.

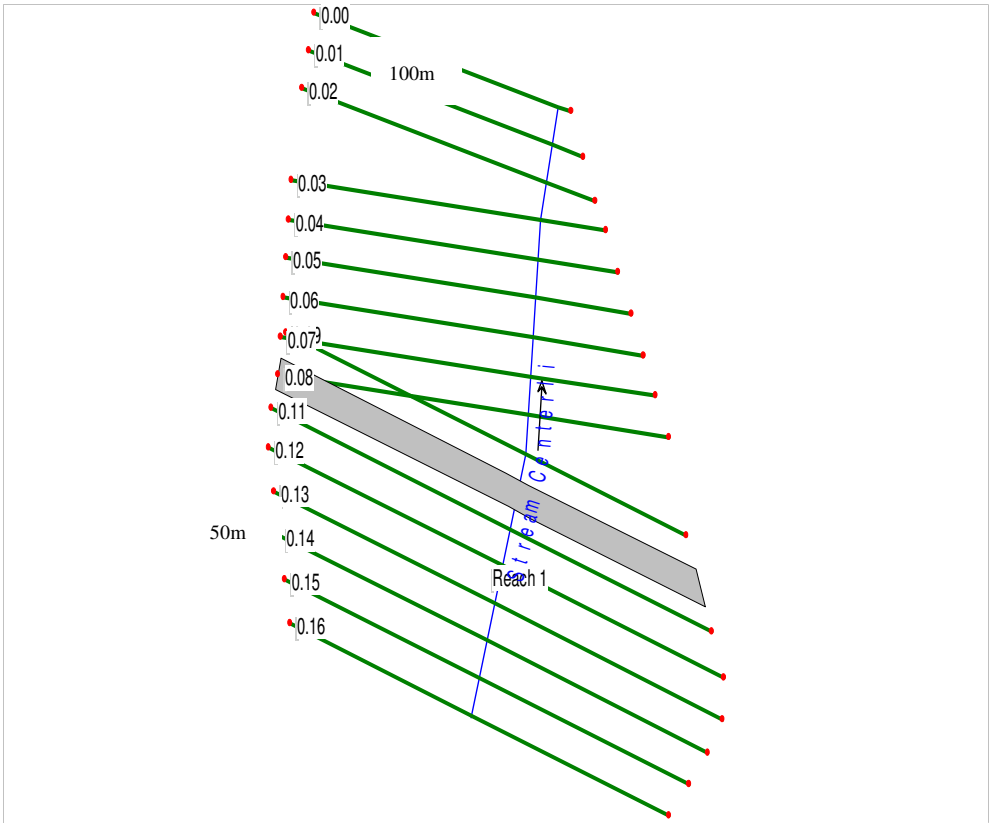


Figure 8A-2 Layout of X-Sections for Hydraulic Analysis of Bridge

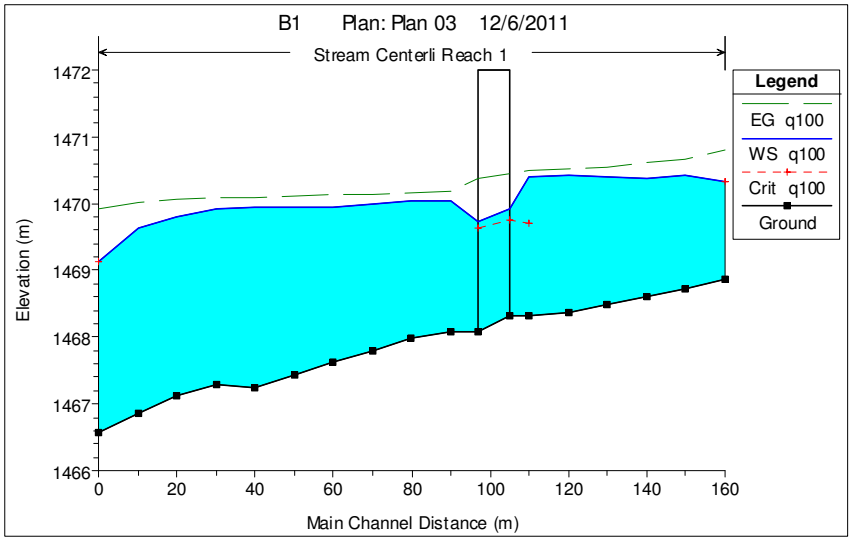


Figure 8A-3 Longitudinal Profiles Along the River

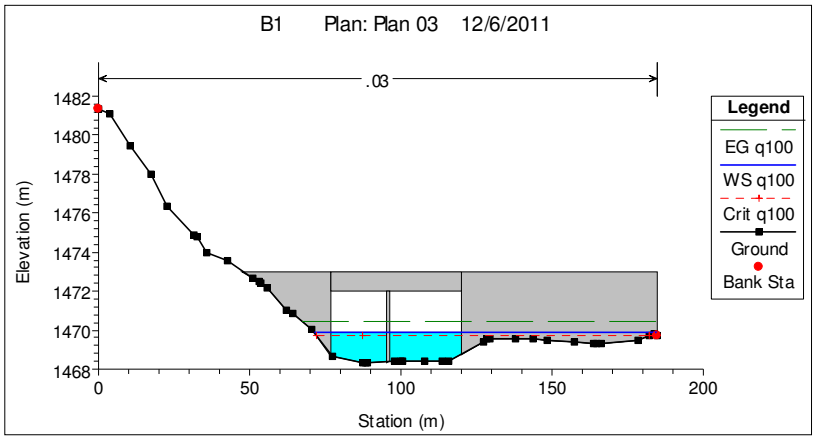


Figure 8A-4 Longitudinal X-Section of the River and Bridge

Hydraulic Calculations for the Bridge

The hydraulic calculations have been carried out by applying the HEC-RAS computer program to determine the appropriate size of the bridge openings sufficient to convey the estimated 100-year return period flood of the river. During computation, the following conditions were considered:

- Flow: Steady flow and normal depth (slope =0.02) reach boundary conditions for Q50 and Q100. Flow type: Sub-critical;
- Bridge modelling approach: Computed by Energy and Yarnell Class A and used the highest energy.

The hydraulic calculation was carried out to determine the size of the bridge to convey the design flood of (Q50 = 172.9m3/s, Q100 = -201.9m3/s).

The modelling result is presented below:

Calculated flood level for 100 year design flood = 1470.49 mm AOD:

Calculated scour depth for the pier:

Left abutment	= 1.2 m
Right abutment	= 1.2 m
Left pier	= 1.56 m
Right pier	= 1.56 m

Worked Example 8-3 – Scour at a Bridge

Problem description Example 8A-3

Consideration is being given to construction a bridge across the Sand River, which is some 730m wide at the proposed bridge site. The Potential scour at the bridge should be determined. This problem was also evaluated with HEC-RAS.

Figure 8A.2 shows a plan view (obtained from the HEC-RAS problem evaluation) and the position of the bridge relative to the other cross-sections. The cross-sectional information for all the sections is available. The bridge will be positioned at cross-section 6.5 (downstream from cross-section 7 and upstream of cross-section 6).

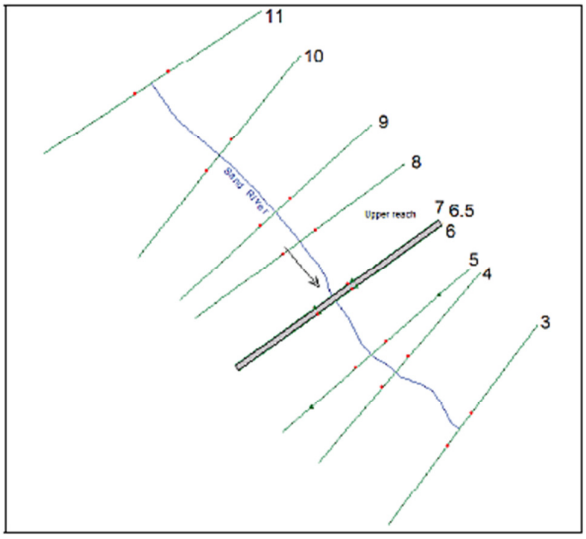


Figure 8A-2: General layout of the Cross-Sections and the Position of the Bridge

The bridge data is described below and the bridge cross-section is shown in Figure 8A-2. The bridge opening between the sloping abutments is approximately 126.61m wide and the bridge is supported by five piers, each with a width of 1.5m (equally spaced). The high (road surface) and low cord (bridge soffit) values for the bridge deck on the upstream side are 6.7 and 5.5m respectively.

The design flow rate for which the scour analyses have to be conducted is the 1:100 year flood (Q_{100}), which has been determined to be 850m^3/s.

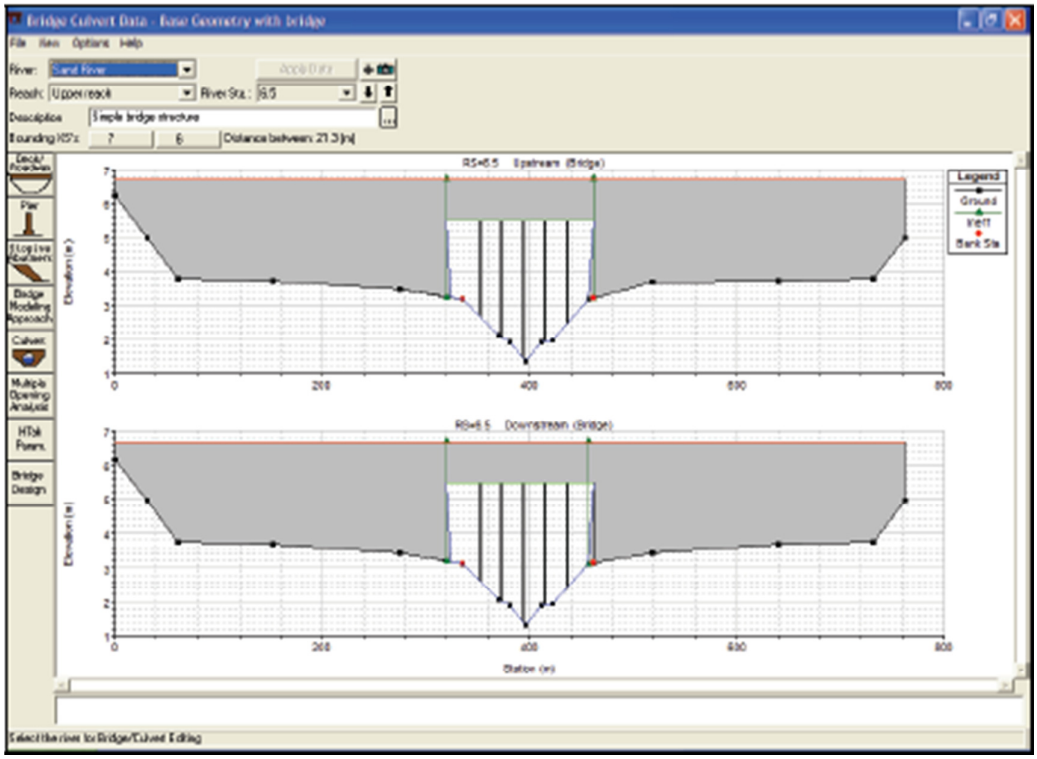


Figure 8A-3 Upstream and Downstream Bridge Cross-Section from HEC-RAS Analysis

The flow in the river is downstream control and the normal flow depth, y_n, could be calculated at the bridge, assuming a representative slope of 0.002m/m.

Bed material characteristics

The sieve analyses (percentage passing) of the bed material revealed the following:

D_{50} = 0.0020 m
D_{90} = 0.0045 m

Cross sectional details

The cross-section details are given in the Table 8A-2 below. These details can be obtained form analysis of the surveyed cross-section information, using software such as HEC-RAS, or computing the variables by hand.

Slope of the river

The general slop of the river is 0.2%.

Determine

(i) Short-term general scour
(ii) Contraction scour
(iii) Local scour at the piers and abutments
(iv) Total scour
(v) Verify the scour depth with the method based on the principle of applied stream power.

Solution Example 8.2

For the analysis the design flood discharge of 850 m^3/s will be used (chapter 5 describes procedures to determine the design flood).

The contracted width at the bridge will be 126.16 m. this will result in a discharge per unit width of 850/126.61 = 6.713 m^3/s.

The normal flow depth (fixed bed), y_n, of the river can be determined by the assumption of the energy slope to be equal to the bed slope 0.002 m/m and by using the Chezy or Manning equations.

It is estimated that the bed roughness under flood conditions will be 0.002m, equal to D50 the representative sediment material size.

Table 8A-2: Details of Cross-Section 6.5 (Obtained form HEC-RAS Analysis)

Section	Y_n (m)	Area (m^2)	Wetted Perimeter (m)	Flow rate (m^3/s)
Left bank	2.98	209.97	288.17	168.75
Main Channel		258.73	126.67	542.60
Right Bank		185.40	283.48	138.65
Total		**654.10**	**698.32**	**850.00**

R = 0.937m and V = 1.299 m/s

Top flow width = 698.2 m for the calculated normal flow depth of 2.98 m

It is assumed that the bed material consists of deep alluvial sand with no cohesion.

(i) Short-term general scour

The regime equations are applied to establish equilibrium conditions at the design flow;

Form Equation 8.6:

$$B = 14Q^{0.5}D_{50}^{0.25}F_s^{-0.5}$$

With F_s = 0.1 form Table 8.2 for salty clay loam, the width B can be calculated.

B = 273 m, which is wider than the proposed bridge of 126.61 m.

Use Equation 8.5 to determine the mean flow depth at the equilibrium width:

$$y = 0.38q^{0.67}D_{50}^{-0.17}$$

q = 850/273 = 3.114 m^3/s. m

Mean depth y = 2.34 m. The maximum depth, Y_{max} = 1.25y = 2.92 m.

The maximum live – bed depth, Y_{max}, is slightly less than the fixed bed depth, y_n of 2.98 m, which reflects that no general short term scour will occur.

(ii) Contraction scour

It has been indicated that contraction scour can be determined by applying either the regime equations (Equations 8.5 and 8.7) or the contraction equations (Equation 8.11 and 8.13 with 8.12).

First apply the ***regime equation*** of the reduced width of 126.16 m, in this case q= 850/126.61 = 6.714 m^3/s.m leading to a mean flow depth, y of 3.915 m. From Table 8.4, Y_{max}, can be determined as follows: Y_{max}, = 1.25 x 3.915 = 4.893 m. This reflects a scour depth, d_s = 4.893 – 2.98 = 1.913 m

Secondly the contraction equations are used to determine the scour depth after it has been established if the flow will be sediment laden or not.

V* can be determined using Equation 8.2, $V^* = \sqrt{gDS}^* = \sqrt{9.81 * 2.98 * 0.002} = 0.242$ m/s and the term,

$V^*D_{50}/v = 483 >> 13$, thus in turbulent flow region (see figure 8.10)

The critical shear velocity $V^*_c = 0.12 \times V_{ss}$ (Equation 8.1) The settling velocity, V_{SS} can be obtained from Figure 8.11 for the representative particle, D_{50}, and the relative density of 2.65 it flows:

V_{ss} = 0.24 m/s, and

V^*_C= 0.029 m/s

From Equation 8.10: $V_c = 5.75\, V^*_C log\frac{12R}{Ks} = 5.75\, [(0.029)\log 12\frac{(0.937)}{(0.002)}$

V_c = 0.625 m/s

The average approach flow velocity of 1.299 m/s > 'critical' velocity of 0.625 m/s, thus sediment will be entrained and Equation 8.11 together with Figure 8.12 can be used to estimate contraction scour.

$$\frac{Y2}{Y1} = (\frac{Qt}{Qc})^{\frac{6}{7}}(\frac{B1}{B2})^{\frac{2}{3}}(\frac{n2}{n1})^{\frac{1}{3}}$$

$\frac{Y2}{Y1} = \left(\frac{850}{542.6}\right)^{\frac{6}{7}}$ = 1.469 (widths and n-values are equal for these sections)

Y2 = (1.469) (2.98) = 4.378 m

Assuming a level bed with a total depth of 4.378 m, the velocity in the contraction can be determined

$$V2 = \frac{850}{(4.378)(126.61-5(1.5))} = 1.63\text{m/s}$$

Note that in this case the downstream area is 521.5 m^2, calculated as follows (4.378x (126.61-5(1.5))). This is larger than the upstream main channel are of 258.78 m^2 (Table 8.16), and thus the flow is expanding. Equation 8.12 is used to determine the contraction scour depth.

$d_s = (Y2 - Y1) + (1 + K)\left(\frac{V_2^2 - V_1^2}{2g}\right)$ and with K = 1 for a sudden transition

$$d_s = (4.378 - 2.98) + (1 + 1.0)\left(\frac{1.63^2 - 1.23^2}{2(9.81)}\right)$$

d_s = 1.50 m

This scour depth (1.50 m) is less than that obtained with the regime theory (1.913 m).

(iii) Local Scour at Piers and abutments

For piers in alluvial cohesion less materials:

Use Equation 8.9 and 8.10 to compute local scour in two different ways. Obtain the factors needed for Equation 8.17 form Tables 8.3 and 8.4. Obtain the factors needed for Equation 8.15 from Tables 8.3, 8.4 and 8.5 and Equations 8.17, 8.18, 8.19 and 8.20. Compare answers obtained from Equations 8.13 and 8.15 and Select a conservative answer using good engineering judgment.

From equation 8.17, with depth y_o in the bridge section as determined from regime equation 8.8:

$d_s = 1.8 y_o^{0.75}\, b^{0.25} - y_o$

$d_s = 1.8\ (3.915^{0.75})(1.5^{0.25}) - 3.915$

d_s = 1.629 m

Note that the scour level is (3.915 + 1.629) = **5.544 m below the design flood level.**

Alternatively Equation 8.15 could be used to calculate the local scour depth at the pier for the longest piers close to the minimum river invert.

$$\frac{Y_s}{b} = 2.0 K_1 K_2 K_3 K_4 \frac{(y_1)^{0.35}}{b} Fr_1^{0.43}$$

With

b = 1.5 m
Y_1 = 2.98 m (normal flow depth upstream of the bridge, Table 8.16)
Fr_1 = 0.468 based on main channel data directly upstream of he pier
K_1 = 1.0 for zero skew angle
K_2 = 1.0 for zero skew angle
K_3 = 1.1 for small dunes

K_4 = 1.0 for uniform sediment (no armouring), then

$\frac{Y_s}{b}$ = (2.0)(1.0)(1.1)(1.0)[2.98]$^{0.35}$ (0.468)$^{0.43}$ = 2.02

Y_s = 3.03m

Note that the scour level is (2.98 +3.03) = 6.01m below the design flood level associated with the normal flow depth and a fixed bed level.

For abutments in alluvial cohesion less materials:

Apply factors in Table 8.10 to the general short-term average scour depth obtained from Equation 8.8.

Form Table 8.10 the factor for flow that impinges at right angles on bank = 2.25; hence the scour at the abutments can be determined as shown below:

d_s (abutments) = (2.25)(3.915 – 2.98) = 2.10m

(iv) Total Scour

Total scour is the sum of the long and short-term general scour, contraction scour and local scour. Table 8A-3 reflects a summary of all the calculated scour depths.

Table 8A-3: Summary of Calculated Scour Depths

Scour Type		Calculated Scour Depth, d_s (m)
Short term general scour		No scour
Contraction scour	Regime equation	1,913
	Contraction Equation	1,398
Local scour	Piers	3,03
	Abutments	2,10
Total expected scour	**Piers**	**4,943**
	Abutments	**4,013**

Review of the contraction or short-term scour using different analyses procedures

The potential general scour at the bridge has been determined in (i) using Equations 8.5 and 8.9. A more correct approach is to estimate contraction scour separately for the main channel and over banks, as is done in HEC-RAS, where the over bank flows may then reflect clear water scour. The scour depth in the channel calculated in the approach used in HEC-RAS is less than 3.1m. With the regime theory reflecting a scour depth of 1.91m and the HEC-RAS result of 3.1m, the contraction scour calculation of 1.5m using Equation 8.174 is too conservative and hence is discarded.

Based on the summary in Table 8.17 the total scour can be determined as follows.

Total scour level at the piers in the main channel below the design flood level (not accounting for backwater) = 1.913 + 3.03 = 4.943m

Total scour at abutments

With the right abutment at the edge of the main channel, the scour would be the sum of main channel contraction scour plus abutment scour, thus:

Total scour at abutments = 1.913 + 2.1 = 4.013m below design flood level

The scour for the left bank abutment would be less.

(v) Verify the scour depth with the method based on the principle of applied stream power (section 8.5.4)

For total scour at piers in alluvial rivers, check the answer against values obtained by means of Equations 8.21, based on the principle of applied stream power.

Equation 8.21 reflects the flowing relationship:

$$\frac{C(Y_1)(v_{ss}k_s)^{1/2}}{q\sqrt{g}} = F$$

with F = 0,8; k_s = 0.002m; v_{ss} = 0.24m/s; q = 542.6/126.61 = 4.29m^3/s.m and C calculated from the Chezy relationship for total section, C = 67.5; it follows that:

Y_1 = 2.03m below design flood level, which is **substantially less than obtained above.**

The designer will experience these conflicting results, which reflect the complexities involved in the mathematical description of scour estimates and shortcomings in the assumption that the material is cohesionless.

Considering the risk of failure of the structure due to scour and the potential consequences, these cases require further evaluation by experienced persons.

9 ENERGY DISSIPATERS

9.1 Introduction

Erosive forces which can be at work in the natural drainage network are often increased by the construction of a highway. Interception and concentration of overland flow and constriction of natural waterways inevitably results in increase erosion potential. In fact, the failure of many highway culverts can be traced to unchecked erosion. To protect the highway and adjacent areas, it is sometimes necessary to employ an energy dissipating device.

Throughout the process of selecting and designing an energy dissipater, the designer should keep in mind that the primary objective is to protect the highway structure and adjacent area from excessive damage due to erosion. An effective design will return the flow downstream of the dissipator to a condition which approximates the natural flow regime.

Energy dissipators may be used at a number of locations within a highway drainage system, including outfalls for culverts, storm sewers, detention ponds and steep ditches.

However, the predominant use of energy dissipating structures will be at culvert outfalls. Thus, this chapter concentrates on the use of energy dissipators for culverts. The designer should be able to easily adapt the methods provided in this chapter to the design of dissipators for other drainage features.

Before specifying an energy dissipator for a culvert site, the designer may wish to investigate modifying the vertical alignment of the culvert to reduce the outlet velocity. The choice between modifying the culvert alignment or providing an energy dissipator would normally be based on a site-specific consideration of the costs for construction and maintenance presented by each option. The designer should also be aware of the discussion of riprap aprons presented in this chapter. There may be situations where a riprap apron may be used in lieu of an energy dissipator, and the designer should be familiar with the criteria for the application of both options.

Although energy dissipators cover a wide range in complexity and cost, they can be grouped into two broad categories. One type of energy dissipator acts by forcing a hydraulic jump in the flow stream leaving the culvert. This is accomplished either by increasing the hydraulic roughness of a segment of the culvert or by directing flows into a basin located at the culvert outfall. The second group of energy dissipators is often referred to as impact basins, even though they are constructed at the stream bed level. Energy is dissipated in these basins as the concentrated flow jet from the culvert outlet impacts on blocks or baffles located on the basin floor. The selection of a particular type of dissipator should be based on consideration of the velocity of the culvert outflow, an assessment of the erosion hazard, and the amount of right-of-way available. The following photographs illustrate some of the culvert outlet energy dissipaters used in Ethiopia. The example photos show that the longitudinal and lateral extent of dissipaters is inadequate and there is a widespread failure of outlet protection devices.



Photo 9-1: Scoured Outlet at Adet Quarit Road, Gojam



Photo 9-2: Culvert Outlet Energy Dissipater, Gondar



Photo 9-3: Drop Inlet Structure, Gojam



Photo 9-4: Stilling Basin Energy Dissipater, Tigray



Photo 9-5: Channel Energy Dissipater, Tigray



Photo 9-6: Chute Energy Dissipater, Tigray



Photo 9-7: Damaged Energy Dissipater at the Outlet of a Culvert, Dire Dawa

This chapter provides design procedures based on Ref. 1 (FHWA-IP-89-016, 1989) and analysis results using Refs. 2 (FHWA-EPD-87-101, 1987) and 3 (FHWA-EPD-86-110, 1983).

9.1.1 Symbols

Table 9-1: Symbols, Definitions and Units

Symbol	Definition	Units
A	Cross sectional area	m^2
A_o	Area of flow at culvert outlet	m^2
d_E	Equivalent depth at brink	m
d_o	Normal flow depth at brink	m
D	Height of culvert	mm
D_{50}	Mean diameter of riprap	mm
DI	Discharge Intensity Modified	--
Fr	Froude Number	--
h_s	Depth of dissipater pool	m
L	Length of culvert	m
L_B	Overall length of basin	m
Ls	Length of dissipater pool	m
Q	Rate of discharge	m^3/s
S_o	Slope of streambed	m/m
TW	Tailwater depth	m
V_d	Velocity downstream	m/s
V_L	Velocity - (L) meters from brink	m/s
V_o	Normal velocity at brink	m/s
W_o	Diameter or width of culvert	mm
W_s	Width of scour hole	m

9.2 Design Criteria

Design criteria are the standards by which a policy is carried out or placed into action. They form the basis for the selection of the final design configuration. Listed below by categories are the design criteria that shall be considered for all energy dissipater designs.

9.2.1 Site Criteria

The dissipater type selected for a site must be appropriate to the location. In this chapter, the terms internal and external are used to indicate the location of the dissipater relationship to the culvert. An external dissipater is located outside of the culvert and an internal dissipater is located within the culvert barrel.

9.2.2 Dissipater Type Selection

Internal Energy Dissipaters

In situations where there is limited right-of-way for an energy dissipator at a culvert outlet and where the culvert barrel is not used to capacity due to inlet control, metal or concrete roughness elements may be placed along a section of the downstream end of the culvert to control outlet velocities. These roughness elements are referred to as internal energy dissipaters. Because the culvert is flowing partially full with inlet control, it is possible to increase the depth of flow near the culvert outlet without creating additional headwater. Because internal energy dissipators may require regular maintenance, they should be used only in box culverts of sufficient size to allow for entry by maintenance personnel. They may be used where:

- The scour hole at the culvert outlet is unacceptable;
- The right-of way is limited;
- Debris is not a problem; and
- Moderate velocity reduction is needed.



Figure 9-1: Roughness Elements Inside of a Box Culvert

(Source: TDOT Drainage Manual, 2008)

Roughness elements decrease flow velocities by either increasing the flow resistance of the culvert barrel or by a phenomenon known as tumbling flow.

Tumbling flow (Figure 9-2) is an excellent energy dissipater on steep slopes. It is essentially a series of hydraulic jumps and overfalls that maintain the flow approximately

at critical velocity on slopes that would otherwise be characterized by high supercritical velocities.

Use of tumbling flow is reasonable for slopes up to 10 or 15 percent. One of the major limitations of tumbling flow as an energy dissipator is that the required height of the roughness elements is closely related to the unit discharge (discharge divided by the width of the culvert).

There may be situations where the element height would have to be half the culvert height in order to maintain tumbling flow. Thus, practical applications of tumbling flow are likely to be limited to low-discharge per unit width (i.e. shallow flow), high-velocity culverts.

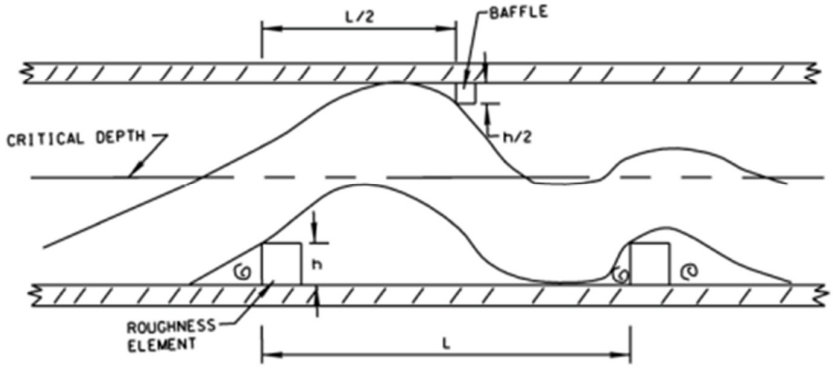


Figure 9-2: Typical Tumbling Flow Energy Dissipater

Tumbling flow can be established rather quickly by using either a very large leading element, or a smaller leading element and a baffle to reverse the flow jet between the first and second rows. The first alternative is not considered to be a practical solution since the element size is likely to be excessive. The baffle has merit since it deflects the so-called "rooster tail" jet back towards the culvert bottom and brings the flow under control very quickly without using a large leading roughness element.

Increased resistance (Figure 9-3) involves using roughness elements to provide greater hydraulic roughness and thus, reduce velocity. Because increasing resistance will also increase the depth of flow, the designer should ensure that the proposed culvert height will be adequate in the roughened section.

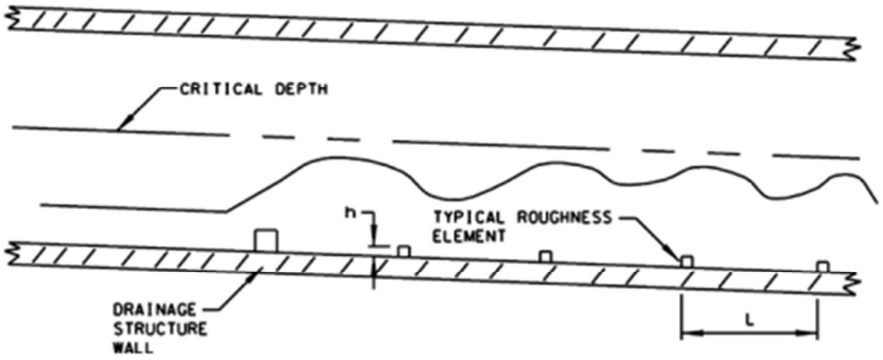


Figure 9-3: Increased Hydraulic Roughness

Whether roughness elements will represent increased resistance or create tumbling flow is largely dependent on the culvert slope. A roughness element on a steep slope may induce tumbling flow, whereas the same roughness element on a relatively flatter slope would represent increased resistance. Further, tumbling flow essentially delivers the outlet flow at critical velocity while increased resistance will deliver outlet velocities which are still in the supercritical flow regime. The designer should carefully evaluate the depth and velocity of the flows leaving the culvert and provide for any additional required erosion protection in the channel. Although internal energy dissipaters may not completely eliminate the need for some form of erosion control at a culvert outlet, they may provide sufficient reduction in outlet velocity or Froude Number to allow a simpler, less expensive form of protection at the outlet.

Although internal energy dissipaters will tolerate a moderate quantity of sand and silt, they should not be used in situations where the stream will transport cobbles and boulders or significant amounts of floating debris. These structures do not require a tailwater to operate efficiently.

External Energy Dissipaters

External energy dissipaters are concrete structures placed at the culvert outfall as either stilling basins or impact basins. External dissipaters are used where:

- The outlet scour hole is not acceptable;
- Moderate amount of debris is present; and
- The culvert outlet velocity (Vo) is moderate, Fr < 3.

USBR Type VI Impact Basin

The USBR Type VI basin, as shown in Photos 9-8 and 9-9, is an impact-type energy dissipator which is contained in a relatively small box-like structure. Inside this box is a vertical baffle which is referred to as a hanging baffle because an opening is provided between the bottom of the baffle and the floor of the box. This type of energy dissipater is attached directly to the culvert outlet in place of a standard end wall.

Energy dissipation is initiated as flow strikes the vertical baffle and is deflected upstream by the horizontal portion of the baffle and by the floor, creating horizontal eddies. Despite its relatively small size, this impact basin yields greater energy dissipation than a hydraulic jump in the same setting.

The baffle is provided with notches which aid in cleaning the basin after prolonged non-use of the structure. If the basin should begin to collect sediment, the notches will provide concentrated jets of water for cleaning. The basin is designed so that the full design discharge can be passed over the top of the baffle should the space beneath it become completely clogged. Although this degrades the performance of the structure, it is acceptable for short periods of time. To provide structural support and aid in priming the device, a short support should be placed under the center of the baffle wall.



Photo 9-8: Typical USBR Type VI Baffled Dissipater

In situations where the culvert entering the basin has a slope greater than 27 percent, the basin should be constructed on a horizontal grade. In addition, the culvert should be provided with a horizontal section at least four culvert widths in length immediately upstream of the dissipater. Although the basin will operate effectively with entrance pipes on slopes up to 27 percent, experience has shown that it is more efficient when the flow jet entering the dissipater is horizontal.

The end of the basin should be provided with a low sill which, where feasible, should be set at the same elevation as the downstream channel. Where this is not possible, a slot should be placed in the end sill to provide for drainage during periods of low flow.

Where needed to retain the roadway embankment, the end of the basin may be provided with an alternate end sill and 45° wing walls as shown in Photo 9-8. It may also be necessary to provide a cut-off wall. Where the velocities of flows exiting the basin exceed 1.524 m/sec, the channel downstream of the basin should be provided with a riprap apron.

To prevent cavitation damage, use of the USBR Type VI basin is limited to installations where the discharge is less than 121.92 m^3/s. Although tailwater is not necessary for the successful operation of the basin, a moderate depth of tailwater will improve its performance. However, the tailwater depth should not be above half of the height of the baffle, or h 3 + h 2 /2, as shown in Photo 9-8. This dissipater is not recommended where potential debris may cause substantial clogging.

Hook Type Impact Basin Energy Dissipater

The hook energy dissipater is a type of impact basin that abates culvert outflow velocities by means of three hook-shaped blocks and an end sill in a uniform trapezoidal channel. The general layout of a hook energy dissipater is shown in Photo m9-9 and a detail of the hook-shaped block is provided in Photo 9-10. Although this type of dissipater was originally developed primarily for large arch culverts, it is also effective for box or circular culverts as shown in Photo 9-7.

Ideally, the width and shape of the uniform trapezoidal channel should generally resemble the natural channel cross section. However, for a given discharge condition, widening the basin and flattening the side slopes will tend to improve the performance of the basin. In practice, the side slopes of the basin should be between 1.5H: 1V and 2H: 1V and the bottom width of the basin should be 1 to 2 times the effective opening width of the culvert.



Photo 9-9: Hook Type Energy Dissipater Basin

Depending on the final exit velocity and local soil conditions, some scour may occur downstream of the basin. Where this is possible, a riprap apron should be provided downstream of the basin. In addition, the end of the basin should be provided with a cutoff wall.

The hook energy dissipater should not be used where large amounts of debris are expected. Coarse sediments may abrade the upstream face of the hooks, while floating debris may catch on them, causing the basin to become choked. These basins may be used where the Froude number of the culvert outflow is between 1.8 and 3.0.

Rip Rap Stilling Basin

Riprap stilling basins are similar to natural scour holes in that their design procedure is based on laboratory studies of the relationships between culvert flow properties and the dimensions of the scour holes that would form in riprap at the outfalls. These energy dissipaters consist of a pool at the culvert outfall, followed by an apron that rises to the channel flow line, and a transition to the natural channel cross section. Concentrated flow at the culvert outfall will plunge into one end of the pool and then form a hydraulic jump at the other end, against the apron. As a result, the flow will generally be well dispersed as it leaves the basin. In some situations, the design method provided in HEC-14 will require that the basin at the culvert outfall will be lined with a heavier class of Machined Riprap. Where this occurs, the apron and transition to the natural valley configuration may be constructed of a smaller class of riprap.

Riprap basins may be used where allowing a natural scour hole to form will not be acceptable. They will be effective when the Froude Number of the culvert outflow is less than 3.

Although a riprap basin may be used where the tail water depth is high, it is recommended that they be used only where the tail water depth is less than 75% of the depth of flow at the culvert outfall. They are not affected by heavy debris loads.

For any site where a riprap basin is feasible, the designer should also check the design for a riprap apron and select the structure type based on cost. Where the tailwater depth is sufficiently low, a riprap stilling basin can be shorter than a riprap apron. Thus, a basin can help to reduce the costs associated with obtaining a permanent drainage easement and environmental permits. In addition, a basin designed in accordance with HEC-14 can often be constructed using Machined Riprap instead of the heavier classes which would be required for an apron in the same setting. On the other hand, riprap aprons are more easily

constructed and can be applied in a wider variety of situations. Typically, the standard cut-off wall depth for the culvert end treatment will be sufficient where a riprap stilling basin is employed.

Stilling Basins are used where:

- The outlet scour hole is not acceptable;
- Debris is present; and
- The culvert outlet velocity (Vo) is high, Fr >3
- Natural Scour holes

This option consists of providing an area in which flows through the culvert will be allowed to form a natural scour hole. The designer should carefully estimate the size of the scour hole and line the channel and the overbanks over an area sufficient to cover the potential scour hole with the class of riprap appropriate for the culvert exit velocity . In general, the size of the scour hole should be estimated assuming cohesion less material to provide a conservative estimate. The designer may, based on sound engineering judgment, choose to assume cohesive soils for this estimate and use the methodology provided in HEC-14. Natural scour holes are usually used where:

- Undermining of the culvert outlet will not occur or it is practicable to be checked by a cutoff wall;
- The expected scour hole will not cause costly property damage; and
- There is no nuisance effect.

Natural scour holes are often unsuitable in Ethiopia due to undermining. Internal dissipaters have limited application and are more useful for long culverts such as would be found under multi-lane highways. Stilling basins are expensive and shall be considered only in extreme circumstances.



Photo 9-10: Scour Hole at a Culvert Outlet with Deposited Material in the Foreground

Debris Control

Debris control if necessary (see Ref. 4, Hec-9) shall be considered:

- Where clean out access is limited; and
- If the dissipater type selected cannot pass debris.

9.2.3 Design Limitations

Flood Frequency

The flood frequency used shall be in accordance with Chapter 2, Table 2-1. A higher frequency may be used, only if justified by all of the following:

- Low risk of failure of the crossing;
- Substantial cost savings;
- Limited or no adverse effect on the downstream channel; and
- Limited or no adverse effect on downstream development.

Maximum Culvert Exit Velocity

The culvert exit velocity shall be consistent with the maximum velocity in the natural channel or shall be mitigated by using:

- Channel stabilization, see Chapter 6: Channels, and
- Energy dissipation.

Tail water Relationship

The hydraulic conditions downstream shall be evaluated to determine a tailwater depth and the maximum velocity for a range of discharges.

Open channels (See Chapter 6)

Lakes, ponds, or large water bodies shall be evaluated using the high water elevation that has the same frequency as the design flood for the culvert if events are known to occur concurrently (statistically dependent). If statistically independent, evaluate the joint probability of flood magnitudes and use a likely combination.

9.2.4 Design Options

Material Selection

The material selected for the dissipater shall be based on a comparison of the total cost over the design life of alternate materials and should not be made using first cost as the only criteria. This comparison should consider replacement cost and the difficulty of construction as well as traffic delay.

Culvert Outlet Type

In choosing a dissipater, the selected culvert end treatment has the following implications:

- Culvert ends that are projecting or mitered to the fill slope offer no outlet protection;
- Headwalls provide embankment stability and erosion protection. They provide protection from buoyancy and reduce damage to the culvert;
- Commercial end sections add little cost to the culvert and may require less maintenance, retard embankment erosion, and incur less damage from maintenance;
- Aprons do not reduce outlet velocity, but if used should extend at least one culvert height downstream. They should not protrude above the normal streambed elevation;
- Wingwalls are used where the side slopes of the channel are unstable, where the culvert is skewed to the normal channel flow, to redirect outlet velocity, or to retain fill.

Safety Considerations

Traffic shall be protected from external energy dissipaters by locating them outside the appropriate "clear zone" distance as per the ERA Geometric Design Manual or shielding them with a traffic barrier.

Weep Holes

If weep holes are used to relieve uplift pressure, they shall be desiigned in a manner similar to underdrain systems.

9.2.5 Related Designs

Culvert

The culvert shall be designed independent of the dissipater design (see Chapter 7) with the exception of internal dissipaters that may require an iterative solution. The culvert design shall be completed before the outlet protection is designed and must include computation of outlet velocity.

Downstream Channel

The downstream channel protection shall be designed concurrently with dissipater design (see Chapter 6).

9.2.6 Alternative Analysis

Choosing alternatives which satisfy:

- Topography; and
- Design policies and criteria.

Analyzing alternatives for:

- Environmental impact;
- Hydraulic efficiency; and
- Risk and cost.

Selecting an alternative which best integrates engineering, economic, environmental and political considerations. The chosen dissipater should meet the selected structural and hydraulic criteria and shall be based on:

- Construction and maintenance costs,
- Risk of failure or property damage,
- Traffic safety,
- Environmental or aesthetic considerations,
- Nuisance considerations, and
- Land use requirements.

9.3 Design Procedures

9.3.1 Computations in Support of Energy Dissipations

A variety of background information is usually needed to properly design an energy dissipater. A portion of that information may be determined by means of computations that support the dissipater design. This section discusses the computations that should be completed prior to beginning the selection and design of an energy dissipation scheme.

9.3.2 Culvert Hydraulic Analysis

A significant portion of the data needed to design an energy dissipater will be obtained from the culvert design file. Detailed procedures for the design of a new culvert are described in Chapter 7.

9.3.3 Computation of Culvert out Flow Conditions

Although the culvert outflow conditions will likely be described in the culvert design chapter, a few parameters will require a more detailed analysis to support the energy dissipater design:

1. ***Outlet Depth (d_o):*** The outlet depth is often provided as a part of the hydraulic analysis of the culvert. Where this is not the case, the outlet depth may be determined using the guidance provided in Chapter 7 of this Manual.
2. ***Area (A_o):*** The cross sectional area of the flow at the culvert outlet should be determined using d_o.
3. ***Top width (T):*** The top width of the flow at the culvert outlet may be determined using d_o.
4. ***Velocity (V_o):*** The culvert outlet velocity should be calculated as follows:

$$V_o = \frac{Q}{A_o} \quad (9\text{-}1)$$

Where: Q = culvert discharge, (m^3/s)

5. ***Equivalent Depth (d_e)***: The equivalent depth is used in a number of computations for non-rectangular culverts. It can be computed as follows:

$$d_e = \left(\frac{A_o}{2}\right)^{0.5} \quad (9\text{-}2)$$

6. ***Froude Number (Fr)***: This parameter is described in Section 9.2 and is an important factor in the design of energy dissipaters. For rectangular shapes, it is calculated as follows:

$$Fr = \frac{V_o}{(g * d_o)^{0.5}} \quad (9\text{-}3)$$

Where: g = acceleration due to gravity, (9.81 m/sec^2)

d_o = depth of flow at outlet, (m)

V_o = culvert outlet velocity, (m/s)

For non-rectangular shapes, the term d_o may be substituted with the equivalent depth, d_e.

9.3.4 Scour Hole Estimation

The estimate of the scour hole size is an essential part of the energy dissipater design procedure. Together with the maintenance history and site reconnaissance information, this estimate can serve to assist in determining an appropriate energy dissipation design.

This section presents a procedure for estimating scour holes in cohesion less materials for the maximum or extreme scour case. The designer may refer to HEC-14, Chapter 5 for detailed information on estimating scour holes in cohesive soils. HEC-14 recommends that soil testing be done at a site where cohesive soils are present to determine the plasticity index and saturated shear strength, which are necessary for the HEC-14 procedure. However, unless the dimensions of the scour hole are critical to the overall design,

sufficient accuracy may be obtained by looking up average values of these parameters in a soil mechanics textbook for the general soil classification at a site.

Results of tests by the U.S. Army Waterways Experiment Station in Vicksburg, Mississippi indicate that scour hole geometry varies with the tailwater conditions. The maximum scour geometry occurs for tailwater depths less than half the culvert height. As shown in Figure 9-4, the maximum depth of scour, d_s, occurs at a location equal to approximately 40% of the length of the scour hole, L_s, measured downstream from the culvert.

Equation 9-4 is an empirical equation that may be used to compute the three dimensions (length, width and depth) of the scour hole. The equation is applied using three coefficients termed α, β, and θ. The value of these coefficients will vary depending on which dimension of the scour hole is being computed. Thus, to compute all three dimensions of the scour hole, the equation would be applied three times. Each time, a different set of values are assigned to α, β, and θ as determined from Table 9-2.

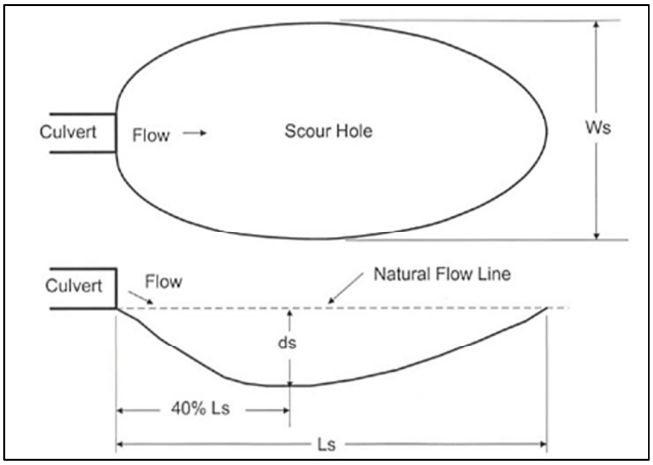


Figure 9-4: Scour Hole at Culvert Outlet

The dimensions of the scour hole will be affected by the slope of the culvert and whether or not a drop exists between the culvert invert and the channel bed. Therefore, two adjustment factors, C_s and C_h, are included in Equation 9-4. C_s is used to account for the slope of the culvert. Values for this factor may be obtained or interpolated from Table 9-3 based on the culvert slope in percent. Values for C_h may be obtained or interpolated from Table 9-4 using the drop height, H_d, expressed in culvert diameters. To use the table, the distance from the culvert invert to the channel bed should be determined and then divided by the culvert diameter to determine H_d. If the culvert is non-circular, the rise of the structure should be used to compute the drop height.

The dimensions of a scour hole will also be affected by the length of time over which flows will occur at the site. The term F_3 in Equation 9-7 is used to account for the duration of the peak flow at the culvert site as compared to the time base of 316 minutes used in the tests by the U.S. Army Waterways Experiment Station. The duration of the peak flow may be estimated if a stream flow hydrograph is available. Lacking this information, it is recommended that a time of 30 minutes be used. It has been found that approximately ⅔ to ¾ of the maximum scour will occur in the first 30 minutes of the flow duration.

$$d_s, w_s, L_s = C_s C_h F_1 F_2 F_3 R_0 \quad (9\text{-}4)$$

and,

$$F_1 = \frac{a}{\sigma^{1/3}} \quad (9\text{-}5)$$

$$F_2 = \left(\frac{Q}{g^{0.5} {R_o}^{2.5}}\right)^{\beta} \quad (9\text{-}6)$$

$$F_3 = \left(\frac{t}{316}\right)^{\theta} \quad (9\text{-}7)$$

Where: d_s = maximum depth of the scour hole, (m)
L_s = length of the scour hole, (m)
w_s = width of the scour hole, (m)
Q = design discharge, (m^3/s)
g = acceleration due to gravity, (9.81 m/sec^2)
t = duration of the peak flow, (minutes), Use 30 minutes if unknown
R_o = hydraulic radius of the cross-sectional flow, (m)
σ = material standard deviation (see following discussion)
α, β, θ, C_s and C_h are coefficients, as shown in Tables 9-1 through 9-3.

The material standard deviation, σ, is a measure of the grain size distribution of the bed material in the channel. When a sieve analysis is available from a geotechnical investigation, the standard deviation may be computed as:

$$\sigma = \left(\frac{d_{84}}{d_{16}}\right)^{0.5} \quad (9\text{-}8)$$

Where:

d_{84} = mean particle diameter at the 84^{th} percentile of the distribution
d_{16} = mean particle diameter at the 16^{th} percentile of the distribution

When a sieve analysis is not available, an approximate value of 2.10 may be used for gravel and an approximate value of 1.87 may be used for sand. An average value of σ is not available for non-cohesive silts; however, a conservative estimate may be obtained by assuming a value of 1.0.

Table 9-1: Coefficients for Computing Scour Hole Dimensions Using Equation 9-4

	α	β	θ
Depth (d_s)	2.27	0.39	0.06
Width (W_s)	6.94	0.53	0.08
Length (L_s)	17.10	0.47	0.10

Table 9-2: Coefficients, C_s, for Culvert Slope Using Equation 9-4

Culvert Slope (%)	Depth	Width	Length
0	1.00	1.00	1.00
2	1.03	1.28	1.17
5	1.08	1.28	1.17
≥ 7	1.12	1.28	1.17

Table 9-3 Coefficients, C_h, for Culvert Outlets Above the Stream Bed1

Drop Height (H_d)*	Depth	Width	Length
0	1.00	1.00	1.00
1	1.22	1.51	0.73
2	1.26	1.54	0.73
4	1.34	1.66	0.73
* Height in pipe diameters			

Using Equation 9-4 coefficients have been derived from experiments with sand bed materials.

9.3.5 Design Procedures

Detailed design procedures for the energy dissipater types described in this Manual are provided in the FHWA document HEC-14, Hydraulic Design of Energy Dissipaters for Culverts and Channels. This document is available on the Internet from the Federal Highway Administration hydraulics home page.

This Chapter provides a general procedure for determining whether an energy dissipater is needed and for selecting the type of dissipater, as well as detailed design procedures for the Stilling Basin and the USBR Type VI Impact Basin.

When it becomes necessary to perform a design for other types of dissipaters, the designer should refer to HEC-14 for detailed guidance and computational procedures. Specific comments and notes helpful in the application of the HEC-14 procedures are provided in the second half of this section.

General Design Procedures

The following design procedure is intended to provide a convenient and organized procedure for manually designing energy dissipaters. The designer should be familiar with all of the equations in Section 9.3 before using this procedure. In addition, application of

the following design method without an understanding of the applicable hydraulic principals can result in an inadequate, unsafe or costly structure.

Step 1: Obtain the culvert design file and assemble the required data. This should include:

- Survey data as defined in the ERA Survey Manual and other site information;
- Design storm frequency and discharge (will be the same as used for the culvert design, detailed in Chapter 2, Table 2.1);
- Tailwater information, including the channel slope, cross section, normal depth and velocity;
- Information on the composition of the downstream bed and bank materials;
- Information on the proposed culvert design, including the culvert type (size, shape and roughness), outlet flow conditions (see Section 9.3.3), culvert slope; and
- The culvert performance curve.

Step 2: Enter the data from Step 1 onto the Energy Dissipator Worksheet provided in the chapter Appendix as Figure 9A-1.

Step 3: Estimate the scour hole size. Enter the required data onto the Energy Dissipater Worksheet and compute d_s, W_s, and L_s using Equations 9-4 through 9-7.

Step 4: Determine the need for an energy dissipater using the criteria presented in Section 9.2.

Step 5: Select dissipater design alternatives based on Section 9.2.3. More than one alternate may be possible. The alternate that provides the best overall fit for the site may become apparent as detailed designs are developed for each one.

Step 6: Develop designs for each of the alternates identified in Step 5. Design procedures and forms for each dissipater type are presented in the Appendix 9A.

Step 7: Design the riprap apron. Many dissipaters may require a riprap apron between the outlet of the dissipater and the natural channel. This provides for a smooth flow pattern between the dissipater and the channel and provides any final erosion protection that may be required. The length and class of riprap for the apron should be determined based on the procedure provided in Chapter 8.

Step 8: Select the cut-off wall depth. Where necessary, energy dissipation structures that are constructed of reinforced concrete should be provided with a cut-off wall of sufficient depth to protect the basin outfall. The cut-off wall depth may be selected based on the criteria provided above.

Step 9: The need for any structural design of a reinforced concrete energy dissipater should be coordinated with ERA. In areas which may be subject to a high water table ERA should also be consulted with regard to the buoyancy of the structure. If the ground is saturated, and tailwater conditions exist, the structure may be subject to buoyant forces that are relative in strength to the volume of water displaced by the structure. Flotation of the structure will occur when its weight is equal to or less than the uplift force exerted by the water. Buoyancy analysis should be performed if the possibility of flotation exists.

Step 10: Review the results. At a minimum, the following items should be addressed:

- If the downstream channel conditions (velocity, depth, or stability) are exceeded, provide a riprap apron or select another type of dissipater;

- If the preferred dissipater affects the hydraulic performance of the proposed culvert, re-compute the culvert performance and insure that the selected dissipater design will still be adequate. Once any needed adjustments are made to the dissipater design, it is not necessary to check the culvert hydraulics any further;
- Ensure that the proposed dissipater will adequately pass debris expected at the site, or that it will not require excessive maintenance. Check whether the proposed energy dissipater, and any needed riprap apron, will be contained within the proposed right-of-way. If not, it may be necessary to obtain a permanent drainage easement to accommodate the structure.

Notes on HEC-14 Procedures

Although the FHWA HEC-14 document provides detailed procedures for the design of the energy dissipaters discussed in this Chapter, there are points at which specific comments may be helpful in applying these procedures. This section provides suggestions and other guidance information intended to aid the designer in developing energy dissipater designs that are consistent with the guidelines set forth in this chapter.

The only dissipater designs for which a detailed procedure is provided in this Chapter are the stilling basin and the USBR Type VI impact basin. The designer should consult HEC-14 for other types of energy dissipater detailed design methods.

HEC-14 Procedure for Riprap Basins

Section 9.2 of this Chapter provides design details for a permanent riprap basin energy dissipater. The procedure provided in HEC-14 should be used to design a riprap basin.

Equation 10.1 in HEC-14 may be used to compute the required depth of a riprap basin (hs in HEC-14) in a ratio with the equivalent depth (de) at the culvert outfall. This equation contains a correction factor, C_o, which varies with the tailwater depth. This correction factor is computed by one of two sets of equations (10.2 or 10.3, HEC-14) depending on whether a more conservative design is desired. In general Equation10.2 will result in basin depths 0.3 to 0.6 m greater than Equation 10.3; however, it will also allow a basin to be used in a greater number of situations and is therefore recommended for use.

The application of Equation 10.1 in HEC-14 is based on assuming a value for the D_{50} of the riprap and then back-checking to ensure that the resulting value for H1 meets the following criteria:

$$\frac{H1}{D_{50}} \geq 2 \qquad (9\text{-}9)$$

Theoretically, the most efficient design would be to select a rock gradation such that the value of this ratio is as close to 2 as possible. For the great majority of situations where a riprap stilling basin will be feasible, the value of H1 determined by Equation 10.1 in HEC-14, will be between 0.5 and 0.8 m, even with using Equation 10.2 of HEC-14 to determine C_o. For many smaller culverts, the value of H1 determined by Equation 10.1 will be zero or negative, especially where the tailwater depth is more than approximately 85% of the equivalent depth at the culvert outfall. It is recommended that a riprap apron be considered for these situations.

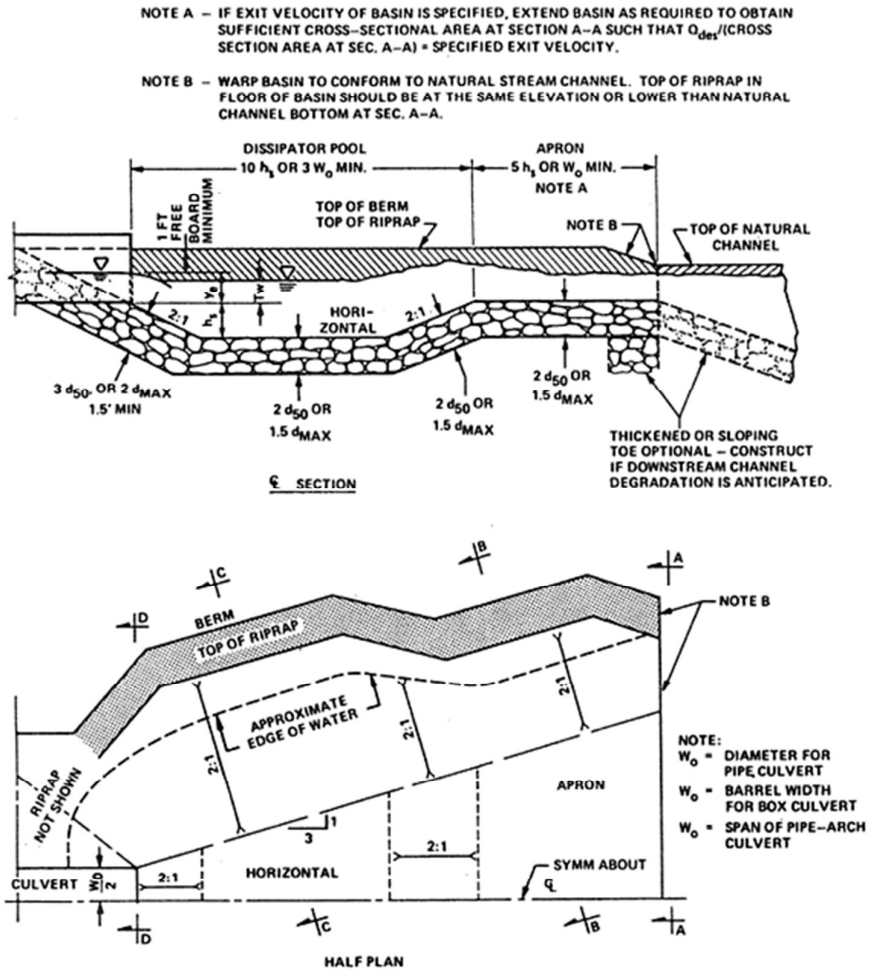


Figure 9-5: Typical Riprap Stilling Basin

(Source: USDOT, FHWA, HEC-14 1983)

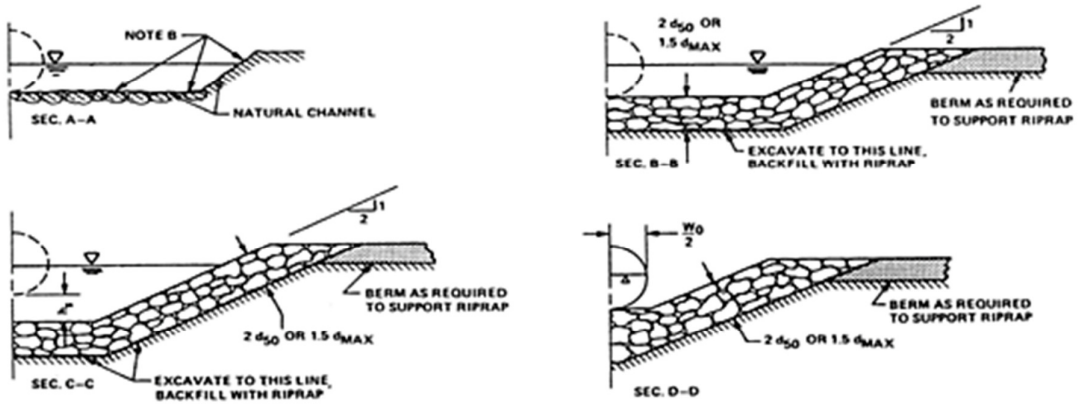


Figure 9-6: Typical Riprap Stilling Basin

(Source: USDOT, FHWA, HEC-14 1983)

HEC-14 Procedures for Internal Energy Dissipater

The FHWA publication HEC-14 provides design procedures for internal roughness elements which serve to dissipate energy either by producing tumbling flow or by presenting increased roughness. The Appendix to this chapter provides design computation worksheets for both types of flow in box culverts. Although HEC-14 includes procedures for tumbling flow or increased roughness in round pipes as well as box culverts, the use of roughness elements in round pipe is not recommended due to concerns regarding the maintenance of such structures.

USBR TYPE VI Impact Basin Design Procedure

The FHWA publication HEC-14 provides a very simple procedure for selecting the dimensions of a USBR Type VI impact basin. However, it does not include detailed instructions for estimating the total energy loss through the structure, nor does it provide specific guidance on the design of any riprap apron that may be required at the outlet. This section presents the HEC-14 method for determining the dimensions of the basin (see Figures 9-1, 9-2, and 9-7) as well as a few additional steps which may be helpful in completing the design.

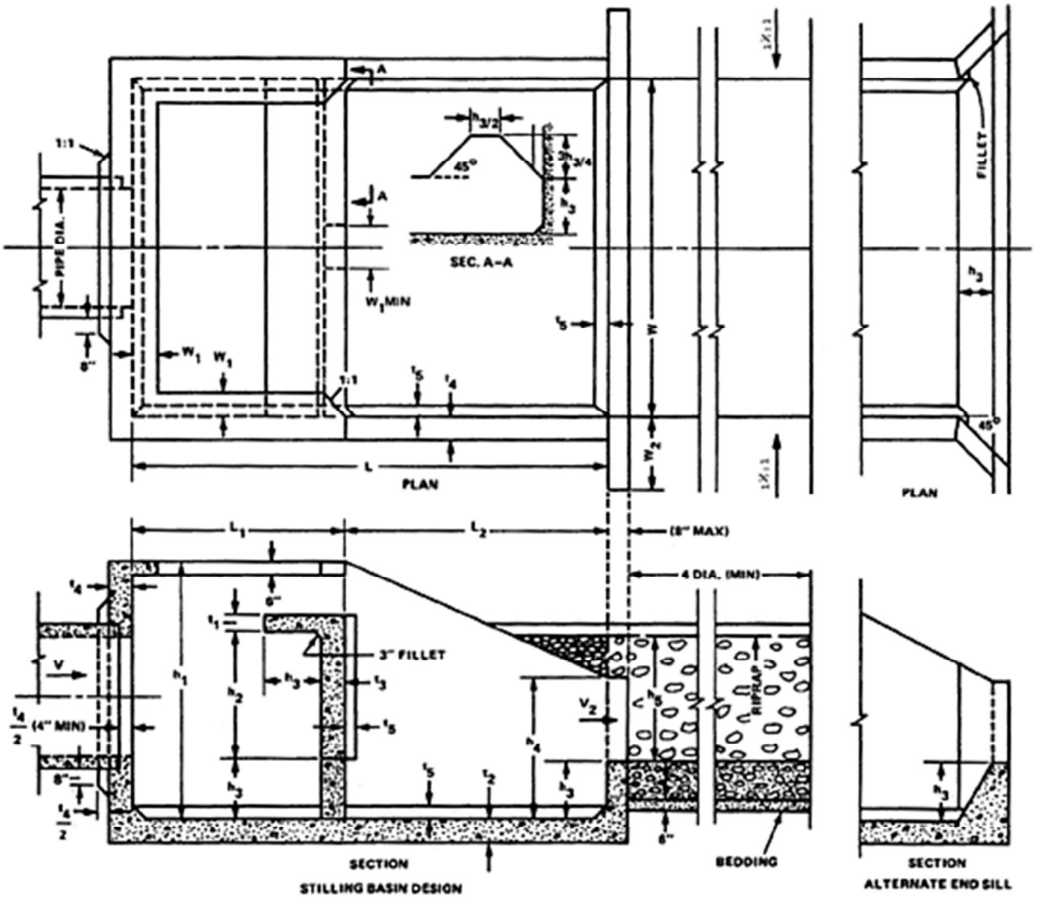


Figure 9-7: Typical USBR Type VI Baffled Dissipator

(Source: USDOT, FHWA, HEC-14 1983)

Step 1: Determine the depth, d_o, equivalent depth, de, velocity, V_o, and Froude Number, Fr_o of the flow at the culvert outlet using the procedures provided in Section 9.3.4.1. In addition, determine the depth of flow (tailwater) in the stream cross section downstream of the basin.

Step 2: Compute the specific energy, H_o, of the culvert outflow using Equation 9-10:

$$H_0 = d_0 + \frac{{V_0}^2}{2g} \quad (9\text{-}10)$$

Where: H_o = specific energy of culvert outflow, (m)
d_o = depth of flow at the outlet, (m)
V_o = velocity of flow at the outlet, (m/s)
g = acceleration due to gravity, (9.81 m/sec^2)

Step 3: Compute a value for the ratio of the outlet specific energy, H_o , to the width the basin, W, from Equation 9-11:

$$\frac{H_0}{W} = 0.0348{Fr_0}^2 + 0.1343Fr_0 + 0.1128 \quad (9\text{-}11)$$

Where: H_o = specific energy of the culvert outflow, (m)
W = width of the impact basin, (m)
Fr_o = Froude Number of the culvert outflow

Determine the required width, W, of the basin by dividing the specific energy computed in *Step 2* by the value for H_o / W determined by Equation 9-11. The result should be rounded to the nearest meter.

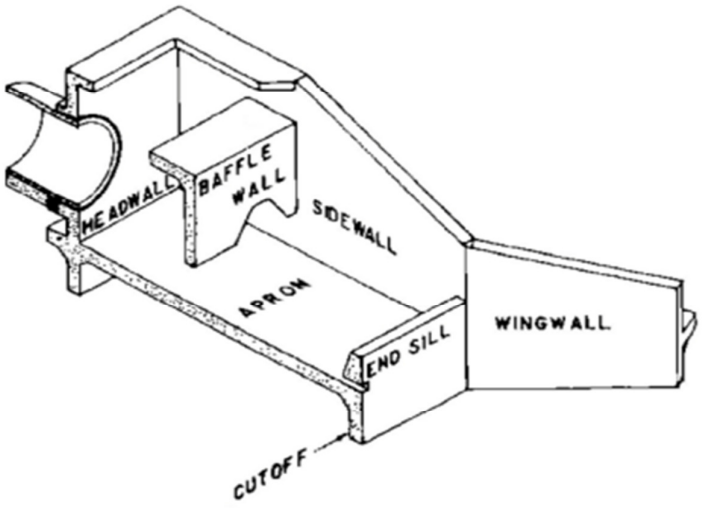


Figure 9-8: "Cut-Away" Isometric View of USBR Type VI Baffled Dissipater

(Source: USDA, NRCS, TR-49 1971)

Step 4: Based on the computed value of W in **Step 3**, obtain values for h_2 and h_3 and verify that Equation 9-12 is true.

$$TW \leq h_3 + \frac{h_2}{2} \quad (9\text{-}12)$$

Where: TW = tailwater depth computed in Step 1, (m)

If Equation 9-12 is not true, the culvert outlet and the basin should be raised such that the height of the tailwater surface above the end sill will make the expression true. This will have the effect of changing the slope of the culvert. Usually, this will require the culvert performance be reanalysed and the procedure would begin again from Step 1.

Step 5: The remaining dimensions of the basin should be determined from Table 9-2 in the Appendix.

Step 6: Compute a value for the ratio of the head lost, HL, in the impact basin to the specific energy, H_o, at the culvert outlet from Equation 9-13. Equation 9-13 utilizes the natural log of the Froude Number at the culvert outlet.

$$\frac{H_L}{H_0} = 0.2718ln(Fr) + 0.2328 \quad (9\text{-}13)$$

Where: H_L = head lost in the impact basin, (m)
H_o = specific energy at the culvert outlet, (m)
Fr = Froude Number of the culvert outflow

The ratio computed above may then be multiplied by H_o to estimate the total energy lost in the basin.

Step 7: Compute the energy, H_E, at the basin outlet as:

$$H_E = H_o - H_L \quad (9\text{-}14)$$

Step 8: Determine the depth of flow, d E , at the basin outlet. There are three possible values for this depth. The first possible value is based on the energy at the basin outlet and may be estimated from Equation 9-39:

$$H_E = d_E + \frac{\left[\frac{Q}{W*d_E}\right]^2}{2g} \quad (9\text{-}15)$$

Where:
H_E = energy at the basin outlet
d_E = depth of flow over end sill, (m)
Q = design discharge, (m^3/s)
W = basin width at the end sill, (m)
g = acceleration due to gravity, (9.82 m/sec^2)

This equation should be solved for d_E using trial and error. Typically, two values for d_E will be possible from this expression, one in the supercritical regime and the other in the subcritical regime. The subcritical solution, which will involve the greater value of d_E, should be considered for use.

The second possible value for d_E is the critical depth of flow across the end sill at the design discharge. This may be computed from the equation:

$$d_{Ec} = \left[\frac{Q}{W\sqrt{g}}\right]^{0.667} \quad (9\text{-}16)$$

Where:
d_{Ec} = critical depth at the end sill, (m)
Q = design discharge, (m^3/s)
W = basin width at the end sill, (m)
g = acceleration due to gravity, (9.81 m /sec^2)

The third possible value for d_E is the depth of flow (tailwater) in the channel cross section downstream of the basin, which was determined in Step 1.

The value to be used for the depth of flow at the end sill may be determined by comparing the tailwater depth with the other possible values for d_E . Where the tailwater depth is greater than d_E, as computed from the energy at the basin outlet, d_E will be equal to TW. Where TW is less than the critical depth, d_{Ec}, the outlet depth will be equal to d_{Ec} .Where TW is between the two values, d_E may be assumed to be equal to the value computed based on energy loss.

Step 9: The flow velocity, V_E, across the basin sill can be computed using Equation 9-17

$$V_E = \frac{Q}{W * d_E} \quad (9\text{-}17)$$

When V_E is significantly greater than the natural stream flow velocity, erosion protection should be provided in the form of a riprap .

Step 10: It is recommended that the basin outlet be provided with a cut-off wall. The depth of this wall may be determined.

HEC-14 Procedure for Hook Impact Basins

HEC-14 provides design information on hook impact basins, which may be constructed as either straight or flared trapezoidal basins. In general, the straight trapezoidal basin is recommended for use.

The dimensions of a hook type impact basin (see Figure 9-9) may be selected using the procedure provided in HEC-14 for straight trapezoidal basins, taking into account the following notes:

- The trapezoidal shape of the basin should be modified to fit the downstream channel as well as possible. Once the effective width of the culvert cross section, Wo in HEC-14, has been determined, the width of the trapezoid floor, W6 in HEC-14, may be any width between Wo and 2 times Wo, to match the existing channel bottom width as closely as possible;
- All three hooks should have the same width, W4, which is equal to 0.16 times the effective culvert width, Wo. Other dimensions necessary for the proper design of the hooks can be determined from Figure 9-10;
- When the guidance provided in HEC-14 is followed, the width between the two upstream hooks, W2, plus the width of the two hooks, should be approximately equal to the effective culvert width. This spacing will not change when the floor width, W6, is greater than Wo;
- Further, when the HEC-14 procedure is followed, the ratio of the spacing between the upstream and downstream hooks, W3, to the hook width, W4, will always be about 1.6. Therefore, it should not be necessary to check that the ratio is greater than one;
- HEC-14 provides a graph which may be used to determine the reduction in flow velocity that will be provided by the proposed basin as a function of the Froude Number of the culvert outflow. This graph provides two efficiency curves, one for W6 = Wo and one for W6 = 2Wo. Where the width of the basin floor, W6, is equal to one of these two values, the curves may be used directly to determine the ratio of the culvert outlet velocity, Vo, to the velocity at the basin outlet, VB. When the floor width falls between these two values, the designer should interpolate between these two curves. As an alternative to using the curves, the designer may choose to

interpolate a value for the ratio of the culvert outlet velocity to the basin outlet velocity (V_o/V_B) from Table 9-5.

Table 9-2: V_o/V_B versus Culvert Outlet Froude Number for Various Floor Widths

Culvert Outlet Froude Number	Floor Width = W_o	Floor Width = 2 times W_o
1.8	1.469	1.352
1.9	1.491	1.429
2	1.519	1.528
2.1	1.550	1.646
2.2	1.585	1.792
2.3	1.633	1.899
2.4	1.701	1.970
2.5	1.773	2.013
2.6	1.858	2.051
2.7	1.949	2.089
2.8	2.031	2.122
2.9	2.110	2.164
3	2.180	2.218

(Source: Adapted from HEC-14)

A value for Vo/V_B may be interpolated from both columns in the table based on a given culvert outlet Froude Number. Based on the proposed basin floor width, the final value would then be interpolated from the two values taken from the table.

- Where VB is significantly greater than the natural stream flow velocity, erosion protection should be provided in the form of a riprap apron;
- It is recommended that the basin outlet be provided with a cut-off wall with a depth as determined.

