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Road Design Manual

Volume 2: Hydrology and Drainage Design

Part 2: Drainage Design

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Foreword

This manual was developed by the Ministry pursuant to The Fourth Schedule of the Constitution which assigns to the National Government the functions and powers of setting standards for the construction and maintenance of all public roads including those under the County Governments.

It is part of a series of manuals that replace the first generation of road manuals developed in the first and second decades after independence. This second generation of the road manuals were developed to cover the entire road project cycle covering planning, appraisal, design, contracts, construction, maintenance, operations and monitoring. The series incorporates best practices, climate change considerations, and recent technologies to enable the provision of road infrastructure that is safe, secure, and efficient.

Under the Kenya Vision 2030 long term plan, infrastructure expansion and modernisation are some of the foundations for the realisation of economic, social and political transformation of Kenya into a rapidly industrialising middle-income country. The plan envisages an integrated, safe and efficient transport and communication infrastructure network consisting of roads, railways, ports, airports, waterways, and telecommunications infrastructure.

The strategies to be pursued under the Vision 2030 plan to improve infrastructure services and to maximise the economic and social impacts of infrastructure development and management include: Strengthening of the institutional framework for infrastructure development and maintenance; Raising efficiency and quality of infrastructure projects; Enhancing local content of identified infrastructure projects to minimise import content; Benchmarking infrastructure facilities and services provision with globally acceptable performance standards; and, Implementing infrastructure projects that will stimulate demand in hitherto marginalised areas.

The first three 5-year Medium Term Plans (MTP) under the Vision 2030 from 2008 to 2022 targeted construction of 1,950 km, 5,500 km and 10,000 km of new paved roads under MTP I, II and III, respectively, totalling 17,450 km. This was a massive infrastructure development program intended to double the paved road network in 10 years compared to 8,600 km developed from independence in 1963 to 2008.

Implementation of MTP I to III resulted in the construction of 14,000 km of paved roads, which extended the paved road coverage to Arid and Semi-Arid regions, that had been previously neglected. However, some key milestones of the Vision 2030 goals have not been realised. This has been due to internal and external challenges. External challenges included: climate change – prolonged droughts; the emergence of COVID-19 pandemic; global supply chain disruptions; exchange rate volatility; and, rising interest rates in the leading economies.

The internal challenges included: inadequate road maintenance equipment; pavement overloading by heavy goods vehicles; huge maintenance backlog of the road network; low contracting and supervision capacity particularly in the Counties; poor quality control and assurance of works; congestion in urban areas; encroachment on road reserves; high costs and delays in payments of land acquisition; lack of harmonisation of cross-border transport regulation and operational procedures; rapid urbanisation; increased traffic volume with exponential growth of motorcycle traffic; high cost/delays in relocation of utilities and services along and across road reserves; inadequate funding of projects and programs; and, delay or default in payments for goods, services and works.

The implementation of MTP III came to an end on 30th June 2023, ushering in the implementation of the Fourth Medium Term Plan (MTP IV), which has been aligned to the aspirations of the Kenya Vision 2030 and the Kenya Kwanza Government's Bottom-Up Economic Transformation Agenda (BETA) planning approach and its key priorities.

BETA is the Government's transformation agenda geared towards economic turnaround through a value chain approach. BETA has targeted sectors with the highest impact to drive economic recovery and growth. This will be achieved through bringing down the cost of living; eradicating hunger; creating jobs; expanding the tax base; improving foreign exchange balances; and inclusive growth. BETA ensures rational resource allocation by eliminating wastage of resources occasioned by duplication, overlaps, fragmentation and ineffective coordination in the implementation of programmes and projects.