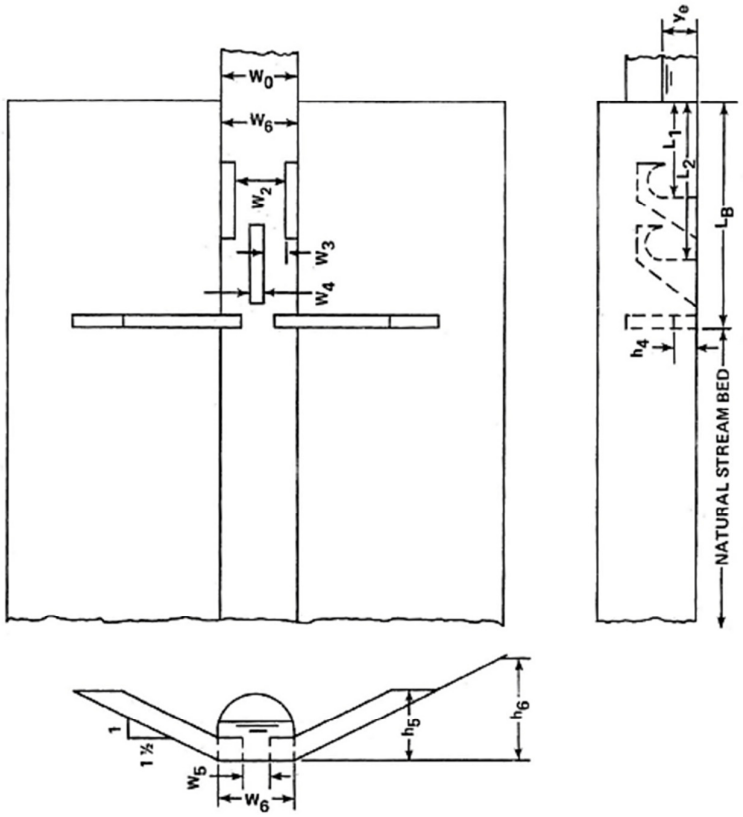
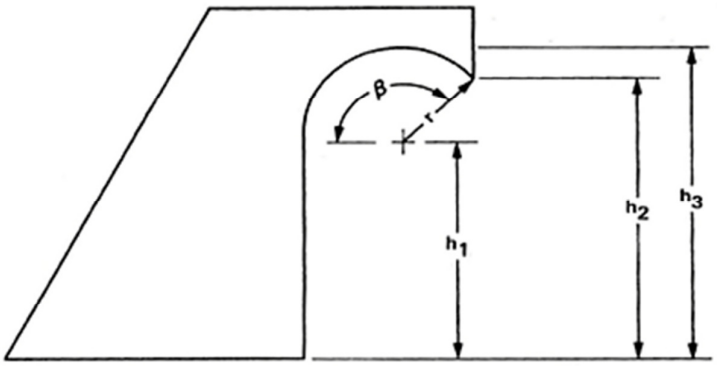


Figure 9-9: Hook Type Energy Dissipater Basin

(Source: USDOT, FHWA, HEC-14 1983)



$h_2 = Y_e$

$h_1 = 0.78\, Y_e$

$h_3 = 1.4\, h_1$

$r = 0.4\, h_1$

$\beta = 135^{o}$

Figure 9-10: Hook Detail

(Source: USDOT, FHWA, HEC-14 1983)

Cut of Wall Depths

Except in areas where the stream bed is composed of competent bed rock, a cut-off wall should be provided at the outfall of the stilling basin. The cut-off wall should be a minimum of 0.9m deep, unless site-specific conditions require a greater depth.

9.4 Acceptable Software

The software discussed in the following sections should be used for the design of an energy dissipater unless special circumstances on the project require other software. The ERA project design manager should approve the use of any other software for these special circumstances.

9.4.1 Computer Program Hy-8

HY-8 is a Windows™ based computer program developed by the FHWA for culvert design. Energy dissipater design computations using the methods prescribed in HEC-14 are available as a module within HY-8. The program is capable of providing design information for all of the dissipater options described in this chapter as well as a number of other dissipater types discussed in HEC-14. Features of the computer program include:

- Scour hole estimation;
- Design of internal energy dissipaters;
- Design of external energy dissipaters;
- Automatic evaluation of the feasibility of the available dissipater types;
- Ability to move seamlessly between the culvert design and energy dissipater design modules;
- Convenient means of quickly analyzing a number of dissipater design alternatives for a given site; and
- Output of results to different file formats (.pdf, .rtf or .xls).

The program is available in the Public Domain from the FHWA Hydraulics internet web page.

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9.6 Abbreviations

ERA	Ethiopian Roads Authority
EPA	Environment Protection Agency
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
Fr	Froude Number
HEC-14	Hydrologic Engineering Circular Number 14
USBR	United States Bureau of Reclamation

APPENDIX 9A-1: ENERGY DISSIPATER WORKSHEET

ENERGY DISSIPATER WORKSHEET, Figure 9A-1

Project_____

Station_____ Structure Type_____

Design Frequency_____ Discharge(m^3/s)_____

Designer_____ Date_____

CULVERT DATA

Type	Size (m)	n-value	Length (m)	Slope (%)	d_c (m)	A_c(m2)	V_c(m/s)	T (m)	fr	End treatment

TAILWATER SECTION (See Section 9.04.1.2)

Bottom Width (m)	Side Slope (X:1)	n-value	Slope (%)	Depth (m)	Flow Area (m2)	Velocity (m/s)	Type of materials in channels	

SCOUR HOLE COMPUTAION

Time (min)	d_s (m)	W_s (m)	L_s (m)	Attach Scour Hole Computation Worksheet	

SITE CONSTRAINTS

Allowable Outlet Velocity (m/s)	Allowable scour Dimensions			Other Restrictions	
	Width (m)	Length (m)	Depth(m)		

Comments:

Example Problems: Scour Hole Estimation Using HY-8, VERSION 7.2

***GIVEN*:**

A concrete culvert has been designed as follows:

- Design Discharge (Q50) = 1.132 m^3/s
- Diameter = 2.22m
- Culvert Length = 30.48 m
- Inlet Invert Elevation = 183.64 m AOD
- Outlet Invert Elevation = 182.88 m AOD
- Computed TW depth = 0.49 m

The natural materials in the channel downstream of the culvert outlet consist of grave and small stones, and the flow line of the channel is at the same elevation as the culvert outfall. The duration of the peak flow may be assumed to be 30 minutes. The downstream receiving channel is trapezoidal in shape, 1.22 m wide, and at the same slope as the culvert. The downstream channel n-value is approximately equal to 0.03.

FIND:

Estimate the dimensions of the scour hole for the design discharge. The parameters to be determined will be:

Depth of the scour hole, d_s
Width of the scour hole, W_s
Length of the scour hole, L_s

SOLUTION:

Step 1: Compute and Record Necessary Channel and Culvert Data Using HY8-v7.2

The designer should review Chapters 6 and 7 of this Manual to develop a basic understanding of open channel hydraulics and culvert flow. For the given culvert conditions, the designer should complete a standard culvert design form for the proposed structure at this location.

Entered/Given Data:

Culvert Shape ..... Circular
Number of Barrels 1
Solving for ..... Headwater
FHWA Chart Number 1
Scale Number 1
FHWA Chart Description ..... CONCRETE PIPE; NO BEVELED RING ENTRANCE
Scale Description ..... SQUARE EDGE ENTRANCE WITH HEADWALL
Overtopping Analysis ..... On
Discharge ..... 1.132 m^3/s
Manning's n 0.0130
Roadway Overtopping Elevation ..... 185.56 m AOD
Inlet Elevation ..... 183.64 m AOD
Outlet Elevation ..... 182.88 m AOD
Diameter 2.22m
Length ..... 30.48 m
Entrance Loss 0.5000
Tail water ..... 0.49 m

Computed Results:

Slope ..... 0.0076 m/m
Velocity ..... 4.1508 m/s
Headwater ..... 184.455 mAOD Inlet Control

Messages and/or Errors:

Inlet head > Outlet head.
Computing Inlet Control headwater.
Headwater: 184.455 m AOD

Discharge flow	Head water elev.	Inlet control depth	Outlet control depth	Flow type	Normal depth	Critical depth	Outlet		Tailwater	
							Velocity	Depth	Velocity	Depth
m^3/s	m	m	m	m^3/s	m	m	m/s	m	m/s	m
0.566	184.18	0.54	0.00	N/A	0.24	0.40	3.39	0.24	0.00	0.47
0.707	184.25	0.61	0.00	N/A	0.27	0.45	3.62	0.27	0.00	0.47
0.849	184.32	0.68	0.122	N/A	0.30	0.49	3.82	0.30	0.00	0.47
0.99	184.39	0.75	0.195	N/A	0.32	0.54	4.00	0.32	0.00	0.47
1.132	184.45	0.81	0.26	N/A	0.35	0.58	4.15	0.35	0.00	0.47

Step 2: Begin Energy Dissipater and Scour Hole Worksheet

The culvert and channel tailwater information obtained in Step 1, along with general project information, will be entered on the top half of the energy dissipater worksheet (see Figure 9A-1). At this time, any known site constraints limiting the geometry of the scour hole may also be entered on the lower portion of the worksheet. The completed energy dissipater worksheet for this culvert site is provided as Figure 9A-2.

The project background and design data should be entered on the natural scour hole computation worksheet (see Figure 9A-2). Additionally, the bed material standard deviation should be determined at this point. Geotechnical analysis may be available for determining this parameter. A default value of 2.10 for the given gravel channel will be used for this example. A copy of the completed scour hole computational worksheet is provided as Figure 9A-12. To complete the scour hole worksheet, proceed to Steps 3 and 4.

Step 3: Perform Scour Hole Computations to determine depth, ds Using the information from the culvert design form, the designer should compute the width, Ws, length Ls, and depth, ds, of the scour hole by applying Equation 9-4 three times, once for each parameter of the scour hole. Cs and C_h of Equation 9-4 are adjustment factors to account for the effects of culvert slope and drop between the culvert exit and the channel bed.

Using the given site information, these factors can be directly obtained or interpolated from Tables 9-3 and 9-4. Enter the values for these coefficients into the depth column at the lower portion of the scour hole computation form.

The terms F_1, F_2, and F_3 of Equation 9-4 can be determined by Equations 9-5 through 9-8. A different value of the coefficients termed α, β, and θ will be used to solve these equations for each of the three iterations performed to solve Equation 9-4. The terms α, β, and θ may be obtained from Table 9-2.

The first computation or iteration of Equation 9-4 should be performed to determine the scour hole depth, as follows:

Solve Equation 9-5 for the term F_1,

$$F_1 = \frac{a}{\sigma^{1/3}}$$

Where, σ = the given material standard deviation and α is obtained from Table 9-2,

$$F_1 = \frac{2.27}{2.10^{0.333}} \qquad F_1 = 1.77$$

Enter the value for F_1 in the depth column on the lower portion of the natural scour hole computation worksheet as shown in Figure 9A-2.

Solve Equation 9-6 for the term F_2 as follows:

$$F_2 = \left(\frac{Q}{g^{0.5} {R_c}^{2.5}}\right)^{\beta}$$

Where, R_c is the hydraulic radius of the culvert flowing full and β is obtained from Table 9-2,

$$F_2 = \left(\frac{1.132}{9.81^{0.5} 0.555_0{}^{2.5}}\right)^{0.39} \qquad F_2 = 1.194$$

Enter the value for F_2 in the depth column of the lower portion of the natural scour hole computation worksheet as shown in Figure 9A-2.

Then, the designer should solve Equation 9-7 for the term F_3 as follows:

$$F_3 = \left(\frac{t}{316}\right)^{\theta}$$

Where, t is the duration of peak flow (see Section 9.3.1.3 for a discussion of t in terms of a base time of 316 minutes) and θ is obtained from Table 9-2.

$$F_3 = \left(\frac{30}{316}\right)^{0.06} \qquad F_3 = 0.868$$

Enter the value for F_3 in the depth column of the lower portion of the natural scour hole computation worksheet as shown in Figure 9A-2.

With all of the terms of Equation 9-4 computed, the depth of the scour hole can now be determined by solving Equation 9-4 as follows:

$$d_s = C_s C_h F_1 F_2 F_3 R_c$$

d_s = (1.037)(1.0)(1.77)(1.194)(0.868)(0.555)

= 1.056 m

Enter the value for ds at the bottom of the depth column on the scour hole computation worksheet as shown in Figure 9A-2.

Step 4: Perform Scour Hole Computations to Determine Width and Length

At this point in the design procedure, the designer should follow the procedure and equations outlined in Step 3 to solve for the scour hole width and length, Ws and Ls, respectively. The appropriate values for α, β, and θ will be obtained from Table 9-2. Enter the width and length columns of Table 9-3 and Table 9-4 to obtain appropriate values for Cs and C_h.

Solving Equation 9-4 for both width and length, the designer obtains values of 5.67 m for the width, and 10.13 m for the scour hole length. These values should be entered at the bottom of the scour hole worksheet. The scour hole worksheet shown in Figure 9A-2 is now complete.

Step 5: Complete Energy Dissipater Worksheet and Verify Results

Using the values obtained in Steps 3 and 4, the designer should now complete the Energy Dissipater Worksheet as shown in Figure 9A-2. The computed depth, width, and length should be compared to any site constraints that may govern maximum allowable values for these parameters. Analysis of the computed values for this example verses the maximum allowable scour dimensions show the computed dimensions will be acceptable

ENERGY DISSIPATER WORKSHEET: Figure 9A-2

Project_
Sample _____

Station_____Structure Type: __**Reinforced Concrete Pipe**__(RCP)_____

Design Frequency_____**50 year**_____Discharge(m^3/s):**1.132**_____

Designer_____Date_____

CULVERT DATA

Type	Size (m)	n-value	Length (m)	Slope (%)	d_c (m)	A_c(m2)	V_c(m/s)	T (m)	Fr	End treatment
RCP	**2.22**	**0.013**	**30.48**	**0.0076**	**0.3474**	**0.2731**	**4.15**	**2.1**	**2.65**	**TYPE "U"**

TAILWATER SECTION (See Section 9.04.1.2)

Bottom Width (m)	Side Slope (X:1)	n-value	Slope (%)	Depth (m)	Flow Area (m2)	Velocity (m/s)	Type of materials in channels	
1.22	**2:1**	**0.03**	**0.0076**	**0.49**	**1.05**	**1.112**	**Gravel and Small Stone**	

SCOUR HOLE COMPUTAION

Time (min)	d_s (m)	W_s (m)	L_s (m)	Attach Scour Hole Computation Worksheet	
30	**1.02**	**5.67**	**10.135**		

SITE CONSTRAINTS

Allowable Outlet Velocity (m/s)	Allowable scour Dimensions			Other Restrictions	
	Width (m)	Length (m)	Depth(m)		
1.112	7.62	15.24	1.22		

Comments:
Velocity of culvert will pose unacceptable risk to road and downstream channel

10 STORM DRAINAGE FACILITIES

10.1 Introduction

Road storm drainage facilities collect storm water runoff and convey it through the carriageway right-of-way in a manner that adequately drains the carriageway and minimizes the potential for flooding and erosion to properties adjacent to the right-of-way. Storm drainage facilities consist of curbs, gutters, storm drains, channels and culverts. The placement and hydraulic capacities of storm drainage structures and conveyances shall be designed to take into consideration damage to adjacent property and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds.



Photo 10-1: Storm Water System Planning

Storm drainage design is an integral component in the design of road and transportation networks. Drainage design for road facilities must strive to maintain compatibility and minimize interference with existing drainage patterns, control flooding of the road surface for design flood events, and minimize potential environmental impacts from road related storm water runoff. To meet these goals, the planning and coordination of storm drainage systems must begin in the early planning phases of transportation projects.

System planning, prior to commencement of design, is essential to the successful development of a final storm drainage design. Successful system planning will result in a final system design that evolves smoothly through the preliminary and final design stages of the transportation project.

10.2 Storm Water Design Objectives

The objective of road storm drainage design is to provide for safe passage of vehicles during the design storm event. The drainage system is designed to collect storm water runoff from the road surface and right-of-way, convey it along and through the right-of-way, and discharge it to an adequate receiving body without causing adverse on- or off-site impacts.

Storm water collection systems must be designed to provide adequate surface drainage. Traffic safety is intimately related to surface drainage. Rapid removal of stormwater from the pavement minimizes the conditions which can result in the hazards of hydroplaning.

Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity.

The objective of storm water conveyance systems (storm drain piping, ditches and channels, pumps, etc.) is to provide an efficient mechanism for conveying design flows from inlet locations to the discharge point without surcharging inlets or otherwise causing surface flooding. Erosion potential must also be considered in the design of open channels or ditches used for storm water conveyance.

The design of appropriate discharge facilities for storm water collection and conveyance systems includes consideration of storm water quantity and quality. Local, regional, and/or Federal regulations often control the allowable quantity and quality of storm water discharges. This regulation is not available in Ethiopia at the present time. To meet these regulatory requirements, storm drainage systems will usually require detention or retention basins, and/or other best management practices for the control of discharge quantity and quality.

10.3 Design Approach

The design of storm drainage systems is a process which evolves as an overall road design develops. The primary elements of the process include data collection, stakeholder coordination, preliminary concept development, concept refinement and design, and final design and documentation. Each of these elements is briefly described in the following:

Step 1. **Data Collection**

This step involves assembling and reviewing technical data and background information as necessary to perform the design. Data requirements are outlined in Chapter 4 of this manual.

Step 2. Stakeholder Coordination

This step includes coordination with regulatory and other impacted or interested stakeholders or groups. Additional information on stakeholder coordination is provided in section 2.4 of this chapter and Chapter 4.

Step 3. Preliminary Concept Design Development

This step involves the development of a preliminary sketch plan and layout for the proposed storm drainage system.

Step 4. Concept Design Refinement: Hydrologic and Hydraulic Design

This step comprises the primary design phase which generally proceeds in the following sequence:

Computation of runoff parameters and quantities based on the preliminary concept layout (see Chapter 5, Hydrology);

1. Refine inlet location and spacing;
2. Refine the storm drain system layout including access holes, connecting mains, outfall control structures, and any other system components;
3. Size pipes, channels, pump stations, discharge control structures, and other storm drain system components;
4. Compute and review the hydraulic grade line;
5. Revise plan and recompute design parameters as necessary.

Through this step the design of the storm drainage system will evolve from the preliminary concept stage to final design as a continuing process. Several levels of system refinement are usually required in response to design changes in the overall transportation process, and input from regulatory and review agencies.

Step 5. Final Design Documentation

This step includes the preparation of final documentation for the design files and construction plans. Final design documentation requirements are typically defined by ERA, and can vary depending on project scope. For detailed discussions of final design documentation refer to Chapter 4. A listing of general documentation to be provided in the final design follows:

1. Hydrology

- Contributing watershed size and identification of source (map name, etc.);
- Design frequency and decision for selection;
- Hydrologic discharge and hydrograph estimating method and findings;
- Flood frequency curves to include design, chosen peak discharge, discharge hydrograph, and any historical floods;
- Expected level of development in upstream watershed over the anticipated life of the facility (include sources of, and basis for these development projections).

2. Open Channels

- Stage discharge curves for the design, peak discharge, and any historical water surface elevations;
- Cross section(s) used in the design water surface determinations and their locations;
- Roughness coefficient assignments ("n" values);
- Methods used to obtain the design water surface elevations; and
- Design or analysis of materials proposed for the channel bed and banks.

3. Storm Drains

- Computations for inlets and pipes (including energy grade lines);
- Complete drainage area map;
- Design Frequency;
- Inflow concerning outfalls, existing storm drains, and other design considerations; and
- Schematic indicating storm drain system layout.

4. Pump Stations

- Inflow design hydrograph from drainage area to pump ;
- Maximum allowable headwater elevations and related probable damage;
- Starting sequence and elevations;
- Sump dimensions;
- Available storage amounts;
- Pump sizes and operation;
- Pump calculations and design report; and
- Mass curve routing diagram.

10.4 Data Requirements

The design of storm drainage system requires the accumulation of certain basic data including the following information (for further details refer to Chapter 4):

Watershed mapping: identifying topographic features, watershed boundaries, existing drainage patterns, and ground cover. Information sources include maps, field surveys, aerial photography, or mapping available from local authorities, districts road authorities, Ministry of water or other planning agencies.

Land use mapping: identifying existing and expected future land uses. This information is typically available from local zoning or planning agencies.

Soils maps: identifying soil types and hydrologic soil groups. This information is available in county soil surveys which can be obtained from Ministry of Agriculture offices.

Flood histories and high water mark elevations: Information of this type may be available from local offices of the Ministry of Water and Energy, planning agencies, local authorities or ERA district offices. Local residents or ERA regional or district maintenance offices may also be able to provide this information.

Description of existing drainage facilities: including size, shape, material, invert information, age, condition, etc. As-built information for existing drainage facilities may be available from the local owner of the facility. If unavailable, field surveys will need to be performed to obtain this information.

Design performance data for existing drainage systems: This information may be available from the local owner of the facility. If the information is not available for the existing system, it will be necessary for the designer to develop the needed information to define how the existing system will function under the new loading from the road runoff.

Utility plans and descriptions. This is normally available from electricity and water utility authorities. If unavailable, field surveys may need to be performed to determine critical design information.

Existing right-of-way information: Available from appropriate ERA right-of-way office, or local maps.

***Federal, Regional, and local regulatory requirements*:** Information can be obtained from local regulatory agencies. Typical regulatory authorities include The Ethiopian Environment Protection Agency, and local governments. Typical regulatory considerations are discussed in Chapter 3.

10.5 Stakeholder Coordination

Prior to the design of a storm drainage system, it is essential to coordinate with regulatory agencies or others that have interests in drainage matters. Regulatory agency involvement may come from any level of government (federal, regional, or local). The concerns of these agencies are generally related to potential impacts resulting from road drainage, and centre on storm water quantity and quality issues.

Others with interests in storm drainage systems include local municipalities, and property developers. Local municipalities may desire to use portions of the road storm drainage system to provide for new or better drainage, or to augment old municipal drainage systems. Local municipalities may be interested in developing cooperative projects where a

mutual economic benefit may exist. Local municipalities may also be aware of proposed private development in the vicinity of the road project which may impact drainage design. These groups may wish to improve or change drainage patterns, redirect storm water to the right-of-way, or propose joint projects which could require the road storm drainage system to carry water for which it would not usually be designed. Early planning and coordination is required to identify and coordinate cooperative projects.

Also important are the concerns of citizens who fear that the drainage facility might impact their business or home. Citizen concerns typically include the road's interruption and redirection of existing drainage patterns, the potential for flow concentration and increased flooding, and water quality impacts to both surface and groundwater. Communication and coordination with local citizens is usually accomplished through local government entities and the public hearing process.

10.6 Preliminary Concept Development

Layout and design of a storm drainage system begins with the development of sketches or schematics identifying the basic components of the intended design. This section provides an overview of the concepts involved in the development of a preliminary concept plan.

10.6.1 Base Map

The first step in the development of a concept storm drainage plan is preparation of a project base map. The base map should identify the watershed areas and subareas, land use and cover types, soil types, existing drainage patterns, and other topographic features. This base information is then supplemented with underground utility locations (and elevations if available), a preliminary road plan and profile, and locations of existing and proposed structures.

10.6.2 Major vs. Minor Systems

A complete storm drainage system design includes consideration of both major and minor drainage systems. The minor system, sometimes referred to as the "Convenience" system, consists of the components that have been historically considered as part of the "storm drainage system." These components include curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, pumps, detention basins, water quality control facilities, etc. The minor system is normally designed to carry run-off from 10-year frequency storm events.

The major system provides overland relief for storm water flows exceeding the capacity of the minor system. This usually occurs during more infrequent storm events, such as the 25-, 50-, and 100-year storms. The major system is composed of pathways that are provided - knowingly or unknowingly -for the runoff to flow to natural or man-made receiving channels such as streams, or rivers. The designer should determine (at least in a general sense) the flow pathways and related depths and velocities of the major system under less frequent or check storm conditions (typically a 100-year event is used as the check storm).

Historically, storm drainage design efforts have focused on components of the minor system with little attention being paid to the major system. Although the more significant design effort is still focused on the minor system, lack of attention to the supplementary functioning of the major storm drainage system is no longer acceptable.

10.6.3 Concept Plan

With the preliminary base map completed and the difference between the major and minor system components determined, a conceptual storm drainage plan can be prepared. The development of this plan includes consideration of both major and minor drainage systems and should consist of the following preliminary activities:

- Locate and space inlets;
- Locate main outfall;
- Locate storm mains and other conveyance elements;
- Define detention strategy and storage location;
- Define water quality control strategy and facility locations; and
- Define elements of major drainage system.

With this sketch, or schematic, the designer will be able to proceed with the detailed process of storm drainage design calculations, adjustments, and refinements as defined in Step 4 of the design approach.

10.6.4 System Components

The components of minor storm drainage systems can be categorized by function as those which collect storm water run-off from the road surface and right-of-way, convey it along and through the right-of-way, and discharge it to an adequate receiving body without causing adverse on- or off-site environmental impacts. In addition, major storm drainage systems provide a flood water relief function.

Storm Water Collection

Storm water collection is a function of the minor storm drainage system which is accommodated through the use of roadside and median ditches, gutters, and drainage inlets.

Roadside and Median Ditches are used to intercept run-off and carry it to an adequate storm drain. These ditches should have adequate capacity for the design run-off and should be located and shaped in a manner that does not present a traffic hazard. If necessary, channel linings should be provided to control erosion in ditches. Where design velocities will permit, vegetative linings should be used.

Gutters are used to intercept pavement runoff and carry it along the road shoulder to an adequate storm drain inlet. Curbs are typically installed in combination with gutters where runoff from the pavement surface would erode fill slopes and/or where right-of-way requirements or topographic conditions will not permit the development of roadside ditches. Pavement sections are typically curbed in urban settings. Parabolic gutters without curbs are used in some areas.

Drainage Inlets are the receptors for surface water collected in ditches and gutters, and serve as the mechanism whereby surface water enters storm drains. When located along the shoulder of the road, storm drain inlets are sized and located to limit the spread of surface water onto travel lanes. The term "inlets," as used here, refers to all types of inlets such as grate inlets, curb inlets, slotted inlets, etc.

Drainage inlet locations are often established by the road geometries as well as by the intent to reduce the spread of water onto the road surface. Generally, inlets are placed at low points in the gutter grade, intersections, crosswalks, cross-slope reversals, and on side

streets to prevent the water from flowing onto the main road. Additionally, inlets are placed upgrade of bridges to prevent drainage onto bridge decks and downgrade of bridges to prevent the flow of water from the bridge onto the road surface.

Storm Water Conveyance

Upon reaching the main storm drainage system, storm water is conveyed along and through the right-of-way to its discharge point via storm drains connected by access holes or other access structures. In some situations, storm water pump stations may also be required as a part of the conveyance system.

Storm drains are defined as that portion of the storm drainage system that receives runoff from inlets and conveys the runoff to some point where it is discharged into a channel, water body, or other piped system. Storm drains can be closed conduit or open channel; they consist of one or more pipes or conveyance channels connecting two or more inlets.

Access holes, junction boxes, and inlets serve as access structures and alignment control points in storm drainage systems. Critical design parameters related to these structures include access structure spacing and storm drain deflection. Spacing limits are often dictated by maintenance activities. In addition, these structures should be located at the intersections of two or more storm drains, when there is a change in the pipe size, and at changes in alignment (horizontal or vertical).

Storm water pump stations are required as a part of storm drainage systems in areas where gravity drainage is impossible or not economically justifiable. Storm water pump stations are often required to drain depressed sections of roads.

Storm Water Discharge Controls

Storm water discharge controls are often required to off-set potential runoff quantity and/or quality impacts. Water quantity controls include detention/retention facilities. Water quality controls include extended detention facilities as well as other water quality management practices.

Detention/retention facilities are used to control the quantity of runoff discharged to receiving waters. A reduction in runoff quantity can be achieved by the storage of run-off in detention/retention basins, storm drainage pipes, swales and channels, or other storage facilities. Outlet controls on these facilities are used to reduce the rate of stormwater discharge. This concept should be considered for use in road drainage design where existing downstream receiving channels are inadequate to handle peak flow rates from the road project, where road development would contribute to increased peak flow rates and aggravate downstream flooding problems, or as a technique to reduce the size and associated cost of outfalls from road storm drainage facilities.

Water quality controls are used to control the quality of storm water discharges from road storm drainage systems. Water quality controls include extended detention ponds, wet ponds, infiltration trenches, infiltration basins, porous pavements, sand filters, water quality inlets, vegetative practices, erosion control practices, and wetlands. Classes of pollutants typically associated with road runoff include suspended solids, heavy metals, nutrients, and organics. Water quality controls should be considered for use as mitigation measures where predictions indicate that road runoff may significantly impact the water quality of receiving waters.

10.7 Sustainable Road Drainage System

10.7.1 Vegetated Filter Strips

Vegetated filter strips, also known as filter strips, grass buffer strips and grass filters, are uniformly graded vegetated surfaces (i.e. grass or close-growing native vegetation) that receive runoff from adjacent roads. Vegetated filter strips typically treat sheet flow or small concentrated flows that can be distributed along the width of the strip using a level spreader. Vegetated filter strips are designed to slow runoff velocities, trap sediment, and promote infiltration, thereby reducing runoff volumes.

Vegetated filter strips, also known as filter strips, grass buffer strips and grass filters, are uniformly graded vegetated surfaces (i.e. grass or close-growing native vegetation) that receive runoff from adjacent impervious areas. Vegetated filter strips typically treat sheet flow or small concentrated flows that can be distributed along the width of the strip using a level spreader. Vegetated filter strips are designed to slow runoff velocities, trap sediment, and promote infiltration, thereby reducing runoff volumes.



Photo 10-2: Example Photo of Vegetative Filter Strip

Advantages/Benefits of Vegetative Filter Strip

- Reduces runoff volumes and peak flows;
- Slows runoff velocities and removes sediment;
- Low maintenance requirements;
- Serves as an effective pretreatment for bioretention cell;
- Can mimic natural hydrology;
- Small filter strips may be used in certain urban settings;
- Ideal for residential settings and to treat runoff from small parking lots and roads;
- Can be used as part of runoff conveyance system in combination with other systems; and
- Little or no entrapment hazard for amphibians or other small creatures.

10.7.2 Constructed Storm Water Wetlands

Constructed stormwater wetlands are stormwater wetland systems that maximize the removal of pollutants from stormwater runoff through wetland vegetation uptake, retention and settling. Constructed stormwater wetlands temporarily store runoff in shallow pools that support conditions suitable for the growth of wetland plants. Like extended dry

detention basins and wet basins, constructed stormwater wetlands must be used with other storm water management plans, such as sediment forebays.



Photo 10-3: Example Photo of Constructed Wetland

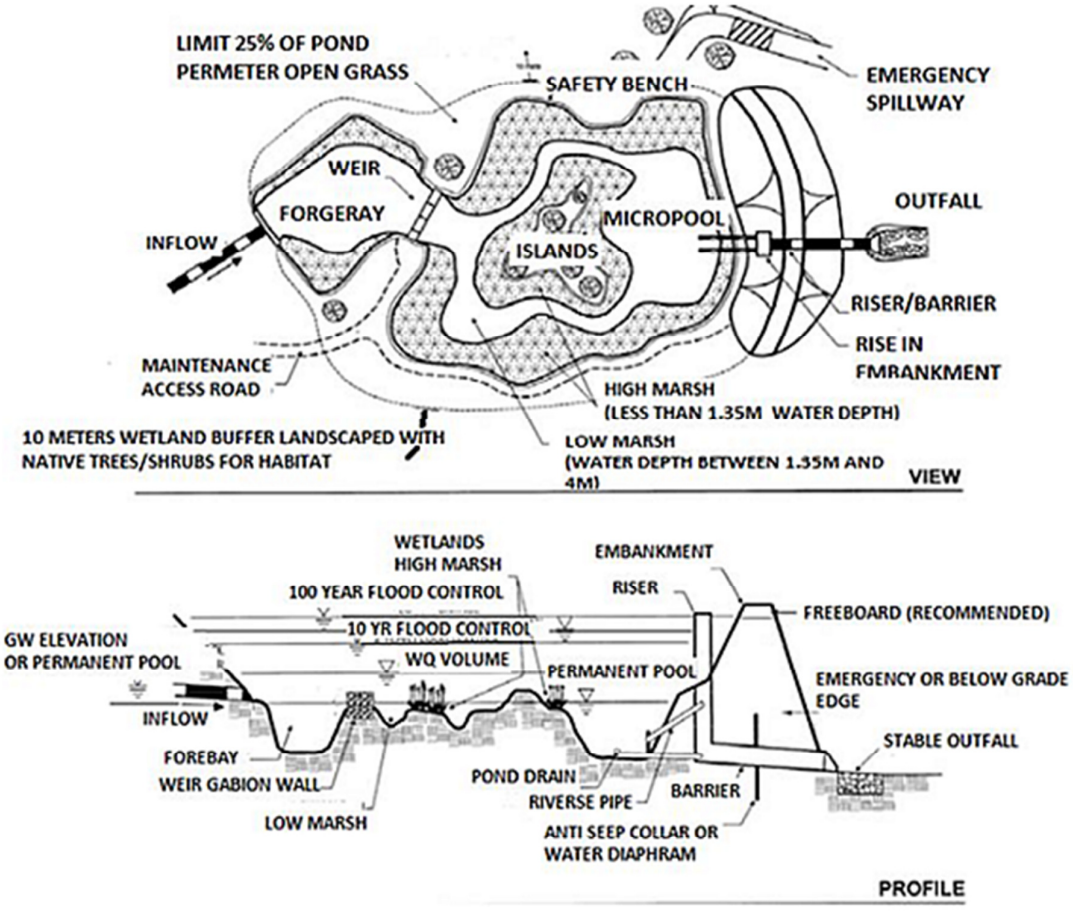


Figure 10-1: Example of Constructed Wetland

(Source: Schueler 1992)

10.7.3 Basin/Wetland Systems (Formerly Pond/Wetland System)

Multiple cell systems, such as basin/wetland systems, use at least one wet basin along with a shallow marsh component. The first cell is a sediment trap that outlets to a wet basin, which removes particulate pollutants. The wet basin also reduces the velocity of the runoff entering the system. Stormwater then travels to the next cell, which contains a plunge pool. The plunge pool acts as an energy dissipator. Shallow marshes provide additional treatment of runoff, particularly for dissolved pollutants. These systems require less space than the shallow marsh systems and generally achieve a higher pollutant removal rate than other stormwater wetland systems.

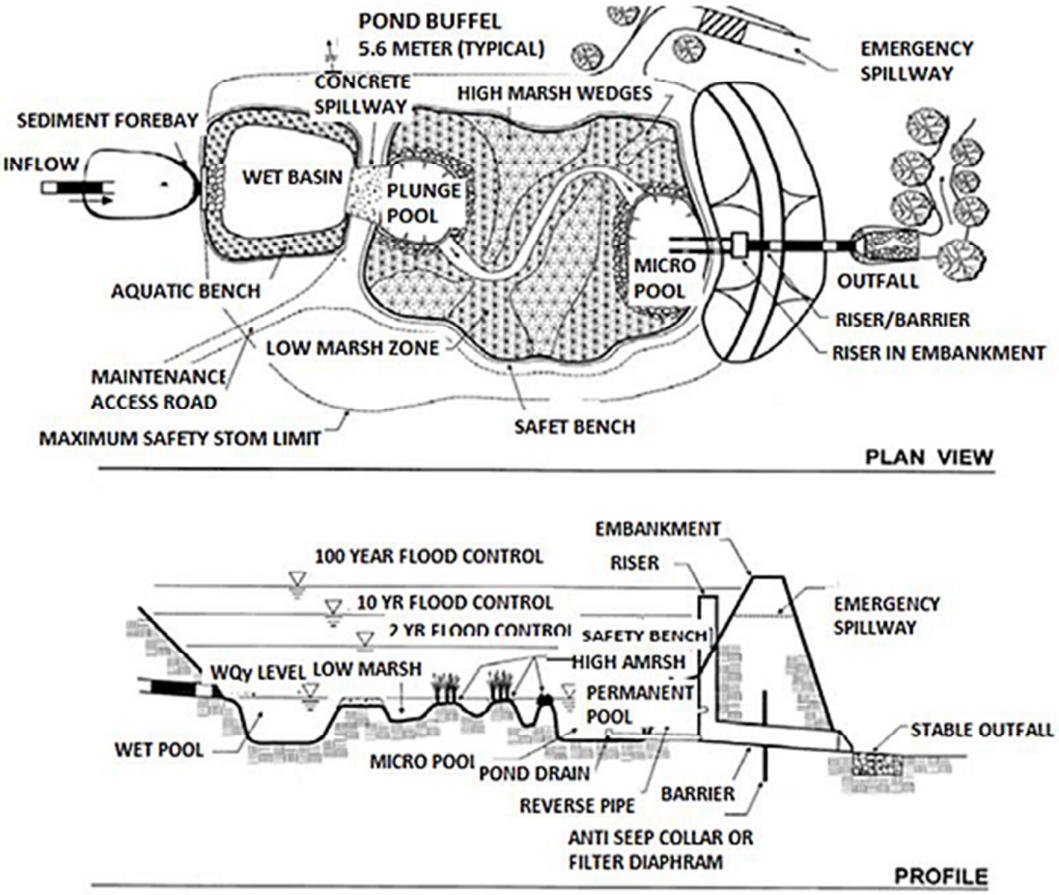


Figure 10-2: Sketch of Basin/Wetland Constructed Storm Water Wetland

(Source: Schueler 1992)

10.7.4 Extended Dry Detention Basin

Extended dry detention basins are modified conventional dry detention basins, designed to hold storm water for at least 24hours to allow solids to settle and to reduce local and downstream flooding. Extended dry detention basins may be designed with either a fixed or adjustable outflow device. Pretreatment is a fundamental design component of an extended dry detention basin to reduce the potential for clogging. Other components such as a micropool or shallow marsh may be added to enhance pollutant removal**.**

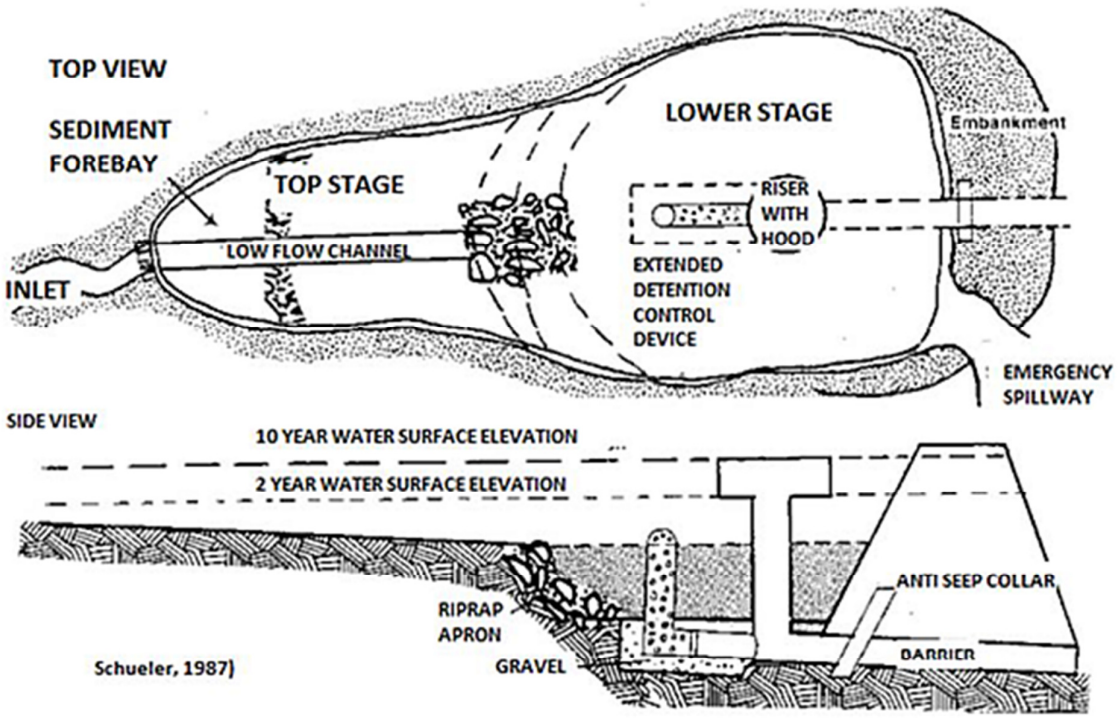


Figure 10-3: Extended Dry Detention Basin

10.7.5 Infiltration Basins

Description: Infiltration basins are storm water runoff impoundments that are constructed over permeable soils. Pre-treatment is critical for effective performance of infiltration basins. Runoff from the design storm is stored until it infiltrates through the soil of the basin floor



Photo 10-4: Example Photo of Infiltration Basin

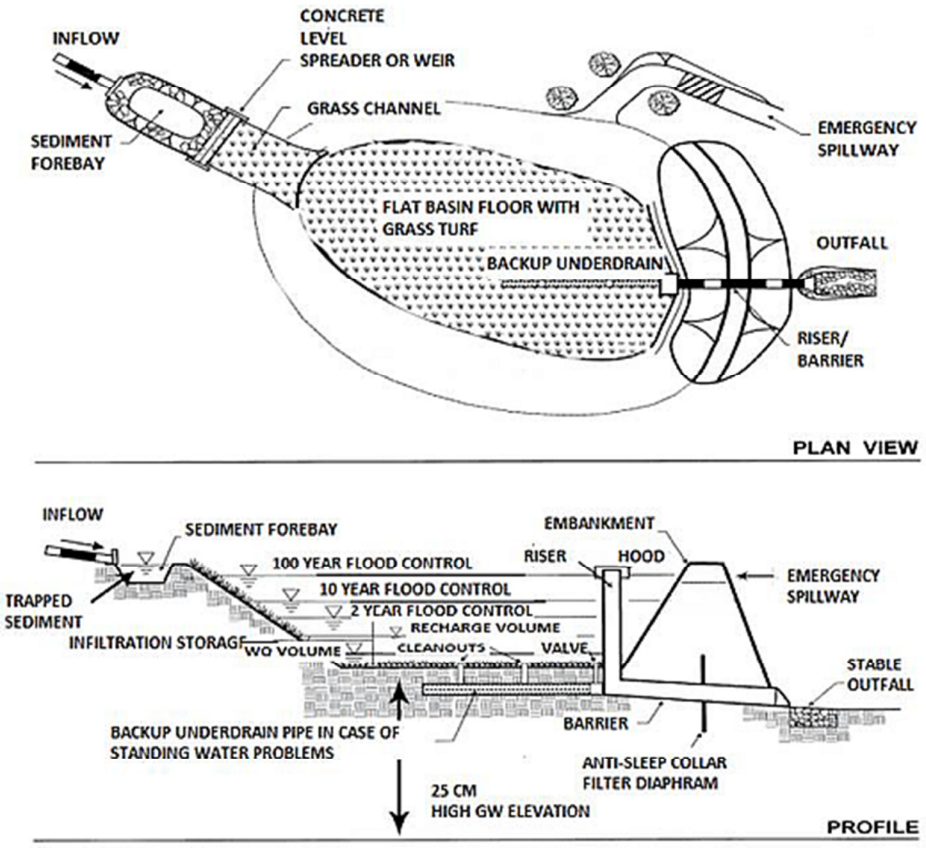


Figure 10-4: Example Plan and Profile of Infiltration Basin

10.7.6 Infiltration Trenches

Infiltration trenches are shallow excavations filled with stone. They can be designed to capture sheet flow or piped inflow. The stone provides underground storage for storm water runoff. The stored runoff gradually infiltrates through the bottom and/or sides of the trench into the subsoil and eventually into the water table.



Photo 10-5: Example Photo of Infiltration Trench

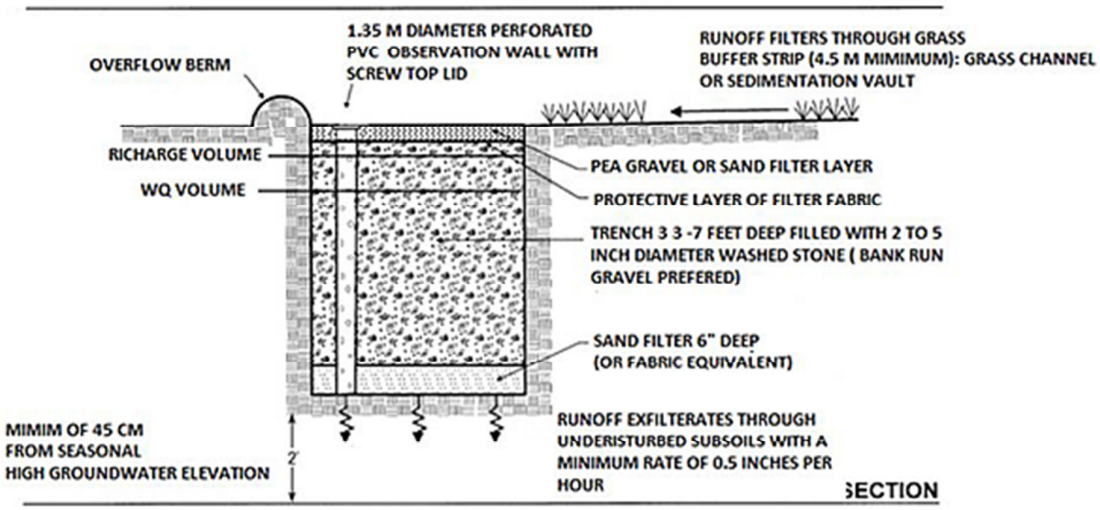


Figure 10-5: Example of Infiltration Trench

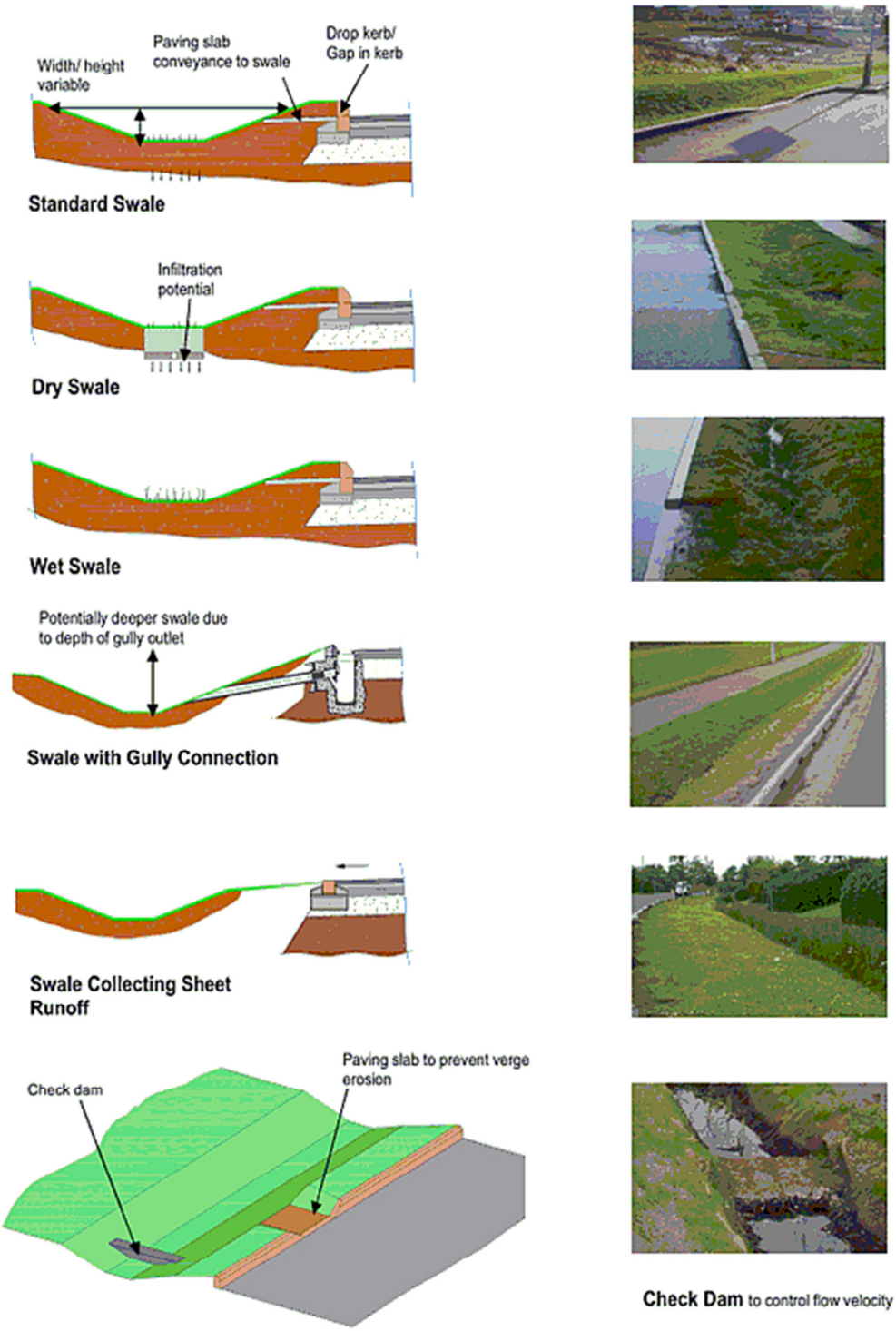


Figure 10-6: Different Types of Sustainable Storm Drainage Storage Devices

(Source: Scottish Road Agency manual,

10.8 Pavement Drainage

Effective drainage of road pavements is essential to the maintenance of road service level and to traffic safety. Water on the pavement can interrupt traffic, reduce skid resistance, increase potential for hydroplaning, and limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. (18)

Pavement drainage requires consideration of surface drainage, gutter flow, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of storm water on the pavement surface. This chapter presents design guidance for the design of these elements.

10.8.1 Design Frequency and Spread

Two of the more significant variables considered in the design of road pavement drainage are the frequency of the design runoff event and the allowable spread of water on the pavement. A related consideration is the use of an event of lesser frequency to check the drainage design.

Spread and design frequency are not independent. The implication of the use of criteria for spread of one-half of a traffic lane is considerably different for one design frequency than for a lesser frequency. It also has different implications for a low-traffic, low-speed road than for a higher classification road. These subjects are central to the issue of road pavement drainage and important to road safety.

10.8.2 Selection of Design Frequency and Design Spread

The objective of road storm drainage design is to provide for safe passage of vehicles during the design storm event. The design of a drainage system for a curbed road pavement section is to collect runoff in the gutter and convey it to pavement inlets in a manner that provides reasonable safety for traffic and pedestrians at a reasonable cost. As spread from the curb increases, the risks of traffic accidents and delays, and the nuisance and possible hazard to pedestrian traffic increase.

The process of selecting the recurrence interval and spread for design involves decision regarding acceptable risks of accidents. It also involves decision for traffic delays and acceptable costs for the drainage system. Risks associated with water on traffic lanes are greater with high traffic volumes, high speeds, and higher road classifications than with lower volumes, speeds, and road classifications. A summary of the major considerations that enter into the selection of design frequency and design spread follows:

- Classification of the road is a good starting point in the selection process since it defines the public's expectations regarding water on the pavement surface. Ponding on traffic lanes of high-speed, high-volume roads is contrary to the public's expectations and thus the risks of accidents and the costs of traffic delays are high (refer to ERA Geometric Design Manual for Ethiopian road classification).
- Design speed is important to the selection of design criteria. At speeds greater than 70 km/hr, it has been shown that water on the pavement can cause hydroplaning.
- Projected traffic volumes are an indicator of the economic importance of keeping the road open to traffic. The costs of traffic delays and accidents increase with increasing traffic volumes.

- Intensity of rainfall events may significantly affect the selection of design frequency and spread. Risks associated with the spread of water on pavements may be more ***in arid areas*** which are subject to high intensity thunderstorm events than in areas accustomed to frequent but less intense events.
- Capital costs are neither the least nor last consideration. Cost considerations make it necessary to formulate a rational approach to the selection of design criteria. "Tradeoffs" between desirable and practicable criteria are sometimes necessary because of costs. In particular, the costs and feasibility of providing for a given design frequency and spread may vary significantly between projects. In some cases, it may be practicable to significantly upgrade the drainage design and reduce risks at moderate costs. In other instances, such as where extensive outfalls or pumping stations are required, costs may be very sensitive to the criteria selected for use in design.

Other considerations include inconvenience, hazards, and nuisances to pedestrian traffic. These considerations should not be minimized and, in some locations such as in commercial areas, may assume major importance. Local design practice may also be a major consideration since it can affect the feasibility of designing to higher standards, and it influences the public's perception of acceptable practice.

Recommended design frequency for depressed sections and underpasses where ponded water can be removed only through the storm drainage system is a 50-year frequency event. The use of a lesser frequency event, such as a 100-year storm, to assess hazards at critical locations where water can pond to appreciable depths is commonly referred to as a "check storm" or "check event".

Table 10-1: Design Frequency and Spread vs. Geometric Design Standard

Geometric Design Standard	Design Frequency (Yrs)	Design Spread
DS1 EW[15]	10-year	Shoulder + 1 meter
At sag point	50-year	Shoulder + 1 meter
DS2	5-year	Shoulder
At sag point	10-year	1/2 driving lane
DS3/DS4/DS5	2-year	1/2 driving lane
At sag point	2-year	1/2 driving lane

10.8.3 Selection of Check Storm and Spread

A check storm should be used any time runoff could cause unacceptable flooding during less frequent events. Also, inlets should always be evaluated for a check storm when a series of inlets terminates at a sag vertical curve where ponding to hazardous depths could occur.

[15] Express way

The frequency selected for the check storm should be based on the same considerations used to select the design storm, i.e., the consequences of spread exceeding that chosen for design and the potential for ponding. Where no significant ponding can occur, check storms are normally unnecessary.

Criteria for spread during the check event are: (1) one lane open to traffic during the check storm event, and (2) one lane free of water during the check storm event. These criteria differ substantively, but each sets a standard by which the design can be evaluated.

10.9 Surface Drainage

When rain falls on a sloped pavement surface, it forms a thin film of water that increases in thickness as it flows to the edge of the pavement. Factors which influence the depth of water on the pavement are the length of flow path, surface texture, surface slope, and rainfall intensity. As the depth of water on the pavement increases, the potential for vehicular hydroplaning increases. For the purposes of road drainage, a discussion of hydroplaning is presented and design guidance for the following drainage elements is presented:

- Longitudinal pavement slope;
- Cross or transverse pavement slope;
- Curb and gutter design;
- Roadside and median ditches;
- Bridge decks.

10.9.1 Hydroplaning

As the depth of water flowing over a road surface increases, the potential for hydroplaning increases. When a rolling tire encounters a film of water on the road, the water is channelled through the tire tread pattern and through the surface roughness of the pavement.

Hydroplaning occurs when the drainage capacity of the tire tread pattern and the pavement surface is exceeded and the water begins to build up in front of the tire. As the water builds up, a water wedge is created and this wedge produces a hydrodynamic force which can lift the tire off the pavement surface. This is considered as full dynamic hydroplaning and, since water offers little shear resistance, the tire loses its tractive ability and the driver has a loss of control of the vehicle. Hydroplaning can occur at speeds of 89 km/hr with a water depth of 2mm (20).

Hydroplaning is a function of the water depth, road geometrics, vehicle speed, tread depth, tire inflation pressure, and conditions of the pavement surface .In problem areas, hydroplaning may be reduced by the following:

- Design the road geometries to reduce the drainage path lengths of the water flowing over the pavement. This will prevent flow build-up;
- Increase the pavement surface texture depth by such methods as grooving of Portland cement concrete. An increase of pavement surface texture will increase the drainage capacity at the tire pavement interface;
- Use of open graded asphaltic pavements has been shown to greatly reduce the hydroplaning potential of the road surface. This reduction is due to the ability of the water to be forced through the pavement under the tyre. This releases any hydrodynamic pressures that are created and reduces the potential for the tyre to hydroplane;

- Use of drainage structures along the road to capture the flow of water over the pavement will reduce the thickness of the film of water and reduce the hydroplaning potential of the road surface.

10.9.2 Longitudinal Slope

Guidance provided by the ERA Geometric Design Manual will provide safe, acceptable pavement drainage. In addition, the following general guidelines are presented (FHW Hydraulic Engineering Circular No. 22, Third Edition):

- A minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement since the water is constrained by the curb. However, flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.
- Desirable gutter grades should not be less than 0.5% for curbed pavements with an absolute minimum of 0.3%. Minimum grades can be maintained in very flat terrain by use of a rolling profile, or by warping the cross slope to achieve rolling gutter profiles.
- To provide adequate drainage in sag vertical curves, a minimum slope of 0.3% should be maintained within 15 meters of the low point of the curve. This is accomplished where the length of the curve in meters divided by the algebraic difference in grades in percent (K) is equal to or less than 50. This is represented as:

$$K = \frac{L}{(G_2 G_1)} \qquad (10\text{-}1)$$

K = Vertical curve constant m/percent
L = Horizontal length of curve, m
G i = Grade of road, percent

10.9.3 Cross (Transverse) Slope

Table 10.2 indicates an acceptable range of cross slopes as specified in AASHTO's policy on geometric design of roads and streets (21). These cross slopes are a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort and safety. These cross slopes represent standard practice. ERA should be consulted be consulted before deviating from these values.

Table 10-2: Normal Pavement Cross slopes

Surface Type	Range in Rate of Surface Slope m/m
High type surface	
2-lanes	0.015 – 0.020
3 or more lanes, each direction	0.015 minimum; increased 0.005 to 0.010 per lane; 0.04 maximum
Intermediate surface	0.015 – 0.03
Low-type surface	0.02 – 0.06
Shoulders	
Bituminous or concrete	0.02 – 0.06
With Curbs	≥0.04

As reported in Pavement and Geometric Design Criteria for Minimizing Hydroplaning, (22) cross slopes of 2% have little effect on driver effort in steering or on friction demand for vehicle stability. Use of a cross slope steeper than 2% on pavements with a central crown line is not desirable. In areas of intense rainfall, a somewhat steeper cross slope (2.5%) may be used to facilitate drainage.

On multi-lane roads where three (3) lanes or more are sloped in the same direction, it is desirable to counter the resulting increase in flow depth by increasing the cross slope of the outermost lanes. The two (2) lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs or portions thereof outward, should be increased by about 0.5 to 1%. The maximum pavement cross slope should be limited to 4% (Table 10.2).

Additional guidelines related to cross slope are:

- Although not widely encouraged, inside lanes can be sloped toward the median if conditions warrant;
- Median areas should not be drained across travel lanes;
- Number and length of flat pavement sections in cross slope transition areas should be minimized. Consideration should be given to increasing cross slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades; and
- Shoulders should be sloped to drain away from the pavement, except with raised, narrow medians and super elevations.

10.9.4 Curb and Gutter

Curbs are normally used at the outside edge of pavements for low-speed, road facilities, and in some instances adjacent to shoulders on moderate to high-speed facilities. They serve the following purposes:

- Contain the surface runoff within the road and away from adjacent properties;
- Prevent erosion on fill slopes;
- Provide pavement delineation; and
- Enable the orderly development of property adjacent to the road.

Gutters formed in combination with curbs are available in 0.3 through 1.0 meter widths. Gutter cross slopes may be the same as that of the pavement or may be designed with a steeper cross slope, usually 80 mm per meter steeper than the shoulder. AASHTO geometric guidelines state that an 8% slope is a common maximum cross slope.

A curb and gutter combination forms a triangular channel that can convey runoff equal to or less than the design flow without interruption of the traffic. When a design flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also shoulders, and portions of the travelled surface.

Spread is what concerns the hydraulic engineer in curbs and gutter flow. The distance of the spread, T, is measured perpendicular to the curb face to the extent of the water on the road and is shown in Figure 10.7.

Where practical, runoff from cut slopes and other areas draining toward the road should be intercepted before it reaches the road. By doing so, the deposition of sediment and other debris on the road as well as the amount of water which must be carried in the gutter section will be minimized. Where curbs are not needed for traffic control, shallow ditch sections at the edge of the road pavement or shoulder offer advantages over curbed sections by providing less of a hazard to traffic than a near-vertical curb and by providing

hydraulic capacity that is not dependent on spread on the pavement. These ditch sections are particularly appropriate where curbs have historically been used to prevent water from eroding fill slopes.

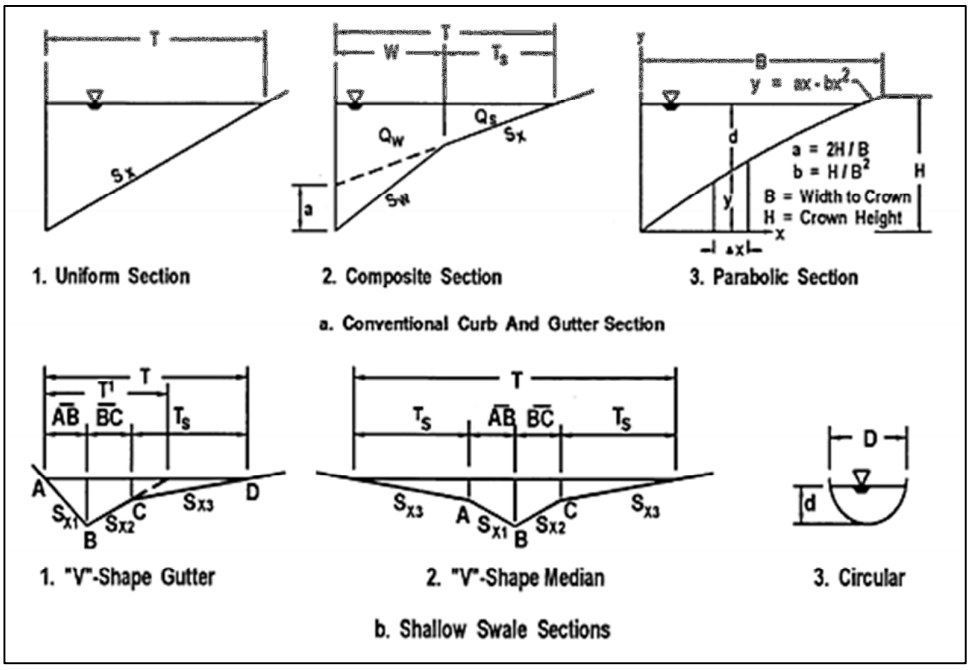


Figure 10-7: Typical Gutter Section
(Source: FHW Hydraulic Engineering Circular No. 22)

10.9.5 Roadside and Median Channels

Roadside channels are commonly used with uncurbed road sections to convey runoff from the road pavement and from areas which drain toward the road. Due to right-of-way limitations, roadside channels cannot be used on most urban areas. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent.

To prevent drainage from the median areas from running across the travel lanes, slope median areas and inside shoulders to a centre swale. This design is particularly important for high speed facilities and for facilities with more than two lanes of traffic in each direction.

10.9.6 Bridge Decks

Bridge deck drainage is similar to that of curbed road sections. Effective bridge deck drainage is important for the following reasons:

- Deck structural and reinforcing steel is susceptible to corrosion from deicing salts;
- Moisture on bridge decks freezes before surface roads; and
- Hydroplaning often occurs at shallower depths on bridges due to the reduced surface texture of concrete bridge decks.

Bridge deck drainage is often less efficient than road sections because cross slopes are flatter, parapets collect large amounts of debris, and drainage inlets or typical bridge scuppers are less hydraulically efficient and more easily clogged by debris. Because of the

difficulties in providing for and maintaining adequate deck drainage systems, gutter flow from roads should be intercepted before it reaches a bridge. For similar reasons, zero gradients and sag vertical curves should be avoided on bridges. Additionally, runoff from bridge decks should be collected immediately after it flows onto the subsequent road section where larger grates and inlet structures can be used. A detailed coverage of bridge deck drainage systems is included in Reference 23.



Photo 10-6: Example of Bridge Deck Drainage Problem (Quarit Jiga Road, Gojjam)

10.10 Flow in Gutters

A pavement gutter is defined, for purposes of this circular, as a section of pavement adjacent to the road which conveys water during a storm runoff event. It may include a portion or all of a travel lane. Gutter sections can be categorized as conventional or shallow swale type as illustrated in Figure 10.7. Conventional curb and gutter sections usually have a triangular shape with the curb forming the near-vertical leg of the triangle. Conventional gutters may have a straight cross slope (Figure 10.7, a.1), a composite cross slope where the gutter slope varies from the pavement cross slope (Figure 10.7, a.2), or a parabolic section (Figure 10.7, a.3). Shallow swale gutters typically have V-shaped or circular sections as illustrated in Figure 10.7, b.1, b.2, and b.3, respectively, and are often used in paved median areas on roads with inverted crowns.

10.10.1 Capacity Relationship

Gutter flow calculations are necessary to establish the spread of water on the shoulder and pavement section. A modification of the Manning's equation can be used for computing flow in triangular channels. The modification is necessary because the hydraulic radius in the equation does not adequately describe the gutter cross section, particularly where the top width of the water surface may be more than 40 times the depth at the curb. To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. (24) The resulting equation is:

$$Q = \left(\frac{0.377}{n}\right) * {S_x}^{5/3} * S^{1/2} * T^{8/3} \qquad (10\text{-}2)$$

Where:

Q = gutter flow rate, m3/s
n = Manning's roughness coefficient
S_x = pavement cross slope, m/m
S = longitudinal slope, m/m
T = width of flow or spread, m

A nomograph for solving equation 10.2 is given in Appendix 10A Figure 10A-1; Figure 10A-3 could also be used. Manning's n values for various pavement surfaces are presented in Table 10-3.

Table 10-3: Manning n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement: Smooth texture	0.013
Asphalt pavement: Rough texture	0.016
Concrete gutter with asphalt pavement: Smooth	0.013
Concrete gutter with asphalt pavement: Rough	0.015
Concrete pavement: Float finish	0.014
Concrete pavement: Broom finish	0.016

For gutters with small slope, where sediment may accumulate, increase above values of n by 0.002. Ref. FHWA, HDS-3 (1961).

10.10.2 Conventional Curb and Gutter Sections

Conventional gutters begin at the inside base of the curb and usually extend from the curb face toward the road centreline a distance of 0.3 to 1 meter. As illustrated in Figure 10.7, gutters can have uniform, composite, or curved sections. Uniform gutter sections have a cross-slope which is equal to the cross-slope of the shoulder or travel lane adjacent to the gutter. Gutters having composite sections are depressed in relation to the adjacent pavement slope. That is, the paved gutter has a cross-slope which is steeper than that of the adjacent pavement. This concept is illustrated in Example 10.1. Procedures for computing the capacity of curb and gutter sections follows.

Conventional Gutters of Uniform Cross Slope

The nomograph in Figure 10A-1, in Appendix 10A is used with the following procedures to find gutter capacity for uniform cross slopes:

Example 10.1

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n.
Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.
Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.
Step 2 Draw a line between the S and Sx scales and note where it intersects turning line.
Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.
Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

Composite Gutter Sections

The design of composite gutter sections requires consideration of flow in the depressed segment of the gutter, Q_w . Figure 10A-2, Appendix 10A can be used to find the flow in a gutter section with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of gutter capacity above the depressed section (Q_s). Example: S = 0.01; S_x = 0.02; S_w = 0.06; W = 0.6m; n = 0.016; Q = 0.057 m^3/s; try Q_s = 0.020 m^3/s.
Step 2 Calculate the gutter flow in W (Q_w), using the equation:
$Q_w = Q - Q_s$

$$Q_w = 0.057 - 0.020 = 0.037 m^3/s \quad (10\text{-}3)$$

Step 3 Calculate the ratios Q_w/Q and S_w/S_x and use Figure 10-2 to find an appropriate value of W/T. (Q_w/Q = 0.037/0.057 = 0.65; S_w/S_x = 0.06/0.02 = 3 W/T = 0.27)
Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3. (T = 0.6/0.27 = 2.22 m)
Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4. (T_s = 2.22 – 0.6 = 1.62 m)
Step 6 Use the value of T_s from Step 5 along with Manning's n, S_w and S_x to find the actual value of Q_s from Figure 10-1. (Q_s = 0.014 m^3/s)
Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

Q_s = 0.014 vs. 0.020 = "no good." Try Q_s = 0.023; then 0.057-0.023 – 0.034; and 0.034/0.057 = 0.6; From Figure 10-2 W/T = 0.23, then T = 0.6/0.23 = 2.61m and T_s = 2.61 – 0.6 = 2.01m. From Figure 10-1, Q_s = 0.023 m^3/s OK. ANSWER: Spread T = 2.61m

Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n, and depth of gutter flow (d). Example: Allowable spread T = 3.05m; W = 0.6m; T_s = 3.05 – 0.6 = 2.44 m; S_x = 0.04; S = 0.005 m/m; S_w = 0.06; n = 0.016; d = 0.13m

Step 2 Use Figure 10-1 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes- Condition 2, substituting T_s for T. From Figure 10-1, Q_s = 0.085 m^3/s

Step 3 Calculate the ratios W/T and S_w/S_x, and, from Figure 10-2, find the appropriate value of E_o. (the ratio of Q_w/Q).(W/T = 0.6/3.05 = 0.2; S_w/S_x = 0.06/0.04 = 1.5; From Figure 10-1, E_o = 0.46

Step 4 Calculate the total gutter flow using the equation:

$$Q = Q_s(1 - E_o) \quad (10\text{-}4)$$

Where:
Q = gutter flow rate, m^3/s
Q_s = flow capacity of the gutter section above the depressed section, m^3/s
E_o = ratio of frontal flow to total gutter flow (Q_w/Q)
Q = 0.085/(1 – 0.46) = 0.157 m^3/s

Step 5 Calculate the gutter flow in width (W), using equation 10.3:

$$Q_w = Q - Q_s = 0.157 - 0.085 = 0.072 m^2/s \quad (10\text{-}5)$$

Note: Figure 10-3 can also be used to calculate the flow in a composite gutter section.

10.11 Drainage Inlet Design

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the road resulting in a hazard to the travelling public.

10.11.1 Inlet Types

Storm drain inlets are used to collect runoff and discharge it to an underground storm drainage system. Inlets are typically located in gutter sections, paved medians, and roadside and median ditches. Inlets used for the drainage of road surfaces can be divided into the following four classes:

1. Grate inlets;
2. Curb-opening inlets;
3. Slotted inlets; and
4. Combination inlets.

Grate inlets consist of an opening in the gutter or ditch covered by a grate. Curb-opening inlets are vertical openings in the curb covered by a top slab. Slotted inlets consist of a pipe cut along the longitudinal axis with bars perpendicular to the opening to maintain the slotted opening. Combination inlets consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part

upstream of the grate. Figure 10.8 illustrates each class of inlets. Slotted drains may also be used with grates and each type of inlet may be installed with or without a depression of the gutter.

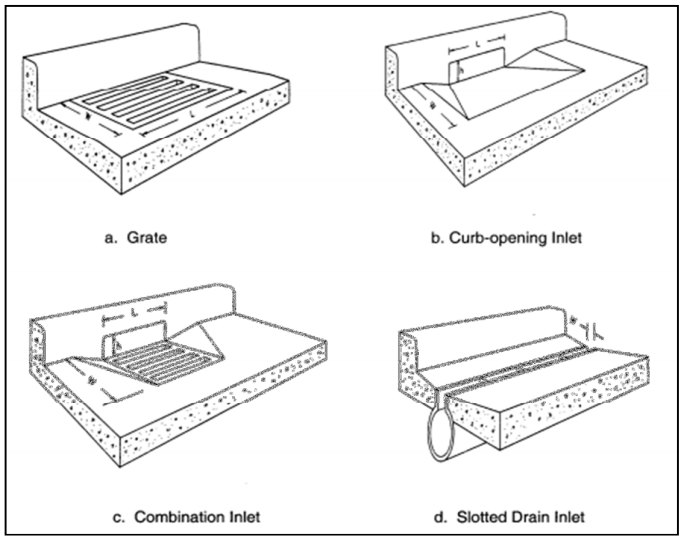


Figure 10-8: Classes of Storm Drain Inlets

10.11.2 Characteristics and Uses of Inlets

Grate inlets, as a class, perform satisfactorily over a wide range of gutter grades. Grate inlets generally lose capacity with increase in grade, but to a lesser degree than curb opening inlets. The principal advantage of grate inlets is that they are installed along the road where the water is flowing.

Their principal disadvantage is that they may be clogged by floating trash or debris. For safety reasons, preference should be given to grate inlets where out-of-control vehicles might be involved. Additionally, where bicycle traffic occurs, grates should be bicycle safe.



Photo 10-7: Vandalised and Clogged Grate Inlet (Addis Ababa, Samit area)

Curb-opening inlets are most effective on flatter slopes, in sags, and with flows which typically carry significant amounts of floating debris. The interception capacity of curb-opening inlets decreases as the gutter grade steepens. Consequently, the use of curb-opening inlets is recommended in sags and on grades less than 3%. Of course, they are bicycle safe as well.

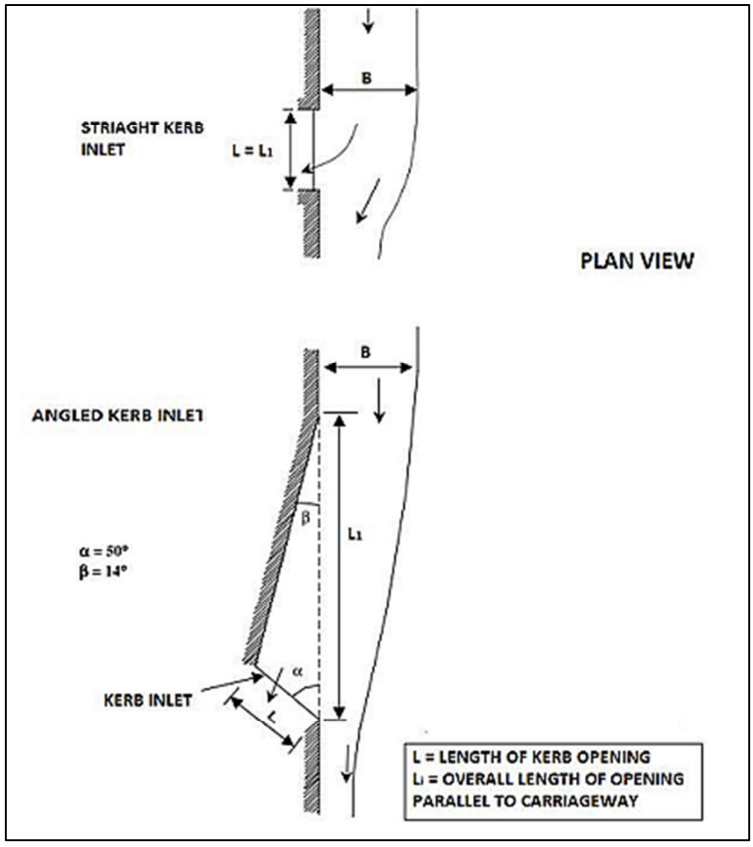


Figure 10-9: Layout of Kerb Inlets

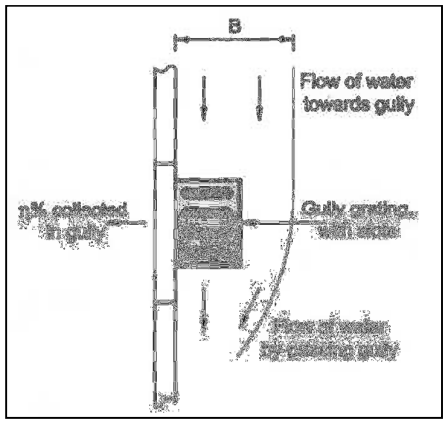


Figure 10-10: Flow of Water Along Kerb and Past Grating

Combination inlets provide the advantages of both curb opening and grate inlets. This combination results in a high capacity inlet which offers the advantages of both grate and curb-opening inlets. When the curb opening precedes the grate in a "Sweeper" configuration, the curb-opening inlet acts as a trash interceptor during the initial phases of a storm. Used in a sag configuration, the sweeper inlet can have a curb opening on both sides of the grate.

Slotted drain inlets can be used in areas where it is desirable to intercept sheet flow before it crosses onto a section of road. Their principal advantage is their ability to intercept flow over a wide section. However, slotted inlets are very susceptible to clogging from sediments and debris, and are not recommended for use in environments where significant sediment or debris loads may be present. Slotted inlets on a longitudinal grade do have the same hydraulic capacity as curb openings when debris is not a factor.

Factors Affecting Inlet Interception Capacity and Efficiency on Continuous Grades

Inlet interception capacity, Q_i, is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet, E, is the percent of total flow that the inlet will intercept for those conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and, to a lesser extent, pavement roughness. In mathematical form, efficiency, E, is defined by the following equation:

$$E = \frac{Q_1}{Q} \qquad (10\text{-}6)$$

where:

E = Inlet efficiency
Q = Total gutter flow, m^3
Q_i = Intercepted flow, m^3/s

Flow that is not intercepted by an inlet is termed carryover or bypass and is defined as follows:

$$Q_b = Q - Qi \qquad (10\text{-}7)$$

Where:

Q_b = Bypass flow, m^3/s

The interception capacity of all inlet configurations increases with increasing flow rates, and inlet efficiency generally decreases with increasing flow rates. Factors affecting gutter flow also affect inlet interception capacity. The depth of water next to the curb is the major factor in the interception capacity of both grate inlets and curb-opening inlets. The interception capacity of a grate inlet depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of flow in the gutter. The efficiency of a grate is dependent on the same factors and total flow in the gutter. Interception capacity of a curb-opening inlet is largely dependent on flow depth (see Figure 10.11) at the curb and curbs opening length. Flow depth at the curb and consequently, curb-opening inlet interception capacity an efficiency, is increased by the use of a local gutter depression at the curb-opening or a continuously depressed gutter to increase the proportion of the total flow adjacent to the curb.

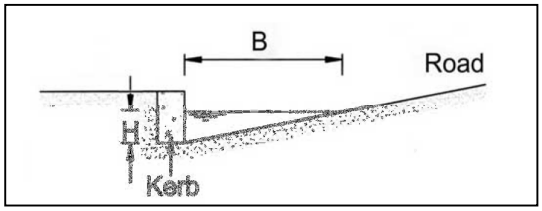


Figure 10-11: Depth of Water Against Curb

Top slab supports placed flush with the curb line can substantially reduce the interception capacity of curb openings. Tests have shown that such supports reduce the effectiveness of openings downstream of the support by as much as 50% and, if debris is caught at the support, interception by the downstream portion of the opening may be reduced to near zero. If intermediate top slab supports are used, they should be recessed several inches from the curb line and rounded in shape.

Slotted inlets function in essentially the same manner as curb opening inlets, i.e., as weirs with flow entering from the side. Interception capacity is dependent on flow depth and inlet length. Efficiency is dependent on flow depth, inlet length and total gutter flow.

The interception capacity of an equal length combination inlet consisting of a grate placed alongside a curb opening on a grade does not differ materially from that of a grate only. Interception capacity and efficiency are dependent on the same factors which affect grate capacity and efficiency. A combination inlet consisting of a curb-opening inlet placed upstream of a grate inlet has a capacity equal to that of the curb-opening length upstream of the grate plus that of the grate, taking into account the reduced spread and depth of flow over the grate because of the interception by the curb opening. This inlet configuration has the added advantage of intercepting debris that might otherwise clog the grate and deflect water away from the inlet.

Factors Affecting Inlet Interception Capacity in Sag Locations

Grate inlets in sag vertical curves operate as weirs for shallow ponding depths and as orifices at greater depths. Between weir and orifice flow depths, a transition from weir to orifice flow occurs. The perimeter and clear opening area of the grate and the depth of water at the curb affect inlet capacity. The capacity at a given depth can be severely affected if debris collects on the grate and reduces the effective perimeter or clear opening area.

Curb-opening inlets operate as weirs in sag vertical curve locations up to a ponding depth equal to the opening height. At depths above 1.4 times the opening height, the inlet operates as an orifice and between these depths, transition between weir and orifice flow occurs. The curb-opening height and length, and water depth at the curb affect inlet capacity. At a given flow rate, the effective water depth at the curb can be increased by the use of a continuously depressed gutter, by use of a locally depressed curb opening, or by use of an increased cross slope, thus decreasing the width of spread at the inlet.

Slotted inlets operate as weirs for depths below approximately 50 mm and orifices in locations where the depth at the upstream edge of the slot is greater than about 120 mm. Transition flow exists between these depths. For orifice flow, an empirical equation derived from experimental data can be used to compute interception capacity. Interception

capacity varies with flow depth, slope, width, and length at a given spread. Slotted drains are not recommended in sag locations because they are susceptible to clogging from debris.

10.11.3 Interception Capacity of Inlets on Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs (vane grates in many situations are efficient). Figure 10A-4, Appendix 10A shows splash-over velocities for various grate configurations, and the portion of frontal flow intercepted by the grate.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features.

They are efficient in debris handling as indicated in Table 10-3. In certain locations where leaves may create constant maintenance problems, the parallel bar grate may be used more efficiently if bicycle traffic is prohibited. Where debris is a problem, consideration shall be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was make to qualitatively simulate field conditions. Table 10-4 presents the results of debris handling efficiencies of several grates.

Table 10-4: Grate Debris Handling Efficiencies

Rank	Grate	Longitudinal Slope	
		(0.005)	(0.04)
1	CV - 3-1/4 - 4-1/	46	61
2	30 - 3-1/4 - 4	44	55
3	45 - 3-1/4 - 4	43	48
4	P - 1-7/8	32	32
5	P - 1-7/8 - 4	18	28
6	45 - 2-1/4 - 4	16	23
7	Recticuline	12	16
8	P - 1-1/8	9	20

(Source: "Drainage of Highway Pavements" (HEC-12), FHA, 1984)

The ratio of frontal flow to total gutter flow, E_o for straight cross slope is expressed by the following equation:

$$E_o = \frac{Q_w}{Q} = 1 - \left(\frac{1-W}{T}\right)^{2.67} \quad (10\text{-}8)$$

Where:

Q = total gutter flow, m^3/s
Q_w= flow in width W, m^3/s
W = width of depressed gutter or grate, m
T = total spread of water in the gutter, m

Figure 10A-2 provides a graphical solution of E_o for either straight cross slopes or depressed gutter sections. The ratio of side flow, Q,, to total gutter flow is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = I - E_o \quad (10\text{-}9)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.295(V - V_o) \quad (10\text{-}10)$$

Where:

V = velocity of flow in the gutter, m/s
V_o = gutter velocity where splash-over first occurs, m/s

This ratio is equivalent to frontal flow interception efficiency. Appendix 10 Figure 10A-4 provides a solution of equation 10.9 which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 10A-4 in appendix 10 is total gutter flow divided by the area of flow.

The ratio of side flow intercepted to total side flow, R., or side flow interception efficiency, is expressed by:

$$R_s = \frac{1}{1+\left(\frac{0.0828V^{1.8}}{S_x L^{2.3}}\right)} \quad (10\text{-}11)$$

Where:

V = velocity of flow in gutter, m/s
L = length of the grate, m
S_x = cross slope, m/m

Figure 10A-1 provides a solution to equation 10.11, with Q = AV.

The efficiency, E, of a grate is expressed as:

$$E = R_f E_o + R_s(1 - E_{o)} \quad (10\text{-}12)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\left[R_f E_o + R_s(1 - E_o)\right] \quad (10\text{-}13)$$

Example Problem

Given:

Drainage Area: 61 m single-family residential strip, Rational Method C=0.4, S=0.05 m/m

3.66 m Lane @ 0.015, 2.44 m shoulder at 0.04 and 0.61 m gutter at 0.06
10 year design, IDF Curve for Addis Ababa from Chapter 5, hydrology
Allowable spread T = 0.61 m, n = 0.016
S_o = 0.01, S_x = 0.04, S_w = 0.06
Use Curves & Nomographs

Find: Maximum allowable flow Q_T

Q_i intercepted by 0.61 m x 0.61 m vane grate
Q_r runby
Location of first and second inlets from crest of hill

Sketch:

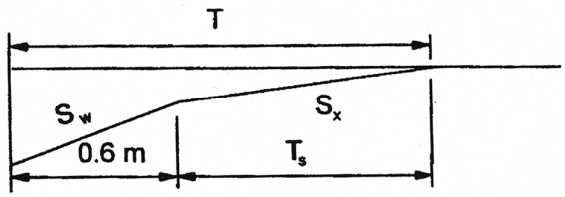


Figure 10-12: Sketch

Solution:

1. Solve for Q_s using Figure 10A-1

   T_o = 2.44 m; S_x = 0.04; Q_s = 0.11 m^3/s

2. Use Figure 10A-2 to find Eo

   S_w/S_x = 0.06/0.04 = 1.5 W/T = 0.61/3.05 = 0.2 E_o = 0.47 = Q_w/Q

3. Find total Q_T (maximum allowable flow)

   $Q_T = Q_s/(1-E_o)$ = 0.11/(1-1.47) = <u>0.21 m^3/s</u>

4. From Figure 10A-6 V = 1.19 m/s

   From Figure 10A-4 R_f = 1.0; From Figure 10A-7 R_s = 0.1

5. Using Equation 10-12

   $Q_i = Q_T [R_f E_o + R_s (1-E_o)]$

   Q_i = 0.21[(1.0 x 0.47) + 0.1(1-0.47)] = <u>0.11 m^3/s</u>

6. $Q_r = Q_T - Q_i$ Q_r = 0.21 – 0.11 = <u>0.10 m^3</u>

7. Locate first inlet from crest using inlet spacing equation 10-14

$$L = \frac{10000 * Q_t}{0.00278 * C * i * W} \quad (10\text{-}14)$$

Where:

L = distance from the crest, m

Q_t = maximum allowable flow, m^3/s

C = composite runoff coefficient for contributing drainage area, m

W = width of contributing drainage area, m

i = rainfall intensity for design frequency, mm/h

To find i, first solve for t_c; From Figure 5-3 residential area

C = 0.4, S = 0.5%, overland flow t_c = 22 min

Gutter flow estimated at V = 1.19 m/s from Figure 10-6

Try 213 m t_c = 213(1.19x60) = 3 min; Total t_c = 22 + 3 = 25 min

From Figure 5-10, i = 72 mm/h

Solve from weighted C value: C = [(61 x 0.4) + (7.32 x 0.9)]/68.32 = 0.454

L = 10000(0.21)/(0.002 78)(0.454)(72)(68.32) = 337 m OK

Place first inlet 337 m from crest

8 To locate second inlet:

Q_T = 0.21 m^3/s, Q_r = 0.10 m^3/s, $Q_{allowable}$ = 0.21 – 0.10 = 0.11 m^3/s

Assuming similar drainage area and t_c, i = 72 mm/h

L = 10000(0.11)(0.002 78)(0.454)(72)(68.32) = 177 m

Place second inlet 177 m from first inlet

10.11.4 Grate Inlets in Sag

A grate inlet in a sag operates as a weir up to a certain depth dependent on the bar configuration and size of the grate and as an orifice at greater depths. For standard gutter inlet grate, weir operation continues to a depth of about 0.12 meters above the top of grate and when depth of water exceeds about 0.4 meters, the grate begins to operate as an orifice. Between depths of about 0.12 meters and about 0.4 meters, a transition from weir to orifice flow occurs.

The capacity of a grate inlet operating as a weir is:

$$Q_i = CPd^{1.5} \quad (10\text{-}15)$$

where:

P = perimeter of grate excluding bar widths and the side against the curb, m
C = 1.66
d = depth of water at curb measured from the normal cross slope gutter flow line, m

The capacity of a grate inlet operating as an orifice is:

$$Q_i = CA(2gd)^{0.5} \quad (10\text{-}16)$$

Where:

C = 0.67 orifice coefficient
A = clear opening area of the grate, m^2
g = 9.81 m/s^2

Example:

The following example illustrates the use of Figure 10A-8.

Given: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.

Q_r = 0.10 m^3/s	Q = 0.23 m^3/s, design storm
Q_r = 0.12 m^3/s	Q = 0.31 m^3/s, check storm
T = 3.0 m, design	S_x = 0.05 m/m
d = TS_x = 0.15 m	n = 0.016

Find: Grate size for design Q and depth at curb for check Q. Check spread at S = 0.003 on approaches to the low point.

Solution: From Figure 10A.8, a grate must have a perimeter of 2.4 m to intercept 0.23 m^3/s at a depth of 0.15 m. Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50 percent covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50 percent. For example if a 0.6 m x 1.2 m grate is clogged so that the effective width is 0.3 m, then the perimeter, P = 0.3 + 1.2 + 0.3 = 1.8 m, rather than 2.4 m, the total perimeter, or 1.2 m, half of the total perimeter. The area of the opening would be reduced by 50 percent and the perimeter by 25 percent. Therefore, assuming 50 percent clogging along the length of the grate, a 1.2 x 1.2, a 0.6 x 1.8, or a 0.9 x 1.5 grate would meet requirements of a 2.4 m perimeter 50 percent clogged.

Assuming that the installation chosen to meet design conditions is a double 0.6 m x 0.9 m grate, for 50% clogged conditions:

P = 0.3 + 1.8 + 0.3 = 2.4 m

For design flow: d = 0.15 m (from Figure 10A-8)

For check flow: d = 0.18 m (from Figure 10A-8), T = 3.6 m

At the check flow rate, ponding will extend 0.6 m into a traffic lane if the grate is 50% clogged in the manner assumed.

Assume geometric policy recommends a gradient of 0.3% within 15 m of the level point in a sag vertical curve.

Check T at S = 0.003 for the design and check flow:

Q = 0.10 m^3/s, T = 2.5 m (design storm) Figure 10A-1 Appendix 10

Q = 0.12 m^3/s, T = 2.7 m (check storm) Figure 10A-1 Appendix 10

Thus a double 0.6 m x 0.9 m grate 50% clogged is adequate to intercept the design flow at a spread which does not exceed design spread and spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet, or curb-opening inlet in a sag where ponding can occur, and flanking inlets on the low gradient approaches.

10.11.5 Location

Inlet structures are located at the upstream end and at intermediate points along a storm drain line. Inlet spacing is controlled by the geometry of the site, inlet pening capacity, and tributary drainage magnitude. Inlet placement is generally a trial and error procedure that attempts to produce the most economical and hydraulically effective system.

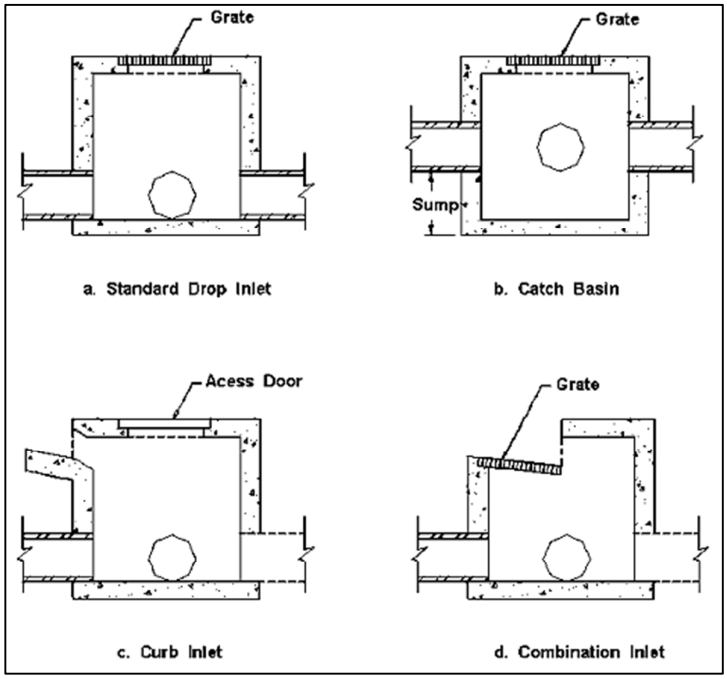


Figure 10-13: Inlet Structure

The following general rules apply to inlet placement:

- An inlet is required at the uppermost point in a gutter section where gutter capacity criteria are violated. This point is established by moving the inlet and thus changing the drainage area until the tributary flow equals the gutter capacity. Successive inlets are spaced by locating the point where the sum of the bypassing flow and the flow from the additional contributing area exceed the gutter capacity. Example in section 10.11.6 illustrates inlet spacing procedures.
- Inlets are normally used at intersections to prevent street cross flow which could cause pedestrian or vehicular hazards. It is desirable to intercept 100% of any potential street cross flow under these conditions. Intersection inlets should be placed on tangent curb sections near corners.
- Inlets are also required where the street cross slope begins to superelevate. The purpose of these inlets is also to reduce the traffic hazard from street cross flow. Sheet flow across the pavement at these locations is particularly susceptible to icing.
- Inlets should also be located at any point where side drainage enters streets and may overload gutter capacity. Where possible, these side drainage inlets should be located to intercept side drainage before it enters the street.
- Inlets should be placed at all low points in the gutter grade and at median breaks.
- Inlets are also used upstream of bridges to prevent pavement drainage from flowing onto the bridge decks and downstream of bridges to intercept drainage from the bridge.
- As a matter of general practice, inlets should not be located within driveway areas.

10.11.6 Curb Inlet on Grade

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or cyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} \quad (10\text{-}17)$$

Where:

K = 0.817

L_T = curb opening length required to intercept 100% of the gutter flow, m

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - \left(\frac{1-L}{L_T}\right)^{1.8} \quad (10\text{-}18)$$

Where:

L = curb-opening length, m

Figure 10a-9 is a nomograph for the solution of equation 10.18, and Figure 10a-10 provides a solution of equation 10.20.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in equation 10.19.

$$S_e = S_x + S'_w E_o \quad (10\text{-}19)$$

Where: S'_w = cross slope of the gutter measured from the cross slope of the pavement,

S'_w = (a/1000W), m/m

a = gutter depression, mm

E_o= ratio of flow in the depressed section to total gutter flow. It is determined by the gutter configuration upstream of the inlet.

Note: S_e can be used to calcuate the length of curb-opening by substituting S_e for S_x in equation 10.19.

Example:

The following example illustrates the use of this procedure.

Given: S_x = 0.03 m/m S = 0.035 m/m n = 0.016 Q = 0.14 m^3/s

Find:

(1) Q_i for 3.0 m curb-opening inlet, uniform cross slope
(2) Q_i for a depressed 3.0 m curb-open inlet with composite cross slope, a = 51 mm W = 0.6 m
(3) Q_i for a depressed 3.0 m curb-open inlet with uniform cross slope

Solution:

(1) From Figure 10A-1, T = 2.4 m

From Figure 10-9, L_T = 12.8 m

L/L_T = 3.0/12.8 = 0.23

From Figure 10A-10, E = 0.35

$Q_i = EQ = 0.35 \times 0.14 = \underline{0.049\ m^3/s}$

(2) $Q_n = 0.14 \times 0.016 = 0.0022\ m^3/s$

$S_w/S_x = (0.03 + 0.085)/0.03 = 3.83$

From Figure 10A-3, T/W = 3.5 and T = 2.1 m

Then W/T (Depress) = 0.6/2.1 = 0.29

From Figure 10A-2, E_o = 0.74

$S_e = S_x + S'_w E_o = 0.03 + 0.085(0.74) = 0.09$

From Figure 10A-9 L_T = 6.33 m, then L/L_T = 3/6.33 = 0.47

From Figure 10A-10 E = 0.69, then $Q_i = 0.69 \times 0.14 = \underline{0.097\ m^3/s}$

(3) $S_w/S_x = 0.03/0.03 = 1$

W/T = 0.6/2.4 = 0.25

From Figure 10A-2 E_o = 0.53

$S_e = 0.03 + (0.085)\ (0.53) = 0.075$

From Figure 10A-9 L_T = 7.6 m, then L/L_T = 3.0/7.6 = 0.4

From Figure 10A-10 E = 0.60, then $Q_i = 0.6 \times 0.14 = \underline{0.84\ m^3/s}$

10.11.7 Curb Inlets in Sag

The capacity of a curb-opening inlet in sag depends on water depth at the curb, the curb opening length and the height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

$$Q_I = C_w(L + 1.8W)d^{1.5} \qquad (10\text{-}20)$$

Where:

C_w = 1.27

L = length of curb opening, m

W = width of depression, m

d = depth of water at curb measured from normal cross slope gutter flow line, m

See Figure 10A-11 for a definition sketch.

The weir equation for curb-opening inlets without depression becomes:

$$Q_I = C_w L d^{1.5} \quad (10\text{-}21)$$

The depth limitation for operation as a weir becomes: d ≤ h

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 x height of curb-opening. The interception capacity can be computed by:

$$Q_i = C_o A \left[2g\left(\frac{d_I - h}{2}\right)\right]^{0.5} \quad (10\text{-}22)$$

Where:

C_o = orifice coefficient (0.67)

h = height of curb-opening orifice, m

A = clear area of opening, m^2

d_i = depth at lip of curb opening, m

Note: Equation 10.24 is applicable to depressed and undepressed curb-opening inlets and the depth at the inlet includes any gutter depression.

Example

The following example illustrates the use of this procedure.

Given: Curb-opening inlet in a sump location

L = 1.5 m h = 0.13 m

(1) Underpressed curb opening

S_x = 0.05 T = 2.4 m

(2) Depressed curb opening

S_x = 0.05 W = 0.6 m

a = 51 mm T = 2.4 m

Find: Q_i

Solution:

(1) d = TS_s = 2.4 x 0.05 = 0.12 m d < h therefore weir controls

$Q_I = C_w L d^{1.5}$ = 1.27 x 1.5 x $0.12^{1.5}$ = 0.079 m^3/s

(2) d = 0.12 m < (1.4 h) = 0.18 therefore weir controls

P = L + 1.8W = 1.5 + 1.8(0.6) = 2.58 m

Q_I = 1.27 x 2.58 x $0.12^{1.5}$ = 0.136 m^3/s (Figure 10A-11)

At d = 0.12 m, the depressed curb-opening inlet has about 70% more capacity than an inlet without depression. In practice, the flow rate would be known and the depth at the curb would be unknown.

10.11.8 Slotted Inlets on Grade

Slotted inlets are effective pavement drainage inlets which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. They can be placed longitudinally in the gutter or transversely to the gutter. Slotted inlets should generally be connected into inlet structures so they will be accessible to maintenance forces in case of plugging.

Longitudinal Placement

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Slotted inlets may have economic advantages in some cases and could be very useful on widening and safety projects where right of way is narrow and existing inlet capacity must be supplemented. In some cases, curbs can be eliminated as a result of utilizing slotted inlets.

The length of a slotted inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} \quad (10\text{-}23)$$

Where: K = 0.817

L_T = slotted inlet length required to intercept 100% of the gutter flow, m

The slot width must be at least 44 mm for equation 10.20 to be valid.

The efficiency of slotted inlets shorter than the length required for total interception is expressed by:

$$E = 1 - \left(\frac{1-L}{L_T}\right)^{1.8} \quad (10\text{-}24)$$

Where: L = slotted inlet length, m

Figure 10A-9 is a nomograph for the solution of equation 10.25, and Figure 10A-10 provides a solution of equation 10.26.

The length of inlet required for total interception by a slotted inlet in a composite section can be found by the use of an equivalent cross slope, S_e, in equation 10.25:

$$S_e = S_x + S'_w E_o \quad (10\text{-}25)$$

Where:

S_x = pavement cross slope, m/m
S_w = gutter cross slope, m/m
$S'_w = S_w - S_x$
E_o = ratio of flow in the depressed gutter to total gutter flow, Q_w/Q (See Fig 10A-2)

Note that the same equations are used for both slotted inlets and curb-opening inlets. The following example illustrates the use of this procedure.

Example

Give: Longitudinal placement of slotted inlet adjacent to curb.

S_o = 0.01 m/m Allowable spread = 3.0 m n = 0.016

(1) Uniform cross slope, S_x = 0.02

(2) Composite cross slope, $S_x = 0.02$ $S_w = 0.06$

(3) Increase S_o to 0.03 and solve for (1) and (2)

Find: (1) Maximum allowable Q

Q_i for a 3.0 m slotted inlet on straight cross slope.

(2) Maximum allowable Q

Q_i for a 3.0 m slotted inlet on composite cross slope.

(3) Same as above with profile grade increased to 0.03 m/m.

Solution:

(1) For T < 3.0 m, Max Q = 0.068 m^3/s from Figure 10-1

L_T = 8.2 m form Figure 10A-9 $L/L_T = 3.0/8.2 = 0.37$

E = 0.55 from Figure 10A-10; $Q_i = EQ = 0.55 \times 0.068 =$ 0.037 m^3/s intercepted

(2) $Q_s = 0.037$ m^3/s from Figure 10-1; $W/T = 0.6/3.0 = 0.2$

$S_w/S_x = 0.06/0.02 = 3$; $E_o = 0.52$ from Figure 10-2

Max $Q = Q_s/(1 - E_o) = 0.037/(1 - 0.52) =$ 0.077 m^3/s

$S'_w = S_w - S_x = 0.06 - 0.02 = 0.04$

$S_e = S_x + S'_w E_o = 0.02 + (0.04 \times 0.52) = 0.041$

L_T = 5.5 m from Figure 10-9 $L/L_T = 3.0/5.5 = 0.55$ E = 0.76 from Figure 10A-10; then $Q_i = EQ = 0.76 \times 0.077 =$ 0.059 m^3/s intercepted

The slotted inlet in the composite gutter section will intercept 1.6 times the flow intercepted by the slotted inlet in the uniform section.

(3) From a similar analysis for $S_o = 0.03$

Uniform Section: Max Q = 0.116 m^3/s and Q_i = 0.040 m^3/s

Composite Section: Max Q = 0.130 m^3/s and Q_i = 0.065 m^3/s

Transverse Placement

At locations where it is desirable to capture virtually all of the flow in the curbed section, a slotted vane drain can be installed in conjunction with a grate inlet. Tests have indicated that when the slotted vane drain is installed perpendicular to the flow, it will capture approximately 0.014 m^3/s per linear meter of drain on longitudinal slopes of 0% of 6%. Capacity curves are available from the manufacturer. The ideal installation would utilize a grate inlet to capture the flow in the gutter and the slotted vane drain to collect the flow extending into the shoulder. Note that a slotted vane drain is shaped and rounded to increase inlet efficiency and should not be confused with a standard vertical riser type slotted inlet.

10.11.9 Slotted Inlets in Sag

The use of slotted drain inlets in sag configurations is generally discouraged because of the propensity of such inlets to intercept debris in sags. However, there may be locations where it is desirable to supplement an existing low point inlet with the use of a slotted drain. Slotted inlets in sag locations perform as weirs to depths of about 0.06 m, dependent

on slot width and length. At depths greater than about 0.12 m, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8LW(2gd)^{0.5} \quad (10\text{-}26)$$

Where:

W = width of slot, m

L = length of slot, m

d = depth of water at slot,m

g = 9.81 m/s^2

For a slot width of 44 mm, the above equation becomes:

$$Q_i = 0.156Ld^{0.5} \quad (10\text{-}26a)$$

The interception capacity of slotted inlets at depths between 0.06 m and 0.12 m can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of slotted inlet. Figure 10A-12 provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

10.11.10 Flanking Inlets

At major sag points where significant ponding may occur, such as underpasses or sag vertical curves in depressed sections, it is recommended practice to place a minimum of one flanking inlet on each side of the inlet at the sag point. The flanking inlets shall be placed to act in relief of the sag inlet if it shall become clogged. Table 10-5 shows the spacing required for various depths at curb criteria and vertical curve lengths defined by K = L/A, where L is the length of the vertical curve in meters and A is the algebraic difference in approach grades. While the *Geometric Design Manual* specifies minimum K values for various design speeds, a maximum sag K of 74 shall be selected for curbed storm drainage.

Table 10-5: Flanking Inlet Locations

Distance In Flanking Inlet In Sag Vertical Curve Locations Using Depth at Curb Criteria (m)												
D↓ K→	6.1	9.1	12.2	15.2	21.3	27.4	33.5	39.6	48.8	50.9	54.9	67.1
0.03	6.1	7.3	8.5	9.8	11.3	12.8	14.3	15.5	17.4	17.7	18.3	20.1
0.06	8.5	10.7	12.2	13.7	16.2	18.3	20.1	21.9	24.4	25.0	25.9	28.7
0.09	10.7	12.8	14.9	16.8	19.8	22.3	24.7	26.8	29.9	30.5	31.7	35.1
0.12	12.2	14.9	17.4	19.2	22.9	25.9	28.7	31.1	34.4	35.4	36.6	40.5
0.15	13.7	16.8	19.2	21.6	25.6	29.0	32.0	34.7	38.4	39.3	40.8	45.1
0.18	14.9	18.3	21.0	23.5	28.0	31.7	35.1	38.1	42.4	43.3	44.8	49.4
0.21	16.2	19.8	22.9	25.6	30.2	34.1	37.8	41.1	45.7	46.6	48.5	53.6
0.24	17.4	21.0	24.4	27.1	32.3	36.6	40.5	43.9	48.8	49.7	51.8	57.3
NOTES:		1 X = (200dK)$^{0.5}$, where x = distance from the low point (m) 2 Drainage maximum K = 74m/% 3 D = depth at curb (m)										

Example

Given: K = 39.6 m/%, S_x = 0.04, allowable spread is 3.0 m

Find: Location of flanking inlets that will function in relief of the inlet at the low point when the inlet at the low point is clogged.

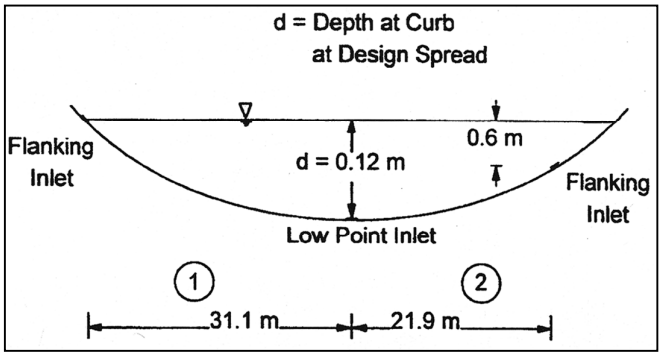


Figure 10-14: Flanking Inlets at Sag Point Example

Solution

(1) Depth over flanking inlet to carry half of the design flow equals 0.63 x 0.12 = 0.08m

(2) Depth from bottom of sag in flanking inlet – 0.12m – 0.08m = 0.04m

(3) Spacing of flanking inlet = 17.6m (from Table 10-5, using d = 0.04)

Inlet Spacing Computations

In order in design the location of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage areas, road profiles, and typical cross sections, grading cross sections, super elevation diagrams and contour maps are necessary. The inlet computation sheet, Form 10A-1, shall be used to document the computations. A step by step procedure is as follows:

Step 1 Complete the blanks on top of the sheet to identify the job by route, date and designer's initials.

Step 2 Mark on the plan the location of inlets which are necessary even without considering any specific drainage area (see Section 10.11 Inlets for additional information).

Step 3 Start at one end of the job, at one high point and work towards the low point, then space from the other high point back to the same low point.

Step 4 Select a trial drainage area approximately 90 to 150m below the high point and outline the area including any area that may come over the curb (use drainage area maps). Where practical, large areas of behind the curb drainage shall be intercepted before it reaches the highway (see Section 10.7 Pavement Drainage).

Step 5	Col 1 Col 2	Describe the location of the proposed inlet by number and station in columns 1 & 2. Identify the curb and gutter type in the Remarks Column 19. A sketch of the cross section shall be provided in the open area of the computation sheet.
Step 6	Col 3	Compute the drainage area in hectares and enter in Col 3.
Step 7	Col 4	Select a C value from one of the tables in Section 5.7 Rational Method or compute a weighted value based on area and cover type as described in Section 10.6 Hydrology and enter in Col 4.
Step 8	Col 5	Compute a time of concentration for the first inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Section 10.6 Hydrology and in *Chapter 5: Hydrology*. The minimum time of concentration shall be 7 min. Enter value in Col 5.
Step 9	Col 6	Using the Intensity-Duration-Frequency curves from *Chapter 5: Hydrology*, select rainfall intensity at the t_c for the design frequency. Enter in Col 6.
Step 10	Col 7	Calculate Q to by multiplying Col 3 x Col 4 x Col 6. Enter in Col 7.
Step 11	Col 8	Determine the gutter slope at the inlet from the profile grade – check effect of super elevation. Enter in Col 8.
Step 12	Col 9 Col 13	Enter cross slope adjacent to inlet in Col 9 and gutter width in Col 13. Sketch composites cross slope with dimensions.
Step 13	Col 11	For the first inlet in a series (high point to low point) enter Col. 7 in Col. 11 since no previous run by has occurred yet.
Step 14	Col 12 Col 14	Using Figure 10A-1 or the available computer model, determine the spread T and enter in Col. 14 and calculate the depth d at the curb by multiplying T times the cross slope(s) and enter in Col 12. Compare with the allowable spread as determined by the design criteria in Section 10.9. If col. 15 is less than the curb height and Col. 14 is near the allowable spread, continue on to step 16. If not OK, expand or decrease the drainage area to meet the criteria and repeat steps 5 through 14. Continue these repetitions until column 14 is near the allowable spread then proceed to step 15.
Step 15	Col 15	Calculate W/T and enter in Col 15.
Step 16	Col 16	Select the inlet type and dimensions and enter in Col 16.
Step 17	Col 17	Calculate the Q intercepted (Q_i) by the inlet and enter in Col 17. Utilize Fig. 10A-1 and 10A-2 or 10A-3 to define the flow in the gutter. Utilize Fig 10A-2, 10A-5 and 10A-6 and equation 10.11 to calculate Q_i for a grate inlet and Fig. 10A-9 and 10A-10 to calculate Q_i for a curb-opening inlet. See Section 10.12 for a grate inlet example and for a curb opening inlet example.

Step 18	Col 18	Calculate the runby by subtracting Col 17 from Col 11 and enter into Col 18 and also into Col. 10 on the next line if an additional inlet exists downstream.
Step 19	Col 1-4	Proceed to the next inlet down grade. Select an area approximately 90 to 120 m below the first inlet as a first trial. Repeat steps 5 through 7 considering only the area between the inlets.
Step 20	Col 5	Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.
Step 21	Col 6	Determine the intensity based on the time of concentration determined in step 19 and enter it in Col 6.
Step 22	Col 7	Determine the discharge from this area by multiplying Col 3 x Col 4 x Col 6. Enter the discharge in Col 7.
Step 23	Col 11	Determine total gutter flow by adding Col 7 and Col 10 and enter in Col 11. Column 10 is the same as Column 18 from the previous line.
Step 24	Col 12 Col 14	Determine "T" based on total gutter flow (Col. 11) by using Figure 10A-1 or 10A-3 and enter in Col 14 (if "T" in Col. 14 exceeds the allowable spread, reduce the area and repeat steps 19-24. If "T" in Col 14 is substantially less than the allowable spread, increase the area and repat steps 19-24).
Step 25	Col 16	Select inlet type and dimensions and enter in Col. 16
Step 26	Col 17	Determine Q_i and enter in Col 17 – See instruction in step 17.
Step 27	Col 18	Calculate the runby by subtracting Col 17 from Col 7 and enter in Col 16. This completes the spacing design for this inlet.

Step 28 Go back to step 19 and repeat step 19 through step 27 for each subsequent inlet. If the drainage area and weighted "C" values are similar between each inlet, the values from the previous grate location can be reused. If they are significantly different, recomputation will be required.

10.12 Access Holes

10.12.1 Location

Access holes are utilized to provide entry to continuous underground storm drains for inspection and cleanout. Some agencies utilize grate inlets in lieu of access holes, when entry to the system can be provided at the grate inlet, so that the benefit of extra stormwater interception can be achieved with minimal additional cost. Typical locations where access holes shall be specified are:

- Where two or more storm drains converge;
- At intermediate points along tangent sections;
- Where pipe size changes;
- Where an abrupt change in alignment occurs; and
- Where an abrupt change of the grade occurs.

Acces holes should not be located in traffic lanes; however, when it is impossible to avoid locating an access hole in a traffic lane, care shall be taken to insure it is not in the normal vehicle wheel path.

10.12.2 Spacing

The spacing of access holes shall be in accordance with the criteria in Table 10-6:

Table 10-6: Spacing of Access Holes

Size of Pipe (mm)	Maximum Spacing (m)
300 - 600	100
675 - 900	125
1050 - 1350	150
1500 - up	300

10.12.3 Types

An access hole with appropriate frame and lid shall be provided for the following storm drain sizes when the depth exceeds 1.2m:

- When the storm drain pipe diameter is 450mm or less, a 1200mm diameter access hole shall be provided.
- When the storm drain pipe diameter is 525mm to 1050mm inclusive, a 1500mm diameter access hole shall be provided.
- When the storm drain pipe diameter is 1200mm or larger, a 1800mm diameter access hole or a special design access hole shall be provided.

10.12.4 Sizing

When determining the minimum access hole size required for various pipe sizes and locations, two general criteria must be met:

- Access hole or inlet structure must be large enough to accept the maximum pipe as shown in Table 10-7;
- Knowing the relative locations of any two pipes, compute:

$$K = \frac{R_1+T_1+R_2+T_2+355mm}{\Delta} \quad (10\text{-}27)$$

Where:R_1 and T_1 are interior radius and wall thickness of Pipe #1 (mm), R_2 and T_2 are interior radius and wall thickness of Pipe #2 (mm), and Δ = angle between the pipes (degrees).

Table 10-7: Access Hole Sizing

Manhole Dia. (mm)	K (mm/°)	Maximum Pipe Size (mm)
675	6.10	375
1050	9.40	675
1200	10.67	750
1350	11.94	900
1500	13.21	1050
1650	14.73	1200
1800	16.00	1350
1950	17.27	1500
2100	18.54	1650
2250	20.07	1800
2400	21.34	1800
2550	22.61	1950
2700	23.88	2100

Example

Given: Pipe # 1 = 1350 mm Pipe # 2 = 1200 mm Δ = 140°

Solution:

$$K = \frac{675mm + 140mm + 600mm + 127mm + 355mm}{140°}$$

$$K = 13.55mm/°$$

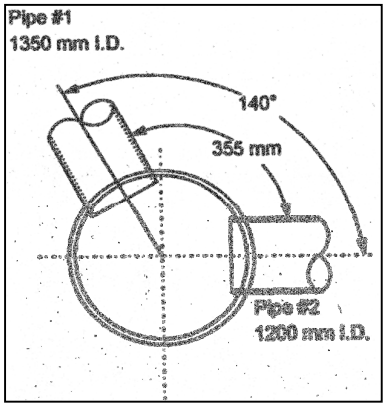


Figure 10-15: Manhole Sizing

The table indicates the minimum access hole barrel to be 1650mm. For the 1650mm AH barrel, the table indicates a maximum pipe size of 1200mm. However, as the maximum pipe size in example is 1250mm, an 1800mm AH must be used.

For this example, spacing is not critical and the pipe size governs. Had the Δ angle been 115^{o} or less, the spacing would be critical and a larger access hole barrel would have been required. If pipes are located at substantially different elevations, pipes may not conflict and the above analysis is unnecessary.

10.13 Storm Drains

After the preliminary locations of inlets, connecting pipes and outfalls with tailwaters have been determined, the next logical step is the computation of the rate of discharge to be carried by each reach of the storm drain, and the determination of the size and gradient of pipe required to convey this discharge.

This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream, reach by reach to the point where the storm drain connects with other drains or the outfall. At access holes where the pipe size is increased, the downstream crown shall be lower than the upstream crown by the amount of the energy loss in the access hole.

The rate of discharge at any point in the storm drain is not necessarily the sum of the inlet flow rates of all inlets above that section of storm drain. It is generally less than this total. The time of concentration is most influential and as the time of concentration grows larger, the rainfall intensity to be used in the design grows smaller. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger using the shorter time even though the entire drainage area is not contributing. The prodent designer will be alert for unusual conditions and determine which time of concentration controls for each pipe segment.

For ordinary conditions, storm drains shall be sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. The Manning's formula is recommended for capacity calculations. In locations such as depressed sections and underpasses where ponded water can be removed only through the storm drain system, a higher design frequency shall be considered to design the storm drain which drains the sag point. The main storm drain downstream of the depressed section shall be designed by computing the hydraulic grade line and keeping the water surface elevations below the grates and/or established critical elevations for the design storm.

10.13.1 Design Procedures

The design of storm drainage systems is generally divided into the following operations:

Step 1 Determine inlet location and spacing as outlined earlier in this chapter.

Step 2 Prepare plan layout of the storm drainage system establishing the following design data:

a Location of storm drains
b Direction of flow.
c Location of access holes.
d Location of existing utilities such as water, gas or underground cables.

Step 3 Determine drainage areas and runoff coefficients, and a time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve from *Chapter 5: Hydrology*, determine the rainfall intensity. Calcuate the discharge by multiplying 0.00278 x A x C x I.

Step 4 Size the pipe to convey the discharge by varying the slope and pipe size as necessary. The storm drain systems are normally designed for full gravity flow conditions using the design frequency discharges.

Step 5 Calculate travel time in the pipe to the next inlet or access hole by dividing pipe length by the velocity. This travel time is added to the time of concentration for a new time of concentration and new rainfall intensity at the next entry point.

Step 6 Calculate the new area (A) and multiply by the runoff coefficient (C), add to the previous (CA), multiply by 0.00278 and the new rainfall intensity to determine the new discharge. Determine the size of pipe and slope necessary to convey the discharge.

Step 7 Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

Step 8 Complete the design by calculating the hydraulic grade line as described in Section 10-14.

The design procedure should include the following:

- Storm drain design computation can be made on forms as illustrated in Form 10-2.
- All computations and design sheets shall be clearly identified. The engineer's initials and date of computations shall be shown on every sheet. Voided or superseded sheets shall be so marked. The origin of data used on one sheet but computed on another shall be given.

10.13.2 25-Year Sag Point

As indicated above, the storm drain which drains a major sag point shall be sized to accommodate the runoff from a 25-year frequency rainfall. This can be done by actually computing the runby occurring at each inlet during a 25-year rainfall and accumulating it at the sag point. The inlet at the sag point as well as the storm drain pipe leading from the sag point must be sized to accommodate this additional runby within the criteria established (see Section 10-9). In order to design the pipe leading from the sag point, it may be helpful to convert the additional runby created by the 25-year rainfall into an equivalent CA which can be added to the design CA. This equivalent CA can be approximated by dividing the 25-year runby by 0.002 78 x I_{10} in the pipe at the sagpoint.

Some designers may want to design separate systems in order to prevent the above-ground system from draining into the depressed area. This concept may be more costly but in some cases may be justified. Another method would be to design the upstream system for a 25-year design so as to minimize the run by to the sag point. Each case must be evaluated on its own merits and the impacts and risk of flooding a sag point assessed.

10.13.3 Hydraulic Capacity

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's formula and it is expressed by the following equation.

$$V = \left(\frac{1}{n}\right) R^{2/3} S^{1/2} \quad (10\text{-}28)$$

Where: V = mean velocity of flow, m/s
n = Manning's roughness coefficient
R = hydraulic radius, m = area of flow divided by the wetted perimeter (A/WP)
S = the slope of the energy grade line, m/m

In terms of discharge, the above formula becomes:

$$Q = VA = \left(\frac{1}{n}\right) AR^{2/3} S^{1/2} \quad (10\text{-}29)$$

Where: Q = rate of flow, m^3/s

A = cross sectional area of flow, m^2

For storm drains flowing full, the above equations become:

$$V = \left(\frac{0.397}{n}\right) D^{2/3} S^{1/2} \qquad Q = \left(\frac{0.312}{n}\right) D^{8/3} S^{1/2} \quad (10\text{-}30)$$

Where: D = diameter of pipe, m

The nomograph solutions of Mannings formula for full flow in circular storm drains is shown in Figures 10A-12 through 10A-14. Figure 10A -15, in Appendix 10A has been provided to assist in the solution of the Manning's equation for part full flow in storm drains.

10.13.4 Minimum Grades

All storm drains shall be designed such that velocities of flow will not be less than 0.9 m/s at design flow. For very flat grades the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system shall be checked to be sure there is sufficient velocity in all of the drains to deter settling of particles. Minimum slopes required for a velocity of 0.9 m/s can be calculated by rewriting Mannings formula as shown in equation 10.28 or by using values given in Table 10-8:

$$S = \frac{(nV)^2}{R^{4/3}} \quad (10.31)$$

Table 10-8: Minimum Slopes Necessary to Ensure 0.9 m/s in Storm Drains Flowing Full

		Minimum Slopes m/m					
Pipe size, mm	**Full Pipe, m^3/s**	**n = 0.012**		**n = 0.013**		**n = 0.024**	
200	0.030	0.006	4	0.007	5	0.025	6
250	0.046	0.004	8	0.005	6	0.019	0
300	0.067	0.003	7	0.004	4	0.014	9
375	0.104	0.002	8	0.003	2	0.011	1
450	0.150	0.002	2	0.002	6	0.008	7
525	0.204	0.001	8	0.002	1	0.007	1
600	0.267	0.001	5	0.001	7	0.005	9
675	0.338	0.001	3	0.001	5	0.005	1
750	0.417	0.001	1	0.001	3	0.004	4
825	0.505	0.000	97	0.001	1	0.003	9
900	0.601	0.000	80	0.001	0	0.003	4
1050	0.817	0.000	70	0.000	82	0.002	8
1200	1.067	0.000	59	0.000	69	0.002	3
1350	1.351	0.000	50	0.000	59	0.002	0
1500	1.668	0.000	44	0.000	51	0.001	7
1650	2.018	0.000	38	0.000	45	0.001	5
1800	2.402	0.000	34	0.000	40	0.001	4

10.13.5 Curved Alignment

Curved storm drains are permitted where necessary. Long radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 1200 mm and larger. Short radius bend sections are also available and can be utilized if there isn't room for the long radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Utilizing large access holes solely for changing direction may not be cost effective on large size storm drains.

10.14 Hydraulic Grade Line

The hydraulic grade line (HGL) is the last important feature to be established relating to the hydraulic design of storm drains. This gradeline aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating from a flood of design frequency.

In general, if the HGL is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. A special concern with storm drains designed to operate under pressure flow conditions is that inlet surcharging and possible access hole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition shall be made. Storm drain systems can often alternate between pressure and open channel flow conditions from one section to another.

The detailed methodology employed in calculating the HGL through the system begins at the system outfall with the tail water elevation. If the outfall is an existing storm drain system, the HGL calculation must begin at the outlet end of the existing system and proceed upstream through this in-place system, then upstream through the proposed system to the upstream inlet. The same considerations apply to the outlet of a storm drain as to the outlet of a culvert. See Figure 7-8 for a sketch of a culvert outlet which depicts the difference between the hydraulic grade line (HGL) and the energy grade line (EGL). Usually it is helpful to compute the EGL first, and then subtract the velocity head ($V^2/2g$) to obtain the HGL.

10.14.1 Tailwater

For most design applications, the tail water will either be above the crown of the outlet or can be considered to be between the crown and critical depth. To determine the EGL, begin with the tail water elevation or (dc + D)/2, whichever is higher, add the velocity head for full flow and proceed upstream to compute all losses such as exit losses, friction losses, junction losses, bend losses and entrance losses as appropriate.

An exception to the above might be a very large outfall with low tail water when a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the tail water, whichever is higher.

When estimating tail water depth on the receiving stream, the prudent designer will consider the joint or coincidental probability of two events occurring at the same time. For the case of a tributary stream or a storm drain, its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the receiving stream. A short duration storm which causes peak discharges on a small basin may not be critical for a larger basin. Also, if may safely be assumed that if the same storm causes peak discharges on both basins, the peaks will be out of phase. To aid in the evaluation of joint probabilities, refer to Table 10-9:

Table 10-9: Joint Probability Analysis

Area Ratio	Frequency for Coincidental Occurence			
	10-Year Design		100-Year Design	
	Main Stream	Tributary	Main Stream	Tributary
10 000 TO 1	1 10	10 1	2 100	100 2
1 000 TO 1	2 10	10 2	10 100	100 10
100 TO 1	5 10	10 5	25 100	100 25
10 TO 1	10 10	10 10	50 100	100 50
1 TO 1	10 10	10 10	100 100	100 100

(Source: USCE, Norfolk District, 1974)

10.14.2 Exit Loss

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as an endwall, the exit loss is:

$$H_o = 1.0\left[\frac{V^2}{2g} - \frac{V_d^2}{2g}\right] \quad (10.32)$$

Where: V = average outlet velocity, m/s

V_d = channel velocity downstream of outlet, m/s

Note that when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with moving water, the exit loss may be reduced to virtually zero.

10.14.3 Bend Loss

The bend loss coefficient for storm drain design is minor but can be evaluated using the formula:

$$h_b = 0.0033(\Delta)\left(\frac{{V_o}^2}{2g}\right) \quad (10\text{-}33)$$

Where: Δ = angle of curvature in degrees.

10.14.4 Pipe Friction Losses

The friction slope is the energy gradient in m/m for that run. The friction loss is simply the energy gradient multiplied by the length of the run in meters.

Energy losses from pipe friction may be determined by rewriting the Manning's equation with terms as previously defined:

$$S_f = \left(\frac{Qn}{AR^{2/3}}\right)^2 \quad (10\text{-}34)$$

The head losses due to friction may be determined by the formula:

$$H_f = S_f L \quad (10\text{-}35)$$

Manning's equation can also be written to determine friction losses for storm drains as follows:

$$H_f = \frac{6.36 n^2 V^2 L}{D^{4/3}} \quad (10\text{-}36)$$

$$H_f = \left(\frac{19.62 n^2 L}{R^{4/3}}\right)\left(\frac{V^2}{2g}\right) \quad (10\text{-}37)$$

Where: H_f = total head loss due to friction, m

n = Manning's roughness coefficient

D = diameter of pipe, m

L = length of pipe, m

V = mean velocity, m/s

R = hydraulic radius, m

g = 9.81 m/s^2

S_f = slope of hydraulic grade line, m/m

10.14.5 Access Hole Losses

The head loss encountered in going from one pipe to another through an access hole is commonly represented as being proportional to the veloicity head at the outlet pipe. Using K to signify this constant of proportionality, the energy loss is approximated as $K(V_o^2/2g)$. Expermental studies have determined that the K value can be approximated as follows:

$$K = K_O C_D C_d C_Q C_p C_B \quad (10\text{-}38)$$

Where: K = adjusted loss coefficient

K_o = Initial head loss coefficient based on relative access hole size

C_D = correction factor for pipe diameter (pressure flow only)

C_d = correction factor for flow depth (non-pressure flow only)

C_Q = correction factor for relative flow

C_n = correction factor for benching

C_p = correction factor for plunging flow

Relative Access Hole Size

K_o is estimated as a function of the relative access hole size and the angle of deflection between the inflow and outflow pipes. See Figure 10-16.

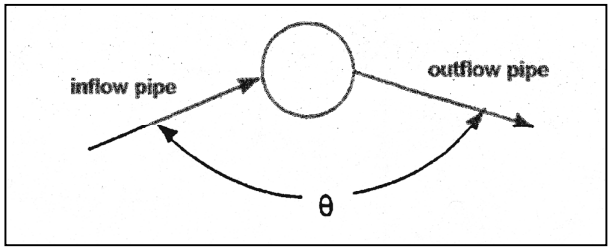


Figure 10-16: Deflection Angle

$$K_o = 0.1\left(\frac{b}{D_o}\right)(1 - sin\emptyset) + 1.4\left(\frac{b}{D_o}\right)^{0.15} sin\emptyset \quad (10\text{-}39)$$

Where: θ = the angle between the inflow and outflow pipes

b = access hole diameter, mm

D_o = outlet pipe diameter, mm

Pipe Diameter

A change in head loss due to differences in pipe diameter is only significant in pressure flow situations when the depth to the access hole to outlet pipe diameter ratio, d/D_o, is greater than 3.2. Therefore, it is only applied in such cases:

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \quad (10\text{-}40)$$

Where: D_i = incoming pipe diameter, mm

D_o = outgoing pipe diameter, mm

Flow Depth

The correction factor for flow depth is significant only in cases of free surface flow or low pressures, when d/D_o ratio is less than 3.2 and is only applied in such cases. Water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated by the following:

$$C_d = 0.5\left(\frac{d}{D_o}\right)^{0.6} \quad (10\text{-}41)$$

Where: d = water depth in access hole above outlet pipe, m

D_o = outlet pipe diameter, m

Relative Flow

The correction factor for relative flow, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed as follows:

$$C_Q = (1 - 2sin\emptyset)\left(\frac{(1-Q_i)^{0.75}}{Q_o}\right) + 1 \quad (10\text{-}42)$$

Where: C_Q = correction factor for relative flow

θ = the angle between the inflow and outflow pipes

Q_i = flow in the inflow pipe, m^3/s

Q_o = flow in the outlet pipe, m^3/s

As can be seen from the equation, C_o is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. To illustrate this effect, consider the access hole shown in Figure 10-17 and assume the following two cases to determine the impact of pipe 2 entering the access hole.

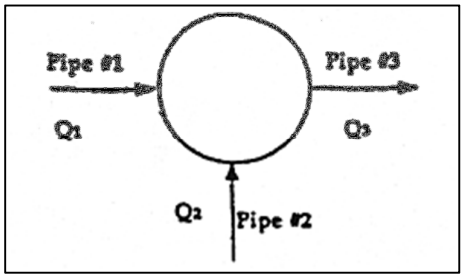


Figure 10-17: Relative Flow Effect

$C_{Q3\text{-}1} = (1\text{-}2 \sin 180^{O})(1 - 0.09/0.12)^{.75} + 1 = 1.35$

Case 1:

$Q_1 = 0.09\ m^3/s$ $Q_2 = 0.03\ m^3/s,$ $Q_3 = 0.12\ m^3/s$ then $C_o = 1.35$

Case 2:

$Q_1 = 0.03\ m^3/s,$ $Q_2 = 0.09\ m^3/s$ $Q_3 = 0.12\ m^3/s$ then $C_o = 1.81$

Plunging Flow

The correction factor for plunging flow, C_p, is calcuated by the following:

$$C_p = 1 + 0.2\left[\frac{h}{D_o}\right]\left[\frac{h-d}{D_o}\right] \quad (10\text{-}43)$$

Where: C_p = correction for plunging flow

h = vertical distance of plunging flow from flow line of incoming pipe to the center of outlet pipe, m

D_o = outlet pipe diameter, m

d = water depth in access hole, m

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the access hole, on the inflow pipe for which the head loss is being calculated. Using the notations in the above figure for the example, C_p is calculated for pipe # 1 when pipe # 2 discharges plunging flow. The correction factor is only applied when h > d.

Benching

The correction for benching in the access hole, C_B, is obtained form Table 10-10. Benching tends to direct flows through the access hole, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Table 10-10: Correction for Benching

Bench Type (see Figure 10-21)	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02
*pressure flow, $d/D_o > 3.2$ **free surface flow, $d/D_o < 1.0$		

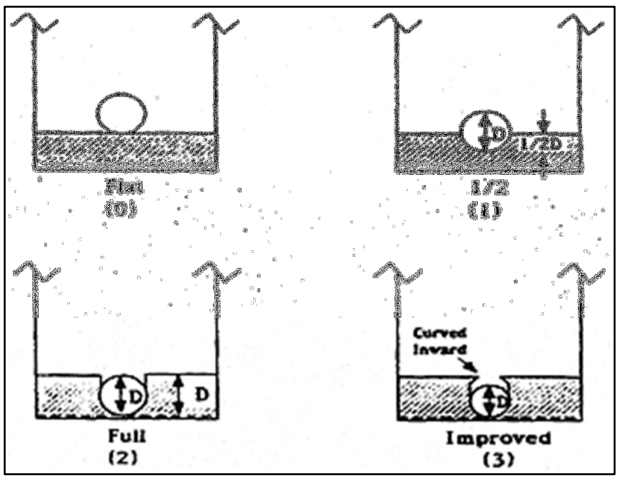


Figure 10-18: Schematic Representation of Benching Types

Summary

In summary, to estimate the head loss through an access hole from the outflow pipe to a particular inflow pipe, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

10.14.6 Hydraulic Grade Line Design Procedure

The equations and charts necessary to manually calculate the location of the hydraulic grade line are included in this chapter. A computer program is recommended for design of storm drains. For example, the HYDRA program will include a HGL analysis and a pressure flow simulation.

A step by step procedure is given to manually compute the HGL. Form 10-3 can be used to document the procedure.

If the HGL is above the pipe crown at the next upstream access hole, pressure flow calculations are indicated; if it is below the pipe crown, then open channel flow calculations shall be used at the upstream access hole. The process is repeated throughout the storm drain system. If all HGL elevations are acceptable, then the hydraulic design is

adequate. If the HGL exceeds an inlet elevation, then adjustments to the trial design must be made to lower the water surface elevation.

See Figure 10-19 for a sketch depicting the use of energy losses in developing a storm drain system.

Step 1 Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. HGL computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2 Enter in Col. 2 the tail water elevation if the outlet will be submerged during the design storm; otherwise refer to the tail water discussion earlier in this section for procedure.

Step 3 Enter in Col. 3 the diameter (D_o) of the outflow pipe.

Step 4 Enter in Col. 4 the design discharge (Q_o) for the outflow pipe.

Step 5 Enter in Col 5 the length, L_o, of the outflow pipe.

Step 6 Enter in Col. 6 the outlet velocity of flow, V_o

Step 7 Enter in Col 7 the velocity head, $V_{02}/2g$.

Step 8 Enter in Col. 8 the exit loss, H_o.

Step 9 Enter in Col. 9 the friction slope (SF_o) in m/m of the outflow pipe. This can be determined by using the equation 10.31. Note: Assumes full flow condtions.

Step 10 Enter in Col. 10 the fricntion loss (H_f) which is computed by multiplying the length (L_o) in Col. 5 by the friction slope (SF_o) in Col 9. On curved alignments, calculate curve losses by using the formula Hc = 0.0033 $(\Delta)(V_n^2/2g)$, where Δ = angle of curvature in degrees, and add to the friction loss.

Step 11 Enter in Col. 11 the initial head loss coefficient, K_o, based on relative access hole size as computed by equation 10.37.

Step 12 Enter in Col. 12 the correction factor for pipe diameter, C_D, as computed by equation 10.36.

Step 13 Enter in Col. 13 the correction factor for flow depth, C_d, as computed by equation 10.38. Note this factor is only significant in cases where the d/D_o ratio is less than 3.2.

Step 14 Enter in Col. 14 the correction factor for relative flow, C_Q, as computed by equation 10.39

Step 15 Enter in Col. 15 the correction factor for plunging flow, C_p, as computed by equation 10.40. The correction factor is only applied when h>d.

Step 16 Enter in Col. 16 the correction factor for benching, C_B, as determined in Table 10-10.

Step 17 Enter in Col. 17 the value of K as computed by equation 10.36.

Step 18 Enter in Col. 18 the value of the total access hole loss, $K V_o^2/2g$.

Step 19 If the tailwater submerges the outlet end of the pipe, enter in Col. 19 the sum of Col. 2 (TW elevation) and Col. 7 (exit loss) to get the EGL at the outlet end of the pipe. If the pipe is flowing full, but the tailwater is low, the EGL will be determined by adding the velocity head to $(d_c + D)/2$.

Step 20 Enter in Col. 20 the sum of the friction head (Col. 10), the access hole losses (Col 18), and the energy grade line (Col 19) at the outlet to obtain the EGL at the inlet end. This value becomes the EGL for the downstream end of the upstream pipe.

Step 21 Determine the HGL (Col. 21) throughout the system by subtracting the velocity head (Col. 7) from the EGL (Col. 20).

Step 22 Check to make certain that the HGL is below the level of allowable high water at that point. If the HGL is above the finished grade elevation; water will exit the system at this point for the design flow.

The above procedure applies to pipes that are flowing full, as shall be the condition for design of new systems. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

10.15 Inverted Siphons

An inverted siphon carries the flow under an obstruction such as sanitary sewers, water mains, or any other structure or utility lane which is in the path of the storm drain line. The storm drain invert is lowered at the obstacle and is raised again after the crossing. The inlet and outlet structures shall be designed by keeping the normal flow to provide the required minimum velocity for self-cleaning and servicing. The criteria for designing inverted siphons can be found in most of the hydraulics text books.

10.16 Under Drains

In certain areas ground water can be a significant problem as it attacks foundations, substructures, subgrades and other aspects of highway components. In most soils where ground water is a problem, a system of under drains, installed for the removal of excess moisture, can be a very useful feature in the overall roadway design. Under drains may take the form of networks of perforated (or otherwise permeable) pipe, French drains, or collector fields. Where such appurtenances are needed, the additional expense in their installation is usually fully justified in terms of future savings in roadway and structure maintenance costs.

Percolation rates for ground water may be obtained from the laboratory or may be simply estimated. Collector pipe sizes and networks may then be established for the removal of that water. French drains can be very useful where the unwanted ground water percolation rates are relatively high. Collector fields may be useful where reasonable outfalls for ground water are not available. All of the above appurtenances may be enhanced by the use of some type of geotextile filter material.

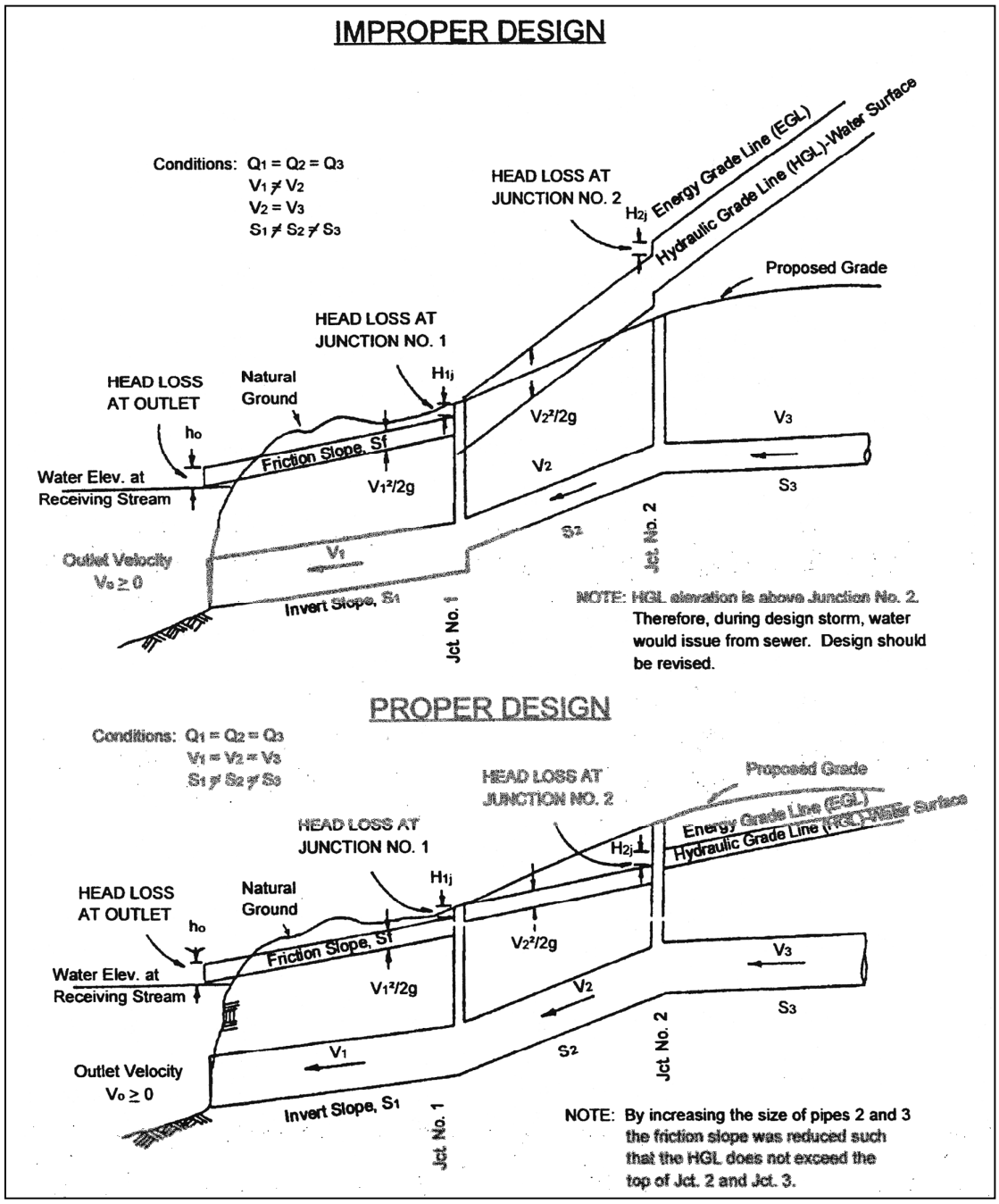


Figure 10-19: Use of Energy Losses in Developing a Storm Drain System

10.17 Computer Programs

To assist with storm drain system design, various microcomputer software modules have been developed for the computation of hydraulic grade line. For example, the computer program XPSTORM, Micro Drainage, Hydro CAD, HYDRA is part of the HYDRAIN system. XPSTORM can be used to check design adequacy and to analyze the performance of a storm drain system under assumed inflow conditions. Include ditch design procedure and recommend shape, type in comparison to various factors.

10.18 Detention and Retention Facilities

Land development activities, including the construction of roads, convert natural pervious areas to impervious and otherwise altered surfaces. These activities cause an increased volume of runoff because infiltration is reduced, the surface is usually smoother thereby allowing more rapid drainage, and depression storage is usually reduced. In addition, natural drainage systems are often replaced by lined channels, storm drains, and curb-and-gutter systems. These man-made systems produce an increase in runoff volume and peak discharge, as well as a reduction in the time to peak of the runoff hydrograph. This concept is illustrated by the hydrograph in Figure 10.20.

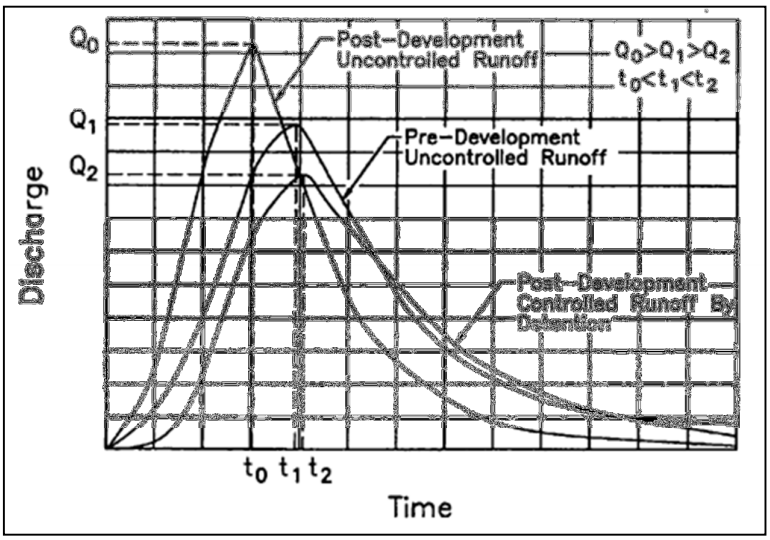


Figure 10-20: Hydrograph Schematics

The temporary storage or detention/retention of excess storm water runoff as a means of controlling the quantity and quality of storm water releases is a fundamental principle in storm water management and a necessary element of a growing number of road storm drainage systems. Previous concepts which called for the rapid removal of storm water runoff from developed areas, usually by downstream channelization, are now being combined with methods for storing storm water runoff to prevent overloading of existing downstream drainage systems. The storage of storm water can reduce the frequency and extent of downstream flooding, soil erosion, sedimentation, and water pollution.

Detention /retention facilities also have been used to reduce the costs of large storm drainage systems by reducing the required size for downstream storm drain conveyance systems. The use of detention/retention facilities can reduce the peak discharge from a given watershed, as shown in Figure 10.20. The reduced post-development runoff hydrograph is typically designed so that the peak flow is equal to or less than the pre-

developed runoff peak flow rate. Additionally, the volume of the post-development hydrograph is the same as the volume of the reduced post-development runoff hydrograph.

10.18.1 Design Objectives

One of the fundamental objectives of storm water management is to maintain the peak runoff rate from a developing area at or below the pre-development rate to control flooding, soil erosion, sedimentation, and pollution.

Specific design criteria for peak flow attenuation are typically established by local government bodies. Some jurisdictions also require that flow volume be controlled to pre-development levels as well. Controlling flow volume is only practical when site conditions permit infiltration.

To compensate for the increase in flow volume, some jurisdictions require that the peak post-development flow be reduced to below pre-development levels.

When storm water management first became common, most detention/retention facilities were designed for control of runoff from only a single storm frequency. Typically the 2-, 10-, or 100-year storms were selected as the controlling criteria. However, single storm criteria have been found to be rather ineffective since such a design may provide little control of other storms.

For example, design for the control of frequent storms (low return periods) provides little attenuation of less frequent but much larger storm events. Similarly, design for less frequent large storms provides little attenuation for the more frequent smaller storms.

10.18.2 Release Timing

The timing of releases from storm water control facilities can be critical to the proper functioning of overall storm water systems. As illustrated in Figure 10.20, storm water quantity control structures reduce the peak discharge and increase the duration of flow events. While this is the desired result for flow tributary to an individual storm water control facility, this shifting of flow peak times and durations in some instances can cause adverse effects downstream.

For example, where the drainage area being controlled is in a downstream portion of a larger watershed, delaying the peak and extending the recession limb of the hydrograph may result in a higher peak on the main channel. As illustrated in Figure 10.21, this can occur if the reduced peak on the controlled tributary watershed is delayed in such a way that it reaches the main stream at or near the time of its peak. On occasions, it has also been observed that in locations where multiple detention facilities have been installed within developing watersheds, downstream storm flooding problems continue to be noticed. In both of these cases the natural timing characteristics of the watershed are not being considered, and certainly are not being duplicated by the uncoordinated use of randomly located detention facilities. It is critical that release timing be considered in the analysis of storm water control facilities to ensure that the desired result is obtained.