The Fourth Medium Term Plan key priorities are clustered under five key sectors, namely: Finance and Production; Infrastructure; Social; Environment and Natural Resources; and Governance and Public Administration. The infrastructure sector seeks to: enhance transport connectivity by constructing 6,000 km of new roads, maintaining rural and urban roads, rail, air and seaport facilities and services; expand communication and broadcasting systems; and promote the development of energy generation and distribution by increasing investments in green energy (geothermal, wind, solar and hydro). The infrastructure gap is expected to be bridged by promoting economic participation of the private sector through public private partnerships in the financing, construction, development, operation and maintenance of infrastructure

The plan entails a shift of focus to fundamentals in project planning and implementation which include: respect for technical input, regulations and standard practices; adherence to project life cycle i.e., planning, feasibility studies and design before procurement of works; public and stakeholder consultation; procurement within budgetary ceilings; shifting focus during project implementation from the finished product ‘black top’ to the construction of the foundation; building local capacity particularly MSMEs by ensuring prompt payments; and capacity building at all levels to enable internalisation of policies and processes.

The first generation of the road manuals were used for 35 to 45 years. It is my sincere hope that the second generation of the road standards which have been developed in alignment with the Government’s strategy will provide guidance in solving most of the above challenges and those expected to emerge in the next 50 years. Implementation of the manuals will enable achievement of the Government aspirations which include: inclusive growth; creation of sustainable employment; building of MSMEs; climate change adaptation and realisation of the UN SDGs; enhanced efficiency in management of infrastructure and transport system; and, laying the foundation for the next national long-term plan at the end of the Vision 2030.

On behalf of the Government of Kenya, I would wish to thank the European Union for financing the development of the first drafts of the manuals in 2009 and the African Development Bank for the financial support in the review and updating of the manuals. I would also like to thank the members of the National Steering Committee and the Technical Task Force for their input. The Technical Administrators, and the Kenya National Highways Authority (KeNHA) for the procurement and able administration of the consultancy Contract. I also thank the Consultant, TRL Limited for their role in providing technical expertise that was essential for the success of the manuals updating exercise. I also wish express my deepest appreciation to our stakeholders and all those who have contributed to this process and the staff of the Ministry for their continued input.

Hon. Davis K. Chirchir, E.G.H
Cabinet Secretary, Ministry of Roads and Transport

Preface

This Manual covers drainage design for rural and urban roads. It is tailor-made for the road sector in Kenya though it can be used in other countries in the region and elsewhere where similar conditions apply. The Manual should be read in conjunction with the Standards Details for Roads and Bridges (Standards Drawings).

In the past context of road design in Kenya the aspects of hydrology, hydraulics and drainage design were treated as cross cutting issues. While the design and the hydraulic dimensioning of side drains was considered part of the RDM Part 1- Geometric Design of Rural Roads, the aspects of pavement drainage and erosion control were dealt with in RDM Part III- Materials and Pavement Design of New Roads. Basic hydrological and hydraulic calculations on catchment runoff and culvert and bridge hydraulics were discussed in the RDM Part IV Bridge Design, Draft Version. In addition, the Road Design Guidelines for Urban Roads (Draft Version) covered certain aspects of urban road drainage. An attempt was made in 2009 draft to merge these aspects into a single manual that covered hydrological and drainage design. But the scope was limited and many design aspects were not adequately covered.

This Manual addresses the missing gaps in the previous manuals. It provides detailed information on planning and hydraulic design of road and urban structures that are based on current best practices. The scope is extensive in the coverage of hydraulics design that is related to road drainage. References are provided in this Manual and include other internationally recognised documents and manuals.

The Manual is targeted at road design practitioners, consultants and contractors, academia, the laboratories, and other users involved in the provision of roads in Kenya and beyond.

While this Manual provides adequate guidance on the design processes, standards and specifications for drainage design, users should use their knowledge and experience to apply the Manual to their particular situations, which may be unique in many respects. Users are encouraged to contribute to future editions noting any necessary improvements through feedback from practice.

Eng. Joseph M. Mbugua, CBS
Principal Secretary, State Department for Roads

Document Management

Document Status

This document has the status of a Manual. Users shall apply the contents there-in to fully satisfy the requirements set out. The content of the manual is based on current practice in Kenya and latest practices in the road sector, both regionally and internationally.