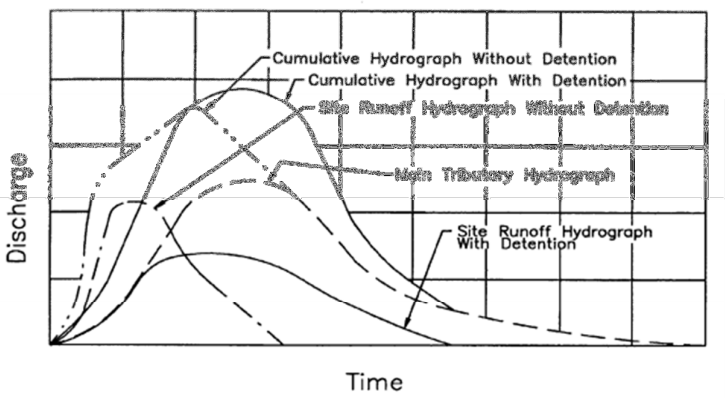


Figure 10-21: Example of Cumulative Hydrograph With and Without Detention

10.18.3 Estimating Required Storage

Estimating the required volume of storage to accomplish the necessary peak reduction is an important task since an accurate first estimate will reduce the number of trials involved in the routing procedure. The following sections present three (3) methods for determining an initial estimate of the storage required to provide a specific reduction in peak discharge. All of the methods presented provide preliminary estimates only. It is recommended that the designer apply several of the methods and a degree of judgment to determine the initial storage estimate.

Hydrograph Method

The hydrograph method of estimating the required volume of storage requires an inflow hydrograph and an outflow hydrograph. The storage required for the basin will be the volume difference between the two hydrographs. The inflow hydrograph will be the one established as the final runoff from the watershed flowing into the detention basin. The outflow hydrograph is unknown at the beginning of the process and is what the routing process will eventually establish. However, for the initial estimation of the needed storage, the outflow hydrograph must be estimated. It may be approximated by straight lines or by sketching an assumed outflow curve as shown on Figure 10.22. The peak of this estimated outflow hydrograph must not exceed the desired peak outflow from the detention basin. After this curve is established, the shaded area between the curves represents the estimated storage that must be provided.

To determine the necessary storage, the shaded area can be planimetered or computed mathematically by using a reasonable time period and appropriate hydrograph ordinates.

Triangular Hydrograph Method

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with standard triangular shapes. This method should not be applied if the hydrographs cannot be approximated by a triangular shape. This would introduce additional errors of the preliminary estimate of the required storage.

The procedure is illustrated by Figure 10.23. The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph as defined by Equation 10.44.

$$V_s = 0.5t_i(Q_i - Q_o) \quad (10\text{-}44)$$

Where:

V_s = Storage volume estimate, m^3

Q_i = Peak inflow rate into the basin, m^3/s

Q_o = Peak outflow rate out of the basin, m^3/s

t_i = Duration of basin inflow, s

t_o = Time to peak of the inflow hydrograph, s

The duration of basin inflow should be derived from the estimated inflow hydrograph. The triangular hydrograph procedure, originally described by Boyd, (47) was found to compare favourably with more complete design procedures involving reservoir routing.

SCS Procedure

The Soil Conservation Service (SCS), in its TR-55 Second Edition Report, (13) describe a manual method for estimating required storage volumes based on peak inflow and outflow rates. The method is based on average storage and routing effects observed for a large number of structures. A dimensionless figure relating the ratio of basin storage volume (Vs) to the inflow runoff volume (Vr) with the ratio of peak outflow (Qo) to peak inflow (Qi) was developed as illustrated in Figure 10.24. This procedure for estimating storage volume may have errors up to 25% and, therefore, should only be used for preliminary estimates.

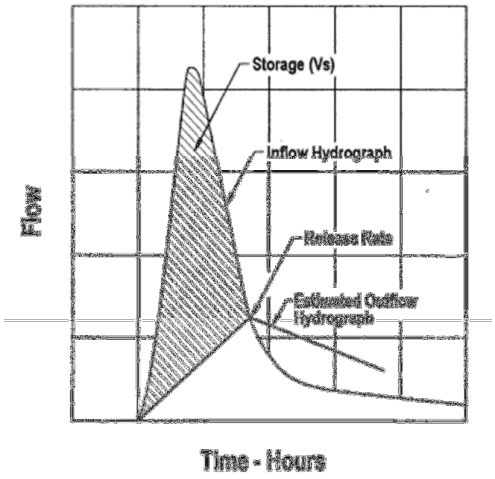


Figure 10-22: Estimating Required Storage Hydrograph Method

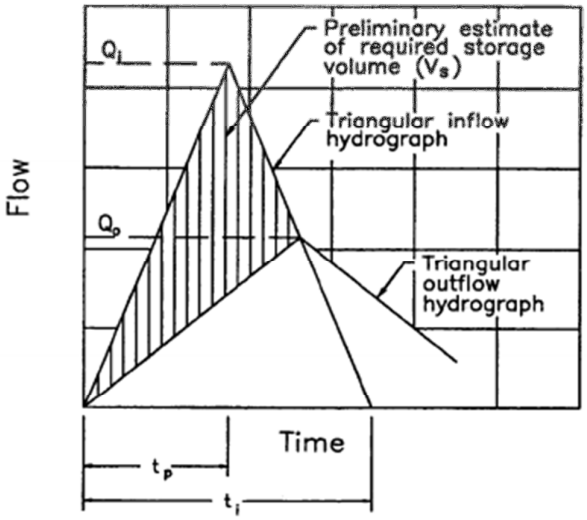


Figure 10-23: Triangular Hydrograph Method

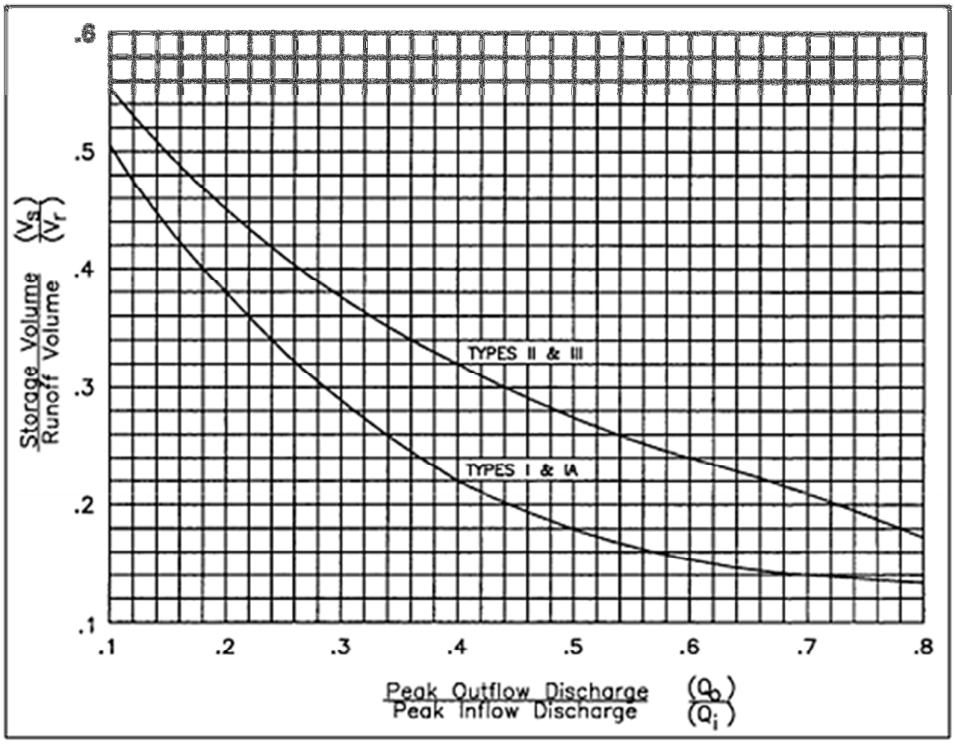


Figure 10-24: SCS Detention Basin Routing Curves

The procedure for using Figure 10.24 in estimating the detention storage required is described as follows: (13)

1 Determine the inflow and outflow discharges Q_i and Q_o

2. Compute the ratio Q_o / Q_i

3. Compute the inflow runoff volume, V_r , for the design storm

$$V_r = K_r QD\ A_m \quad (10\text{-}45)$$

Where:

V_r = Inflow volume of runoff, ha-mm

K_r = 1.00

QD = Depth of direct runoff, mm

A_m = Area of watershed, ha

4. Using Figure 10-24, determine the ratio Vs /Vr

5. Determine the storage volume, V_s as

$$V_s = V_r \left(\frac{V_s}{V_r}\right) \quad (10\text{-}46)$$

10.18.4 Stage-Storage Relationship

A stage-storage relationship defines the relationship between the depth of water and storage volume in the storage facility. The volume of storage can be calculated by using simple geometric formulas expressed as a function of storage depth. This relationship between storage volume and depth defines the stage-storage curve. A typical stage-storage curve is illustrated in Figure 10.25. After the required storage has been estimated, the configuration of the storage basin must be determined so that the stage-storage curve can be developed. The following relationships can be used for computing the volumes at specific depths of geometric shapes commonly used in detention facilities.

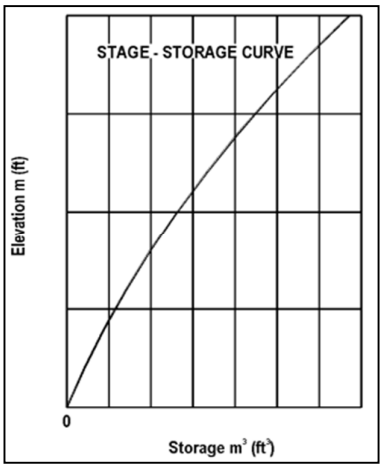


Figure 10-25: Stage–Storage Curve

10.18.5 Orifices

For a single orifice as illustrated in Figure 10.26 (a), orifice flow can be determined using Equation 10.47

$$Q = C_o A_o (2g\ H_o)^{0.5} \quad (10\text{-}47)$$

Where:

Q = Orifice flow rate, m^3/s

C_o = Discharge coefficient (0.40 – 0.60)

A_o = Area of orifice, m^2

H_o = Effective head on the orifice measured from the centroid of the opening, m

g = Gravitational acceleration, 9.81 m/s^2

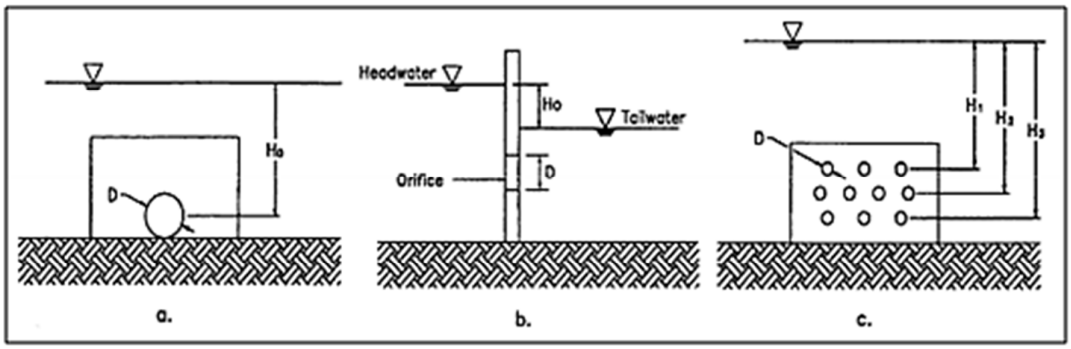


Figure 10-26 : Definition Sketch for Orifice Flow

If the orifice discharges as a free outfall, then the effective head is measured from the centerline of the orifice to the upstream water surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the upstream and downstream water surfaces. This latter condition of a submerged discharge is shown in Figure 10.26(b).

For square-edged, uniform orifice entrance conditions, a discharge coefficient of 0.6 should be used. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, a value of 0.4 should be used.

For circular orifices with Co set equal to 0.6, the following equation results:

$$Q = K_{or} D^2 {H_o}^{0.50} \quad (10\text{-}48)$$

Where:

K_{or} = 2.09 in SI units

D = Orifice diameter, m

Pipes smaller than 0.3 m in diameter may be analyzed as a submerged orifice as long as H_o/D is greater than 1.5. Pipes greater than 0.3 m in diameter should be analyzed as a discharge pipe with headwater and tail water effects taken into account, not just as an orifice.

Flow through multiple orifices (see Figure 10.26(c)) can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

10.18.6 Weirs

Relationships for sharp-crested, broad-crested, V-notch, and proportional weirs are provided in the following sections:

Sharp Crested Weirs

Typical sharp crested weirs are illustrated in Figure 8-13. Equation 8-19 provides the discharge relationship for sharp crested weirs with no end contractions (illustrated in Figure 8-13a).

$$Q = C_{scw} L H^{1.5} \quad (10\text{-}49)$$

Where:

Q = Discharge, m 3 /s
L = Horizontal weir length, m
H = Head above weir crest excluding velocity head, m
C_{scw} = 1.81 + 0.22 (H/H_c)

As indicated above, the value of the coefficient C_{scw} is known to vary with the ratio H/H_c (see Figure 10.27c for definition of terms). For values of the ratio H/H_c less than 0.3, a constant C_{scw} of 1.84 is often used. Equation 10.50 provides the discharge equation for sharp-crested weirs with end contractions (illustrated in Figure 10.27(b)).

$$Q = C_{scw}(L - 0.2H)H^{1.5} \quad (10\text{-}50)$$

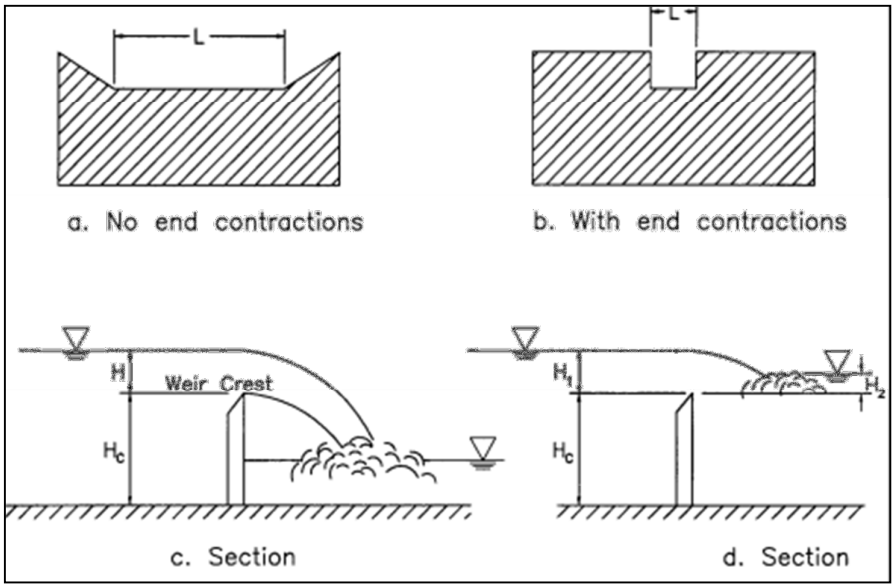


Figure 10-27: Sharp Crested Weirs

Sharp crested weirs will be affected by submergence when the tailwater rises above the weir.

Broad-Crested Weirs

The equation typically used for a broad-crested weir is:

$$Q = C_{BCW} L H^{1.5} \quad (10\text{-}51)$$

Where:

Q = Discharge, m 3 /s

C_{BCW} = Broad-crested weir coefficient, 1.35 – 1.83

L = Broad-Crested weir length, m

H = Head above weir crest, m

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 1.70. For sharp corners on the broad crested weir, a minimum value of 1.44.

V-Notch Weir

The discharge through a v-notch weir is shown in Figure 1.28 and can be calculated from the following equation: (49)

$$Q = K_u \left[tan\left(\frac{\theta}{2}\right) \right] H^{2.5} \quad (10\text{-}52)$$

Where:

Q = Discharge, m^3/s

θ = Angle of v-notch, degrees

H = Head on apex of v-notch, m

K_u = 1.38

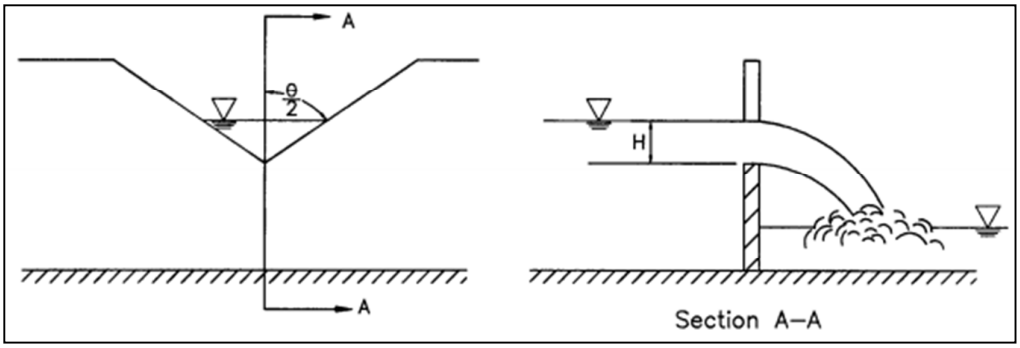


Figure 10-28: V-Notch Weir

10.19 Land-Locked Retention

Watershed areas which drain to a central depression with no positive outlet can be evaluated using a mass flow routing procedure to estimate flood elevations. Typical examples would be retention basins in karst topography or other areas having high infiltration rates. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrologic phenomenon that requires good field measurements and a

thorough understanding of local conditions. Since outflow rates for flooded conditions are difficult to calculate, field measurements are desirable.

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INLET COMPUTATION SHEET

DATE _____ SP _____ ROUTE _____

COMPUTED BY _____ SHEET _____ OF _____

LOCATION		OUTTER DISCHARGE Design Frequency _____					OUTTER DISCHARGE Allowable Spread _____								INLET DISCHARGE			REMARKS
IN-LET NO	STAT	DRAIN AREA "A" (ha)	RUN-OFF COEF "C"	TIME OF CONC "t_c" (min)	Rain Intensity "I" (mm/h)	Q-0.002 78 CIA (m^3/s)	GRADE "S_o" (m/m)	CROSS SLOPE S_x (m/m)	PREV RUNBY (m^3/s)	TOTAL QUT-TER FLOW (m^3/s)	DEPTH "d" (m)	OUT-TER WID TH "W" (m)	SPREAD "t" (m)	W/T	INLET TYPE	INTER-CEPT "Q_i" (m^3/s)	RUNBY "Q_t" (m^3/s)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

Form 10-1 Inlet Spacing Computation Sheet

Computed_____Date_____ Roule_____
Section_____
Checked_____Date_____ County_____

Station		Length (m)	Drainage Area A (ha)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (mm/h)	Total Runoff 0.0278 CIA=Q (m³/s)	Diameter Pipe (mm)	Capacity full (m³/s)	Velocity (m/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (m/m)
From	To		Increamen	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

Form 10-2: Storm Drain Computation Sheet

Station	TW	D_o	Q_o	L_o	V_o	$V_o^2/2g$	H_o	SF_o	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K/V_o^2/2g)$	EGL_o 2 + 7	EGL_o 10+18+19	HGL EGL-7	TOC Elev.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

Form 10-3 Hydraulic Grade Line Computation Form References

APPENDIX 10A - NOMOGRAPHS

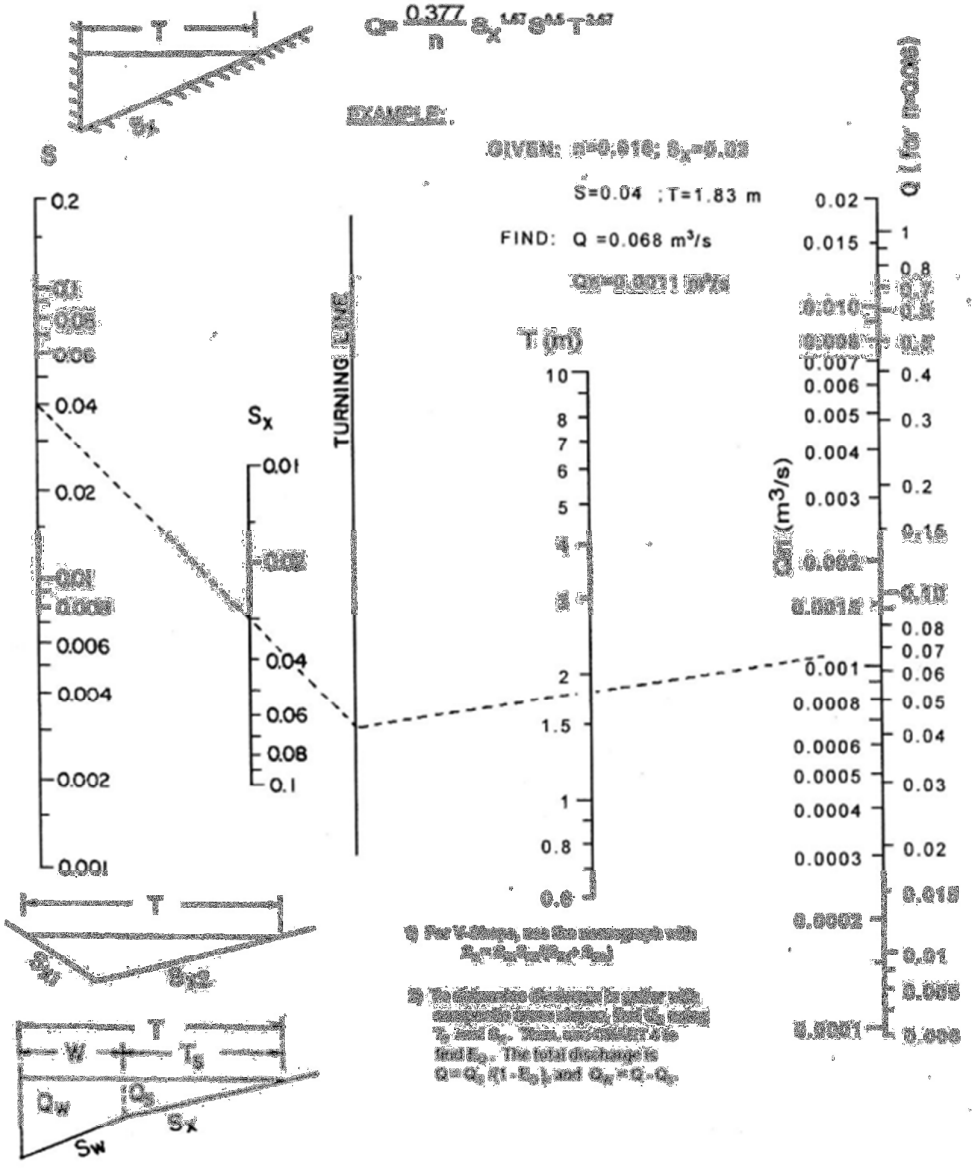


Figure 10A-1: Flow in Triangular Gutter Sections

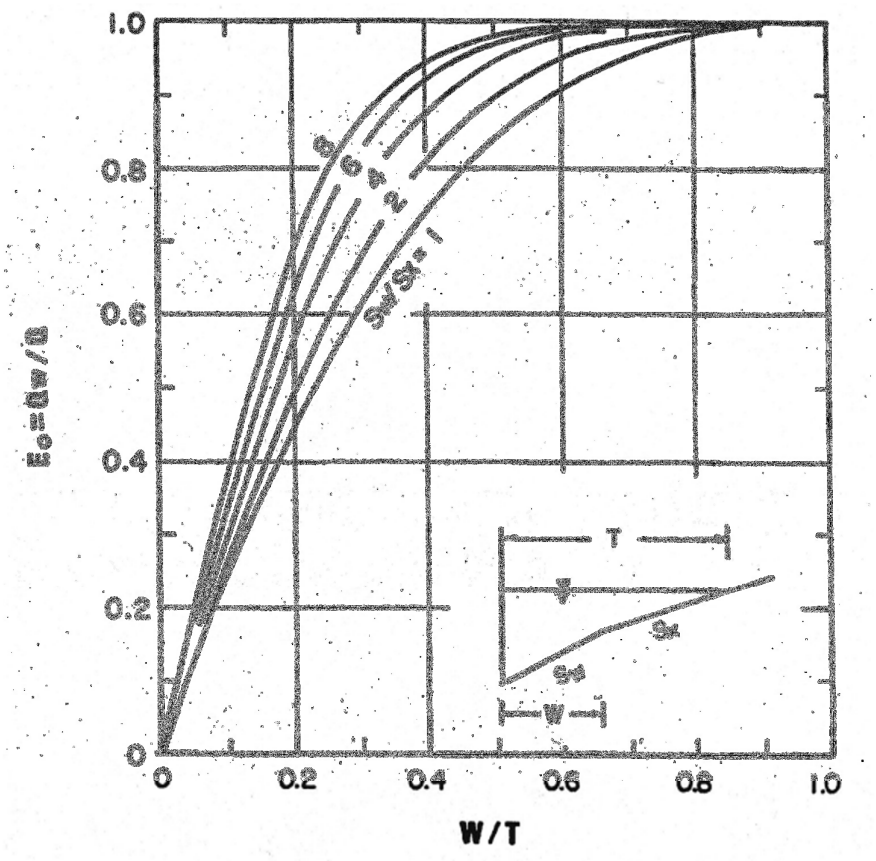


Figure 10A-2: Ratio of Frontal Flow to Total Gutter Flow

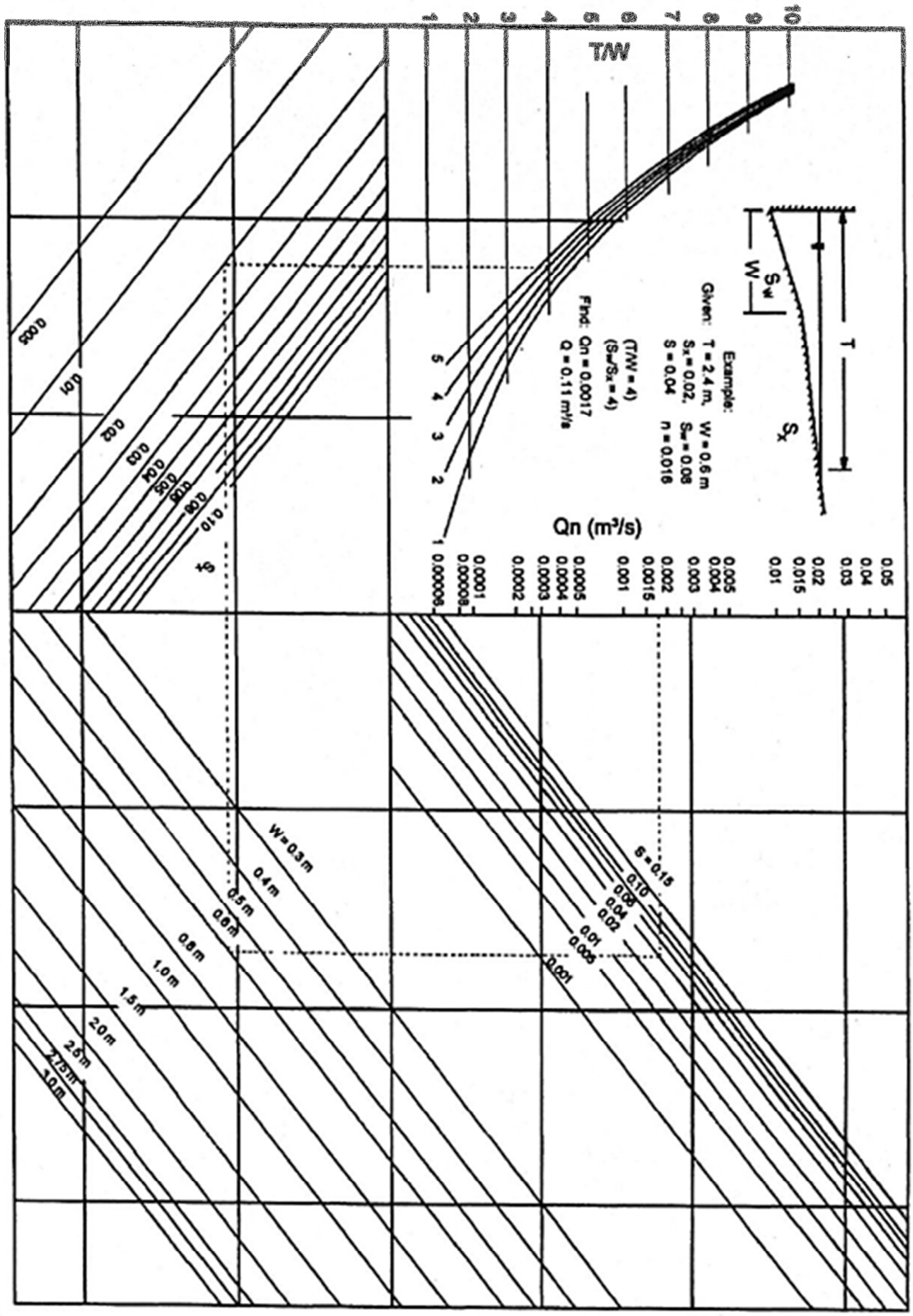


Figure 10A-3: Flow in Composite Gutter Section

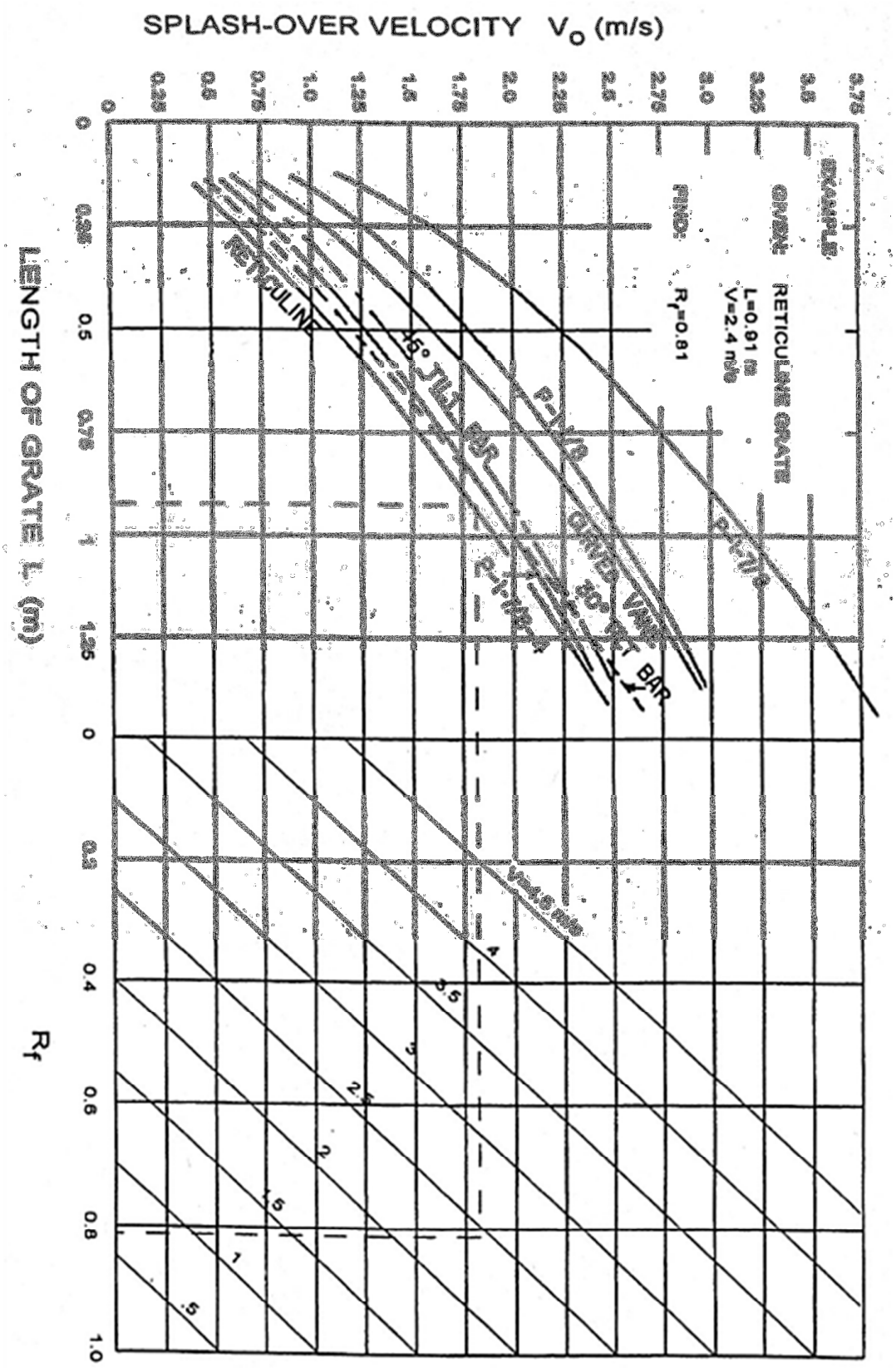


Figure 10A-4: Grate Inlet Frontal Flow Interception Efficiency

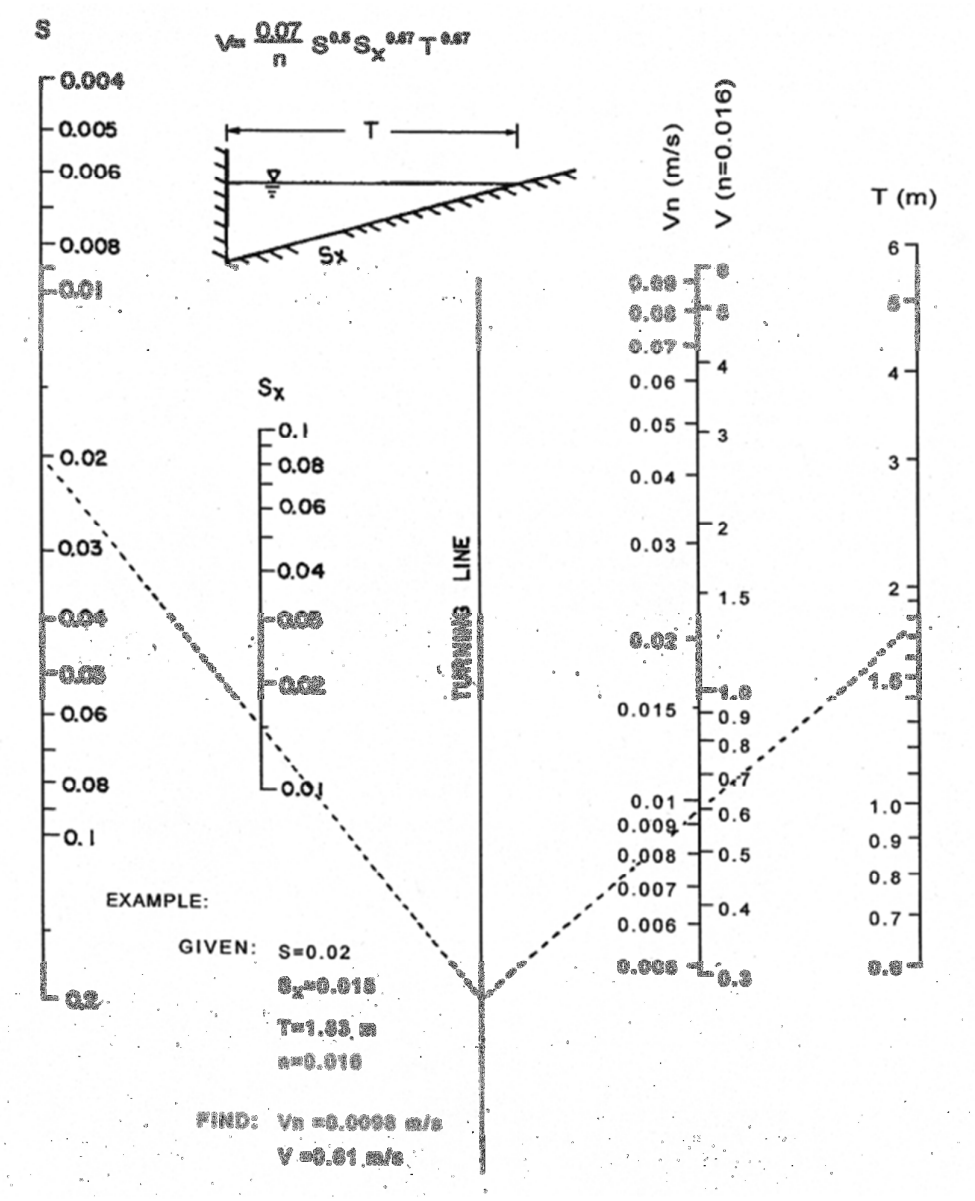


Figure 10A-5: Velocity in Triangular Gutter Section

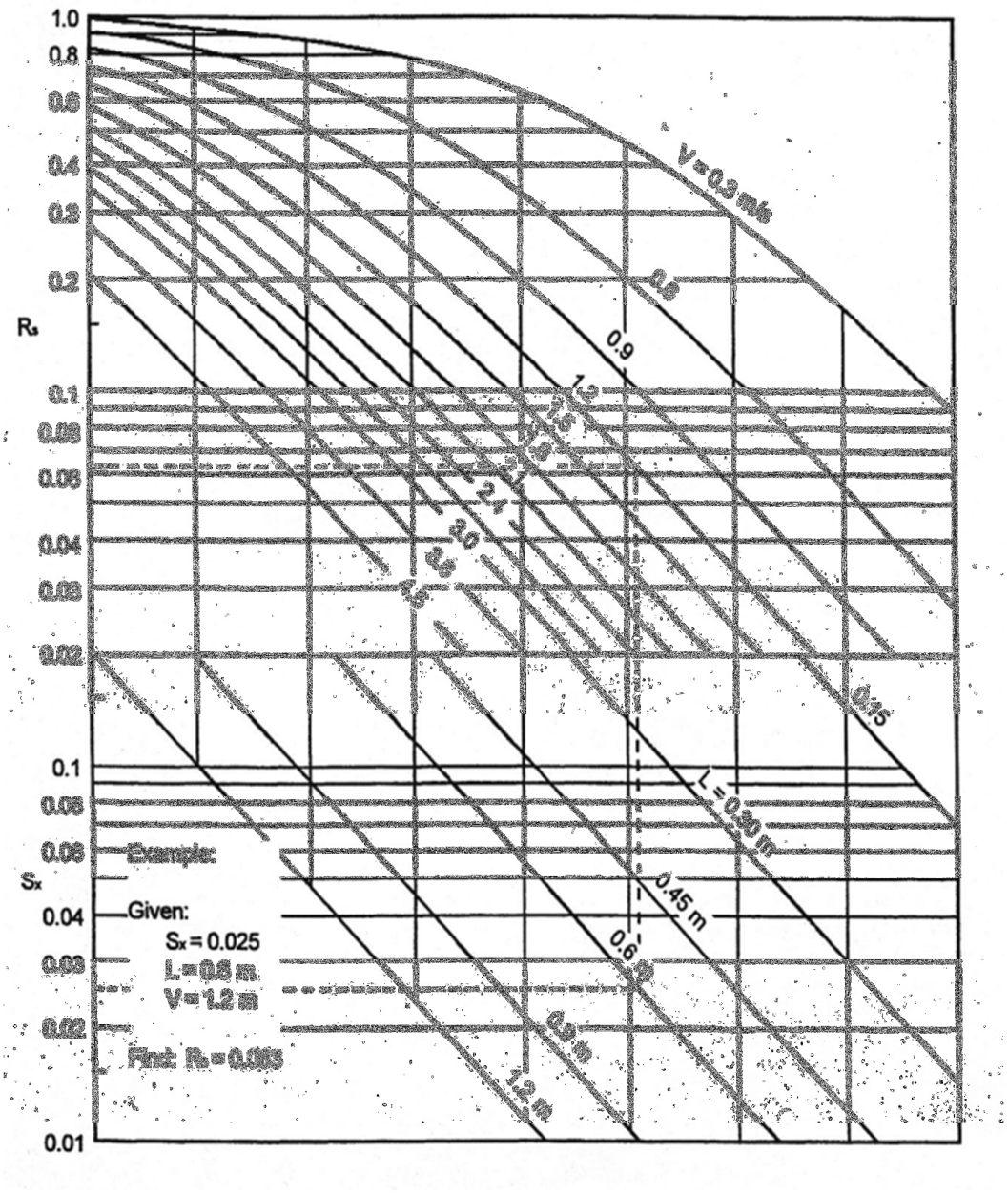


Figure 10A-6: Grate Inlet Side Flow Efficiency

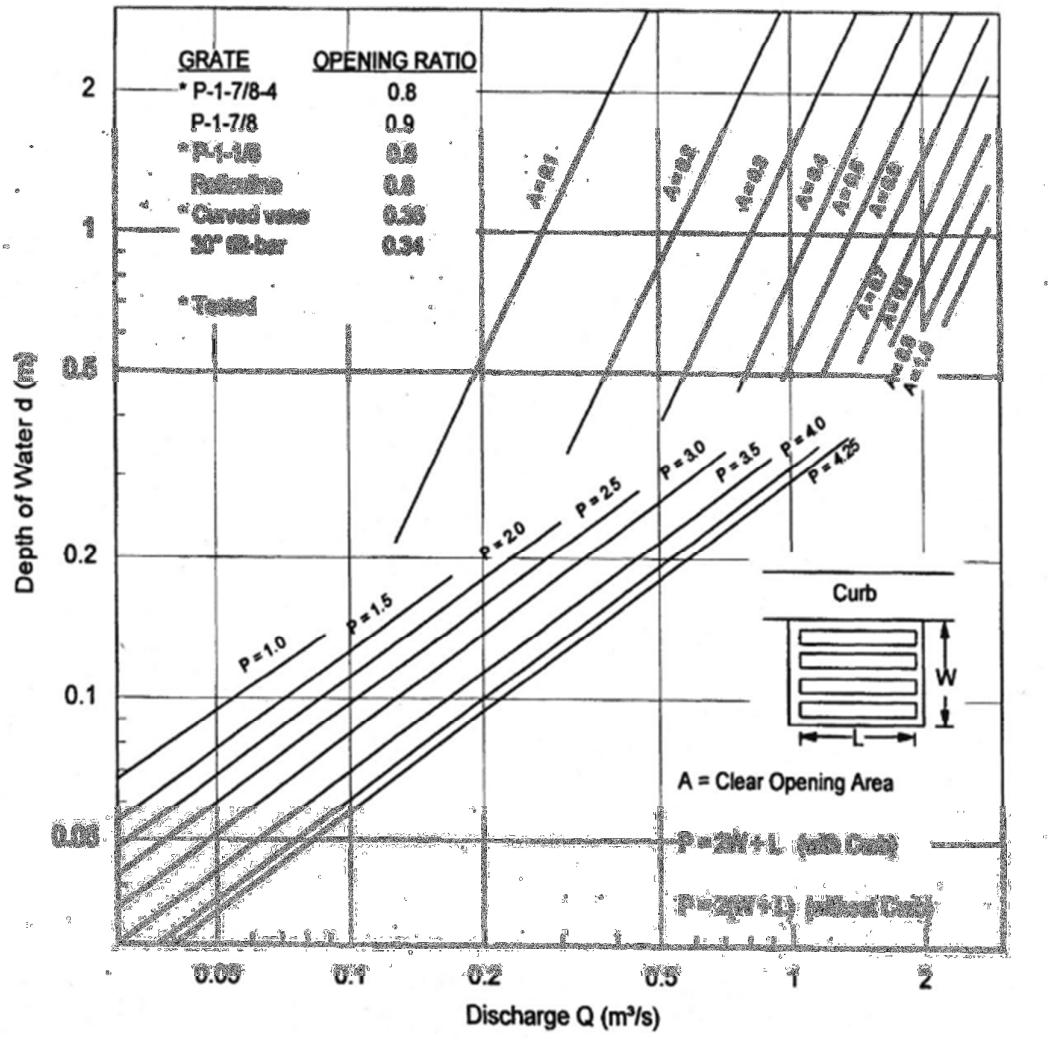


Figure 10A-7: Grate Inlet Capacity in Sump Conditions

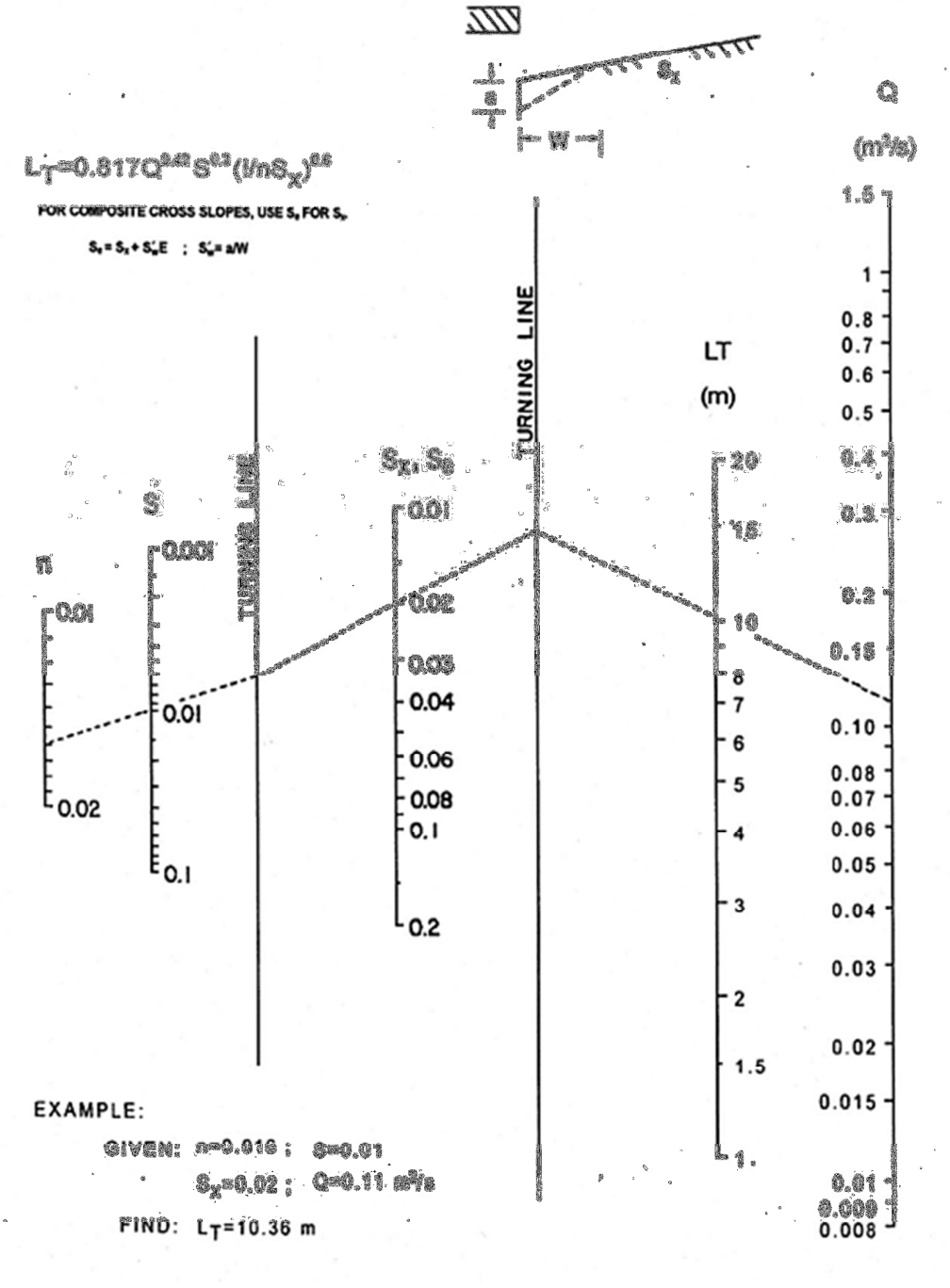


Figure 10A-8: Curb and Longitudinal Slotted Drain Inlet Length for Total Interception

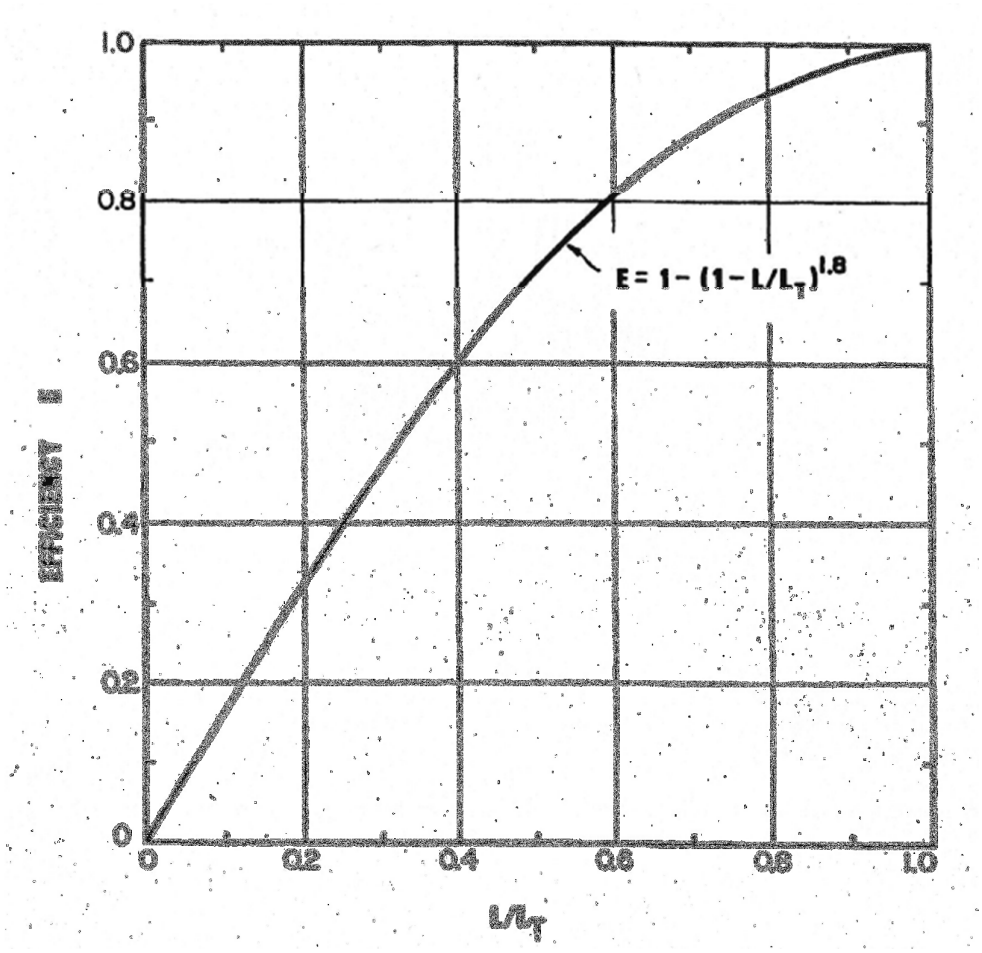


Figure 10A-9: Curb-Opening and Slotted Drain Inlet Interception Efficiency

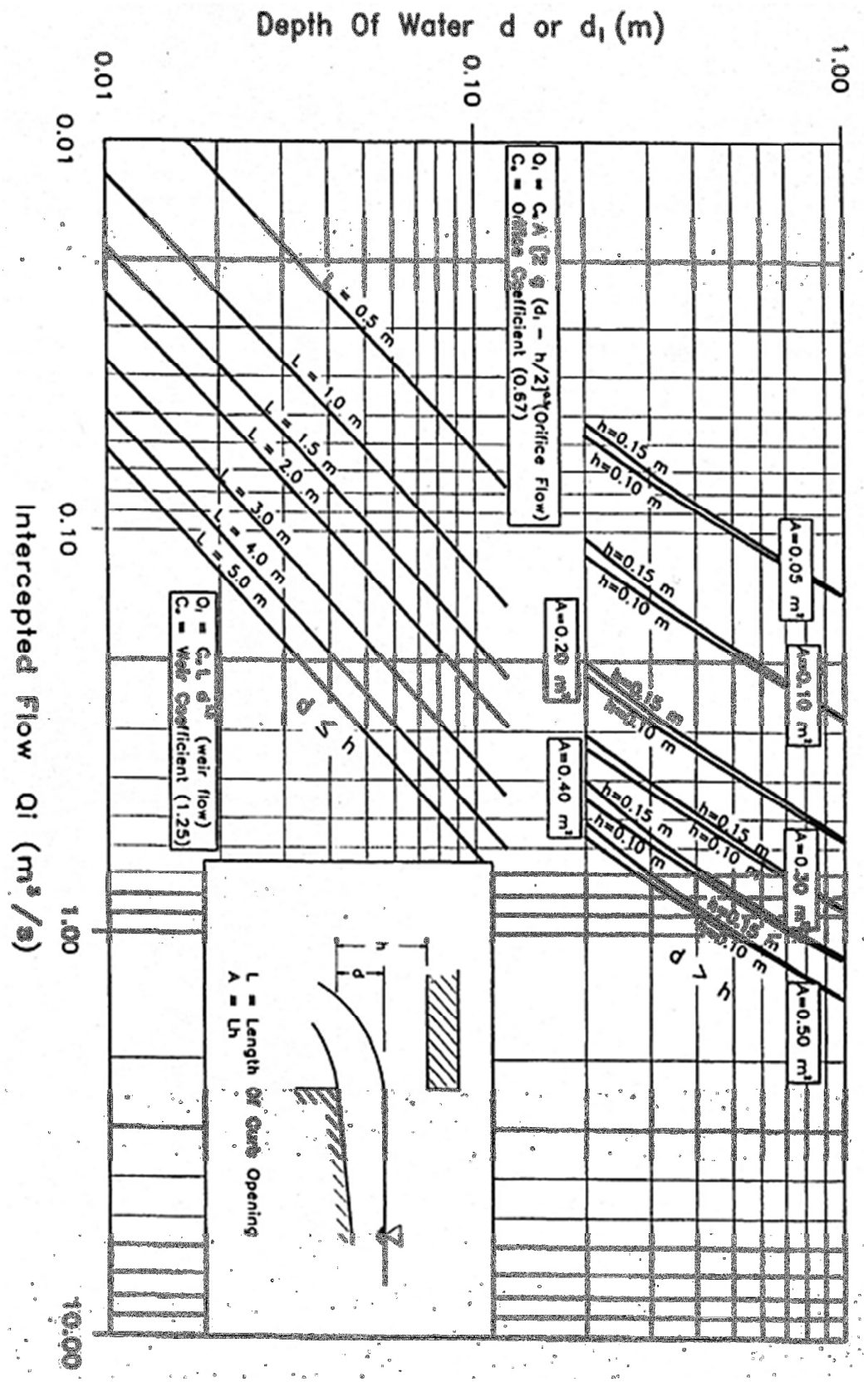


Figure 10A-10: Depressed Curb-Opening Inlet Capacity in Sump Locations

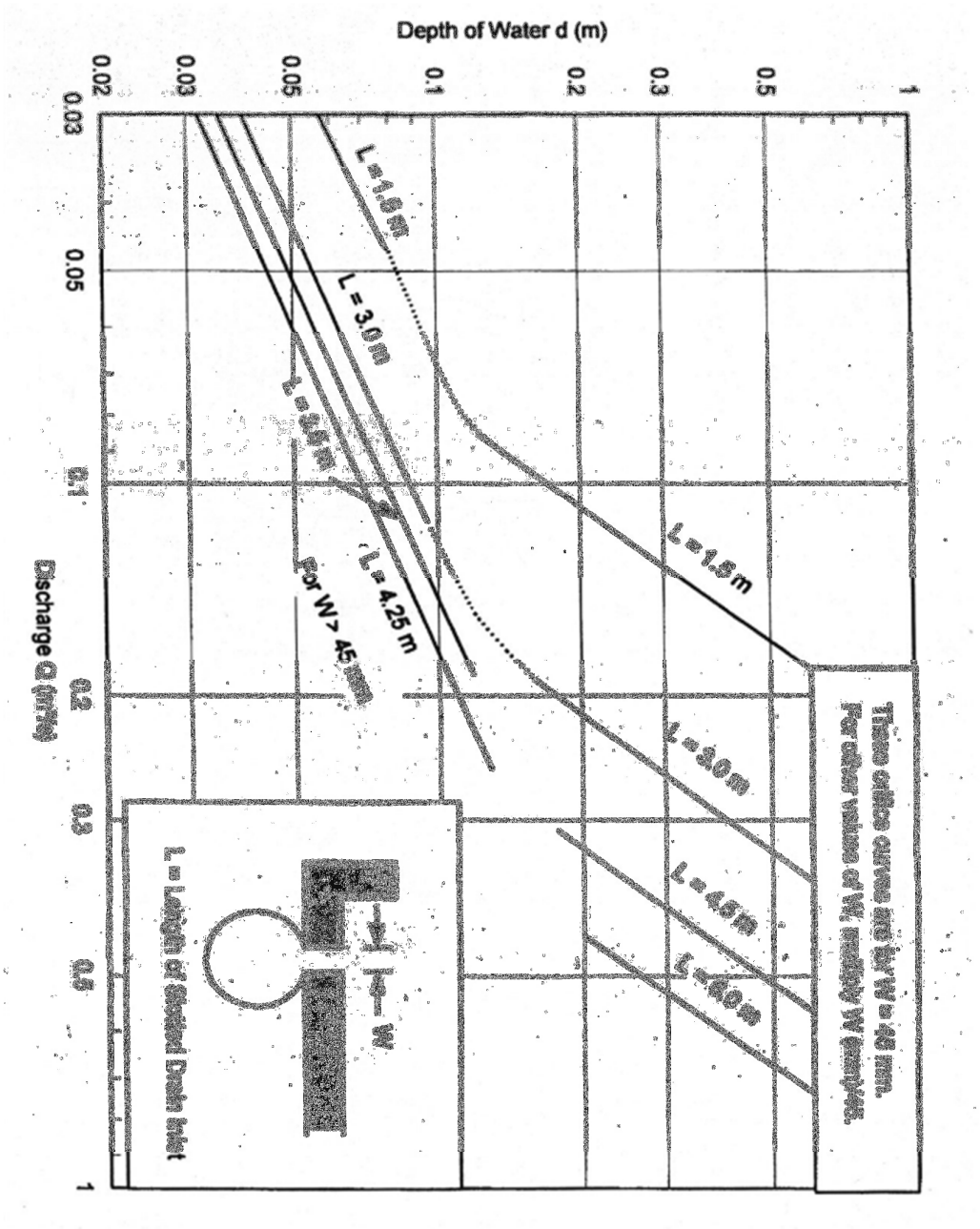


Figure 10A-11: Slotted Drain Inlet in Sump Location

The nomograph solution of Manning's formula for full flow in circular storm drains is shown in Figures 10A-12 through 10A-14. Figure 10A-15 has been provided to assist in the solution of the Manning's equation for part full flow in storm drains.

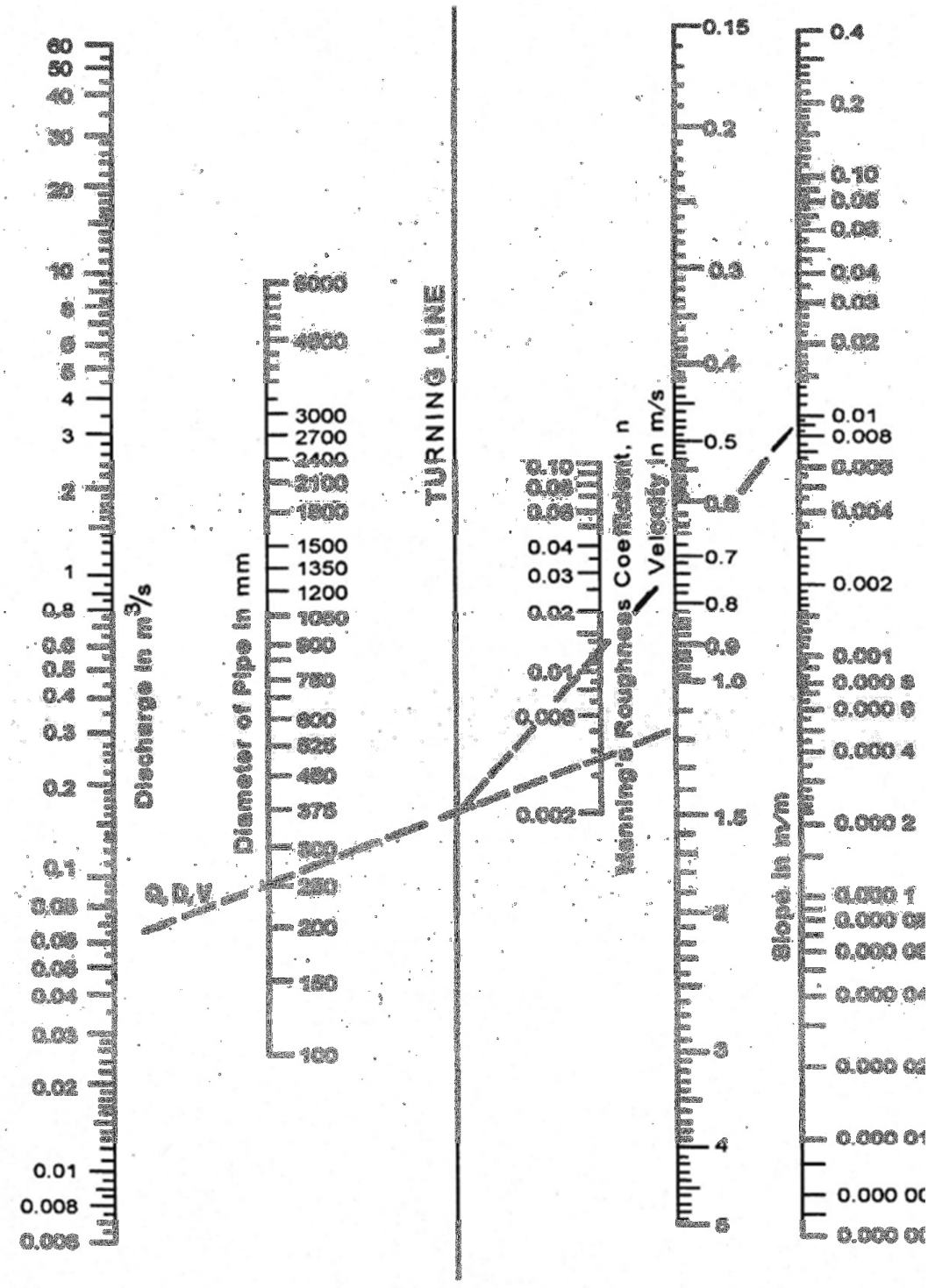


Figure 10A-12: Manning's Formula for Flow in Storm Drains

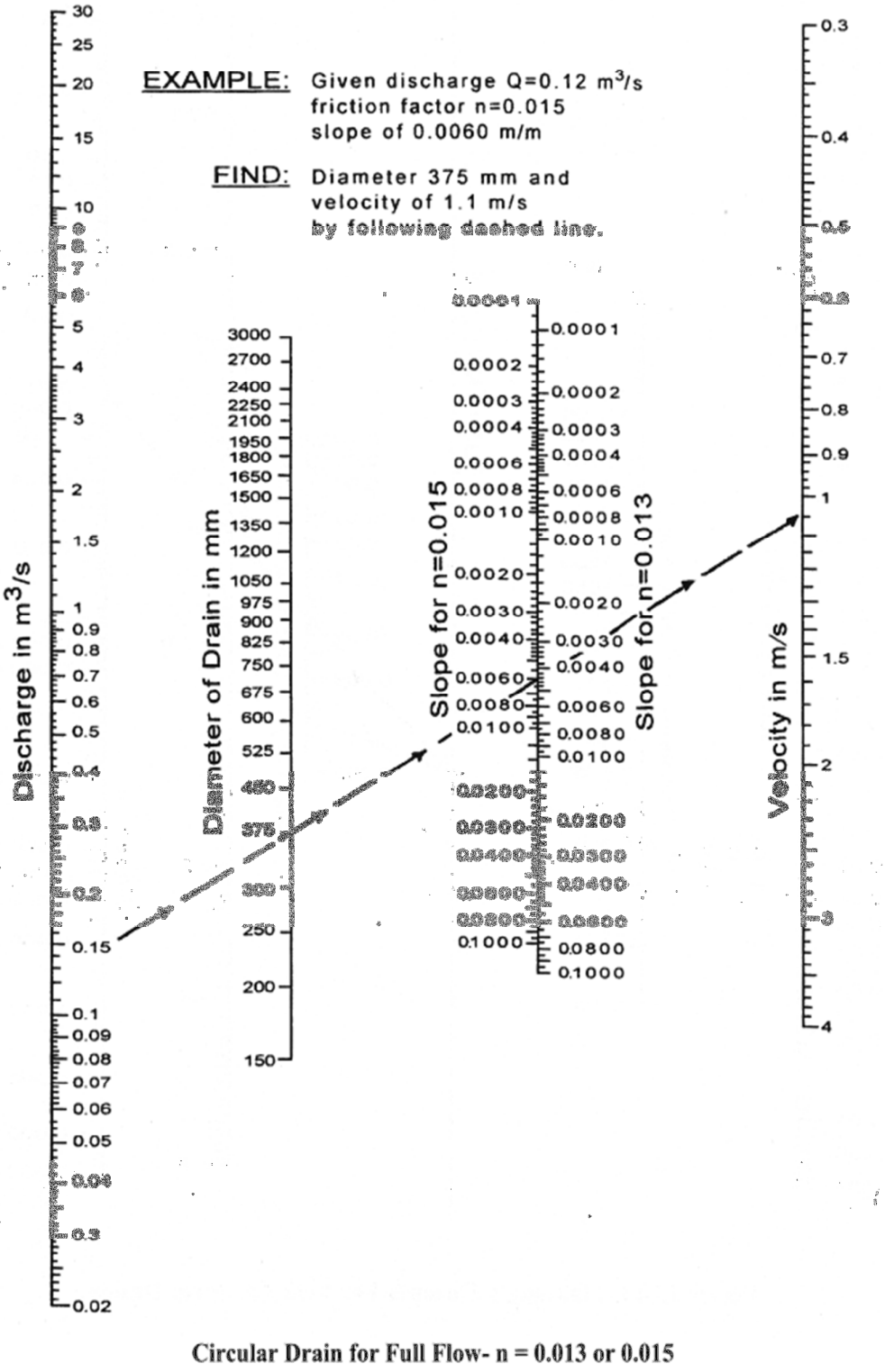


Figure 10A-13: Nomograph for Computing Required Size of Storm Drain

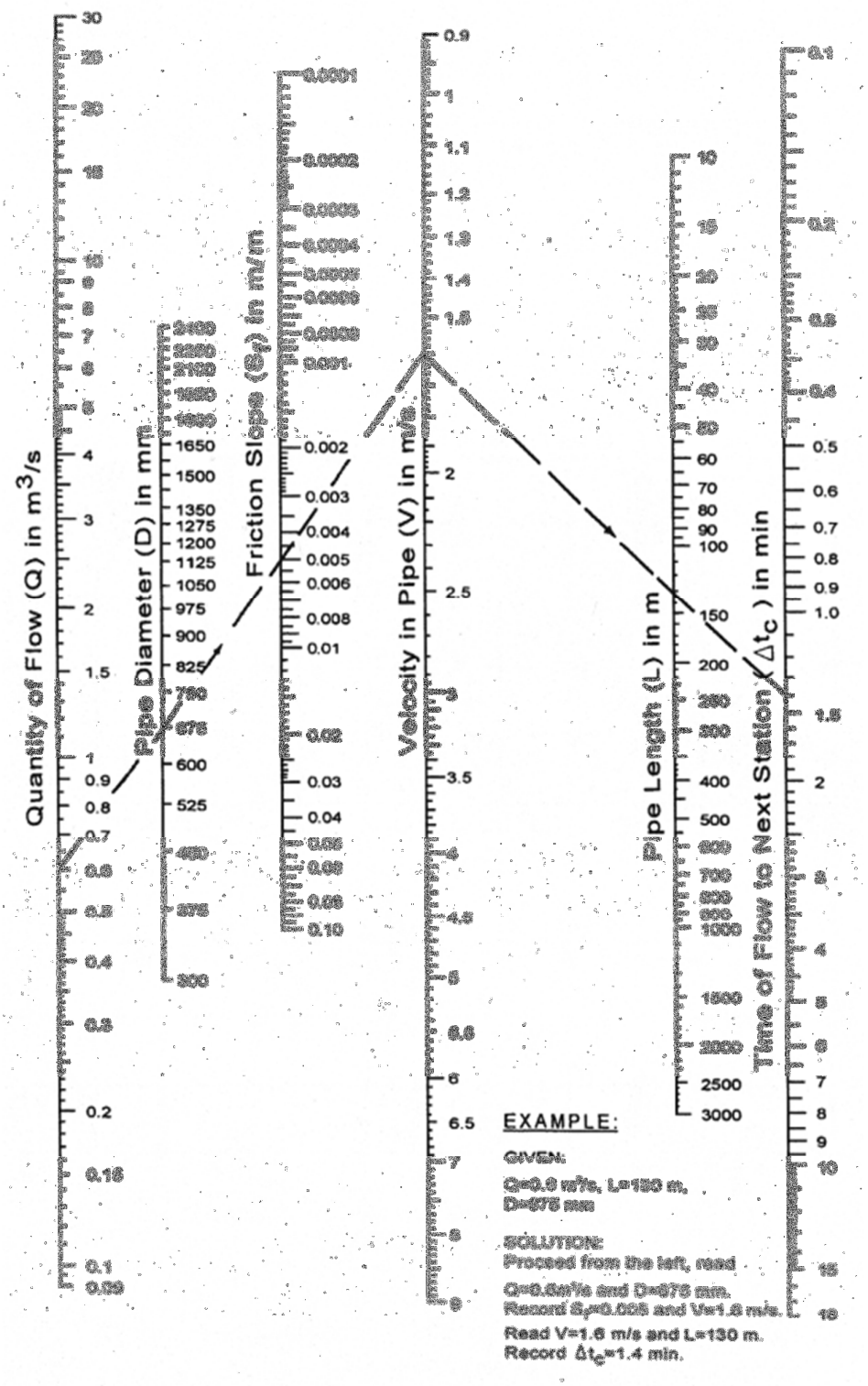


Figure 10A-14: Concrete Pipe Flow Nomograph

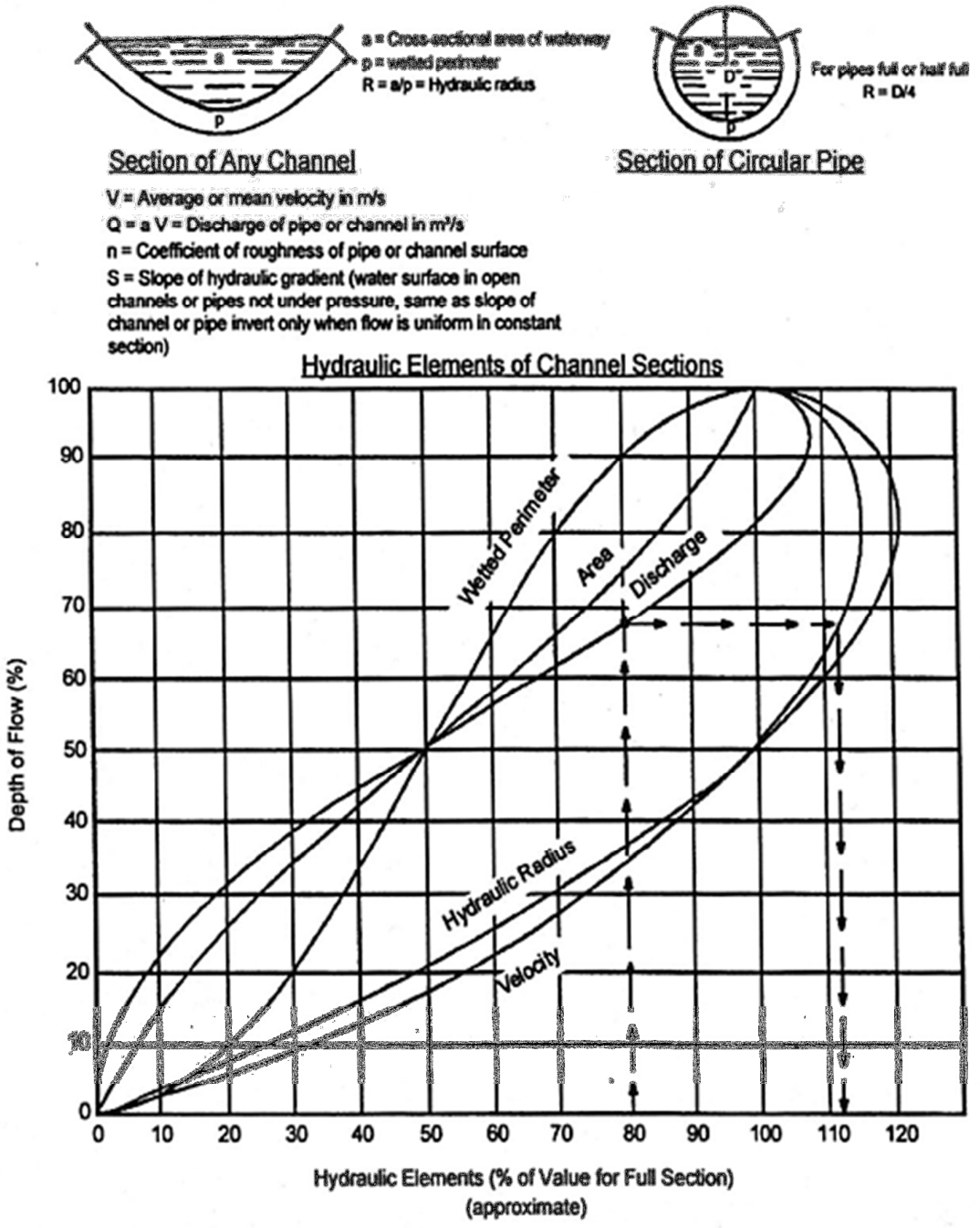


Figure 10A-15: Values of Hydraulic Elements of Circular Section for Various Depths of Flow

11 SUBSURFACE DRAINAGE

11.1 Introduction

Paved and unpaved roads are subjected to problems associated with excess water within the foundation structure of the roadway. This excess water originates from water infiltrating along the roadway surface, groundwater seeping in from upslope areas, high water in roadway ditches or groundwater rising up from beneath the roadway.

The excessive wetness of the road foundation leads to a weakening of the road foundation and, eventually, failure of the surface, whether it is paved or unpaved. The national economic cost of pavement damage as a result of excess water can be estimated in tens of millions of Birr annually. While surface drainage practices do help to alleviate some of the problems associated with excess water conditions, the principal way of handling the problem is to use subsurface drainage.

Sub-surface drainage of highway pavements comprises the measures incorporated in the design in order to control levels of groundwater, and drain the road foundation. Sub-surface drainage is normally necessary in order to remove any water which may permeate through the pavement layers of roads in both cut and fill situations.

Sub-surface drainage in cuttings must provide not only for the necessary drainage of pavement layers, but also for the removal, to an adequate depth, of any groundwater, which may be present in the cutting.

Groundwater may be subject to seasonal variations consequential to rainfall conditions and soil permeability, and the best possible analysis of groundwater conditions should be undertaken during ground investigation. Water moves partly by gravity and partly by capillary action, and these movements are susceptible to control by subsoil drainage.

The hydraulic design of subsurface drainage systems within a road structures is not a precise science as soil layers within a road structure are not uniform. Water table is usually ill-defined and varies significantly from location to location within the road project. Geotechnical investigation before the design of new roads or the rehabilitation of existing roads should be undertaken to identify the need for subsurface drainage. This section of the ERA manual introduces subsurface drainage by discussing some of the important aspects and considerations of subsurface drainage.

Sub-surface drainage is affected by installation of longitudinal sub-surface drains at the low edges of road pavements. These serve to drain the pavement layers and the pavement foundation. They also prevent ingress of water from verge areas adjacent to the pavement.

It is also essential that water be not retained within the sub-base and for that matter the capping layer. Water reaching the formation and sub-formation must be drained to longitudinal sub-surface drains by adequate shaping of the formation and sub- formation such that no undrainable low spots occur.

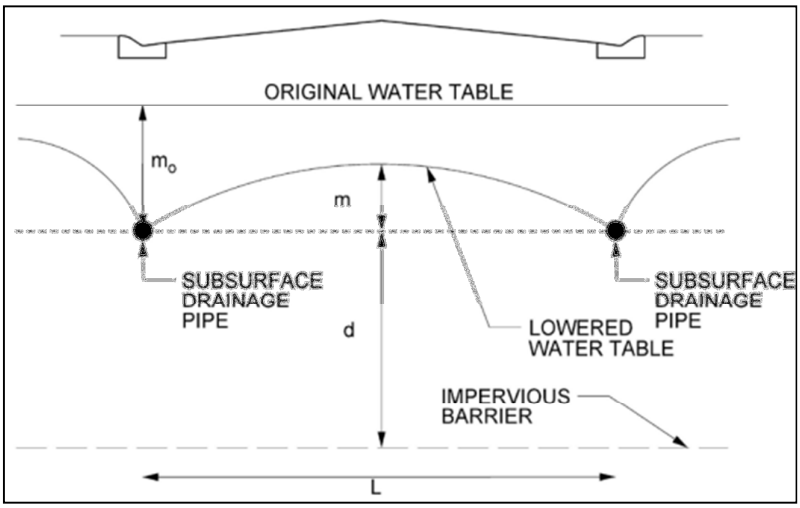


Figure 11-1: Geometry of the Drainage Problem and Effect of Subsurface Drains
(Source: Adapted from Gerke, 1987)

The construction and installation of subsurface drainage systems needs to be conducted with great care to be assured of a positive outcome. One of the leading causes of failure in subsurface drainage systems is inadequate care in the construction and installation phases of a project. Care needs to be taken to assure the proper alignment of drains, proper out letting of drains, and adequate compaction of backfill for drain trenches. It is essential to make sure that construction equipment does not cause misalignment or damage to the drain. Care also needs to be taken by construction inspectors to make sure that the finished product meets the ERA specifications. The contractor is responsible for making sure that the drains are properly installed.

The maintenance of subsurface drains is an essential step in protecting the investment represented by the system. Not only is the capital cost of the drainage system at risk, but the road pavement is as well, because a drainage system operating inadequately will lead to moisture damage to the road. The cost of that damage is several orders of magnitude higher than the cost of a good maintenance program. A sound maintenance program involves periodic inspections of drains and cleanout of drains that are plugged.

11.2 Purpose of Subsurface Drainage System

An important component of pavement design is determining the need for incorporation of a drainage system in new and/or existing pavement structures. The key factors determining the need for subsurface drainage may be categorized as:

- Traffic loads, which include volume and weight (axle):
- Factors influencing the amount of free water entering the pavement system, which include:
  - Climatic factors of rainfall and temperature (freezing and thawing);
  - Ground water;
  - Roadway geometry;
  - Pavement type and condition;
- Factors that increase potential for moisture-related pavement damage, such as;
  - Subgrade type, strength, and condition ;
  - Type of pavement material used; and
  - Design features such as pavement thickness, etc.

Accumulation of moisture introduced into the pavement subgrade from any of the sources can adversely affect pavement performance, leading to accelerated pavement deterioration. Pavement problems associated with infiltrated water may fall into three categories:

- Softening of the pavement layers and subgrade by becoming saturated and remaining so for prolonged periods;
- Degradation of the quality of pavement and subgrade material due to interaction with moisture; and
- Loss of bonding between pavement layers due to saturation with moisture.

Likewise, failures occurring due to groundwater and seepage may be classified into two categories:

- Those causing piping or erosion failures; and
- Those caused by uncontrolled seepage patterns leading to saturation, internal flooding, excessive uplift, or excessive seepage forces.

11.3 Planning of Subsurface Drainage

In the planning of an underground drainage system, the first step is to undertake a groundwater investigation at the site. There are two basic groundwater conditions for which drainage systems are essential:

- **Groundwater with a hydraulic gradient smaller than the slope of the ground**. Typical warning signs are wet patches and visible outflows on the side of a cut. Interception drains should be installed in such cases.
- **A groundwater table close to the surface**: Signs of this are collapsing wet spots in flat areas. This condition arises where water infiltrates from high areas or through a leaking surface layer. Layer drainage should be installed in this case.

Observation should be carried out on:

- **Geology** – fractured, fissured or jointed rock, impermeable dykes or alternating layers of permeable and impermeable material;
- **Vegetation** – variations in colour and vigorous growth or hydrophilic vegetation;
- **Topography** – shape of the land, depression, valley lines, catchments , etc.; and
- **Road surface failures** – pumping, rutting, tension cracks.

A design method should be chosen to calculate the capacity of the drainage system (Note: storm water many not be discharged into a subsurface system).

In the case of groundwater intercepted from cuts, it is generally not practical to carry out a sophisticated calculation, probably because too wide a variety of materials is found in a cut, and seasonal changes have a strong influence on the groundwater discharge. In practice, the capacity of an intercept drain may be determined by in-situ flow measurement (during the wet season after the channel has been constructed) or, if excessive quantities of groundwater have not been observed, or there is no groundwater in the dry season, normal drainage may be provided.

11.4 Sources of Moisture

An important component of design and installation of an effective pavement subsurface drainage system is the knowledge and understanding of the sources of moisture reaching the subsurface layers of the pavement structure. Designers should be knowledgeable about the various sources of water in the structure for them to identify the best methods for preventing the moisture from entering the system or removing it once it has entered the system (ERES, 1999). The moisture in the pavement subgrade may come from many sources, as is illustrated in Figure 11.2. The main source of water infiltrating into pavement structural sections is generally from precipitation (Moulton, 1980).

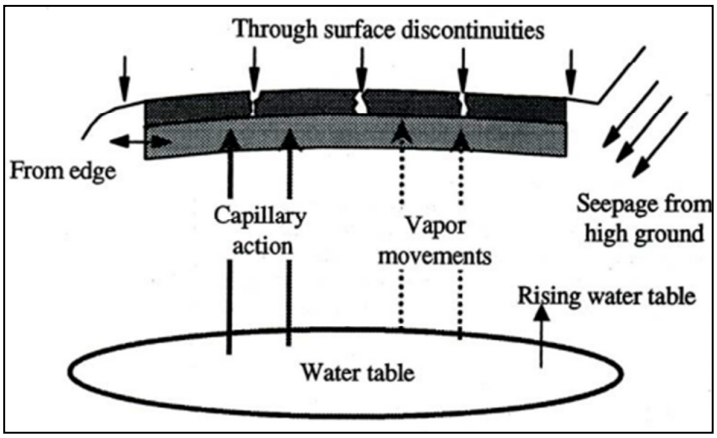


Figure 11-2: Sources of Moisture Reaching Subsurface of the Pavement System

(Source: ERES, 1999)

11.5 Effects of Moisture on Pavements

Moisture related problems in pavements can be minimized when designers of the structure make conscious efforts to keep the base, sub-base, subgrade, and other susceptible paving materials from becoming saturated or exposed to constant high moisture levels. Three effective approaches to controlling or reducing pavement problems are:

- To provide adequate cross slopes and longitudinal slopes to quickly drain moisture from pavement surface, thereby minimizing infiltration into the pavement structure;
- To use material and design features, such as stabilized cement (CTB) or lean concrete bases (LCB) in Portland cement concrete, also known as PCC pavement, that are not sensitive to the effects of moisture; and
- To remove moisture that enters the pavement system promptly.

For effective control of moisture related problems in pavements over the life of the pavement, it often is necessary to employ these approaches in combination.

11.6 Quantifying Net Inflow by Source

During the design process of the pavement drainage system, the design net inflow, q_n, should include inflow from all possible sources. The major sources of inflow include surface infiltration, inflows from gravity drainage of groundwater and artesian sources from below the pavement (FHWA, 1992). Discussion on these sources, and the methods of computing design inflow rates are presented in the following sections.

When considering all the important probable combinations of inflows and outflows, the following relationship for computing net design inflow is important:

$$q_n = q_i + q_g + q_m + q_a \quad (11\text{-}1)$$

Where:

q_n = the design net inflow;
q_i = the inflow from infiltration;
q_g = the inflow from gravity flow of groundwater;
q_m = the inflow from melt water from thawing ice lenses; and
q_a = the inflow from an artesian source below the pavement.

Each of these sources will be discussed in the sections to follow, and details of calculations are presented at the end of this chapter.

There are two types of hydraulic design approaches used in the design of pavement drainage systems. They are known as the steady-state flow approach and the time-to-drain approach.

Both of these have a part to play in the design even though the time-to-drain approach is preferred (FHWA, 1992). This preference is due to the fact that the steady-state approach requires estimates of the inflows from the various sources, one of these being the infiltration source.

The first problem with quantifying the infiltration flux is in estimating the design rainfall rate. Hydraulic engineers need to decide on the proper selection of the storm frequency and the time of concentration (storm duration), which are required for estimation of a design rainfall. The second problem is estimating the portion of rainfall that enters the pavement. However, putting these concerns aside, it is appropriate to consider both analyses for the design of the drainage system.

The steady-state flow analysis assumes that constant flows from the various sources are entering the pavement structure. The drainage system, including the drainable base course, as well as the drain and drain outlet, is designed to enable removal of this flow without allowing the base to become saturated.

The Time-to-Drain analysis considers the situation where the base becomes saturated due to some design rainfall event, and determines the capacity of the drainage system necessary to remove this excess water within a desired period of time. Whichever of these two analyses yields the maximum required drainage capacity will be the result that is selected for the design.

11.6.1 Infiltration, q_i

Water arriving at the pavement surface would infiltrate into the subgrade layers through surface discontinuities such as joints, cracks, shoulder edges and any other defects in the pavement surface. Studies have shown surface infiltration to be the single largest source of moisture-related problems in PCC pavements (FHWA, 1994). Hagen and Cochran (1995) discovered that 40 percent of rainfall enters the pavement. Although AC pavements lack joints, their surface cracks, longitudinal cold joints that crack, and pavement edges provide pathways for water to infiltrate the pavement structure.

Pavement infiltration (cu.m/day/sq m of pavement) is the volume of water entering through a specified area of pavement, and can be determined by either the infiltration ratio method or the crack infiltration method (FHWA, 1992). Of these two methods, the Crack

Infiltration method is preferred because parameters can be estimated more easily and with greater confidence (Moulton, 1980).

11.6.2 Groundwater, q_g, q_a

The seasonal fluctuations of the water table can be a significant source of water moving into pavement sections. Although this flow varies with season, the rate of change in flow is sufficiently small so one can justifiably treat the flow as steady-state.

Two possible sources of groundwater which should be considered during design of subsurface drainage systems are gravity drainage, which is water moving laterally towards the pavement section (see Figure 11.3a), and artesian flow, which is upward flow from confined aquifers (see Figure 11.3b) (Moulton, 1980).

While it is feasible in some situations to intercept all of the groundwater flowing towards the pavement structure, in many instances it will not be possible, especially with regard to water originating from an artesian aquifer system. When some, but not all, of the groundwater is intercepted, it is necessary to include seepage from this source while designing pavement drainage.

The contribution of water flow to the pavement from these two sources of groundwater can be estimated using information about hydraulic conductivity of the underlying soil and the water pressures in the soil alongside the road and in the confined aquifer.

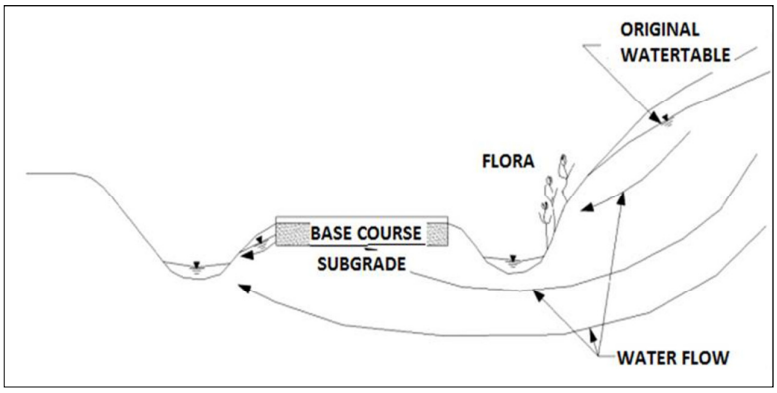


Figure 11-3a: Lateral (Gravity) Flow of Groundwater towards the Road

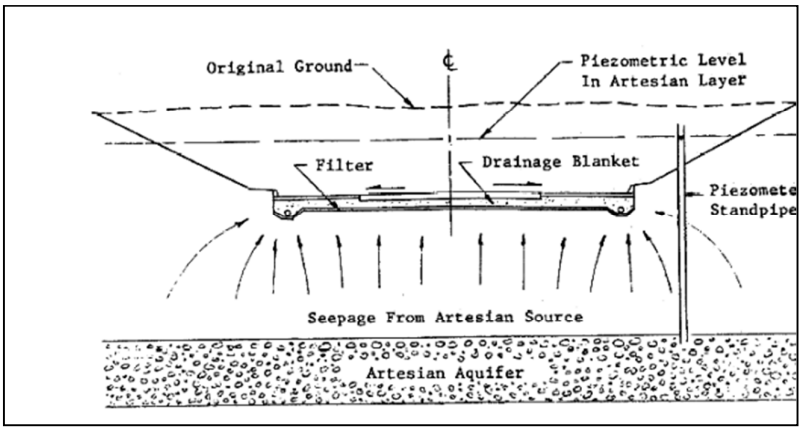


Figure 11-4b: Flow of Water from a Confined (Artesian) Aquifer

(Source: Moulton, 1980)

11.7 Pavement Geometry

The geometry of a highway plays an important part in the design of a pavement drainage system. Therefore, good geometric designs which facilitate surface drainage of a pavement and median in both the transverse and longitudinal directions is an important design consideration.

Comprehensive guidelines on geometric design to provide adequate surface drainage are described in the AASHTO manual (AASHTO, 1990; Johnson and Chang, 1984; Anderson and Reed, 1998). A well designed pavement would provide for a system that has an effective method of preventing surface water from infiltrating into the pavement system. A pavement which does not allow moisture to stay on its surface for long would prevent the moisture from entering the pavement base layers through cracks, joints, or pavement surface infiltration. This can be accomplished by providing adequate cross slopes and longitudinal slopes to quickly drain moisture from the pavement surface.

An example of a typical pavement constructed in this manner is one with crowned sections having transverse slopes ranging from 1.5 to 2.5 percent for the surface layer and 3.5 to 6 percent for the shoulders (Yu et al., 1998). Figure 11.4 illustrates recommended cross sections for various pavement designs.

11.7.1 Longitudinal Grades

A requirement in the design of subsurface drainage systems is a set of road cross-sections showing the original ground and the gross features (i.e. cut and fill slopes, ditches, etc.) of the proposed construction. It is also desirable to have a topographic map of the highway corridor upon which the final highway alignment has been superimposed. This map should be of a large enough scale (100 or 200 scale) that features pertinent to surface and subsurface drainage, such as streams, lakes, and the seasonally wet areas above the highway, can be clearly identified.

The flow of water across the surface of a paved road is controlled to a large extent, by the longitudinal grade of the roadway, g, and its cross slope, S_c. Figures 11.4 and 11.5 illustrate paths of water movement in the pavement surface and subsurface, and possible routes the water would follow in permeating the subbase. The length of the flow path, L, can be expressed in equation:

$$L = W\sqrt{1 + \left(\frac{g}{S_c}\right)^2} \qquad (11\text{-}2)$$

Where:

W = the width of the drainage layer

g = longitudinal grade of the roadway

S_c = roadway cross slope

The slope of the flow path, S, can be evaluated using the expression:

$$S = W\sqrt{{S_c}^2 + g^2} \qquad (11\text{-}3)$$

After the determination of the various combinations of longitudinal and transverse grades to be encountered on the project at hand, the data should be tabulated in a form convenient for the calculation of L and S required in the design and analysis. An anomaly with the

equation for determination of the flow path, L, is that whenever the transverse grade approaches zero, the length of the flow path given by equation (11, 3) approaches infinity. In practice, the relationship between longitudinal and transverse grades will be a local one, and length of the flow path will be governed by the grades of adjacent sections of roadway and/or the distance to the nearest transverse drain.

Another anomaly is that if either the cross slope or the longitudinal grade is varying with the stationing along the road, the flow path cannot be linear, but will be curved as shown in Figure 11.4.

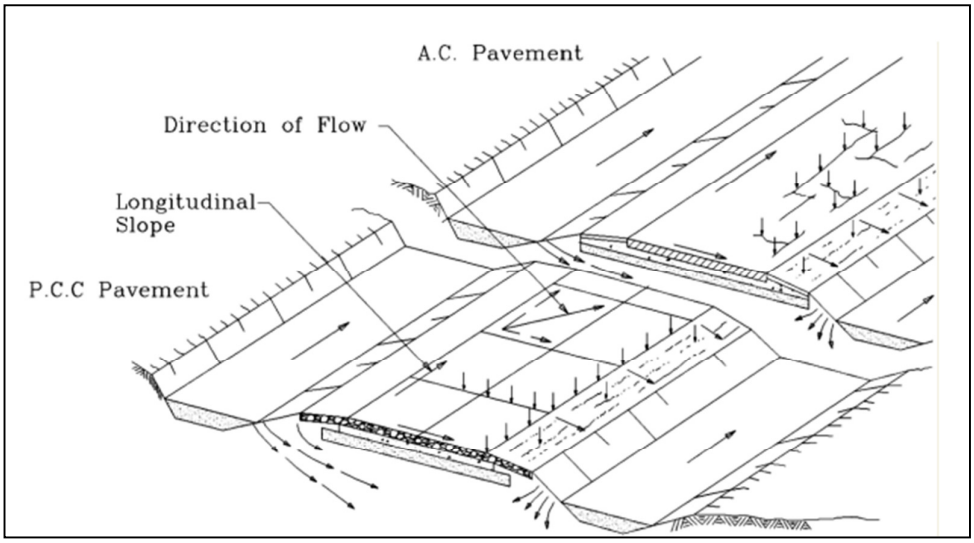


Figure 11-5: Points of Entrance of Water into the Highway Pavement

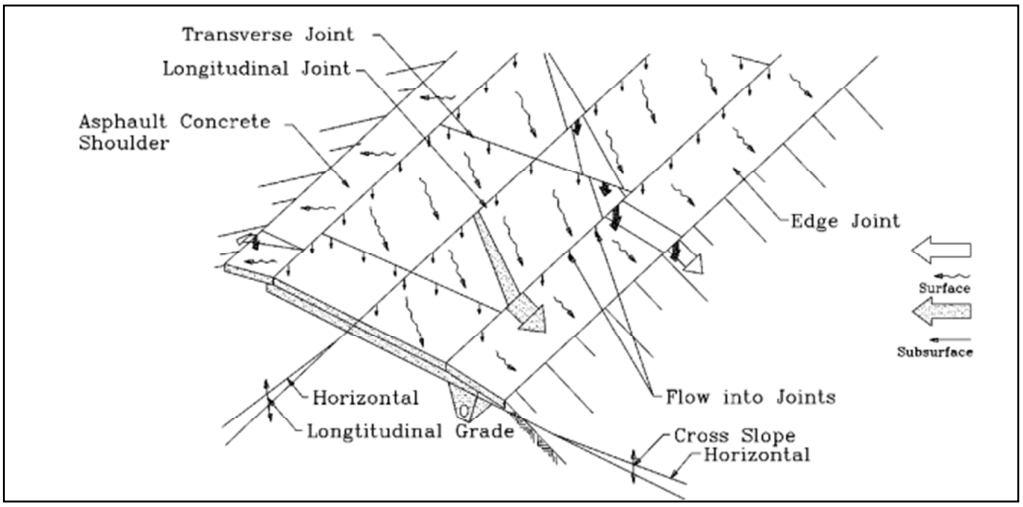


Figure 11-6: Paths of Flow of Subsurface Water in Portland Cement Concrete Pavement

11.7.2 Subsurface Geometry

It is necessary to establish as accurately as possible the nature and limits, especially the subsurface boundaries, of the flow domain. To accomplish this would require a thorough and detailed exploration and geologic evaluation of the subsurface, leading to the development of soil and rock profiles, as well as to define the prevailing groundwater

conditions. A good subsurface exploration is a vital part of the basic design procedure for highways. There are agricultural and geological maps available for some of Ethiopia that can be used while planning the subsurface exploration program.

11.8 Types of Subsurface Drainage Systems

Pavement design that did not incorporate subsurface drainage systems are shown in Figures 11.6 through 11.8. Pavement without subsurface drainage has water which enters the base and subbase layers and is trapped there. The subgrade and shoulder back-fill are fine- grained soils, acting as barriers which prevent water from exiting, hence resulting in the system filling with water over time. This condition is commonly known as "trench". If this water does not find a way out of the pavement structure, pavement problems are likely to be manifested. Construction of pavements without providing a subsurface drainage system may be acceptable in areas with special conditions, such as areas where coarse-grained subgrade is present, which allows water to drain vertically through the subgrade (ERES, 1999).

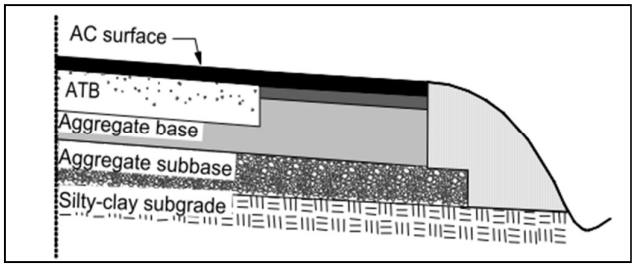


Figure 11-7: Typical AC Pavement Section

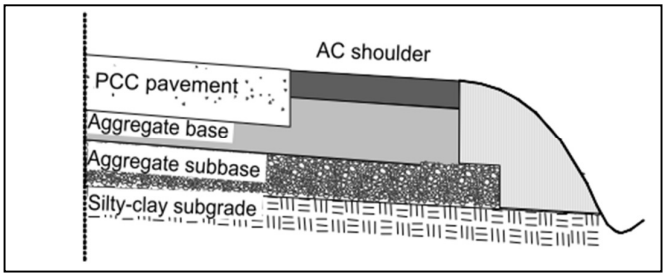


Figure11-8: Typical Undrained PCC Payment Section

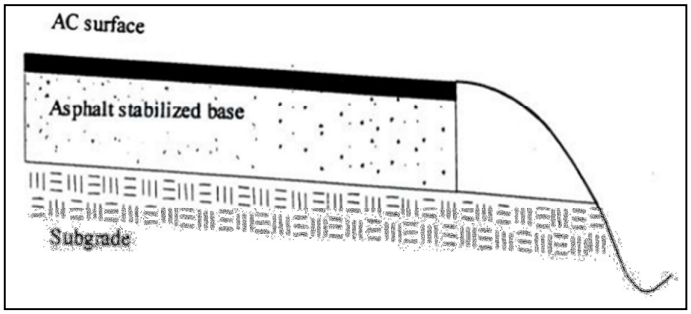


Figure 11-9: Typical Full-Depth Asphalt Concrete Section

Groundwater control systems are drainage systems designed to remove and/or control the flow of groundwater, while infiltration control systems are designed to remove water that seeps into the pavement structural section. There are situations where both subdrainage

systems may be required to control water from both sources (Moulton, 1980). Subsurface drainage systems are commonly identified in terms of their location and geometry (ERES, 1999). In this classification, subsurface drainage systems are typically divided into five distinct types. These are:

- Longitudinal edge drains;
- Transverse and horizontal drains;
- Permeable bases;
- Drainage blankets; and
- Well systems.

Highway subsurface drainage systems can be classified according to the source of the subsurface water they are designed to control, the function they perform, and their location and geometry.

Well designed subsurface drainage systems are capable of performing different functions, including interception or cut-off of the seepage from above an impervious boundary, draw-down of the water table, and collection of flow from other drainage systems (Moulton, 1980).

Even though they are commonly designed to serve one function, subsurface drainage systems may be expected to serve other functions, such as an interceptor drain, used to cut off side-flow, and as a means to draw down the water table.

A typical, well-designed drainable pavement system should consist of the following design elements and features:

- Full-width permeable base, or non-erodible base under the AC or PCC surfaced pavement;
- A separator layer under the permeable base to prevent contamination from the subgrade materials; and
- Longitudinal edge drains with closely spaced outlets, or edge drains 'day lighting' directly into a side ditch.

Designs which do not incorporate these combinations of features cannot be expected to function properly.

11.8.1 Infiltration

The most commonly used approaches to address surface infiltration water for new construction are day lighted dense-graded or permeable bases and permeable bases with longitudinal edge drains. For existing pavements, retrofit edge drains are the common means of improving drainage of existing pavements.

Figures 11.9 and 11.10 show details of the use of different subsurface drainage systems for different types of pavement. The realization of the importance of drainage to pavement performance has led to a trend where older pavements not originally provided with subsurface drainage are being retrofitted with drainage features such as longitudinal edge drains. The functions of different types of subsurface drainage systems are illustrated in Figures 11.9 through 11.11.

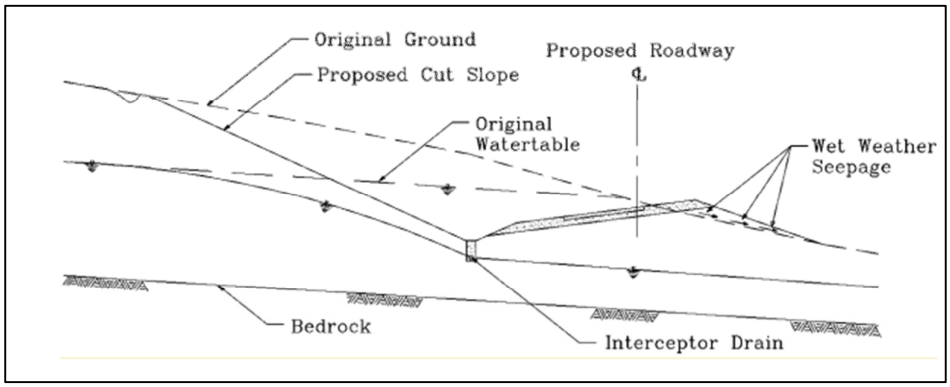


Figure 11-10: Longitudinal Interceptor Drain used to Cut Off Seepage and Lower the Groundwater Table

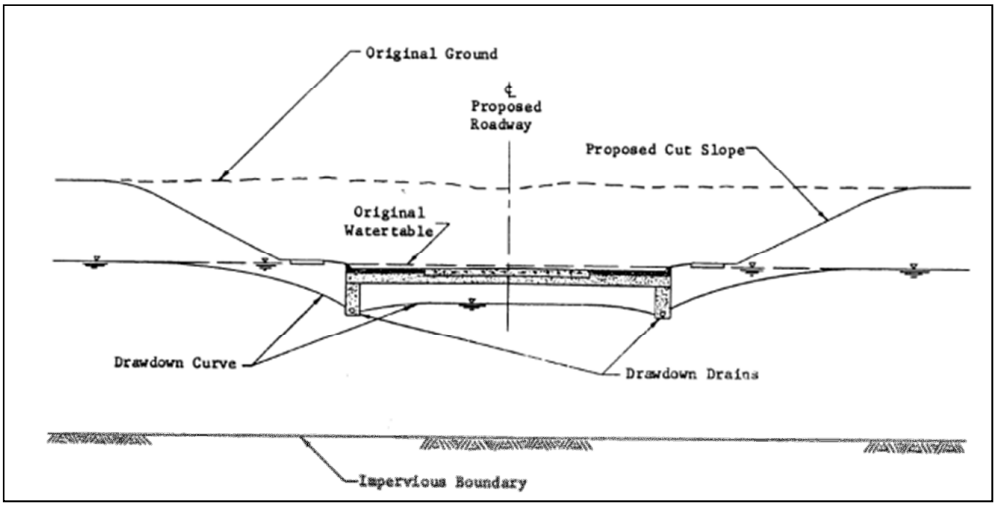


Figure 11-11: Symmetrical Longitudinal Drains used to Lower the Groundwater Table and to Collect Water Infiltrating the Pavement

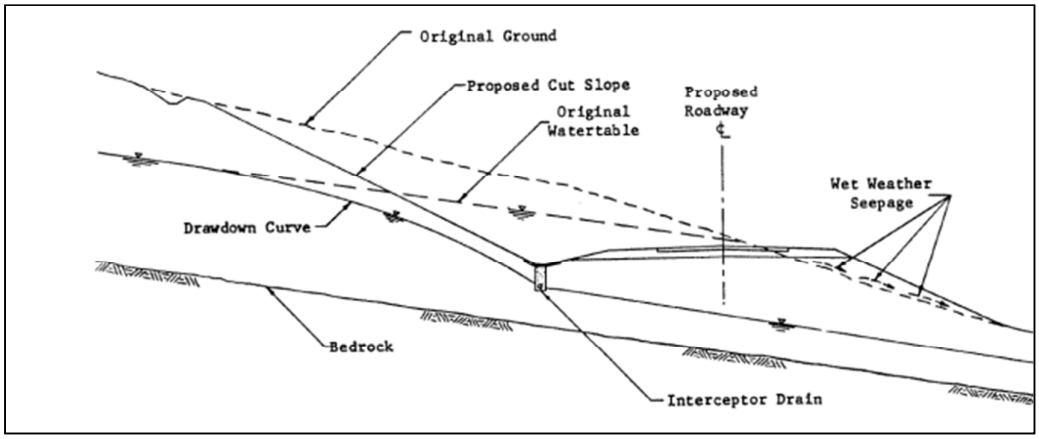


Figure 11-12: Multiple Interceptor Drain Installation from Groundwater Control

11.8.2 Positive Drainage of the Permeable Base

Permeable bases beneath pavements must be provided with positive drainage, usually in the form of longitudinal edge drains with outlet pipes (FHWA, 1994). A drainage alternative often used is day lighting the permeable base into ditches. However, this has proved ineffective over the long run because the day lighted layers often get clogged by roadway debris and vegetation.

The type of subsurface drainage system an engineer is likely to select depends, to a large extent, on the three selection criteria suggested by Moulton (1980) above.

11.8.3 Longitudinal Drains

Longitudinal drains are normally located parallel to the road centreline, with both horizontal and vertical alignments. This type of drainage usually includes a trench of substantial depth, a collector pipe, and a protective filter of some kind. They may also be less elaborate. Examples of types of longitudinal drains commonly used in control of seepage and groundwater are shown in Figures 11.12 and 11.13.

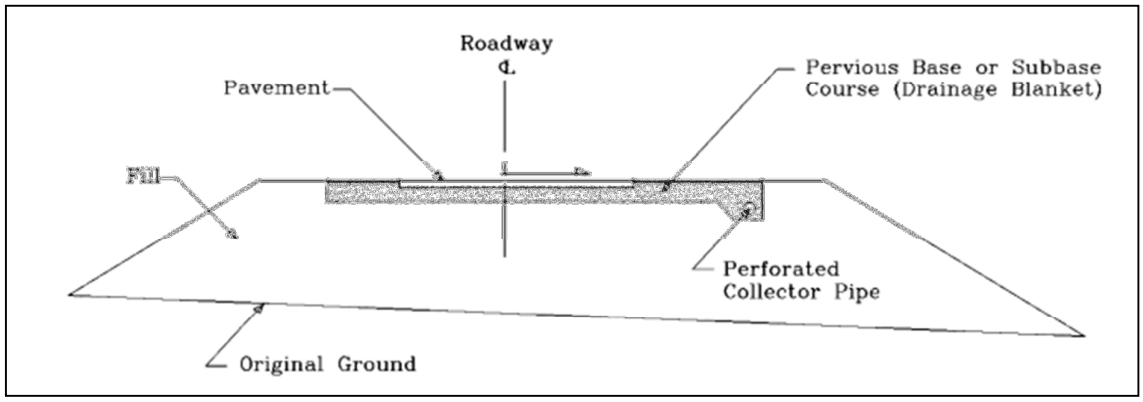


Figure 11-13: Longitudinal Collector Drain used to Remove Water Seeping into Pavement Structure Section

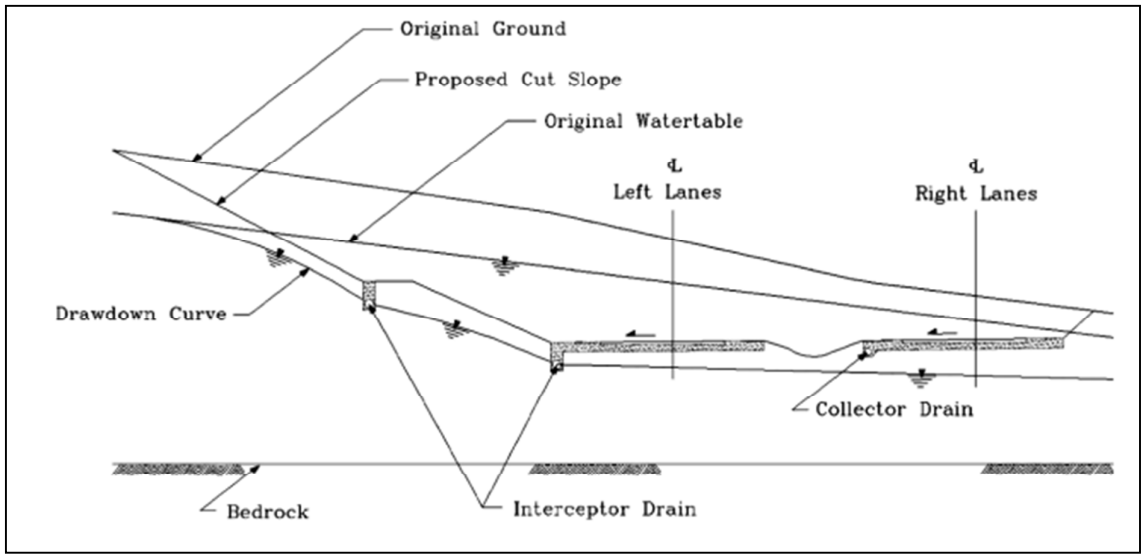


Figure 11-14: Multiple Multipurpose Longitudinal Drain Installation

Sometimes, a multiple drain installation is needed for control of water under certain situations. Figure11.13 shows an example of a multiple longitudinal drain in a section of an expressway cut in a wet hillside. In order to intercept the flow and draw down the water table below the left cut slope, it was necessary to use two lines of relatively deep longitudinal drains.

In addition to intercepting water flowing from the hill slope, the interceptor drain beneath the left shoulder of the left lanes drains any water that may enter the base or subbase of the left lanes from infiltration or frost action. The shallow collector drain along the left edge of the right lanes performs this same function.

In many cases it is not possible to compact the subgrade material to the desired specification. The material is then removed and other more suitable material is transported in to replace it. The resulting backfilled subcuts are then susceptible to the 'bathtub effect', meaning that water will accumulate in the volume of replaced material. It is important to provide drainage for these subcuts. The drainage of subcuts can be accomplished with longitudinal drains if the subcuts are continuous along the pavement, or the drains might be placed on a transverse angle to the pavement if the subcut's volumes are localized. The design of drains for longitudinal subcuts follows the same procedures used for longitudinal edge drains. For more localized subcut situations the design of the drains can follow the procedures used for transverse drains.

11.8.4 Transverse and Horizontal Drains

Transverse drains are a class of subsurface drains that run laterally beneath the road. The common placement of these drains is at right angles to the road centreline, although they may be skewed in some cases, creating what is often referred to as the "herringbone" pattern. This type of drainage system is often used at pavement joints to drain infiltration and groundwater which may be in the bases and subbases. Transverse drains may be used in conjunction with a horizontal drainage blanket and longitudinal collector drain system, which provides an effective means for rapid removal of water from the pavement section.

Transverse drains may involve a trench, collector pipe, and protective filter, as shown in Figure 11.14, or they can consist of simple "french drains" (shallow trenches filled with open graded aggregate), although this is not generally recommended. The degree of sophistication employed in the designs of this type of drainage system depends on the source of the subsurface water and the function of the drain. This type of drain is especially effective when used in situations where the general direction of the groundwater flow tends to be parallel to the roadway (common when the roadway is cut more or less perpendicular to the existing contours). This application is illustrated in Figure 11.15.

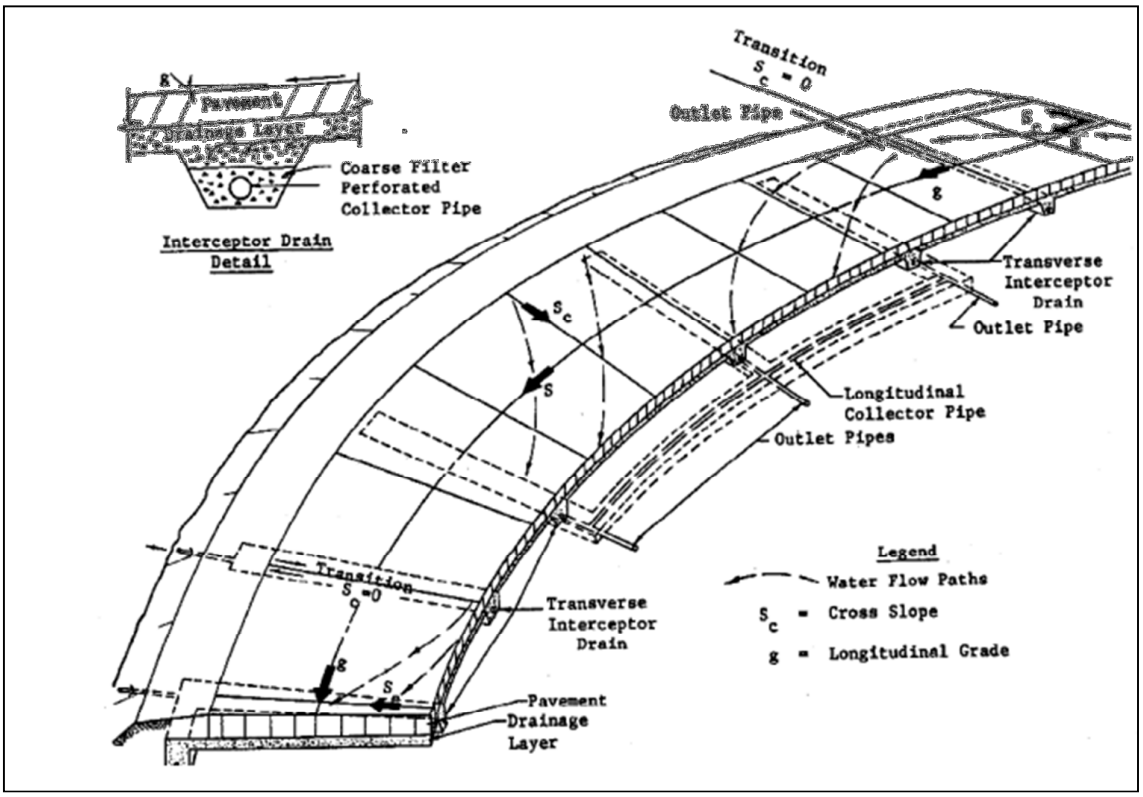


Figure 11-15: Transverse Drains on Super-Elevated Curve

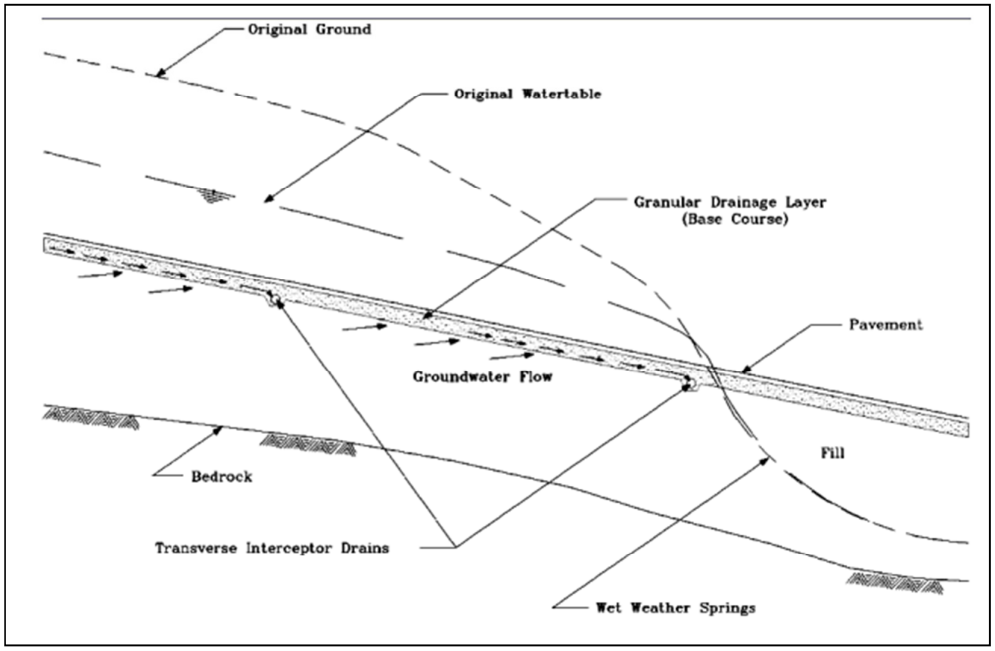


Figure 11-16: Transverse Interceptor Drain Installation in Road Cut with Alignment Perpendicular to Existing Contours

Horizontal drains consist of nearly horizontal pipes drilled into cut slopes or side hill fills to tap springs and relieve pore water pressures.

11.8.5 Medians

Subsurface drains (Figure 11.16) should be considered along the:

- Low side of a dished median where the median drain invert level is less than 0.2 m below subgrade level of the adjacent pavement;
- Low side of a kerbed median where the cross-slope is 0.10 m/m or more;
- Sides of a median with a fixed watering system or wider than 6m; and
- The centre of flat grassed medians without fixed watering systems and less than 6m wide.

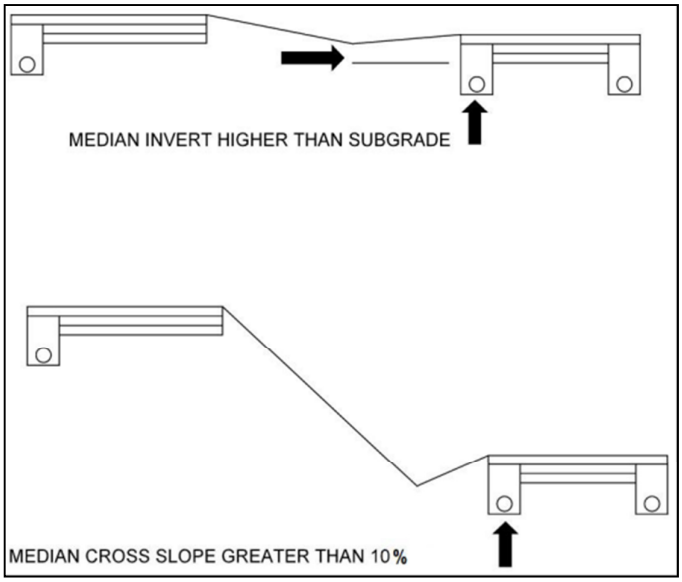


Figure 11-17: Median Subsurface Drain

11.8.6 Drainage Blankets

Drainage blanket is a term generally applied to a very permeable layer whose width and length in flow direction is large relative to its thickness. These drainage systems, if properly designed, can be used for effective control of both groundwater and infiltration. The horizontal drainage blanket can be placed beneath or serve as an integral part of a pavement structure to remove infiltrated water or to remove groundwater from both gravity and artesian sources. To function as drainage blankets, the systems must be specifically designed and constructed to do so. They must be designed with adequate thickness of material with a very high coefficient of permeability, a positive outlet for the water collected, and, in some instances, the use of one or more protective filter layers (Cedergren, 1973a; Cedergren, 1974a).

Types and applications of horizontal drainage blanket systems are shown in Figures 11.17 through 11.19. In Figure 11.17 a horizontal blanket drains used in connection with shallow longitudinal collector drains to control both infiltration and the flow of groundwater from an artesian source. In Figure 11.18 a horizontal blanket drain is used to remove water that has seeped into the pavement by infiltration alone. An outlet can be provided by "day lighting" the drainage blanket. However, it should be noted that it is not uncommon for this type of outlet to become clogged and cease to function effectively. A more positive means of out-letting the drainage blanket would be to use the longitudinal drain as shown dashed in the illustration.

Drainage blankets can be used effectively to control the flow of groundwater from cut slopes and beneath side hill fills, as illustrated in Figures 11.19 and 11.20. When the drainage blanket is used in connection with a longitudinal drain, this will help improve the surface stability (relieve sloughing) of cut slopes by preventing the development of a surface of seepage.

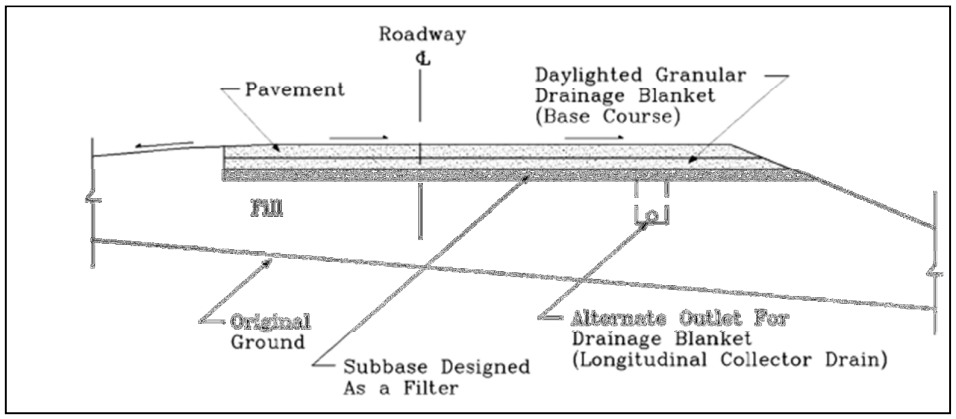


Figure 11-18: Application of Horizontal Drainage Blankets

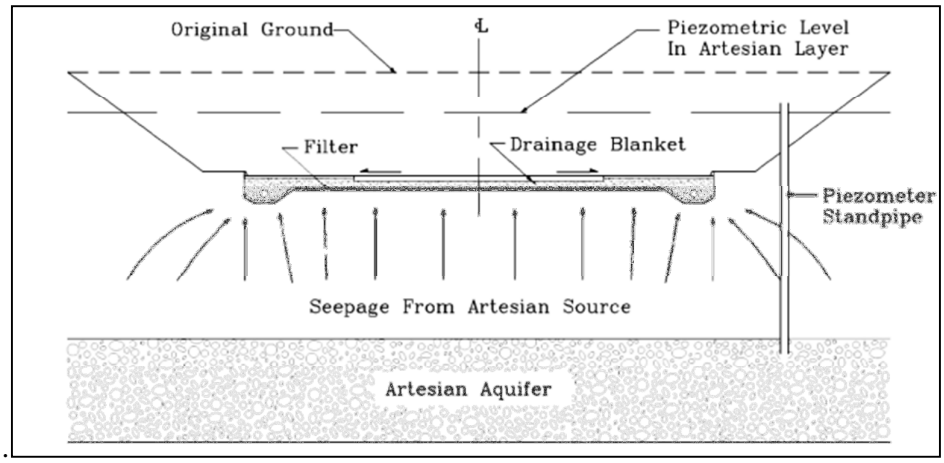


Figure 11-19: Application of Horizontal Drainage Blankets

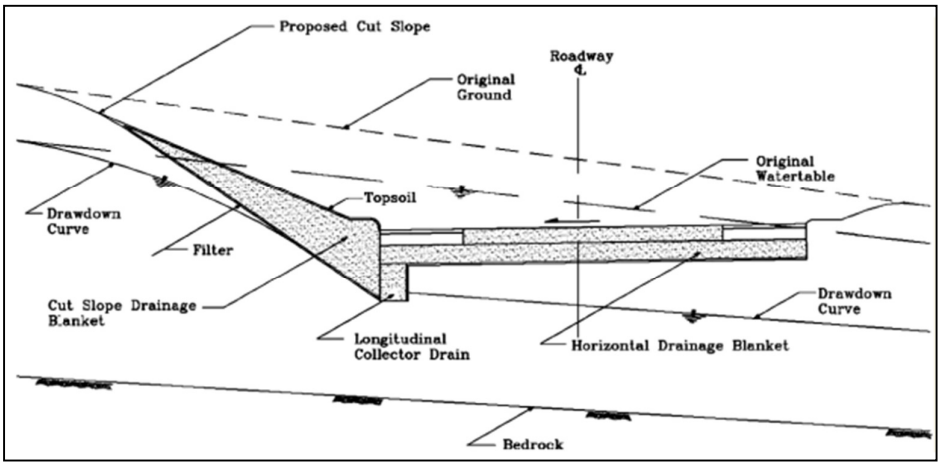


Figure 11-20: Drainage blankets on Cut Slope Drained by Longitudinal Collector Drain

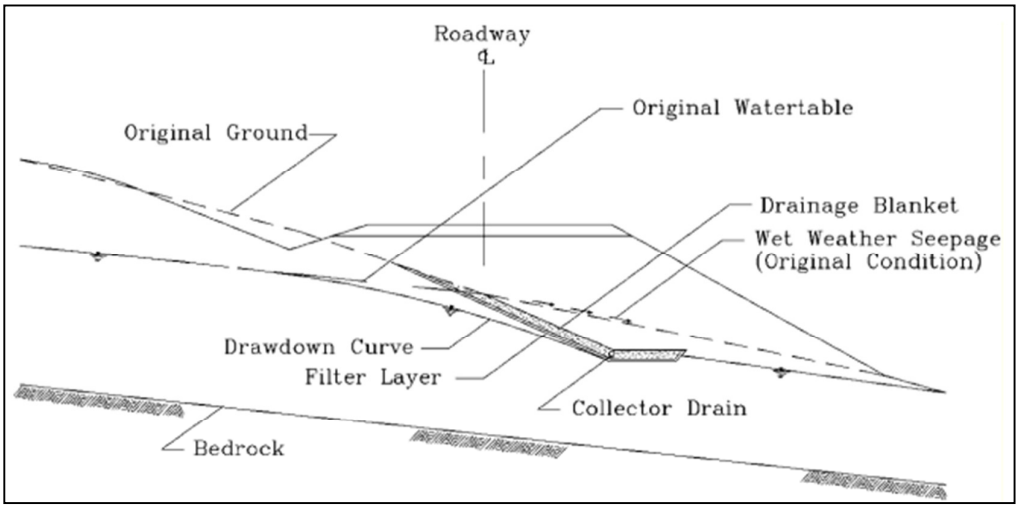


Figure 11-21: Drainage Blanket Beneath Side Hill Outletted by Collector Drain

11.8.7 Interceptor Drains

In many instances hill slopes along the side of road can have ground water seeping from higher ground, which leads to instability of the hill slope in many instances. This ground water is also a source of water for the pavement foundation. An illustration of a field situation near a pavement section with hill slope seepage is shown on Figure 11.21. The soil profile has a bottom boundary layer which is considered to be effectively impervious. The ground water flow toward the highway shows that the water table intersects with the hills lope surface near the road ditch, and ground water is seeping through the slope into the ditch. In addition, ground water is flowing beneath the road and entering into the subgrade and base course material.

Placing an interceptor drain up gradient from the ditch, or beneath the ditch itself, can help to control the hill slope seepage and decrease or even eliminate the flow beneath the roadway, thus removing the source of water from entering into the pavement foundation. An illustration of the situation with an interceptor drain is shown in Figure 11.22. There it

is seen that the water table is drawn down by the interceptor drain to the level of the drain. The water table down gradient of the interceptor drain may rise up above the level of the drain due to seepage flowing under the drain.

The design of an interceptor drain requires an estimate of the hydraulic conductivity of the hill slope soil, k, the thickness of the saturated zone for the ground water, which is shown as height H in Figure 11.21, the slope of the bottom boundary of the soil profile, S, and the height of the drain above the impermeable boundary, Ho . If we want to prevent ground water from entering into the subgrade and base course material, then the interceptor drain needs to be placed at an elevation below the elevation of those foundation layers, as shown in Figure 11.22.

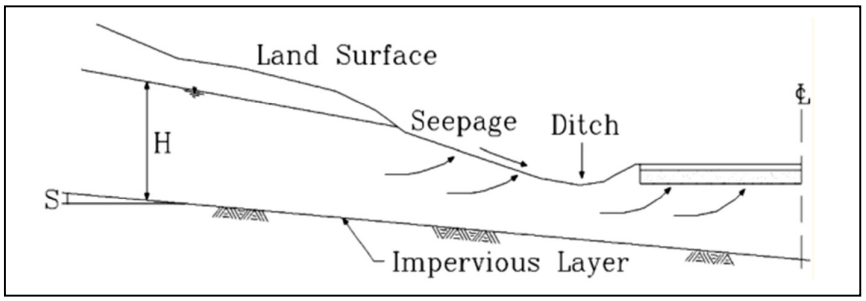


Figure 11-22: Groundwater Flow along a Sloping Impervious Layer Towards a Road

Groundwater seeps through the slope where the water table intersects the land slope, and groundwater flows beneath the payment while also entering the pavement foundation materials.

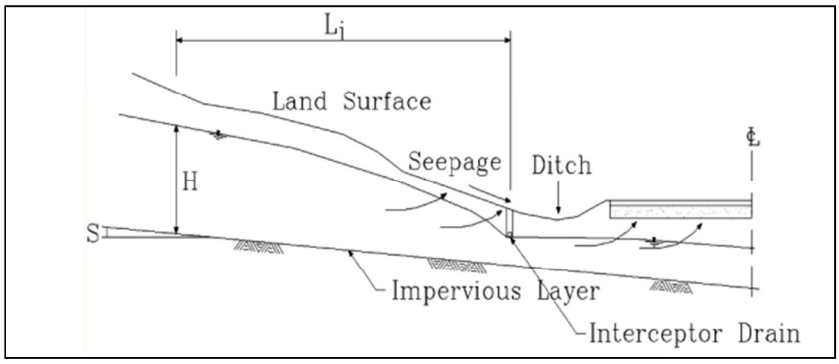


Figure 11-23: The Effect of an Interceptor Drain on Drawdown of Groundwater Table

11.8.8 Well Systems

Under certain conditions, such as potentially troublesome highway slopes, systems of vertical wells can be used to control the flow of groundwater and relieve pore water pressures. When necessary, these systems are pumped for temporal lowering of the water table during construction, or may otherwise be allowed to overflow for the relief of artesian pressures. A common practice is to provide them with some collection system, such as tunnels, drilled-in pipe outlets, or horizontal drains, so they can be drained freely at the bottom.

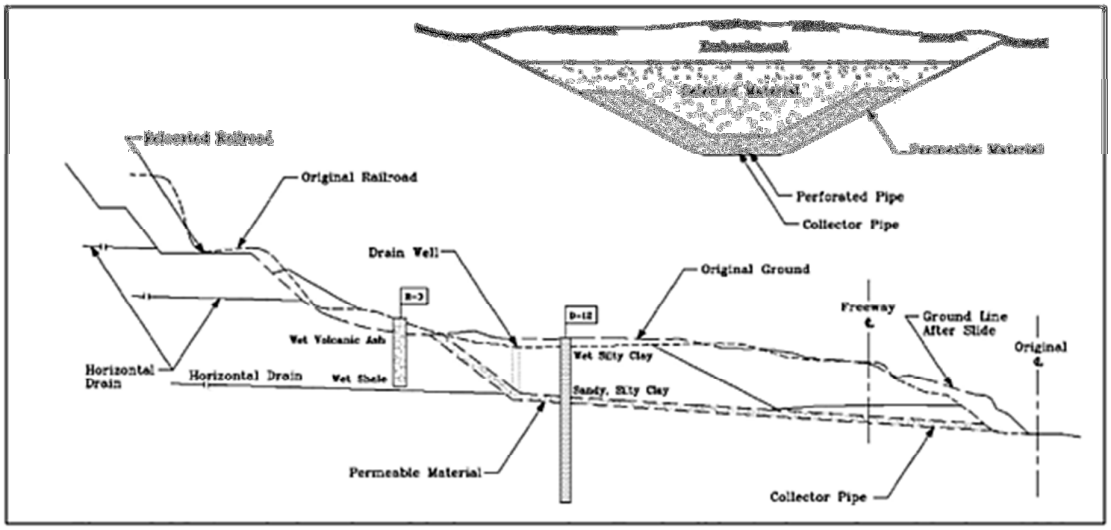


Figure 11-24: A Typical Section of Drainage Trench

Figure 11.23 shows typical well drainage systems that can be used in the stabilization of wet slopes. The sand filled vertical wells can be used for accelerated drainage of soft and compressible foundation materials which are undergoing consolidation as a result of the application of a surface loading (Barron, 1948; Cedergren, 1977).

11.9 Design of Subsurface Drainage Systems

The primary factors to consider in developing design alternatives for both new construction and rehabilitation are:

- **Traffic –** cumulative heavy axle loading measured over the life of a pavement is a necessary factor in the design for subsurface drainage system. This is indicative of potential for damage to the pavement structure from axle loading. Total traffic volume must be accounted for in the geometric design of the road;
- **Soil characteristics –** are key factors influencing design features for reduced moisture damage. Strength, deformation, gradation, and permeability properties of the subgrade soil influence pavement design and the need for subsurface drainage;
- **Climatic conditions –** such as rainfall, freeze-thaw, and daily and seasonal temperature cycling all influence subgrade soil and pavement layers, and are therefore important in selection of alternate designs. Pavements located in regions with little or no rainfall and no freeze-thaw will not require subsurface drainage;
- **Construction considerations –** there is need for an assessment to determine the time required for initial construction, the period before major rehabilitation is necessary, and the frequency of future maintenance. These are important, especially for urban roads and other high-volume routes, where traffic control is costly and lane closure time must be minimized;
- **Cost comparisons –** federal and regional authorities recognize the need for assessing all costs of a highway improvement over a certain design analysis period rather than comparing only the initial costs of construction of different alternatives;

- **Economic analysis –** which compares major costs of a highway improvement over a chosen analysis period must consider initial construction costs, maintenance costs, rehabilitation costs, and road user costs; and
- **Maintainability –** of the system and expected performance should be considered.

These primary factors are of paramount importance in an engineering analysis of design alternatives. However, there is need to consider additional factors that are specific to subsurface drainage, including:

- Type of construction (new or a rehabilitation of an existing structure);
- How similar subsurface drainage designs in the area are performing;
- Success of local contractors in constructing drainage design alternatives;
- Characteristics of surface drainage;
- Type of pavement and other design features;
- The quality of local materials;
- Condition of the Pavement for retrofitting edgedrain design; and
- Topography.

It is therefore critical to consider the interrelationship between subsurface drainage, other design features, and specific pavement performance if optimal pavement design is to be obtained.

11.9.1 Data Requirements for Subsurface Drainage Design

It is important to note that data requirements for design of subsurface drainage systems will vary depending on whether the system is a retrofit in an existing pavement, or installations in a new pavement are to be constructed, or if it is a re-construction. In the situation where an existing pavement is experiencing drainage problems, the most common type of drainage system used for retrofits have been pipe edge drains placed in geotextile wrapped trenches (Mathis, 1990). Retrofit longitudinal edge drains can be grouped into three basic types known as pipe edge drains, refabricated geocomposite edge drains (PGED), or fin drains, and aggregated renches, or French drains (ERES, 1999).

The discussion in the following sections will focus on subsurface drainage design requirements for both new and existing pavement systems.

Roadway Geometry

Because many of the geometric design features of a highway can exert some influence on the analysis and design of subsurface drainage, the designer should be armed with as much information as possible on these features before undertaking the work.

Important resources to have at hand are sufficiently detailed profiles and cross-sections, which permit assembly of the following data for each section of roadway under consideration:

- Longitudinal grades;
- Transverse grades (including super-elevation);
- Widths of pavement and shoulder surface, and the base and subbase;
- Required thickness of pavement elements based on normal structural design practice for the particular area under consideration;
- Depths of cuts and fills;
- Recommended cut and fill slopes; and
- Details of ditches and other surface drainage facilities.

Design of Edge Drain Collector System with Outlet Pipe

Edge drains are perhaps the most effective subsurface drainage systems for removing water infiltrating joints and cracks in PCC pavements (Jeffcoat et al., 1992). Since the effectiveness of any system can be highly site specific, it is essential that careful evaluation of site conditions be carried out when considering retrofitting edge drains because addition of edge drains in areas with highly erodible subgrade or base material may accelerate erosion problems. This is due to the fact that the fines can be lost through the edge drains (Gulden, 1983).

The longitudinal edge drains, when used in existing pavements, just like those installed during initial construction, can be grouped into three basic types known as pipe edge drains, PGED or "fin drains," and aggregate trenches or "French drains." Refer to Figures 11.24 to 11.26 below.

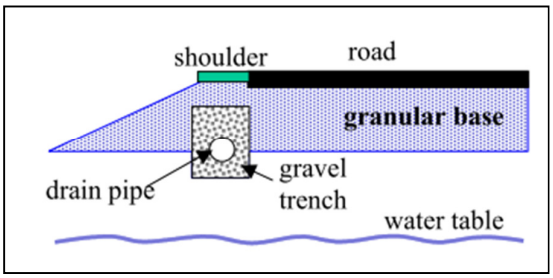


Figure11-25: Schematic of Edge Drain

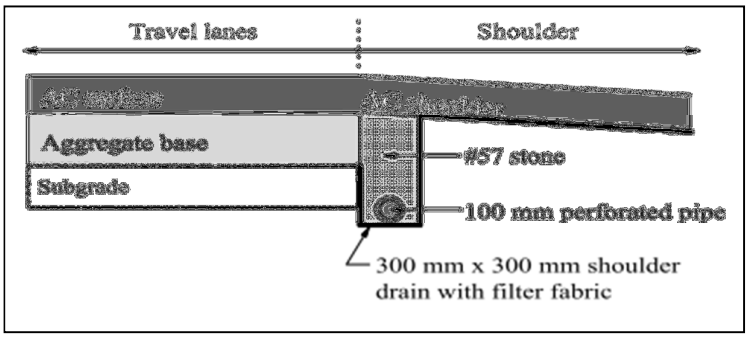


Figure 11-26: Typical AC Pavement with Pipe Edge Drains

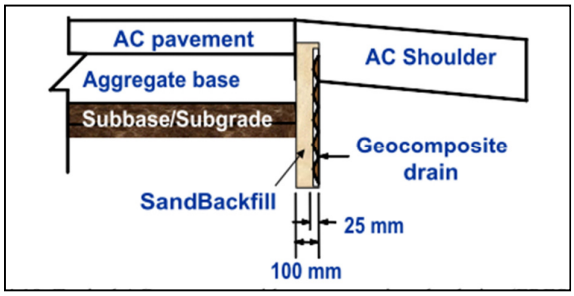


Figure 11-27: Typical AC Pavement with Geocomposite Edge Drains

Providing an AC layer of adequate thickness above the permeable base is essential for obtaining good performance (Yu et al., 1998b). Dense-graded bases that are day lighted have been determined not suitable for providing drainage to newly constructed or reconstructed AC pavements. However, day lighting of the dense-graded bases will provide positive drainage, and would hence be far superior to bathtub design (Kersten and Skok, 1968).

Another advantage of DGAB is that a day lighted permeable base is able to breathe, thus preventing build-up of water vapour pressures under the AC surface from hydrogenesis (Fehsenfeld, 1988). Asphalt concrete pavements with granular bases are particularly susceptible to hydrogenesis, which can lead to stripping (Hindermann, 1968).

Edge Drain Capacity and Outlet Spacing

The goal of installing subsurface drainage systems in pavement structures is to remove water entering the base and subgrade layers as quickly as possible. It is imperative that the edge drain capacity should be designed so as not to be an impediment to the removal of water. A common recommendation is that the capacity of the edge drain system should always increase as the water flows through the system (FHWA, 1992). This would be accomplished if the combination of edge drain capacity and outlet spacing are adequate to handle the design flows. Refer to Figure 11.28 for Typical Subsurface Drain Outlet.

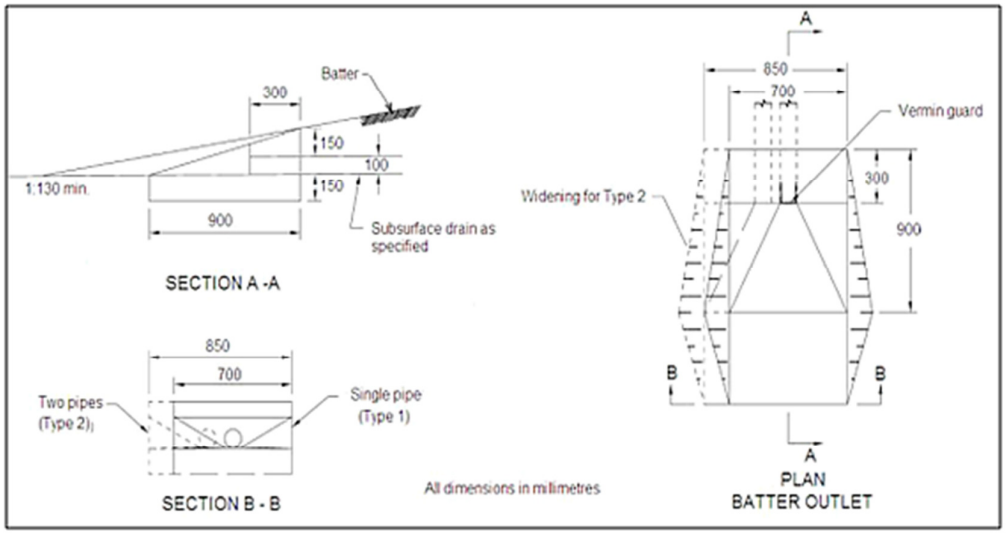


Figure 11-28: Typical Subsurface Drain Outlet

(Source: Adapted from VicRoads, 2005)

The required pipe capacity and outlet spacing can be determined by one of three design approaches. These are:

- The pavement infiltration discharge rate (q_i);
- Permeable base discharge rate; and
- Time-to-drain discharge rate.

The engineer needs to select the design approach that meets the field conditions. The design pipe flow for this approach is determined by the following equation.

$$Q_p = q_i WL \qquad (11\text{-}4)$$

Where:

Q = Discharge flow rate for pipe flow, cu. m/day
q_i = Pavement infiltration, cu m /day/sq m
W = Width of permeable base, m
L = Outlet spacing, mt

To determine the required pipe flow, the design discharge rate from the permeable base need to be adjusted. The resulting equation is:

$$Q_p = kS_R HLcos(A) \quad (11\text{-}5)$$

Where:

Q_p = Design flow rate for pipe flow, cu m/day

k = Coefficient of permeability, m/day

S_R = Resultant slope, m/m

H = Thickness of base, fm

L = Outlet spacing, m

A = Angle between roadway cross slope and resultant slope.

In the time to drain discharge rate approach, the edge drain system is required to be capable of handling the flow generated by draining the permeable base. The pipe flow rate is determined by the equation below:

$$Q_p = (WLHN_c U)\left(\frac{1}{t_D}\right) * 24 \quad (11\text{-}6)$$

Where:

Q_p = Design flow rate for pipe, cu. m/day

W = Width of permeable base, m

L = outlet spacing, m

H = Thickness of base, m

N_e = Effective porosity, %

U = Percentage drained, expressed as a decimal

t_D = Drainage time period, hours

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APPENDIX 11A - WORKED EXAMPLES

Example 11A-1

Given a hypothetical PCC pavement designed with a permeable base drainage width of 0.32m, determine the design pavement discharge rate required to be removed by edge drains using the pavement infiltration discharge rate method.

Solution

A pavement infiltration rate of 0.0113 $m^3/day/m^2$ is selected for the PCC pavement. The design pavement discharge rate is calculated as follows:

qd = qiW = (0.0113 $m^3/day/m^2$)(24m) = 0.271 $m^3/day/m$

The spacing between outlets for this system can be determined once the capacity of the drain is computed. This is done in Example 11A-2.

Example 11A-2

A PCC pavement is being designed for a collector road. The proposed pavement section consists of a 0.1524 m permeable base with a coefficient of permeability of 457 m/day. The resultant slope is 0.0061 m/fm, and the angle between the roadway cross slope and the resultant slope is 10^o. The longitudinal edgedrain is 10.16cm (0.1016m) on a slope of 0.00122m/m. Given these conditions, determine the design pavement discharge rate using the permeable base discharge rate approach, and determine the spacing of drain outlets.

Solution

For this structure design pavement discharge rate can be determined as follows:

$q_p = kS_R$ Hcos (A) = (457 m/day) (0.0061 m/m) (0.1524) cos(10)= 0.419 $m^3/day/m$

The flow capacity of an edge drain, which is a circular pipe, can be determined by Manning's equation assuming the pipe is flowing full with no back pressure:

$$Q = \frac{1}{n} D^{2/3} S^{1/2}$$

Where:

Q = Pipe capacity, m^3 /day.
n = Manning's roughness coefficient.
D = Pipe diameter, inches.
S = Longitudinal slope, mm.