Sources of the Document

Copies of the document can be obtained from:
The Principal Secretary, State Department for Roads, Ministry of Roads and Transport, Works Building, Ngong Road, P.O. Box 30260 - 00100, NAIROBI Email: ps@road.go.ke

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Notification of Errors and Requests for Amendments

While all care and consideration has been applied in the compilation of this document, the Ministry accepts no responsibility for failure in any way related to the application of this manual or any reference documents cited in it.

Requests for edits and corrections can be freely sent to the following address:
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Amendments Request Form

Request No.	Name	Organisation	Chapter	Page	Section/ Clause	Ref. to: Figure/ Table/	Type of Request	Request

Type of request: General – G; Editorial – E; Technical - T

Amendments to Date

Amendment No.	Description	Amendment Effective Date	Amended Approved by

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The technical work was undertaken under the guidance of a Technical Task Force, chaired by Eng. David Maganda, with the following gazetted members: Francis Gichaga (Prof.) (Eng.), Andrew Gitonga (Eng.), Timothy Nyomboi (Dr.) (Eng.), Rosemary Kungu (Eng.), Charles Obuon (Eng.), Sylvester Abuodha (Prof.) (Eng.), Samuel Kathindai (Eng.), Nicholas Musuni (Eng.), Charles Muriuki (Eng.), Tom Opiyo (Eng.), John Maina (Eng.), Fidelis Sakwa (Eng.), Daniel Cherono (Eng.), Maurice Ndeda (Eng.), Theo Uwamba (Eng.).

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Abbreviations

AASHTO	American Association of State Highway Transport Officials.
CBR	California Bearing Ratio.
DFL	Design flood level.
HFL	High flood level.
HEC - RAS	Hydrologic Engineering Centre - River Analysis System.
IDF	Intensity duration frequency.
LWL	Low water level.
SCS	Soil Conservation Service.

Symbols

a	Pier width.
A	Catchment area.
b	Pier width perpendicular to flow direction.
B	Barrel width.
C_d	Coefficient of drag.
D	Diameter.
D_r	Relative density of soil.
D_{50}	Median diameter of the bed material, diameter which 50% of the sizes are smaller.
D_{84}	Diameter of the bed material of which 84% are smaller.
D_{90}	Diameter of the bed material of which 90% are smaller.
d	Depth of flow.
e	Void ratio of soil.
E_o	Ratio of frontal flow to total gutter flow Q_w/Q
Fr	Froude Number $V/(gy)$½
g	Acceleration of gravity.
H	Height (thickness)
H_b	Distance from the low chord of the bridge to the average elevation of the stream bed before scour.
H_e	Entrance head loss in pipe flow.
H_f	Friction head loss.
H_b	Bend head loss.
H_L	Friction and form loss.
H_O	Hydraulic grade line height above outlet invert.
H_v	Velocity head loss.
h	Sum of $H_e + H_f + H_o$
$h_{1\text{-}2}$	Head loss between sections 1 and 2.
I	Rainfall intensity.
J_s	Relative orientation parameter.
K	Various coefficients in equations as described in the main text.
	Conveyance in Manning equation $(1/n)AR^{2/3}$.
L	Length.

M	Mass.
n	Manning's coefficient n.
P	Instantaneous stream power.
	Perimeter.
Q	Discharge or flow rate.
Q_{100}	Storm-event having a probability of occurrence of one every 100 years.
Q_{500}	Storm-event having a probability of occurrence of one every 500 years.
q	Discharge per unit width.
R	Hydraulic radius.
S or Y	Spacing between columns of piles, pile centre to pile centre.
	Stopping distance for debris mass.
	Slope.
S or S_x	Crown, Cross or Longitudinal Slope of pavement.
S_W	Depression section slope
SBR	Set-back ratio of each abutment
S_s	Specific gravity of bed material. For most bed material this is equal to 2.65
T	Top width of water surface (spread on pavement)
t_c	Time of concentration.
T_s	Spread above depressed section.
t	Time from the beginning of the total cycle.
	Duration of flow.
	Thickness.
V	Velocity.
W	Width.
y	Depth of flow in approach gutter.
	Depth of flow.
Z	Vertical offset to datum.
z	T/d, reciprocal of the cross slope.
α_A	Amplification factor for abutment scour.
σ	Sediment gradation coefficient (D_{84}/D_{50}).
τ	Design shear stress.
γ, γ_w	Specific weight of water.
ρ_w	Density of water.
ρ_s	Density of sediment.
Θ	Angle of repose of the bed material (ranges from about 30 to 44).
ω	Fall velocity of the bed material of a given size.