The flow capacity is also just equal to

$Q = q_d L$

thereby allowing us to compute the outlet spacing L . The following are suggested values of Manning's roughness coefficient (FHWA, 1994):

Smooth pipe: n = 0.012

Corrugated pipe: n = 0.024

For this example we use the corrugated pipe (n=0.024). The flow capacity of the pipe is then

$$Q = \frac{1}{0.024} 0.1061^{2/3} 0.0061^{1/2} = 1.54 m^3/day$$

The spacing between the outlets is then

$$L = \frac{Q}{q_d} = \frac{1.54}{0.272} = 5.68m$$

Example 11A-3:

Calculation of flow rate to interceptor drain.

Known:

Height of the water table upgradient of the drain is (H) = 3.048 m.

Slope of the bottom boundary of soils (S) = 0.0122 m/m

Height of the drain above the impermeable barrier (H_o) = 1.22 m

Hydraulic conductivity of the soil (K) = 0.61 m/day

Solution

First, we calculate the length of influence (L_i)

$L_i = 3.8 (H-H_o) = 3.8 (3.048 -1.22) = 1.828$ m

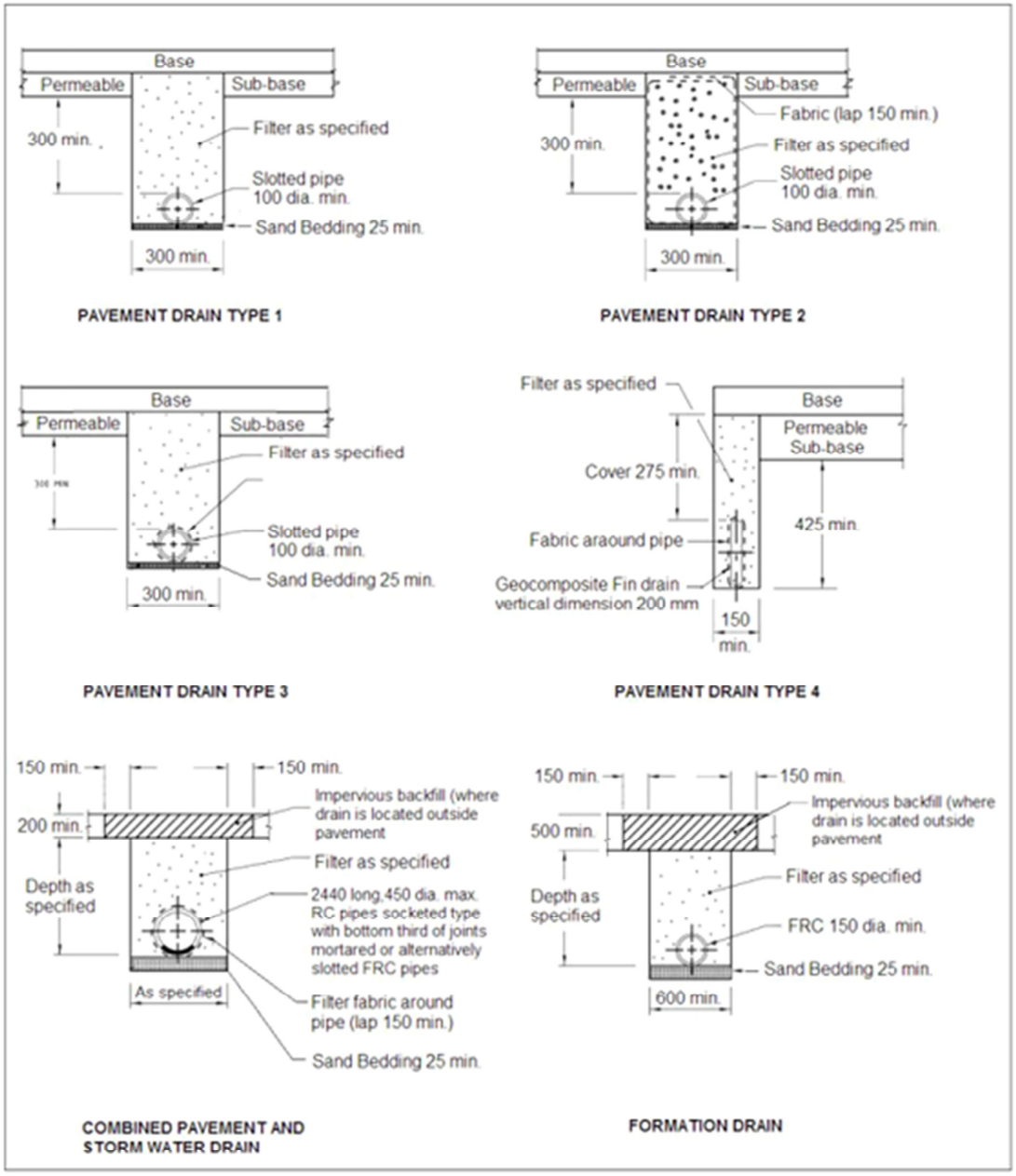
$$The\ ratio \left(\frac{H_o}{H}\right) = \frac{1.22}{3.048} = 0.4$$

$$The\ ratio \left(\frac{SL_i}{H}\right) = \frac{(0.0061)(1.828)}{3.048} = 0.0004$$

and this result leads to

q_d = (5.5)(k)(H)(S)= (5.5)(2)(3.048)(0.0061) = 2.05 cu.m/day linear meter of the interceptor drain. With this flow rate we can determine the required diameter and the grade for the interceptor drain.

APPENDIX 11B – CONSTRUCTION DETAILS OF SUBSURFACE DRAINAGE TYPES



Source: Adapted from VicRoads (2005)

12 CONSTRUCTION

12.1 Project Management

In most highway agencies, different personnel perform the design and construction functions. Adequate communication between the two is essential. Design personnel shall be aware that there are construction related design considerations and construction personnel shall be aware that there are design-related construction considerations. Thus it is very important to have good communication between design and construction personnel.

A communication strategy should be put in place with clear lines of communication between the contractor and designer. This ensures that the correct personnel are advising on specific issues and a level of accountability goes with this strategy which negates the need for making ad-hoc decisions on site. Regular communication and meetings, clear agreed objectives and agreed methods of working (method statements) are recommended to build and enhance relationships and trust between the designer and contractor. For the highway designer for instance, this will continue throughout the project period with input from the various associated specialists, hydrologists, geo-technical engineers, environmentalists – ecology etc.

At the pre-construction, an example of this would be say during the route selection, where the hydrologist, geo-technical engineer and environmentalist would advise on the constraints on the various route selections based on their specific discipline. During pre-construction and construction period when the contractor is appointed, this communication strategy will be widened to incorporate the contractor and their various personal to ensure the objectives and agreed methods of working are understood and agreed at the outset.

12.1.1 Construction Considerations

Construction related hydraulic considerations are a necessary part of the planning and design phases. Factors which will affect construction timing and methods need to be kept in mind as project development proceeds. Those responsible for contract administration and actual construction may need to coordinate their scheduling and construction procedures with the designer in order to achieve results intended. Any special or unique construction requirements shall be communicated to the designer prior to the final design phase of the project.

The designer shall be present at the pre-construction conference to explain special features and planned construction phasing where these considerations are necessary to the proper functioning of the design. It may be advisable and necessary to specify certain time limits and special instructions as to how the work will be accomplished. Phased construction to accommodate seasonal variations, floods, fish passage, irrigation, etc., may be needed. In addition, the need for special considerations related to needed temporary work, detours, and public safety issues can be outlined and discussed. It shall be emphasized by the designer that any and all revisions of drainage designs as contained on construction plans shall be discussed with the designer prior to execution.

12.1.2 Cost Considerations

Cost is an important consideration in any design. The primary components of first costs are related to materials and construction. Future maintenance costs are an important related design consideration. The designer must achieve the proper balance of material and construction costs.

Ordinarily, material costs are optimized by: using available materials in a consistent manner; recycling materials; research programs to identify (continued) potential construction materials, and how they may be utilized efficiently; using reasonable safety factors in design; and encouraging and allowing alternatives where possible. In some cases, the least expensive material may not be the proper choice. The reason for this is that costs are greater than for a more expensive material or future maintenance/replacement costs override the material cost advantage. Climate Change should also be taken into account with respect to predicted increases in rainfall within Ethiopia.

Construction costs are affected by the:

- Relative difficulty of construction;
- Laws, rules, and/or regulations governing construction procedures;
- Degree of competition among contractors;
- Construction latitude allowed by the specifications;
- Quality of the construction plans; and
- Degree of supervision and inspection.

The choice of a more complicated and expensive construction procedure may be acceptable if it allows for the use of more economical materials which decreases maintenance costs, and eliminates or reduces the need for replacements.

12.1.3 Construction Plans

The designer must be aware of the above relationships and how they affect costs, and consider them in design. Additionally, construction plans should reflect these considerations by containing:

- Suggested construction sequences that consider construction costs and environmental considerations as well as public convenience;
- Subsurface soil borings;
- Complete descriptions of utilities; and
- Consistent plan format which enhances the contractor's ability to assimilate and understand the plans.

Despite the best of efforts, construction changes occasionally occur. The designer shall be consulted when these changes may affect the proper functioning of the drainage facilities. The post-construction inspection following completion of the project should document any deviations from the original plans as well as an initial assessment of the hydraulic performance. Construction personnel shall be encouraged to inform the designer of any design-related difficulties which are encountered and suggestions to improve future designs. Changes must be incorporated into "as-built" plans for future reference.

Plans shall be checked to verify that site conditions have not changed between location surveys and construction. Meander migration, bank caving, aggradation, head cutting or other natural or man-induced changes in the channel may have occurred which would require that the designer reconsider decisions made on the basis of conditions which were different from those which exist at the beginning of construction. This is best accomplished with a joint field inspection by the designer, contractor, and maintenance personnel. Additional objectives of the inspection are to assure location survey accuracy and to ascertain if the designer has properly visualized existing situations and designed accordingly.

The changed conditions may require river control works, revisions to pier locations and orientation, rearrangement of spans, or other modifications of the design to accommodate the changes that have occurred. Plan changes required because of differences between location surveys and construction field inspections shall be made in consultation with the designer. Some changes could significantly affect either the hydrology or the hydraulic performance of the drainage feature designed for the site.

12.1.4 Effects of Change

Land use changes in the watershed can modify the hydrology and debris considerations used in the design. New development/urban expansion and de-forestation along the project could change damage risk considerations for the design. Dependent upon the time that has elapsed between completion of the design plans and the beginning of construction, changes in land use could significantly affect the validity of design considerations. Commercial mining of materials for construction is a rather common practice that can change flow velocities, volume and character of bed load, and flow direction and distribution at the crossing site.

Land clearing for agricultural purposes may create a need to reconsider the location and size of waterway openings. Land development near the site could change damage risk considerations for the crossing. The designer shall be consulted regarding the need to modify the design at any drainage facility site which has changed significantly from the conditions which existed during design.

Changes in stream alignment and profile can result in different flow conditions than those used in the outfall or cross drain design. Drainage area changes due to diversions or site grading can affect inlet and outlet locations and type, as well as storm drain or roadside ditch designs.

Utilities added after the survey may require extensive redesign of storm drain systems to avoid conflicts; this reinforces the need for good utility surveys prior to design in order to forestall costly redesigns and delays. Other typical changes which may affect drainage design are:

Possible Errors or Omissions

- Incorrect existing structure and/or invert elevations;
- Incorrect drainage area size;
- Channel alignment, profile;
- Unreported utility;
- Existing structure condition as related to service life, outlet scour, siltation, etc.;
- Local flooding not documented;
- Existing slope erosion not reported;
- Sensitive receiving waters not reported;
- Unreported debris problems; and
- Attractive nuisance problems.

Possible Changes

- Increased development/urbanisation;
- Channel improvements;
- Diversion or site grading changes;
- New utility; and
- Loss of outfall due to development.

Miscellaneous

- Incorrect typical section choice, and/or incorrect grade.

In some cases, a considerable amount of time may elapse between design and construction. In other cases, designs may change before construction is begun. Any changes in the plans, specifications, and estimates shall be reflected in the final plans. If questions arise, the construction personnel should check with the designers to determine if changes have been made and how construction should proceed.

12.2 Preconstruction Conference

It is important for the designer to be present at the preconstruction conference to explain special features of the designs and planned construction phasing, where these considerations are necessary for proper functioning of the design. It may be advisable and necessary to specify certain time limits and special instructions as to how the work will be accomplished. The designer should answer any construction-design questions that the contractor's construction personnel have or endeavour to obtain answers as soon as possible.

The designer should go over the job with several key personnel including, resident engineer, contractor's project manager, site manager, surveyors, and others who may have a direct interest in the project. Such a review at this time will aid materially in clearing up reasons for certain design features such as: erosion treatments and there location, outfall locations, materials proposed, foundation treatment; potential slides; environmental commitments; and potential drainage and maintenance problems, including water pollution.

The purpose of the meeting is to discuss the design and construction aspects of the project, thus affording all parties a common understanding of the proposed work and the problems and possible solutions which may be expected.

12.2.1 Other Concerns

Several other concerns shall be discussed at the preconstruction conference including drainage maintenance during construction, water pollution, and erosion control. Drainage work on some projects may be completed several months before total project completion. During this period, vegetative erosion control measures are not well established and maintenance to correct erosion and sediment deposition in the newly constructed channels is important to achieving the results intended. The contractor should provide for maintenance during the term of the contract, require interim protective measures, and/or advance its own maintenance schedule to assure that minor damage will not develop into major damage which will require costly repairs or replacement when it assumes the permanent maintenance responsibility.

During the preconstruction conference, provisions of the contract relating to pollution control shall be reviewed. The contractor should submit a program to control water pollution prior to beginning work (or waive the specified requirement if such action is in order).

12.3 Factors Influencing Construction

Problems may be avoided during construction when important drainage or other water related factors are considered during the location and planning phases of the project. If at all possible, problem locations shall be avoided. A site may be considered a problem location because of geological aspects, environmental concerns, other existing facilities, or other reasons which might conflict with the proposed project.

The concerns of erosion and sediment, where they might occur, and how to control them, must be considered, at least in broad terms, during the early phases of location. As an example, the designer may be involved in the geological investigations because of underground water so that proper measures can be taken to prevent problems before they occur.

The time of the year and total construction time shall be taken into consideration. Certain elements, such as embankments along a stream, shall be completed before the anticipated flood season. The use of temporary structures must also be planned. Often a temporary crossing can be smaller than normal if it is only going to be utilized during the dry months. If it will be used for more than one year, perhaps it needs to be sized for a lower frequency flood.

This consideration may change the concept of the project or at least the type of structure designed. Many constructions related hydraulic problems are related to scheduling. Although these problems will be studied in more detail during the design phase, they shall be initially considered, at least in a preliminary manner, as early as possible. Commitments regarding water resource related items must be made known to the personnel who will be involved in the actual construction.

12.3.1 Environmental Concerns

The designer can work with other disciplines to devise and construct mitigation measures which reduce adverse effects. The designer can recommend locations and sizes for hydraulic facilities such as for culverts, bridges and channels; they may also identify spoil disposal areas and geometry, and various construction alternatives. They may also assist in developing programs for protecting surface waters during construction. These programs include such things as levees and ponds to collect various types and quantities of pollutants including those from construction equipment or which are accidentally spilled. In addition methods to reduce erosion and sedimentation would also be proposed.

12.4 Hydrology

12.4.1 Low Flow Discharges

Construction and maintenance of highways may require knowledge of low flow discharge properties such as discharges, flow stages, flow durations and related flow variables. For example, the construction of a culvert or a bridge may require knowledge of the time frame at which flows are below certain levels, or below certain magnitudes. This knowledge might be useful in scheduling construction (wet or dry season), or designing temporary construction facilities. With some facilities it is often necessary to avoid long periods where the facilities are unavailable to the user due to prolonged occupation of a portion of the facility by frequent low flows.

12.4.2 Timing and Risk

The Ministry of Water & Energy compiles water resources data for gauged streams listing mean daily discharge. Based on these daily records, a low flow analysis may determine an acceptable discharge for the hydraulic design of temporary construction facilities. A rigorous flood frequency analysis is not generally required for these low flow studies. Flow discharges may be quickly determined based on a visual examination of monthly mean discharge data as determined from the mean daily discharge values for all years of record and with consideration given to construction timing and degree of risk.

Acceptable methods that can be used to transfer data from one location to another and hydrologic regression equations are given in Chapter 5: Hydrology of this manual.

12.4.3 Water Quality

Water quality of streams and lakes is a very sensitive issue. Drainage construction may deliver such things as sediment and chemicals to streams, rivers and lakes unless precautions are taken. Annual runoff hydrographs may indicate that very low stream flows will occur during the late autumn and winter months. During these periods even small amounts of additional sediment or chemicals entering the stream from construction areas could be detrimental because of the low dilution effect provided by the receiving waters. The effects of sediment or chemicals due to highway construction during the low flow periods shall be investigated for those sensitive areas such as where stream flow is used for municipal water supply. This investigation may include periods of water quality monitoring and testing. If the investigation concludes that the amount of sediment or chemicals will exceed an acceptable threshold value, the construction periods may have to be rescheduled or mitigation measures taken. This threshold's to be set by Ethiopian Government.

12.5 Erosion, Sediment and Pollution Control

By the time construction begins, all erosion, sediment pollution control measures made during the planning, location, and plan development phases shall be contained in the plans, specifications, and special provisions provided to the for accomplishment of the project construction.

The resident engineer should thoroughly familiarize themself and their inspection staff with the erosion, sediment and pollution sensitive areas of the project and the control measures contained in the plans. This information shall be shared with the contractor for his formulation of a work plan.

The contractor should utilize an erosion, sediment and pollution control schedule which sets out the proposed construction sequences and the measures that will be employed. This schedule allows the contractor and engineering personnel to plan ahead and control erosion, sediment and pollution before it becomes a problem rather than adding measures after damages have occurred.

Adequate inspection during construction is essential for erosion and sediment control. If deficiencies in the design or performance of control measures are discovered, the supervising engineer should take immediate steps for correction, including notification of the designer to avoid a reoccurrence of the problem.

Periodic field reviews and inspections by the design and construction engineers to correct deficiencies and improve control procedures are highly recommended.

12.5.1 Erosion Control

Soil erosion is the process by which the land surface is worn away by the action of wind, water, ice, and gravity. The natural rate of erosion is increased greatly by many urban activities – especially construction activities. Any activity that disturbs the natural soil and vegetation has the potential to increase erosion because bare, loose soil is easily moved by wind or water.

Factors affecting the erosion potential of any site include soil type, geology, vegetative cover, topography, climate, and land use. Physical properties of soils such as particle size, cohesiveness, and density affect its erodibility. Loose silt and sand-sized particles are more susceptible to erosion than clay soils. Rocky soils are less susceptible to wind erosion, but are often found on steeper slopes that are subject to water erosion. When surface cover and soil structure are disturbed, the soil's erodibility potential increases.

Construction activities disrupt the soil structure and its vegetative cover. Vegetation plays an important role in controlling erosion. Roots bind soil particles together, and the leaves or blades of grass reduce raindrop impact forces on the soil. Grass litter and other ground cover traps rain which allows infiltration and reduces runoff velocity.

Vegetation reduces wind velocities at the ground surface, and provides a rougher surface which will trap particles moving along the ground. Once vegetation is removed, erosion can proceed unchecked.

The factors that influence land erosion are:

- Run-off velocity
- Run-off volume
- Soil type
- Vegetation cover
- Machinery and plant

12.5.2 Erosion Control Procedures

The objective of erosion control is to limit the amount and rate of erosion occurring on disturbed areas. Erosion of SUDS techniques will reduce their effectiveness, and add to the silt load that any other technique downstream will have to deal with. Erosion controls are surface treatments that stabilise soil exposed by excavation or grading.

The objectives for erosion control during construction include the following:

- Conduct all land-disturbing activities in a manner that effectively reduces accelerated soil erosion and reduces sediment movement and deposition off site;
- Schedule construction activities to minimise the total amount of soil exposed at any given time to reduce the period of accelerated soil erosion;
- Establish temporary or permanent cover on areas that have been disturbed as soon as possible after final grading is completed;
- Design and construct all temporary or permanent facilities for the conveyance of water around, through, or from the disturbed area to limit the flow of water to non-erosive velocities;
- Remove sediment caused by accelerated soil erosion from surface runoff water before it leaves the site; and
- Stabilise the areas of land disturbance with permanent vegetative cover.

Permanent or temporary soil surface stabilisation should be considered for application to disturbed areas and soil stockpiles as soon as possible after final grade is reached on any portion of the site. Soil surface stabilisation should also be considered for disturbed areas that may not be at final grade but will remain undisturbed for a significant period of time.

Roads and other hard standings should be covered as early as possible with the appropriate aggregate base course where this is specified as part of the pavement.

12.5.3 Erosion Protection Techniques

A number of erosion protection techniques can be used, such as:

- Vegetation, which reinforces the soil due to the binding effects of the root structure. It helps protect areas downstream by the friction effect of the vegetation decreasing the runoff velocity e.g. hollows or pits. Refer to Photos 12.1 and 12.2.
- Geotextiles and erosion control fence and fabrics, which reinforce the soil structure reducing the potential for particle stripping.
- Gravel trenches, which can be located upstream of exposed land. They intercept runoff flow that then enters a perforated pipe system to an outfall or infiltrates into the ground. Refer to Photo 12.3.
- Flat sites or slack gradients within a site, which will help reduce the velocity of the runoff.
- Impermeable area run-off should not be allowed to flow directly over areas of exposed ground.

Some practicable examples with respect to preventing soil erosion during or after construction are shown below.

Example 1: Hollows or Pits



Photo 12-1: Before: Bare Capped Soil and after: Hollows Vegetated and Stabilising Soil

Example 2: Vehicular Access - Track Treatment



Photo 12-2: Before: A Typical Eroded Track and after: Mulch on the Track to Prevent Erosion and Establish Plant Cover

Example 3: Gravel Trench to Stop Erosion at Source



Photo 12-3: Gravel Trench Edge of Highway and Gravel Trench on Embankment

12.5.4 Sedimentation

During a rainfall event, runoff normally builds up rapidly to a peak and then diminishes. Because the amount of sediment a watercourse can carry is dependent upon the velocity and volume of runoff, sediment is deposited as runoff decreases. The deposited sediments may be re-suspended with subsequent runoff. In this way, sediments are moved progressively downstream.

Wind-blown silt and sand particles are deposited whenever the force of the wind lessens. Much of the wind-eroded material is deposited behind fences, in landscaped areas or downwind of buildings or other obstructions to the wind. Materials transported by bouncing or creeping along the surface are often trapped in surface irregularities near the point of initial movement.

12.5.5 Sediment Control Procedures

Sediment entrapment facilities are necessary to reduce sediment discharges to downstream properties and receiving waters. Sediment entrapment facilities include sediment control fences, stone gabions to trap large volumes of silt and stilling ponds to accommodate large volumes of storm water runoff heavily laden with silt. The type of sediment entrapment facility to be used depends on the catchment area and site slope.

All runoff leaving a disturbed area should pass through a sediment entrapment facility before it exits the site and flows downstream. A silt fence is made of a woven synthetic material, geotextile, and acts to filter runoff.

Silt fencing can be placed as a temporary barrier along the contour at the base of a disturbed area, but is not recommended for use in a channel or swale. The material is durable and will last for more than one season if properly installed and maintained. Silt fencing is not intended to be used as a perimeter fence or in areas of concentrated flow. If concentrated flow conditions exist, a more robust filter should be considered. Refer to Photos 12.4 to 12.8.

Example 4: Sediment Silt Fences and Stone Gabion



Photo 12-4: Fences to Trap Sediment within Overland Flows



Photo 12-5: Fences with Mulch to Check Overland Flows and Restrict Sediment Flow



Photo 12-6: Wire Netting and Mulch used to Check Overland Flows



Photo 12-7: Stone Gabions

Example 5: Stilling Pond



Photo 12-8: Cascading Stilling Ponds to Remove Silt from Surface Water Runoff

12.5.6 Pollution Control

Pollution Prevention

The main requirements are to control surface water runoff and pumped water from sites, for example by the use of settling tanks, to ensure that it does not pollute controlled waters. The safe storage of materials and fuels is also important so that if spills occur they are contained (by the use of berms, check ditches or other techniques) and do not cause a pollution incident.

Before mobilisation, the site layout should be planned to consider fully issues such as the location of stockpiles, fuel stores, storage areas, waste disposal, refuelling points, wash down areas, etc. These should be located in areas where they are least likely to affect controlled waters or infiltrate groundwater. Planning should also address subjects such as the diversion of watercourses, prevention of upstream runoff entering the site and the design of haul roads, including the use of road bridges over watercourses to stop vehicles driving through streams and rivers.

An environmental plan should also be put in place. The plan should include: the environmental risk assessment with control measures, location of foul drainage disposal routes, location of surface water systems that discharge into watercourses, requirements for discharge and abstraction permits, location of spillage kits and an action plan in the event of an environmental incident.

12.5.7 Pollution Sources and Prevention

Suspended Solids

Suspended solids are one of the major sources of construction site pollution. The following list indicates a number of sources and the measure that can be taken to help prevent pollution:

- Excavated ground and exposed ground. The effect of having no vegetation and being recently disturbed allows for relatively low velocity run-off to erode the surface. To help prevent the pollution from entering a watercourse, silt fences or stilling ponds should be placed downstream. To limit the volume of run-off reaching the exposed ground, runoff diversion or interception devices should be placed upstream.
- Stockpiles. The effects of erosion on a stockpile will depend on the type of material being stored. Fine sand and topsoil stockpiles will be eroded far more readily than heavy granular materials. Stockpiles should be located away from a watercourse or site drainage system. Protective coverings will help prevent runoff stripping the stockpile.
- Plant and wheel washing. Plant and wheel washing should take place in designated locations. The area should be tanked and should not be allowed to discharge into a watercourse or infiltrate groundwater, as the wastewater from these devices is highly contaminated with silts, sands, and hydrocarbons. Some proprietary vehicle washing systems offer a recycling facility, which filter and settle solids, with the effluent being pumped back into the system. The solid waste materials from this process need to be treated as contaminated waste due to the high hydrocarbon content.
- Haul roads. The runoff from haul roads contains a large amount of suspended solids as well as hydrocarbons. Haul roads should be designed so that the length is kept to a minimum, but still serves it purpose. The gradient should be shallow to prevent increasing runoff velocity and, if possible, bunds and/or discrete ditches constructed to intercept the runoff. Haul roads should be sprayed regularly to keep down dust. If any section of a haul road is hard surfaced, then it should be swept on a regular basis to prevent the accumulation of dust and mud.
- Disturbance of riverbeds or banks. Excavation of riverbanks or beds can generate silty water, as the excavated and exposed material is washed downstream. The amount of such excavation needs to be limited and, if undertaken, the water area downstream needs to be protected by booms. For larger projects, consideration should be given to diverting the river during excavations.

- Dewatering operations. Groundwater discharge as a result of excavation activity is likely to be heavily polluted with suspended solids, and should not be discharged directly into a watercourse. To help reduce the amount of suspended solids within the runoff a number of techniques can be adopted:
  - Passing the discharge water over a grass area;
  - The discharge velocity has to be monitored and kept sufficiently low to promote settlement;
  - Passing the discharge water though a temporary gravel strip;
  - Controlled use of skips and/or tanks to act as stilling basins; and
  - Controlled use of stilling ponds.

Oils and Hydrocarbons

The use of oils and hydrocarbons on construction sites provide an inherent risk of leakages and spillages leading to pollution incidents. Table 12.1 below details the potential sources of oil pollution.

Table 12-1: Sources of Oil Pollution

Sources	Potential problem initiators
Storage tanks	Leaking valves. Leaking pipe work. Corrosion. Frost damage. Vandalism. Leaking bund. Removal of waste.
General operation and maintenance	Refuelling. Leaking pumps, browsers, generators, plants, machinery. Disposal of waste oil.
Accidents/incidents	Spillages (greatest risk at refuelling). Over-turning (drums and buckets). Mechanical failure e.g. rupture of pipes. Inadequate bunded area. Vandalism.

Simple measures can be taken to prevent oil and hydrocarbons becoming pollutants, such as:

- Maintenance of machinery and plant;
- Drip trays;
- Regular checking of machinery and plant for oil leaks;
- Correct storage facilities;
- Check for signs of wear and tear on tanks;
- Care with specific procedures when refuelling;
- Designated areas for refuelling ;
- Emergence spill kit located need refuelling area;
- Regular emptying of bunds;
- Tanks located in secure areas to stop vandalism; and
- Booms installed on watercourses.

12.5.8 Construction Feedback

An important consideration in the decision to utilize any erosion, sediment and pollution control measure is its effectiveness in the particular circumstances of planned use. There is no better way to answer this question than through experience. For this reason it is very critical to the development of good erosion, sediment and pollution control program that communication exists between design and construction personnel. One method of establishing communication is to have regularly scheduled project field reviews or meetings involving those responsible for design and construction. During these meetings, problems and successes with particular items can be evaluated. Different ideas and procedures that have been successfully employed by a contractor can be studied to determine if they warrant consideration for widespread use. Also of importance for discussion is possible modification to standard design items that would facilitate their construction and/or perhaps reduce their cost.

This feedback procedure extends beyond construction into the long term maintenance of erosion, sediment and pollution related items. Drainage maintenance personnel must check and correct any deficiencies in the permanent erosion, sediment and pollution control measures. Design personnel shall be apprised of any persistent problems so that an analysis can be made to determine if any alteration of design or construction practices is warranted to reduce maintenance problems.

12.6 Culverts

12.6.1 Preparation

The plans, specifications, and other construction documents shall be reviewed to ensure that the design fits current site conditions. Design personnel shall be informed and involved in all changes to the plans and specifications.

As soon as final locations are determined, furnish the contractor a revised culvert list, including those culverts which have been added or altered by change order.

12.6.2 Installation

Assembly or construction, bedding and backfill are as important to culvert service as the hydraulic and structural design. Culverts shall be protected from damage during construction operations and shall be periodically inspected. A particularly critical time for inspection is upon completion of grading operations and prior to the start of surfacing operations. It is as important to inspect culverts which are not under the roadway as it is for those structures that are under the roadway. Prior to the acceptance of the installation, all culverts shall be inspected and cleaned as necessary.

12.6.3 Records

Records shall be kept of the construction of each culvert installation. The final location and slope of the culvert shall be recorded on the 'as-built' plans. This information is useful for evaluating overall performance of the installation. The following records shall be kept for each installation:

- Inspection tags;
- Location and layout including:
- Station;
- Skew(s);

- Location of inlets, outlets, junctions;
- Flowline elevations in inlets, outlets, junctions, camber;
- Alignment;
- Grade;
- Daily reports;
- Structure summary sheet containing:
- Measurements;
- Calculations; and
- Pay quantities.

Any deviation from the original design must be reported to the designer, as any change may have a significant impact on the operation of the culvert.

12.7 Bridges

12.7.1 Hydraulic Considerations

In some cases, the Ethiopian Roads Authority (ERA) may include construction-related details in the plans and specifications, in order to mitigate potential environmental effects or to assume or reduce the risk of failure during construction. In addition, other special provisions related to the construction phase of the bridge construction may be specified in the plans. Hydraulic considerations during construction usually differ from the design considerations for the completed facility.

The risk and responsibility that lay with the ERA will be in accordance with the Conditions of Contract and relevant proclamations.

12.7.2 Hydraulic Information

A hydrograph of superimposed mean daily flows and a plot of the rating table for a stream gauging station near the crossing site are useful for the design of cofferdams, false work and temporary crossings, in scheduling of the work, and in selecting the location of work and material storage areas.

In the event a gauging station is not located near the stream crossing site, records from upstream or downstream gauges may be useful as an indication of the usual magnitude, duration and time of flood events.

12.7.3 Foundation and Scour

Any site conditions that might impact on the foundation design or create unusual scour problems shall be discussed with the designer to determine if design changes are required. If specific elements of the bridge design are dependent on special foundation or scour considerations, construction personnel shall be informed so they can identify possible problems. (Refer to Photos 12.9 and 12.10 for scour and its impact on bridge abutments).




Photo 12-9: Scour Mechanism in Action Photo 12-10: Result of Scour

12.7.4 Coffer Dams, Caissons and Falsework

Cofferdams, falsework, and occasionally contractor's equipment, such as barges, constrict the stream channel more than the completed substructure and consequently have greater potential for causing scour and bank caving, and for collecting debris. Scheduling of work to avoid flood seasons is especially important if these types of operations will be involved.

12.7.5 Crossings and Detours

Temporary stream crossings necessary for the construction of bridges are to be designed by the contractor and checked by the Engineer. Design of such crossings shall be to a level to minimize or mitigate the adverse effects on the stream environment, to facilitate securing permits, or to reduce risk and thereby reduce construction costs.

The risk and responsibility that lay with the ERA will be in accordance with the Conditions of Contract and relevant proclamations.

In addition, stream crossings for detours are built to much lesser standards than permanent crossings (Refer to Photos 12.11 and 12.12). The criteria used for the hydraulic design of detour stream crossings shall be based on risk factors which shall be evaluated considering the probability of flood exceedance during the anticipated service life of the detour (the construction period for the crossing), the risk to life and property and traffic service requirements. Figure D-1 can be used to assist in designing temporary stream crossings and detours.



Photo 12-11: Temporary Stream Crossings



Photo 12-12: Temporary Stream Crossings – Repaired as and when washed away

As in the case of the design of highway stream crossings, detour designs should accommodate floods larger than the event for which they are designed in order to avoid undue liability for damages from excessive backwater and to reduce the probability of losing the detour stream crossing structure during a larger flood. In most instances, the conveyance of floods larger than the detour design flood is provided for by a low roadway profile which allows overflow without creating excessive velocities or backwater.

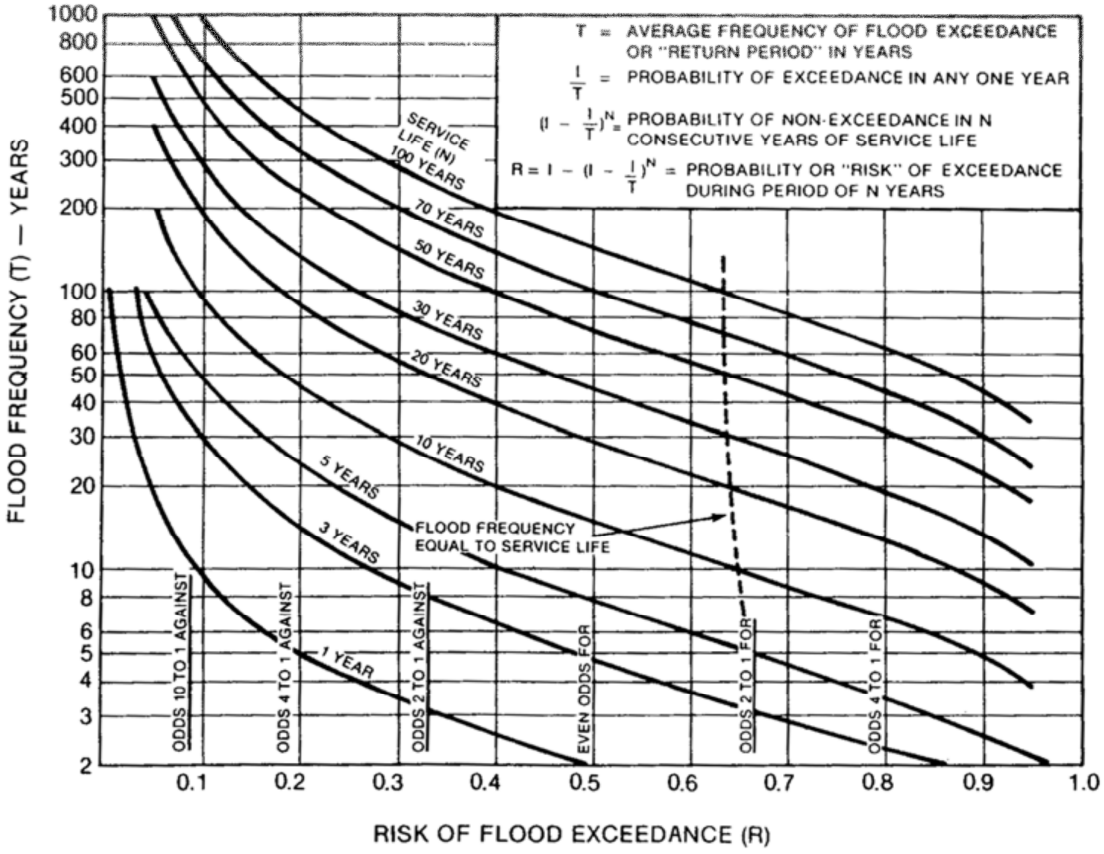


Figure 12-1: Probability or Risk of Exceedance of a Flood Event vs. Service Life of a Highway Encroachment

12.7.6 Environmental Aspects

Minimum disturbance of the banks and bed of a stream during the construction period will reduce erosion damage to the banks, sedimentation and harm to fish and wildlife. Embankments in or along streams shall be constructed of erosion resistant material and/or protected against erosion to avoid adverse sediment concentrations which contribute to the turbidity of the stream.

Consideration shall be given to precluding in-stream operations that would cause turbidity during the spawning season of certain types of fish. Detours and construction roads are other sources of turbidity and should either be constructed at a time that fishery activities will not be disturbed, or provisions shall be made to control any harmful effects of erosion.

Silts and clays will generally flush out of the substrate over a period of time but sands tend to become embedded. Gravel and rock similar to the gradations found in the existing substrate will do the least damage to the aquatic habitat. Pumping of cofferdams and other dewatering operations may have a discharge of unacceptable quality to the receiving stream. Mitigation measures such as stilling ponds may be necessary if the ecosystem of the stream would be upset by the temporary degradation of water quality.

12.7.7 Feedback

Designers are required to participate in the construction of the works that they have designed. Designs can be improved upon for construction purposes when the designer is informed of the deficiencies. Designers are required to visit construction sites to discuss problems with designs and possible improvements in future designs. This is especially important for major projects like bridge construction. Upon completion of a project, a design critique conducted jointly by designers and field personnel can be a very useful learning experience for both. This critique should include difficulties encountered in the construction and possible design changes to prevent such difficulties in the future.

12.8 Open Channels

12.8.1 Bank Stabilisation

Bank stabilization is an important aspect of open channel construction. Since in some cases a considerable length of a stream or channel system may be disturbed by construction, great care shall be exercised in scheduling and implementing stabilization measures. Immediately prior to the commencement of construction of bank stabilization measures, the site manager/resident engineer should inspect the site to ensure that measures proposed are not inappropriate because of bank movement subsequent to completion of design surveys.



Photo 12-13: Bank Stabilisation

Recognizing that an entire reach of stream may require stabilization, highway responsibility may well be much more limited in scope and a total solution not possible.

12.8.2 Excavation

Channel excavation work on some projects may be completed several months before the project is completed. The time between completion of channel excavation and total project completion is usually longer when grading and structure projects are separated from the contract for paving or stabilization. During this period, vegetative erosion and control measures are not well established and maintenance to correct erosion and sediment deposition in the newly constructed channels is important to achieving the results intended. The construction contract should provide for maintenance by the contractor during this period of the contract, where interim protective measures, are put in place to assure that minor damage will not develop into major damage which will require costly repairs or replacement when it assumes the permanent maintenance responsibility.

12.8.3 Access

Damaged channels can be both expensive to repair and hazardous to traffic. To facilitate repair and maintenance, channels shall be designed recognising that periodic maintenance, inspection and repair will be required. Where possible access shall be incorporated for personnel and equipment during the construction period and afterwards.

Consideration shall be given to the size and type of equipment which will ordinarily be required in assessing the need for access easements, entrance ramps and gates through right-of-way.

12.9 Subsurface Drainage

12.9.1 Plan Revisions

Plans for subsurface drains are seldom as complete as those for culverts. The discovery of damaging amounts of groundwater during preliminary materials investigation is difficult. During dry seasons, or following a long, dry cycle, indications of groundwater problems may be missing entirely. However, with the return of a wet season serious problems may occur if needed subsurface drains are not installed. Therefore once a horizontal and vertical route with respect to the alignment of a highway has been agreed a site investigation along

this route should be undertaken and groundwater profiles monitored a minimum 18 months before pre and post construction to assess the impact on the makeup of the highway.

Installations shall be carefully reviewed and plans revised as necessary to fit field conditions in consultation with the designer. It is seldom necessary to decrease the number of planned subsurface drains, the contrary is usually the case. Also the location of subsurface and other drains may need to be changed to locate these facilities in stable areas and at low points or other locations where the drainage of surface water can be intercepted and allowed to efficiently enter the storm drain system.

During the clearing and grading operations, groundwater problems may become evident. Swamps, bogs, springs, and areas of lush growth are possible indicators of excess groundwater. Hence the need for a geo-technical along the alignment of the highway to inform the design process and mitigate any problems prior to arriving on site.

12.10 "As Built" Plans

“As Built” plans serve many functions related to the design and construction process including documentation of the final location of all elements of the drainage system and related facilities, any changes that were made in the design during the construction process (i.e., size of facilities, materials used, addition or elimination of facilities), and any variation between the original plans and specifications and the final installed facilities.

The completion of accurate and complete “As Built” plans can be invaluable in documenting changes that can be incorporated in future designs and to future investigations of the project if problems are encountered or there is some need to analyse the facilities performance. Possible future legal action makes the documentation of “As Built” plans very important.

12.11 Temporary Hydraulic Facilities

Temporary hydraulic facilities include all channels, culverts or bridges which are required for haul roads, channel relocations, culvert installations, bridge construction, temporary roads, or detours. They shall be designed with the same care which is used for the primary facility.

These designs are to be included in the contract documents for the project. Approval is required from the Engineer. Temporary facilities shall be designed by the Contractor except in exceptional circumstances.

12.11.1 Detours and Temporary Roadways

Drainage systems for these shall be designed for a two-year frequency if the roadway is required for a year or less and a five-year frequency if required for longer than a year. All other temporary hydraulic facilities connected with these roads shall be designed to a five year frequency.

12.11.2 Haul Roads

Hydraulic facilities for haul roads which cross or encroach into a watercourse shall be designed for a frequency as determined by using a Design Risk of 50%. As a general rule, to avoid excess upstream flooding, the profile of the road should connect the tops of the channel embankments and the road shall be designed to be overtopped by those events which exceed the design discharge. Sufficient cover must be provided over the temporary

conduit to insure structural integrity. The structural analysis of the conduit is to be included within the design.

The plan is to include a warning to the contractor that this road is expected to be under water during certain rainfall events for undetermined lengths of time.

12.11.3 Design Procedure

The selection of a design flood frequency for the remaining temporary hydraulic facilities involves consideration of several factors. These factors are rated considering their severity as 1, 2, or 3 for low, medium or high conditions.

12.11.4 Selection Factors

Potential Loss of Life - If permanent or temporary inhabited structures are inundated or are in the path of a flood wave caused by an embankment failure, then this item will have a multiple of fifteen applied.

Property Damage - Private and public structures (houses, commercial, or manufacturing); facilities such as sewerage treatment and water supply; and utility structures either above or below ground, are to have a multiple of ten applied. Active cropland, parking lots, and recreational areas are to have a multiple of five applied. All other areas shall use the severity determined by site conditions.

Traffic Interruption - Includes consideration for emergency supplies and rescue; delays; alternate routes; buses; etc. Short duration flooding of low volume roadway might be acceptable. If the duration of flooding is long (more than a day), and there is a nearby good quality alternate route, then the flooding of a higher volume highway might also be acceptable. The severity of this component is determined by the detour length multiplied by the average daily traffic projected for bi-directional travel.

Detour Length - The length in kilometres of an emergency detour by other roads should the temporary facility fail.

Height Above Streambed - The difference in elevation in meters between the travelled roadway and the bed of the waterway.

Drainage Area - The total area contributing runoff to the temporary facility, in square kilometers.

Average Daily Traffic - The average number of vehicles travelling through the area both ways in a twenty-four hour period.

The example below illustrates a method of determining the design discharge. The severity and rating of each component is determined and entered in the Impact Rating Table. The total impact rating determines the Precent Design Risk, and the construction time is then considered to find the design frequency. A ratio corresponding to the frequency is used with the 50-year and 100-year storm to determine the design discharge.

Note: If sufficient discharges have been developed either by the designer or the Ministry of Water & Energy then a frequency curve shall be plotted to determine the Design Discharge instead of the final formula using the ratio.

Table 12-2: Rating Selection

Factor	Rating		
	1	2	3
Loss of Life	See Instructions		
Property Damage	See Instructions		
Traffic Interruptions	0-2000	2002-4000	4001-6000
Detour Length	< 5	5-10	> 10
Height above Streambed (m)	> 6	3-6	< 3
Drainage Area (km^2)	< 1	1-10	> 10
Rural ADT	0-400	401-1500	> 1500
Suburban ADT	0-750	751-1500	> 1500
Urban ADT	0-1500	1501-3000	> 3000

Table 12-3: Impact Rating Form

Loss of Life x 15 =	_____	
Property Damage x 10 or x 5=	_____	
Traffic Interruption =	_____	
Detour Length =	_____	
Height above Streambed =	_____	
Drainage Area =	_____	
Average Daily Traffic =	_____	
Total Impact Rating = (sum of the above) =	_____	
Design Frequency =	_____	years

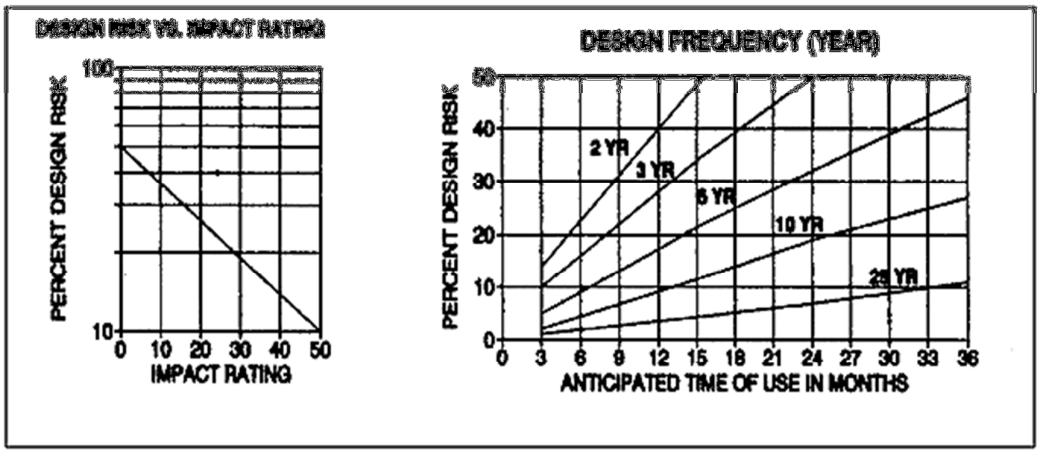


Figure 12-2: Design Risk vs. Impact Rating and Design Frequency (Year)

Table 12-4: Flow Ratio

Year	Ratio	Year	Ratio	
2.0	.8	10.0	1.9	
2.33	1.0	25.0	2.7	Ratio = ___x 0.27 (Q_{50}__) = __ m^3/s
3.0	1.2	50.0	3.7	
4.0	1.3	100.0	5.0	Ratio =____x 0.20 (Q_{100}_) = __ m^3/s
5.0	1.4			

EXAMPLE 1

1. Loss of Life. No potential loss of life. Use same severity as used for ADT (see 7 below) = 2
2. Property Damage. Active cropland, use Rating = 5
3. Traffic Interruption. Detour length = 6 kms, ADT = 1200, 6 x 1000 = 6000, Rating = 3
4. Detour length = 6kms, Rating = 1
5. Height above streambed = 4 meters, Rating = 2
6. Drainage Area = 1800 hectares, Rating = 2
7. ADT (Rural) = 1000, Rating = 2
8. Total impact Rating = 17

IMPACT RATING TABLE

Loss of Life x 15 =	_____2_____
Property Damage x 10 or x 5=	_____5_____
Traffic Interruption =	_____3_____
Detour Length =	_____1_____
Height above Streambed =	_____2_____
Drainage Area =	_____2_____
Average Daily Traffic =	_____2_____
Total Impact Rating = (sum of the above) =	_____17_____

From Figure 2, for an impact rating of 17, Percent Design Risk = 30%.

Construction time is estimated at 24 months, Figure 12-2 indicates 5 year design frequency.

From Table 12-4, a 5-year design = 1.4 ratio.

If Q_{50} = 4.3 m^3/s; Q_{100} = 5.7 m^3/s, design for:

Q_{50} = 1.4 x 0.27 x 4.3 m^3/s = 1.63 m^3/s

Q_{100} = 1.4 x 0.27 x 5.7 m^3/s = 2.15 m^3/s

12.12 References

1. AASHTO Guidelines 1,2,3,4,5,6, and 7.
2. The SUDS Manual - CIRIA C697.
3. Practical Soil Erosion Control and Veld Rehabilitation in the Little Karoo – February 2011.

13 OPERATION, MAINTENANCE AND REMEDIATION

13.1 Introduction

Appropriate maintenance of drainage infrastructure plays a crucial part in its effective operation. This also minimises environmental harm and provides a level of safety to users of the road corridor.

This chapter of the manual mainly focuses on managers of road maintenance operations and maintenance contractors. It outlines the maintenance process and uses examples of drainage failures to illustrate the need for effective maintenance operations. It also provides steps for the remediation of problems or deficiencies.

The process outlined in this chapter relies on the design process undertaken in previous chapters and reference to the design criteria, assumptions, calculations and assessments may be required.

The purpose of this chapter is to use the maintenance process for identifying failures in the drainage system and to assist learning from these failures to prevent future failures.

13.2 Legal Requirements

The legal requirements associated with operation, maintenance and remediation works are outlined in Chapter 3 - Planning and Policy. In addition ERA has a legal responsibility / duty of care to ensure that the roads under its jurisdiction are maintained to provide an acceptable level of safety to the public, road users and to protect the environment from harm. It is important that staff overseeing these activities understand the applicable requirements of the legislation to ensure compliance.

To ensure appropriate and timely maintenance, it is important that regular inspections, followed by appropriate remediation works (where required), be conducted. With respect to road drainage, it is recommended that inspection of this infrastructure should be conducted shortly after significant rainfall / flood events when failures are more likely to occur. Any remediation work would depend on the severity of any damage / failure identified.

Furthermore, ERA must also ensure prompt response to emergency situations (such as water over the road or subsidence of the roadway occurs) where rapid remediation works are required. Failure to act appropriately exposes ERA, and its officers, to increased risk of investigation and/or legal action.

13.3 Operation

The operation of the road and in turn the drainage system commences immediately after the road is opened to traffic. This section deals with the period immediately after construction while Section 13.4 discusses the ongoing maintenance considerations and activities which span the life of the road.

An important function or activity that should be conducted in the period after construction is the inspection/check of the drainage system's actual operation or performance against the design intent. This can only happen after a reasonable rainfall/storm event and will either validate the design or identify deficiencies. This performance check is particularly important for drainage devices protecting/maintaining water quality. Depending on the deficiency, remedial works may be covered under the defects liability component of the construction project.

It is important to note that the inspection period for the site should be extended to check the performance of any remedial work.

13.3.1 Performance

Drainage infrastructure is constructed to primarily carry / transfer and possibly treat stormwater. These devices are designed for a certain discharge and/or capability. With respect to drainage devices constructed to protect and/or maintain water quality, they have been designed to meet specific water quality requirements. It is important that the performance of these devices be checked to ensure that the devices are achieving the design requirements. If it is found that the requirements are not being met, the site must be:

- Fully investigated to determine the reasons why the device is or devices are not achieving required targets;
- Appropriate remedial action planned / designed to correct deficiency; and
- Remedial work undertaken.

This investigation and remedial work must be undertaken as soon as possible after the deficiency has been identified as the risk of causing harm or damage will remain elevated until the work is completed.

13.3.2 Reporting of Deficiencies

It is important that deficiencies in the drainage system be identified, investigated and reported. The reports must identify the reason or cause for the deficiency. While reports are processed and subsequent remedial works actioned by regional/district offices, it is requested that copies of reports and determined remedial action be forwarded to the ERA. This will enable a review of drainage failure/deficiencies which may lead to changes in design/construction methodology, and/or departmental documents and policy.

13.3.3 Health and Safety

To comply with the Health and Safety Regulations, designers must assess all foreseeable risks during construction and maintenance and the design must minimise them by the following (in order of preference):

- Avoid;
- Reduce; and
- Identify and mitigate residual risks**.**

Designers must also make contractors and others aware of risks. All those responsible for maintenance should take appropriate health and safety precautions for all activities (including lone working, if relevant) and risk assessments should always be undertaken.

13.4 Maintenance

13.4.1 General

Drainage structures perform the function of removal of water from highway sections, parking areas, and other drainage areas and the protection of the structures from the effects of the water.

These drainage structures include drop inlets, storm drains, bridges, culverts, underdrains, ditches, check dams, slope protection, and erosion control devices. In order for these structures to function as designed and constructed, they must be properly maintained. Full

consideration must therefore be given to this activity during their design. Designing drainage structures that are as maintenance free as practical will often result in cost savings that, over the service life of the drainage feature, equal or exceed initial construction cost. Good drainage design practices recognize that all structures require periodic maintenance inspections and repairs. Reasonable access for maintenance personnel and equipment must be provided for this necessary function.

Communications between designers and maintenance personnel are essential. Design personnel are encouraged to contact maintenance personnel for their input on difficulties they identify in maintaining drainage structures. Suggestions from maintenance personnel on how drainage structures and future designs may be improved for efficient and effective maintenance shall be invited.

Conditions that appear to require extensive repair or that incur frequent recurring maintenance shall be referred to the Hydraulic Engineer for review. Investigation may reveal that a complete redesign is more cost effective than repetitive repair. Reports by the maintenance forces of both effective and non-effective hydraulic installations aid designers in future work.

Maintenance Program

Highway maintenance requires the upkeep and restoration of all carriageway structures to a standard which aims to retain the condition of the structures to its immediate post construction state. This approach will ensure that the structures are fit for purpose.

Maintenance functions relative to highway drainage vary from establishing and caring for vegetative cover on slopes for erosion control purposes to extensive emergency repairs resulting from major storm damage.

Minor improvements to drainage structures and betterment work, although often performed by maintenance forces are not generally classified as maintenance functions.

Routine maintenance of highway drainage structures include:

- Performing periodic inspection to ensure that the structures are functional and constructed as designed;
- Performing miscellaneous preventative maintenance acts which will forestall deterioration of the structures;
- Cleaning accumulation of debris, sediment, and vegetation from the structures; and
- Repairing or replacing damaged elements of the structures.

Storm damage to major channels and structures can be both expensive to repair and hazardous to traffic. In addition to periodic maintenance, the maintenance program should include inspections and repairs following major storms and floods.

Records

Appropriate records of significant repairs made to drainage structures shall be kept by ERA regional offices in charge of maintenance.

They shall include description of all repair works carried out. A record should also be made of exceptional high water, unusual flow conditions, and any other peculiar conditions such as scour of the streambed or bank altering the channel during high water periods. A record of the highest high water mark for major streams shall be indicated by painting a mark and date on abutment, pier, or column.

Problems

Carriageway maintenance operations that require lane closures on heavily traveled highways, particularly urban sections of trunk roads, are costly and may compromise the safety of maintenance workers and the travelling public. Good drainage designs eliminate or reduce the need for traffic control measures for cleaning drainage systems and structures. Drainage inlet types and pipe drains that are designed to be as self-cleaning as possible shall be specified wherever it is practical.

The routine cleaning and minor repairs of highway drainage features often require that labor intensive hand methods be used. Adequate access for maintenance personnel and equipment to get to the site and do work on drainage structures shall be provided for in the plans. In addition, most costly maintenance work might easily be avoided, or more efficiently accomplished, if designers were to give more attention and thought to the shape and location of drainage features. For example, a "V" shaped roadside gutter that is contiguous to the shoulder can be efficiently reshaped and cleaned with a motor grader. Small trapezoidal and other shaped roadside ditches will be easier to clean by hand.

Locating drainage inlets where trash and sand naturally tend to accumulate causes clogging of grate-type inlets and pipe drains. Wherever practical, drainage inlets shall be located where cleaning operations will not deposit additional debris in drainage structures. Inlet and junction boxes and other minor drainage structures that are subject to periodic cleaning must be made large enough that maintenance workers can enter them and work with either hand tools or machine equipment.

Several types of bank protection and erosion control materials are classified as flexible or self-adjusting and, as a rule, are less of a problem to repair and maintain than are the rigid category of linings. Whenever practical, materials requiring less maintenance attention shall be specified.

Inspection

Maintenance of drainage structures is very important both during the development and construction of a project and afterwards. In some areas maintenance of natural drainage systems presents minimal problems while in other areas major resources will need to be allocated for maintenance related tasks.

Drainage inspections shall be made regularly and after (or during) each major storm to confirm that satisfactory conditions exist, or to evaluate the need for cleanup and repair. Inspection schedules should include mandatory inspection of known trouble areas, and inspection of other areas as appropriate. Tables 13-1 and 13-2 give indicative frequencies for routine and periodic inspections of the most common structures. These frequencies will be adapted to the local conditions and the particular troubles encountered.

Table 13-1: Routine Inspection Frequency for Different Types of Drainage Structures

Type of structure	Longitudinal Ditches	Small Pipe and Box Culverts (Section < 1 x 1 m)	Large Pipe and Box Culverts (Section > 1 x 1 m)	Slab Culverts and Small Bridges
Type of Inspection	Visual	Visual	Visual + Inside Visit	Visual + Inside Visit
Frequency of Inspection	Twice a year : one before and one after the main raining period	Twice a year : one before and one after the main raining period	Once a year	Once every other year

Table 13-2: Periodic Inspection Frequency for Different Types of Drainage Structures

Type of structure	Large Pipe and Box Culverts (Section > 1 x 1 m)	Slab Culverts and Small Bridges
Type of Inspection	Detailed Visual + Structural tests	Detailed Visual + Structural tests
Frequency of Inspection	After any major flood and at least once every five years	After any major flood and at least once every ten years

The best time to look at drainage structures is often during a storm. It is easy then to see where water ponds and where drainage structures are over-flowing. The inspector shall be alert to any pavement cracks or settlements that appear after a severe storm even if these defects are small, as they may be evidence of erosion caused by a break in the pipes. Areas that generate large amounts of sediment and debris shall be identified. Also, locations within the drainage system where debris and sediment accumulate shall be identified and included in any preventive maintenance schedule.

An example of a form used for culvert inspections presented as Table 13-5.

A record of the inspection shall be kept by ERA, with any deficiencies recorded.

Organisation

The organization of agencies responsible for maintenance of drainage structures is not within the scope of this manual. An example of a typical Maintenance/Inspection sheet is given in Table 13-3 and Table 13-4 respectively at the end of this section. The use of this kind of form, when properly and regularly filled in can help improving the efficiency of the recording system. It must be adapted to the organisation in charge of maintenance in the country and discussed with the people involved.

13.4.2 Detention Structures

Detention structures are often used in urban drainage systems to temporarily store or detain excess storm water runoff and then release it at a regulated rate to downstream areas. Using this approach, runoff is stored in constructed or natural basins from which it is released continually until the water elevation in the structure reaches its design dry-weather stage. To function properly the storage volume must be maintained at its design level and outlet structures must be kept open and free from obstructions or clogging.

Maintenance

A good maintenance program should include preventive maintenance as well as corrective maintenance. The maintenance program should include:

- Periodic inspection, adjustments, replacement;
- Preventive maintenance - doing maintenance so that problems do not occur such as removal of debris at inlets; and
- Corrective maintenance - making changes to the system so that system functions as intended at the lowest annualized cost, such as resetting an inlet to reduce ponding.

The major prerequisite for a preventive and corrective type of maintenance programme is an assured source of funds.

Inspection Intervals

Inspection of major detention structures should be made as frequently as experience shows necessary, perhaps monthly as a minimum in wet seasons. Where debris is a problem, inspections must be spaced according to debris generation. In any event, it is important to conduct inspections and clean-up work following major runoff events. It is sometimes necessary to make inspections during rainstorms when intense rainfall occurs.

Maintenance Tasks

Maintenance tasks can be grouped in three general categories:

- Aesthetic maintenance;
- Nuisance maintenance, and
- Operation maintenance.

Of these, the most important, from the standpoint of health and safety, is operation maintenance.

Operation Maintenance

This category can be characterized as that level of maintenance required to ensure against failure of major structural components and/or flow controls, and to ensure that the structure continues to function as designed. Neglecting this level of maintenance could cause dam failure and subsequent property damage as well as possible loss of life. In addition, neglect often causes a structure to cease functioning as it was originally designed to do. A program of scheduled, periodic inspections of the structure is essential to recognize potential structural maintenance needs. The following is a partial list of items that should be checked periodically and corrective action taken as required:

- Settling of detention pond dam;
- Woody growth on the dam (roots can create channels for dam leakage and eventual failure);
- Signs of piping (leakage);
- Signs of seepage or wet spots on the downstream face of a dam (may require toe drains or chimney drains to solve problems);
- Riprap failures;
- Deterioration of principal and emergency spillways;
- Various stage/outlet controls;
- Effectiveness of debris racks;
- Outlet channel conditions;

- Safety features (access controls to hazardous areas);
- Mechanical and electrical equipment (pumps, generators, automatic controls, etc.);
- Access for maintenance equipment;
- Availability of manufacturer's mechanical and electrical information manuals; and
- Availability of design information such as rating curves and tables for spillway flow, bypass flow, total flow, and storage and pump out calculations.

In addition, the following actions should be taken on each dam as required:

- Replace soil removed by rodent burrows;
- Inspect drainage systems and relief wells periodically, at least annually, for proper functioning and clean out or replacement as necessary;
- Maintain riprap or other wave-protective measures and replace as needed;
- Remove and/or stabilize slide material as soon as practical. It may be necessary to construct a berm or flatten the slope;
- Replace eroded material and establish vegetation in eroded areas in emergency spillways, swales and other areas; and
- Repair any unusual seepage, boils or settlement in fill areas, or sinkholes in pool areas.

Also, observations should be made of any changes in topography, downstream drainage systems or land uses that may have a bearing on the operational effectiveness and safety of detention structures.

Volume Maintenance

One of the most important variables in the design of a detention structure is the volume available for storage of runoff. If a detention structure is allowed to accumulate sediment and debris which will decrease the storage volume, the ability of the structure to function as designed can be greatly reduced.

Thus it is essential to maintain the design volume. To facilitate the inspection of the structures for volume control, it is recommended that some marker be installed in the detention structure to indicate the maximum level for silt build-up before the structure must be dredged or cleaned. This marker could be a small pipe with a stripe or suitable indicator at the cleanout level. A suitable indicator could also be place on the outlet device or in some location which can be easily identified during the inspection process.

Sediment Basins

Often detention structures are used as temporary sediment basins. To control the maintenance of these structures some criteria must be established to determine when they should be cleaned and how much of the available storage can be used for sediment storage.

Example 1

The following is an example of possible criteria and procedures that can be used for sediment basin maintenance.

Figure 13-1 can be used to estimate sediment trap efficiency for sediment basins with different volumes. The procedure for using this figure is as follows.

1. Establish sediment generation criteria (e.g., 125m^3 silt per disturbed hectare draining to the sediment basin).

2. Estimate total volume available for sediment storage from the geometric shape of the basin (e.g., 550 m^3).
3. Calculate minimum silt storage needed given the silt generation criteria (e.g., 125 m^3 per disturbed hectare x 4 hectares of disturbed area = 500 m^3).
4. Trap efficiency can be estimated from Figure 13-1 as follows 550 m^3 available storage/ 4 ha x 1 ha/10 000 m^2 = 0.01375 m 0.01375m x 1000 mm/m= 13.75 mm – storage/drainage area hectare from Figure 13-1 at 13.75mm- Trap Efficiency = 55%
5. If you were required to clean out the structure when the efficiency reached 50%, the clean out elevation could be determined as follows:
   - From Figure 13-1 at 50% trap efficiency, the storage/drainage area = 10.25 mm,
   - Storage = 10.25mm x 4 ha x 1 m/1000 mm x 10 000 m^2/1ha = 410 m^3, and
   - Given the basin geometry, try different depths until you have 410 m^3 of storage still available for sediment storage. This is then the depth where the basin should be cleaned to ensure that the trap efficiency does not go below 50 percent.

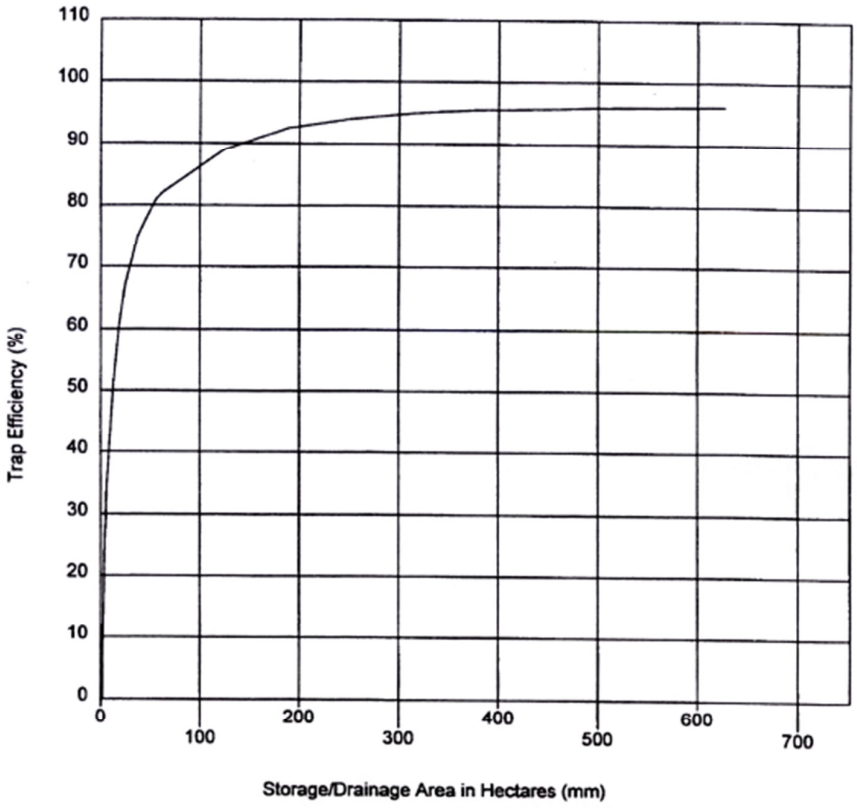


Figure 13-1: Efficiency of Sediment Basins

Note: Capacity is total sediment basin volume up to emergency spillway crest. From Brune, Gunnar M. – Trap Efficiency of Reservoirs Trans. AGU.

This example gives a procedure that can be used to estimate trap efficiency and also establish clean out levels for a given efficiency. Different efficiencies might be established according to the damage potential downstream.

13.4.3 Storm Drains

Maintenance Problems

Storm drains convey the water collected from catch basins and drop inlets to the natural watercourses. The maintenance involved in storm drain systems is the removal of any sand, silt, or debris and the maintenance of a tight seal at each pipe joint. There are occasions when abrasive material is present in the water (or some chemical that has a deleterious effect on the pipe) that causes the pipe material to be worn away. This necessitates relining the pipe to preserve its integrity. It is recommended that the entire storm system be inspected every 10 years and the catch basins twice yearly.

Cleaning

Water flushing can remove some partial clogs. Storm drains can be cleaned by using drain-clearing rods or by inserting a rodding machine (heavy-duty sewer snake) in one manhole and running it through to the next manhole.

13.4.4 Culverts

Culvert Maintenance

Culverts must be kept free of obstructions. Sand or sediment deposits shall be removed as soon as possible. Inlet and outlet channels shall be kept in alignment and vegetation controlled in order to minimize any significant restriction of flow. Reinforced concrete box culverts require little maintenance, but they shall be inspected every 2 - 4 years for cracks, bottom erosion and undermining at outlets. Undermining is the result of high outlet velocities in erodible materials. Correction of undermining usually requires adding an energy dissipater or protective measures.

Cleaning

Culverts may become clogged if the flow-line grade prevents self-cleaning. A permanent correction is to relay the pipe on a steeper grade, but this is not always possible and is often very expensive. The alternative is to clean the pipe frequently (e.g. every year, at the end of the dry season). Small culverts may be cleaned by flushing away debris with water pressure. An alternate method of cleaning small culverts is to use hand tools supplemented by mechanical means.

Some large culverts over 0.75 meters diameter must be cleaned by hand. A small sled or wagon is useful for transporting material from inside the barrel to the culvert ends. In some cases a small rubber-tired tractor, equipped with a push blade, may be used to remove sand and silt deposits from the larger concrete culverts. Cleaned material and debris must be removed from the site.

Repairing

If the invert of a metal or concrete culvert becomes worn or eroded, it can be repaired by relining with concrete grout, shotcrete, or asphalt cement. If the hydraulic capacity of the culvert is not critical, a smaller pipe can be placed inside, and the space between the pipes can be filled with pressure pumped cement grout, or shotcrete.

High velocity flows, containing large quantities of stone and rock, scour the culvert bottom. Scour may be reduced by securing steel plates longitudinally along the bottom. Scour around footings, cut-off walls, and headwalls may be repaired by replacing the eroded material in kind or by filling the void with riprap or sacked concrete. In an emergency a bituminous mix may be used.

When concrete pipe culverts settle, joints pull apart. Joints may be repaired by tamping or rodding grout into the cracks.

In order to prevent erosion, energy dissipaters are sometimes placed at outlets of culverts and drains. It is important that these be inspected periodically, particularly after major flows, to ensure that they are in place and functional.

13.4.5 Ditches

Ditch Maintenance

Side ditches convey water away from carriageways and other areas. Ditches may be unlined or lined with cement concrete, shotcrete, masonry, quarry rock, bituminous concrete or vegetative material. Ditches shall be kept free of silt, debris, large amounts of vegetation, or any other material that restricts the flow of water. A complete cleaning shall be carried out at least once every year.

The flow lines of unlined roadside ditches may be maintained by hand or by motorized equipment supplemented with hand work. Large roadside ditches are sometimes located at an elevation well below the carriageway and not accessible to a motor grader. These may be reached with a truck mounted hydraulic excavator operated from the shoulder.

Interceptor ditches on slopes and along excavation or embankment benches, and outlet ditches from culverts may require hand cleaning by using shovels and wheelbarrows.

Repairing

Joint separation is a common problem associated with concrete lined and masonry lined ditches. Once water gets under the concrete or masonry, the underlying soil is removed and deterioration may be rapid, so frequent inspection is vital (after any heavy rain) and fast repairs a necessity if the investment is to be protected. If not immediately repaired, erosion occurs under the lining, causing it to crack and sometimes drop.

Erosion and Vegetation

Ditch erosion is the loss of soil caused by the rapid flow of water. It is controlled by paving the ditch with bituminous asphalt aggregate mix, placement of masonry, grouting rock, establishing erosion resistant vegetation, or by constructing checkdams. Since erosion is serious, any case of erosion shall be reported to the person in charge. Ditches lined with bituminous material oxidize or weather rapidly and shall be sprayed with asphalt emulsion.

Since erosion is one of the major problems with ditches, the growth of vegetation is encouraged. The vegetation may be maintained by adjoining property owners, but it shall be checked by the District Maintenance Organization. The control of weeds is one of the major problems when vegetation is used to control erosion in ditches.

13.4.6 Stormwater Inlets

Stormwater inlet structures are designed to intercept water in gutters and drainage courses. They also act as settling basins to collect heavy solids, and they prevent debris from entering culvert systems. Hand equipment and tools are usually used to clean sediments from catch basins.

Grates/grills on catch basins are used to prevent large objects and debris from entering the system. Frequent inspections are required because debris such as pieces of cardboard, newspapers, or flat metal can get caught on the grate/grill and can prevent water from

entering the catch basin. Many times the duration of rainfall is too short to allow for self-cleaning of the system and debris tends to buildup over time.

The frequency of inspection and cleaning shall depend on the environment and on the design of the inlet. Large catch basins constructed without a grate may collect large quantities of rock. This rock may be removed by mechanical means from the catch basin. Muck may be removed by a wheelbarrow.

13.4.7 Slope Drains

Slope drains are paved or metal pipes used to carry water from a collector drain, gutter, or ditch, into a roadside channel or natural watercourse. They should have firm contact with the supporting surface. If connected to pavement or dike, a tight seal shall be maintained.

The outlet end of slope drains shall be inspected regularly for erosion, at least once every year. Eroded areas shall be repaired. Sometimes the repair may include extending the drain.

Metal pipes used for slope drains on high embankments or benched excavations shall be rigidly attached to the surface with pipe anchors. Anchors are designed to prevent the pipe from separating at joints, but in spite of this, separation occasionally will occur. Repairs consist of removing, re-installing and anchoring all pipes below the separation.

13.4.8 Check Dams, Energy Dissipaters and Bank Protection

Check Dams

Check dams are used to collect water, redirect its flow, provide settlement basins for siltation control, and control rate of flow. They are constructed of masonry, reinforced concrete or grouted stone.

Check dams are inspected for undermining after each period of heavy water runoff. Undermining shall be repaired by filling voids with large rocks, riprap, or sacked concrete.

Energy Dissipaters

Energy dissipaters are used at the outlets to culverts flowing with inlet control and therefore supercritical velocities. All energy dissipaters lessen outlet velocities, resulting in better maintenance of the downstream channels. The energy dissipater may be a level spreader, discharge apron, or hydraulic jump. They should discharge into an area stabilized, if possible, by vegetation as a minimum. They shall be inspected for undermining after periods of heavy runoff, and any undermining shall be repaired.

Grouted Riprap Protection

Undermining causes failure of this type of bank protection. Undermining leaves a void under the grouted riprap that can rupture and cause the riprap to collapse into the void. Repair consists of filling the voids and collapsed surfaces with rock and grouting.

Slope Paving

Slope paving is a rigid type of slope protection composed of concrete reinforced with wire mesh, or shotcrete. It is subject to damage by undermining, hydrostatic pressure, and material being washed through cracks. If degrading of a stream or scour exposes the toe, the toe shall be protected with grouted rock. Weep holes shall be placed in the slope paving if there is a possibility of water seeping behind the paving. Weep holes may be placed by drilling approximately 50mm diameter holes in the slope paving. If cracks develop which

permit the backing material to be washed out, they shall be sealed with concrete grout. When the toe wall of concrete slope paving becomes exposed, it shall be repaired using rock or concrete.

Retards and Permeable Jetties

Retards and permeable jetties are frame structures made of wire mesh, chain-link, steel rails or timber, filled with earth or stone ballast designed to absorb energy and to keep erosive channel flows away from a bank. They are designed to decrease velocity and induce sediment deposition or accretion.

The design of this type of structure is subjected to analysis of the flow and of the sediment deposition and it must be carried out by hydraulics specialists.

Maintenance of such structures is confined primarily to the replacement of stone fill.

Gabions

The use of gabions is encouraged as the preferred alternative to check dams and grouted riprap. To be effective, they need to be used in conjunction with a filter cloth/geotextile backing. Maintenance problems associated with gabions are minor in nature and consist mainly of patching wire mesh and the addition of wire and clips.

13.4.9 Underdrains

Underdrains shall be checked in the early part of the wet season to ensure that they have not become clogged with sand or roots, and that outfalls are free to drain. Presence of silt or dirty water coming out of an underdrain indicates a possible break in the pipe. This shall be reported at once so that an investigation and remedial measures can be initiated.

Manual cleaning is required to clean out long sections of clogged pipe. When the pipe becomes clogged, the filter material probably has become silted and its effectiveness has been reduced to a level that makes it necessary to consider replacement.

13.4.10 Bridges

Bridges must be kept free of obstructions. Debris and vegetative growth under a bridge may contribute to scour, create a potential fire hazard and reduce freeboard for debris during high-water flows, resulting in a serious threat to the bridge. A reduced effective flow area under the bridge may also result in excessive bridge backwater damage, more frequent carriageway overtopping and a hazard to the traveling public.

Maintaining a channel profile record and revising it as significant changes occur provides an invaluable record of the tendency toward scour, channel shifting, and degradation or aggradation. A study of these characteristics can help predict when protection of pier and abutment footings may be required. Being able to anticipate problems and taking adequate protective steps will avoid or minimize the possibility of future serious difficulties.

Maintenance inspection must be commensurate with the risk involved. Where probing and or diving are necessary, the inspection shall be scheduled at the season of lowest water elevation. High-water and debris marks with the date of occurrence shall be recorded for future reference.

Maintenance Problems

The following are some of the maintenance problems that can be encountered:

- Clogging of bridge deck drains and scuppers, which may create a hazard to traffic and contribute to deck deterioration;
- Discharges of bridge deck drains that are detrimental to other members of the bridge, and those spilling onto a carriageway below. In addition, discharges that may cause fill and bank erosion;
- Clogging of air vents in the superstructure or deck of bridges subject to overtopping which may increase buoyancy forces and the possibility of bridge washouts;
- Accumulation of debris in the open space between the handrails of bridges subject to overtopping which may induce additional lateral forces on the bridges and increase the risk of washouts;
- Channel aggradation or degradation;
- Scour at piers and abutments caused by accumulation of debris and or excessive velocities;
- Damage to bridge approach embankment caused by channel encroachment;
- Loss of riprap due to erosion, scour, and wave action; and
- Damage to bridge elements due to debris and excessive velocities.

Maintenance Measures

Maintenance measures include the following:

- Repair of damaged bridge elements;
- A schedule for removal of debris after major floods;
- Removal of sand and gravel bars in the channel that may direct stream flow in such a manner as to cause harmful scour at piers and abutments;
- Cleaning bridge deck drains and keeping their outlets away from traffic underneath. Also providing riprap or other means of protection at outlets to avoid fill and bank erosion;
- Removal of debris caught between bridge handrails and opening vent holes designed to reduce buoyancy;
- Making a channel change when necessary to redirect the flow away from bridge approaches and in line with the bridge skew;
- Dredging of channels that are subjected to a high degree of aggradation in order to maintain waterway adequacy;
- Constructing cut-off walls to reduce or stop progressive channel degradation;
- Replacing lost dirt in scour holes and constructing riprap mats or other means of protection for undermined piers and abutments;
- Replacing missing riprap on embankment slopes, channel banks, spur dikes, etc.;
- Constructing additional openings to accommodate increased urbanization in the drainage area upstream from the bridge; and
- Modifying or increasing existing protective measures when needed.

Table 13-3: Maintenance and Inspection Sheet

This maintenance and inspection sheet is given as an example and shall be adapted to each organization in charge of maintenance

Description of structure :…… ……..	Items to be inspected : 1…… 2…… 3…… 4…… 5…… 6……	Frequency of Inspection ……
Location :…… …… ……		Person in charge

Date of next planned inspection	Date of inspection	Team / person in charge of inspection	Damage /clogging reported	Maintenance / Cleaning works to be carried out	Team/ person in charge of maintenance/ cleaning works	Date of maintenance / cleaning work

Description of structure:…Pipe culvert ……	**Items to be inspected :** **1** Clogging of pipe……	**Frequency of Inspection** Every Year……

Table 13-4: Maintenance and Inspection Sheet Example of use

…… …… …… ……..	**2** Scouring at outlet …… **3** Bottom erosion…… **4**…… **5**…… *Whenever a new type of damage is reported, it shall be added in this section*	**Person in charge of supervision**
Location :…Km 45-100……..…. …… …….....		

Date of next planned inspection	**Date of inspection**	**Team / person in charge of inspection**	**Damage /clogging reported**	**Maintenance / Cleaning works to be carried out**	**Team/ person in charge of maintenance/ cleaning works**	**Date of maintenance / cleaning work**
01/01/2002	07/01/2002	Mr X	Nothing to report			
07/01/2002	05/01/2002	Mr X	1 Pipe clogged	Cleaning	Mr S	To be done
			2 Scouring at outlet	Build energy dissipator	Dept A	28/01/2002
05/01/2003	To be done					

Table 13-5: Culvert condition Survey Maintenance format

ETHIOPIAN ROADS AUTHORITY		**CULVERT CONDITION SURVEY**	Date :
ROAD SECTION :			**Km …….. to Km ……..**
CULVERT	Location : Km Type (Check box)	Pipe ☐ Box ☐ Single slab ☐ Double slab ☐	
	Section and dimensions :	Diameter : Length : Height : Width :	
	Nature (Check Box)	Reinforced Concrete ☐ Steel ☐ Other (Specify) :	
CONDIDITON OF STRUCTURE (Enter your comments)	Wing walls inlet outlet		
	Slabs inlet outlet		
	Culvert body		
CONDIDITON OF ENVIRONMENT (Enter your comments)	Channel entrance exit		
	Embankment top entrance side exit side		
MAINTENANCE WORKS REQUIRED	Structure		
	Environement		

13.5 Drainage Failures

A failure in road drainage may be caused by any number of problems or combination of problems and can occur during the construction or operation of the road. Erosion at culvert outlets, undermining of pavement and drainage structures, soil loss on steep batters and sedimentation of drains are all common failures.

While flooding of the road corridor is the most common problem as a result of insufficient drainage capacity or a blockage of cross drainage, there are other issues that can occur as a result of failure (reduced performance/capability).

13.5.1 Causes of Failure

Failures in the drainage system or device/component may result from a number of situations. Some of the more prominent causes of failure are:

- Inadequate/inappropriate design;
- Poor construction practice/post construction inspection;
- Changes to the site's physical conditions (e.g. alteration to landform, vegetation, and surrounding land use);
- Poor maintenance inspection/practice;
- An extreme rainfall event which delivers a storm much greater than the design storm the drainage infrastructure was designed to handle; and
- Out of specification materials.

Failure may also be caused by site maintenance operations. It is recognised that maintenance of the drainage infrastructure plays a crucial part in its effective operation.

13.5.2 Types of Failure

A number of types of failures are evident in the road drainage system. The more common failures include:

- Surface/slope/bank erosion;
- Undermining or piping;
- Sedimentation;
- Debris accumulation (e.g. litter and vegetation); and
- Structural failure of the drainage device/component.

These are discussed below in further detail.

13.5.3 Erosion

Erosion is the most common failure. Roads tend to concentrate storm water flows which in turn increases flow velocities and energy. This combination increases the risk of erosion and scour. Photo 13.1 below shows examples of erosion.



Photo 13-1: Erosion of Roadside Edge

Erosion can occur at or in:

- Culverts (inlets and outlets);
- Bridges;
- Floodways;
- Bypass channels;
- Table drains; and
- Unsealed shoulders, verges and batters.

13.5.4 Undermining or Piping

Undermining refers to the loss of soil from underneath some part of the road infrastructure (e.g. pavement surface, concrete lined drain, culvert apron). This can result in direct damage to the road infrastructure such as cracking or slumping.

Piping is the term used to describe the mode of embankment failure that involves the washing out of the smaller soil particles from a section of the road embankment by water leaking through a weak point in its structure. The weak point can occur either from the side on the embankment, through the top surface of the embankment via cracks in surfacing or unsealed shoulders or through gaps in disjointed culverts. The progressive removal of fine soil particles will further increase the rate of flow of water and the rate of removal of the soil particles. As the rate of leakage accelerates, larger soil particles can be transported. Eventually localised collapse of the embankment will occur. Photo 13-2 below shows an example of a dam failure due to piping.



Photo 13-2: Failure of Earth Dam due to Piping

13.5.5 Sedimentation

Sedimentation is another common cause of failure and is the deposition of soil that has been transported by flowing water. Soil particles settle once the flowing water has slowed or stopped. This often occurs in culvert inlets and outlets as well as creeks and other watercourses. Photo 13-3 below shows examples of sediment deposition in culverts. This failure is also termed blockage and reduces the capacity of the culvert, which in turn can increase flooding (afflux) upstream.



Photo 13-3: Sedimentation

13.5.6 Debris Accumulation

Debris accumulation includes the accumulation of vegetation, litter and other gross pollutants in the drainage system. This may be caused by insufficient hydraulic capacity of the drainage structure, the size of debris entering the drainage system or lack of maintenance. Photo 13-4 below shows debris accumulation via the growth of vegetation in the outlet of a culvert.



Photo 13-4: Blockage by Vegetation

13.5.7 Structural

Structural failure is the failure of a drainage structure either by separation of units making up a single structure (for example, the disjointing of culvert pipes or box units or the headwall separating from the barrel) or the actual structural failure of a unit (for example, the collapse of a concrete pipe unit due to excessive loading or the collapse of a steel culvert due to weakening by rust). Photo 13-5 below shows failure of the structure of a culvert.



Photo 13-5: Garno River Bridge Failure in Gondar

13.5.8 Environmental Impacts of Failures

Drainage failures have the potential to cause environmental harm. Some common impacts include:

- Disruption to vegetation (direct terrestrial and aquatic habitat loss);
- Soil erosion and sedimentation;
- Altered stream hydrology;
- Altered overland flow paths causing soil moisture changes; and
- Weed invasion.

13.5.9 Identifying Failures

As discussed, routine maintenance of the road corridor is generally carried out by a maintenance contractor. Part of the contractor's role is to undertake regular surveys and inspections of the corridor to identify and prioritise maintenance works.

Significant failures need to be identified quickly so that appropriate remedial works can be determined, approved and undertaken.

The inspection and reporting process of failures could form part of the Maintenance Contractor's Quality Plan. Any failure reports could also be submitted as part of the contractor's quarterly progress report, if required.

Reports should include the following detail:

- The identified drainage failure(s);
- Relevant field notes/photographs; and
- Edentification of potential remediation types.

This information can then be used to assist in the development of an appropriate remediation solution.

Following the completion of the report, the maintenance contractor, in consultation with the department, should prioritise remediation works and/or report maintenance requirements.

13.5.10 Reporting of Failures

It is important that failures of drainage devices or system be identified, investigated and reported. Reports must identify the reason or cause for the failure.

While reports are processed and subsequent remedial works actioned by Federal/regional offices, it is proposed that copies of reports and determined remedial action be forwarded to designers. This will enable a national review of drainage failure / deficiencies which may lead to changes in design / construction methodology, and/or departmental documents and policy.

13.6 Restoration

13.6.1 Planning

A substantial part of highway expenditures is for the purpose of rehabilitating and upgrading existing highways. The type of improvements can include improvements in geometrics, level of service, safety, and/or repair and rehabilitation of various highway elements. All of these improvements may involve highway drainage.

The hydraulic consequences of a highway improvement need to be addressed during the planning phase of the project. Failure to assess the hydraulic aspects of the improvement could result in an increase in damages to adjacent property as well as the highway facility.

13.6.2 Coordination and Cooperation

Opportunities for coordination, cooperation and input from utilities, individual property owners or with other agencies are sometimes present in highway improvement projects. Cost sharing, maintenance, and other responsibilities for cooperative projects need to be addressed when planning the improvement. It is essential that coordination with other agencies and the public be initiated early in the project location phase and continued as necessary through subsequent stages of project development.

13.6.3 Other Agencies

Several Federal Ministries, regional, or local agencies could have an interest in the drainage aspects of a particular improvement project. The modification or replacement of a drainage structure could:

- Affect an existing or planned project by another agency;
- Render an action feasible that may not have been practicable with existing conditions;
- Afford a flexibility in the channel bed elevation, size and/or conveyance characteristics at the crossing;
- Involve preserving existing vertical and horizontal clearances in navigable or recreational waters;
- Present the opportunity for a cooperative storm drain project with a local jurisdiction city or county; and
- Expedite the development of another project.

13.6.4 Utilities

It is important that every effort be made to identify all utilities that may impact a highway improvement project because they can be a major consideration both in the design and cost of the improvement project.

13.6.5 Approvals

Highway improvement projects are subject to the same approval requirements as highways on new alignments. Applications for approvals required by ERA, regional, local and other agencies shall be made as soon as practicable in the process of project development.

13.6.6 Types of Restoration

There are many kinds of highway improvement projects ranging from minor pavement patching to complete reconstruction of several kilometers or more of existing highway. Whenever these projects are undertaken, drainage structures shall be analyzed to determine if existing or proposed structures are adequate.

13.6.7 Horizontal or Vertical Adjustments

Adjustments to the horizontal or vertical alignment of a highway are actually changes in the location of the highway. Because of this, the highway is usually completely reconstructed except where a part of the old embankment is used. Reconstruction can:

- Result in changes in roadside ditch drainage;
- Require either the replacement or the modification of existing drainage structures;
- Change the hydraulic performance of the structure; and
- Cause increased headwater, which affects adjacent property.

13.6.8 Cross Section Change

The roadway cross section is significantly changed in many highway improvement projects. When a change to the roadway cross section is proposed, the effects of the change on highway drainage structures must be recognized and dealt with, including:

- Increase in runoff due to a greater impervious area or more direct ditching;
- Changes in roadside drainage from open to closed systems;
- Changes in the hydraulic performance of drainage structures; and
- Adjustment of existing storm drains and utilities.

13.6.9 Roadway Widening

Pavement widening is sometimes accomplished without any significant change in the highway cross-section and consequently no significant effect on drainage results. Other widening may affect drainage, and for these projects the hydraulic characteristics of the drainage system shall be analyzed.

13.6.10 Flattening Foreslopes

Whenever foreslopes are flattened the culverts need to be extended and the effects of the project on existing roadside ditches must be determined. This type of project can also affect the backslopes and right-of-way.

13.6.11 Survey

The survey items required for highway improvement projects are substantially the same as for a highway on a new alignment. In addition information on the condition of the structure being improved is required, as is specific information about the past hydraulic performance of structures, and identification of any other problems with the existing structure.

13.6.12 Existing Structures

Most highway improvement projects require some modification to the existing structures or to the roadway over the structures. Necessary structural repairs to the bridges, culverts, and storm drains that lie within the limits of the improvement project shall be included with the other project work. Culverts in particular shall be inspected throughout their length before they are extended. The condition survey should provide information about:

- Scour, settlement, joint separation and piping;
- Alignment with the stream;
- Susceptibility to plugging;
- Corrosion and/or abrasion damage;
- Inlet blockage damage, crushed inlets, etc.;
- Recent levees, fills, or other man-made features which affect the hydraulics;
- Vegetative growth; and
- High water information.

Information about the hydraulic performance of existing culverts and bridges is not always available, but an effort to obtain this type of information should always be made because it is extremely valuable input into the hydraulic design of an improvement.

A side-tapered improved inlet shall be routinely considered for all inlet control culverts in urbanizing areas with attendant discharge increases. Incorporating this type of inlet improvement within the expanded highway template will increase the hydraulic capacity of inlet control culverts, negating the need for additional culvert size in some instances.

13.6.13 Other Existing Conditions

There are a number of other hydraulic related conditions that may affect the drainage requirements including:

- Stream stability or the rate and amount of aggradation, degradation, or bank (meander) movement;
- Debris problems;
- Changes in land use, either past or proposed;
- Pier scour and bank scour at bridges, or outlet scour at culverts;
- Upstream or downstream structures that have caused or could cause difficulties to an existing structure; and
- Channel improvements that have been made or are planned.

13.6.14 Hydrology

Hydrologic studies for a replacement bridge or culvert are in many respects the same as those required for structures on a new alignment. There are, however, important differences in the information available at replacement structure sites that can provide valuable input into the hydrologic analysis.

13.6.15 Observed Flood Discharge

Knowledge of how an existing bridge or culvert performs during high runoff events is important input into the design of drainage structures. Survey data collection will normally provide information about past flood events. This survey information should include high-water data. The designer can relate this historical high-water data to a discharge. At those

sites where discharge information can be obtained from a gauging station, the task of determining a discharge for the historical flood is simplified.

It is important, when evaluating past floods, that the designer be aware of conditions at the structure site that could affect the evaluation such as the following:

- Changes in hydrologic techniques;
- Flow obstructions;
- Stream stability;
- Levees or spoil banks;
- Dams, reservoirs, ponding;
- Downstream controls;
- Land use changes;
- Backwater conditions;
- Changes in vegetation density;
- Upstream storage; and
- Changes in stream confluences.

13.6.16 Hydrologic and Hydraulic Investigations

The hydrologic/hydraulic investigation for any highway drainage structure shall be commensurate with the importance of the structure. This general rule shall apply to structures on new highways as well as to existing structures that are being improved. Problem structures include locations where:

- Frequent maintenance of the structures is required;
- Complaints regarding the structure's performance have been received;
- The highway is frequently overtopped;
- The replacement cost is very high;
- River meanders threaten the highway;
- Aggradation or degradation is evident;
- There is high offsite flood damage potential;
- Environmentally sensitive areas are present;
- There is evidence of damage to the structure (wash out, scour, corrosion, abrasion, etc.); and
- Plugging with debris often occurs.

It is also important to be aware of the possibility that replacement structures could cause new problems, such as drift and scour, due to added piers, less freeboard because of deeper superstructures, changes in flow distribution, and changes in downstream peak flows.

13.6.17 Hydraulic Considerations

Most highway improvement projects will require some modification of the existing drainage structures. This modification can vary from extension of small culverts to complete replacement of bridges. The effects on the hydraulic performance of the existing drainage structures can be significant or they can be negligible. If the hydraulic performance of a drainage structure is changed, the change shall be investigated. The investigation should include both upstream and downstream effects. Some of the types of structure modifications and the possible consequences of change in hydraulic performance are discussed in the following subsections.

13.6.18 Culvert Replacement and Rehabilitation

The following sections discuss the advantages and disadvantages of culvert re-placement versus culvert rehabilitation.

Culvert Replacement

When an existing culvert is to be replaced, an analysis shall be made to see if the size of the existing culvert is either smaller or larger than necessary.

- Where an existing culvert is larger than needed for hydraulic purposes, but adequate structurally, it may be more economical to extend the length of culvert than replace it.
- When a culvert must be replaced for structural purposes, and a culvert with less hydraulic capacity is considered, one possible consequence is a significantly higher headwater at the culvert inlet.
- If enough water is ponded, the peak discharge from that drainage area may be lowered downstream.
- The consequences of replacing a hydraulically small culvert with a larger one could be reduced inlet headwater, increase in the downstream flood flow, and under very limited conditions, could change pressure flow to open channel flow.

Culvert Rehabilitation

Any proposed culvert rehabilitation scheme shall be analyzed for hydraulic adequacy and outlet protection. If the cross sectional area resulting from culvert rehabilitation is reduced, this could lead to higher headwater elevations.

However, this effect may be insignificant if there is storage upstream or if the potential for damage is minimal. Routing of flood hydrographs for the proposed design is a good way of assessing increased risks where the upstream channel and floodplain contain significant storage.

Corrugated metal culverts greater than 1,200mm in vertical dimension should have at least a partial concrete headwall with anchorage of the metal to the concrete to minimize potential failure of the inlet and subsequent failure of the entire structure.

13.6.19 Culvert Extensions

The extension of an existing culvert can result in significant changes to the hydraulic performance.

- Extending the inlet of a culvert operating in inlet control will raise the water elevation of the inlet by an amount equal to the difference in invert elevation between the original and reconstructed culvert, all other inlet conditions being equal.
- Extending a culvert that operates under outlet control will increase the headwater.
- Long culvert extensions would rarely cause a culvert to switch from inlet control to outlet control, but the designer should keep this in mind.

13.6.20 Bridge Replacement

Where a proposed new bridge is larger than the existing bridge, there are rarely any significant detrimental consequences because of the change in size. An exception to this would be where the highway profile is raised, eliminating highway overflow. This may increase bridge backwater. If the grade is lowered, the flow pattern and the amount of flow directed over the road and into downstream property could be increased. Backwater could be decreased. When replacement bridges have shorter spans than the existing bridge, the

resulting increase in the number of piers could add to debris and scour problems or increase backwater.

Where a proposed bridge is smaller than an existing bridge there may be some increase in backwater and in velocities in the bridge opening.

13.6.21 Type of Structure

At most sites, the replacement structure will be of the same general type as the existing structure; that is, bridges will replace bridges and culverts will replace culverts. At some locations, though, the hydraulic requirements of the site can be satisfied by either a bridge or by a large culvert.

13.6.22 Bridge Replacing Culvert

Locations where it may be advantageous to replace a culvert with a bridge would include sites where:

- A larger structure is needed;
- Maintenance problems are serious;
- Construction costs would be less;
- There may be a physical obstacle to a culvert;
- Future channel improvements can be better accommodated by a bridge;
- Clearance for boats and water oriented recreation is needed;
- Abrasive sediment could make a bridge a better choice; and
- Environmental considerations, foundation conditions, and channel configuration favor a bridge.

13.6.23 Culvert Replacing Bridge

Locations where it may be advantageous to replace a bridge with a culvert would include sites where:

- Maintaining traffic through a construction site is made easier,
- Bridges would create safety problems near intersections,
- A culvert could stop stream degradation by providing a fixed bottom,
- A culvert would take less time to construct,
- The culvert's freeboard requirements are not as critical,
- A long bridge was originally built because of available construction methods,
- Foundation conditions favor a culvert, and
- Culverts provide more flexibility with horizontal and vertical alignments and have lower maintenance costs.

13.6.24 Existing Structures

The following sections will discuss the influence that existing structures might have on the highway drainage structures.

Upstream Structures

Structures that have been built immediately upstream from a highway since the original highway construction can influence the design for reconstructing a drainage structure. Upstream culverts that are undersized and small dams could detain a large enough volume of water during peak runoff events to reduce peak flows at the highway site. On the other hand,

an inadequate structure may wash out and increase flows to the downstream structure. Accordingly, the highway culvert shall be redesigned considering the permanence and stability of upstream structures, any overtopping floods, and the potential risks involved.

The replacement structures can also affect existing structures upstream and downstream. If the grade of the highway is raised and an upstream structure is not raised, flooding of the upstream structure could result. This could be avoided by sizing the highway structure considering its effect on the upstream structure.

Downstream Structures

Structures located downstream from an existing highway can influence tail water elevations at the drainage structures, causing them to operate under outlet control. Downstream structures also influence the position of the highway structures. Thus the effects of all downstream structures shall be analyzed.

13.6.25 Improvements in Hydraulic Performance

Methods of improving hydraulic performance include:

- Improved inlets on culverts;
- Local channel modifications at bridges to reduce backwater;
- Bridge structure modifications;
- Debris control measures to prevent blockage of waterways;
- Spur dikes at bridge sites to improve flow alignment and reduce scour;
- Superstructure modifications to increase freeboard; and
- Removal of obsolete structures.

13.6.26 Added Structures

When improving an existing highway, there is often a need for adding drainage or multiple-use structures. A side-tapered inlet shall be routinely considered for all inlet control culverts in urban areas. Incorporating this type of inlet will increase the hydraulic capacity of inlet control structures, negating in some cases the need to employ a larger structure.

Additional drainage structures could be required for one or more of the following reasons:

- Need to provide more waterway opening;
- Anticipated future drainage needs;
- A multiple-use structure may not be functioning well and an additional drainage structure may be warranted;
- A drainage improvement may require a lower flow line and an additional small structure to carry the smaller flow;
- Elimination of a roadway overflow section due to a grade raise;
- Need to redistribute flood flows;
- Need to equalize flow such as the flow in roadside ditches; and
- Need to add storm drain systems in urban areas where widening projects eliminate roadside ditches.

13.7 References

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14 ECONOMIC EVALUATION OF HIGHWAY DRAINAGE STRUCTURES

14.1 Introduction

Economic evaluation of a project is the process of identifying and then evaluating options in order to select the one that most closely satisfies the defined project objectives. In the context of highway drainage these objectives include:

- Identifying a solution that is technically sound and is fit for its purpose;
- Being environmentally and socially acceptable while ensuring a sustainable solution;
- Identifying and mitigating the risk to people, the natural environment and project finances; and
- Ensuring best value for money from a national perspective.

The emphasis throughout this chapter is on the practical use of techniques for evaluating typical highway drainage activities. No attempt has been made to provide full coverage of the theoretical or philosophical basis behind the methods presented.

14.1.1 The Evaluation Process

The iterative nature and phases of project evaluation are shown in Figure 10.1 below. Ideally the problem identification and objective definition will be done within a strategic framework in order to encourage the widest possible impact assessment and option development. However it is acknowledged that this is not always the case where local issues are the main drivers. The process consists of:

- Defining;
- Developing;
- Comparing; and
- Selecting options.

The process uses economic, risk, environmental, social and technical considerations at each stage to build up a comprehensive evaluation that pulls together all the relevant information, leading to logical decision making and best option selection.

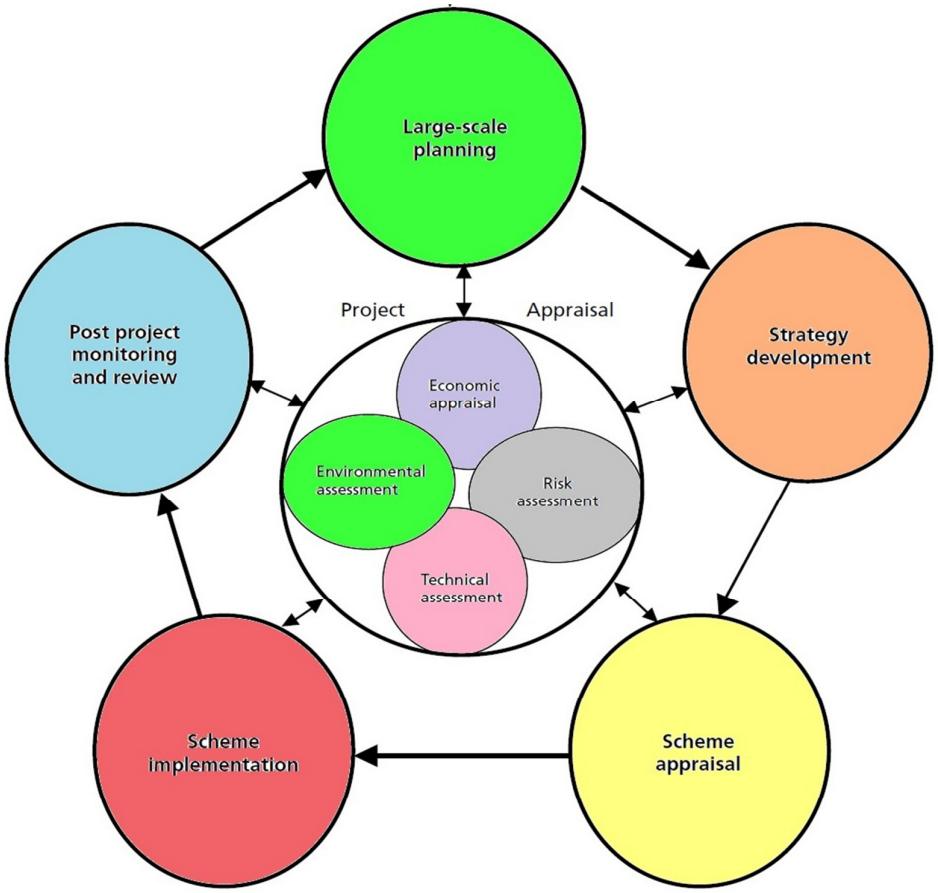


Figure 14-1: Stages of Project Planning and Development[16]

14.1.2 Decision Making

The purpose of an economic evaluation of a project is to improve decision making. Clear and transparent decisions are most likely to result from considering all economic, environmental, social and technical issues for a full range of options, together with a proper consideration of risk and uncertainty. By balancing these issues – even where it is difficult to put a monetary value on such effects – the most viable scheme should be identified. Good decision making will always include a role for:

- Informed consultation and negotiation; and
- Appeals and formal adjudication.

A poor economic evaluation, failing to properly consider potentially viable options, can open the door to extensive consultative problems if confidence is lost in the validity of the appraisal.

[16] Flood and Coastal Defence Project Appraisal Guidance 1

14.2 Basic Principles

14.2.1 Economic Costs and Shadow Pricing

A financial analysis considers the monetary costs and revenues to the business contemplating the investment. These monetary costs are the prices of goods and services in the marketplace.

In many instances the market prices for goods and services do not equate to their economic costs. This difference may occur from transfer payments, such as taxes, duties and subsidies, or of market imperfections or other factors.

It is necessary, when performing any economic evaluation, to substitute the market price of items with a value that takes account of these differences. This technique is termed shadow pricing.

A basic example of this would be where an individual household or organisation involved in a flood event claims for the loss of a 10 year old television. This loss is counted as the market price of a new television because VAT is included as are other indirect taxes as they affect the individual household or organisation involved. However, if we take the view of the nation as a whole – one person's loss can be another person's gain. The real loss to the country is a ten year old television; the depreciated value of that ten year old television is taken as the loss, VAT is excluded, as are other indirect taxes, because they are money transfers within the economy rather than real losses or gains.

Shadow pricing is assigning a monetary value to all parts of a production process, including intangible assets. This technique is used in cost-benefit analysis. Tangible assets comprise of bridges, culvert crossings, roads etc. something that an economic value can easily be put to. An intangible asset could comprise of abstract commodities, e.g. visiting parks or other facilities where costs are not easily attributed to. Where access is not achievable as a result of a road to a park being washed away for example (intangible asset), the economic value is usually identified by what customers are willing to pay or sacrifice for this benefit or what they are willing to pay so as not to suffer disappointment.

14.3 Assessing the Benefits

14.3.1 Benefits and Costs

In deriving a benefit–cost ratio (BCR), there is no universally agreed basis for classifying a particular item as either a positive cost or a negative benefit (disbenefit) or vice versa. While the particular approach that is adopted will have no effect on the net present value (NPV) of an option, it can have a significant effect on the benefit–cost ratio.

To ensure a consistent approach between options and between schemes, it is important to have a common rule. In the analysis of highway drainage structures, the following conventions have been adopted:

- Any 'negative costs' should be regarded as positive benefits.
- Any 'negative benefits' should be regarded as costs.

Benefits arising from a project, equate to the net difference between total present values damages with and without the proposed works (i.e. the damage avoided in comparison with the 'do nothing' option). Any negative benefit, or dis-benefit, arising from a project represents a loss to society and hence should be considered as a cost to the project. Conversely resources which become available to society as a result of project implementation should be regarded as a benefit.

For example, if an option restricts access of the public to a particular area, this loss of amenity should be counted as a scheme cost; if widening or dredging a river for flood alleviation yields gravel which can be sold then this is a benefit of the scheme.

14.3.2 Infrastructure – Tangible Asset

The market value of a residential property can be expected to include the value of the immediate services to that property and such local services have no economic value once the property is lost. However, separate valuations will usually be appropriate for infrastructure serving a wider area including trunk sewers, main road and rail routes, major pipelines, cables, pumping plant and other such facilities. In general, their loss can be treated as the cost of replacing those facilities elsewhere or rerouting them.

Appropriate adjustments should be made for depreciation and obsolescence. It should not, for example, be assumed that a trunk sewer, railway line or trunk telephone cable would necessarily be replaced, or that its economic value is necessarily the full replacement cost if it was likely to be abandoned or renewed, in any event, during the life of the scheme.

Bridges, culverts and surface water systems constructed primarily to convey flows of traffic and flood waters have a functional value only and including a value for such assets is likely to lead to double counting. However, they may also have a use value for recreation, which may be taken into account, where appropriate.

14.3.3 Agriculture – Tangible Asset

The analysis should be considered in terms of changes in agricultural net product (the change in net output less any change in inputs). Therefore any gains at the national level will almost certainly be less than those at the individual farm level.

Where the land in question is lost for a year it is assumed that production will transfer to other areas, leaving output of the commodities in question unchanged at the national level but displacing production in other sectors of agriculture. However, this is not always the case and where large areas of farming lands are lost as a result or in part to infrastructure failure (capacity of bridge openings and culverts reduced by silt from previous events resulting in failure of the structure) this will need to be taken into account when there is a loss of output to the economy.

14.3.4 Recreational and Environmental- Social Values – Nontangible/Tangible Assets

There are several potential approaches to the inclusion of recreational and environmental benefits, however this chapter only concentrates on one basic concept.

Recreational gains or losses should, as far as possible, be included in the benefit–cost analysis. Where only marginal changes in recreation or amenity are likely then it will seldom be worthwhile to carry out such valuations.

The benefits of avoiding a loss in recreational value, or of an increase in recreational value, can be calculated by estimating the number of visits made to the site and multiplying this by the change in the value of enjoyment per visit. Therefore, the number of visits made to the site is a primary indicator of the likely magnitude of those benefits. If there are no visitors, then there are no benefits. Therefore, any analysis should start with a preliminary estimate of the number of visits made by adults to the site.

Environmental assets have both use value (e.g. as measured through recreational use) and non-use values (e.g. existence value). Whether valued in money terms or not, environmental impacts should always be considered. Where such choices have to be made, and in a number of other situations, it will be desirable to place an explicit economic value on an environmental site or asset, in addition to any recreational value it may have. These include:

- Schemes where all, or a significant part, of the benefits arise from the protection of a site of environmental or heritage value; or
- Option evaluation where some choices have a significant environmental benefit or dis-benefit.

However, it must be recognised that it may not always be possible to express all use and non-use values in monetary terms, or even to quantify them meaningfully in some other way, and this should be taken into account in the overall appraisal. In general the least contentious and lowest cost method of deriving a proxy for the lower bound economic value of an environmental asset gained or lost as a result of a scheme can be taken as the lowest of:

- The cost of creating a similar site elsewhere of equivalent environmental value (e.g. a wetland site);
- The cost of relocating to another site (e.g. historic buildings or relocation of specially protected species); and
- The cost of local protection (e.g. a local flood embankment).

The use of such proxy values, however, will depend on there being broad agreement that the value of the asset in question is at least equal to the lowest of these figures.

The cost of re-creation or removal should be the expected present value economic cost only of those actions required to acquire the site and make the necessary changes. The normal expectation should be that long-term management costs would not change.

14.3.5 Indirect Losses[17]

Indirect losses are conceptually more difficult to measure than direct losses. Commercial firms may suffer major losses in sales but these financial losses usually do not represent national economic losses.

Consumer losses: If a shop or factory is flooded, then that company will lose sales and its customers may be inconvenienced. Potentially, therefore, there are two forms of indirect losses: to the supplier and to the consumer. In general, it is the loss to the consumer that is the economic loss; the loss to the supplier is usually a financial rather than an economic loss.

If the consumer can buy the same goods at the same cost from an alternative supplier immediately then there is no loss to the consumer. If they have to make do with inferior goods or they incur higher costs, then there may be an economic loss. However, it will only be appropriate to evaluate such losses in special circumstances if, for example, long-term loss of a rural retail outlet would likely involve significant extra travel.

Supplier Losses: If other shops or factories make up the consumers' purchases, then this is simply a transfer payment, unless those other shops or factories incur higher costs. The sales lost by one company are gained by another. The only exception is when those purchases are made up by additional imports or lost exports. Indirect losses do not normally arise from

[17] Flood and Coastal Defence Project Appraisal Guidance 3

disruption to commercial and retail activity because there are typically many alternative outlets offering the same services immediately and this need only be considered in exceptional circumstances, for example when highly specialised products are being produced.

Traffic Disruption: Disruption of road and rail networks, and also of trunk utility networks, can also result in significant indirect losses. For road networks, it will not generally be worth evaluating these unless a major through road is closed by at least the 1 in 10 year event. If flooding occurs at below the five-year return period event and a significant part of a road network carrying through traffic is affected, then the benefits of reducing traffic disruption can be very large, both in total and as a proportion of scheme benefits.

If roads are affected by flooding the traffic that usually uses those roads will have to divert to other roads. That traffic will have to travel further, and/or for longer, incurring both resource and time costs. Since the speed of traffic depends upon the volume of traffic, the existing traffic on the diversion routes will also travel slower, increasing resource and time costs.

The benefits of flood alleviation include reducing the disruption of road traffic that occurs when roads are inundated. The losses from disruption are the additional costs to each vehicle using the road network incurred by taking more time and/or travelling further to make the desired journey. Those additional resource costs are:

- The additional travel time costs
- The additional vehicle operating costs

The additional costs incurred in a flood are estimated by: number of vehicles delayed x additional cost per vehicle x number of hours that the flood disruption lasts.

In this equation it is the total number of vehicles that take longer to make journeys that is important – including the traffic already on those roads on to which traffic is diverted to avoid the flooded roads. Excluded from the equation are those vehicles that are travelling to or from an address that is also flooded.

Example[18]:

Suppose 15,000 vehicles travel through the local network each hour and they will have to travel an average 2 kilometres further with flood disruption but their average speed (40 km/h) remains the same.

The cost of that flood event will be (15,000 x $0.50 x 2) per hour (the $0.50 is estimated). If the flood lasts six hours, the costs of traffic disruption would amount to $90,000.

The problems associated with establishing the cost to the economy of traffic disruption is identifying daily traffic flows, costs per vehicle and the length of flooding. Another problem is that of estimating the diversion routes. The second issue is that the progressive development of flooding may progressively close different roads in the local network, so inducing a cascade of traffic diversions as one road after another is closed (hydraulic modelling is needed for this).

[18] The Benefits of Flood and Coastal Risk Management - A Handbook of Assessment Techniques - 2010

Where traffic disruption is likely to be severe and extend over a prolonged period, it may be more realistic to equate the economic loss to the cost of reconstructing the road, or making sufficient improvements to alternative routes to avoid the cost of delays. This should be applied only where the present value is likely to be less than that of the long-term costs of disruption.

14.3.6 Double Counting of Benefits

The benefits listed in this chapter relate to the total economic impact of positive and negative costs.

Certain external impacts of activities, such as increased land values, may arise because of the improved level of service and accessibility to nearby areas. These impacts shall be excluded from the evaluation because including them would be double counting.

Further examples of double counting are identified below:

- It would be double counting to claim increased land values as additional benefits if these benefits are merely a capitalisation of road user benefits.
- Bridges, culverts and highway drainage are constructed primarily or in part to facilitate flood waters and only have a functional value in terms of their main objective. Including the value of such assets is likely to lead to double counting.
- It would be double counting to include 'saved energy' benefits in terms of reduced travel distances.

Therefore, it is necessary to ensure a common baseline to avoid double counting of any benefits calculated (positive or negative).

14.3.7 Transfer Payments

A transfer payment occurs when a change simply affects either who gets the product or who provides the product, but there is no change in the national total of either product or all the providers required to generate that product.

The test for a transfer payment will be whether there any changes either or both in the total value of product, or in the providers required to provide that product. If not, then only a transfer payment is involved.

Examples of transfer payments are identified below:

- Tax and excise duties are always transfer payments and must be netted out of the analysis. If less petrol is sold, then the government will simply find different ways of raising taxes.
- If a hotel is lost as a result of a flood, the trade would simply transfer to other hotel.
- The losses of trade to commercial or retail outlets will be a transfer payment except in the circumstances given below.

Examples of changes which are not a transfer payment are:

- A 'green tax', which is intended to reflect a real economic cost, but appears identical to other forms of taxation. If, for example, a charge were to be levied on aggregates which reflected the real environmental damage caused by aggregate extraction then this would reflect the additional economic loss resulting from mineral workings. Therefore, an increase in aggregate extraction would result in additional economic losses to the country, in addition to the resource costs of extraction and transportation.

- Losses of trade to commerce and retail outlets result in real losses if the consumer cannot obtain an equivalent good at the same time and at the same cost. If all three conditions hold then an economic loss is involved. However, the normal expectation is that consumers will be able to obtain equivalent goods at no extra cost and therefore any differences will not be worth evaluating.

The test can also be applied to non-priced goods, such as visits to a public park. If consumers can go somewhere else and get the same amount of enjoyment at no extra cost, then the change in park visiting results in no real economic cost. If they cannot then the net value of the loss in enjoyment plus any increase in cost to the visitor measure the economic loss.

14.4 External Impacts

14.4.1 Benefits and Disbenefits

External impacts are benefits or disbenefits stemming from an activity that do not reside with the confines of the project. However because cost benefit analysis (CBA) takes the national viewpoint, external impacts must also be considered. An example of external impact is identified below and how it needs to be treated within the overall analysis:

Environmental Impacts: The impact of environmental issues is an important subset of external impacts of any project. Most countries impose a duty when preparing activities to assess the effect of the activity on the environment and environmental sustainability. The emphasis is on avoiding adverse effects on the environment. This can be achieved by:

- Reducing the negative impacts of the transport system on land, air, water, communities and ecosystems; and
- Actively moving the transport system towards reducing the use of non-renewable resources and their replacement with renewable resource.

Mitigation of external impacts: Where a scheme needs to address an external impact which will require mitigation, the costs of the preferred mitigation measure shall be included.

Quantifying and valuing external impacts: Where impacts are valued, they should be included as benefits or dis-benefits in the economic efficiency evaluation. Non-monetised impacts should be quantified, where possible, and reported as part of the funding assessment.

Transferred External Impacts: External impacts are not included in the economic evaluation when they merely represent a transfer of impact from one person to another. This is similar to any other transfer payment.

14.5 Stages in a Benefit – Cost Analysis

14.5.1 Do-Nothing – Do Something

The main stages in a benefit–cost analysis with the iterative loops are identified in Figure 14.2 below. The identification of the **'do-something'** options and the estimation of the benefits and costs of those options can be progressively refined.

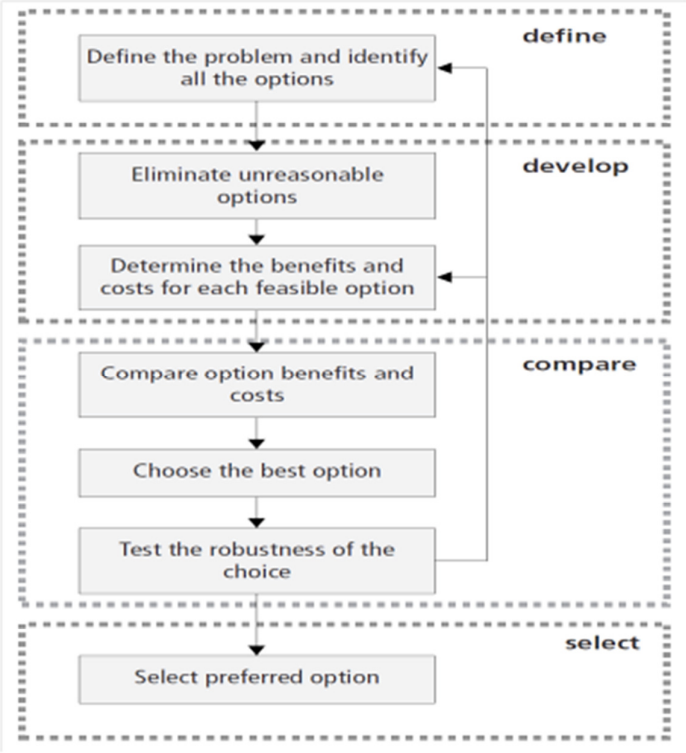


Figure 14-2: Stages in a Benefit Cost Analysis[19]

Identifying the 'do nothing' option benefits and costs for all **'do-something'** options need to be compared. This provides a convenient common baseline against which alternative options can be assessed and different investments, in different parts of the country, can be compared. It is particularly important that such a common baseline is adopted when, in a situation of national shortage of funds, the benefit–cost ratio is an important element in the ranking of schemes. Scheme benefits are calculated from the losses avoided by carrying out the proposed works rather than doing nothing.

Identifying the **'do-nothing'** option correctly is therefore critical to the analysis and needs careful consideration. Where there is no scheme in existence, then the **'do-nothing'** option is obvious; no action is taken to intervene with natural processes. Where there is a scheme at present, the option will be to **'walk away'** and abandon all maintenance and repair to the existing structure, allowing nature to take its course. Simply continuing with maintenance and repair of the existing structure is then one of the 'do something' options to be considered.

For many highway drainage activities, it is often not practical to do nothing. A certain minimum level of expenditure may be required to maintain a minimum level of service. This minimum level of expenditure is known as the **'do-minimum'** and can be used as the basis for evaluation, rather than the do-nothing.

[19] Flood and Coastal Defence Project Appraisal Guidance 1

Typically the economic value of 'do-nothing' losses will be represented by:

- The economic cost of reinstating an asset to its condition prior to flooding (with appropriate deductions for depreciation); plus
- Temporary losses due to disruption; or
- The cost of permanent replacement or relocation to an alternative location.

Typically the economic value of 'do minimum' losses will be represented by:

- Do minimum by continuing maintenance of the structures etc.; and
- Undertake minimum reconstruction works.

Typically the economic value of 'do-something' losses will assess a range of options. These should normally involve assessing:

- As wide a range of options as possible, before quickly reducing these options to a smaller range of standards and options for detailed analysis;
- Different standards of protection;
- Alternative alignments for road or defences;
- Alternative timing of works; and
- Different approaches to solution of the problem.

14.5.2 Defining the Boundaries of the Benefit – Cost Analysis

Boundaries in space and time have to be drawn for any benefit–cost analysis. A scheme option may have an influence outside that of the immediate area of the scheme, but, in addition, the consequences of the different options may depend upon what happens outside the project boundaries. For example, urbanisation or deforestation of the catchment upstream of a project would have an effect upon the flood risk in the project area. All appraisals of the consequences of the different options will therefore be conditional upon the assumptions made about these external conditions. The assumptions made should be realistic and not simply convenient.

14.5.3 The Time Span of the Analysis

A benefit–cost analysis should generally be undertaken to cover the engineering life of the longest lasting component in the longest lasting scheme, since shorter time horizons can bias the analysis towards options with short lives or high operating costs. The period chosen should allow appropriate comparison of options. There is usually little point in undertaking the analysis over a time horizon of more than 50 years because discounting means that consequences far in the future have little effect on present values.

For any economic appraisal the anticipated benefits must be compared with the expected whole-life costs of works and management required to achieve them. All relevant capital, maintenance and running costs must be included to assess the whole-life cost of a project. Maintenance costs should include the expected value of the costs of repairs as a result of damage and, where significant, decommissioning costs.

Only benefits and costs resulting from implementing each option should be considered. Sunk costs, which have already been incurred, should not be included in the appraisal.

Sunk Costs are any expenditure which has already been incurred and which consequently cannot be changed as a result of any decision about future options. Examples of sunk costs are:

- Previous investments in defences and expenditure on feasibility studies are common examples of sunk costs; or
- Alternatively, those at risk of say flooding may have invested in individual protective measures. The cost of these measures is, however, irrelevant to the appraisal of any wider scheme.

Inflation: Benefit–cost analysis should be undertaken using real prices; that is, inflation is ignored where 'inflation' has its everyday meaning of the price of a resource increasing without its relative value also increasing.

Constant/relative prices: Commonly, the relative prices of the different streams of costs and benefits are assumed to be constant over time; this is generally a conservative practice. However in reality, relative prices may change over time.

14.5.4 Damage Calculation

The damages caused by flooding are a function of the depth, duration and velocity of flooding, along with the sediment load and pollutants carried by the floodwater. Floods in Ethopia range from flash flooding in steep catchments to prolonged flooding in the lower valley areas. Therefore in the flashy catchments, flooding will result in failure of protective structures, high flood velocities and additional losses may result, for example, from partial or complete structural failure of structures.

Prolonged flooding can result in highway infrastructure being impassable for long periods of time as well as some structural damage to highway assets and increased maintenance (flood waters containing high sediment load). The sediment, debris and degree of contamination by sewage borne by the flood will affect the costs of cleaning up after the event.

14.5.5 Calculating the Benefits

The benefits of alleviation are calculated as the expected value of annual flood losses averted. As floods are assumed to be random events, it is not possible to predict when they will occur and the expected value of annual flood losses is calculated as the probability of a range of events multiplied by the loss that such an event would incur. In practice, the losses are measured by the difference in the areas under loss–probability curves for the 'do nothing' and with scheme options. This difference in area is the expected value of the reduction in flood losses each year over the life of the scheme; the average annual benefits. Refer to Figure 10.3 below. These are then discounted over the life of the scheme to give the present value of the benefits.

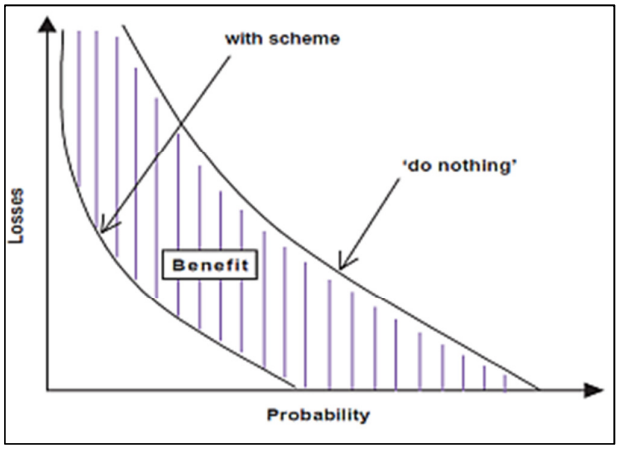


Figure 14-3: Average Annual Benefits

Changes over Time: Where changes are anticipated in the expected probabilities of flooding over the life of the scheme, it is necessary to calculate a number of different average annual benefits corresponding to different conditions. Such changes include, for example, predicted changes in rainfall patterns due to climate change or other expected changes in the catchment which are predicted to change the rate of run-off and the frequency of flooding (urbanisation, deforestation etc.). Alternatively, changes over time in the use of the flood plain may change the losses expected from a flood of a given magnitude. In these cases, average annual benefits should be calculated for appropriate years, and values interpolated for intervening periods.

Upper Limit of Losses: Care should be exercised where the total present value of losses exceeds the current write-off value of the asset. In the case of domestic or commercial property, it will usually be prudent to assume that the long-term economic loss cannot exceed the current capital value of the property. In the case of other assets, such as roads, railway lines, pipelines or cables, some very large values can be generated for long term disruption.

It will often then be reasonable to assume that the maximum economic benefit derived from flood protection is equal to the economic cost, depreciated to allow for the age of the existing asset, of reconstructing an equivalent facility at a higher level or on an alternative alignment which avoids the flood risk.

14.5.6 Assessing Probability Events to use in the Analysis

The loss–probability curve is generally calculated using only a very small sample of the possible probability events (return periods) that might be considered. This will be informed by the proposed design life of a particular project. The overall form of the curve, and the area under it, is then calculated by drawing straight lines between the calculated points. This can, potentially, result in wrong estimates of the area under the curve. Refer to Figure 14.4 below. In this illustration, the choice of return periods, when compared to the 'true' relationship, has resulted in a significant overestimate of the overall losses.

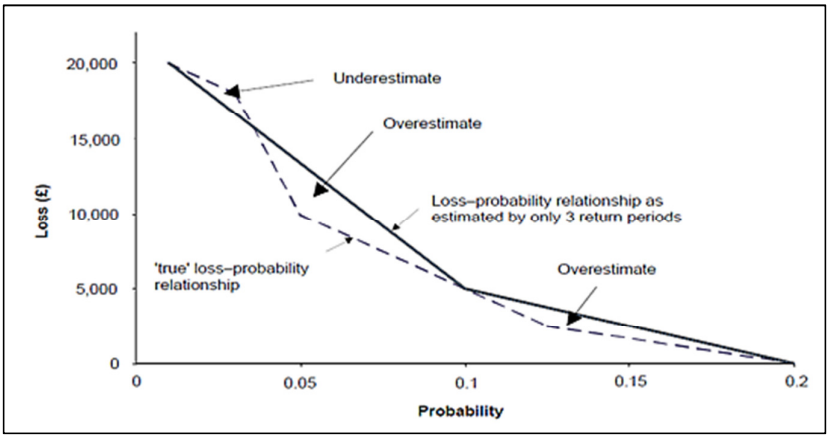


Figure 14-4: Accuracy of Estimation of the Loss-Probability Curve

Determining how many and which probability events to include is a sampling problem. The aim is to obtain a reasonably close approximation to the loss–probability curve representing an infinite number of probability events if these were to be modelled. The ideal number of probability events to use is those which are located at discontinuities on the loss–probability curve.

First, it is important to locate the probability event of the threshold flood event: (i.e. the most extreme flood that does not cause any damage). Secondly, engineering judgement should be used to assess where the discontinuities are likely to be, since these occur, when new areas of property start to flood. Thus, for example, they can be expected to occur when an existing natural or man-made structure is overtopped or a culvert or bridge reaches its capacity. Thirdly, the greatest proportion of benefits generally arises from the shorter return period events. Consequently, the sample of events included should usually be biased towards these events.

Thus, it is likely to be better to include the 5, 10, 25, 50 and 100-year events rather than the 5, 20, 35, 50 and 100- year events when assessing the benefits of a scheme with an expected 1 in 50-year design standard of protection. However, a few judiciously chosen events at appropriate points of discontinuity will generally produce a more realistic result than a larger number of events at standard intervals.

Discontinuities on this curve indicate those events that should be included, provided that the information that produces the discontinuities was accurately provided in the data input.

As a general rule the following is recommended:

- The benefits should be calculated using a minimum of three events and the choice of those three events should be considered carefully; and
- One of these events should normally be the threshold flood event.

14.6 Present Value and Discounting

14.6.1 Present Value

Present Value is a technique used to compare costs and benefits that occur in different time periods. A community places a higher value on benefits and costs that occur in the near future, compared with those that occur at a later date. This is known as ‘time preference’.

The discount rate is used to convert all costs and benefits to 'present values', so that they can be compared. The recommended discount rate is between 5%[20] and 10%[21]. Calculating the present value of the differences between the streams of costs and benefits provides the net present value (NPV) of an option. The NPV is one of the primary criteria for deciding whether government action can be justified.

Discounting to present value involves calculating the current equivalent value of a cost or benefit associated with a project, given a prevailing interest (or discount) rate. The current equivalent monetary value of a cost or benefit that will be received in the future is called its present value. The formula used to calculate the present value of a future cost or benefit in monetary terms is:

$$PV = \frac{F}{\sum(1+r)^n}$$

Where:

PV = present value

F = future value of cost or benefit in monetary terms

r = the rate of discount

n = no. of periods under consideration (e.g. years)

Example 1

The following example identifies where present value would be applied on a highway scheme. An existing road is being washed away on a regular basis. If the exiting culvert was upgraded the benefit in one years' time are estimated at $30, 000. The discount/interest rate for this project has been assumed to be 5% (this figure to be set by national bank). We want to know what the present value of project will be in one years' time. Using the above formula the present value of $30,000 in one years' time will equate to:

$$PV = \frac{\$30{,}000}{\sum(1+0.05)^1} = \$28{,}571.43$$

Table 14.1 below shows how the present value of $1,000 declines in future years with a discount rate of 3.5 per cent.

Table 14-1: Present Values and Discount Rate

Time	0	1	2	3	4	5	6	7	8	9	10
PV of payment	$1,000	$966	$934	$902	$871	$842	$814	$786	$759	$734	$709

14.6.2 Net Present Value and Benefit Cost Ratio

Benefits and costs which have been monetised can be directly compared to assess whether a proposed option is cost-beneficial. Whether a BCR or an NPV is used as the main summary statistic of appraisal, the chosen method will depend partly on standard practice amongst

[20] Social Benefits in Transport Planning A Case Study of Ethiopia – Jan 2004
[21] Ethiopia's Climate-Resilient Green Economy – Sept 2011

organisations. Although Net Present Value gives the overall net "worth" of an option, it tends to favour larger or more involved projects. However, organisations with limited budgets tend to use the Benefit-Cost Ratio on the options especially if they are interchangeable, which gives a measure of return per dollar of cost, regardless of project size.

Net Present Value

The Net Present Value (NPV) of a project is equal to the difference between the present value of the benefits and the present value of the costs. This can be expressed using the following equation:

NPV = Σ PV (B) - Σ PV (C)

Where,

NPV = net present value

PV (B) = present value of the benefits

PV (C) = present value of the costs

This equation is also sometimes written as:

NPV = Σ present value of total future benefits - Σ present value of total future costs

The outcome of the NPV calculation provides an easy to interpret result for the project. If NPV > 0, then it follows that the project has economic justification to go ahead. There is economic justification for the project as long as the sum of the present value of the benefits is greater than the sum of the present value of the costs.

Example 2

The following examples identifies where net present value (NPV) would be applied on a highway scheme. A major highway crossing is being washed away on average once every five years. The present value (PV) associated with reconstructing the crossing is $750,000. The design life of the road is 25 years and is a major artery to and from urban and rural areas. It has been identified that a new crossing can be constructed for a cost of $1,250,000 and will not be washed away during its design life. With a discount rate of 5% the NPV of the project is as follows:

Assuming on average the highway crossing will be washed away every 5 years, the present value of repairs over the design life is as follows:

- The PV of repairs in 5 years equate to $587,644.62
- The PV of repairs in 10 years equate to $460,434.94
- The PV of repairs in 15 years equate to $360,762.82
- The PV of repairs in 20 years equate to $282,667.11

Total present value cost of repairs over the design life equate to $1,691,509.49

The present value construction cost equates to $1,250,000.00

The net present value = $1,250,000.00 - $1,691,509.49 = -$441,509.49

Therefore the above figure shows that in terms of net present value, the more expensive scheme should be built now rather than replacing the crossing every five years.

However, applying a discount rate of 10% the NPV indicates that replacing the crossings every five years provides a more favourable NPV.

Example 3

A second example on NPV looks at a construction project with initial costs of $1.5m. Benefits arising from this project are $100,000 at the end of year 1, $200,000 at the end of year 2, $500,000 at the end of year 3, and $1m at the end of year 4.

With no discounting the NPV of this project is as follows:

NPV = Benefits - Costs

NPV = $1.7m - $1.5m

NPV = $200,000

With a discount rate of 5% the NPV of the project is as follows:

$95,238.10 + $181,405.90 + $431,918.8 + $822,702.47

NPV (B) = $1,531,265.30

NPV = $1,531,265.30 - $1.5m

NPV = $31,265.30

With a discount rate of 10% however the NPV of the project is as follows:

$90,909.09 + $165,289.26 + $375,657.40 + $683,013.46

NPV (B) = $1,314,869.20

NPV = $1,314,869.20 - $1.5m

NPV = -$185,130.79

In this example without discounting there is economic justification for the project to go ahead. With a discount rate of 5% there is justification for the project to go ahead. However, with a higher discount rate of 10% the costs outweigh the discounted benefits by almost $200,000, suggesting the project should not go ahead. In this case the choice of discount rate could have a significant impact on the viability of the construction project.

Benefit Cost Ratio

The Benefit Cost Ratio (BCR) can be used as a method of obtaining a result for a cost benefit analysis (CBA). It provides a value by calculating the ratio of the sum of the present value of the benefits associated with a project and comparing them against the sum of the present value of the costs associated with a project. The BCR can be expressed as follows:

$$BCR = \frac{\sum PV(B)}{\sum PV(C)}$$

Where;

BCR = benefit cost ratio

PV (B) = present value of the benefits – as a result of a scheme in place

PV (C) = present value of the costs – without a scheme in place

BCR can also be expressed as:

$$BCR = \frac{\Sigma \text{ present value of total future benefits}}{\Sigma \text{ present value of total future costs}}$$

In obtaining a result using BCR you would interpret the BCR value as follows. It would not be rational to proceed with the project if the BCR value is below unity (1). The greater the value above unity (1), the greater the benefits associated with the alternative considered. Using BCR the scheme with the greatest BCR should be chosen.

Choosing an Option - Comparing NPV and BCR

It should be noted that although both NPV and BCR will provide the same positive or negative outcome for an alternative, where a number of options are considered, the two methods will not always give the same preferred outcome. This is important as the choice of calculating the outcome of the CBA using only one of these methods could result in the CBA not considering an alternative that actually offers a positive outcome.

Example 4:

The following example, considers a project seeking to increase the level of flood protection to an urban area and its infrastructure. Two alternatives have been considered for this project (not including a 'do-nothing' approach).

Alternative 1 involves building a brand new flood defence wall, providing additional protection to existing structures to prevent scour and erosion, together with implementing a dredging project. The total present value of the costs associated with the alternative equal $100m. The total present value of the benefits is equal to $140m.

Alternative 2 involves upgrading the existing defences and repairing where necessary. This will also involve providing some minor protection to existing structures to prevent scour but excludes the dredging project. The total present value of the costs in this project alternative is equal to $20m. The total present value of the benefits is equal to $35m.

Calculating the NPV and BCR for these alternatives gives the following results:

Project Alternative 1

Costs = $100m

Benefits = $140m

NPV = $140m - $100m = $40m

BCR = $140m/$100m = 1.40

Project Alternative 2

Costs = $20m

Benefits = $35m

NPV = $35m - $20m = $15m

BCR = $35m/$20m = 1.75

From this simple example it can be seen that while both alternatives provide a net positive outcome, the NPV and BCR methods of obtaining results provide slightly different outcomes. Using NPV suggests project Alternative 1 provides the better outcome as the NPV of $40m is greater than the NPV of alternative 2 ($15m). However, using the BCR method Alternative 2 would be chosen as a BCR of 1.75 is greater than Alternative's 1 BCR of 1.40.

In this case the overall result of the CBA may be determined by considering the costs involved in Alternative 1 which are much greater, or may be determined by considering the overall much greater benefits (in monetary terms) obtained by choosing Alternative 1.

It may be appropriate for the assessor to provide the client or decision maker with all the most recent available information with which to make their final decision. However, some analysts (and textbooks) argue that the BCR can confuse and provide inaccurate information as it is a ratio and therefore ignores the overall monetary value of the benefits associated with an alternative. In addition, the BCR is more sensitive to willingness-to-pay.

The method of choice will be left to the bodies undertaking the work however, both methods should be undertaken to ensure value for money is achieved.

Incremental Cost Benefit Analysis

Where activity alternatives and options are mutually exclusive, incremental cost benefit analysis of the alternatives and options shall be used to identify the optimal economic solution.

The incremental BCR indicates whether the incremental cost of higher-cost activity alternatives and options is justified by the incremental benefits gained (all other factors being equal). Conversely, incremental analysis will identify whether a lower cost alternative or option that realises proportionally more benefits is a more optimal solution.

An example of this is where benefits and costs vary fairly smoothly with no major discontinuities. It might be a river embankment reconstruction where the height and width of the embankment are directly related to the risk of breach or overtopping. Typical benefit and cost curves are S-shaped; although the upper and lower limbs are by no means symmetrical and may even appear truncated. A common situation is that schemes find themselves on the convex (upward) part of the benefits curve and the concave part of the cost curve. This is the apocryphal situation known as ‘the laws of diminishing returns.

Example 5:

The following example identifies how this philosophy is implemented. For the purpose of this exercise, an indicative standard of protection for various land uses are identified in Table 10.2 below.

Table 14-2: Indicative Standards Of Protection

Indicative standards of protection			
Land Use Band	**Description**	**Return Period (Yrs)**	**Annual Probability**
1	Heavily urbanised	100 - 300	0.01 - 0.003
2	Medium dense urban area	50 - 200	0.02 - 0.005
3	High grade agricultural land and less dense urban areas	10 - 100	0.10 - 0.010
4	Standard/typical mixed grade agricultural land with some properties	2.5 - 20	0.40 - 0.05
5	Typical Low grade agricultural land, at risk of flooding	< 5	> 0.20

The two situations shown below demonstrate different standards of protection for the scheme of choice dependent upon both the indicative standard of protection and the economic efficiency of the scheme. It is of interest to note that, because of the interplay between costs and benefits there is very little difference in practical terms between the benefit–cost ratios for schemes in the 1 in 10-year to 1 in 100-year range.

Other factors, including affordability may well dominate the actual final choice within this range.

However, in economic terms, there is no point in spending more on a higher standard of defence than the expected losses that would be saved. The higher indicative standards for urban areas recognise the higher probability that there will be un quantified intangible benefits. In this case raising the standard from 1 in 100 to 1 in 200 would cost an extra $1.1m but would achieve only $0.5m of benefits. Clearly there will be a better use for this money elsewhere.

Table 14-3: Costs, Benefits and Benefit–Cost Ratios against Standard of Protection

Return period (Yrs)	Flood Levels (m)	Benefits ($m)	Costs ($)	Benefit-cost ratio	Incremental benefit-cost ratio
1	3.50	4.50	1.500	3.00	
2.5	3.70	7.25	1.950	3.72	6.11
10	4.00	11.75	2.500	4.70	8.18
25	5.00	15.50	2.875	5.04	7.33
50	5.50	16.00	3.250	4.92	4.00
100	6.00	16.75	3.625	4.62	2.00
200	6.50	17.25	4.750	3.63	0.44
300	6.80	17.50	5.50	3.18	0.33

14.6.3 The Internal Rate of Return

The break even discount rate is sometimes referred to as the Internal Rate of Return (IRR). The value of the break even discount rate or IRR can be used in two ways. It can be used to compare projects. For example, a project with an IRR of 6% is equivalent to another project of IRR 6%, at least in economic terms. This is particularly useful for projects when comparing with a 'do nothing' approach. If the IRR value, that is the value at which NPV is equal to zero, is higher than the interest rate used in the CBA, then this provides economic justification for the project to go ahead. If, however, the IRR is lower than the interest rate, this suggests the project should not go ahead.

The IRR method has a number of disadvantages compared with the NPV method which are as follows:

- It ignores the relative size of the investments;
- There are problems with its use when a project has non-conventional cash flows or when deciding between mutually exclusive projects; and
- Discount rates which differ over the life of a project cannot be incorporated into IRR calculations.

Example 6:

A flood defence wall with a design life five years is proposed to be constructed to protect existing assets that re to be removed at the end of five years (major electricity sub-station). The cost of this sub-station equates to $1,300,000 and the benefits afforded to the substation equates to $250,000 in year 1, $350,000 in year 2, $600,000 in year 3, $100,000 in year 4 and $400,000 in year 5.

The internal rate of return is where the NPV $_{income}$ = NPV $_{expenditure}$ = (NPV= 0)

$$NPV_{income} = \frac{\$250{,}000}{(1+i)^1} + \frac{\$350{,}000}{(1+i)^2} + \frac{\$600{,}000}{(1+i)^3} + \frac{\$100{,}000}{(1+i)^4} + \frac{\$400{,}000}{(1+i)^5}$$

$$NPV_{expenditure} = \frac{\$1{,}300{,}000}{(1+i)^0}$$

$$\boldsymbol{Therefore}: \frac{\$250{,}000}{(1+i)^1} + \frac{\$350{,}000}{(1+i)^2} + \frac{\$600{,}000}{(1+i)^3} + \frac{\$100{,}000}{(1+i)^4} + \frac{\$400{,}000}{(1+i)^5} = \frac{\$1{,}300{,}000}{(1+i)^0}$$

Solving this equation (solving for i) is an iterative process which results in IRR of 9.525%

Therefore the internal rate of return for this project needs to be 9.525% minimum or greater to make it a viable option.

14.7 Sensitivity Analysis

14.7.1 Sensitivity Criteria

In order to ensure that the discount rate (and/or horizon value) used in the analysis is appropriate and is not solely responsible for the outcome of a project appraisal, it would be prudent to undertake a sensitivity analysis. This can be used to improve the robustness of a CBA, and is particularly useful where there is uncertainty over the discount rate.

14.7.2 Testing Sensitivity by Varying the Discount Rate and Horizon Value

The analysis may vary the discount rate and the horizon value in order to test the sensitivity of the CBA. This is a particularly simple procedure to complete when using a spreadsheet to conduct a CBA. Projects, whose costs are all incurred early, for example many construction projects, may be especially susceptible to discount rate sensitivity. This is because a higher discount rate reduces the present value of the benefits which are accrued over time, while having no effect on the initial costs involved in the project.

14.7.3 Other Forms of Sensitivity Analysis

In addition to testing for sensitivity relating to discount rate and horizon values, it may also be necessary to consider the sensitivity of the CBA to other measures. A range of situations may call for the use of sensitivity analysis of some form. These include exploring the effects associated with changing:

- Values given to time;
- Wage rates used;
- The value of life, or risks associated with certain occupations;
- The multiplier effect applied to the economic impact of an investment or scheme; and
- Environmental impacts.

The analysis must determine how changes in these measures affect the outcome of the CBA. An analysis may wish to perform additional sensitivity analysis in any of these cases, especially if the result of the CBA is close to zero (using NPV), or the BCR is close to one. However, if a particular cost or benefit that accounts for a large proportion of the overall costs or benefits associated with a project is calculated using a measure which is subject to some sort of arbitrary value or fraction, sensitivity analysis may be appropriate.

14.8 References

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2. American Public Works Association. Urban Stormwater Management, Special Report No. 49. 1981. American Public Works Association, 1313 East Sixtieth Street, Chicago, Illinois.
3. U.S. Department of Transportation, Federal Highway Administration, Culvert Inspection Manual, FHWA-IP-86, July 1986.
4. U.S. Department of Transportation, Federal Highway Administration, Integration of Maintenance Needs into Preconstruction Procedures, FHWA-TS78-216, 1978.
5. AASHTO, 1992. Highway Drainage Guidelines, Vol. 8 - Hydraulic Aspects of Restoration and Upgrading Highways.