Glossary of Terms

Abrasion — Removal of stream bank material due to entrained sediment, ice, or debris rubbing against the bank.

Abutment — The support at either end of a bridge, usually classified as spill-through or vertical.

Accretion — A process of accumulation by flowing water whether of silt, sand, or pebbles. May be due to any cause and includes alluviation, 2. The gradual building up of a beach by wave action, 3. The 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, floodplain, 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.

Armor — Artificial surfacing of channel beds, banks, or embankment slopes to resist scour and lateral erosion.

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.

Aquifer — A porous, water-bearing geologic formation. Generally restricted to materials capable of yielding an appreciable supply of water.

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.

Artesian — Pertains to groundwater that is under pressure and will rise to a higher elevation if given an opportunity to do so.

Avulsion — A sudden change in the course of a channel, usually by breaching of the banks during a flood.

Backwater — The increase in water-surface profile, relative to the elevation occurring under natural channel and flood-plain conditions, is 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.

Glossary of Terms *(continued)*

Bank	The particle diameter at the 85th percentile point on a size weight distribution curve.
Bar	An elongated deposit of alluvium, not permanently vegetated, within or along the side of a channel.
Base Flood	The 100-year flood.
Basin, Drainage	The area of land drained by a watercourse.
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.
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 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 carry-over.
Capacity	A measure of the ability of a channel or conduit to convey water.
Catch Basin	A structure with a sump for in-letting 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 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 man-made 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.

Glossary of Terms *(continued)*

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 (flood)	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.
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 (kerb) 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/ Contraction Scour	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 floodplain 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 $K = (1/n)\, A\, R^{2/3}$ (SI units).
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 judgement is based.
Critical Depth	The depth at which water flows over a weir; this depth being obtained automatically where no backwater forces are involved. It is the depth at which the energy content of flow is a minimum.
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.
Curb/Kerb-Opening Inlet	Drainage inlet consisting of an opening in the roadway kerb.
Cumulative Conveyance	A tabulation or graphical plot of the accumulated measures of conveyance; proceeding from one stream bank to the other.
Cut-off Wall	A wall that extends from the end of a structure to below the expected scour depth or scour-resistant material.
D50	Median size of riprap. The particle diameter at the 50th percentile point on a size weight distribution curve.

Glossary of Terms *(continued)*

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) is used as a basis for the design discharge.
Design High Water 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 localised at a structure or may be along an alternate route.
Dike (Dyke)	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.
Drop Inlet	Drainage inlet with a horizontal or nearly horizontal or nearly horizontal opening.
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.
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.

Glossary of Terms *(continued)*

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.
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.
Evapotranspiration	Plant transpiration plus evaporation from the soil. Difficult to determine separately, therefore used as a unit for study.
Filter	A material which allows water to pass into and through it while preventing soil particles from entering or passing through it.
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.
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 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, that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.
Flow Concentration	A preponderance of the stream flow.
Flow Distribution	The estimated or measured spatial distribution of the total streamflow.
Flume	An open or closed channel used to convey water.
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.

Glossary of Terms *(continued)*

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.
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 localised 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	A drainage inlet composed of a grate in the carriageway section or at the roadside in a low point, swale or channel.
Groyne	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.
	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 kerb that is used to convey storm runoff water. It may include a portion, or all, of a travelled lane, shoulder or parking lane, and a limited width, adjacent to the kerb.
Head	The height of water above any datum.
Head loss	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, Hw	That depth of water impounded upstream of a culvert due to the influence of the culvert constriction, friction, and configuration.
High Water Elevation	The water surface elevation that results from the passage of flow. It may be 'observed high water elevation' as a result of an event, or 'calculated high water elevation' as part of a design process.
Historical flood	A past flood event of known or estimated magnitude.
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. The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).

Glossary of Terms *(continued)*

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 the cross-sectional area of flow divided by the wetted perimeter. For wide shallow flow, the depth hydraulic radius can be approximated by the average.
Hydraulic Roughness	A composite of the physical characteristics that influence the flow of water across the earth's surface whether natural or channelised, 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.
Hydrology	The study of the occurrence, circulation, distribution, and properties of the waters of the earth and its atmosphere.
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 centimetres per hour or day, or cubic meters per second.
Inflow	The rate of discharge arriving at a point (in a stream, structure, or reservoir).
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 centimetres 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.
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.

Glossary of Terms *(continued)*

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 (TL = 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 floodplain that is localised at a pier, abutment, or other obstruction to flow. The scour is caused by the acceleration of the flow and the flow development of a vortex system induced by the obstruction to the flow.
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 litter 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).
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.

Glossary of Terms *(continued)*

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.
Precipitation	The process by which water in liquid or solid-state falls from the atmosphere.
Pressure Head	Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.
Principal Spillway	Conveys all ordinary discharges coming into a reservoir and all of an extreme discharge that does not pass through the emergency spillway
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 centimetres 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.
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.
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 super-elevation 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 kerb or barrier is also called a scupper.
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.

Glossary of Terms *(continued)*

Skew	A measure of the angle of intersection between a line normal to the roadway centreline 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 Inlet	A drainage inlet composed of a continuous slot build into the top of a pipe which serves to intercept, collect, and transport the flow.
Splash-Over	Portion of the frontal flow at a grate which skips or splashes over the grate and is not intercepted.
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 evapo-transpiration takes place.
Spread	The width of flow in the gutter measured laterally from the carriageway kerb.
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 dissipator placed at the outlet of a structure.
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.
Storm Drain	A storm drain is that portion of the storm drainage facilities that receives runoff from the inlets and conveys the runoff to an adequate outfall. Culverts discharging to the storm drainage system are considered part of the system.
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.

Glossary of Terms *(continued)*

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.
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.
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
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.
Uniform Flow	Flow of constant cross-section and average velocity through a reach of channel during an interval of time.
Unsteady Flow	Flow of variable cross-section and average velocity through a reach of channel during an interval of time.
Velocity Head	Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water.
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 , 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.

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1 Introduction

1.1 General

This Manual was prepared by the Ministry as part of a series of manuals that cover the entire project cycle. The series incorporate best practices, climate change considerations, and recent technologies thereby enabling the provision of road infrastructure that is safe, secure, and efficient.

The roads manual series is as follows:

Project Cycle Stage	Manual: Volume or Part/Chapter	Code
A. General	**Procedures and Standards Manual**	**PSM**
	1. General	
	2. Policies	
	3. Procedures Guidance	
	4. Codes of Practice	
	5. Guidelines	
	6. Product/Testing Standards	
B. Planning	**Network and Project Planning Manual**	**NPM**
	1. Road Classification	
	2. Route/Corridor Planning	
	3. Route/Corridor Planning	
	4. Highway Capacity	
	5. Project Planning	
C. Appraisal	**Project Appraisal Manual**	**PAM**
	1. Environmental Impact Assessment and Audit	
	2. Social Impact Assessment	
	3. Traffic Impact Assessment	
	4. Road Safety Audits	
	5. Project Appraisal	
	6. Feasibility Studies	
D. Design	**Road Design Manual**	**RDM**
	1. Geometric Design	
	2. Hydrology and Drainage Design	
	3. Materials and Pavement Design for New Roads	
	4. Bridges and Retaining Structures Design	
	5. Pavement Maintenance, Rehabilitation and Overlay Design	
	6. Traffic Control Facilities and Communication Systems Design	
	7. Road Lighting Design	
E. Contracts	**Works and Services Contracts Manual**	**WSCM**
	1. Forms of contracts	
	2. Standard Specification for Road and Bridge Construction	
	3. Bills of Quantities	
	4. Standard/Typical Drawings	
F. Construction	**Road Construction Manual**	**RCM**
	1. Construction Management	
	2. Project Management	
	3. Site Supervision	
	4. Quality Assurance	
	5. Quality Control	
G. Maintenance	**Road Asset Management Manual**	**RAAM**
	1. Maintenance Management	
	2. General Maintenance	
	3. Pavement Maintenance	
	4. Bridges and Structures Maintenance	

This table continues onto the next page...