15 WEB-BASED LINKS AND SUPPORTING SOFTWARE

15.1 Introductions

The main purpose of this chapter is to provide relevant web-based links to freely available software and supporting reference materials which can assist the road drainage designer to accomplish their task in a timely and an efficient manner. A supporting DVD is enclosed at the back of the revised ERA drainage manual which contains linmks to additional reference materials, freely available data and computer software. Theses item are discussed below.

15.2 Web-Based Software and Reference Materials

Links to freely available software and reference materials relevant to the ERA Road Drainage Design Manual is included in this section.

15.3 Supporting DVD

A DVD is enclosed to this manual which contains supporting documentation on a number of topics that can be downloaded from the internet. Photographs of culverts, bridges and other road drainage features collected during the manual review site visit are catalogued in the DVD. The DVD contains the following information:

- The revised drainage design manual;
- Links to data and reference materials;
- Site and hydraulics srtruacture photographs;
- Links to drainage design software; and
- Other links to relevant websites.

15.4 Computer Programs

Links to relevant and freely available hydrologic and hydraulic software which are used in road drainage design includes the following:

15.4.1 Hydrologic Computer Programs

The rapid development of computer technology in recent years has resulted in the development of many mathematical models for the purpose of calculating runoff and other hydrologic phenomenon. Hydrograph methods can be computationally involved, so computer programs such as HEC-1, HEC-HMS (developed by COE), TR-20 (developed by NRCS), Storm Shed and HYDRAIN (developed by FHWA), are almost exclusively used to generate runoff hydrographs. Many of these programs will also hydraulically route these hydrographs, which is important in the design of storm water detention, other water quality facilities, and pump stations. They can also be used to evaluate conveyance flows by routing through large storm drainage systems to more precisely reflect flow peaking conditions in each segment of complex systems. Since the above-referenced techniques require a large amount of time and data to perform basin-wide computer simulation and rainfall runoff modeling, these should be reserved for larger or special projects that justify applying the additional time and manpower. The additional time and cost must be balanced against the need for a higher level of detail and accuracy that can be obtained. As a word of caution, the designer should refrain from accepting the computer generated input without

questioning the reasonableness of the results obtained from a hydrologic viewpoint. A sanity check should always be undertaken on results generated by any software.

Depending upon the amount of detail and accuracy of the input data, these models can be very efficient at showing conveyance system response to particular designs and assist the designer in preparing an adequate conveyance or flood protection system in the most efficient and cost effective manner.

HEC-HMS

Step by step hydrologic modelling using HEC-HMS is presented in the following section. The intent of this exercise is to introduce the drainage designer to the structure and some of the functions of the HEC-Hydrologic Modelling System (HEC-HMS).

Computer Requirements

The designer must have a computer with the latest windows operating system, and HEC-HMS v.3.5 or the latest version installed.

HEC_HMS v.3.5 or the latest version for windows is available for free from the Hydrologic Engineering Centre's home page at: http://www.hec.usace.army.mil/software/hec-hms/download.html. A user's and technical manuals are also available at this location.

Data Requirements

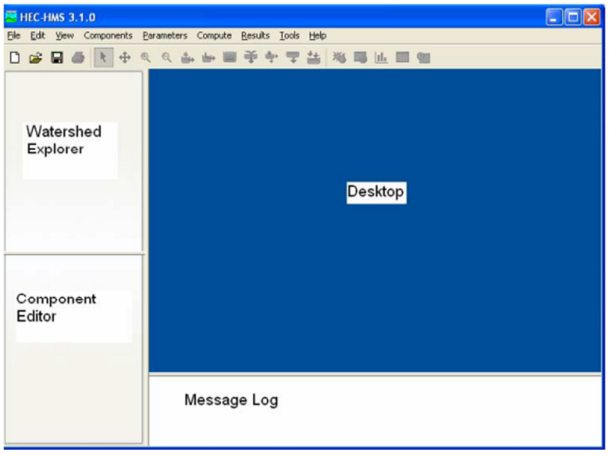
To run the HEC-HMS model, a basin file is needed to specify the physical parameters of the watershed, and a map file to give the outline of the drainage areas and streams. For example files for Waller stream (zipped as waller.zip) can be downloaded from the following link:

http://www.meconsultingm.com/projctes/modelling/waller.zip This data is reproduced from a project undertaken at Purdue University.

Getting Started

Start HEC-HMS by clicking on the HEC-HMS icon by going to Start/Programs/HEC-HMS/HEC-HMS 3.5

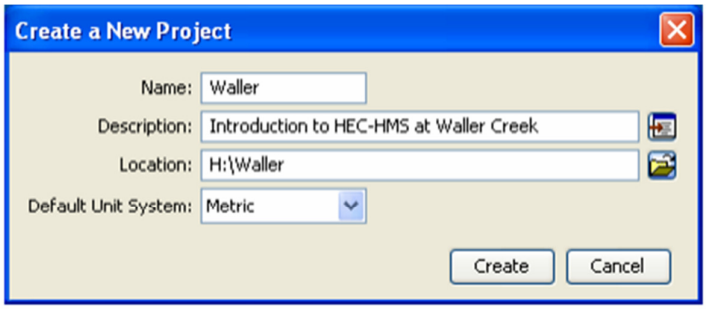
After a few seconds, the following should appear:



Henceforth, this window will be referred to as HMS Interface. The HEC-HMS interface consists of a menu bar, tool bar, and four panes. These pages are referred to as the Watershed Explorer, the Component Editor, the Message Log and the Desktop. More description on these pages, menus and tools is provided when they are used later in the exercise.

Creating a new HEC-HMS project

The first step is to create a new HEC-HMS project by selecting *File/New* in the menu bar. Enter the project name and description as shown below, and specify your working directory (where waller.zip is downloaded).



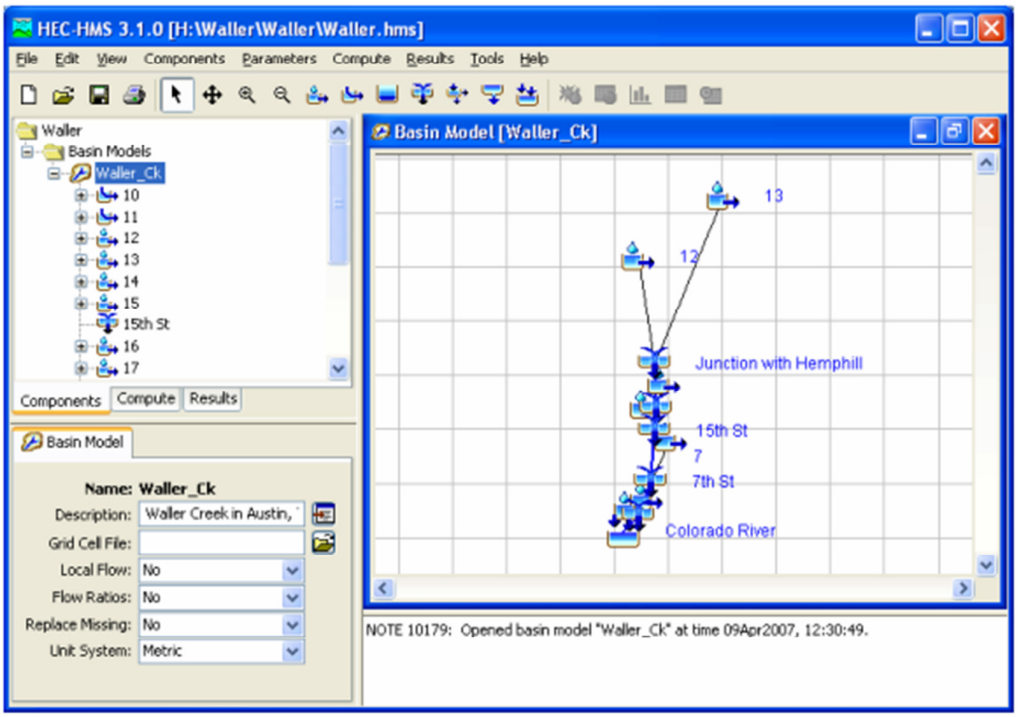
Click Create to create the Waller project.

HMS Model Components

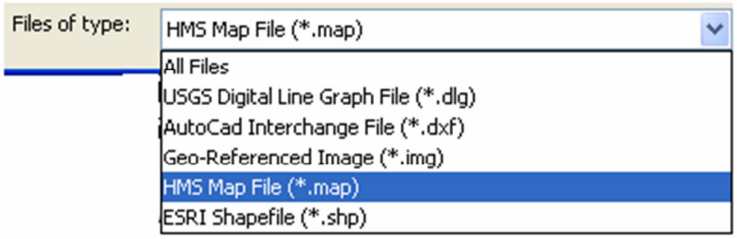
HMS has four main model components (you can see these by selecting components on the menu bar): ***basin model, meteorologic model, control specifications and input data (time series, paired data and gridded data).*** The Basin Model, for instance, contains information relevant to the physical attributes of the model, such as basin areas, river reach connectivity, or reservoir data. Likewise, the Meteorological Model holds rainfall data. The Control Specifications section contains information pertinent to the timing of the model such as when a storm occurred and what type of time interval you want to use in the model, etc. Finally, the input data component stores parameters or boundary conditions for basin and meteorological models. Each of the sections is explored below individually.

Importing a Basin Model

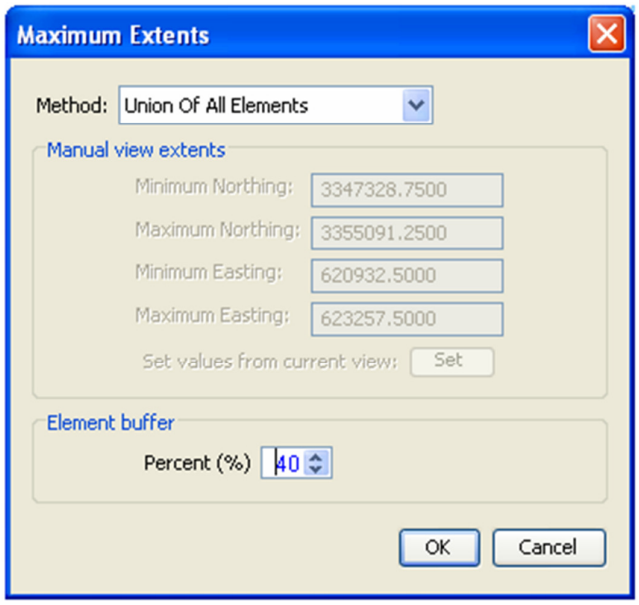
The first of the Components we will consider is the ***Basin Model***. To create a hydrologic model of Waller Creek, you need to import the basin file that you just downloaded. In HMS interface, use ***File/Import/Basin*** Model… Browse to Waller_Ck.basin in the working directory, and click Select to add it to the HMS project. To see the Waller Creek basin model in the Desktop window, expand (by clicking on the **+** sign) the folders in the Waller folder until you get to Waller_Ck as shown below.



If you expand Waller_Ck, you will see different hydrologic elements in the basin in the watershed explorer. Before we get into the details of the basin, go ahead and add the basin map which is stored in the Hms.map file. In the HMS interface, select ***View/Background Maps***. This will prompt a Background Maps window. In the Background Maps window, click on Add, ***Browse to the Hms.map*** file in the working folder (make sure you change the Files of type to HMS Map File (*.map) as shown below), and select it.



This will add Hms.map in the Background Maps window. Click OK to add it to the HMS You should then see a schematic of Waller Creek showing the watershed and stream map and an overlay of the hydrologic elements. You will notice that the whole map of the basin does not fit in the Desktop window. You can change the Desktop settings by selecting View→Maximum Extents in the Menu bar. Select the display method as Union of All Maps and Elements with an element buffer of 40 % or more as shown below:



Now you should see the whole basin in the desktop window. Save the project.

Navigating the HMS Desktop

You can use the following four tools in the tool bar to navigate through the HMS desktop:



The arrow tool lets you select any hydrologic element in the basin. You can use the zoom-in tool to zoom-in to a smaller area in the desktop and zoom-out tool to zoom out to see a larger area. The pan tool can be used to move the display in the desktop. Go ahead and experiment with these buttons to understand better how each works.

Now let us explore the basin information.

15.4.2 Hydrologic Elements

The Waller_Ck basin contains different hydrologic elements. The following description gives brief information on each symbol that is used to represent individual hydrologic element.

Subbasin **–** Used for rainfall-runoff computation on a watershed.

Reach – Used to convey (route) streamflow downstream in the basin model.

Reservoir – Used to model the detention and attenuation of a hydrograph caused by a reservoir or detention pond.

Junction **–** Used to combine flows from upstream reaches and sub-basins.

Diversion – Used to model abstraction of flow from the main channel.



Source – Used to introduce flow into the basin model (from a stream crossing the boundary of the modeled region). Source has no inflow.

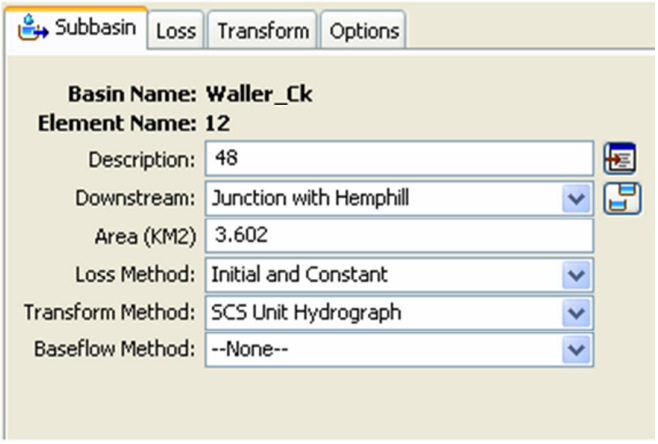


Sink – Used to represent the outlet of the physical watershed. Sink has no outflow.

The example model contains only 4 of these kinds of elements. There are 18 hydrologic elements in the model, made up of 7 sub basins, 5 river reaches, 5 junctions, and 1 sink at the point where the stream flows into the main River. Notice that when a stream flows through a watershed, the additional local runoff from the drainage area around the stream is not accounted for until the downstream end of the reach where its flow is combined at a junction with the flow coming from the upstream reach. The junctions have been located at points where roads cross the steam.

15.4.3 Editing a Basin Model

Make sure the stream basin is expanded in the watershed explorer to see all the hydrologic elements in the basin. Select the Arrow tool from the tool bar, and click on sub-basin 12 icon in the watershed explorer. After this sub-basin is highlighted, information related to this sub-basin will appear in the component editor window (lower left corner).

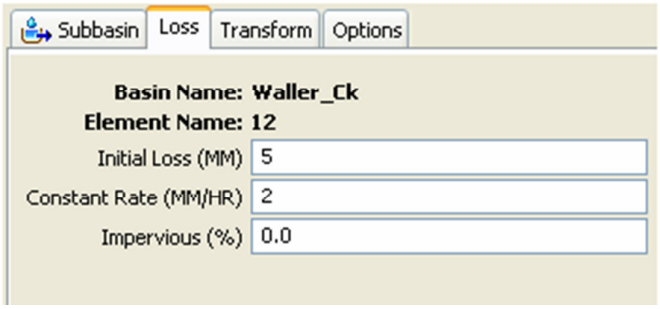


Remember the sub-basin element is used to convert rainfall to runoff. So the information on methods used to compute loss rates, hydrograph transformation and base flow is required for each sub-basin element. The loss method allows you to choose the process which calculates the rainfall losses absorbed by the ground. Click (do not select any!) on the drop down menu to see your choices. Some options are SCS Curve No. and Green & Ampt. In this model, Initial and Constant has been selected. This loss relationship means that a quantity of rainfall will be absorbed by permeable soil initially, and a constant rate will be absorbed over the time frame of the model. The loss method will convert the rainfall hyetograph to excess rainfall (chapter 5 in the manual).

The Transform method allows you to specify how to convert excess rainfall to direct runoff. Again, click on the drop down menu to view your options. This model employs the SCS technique (dimensionless unit hydrograph Chapter 5). The modClark model takes gridded rainfall data, subtracts the losses as specified through the Loss Rates, and converts the excess rainfall to a runoff hydrograph using a variation of what is known as the Clark unit hydrograph. There is no base flow method specified for this model, but you can look

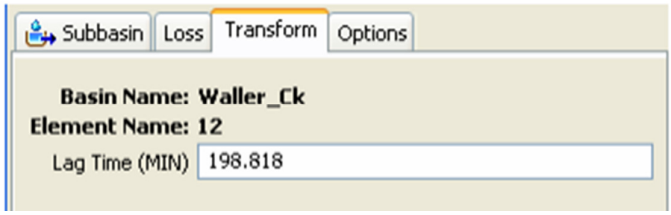
at the available options. If we specify base flow, this base flow will be added to the resulting direct run-off hydrograph to produce total stream flow hydrograph.

Once the loss and transform methods are chosen for the sub-basin, the next step is to specify the parameters for these methods. Select the Loss tab in the component editor to look at the parameters for the loss method.



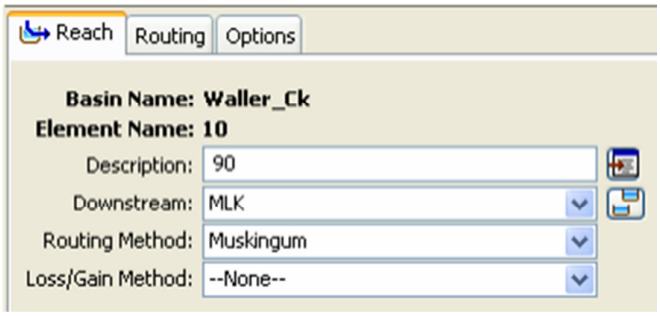
Each sub-basin requires an initial loss quantity, a constant loss rate, and a percent imperviousness. These values have been selected arbitrarily. If the % impervious value differs from 0, that % of the land area is assumed to have no losses and the loss method is applied only to the remainder of the drainage area

Similarly select the Transform tab to look at the parameters for the transform method.



Note that the SCS unit hydrograph method requires only one parameter for each sub-basin: lag time between rainfall and runoff in the sub-basin. The parameter that is specified here is tp, and the program will compute Tc (time of concentration) and qp (peak flow) to rescale the SCS dimensionless unit hydrograph (Chapter 5). This is then used to compute the direct runoff hydrograph for this sub-basin.

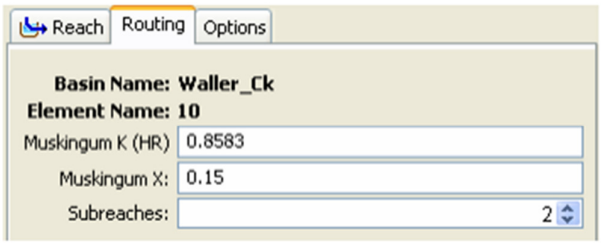
The Options tab is used to enter observed stream flow and stage data which is left blank for this model. After the sub-basin element, let us look at a reach element. Click on reach 10, and look at its parameters in the component editor.



Since the reach element route flows, only one method (routing) is associated with it.

Click on the drop-down menu to look at choices available for routing flows. The Muskingum method is specified here, which is the routing technique used for the reaches in this model. Routing is described in chapters 5 and 10 in this manual.

Select the Route tab to look at the parameters for the routing method (Muskingum).

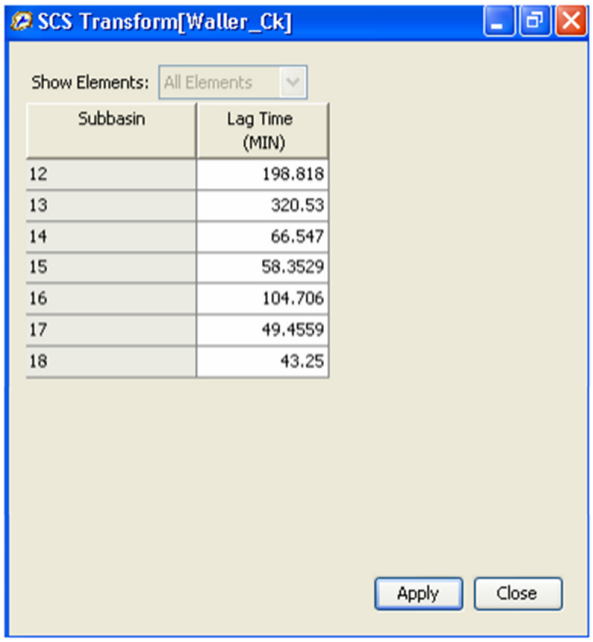


This simulation routes the water through the reaches by the Muskingum method in which

K is the travel time of a flood wave passing through the reach, X is a measure of the degree of storage (X = 0 means a level-pool reservoir or maximum storage, X = 0.5 means a pure transmission reach in which there are no storage effects, and X ranges between 0 and 0.5). The reach is divided into a number of sub-reaches if necessary to keep the computations numerically stable.

You can explore the junctions, source and sink elements to see how they are specified.

There are a couple of ways to look at Basin model. If you expand the sub-basin model (by clicking the + sign next to it) in the watershed explorer, you will see the methods specified for the sub-basin, and when you click on the method, you see the parameters in the component editor. Alternatively, you can look at the parameters for all hydrologic elements by selecting Parameters in the menu bar and selecting a method. For example, by selecting Parameters →Transform →SCS Unit Hydrograph gives a list of lag times for all the sub-basins in the model as shown below:

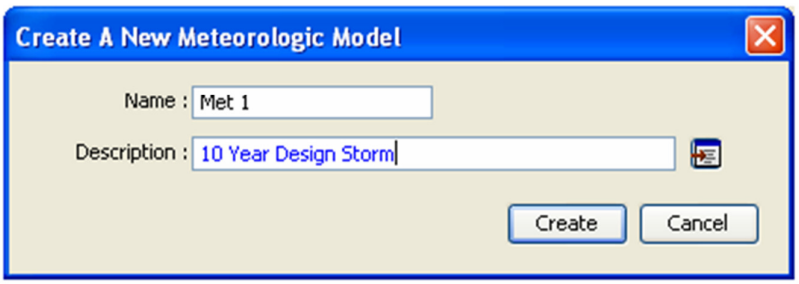


15.4.4 Creating a Meteorological Model

Having established the Basin Model, we will now create the Meteorological Model. The precipitation and evapotranspiration data necessary to simulate a watershed are stored in the meteorological model. From a statistical study of extreme storm rainfall data recorded at gages, tables and maps have been prepared for the area which specifies the storm precipitation depth to be expected as a function of the return period of the event and the duration of the rainfall. A table of such values is shown below for demonstration purposes, in which the crossing is located. For example, for a 10 year return period event, we expect that in 5 min 21.59 mm of rainfall will fall, in 15 min 42.16 mm, and so on, up to 720 min (12 hours) and 1440 min (24 hours). These precipitation depths are the values which would be equalled or exceeded on average once in 10 years when considering a very long period of data. As rainfall duration increases, the cumulative depth of rainfall increases but the average intensity over the duration decreases because severe rainfall cannot be sustained for very long.

Design Precipitation Depth for the Crossing Site (mm)						
	Return Periods (years)					
Duration (min)	**2**	**5**	**10**	**25**	**50**	**100**
5	15.24	19.05	21.59	24.89	27.18	31.50
15	29.21	37.34	42.16	49.02	53.85	61.72
60	49.28	64.77	73.91	86.61	96.52	109.47
120	59.69	79.50	90.68	106.93	120.40	135.89
180	66.04	88.14	101.09	119.63	135.64	152.91
360	76.96	104.39	120.65	143.51	163.83	184.91
720	89.92	123.70	141.48	170.43	196.85	220.73
1440	103.12	144.02	165.61	201.68	235.20	263.91

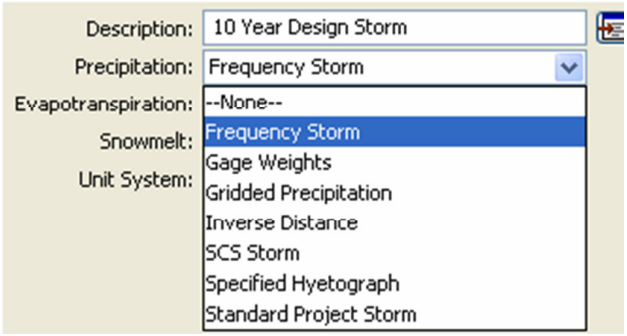
We will input the rainfall to HMS. To import an existing basin or meteorological model, one can use the menu bar by selecting File →Import (like you did for the basin model). To create a new model, we use Components menu. To create a new Meteorological Model, select Components →Meteorological Model Manager. In the Meteorological model manager, click New and enter the following information:



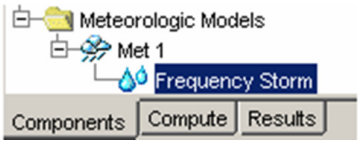
Click Create and close the Meteorological model manager. This will add Meteorological Models folder to the watershed explorer. Expand the Meteorological Models folder, and select the Met 1 model as shown below:



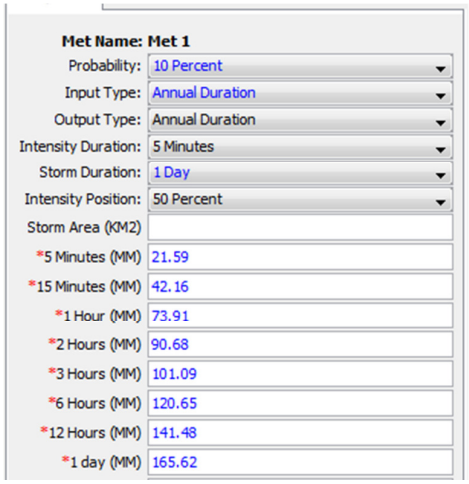
Once the Met 1 model is selected in the watershed explorer, its details will appear in the component editor. The description box includes the description you entered earlier when creating the model. By default, the precipitation data type is Specified Hyetograph (which is the most common data type). In this exercise, however, we will use design precipitation data for the site. From the drop-down menu for precipitation, choose Frequency Storm, and specify units as shown below:



Similarly select the Basins tab and specify to include sub-basins. Once we specify the precipitation type, the next step is to enter the data. Unlike Basin model, there is no extra tab in the component editor to enter the data after you choose the precipitation type. To enter the data, you need to expand the Met 1 model in the watershed explorer and then choose the precipitation type (Frequency Storm) as shown below:



This will prompt a precipitation tab in the component editor. Fill in the values shown in the site table above for a 10 year storm (10% chance of being equaled or exceeded in any year). The storm is configured by selecting the exceedance probability (10 % or 10 year return period), output type (Annual duration), duration of maximum intensity (5 mins), storm duration (one day), and percentage of storm that occurs before the peak intensity (50 %) as shown below:

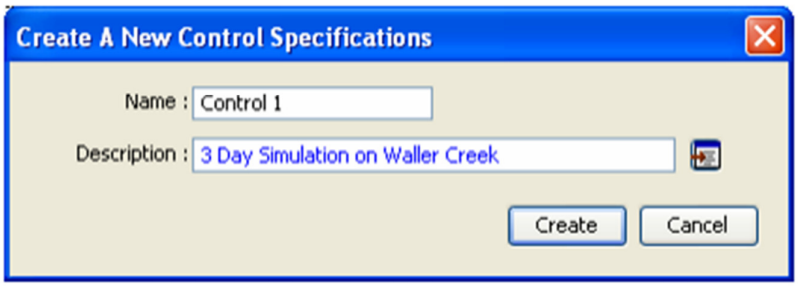


Save the project. For each project, the HMS creates an output Data Storage System DSS file which stores calculated data from all runs for a given project so that results from a previous run can be directly compared to results from a more current run.

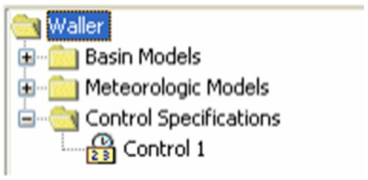
15.4.5 Defining the Control Specifications

The final task in the model setup involves establishing the model's time limits.

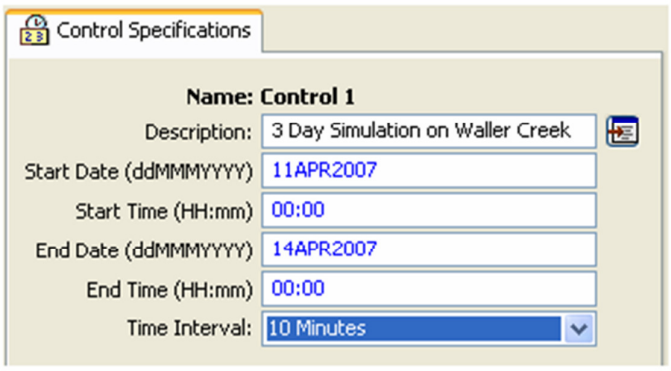
Select Components → Control Specifications Manager. Select New in the control specifications manager and type the following information:



Click Create and close the control specifications manager. This will add a Control Specifications folder in the watershed explorer. To see the control specifications file, expand the folder, and select Control 1



This will prompt the control specifications tab in the component editor. Specify the duration of the simulation in date and time, and also the time interval of the calculations (10 minutes) as shown below:

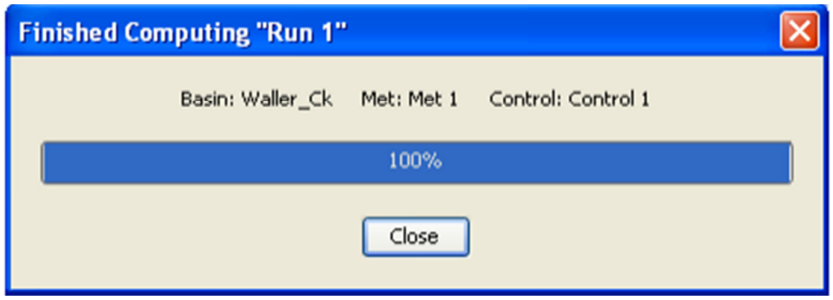


In this case, the duration is arbitrary; long enough to depict the runoff from a 1-day storm, but the 10 minute time interval is part of the Basin file model setup and should remain fixed for this stream model.

15.4.6 Executing the HMS Model

Finally, you have finished perusing the data involved in creating the stream model.

The last step is to run the model. Select Compute →Create Simulation Run. Accept the default name for the run (Run 1), click Next to complete all the steps and finally Click Finish to complete the run. Now to run the model, select Compute →Select Run →Run 1, and then go to Compute →Compute Run [Run 1] to see the following window (alternatively you can click the compute run tool  in the tool bar):



Click Close. You will see a log in the message log as program executes the model. If there are errors in the model, you will see them in red color. For this model, there are no errors.

If you want to make runs with alternative model files, you can do so by first creating/importing new model files (basin, Meteorological and control specifications), creating a new run (say Run 2) by going to Compute →Create simulation run and selecting the new files while creating Run 2. Though the model used in this exercise has one dataset each for basin, meteorology and control specifications, HMS is slick in that it allows the user to have multiple data sets available to include conveniently in different runs.

15.4.7 Viewing HMS Results

The HMS allows you to view results in tabular or graphical form. To view a global results table, select Results →Global Summary Table (alternatively you can click the Global Summary tool  in the tool bar). You will get a window like the one shown below which summarizes the peak discharge and time, the total volume of storm runoff and the drainage area from which it came.

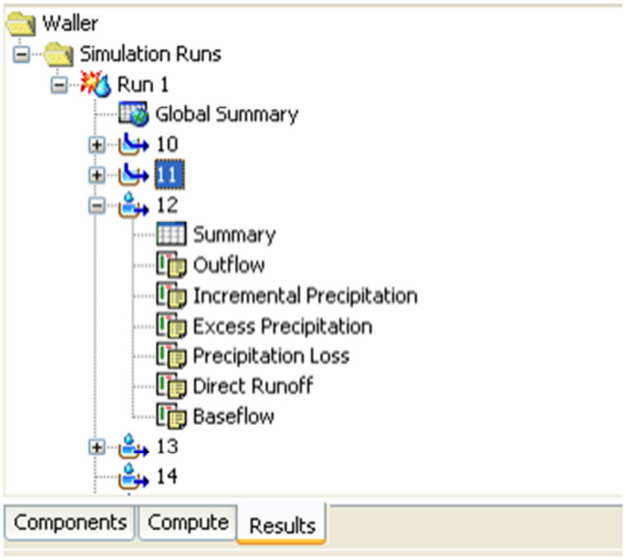
Project: Waller Simulation Run: Run 1

Start of Run: 11Apr2007, 00:00 Basin Model: Waller_Ck
End of Run: 14Apr2007, 00:00 Meteorologic Model: Met 1
Compute Time: 09Apr2007, 13:02:02 Control Specifications: Control 1

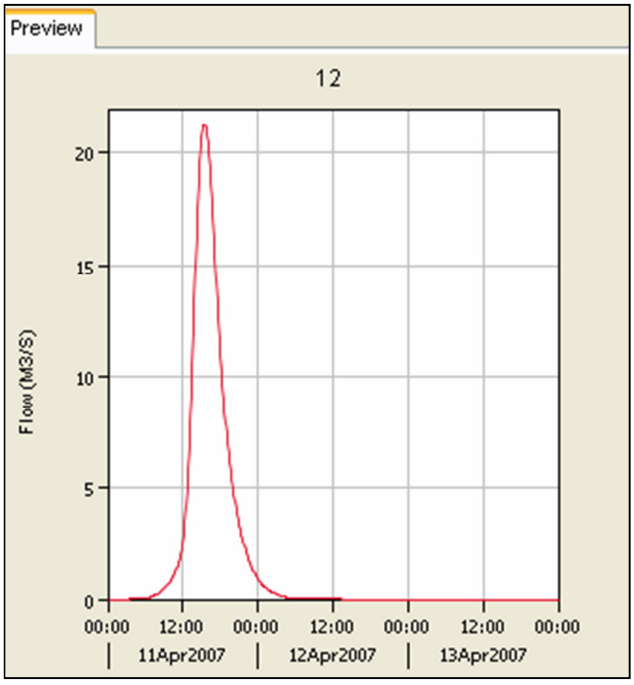
Volume Units: (•) MM ( ) 1000 M3

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
10	10.514	44.0	11Apr2007, 17:10	117.07
11	11.480	44.2	11Apr2007, 17:40	117.11
12	3.602	21.3	11Apr2007, 15:30	117.27
13	6.912	27.6	11Apr2007, 17:30	116.97
14	0.966	12.8	11Apr2007, 13:10	117.51
15	0.563	8.1	11Apr2007, 13:00	117.55
15th St	12.043	44.5	11Apr2007, 17:40	117.13
16	1.633	15.8	11Apr2007, 13:50	117.45
17	0.437	7.0	11Apr2007, 12:50	117.56
18	0.252	4.4	11Apr2007, 12:50	117.58
1st St	14.113	44.3	11Apr2007, 19:20	117.18
7	12.043	43.6	11Apr2007, 18:40	117.13
7th St	13.676	44.6	11Apr2007, 18:40	117.17
8	13.676	44.2	11Apr2007, 19:20	117.17
9	14.113	44.0	11Apr2007, 19:40	117.18
Colorado Ri...	14.365	44.0	11Apr2007, 19:40	117.19
Junction wit...	10.514	44.9	11Apr2007, 16:20	117.07
MLK	11.480	44.6	11Apr2007, 17:10	117.11

In addition to viewing global results, you may also view results for each element within the model. Again there are a couple of options to do this, and each option provides output in different ways. One option is to use the watershed explorer and component editor tab. To view results, you select the Results tab in the watershed explorer, expand the Simulation Runs folder, and expand Run 1. To see results for any element, expand that element as seen below:

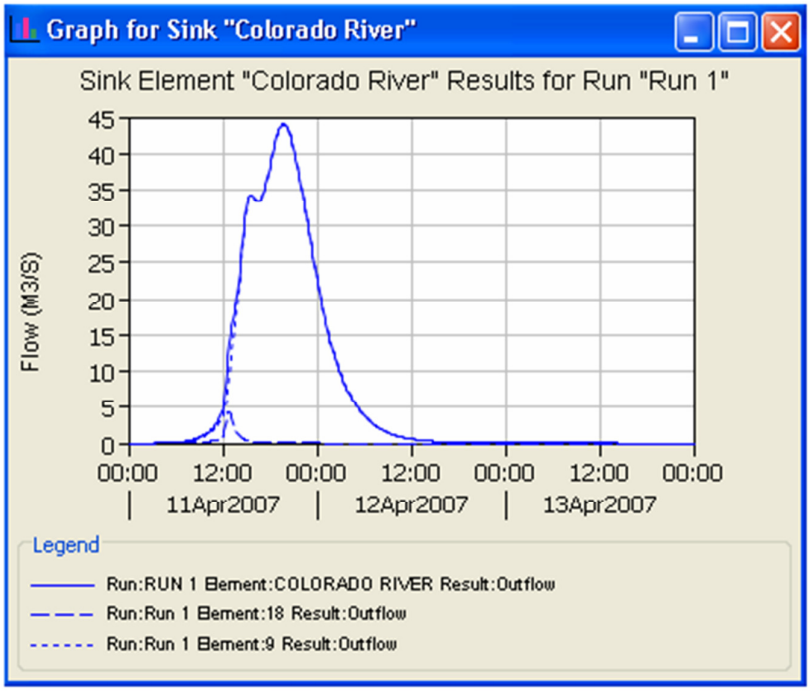


To see the outflow from sub-basin 12, you can select outflow and see the outflow hydrograph in the component editor as shown below:

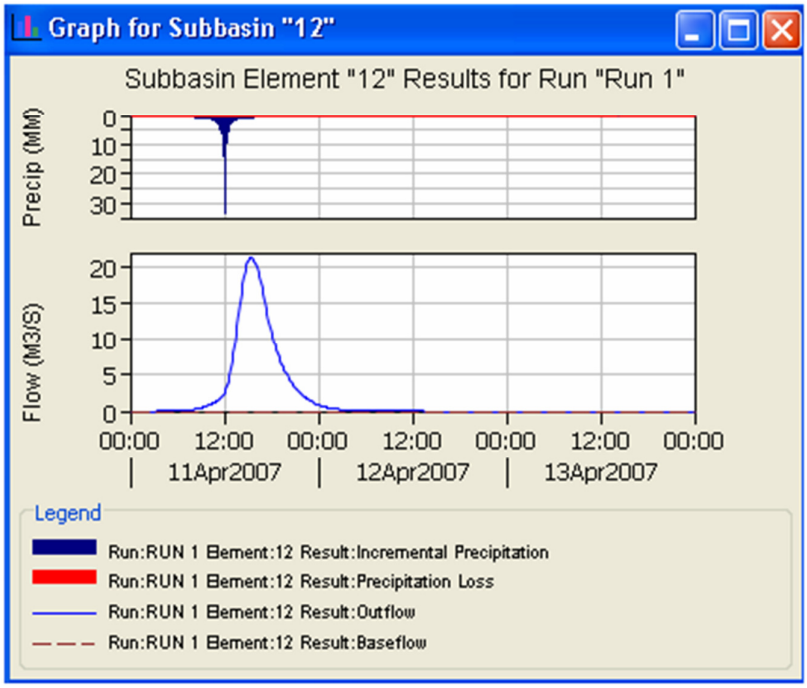


Similarly you can look at other graphs in the component editor by selecting the variable in the watershed explorer. You can select a reach element and see the attenuation in the inflow and outflow hydrograph by selecting the combined inflow and outflow option in the watershed explorer. Each element also has a summary option that gives the results from the global summary table (a single row of the table) for that particular element. Another way of looking at results is by using the tools on the tool bar, which show results in a different way than the component editor. However, to use these tools, you need to select the element by using the component tab in the watershed explorer. For example, select Example River

(Sink) in the watershed explorer, and click on the view graph tool  to get the following graph:



The dashed line hydrographs are the inflow data from the sub-basins immediately upstream of the River which is added to the routed flow in the channel to produce the total outflow curve. If you click on a sub-basin (12), you see the rainfall at the top and the runoff at the bottom as shown below:



Unlike a single graph in the component editor, you get to see all graphs (input precipitation, outflow hydrograph, baseflow, precipitation losses) in a single window using this option. You can also see the results in tabular form by using the view time series table tool  in the toolbar. These functions are also accessed through the Results menu on the menu bar.

OK, you are done with learning the basics of HEC-HMS for event based modeling

15.4.8 Hydraulic Analysis Programs

HEC-RAS

HEC is an abbreviation for Hydrological Engineering Center. It is part of the US Army Corps of Engineers. Over the years, the organization has made several computer programs for water flow problems, named HEC1, HEC2, etc. HEC2 computed the water surface profile for a steady water flow in a natural river in one dimension. The original HEC2 program does not have a user interface. It read an ASCII input file with all the necessary information about water discharge, friction factors, geometry etc. The result was an output file with the computed water levels.

Later, a graphic user interface for the program was made, with interactive input of data and visualisation of results. This program version is called HEC-RAS. The most recent version of HEC-RAS, Version 4.1 of the River Analysis System (HEC-RAS) is now available. This version supersedes version 4.0, which was released in March of 2008 to the general public, and all previous versions. Several new simulation features have been added to the program since that time. Version 4.1 of HEC-RAS includes the following new features:

1. New RAS Mapper Floodplain Delineation Capabilities;
2. Hydrologic Routing Reaches within an Unsteady Flow Model Run;
3. New Flow Data and Boundary Conditions Editor for Unsteady Flow;
4. Contraction/Expansion losses for Unsteady Flow;
5. Minor Losses for Unsteady Flow;
6. New Junction Hydraulics Option for Unsteady Flow;
7. Groundwater Leakage for Storage Areas;
8. Water Quality Modelling Enhancements;
9. Sediment Transport Modelling Enhancements; and
10. User's Manuals and Help System

For the background theory of the bridge hydraulic analysis and modelling procedures within this program, the designer is recommended to refer to the HEC-RAS user's and technical manual's which can be downloaded with the program free of charges at www. hec.usace.army.mil/

Step by step hydrologic modelling using HEC-HMS is presented in the following section. The intent of this exercise is to introduce the drainage designer to the structure and some of the functions of the HEC-RAS Modelling System.

Project File: LAB_1.PRJ

HEC-RAS input files for example problem with single triangular open channel. Input data for open channel, copy cross sections, and interpolate cross sections.. Generate program files, perform analysis and review output. Discussion of output formats available from HEC-RAS (e.g., charts, tables, figures).

Problem Statement: One river reach (River TREEO, Reach **Main**) discharges into a downstream basin outlet. The peak discharge has been calculated at 14 m^3/s, using HEC-HMS software for input into HEC-RAS. The reach is 800 feet long.

To Do: Prepare a backwater profile analysis along the stream.

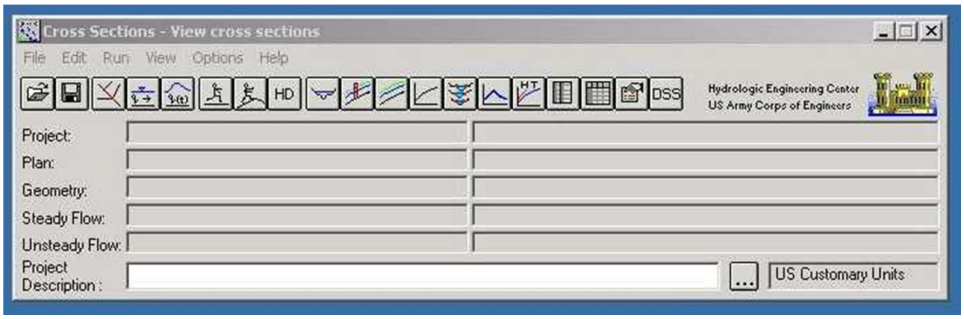
NOTE:

Projects consist of multiple files with multiple file extensions.

Extension:	**File Description**
PRJ	**Main Project File**
G**	Geometry File
F**	Steady Flow File
P**	Plan File
R**	Run File (contains results)
O**	Output File (used to create graphs and tables)

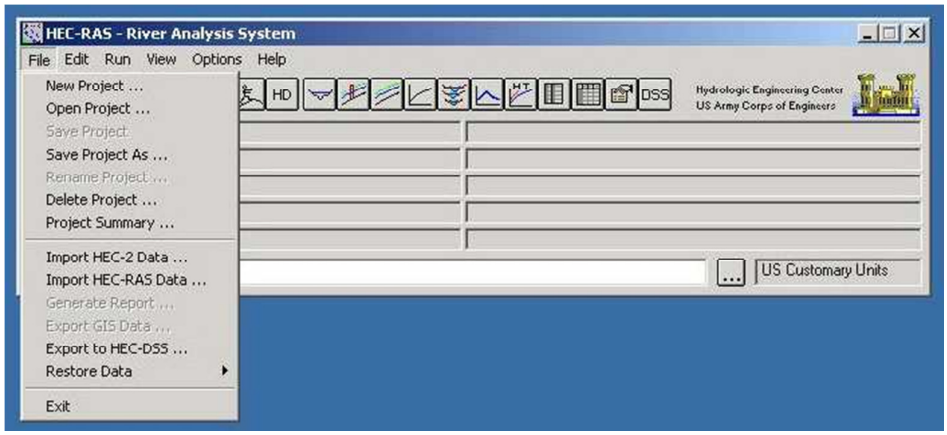
Step 1: Steps to Develop the HEC-RAS Model:

Open HEC-RAS

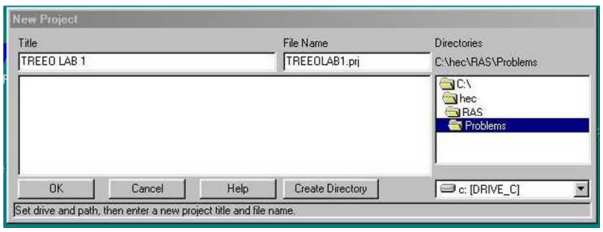


Go to **Windows Start**, **Programs**, **HEC**, and **click on the HEC-RAS** icon.

Enter Project Information



Select **File**, **New Project**



Create Directory: Problems (enter)

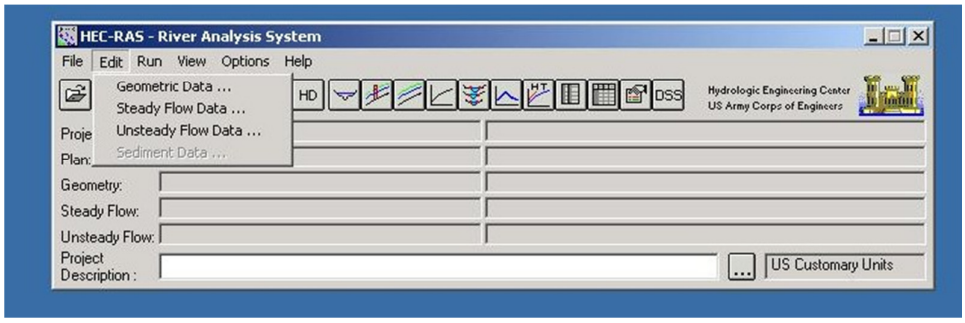
Enter Short Project ID and Project Filename (extension is added automatically)

Short Project ID: Treeo

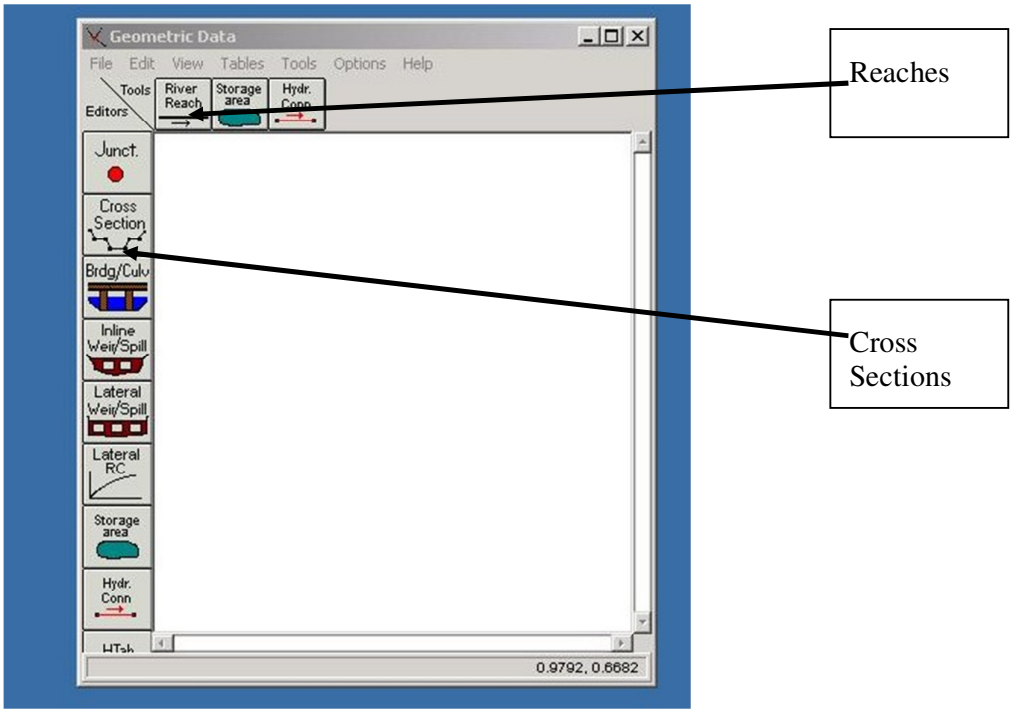
Project Filename: main (extension is added automatically)

Add Project Description: RAS Treeo (optional information)

Begin building model.



Start Geometric Data



Select **Edit**, **Geometric Data**

Add Reaches and Cross Sections

Draw Reach by Selecting the Reach Button **(Click Once)**

Move over screen to draw reach

Define Reach proceeding from upstream to downstream

With cursor in draw screen, **click mouse button once to start**

Move cursor, click mouse button once to add point, change direction

At downstream point, **double click mouse button to end**

Enter **River Name (Awash)** and **Reach Name (MAIN)**

1. Add Cross Sections to Reach

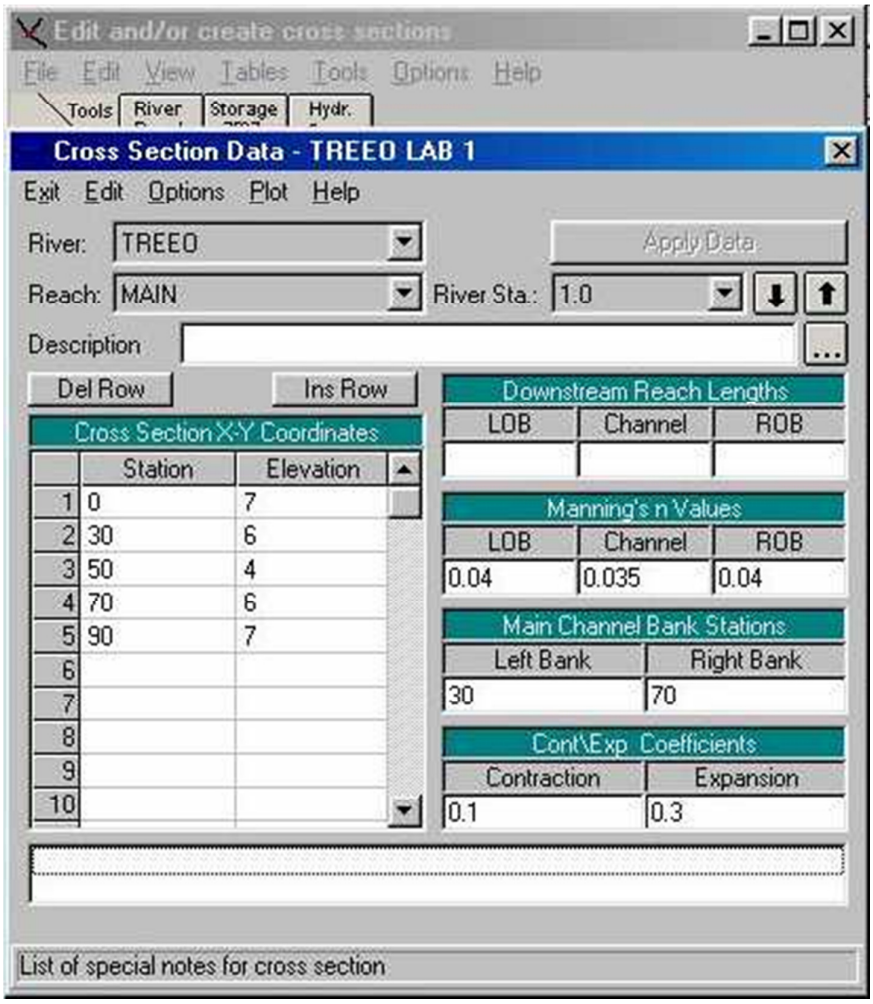
Select Cross Section Button (click once)

Select Options, Add a new Cross Section

Enter Station Location (**1.0**) and Cross Section Description (optional)

Station must be numerical value and are defined as follows:

Lower station downstream, higher stations upstream



Cross Section Data

Cross Sections typically **enter left to right looking downstream**

Cross Section **Stationing in left column**

Elevation Data in right column

Triangular Channel

Station	Elevation	
0	7	
30	6	THIS IS A TRIANGULAR
50	4	CHANNEL, 10:1 SIDE SLOPE
70	6	
90	7	

Enter downstream reach lengths for main channel and overbanks

No downstream reach lengths for most downstream section

Furthest downstream section defined by boundary conditions

Add Manning's n value for main channel and overbanks

.04 .035 .04

Outer Banks are outside Main Channel.

Must correspond to a cross section station.

Left Main Channel Bank at 30 Right Main Channel Bank at 70

Repeat for additional Cross Sections

Use Add, Copy, Interpolate

Copy - Define New Cross Section Name: **9.0**

Adjust Elevation: +4 feet (1/2 percent slope)

Revise Downstream Reach Lengths: 800 feet

Exit Cross Section Data Editor

Interpolate - On main geometry screen

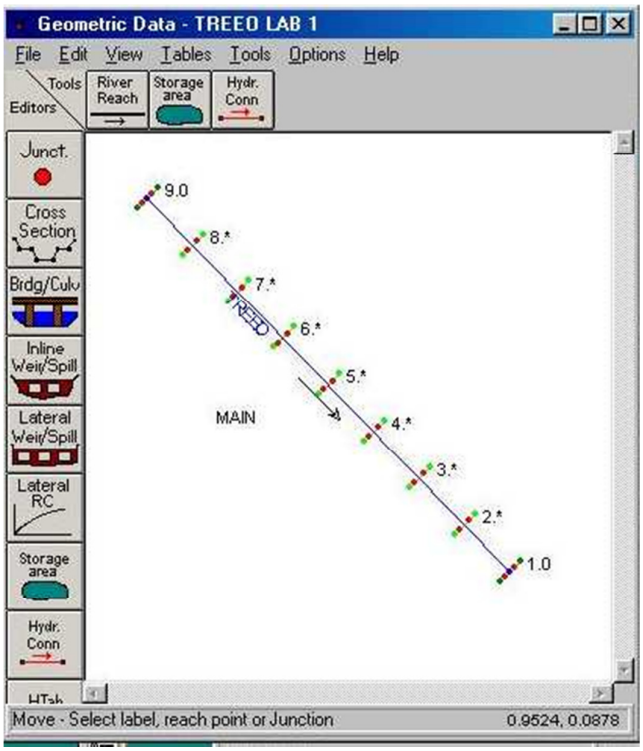
Tools, XS Interpolation, Within a Reach

Maximum distance: 1000 feet

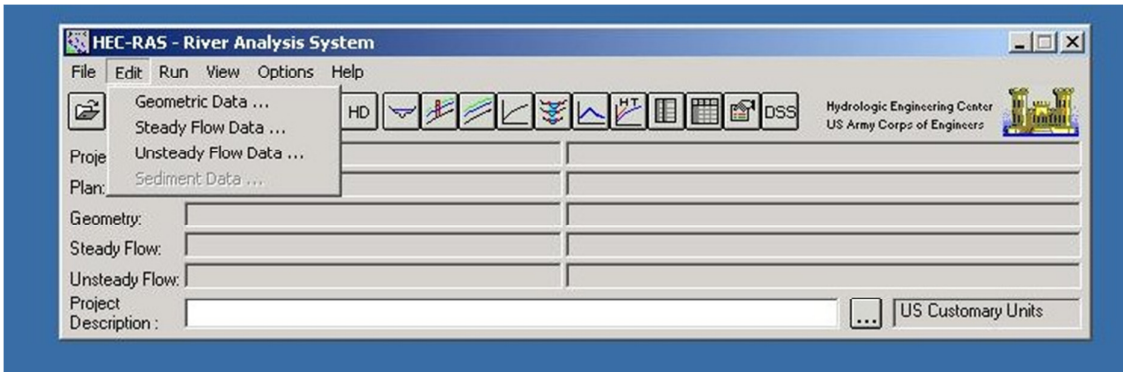
Interpolate XSs, Close

Interpolated Cross Sections have "*" by station name

Interpolated Cross Sections can be made permanent by editing the name in the cross section editor and removing the asterisk



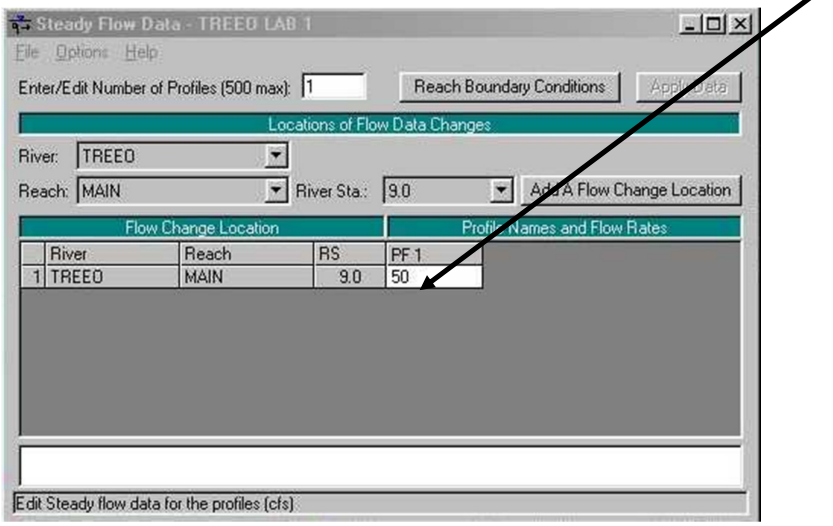
SAVE GEOMETRIC DATA, file name automatically assigned



Start Steady Flow Data

Select Edit, **Steady Flow Data**

Enter the Flow for the River TREEO and Reach Main: 50 cfs

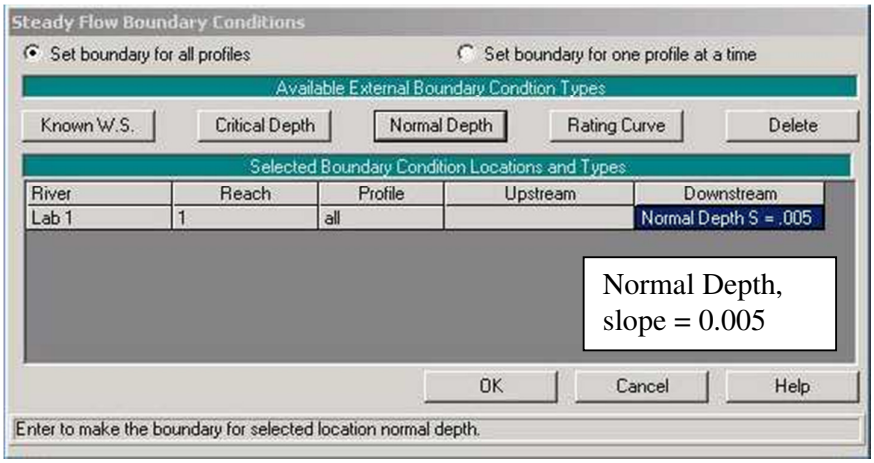


(This is the flow at the upstream cross section of each)

Enter Boundary Conditions

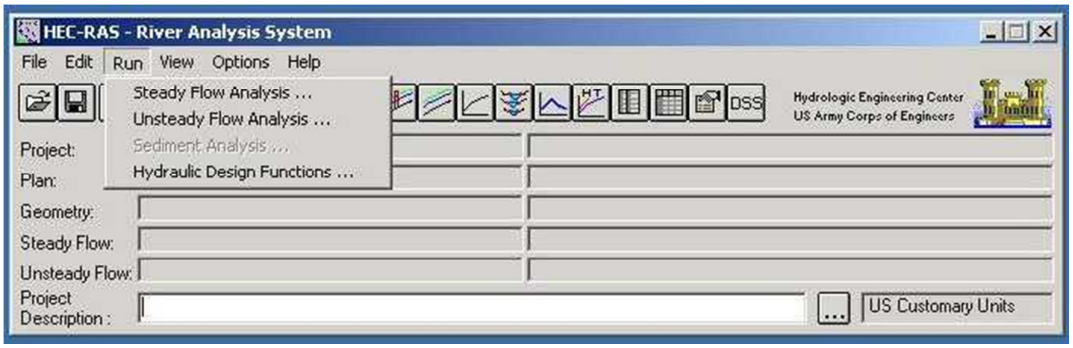
Most Downstream Channel

- use Known Water Surface Elevation
- Critical Depth, determined by size of channel
- **Normal Depth defined by channel shape, slope, and flow**
- Rating Curve (user defined)
- HEC-RAS will normally set these automatically



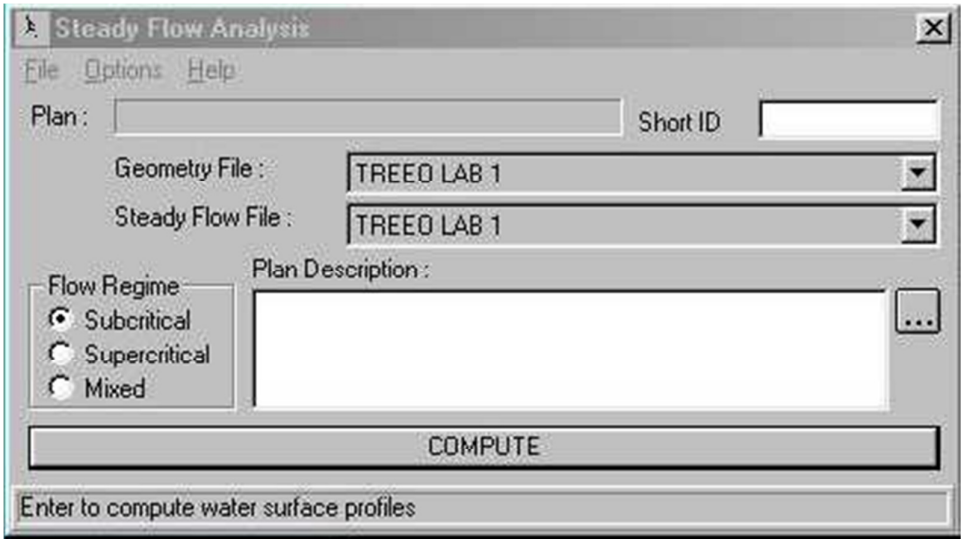
SAVE STEADY FLOW DATA, file name automatically assigned

Simulate Backwater Analysis



Select **Run**, **Simulate**

Select Geometry File: TREEO LAB 1



Select Steady Flow File

Save Steady Flow Analysis File

Description: TREEO LAB 1 **Short Form: LAB 1**

COMPUTE

Options within this segment allow for error checking, precision, etc.

See FINISHED COMPUTATION screen for errors and warnings.

Close Window

EXIT STEADY FLOW ANALYSIS

View Results

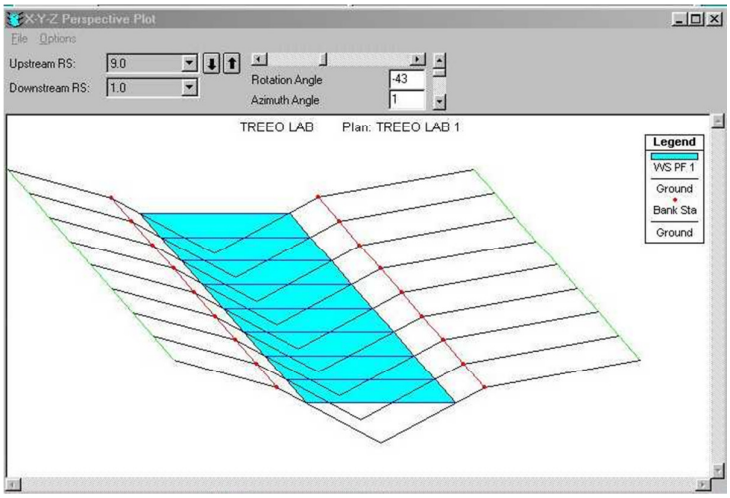
View Graphics

Cross Sections

Profiles

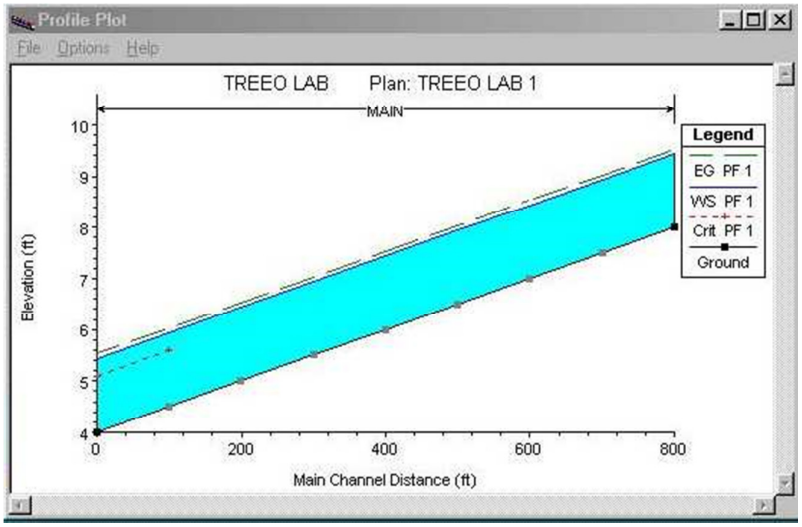
Rating Curves (multiple profiles)

X-Y-Z Perspective (3-D)



View, X-Y-Z Perspective Plot

View, Water Surface Profile



print using File, Print from screen or copy to Windows clipboard

View Tables

Cross Sections/Culverts/Bridges

Profile Table



E.G. Elev (ft)	6.03	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.09	Wt. n-Val.		0.035	
W.S. Elev (ft)	5.94	Reach Len. (ft)	100.00	100.00	100.00
Crit W.S. (ft)	5.59	Flow Area (sq ft)		20.79	
E.G. Slope (ft/ft)	0.004997	Area (sq ft)		20.79	
Q Total (cfs)	50.00	Flow (cfs)		50.00	
Top Width (ft)	28.84	Top Width (ft)		28.84	
Vel Total (ft/s)	2.40	Avg. Vel. (ft/s)		2.40	
Max Chl Dpth (ft)	1.44	Hydr. Depth (ft)		0.72	
Conv. Total (cfs)	707.3	Conv. (cfs)		707.3	
Length Wtd. (ft)	100.00	Wetted Per. (ft)		28.98	
Min Ch El (ft)	4.50	Shear (lb/sq ft)		0.22	
Alpha	1.00	Stream Power (lb/ft s)		0.54	
Frctn Loss (ft)	0.50	Cum Volume (acre-ft)		0.05	
C & E Loss (ft)	0.00	Cum SA (acres)		0.07	

Summary Errors/Warnings

print using File, Print from screen or copy to Windows clipboard

Cross Section 2.0: Water Surface Elevation = 5.94 feet

Generate Report - File, Generate - Select Filename

SAVE PROJECT, file name assigned in step 1.

HY-8

HY-8 7.3 builds on previous versions of the software and incorporates improvements from user feedback. HY-8 7.3 expands the capabilities to analyze special hydraulic

situations and culvert shapes and includes an upgrade to the source code to make the program run more smoothly and efficiently.

HY-8 is a 32-bit program, but it is fully compatible with 32 and 64-bit Windows-based operating systems (XP, Vista, and Windows 7) and runs in 32-bit mode on these operating systems. When installing HY-8, make sure you have permissions on your computer to install programs (usually this requires administrator permissions or a username and password for an administrator account). If you do not have administrator permissions on your account, you need to right-click on the installation program and select the "Run as Administrator" option if running Windows Vista or Windows 7. Windows XP users should login to an account that allows you to install software before installing HY-8.

ISIS FREE

ISIS is developed by Halcrow Ltd (now part of HM2 Hill) in the UK. It is similar in functionality to HEC-RAS and Mike 11, with a graphical user interface for interactive data entry and result visualisation and computational modules. ISIS also has a number of add-on-modules including 2-dimenstional program. However, the free version of ISIS is limited to 250 model nodes and 2500 2-dimensional grid cells. This is sufficient for bridge hydraulic analyses in Ethiopia. The latest version of ISIS is 4.1 released in August 2012.

Simulation of Hydraulic Structures

A key strength of ISIS is the ability to model a wide range of hydraulic structures including all common types of bridges, sluices, culverts, pumps and weirs. Wherever possible, standard equations or methods are incorporated into the software so that the calculation of level and discharge relationships is fully handled by the software. For structures with automatic operation such as pumps and sluices ISIS allows the user to incorporate logical control rules.

Advanced Backwater Solutions

A backwater solution is all that is needed for many river engineering applications. ISIS incorporates a powerful steady state solver, providing engineers with an easy to use tool for day-to-day application in engineering design. Although easy to apply to simple systems, the steady state solver includes the full range of hydraulic structures and is able to model complex looped channels and transcritical flows in steep rivers.. The modular design of ISIS allows any model constructed for backwater modelling to be used for unsteady simulation if an unsteady solution is subsequently required.

Rainfall-Runoff Modelling

An event-based hydrological module is available. The US Soil Conservation Service (US SCS) method is available for rainfall runoff modelling. The software allows the user to specify observed or predicted rainfall profiles and unit-hydrographs.

Mike 11

The Danish Hydraulic Institute develops Mike 11. It is a one-dimensional program with both steady water surface profile computation and solution of the full Saint – Venant Equations. The program has a user interface, and program includes connections with GIS systems. Mike 11 has a number of add-on- modules, computing for example:

- Rainfall/runoff;
- Water Quality;

- Sediment transport; and
- Groundwater analysis.

The various modules makes Mike 11 very well suited for solving a number of different hydraulic problems.

Scour Model

Sediment transport computer models can be used to determine long-term aggradation or degradation trends. These computer models route sediment down a channel and adjust the channel geometry to reflect imbalances in sediment supply and transport capacity. The USACE HEC-RAS (USACE 2010a) model is an example of a sediment transport model that can be used for single event or long-term estimates of changes in bed elevation. The information needed to run these models includes:

- Channel and floodplain geometry
- Structure geometry
- Roughness
- Geologic or structural vertical controls
- Downstream water surface relationship
- Event or long-term inflow hydrographs
- Tributary inflow hydrographs
- Bed material gradations
- Upstream sediment supply
- Tributary sediment supply
- Selection of appropriate sediment transport relationship
- Depth of alluvium

In sediment routing, the sediment transport capacity is used to update cross section geometry, which is then used to update the hydraulic calculations. The geometry is updated for individual cross sections, though the hydraulic variables can be weighted with up- and downstream cross sections. A flood hydrograph or long-term flow hydrograph is entered as a series of constant flows. Within each flow time step, many sediment transport and cross section updating time steps are often required. The model does not assume that transport capacity is reached at every cross section, but limits erosion based on potential entrainment rates and limits deposition based on fall velocity, flow velocity and water depth. Sediment layer depths, as well as lateral limits for erosion and deposition are also input. Sediment transport modelling generally requires greater model upstream and downstream extent than a hydraulic flow model, as well as careful consideration of all boundary conditions (hydraulic and sediment).

15.4.9 Computer Program for Storm Drainage

Storm drain design by computer analysis offers some distinct advantages over calculations performed by hand. Chief among these advantages is the ability to perform complex flow routings through the system, which can take into account the detention due to storage within the pipes and structures and overlapping of various basin runoff hydrographs. This analysis develops a more accurate picture of the hydraulic grade lines and flow conditions within a storm drain system for the design storm peak conditions.

Most computer programs will have the ability to process various runoff hydrograph calculation methods that use the Rational Method or NRCS curve numbers, as well as the

ability to enter storm distributions and rainfall intensity data unique to a specific location. They also will allow selection of different routing methods. Most of these programs will have sub-routines for designing detention within a system or accepting the input of additional detention storage and discharge controls. The designer should develop a full understanding of the various runoff and routing methodologies and in particular, select those that are best for the application when using these programs. There are several commercially-available computer programs for storm drain design. Xpstorm, FlowMaster, HydroCAD and MicroDrainage are some of the software commercially available.