Project Cycle Stage	Manual: Volume or Part/Chapter	Code
H. Operations	**Road Operation Manual**	ROM
	1. Traffic Management	
	2. Vehicle Load Control	
	3. Emergency Services	
	4. Tolling	
I. Monitoring and Evaluation	**Road Design Manual**	MEM
	1. Performance Monitoring Manual	
	2. Technical Audits	
	3. Poverty, Gender Equality and Social Inclusion Monitoring	

This Road Design Manual, Volume 2, Part 2 – Drainage Design is part of the Roads Design Manual made up of a series of volumes as shown below:

Table 1.1 Road Design Manual (RDM) Coding Structure

Vol.	Manual Title	Part Name	Code
1	**Road Design Manual: Vol. 1** Geometric Design	**Part 1** - Topographic Survey	RDM 1.1
		Part 2 – Traffic Surveys	RDM 1.2
		Part 3 – Geometric Design of Highways, Rural and Urban Roads	RDM 1.3
2	**Road Design Manual: Vol. 2** Hydrology & Drainage Design	**Part 1** – Hydrological Surveys	RDM 2.1
		Part 2 – Drainage Design	RDM 2.2
3	**Road Design Manual: Vol. 3** Materials & Pavement Design for New Roads	**Part 1** – Ground Investigations and Material Prospecting	RDM 3.1
		Part 2 – Materials Field and Laboratory Testing	RDM 3.2
		Part 3 – Pavement Foundation and Materials Design	RDM 3.3
		Part 4 – Flexible Pavement Design	RDM 3.4
		Part 5 – Rigid Pavement Design	RDM 3.5
4	**Road Design Manual: Vol. 4** Bridges & Retaining Structures Design	**Part 1** – Geotechnical Investigation and Design	RDM 4.1
		Part 2 – Bridge and Culvert Design	RDM 4.2
		Part 3 – Retaining Structures Design	RDM 4.3
		Part 4 – Reinforced Fill Structures Design	RDM 4.4
		Part 5 – Bridges and Structures Condition Survey	RDM 4.5
		Part 6 – Bridge Maintenance Design	RDM 4.6
5	**Road Design Manual: Vol. 5** Pavement Maintenance, Rehabilitation & Overlay Design	**Part 1** – Pavement Condition Survey	RDM 5.1
		Part 2 – Pavement Maintenance, Rehabilitation and Overlay Design	RDM 5.2
6	**Road Design Manual: Vol. 6** Traffic Control Facilities & Communication Systems Design	**Part 1** – Road Marking	RDM 6.1
		Part 2 – Traffic Signs	RDM 6.2
		Part 3 – Traffic Signals and Communication System	RDM 6.3
		Part 4 – Other Traffic Control Devices	RDM 6.4
7	**Road Design Manual: Vol. 7** Road Lighting Design	**Part 1** – Grid-connected Road Lighting	RDM 7.1
		Part 2 – Solar Road Lighting	RDM 7.2

This manual must be applied sensibly and flexibly in conjunction with the skill and judgement of the designer. Compliance with the guidance given in the manual does not relieve designers of the responsibility for establishing that their design is suitable, appropriate, safe, adequate and sustainable for the purpose stated in the project requirements.

1.2 Objectives of the Manual

This Manual provides procedures recommended for analysing and designing effective road drainage facilities. Hydraulic facilities include open channels, bridges, culverts, storm drains, and storm-water quantity and quality control systems. Each can be part of a larger facility that drains water. In analysing or designing drainage facilities, investment of time, expense, concentration, and task completeness should be influenced by the relative importance of the facility.

1.3 Scope of this Part

This Manual has been developed to provide engineers with basic working knowledge of hydraulics. Necessary basic design elements are included such that the engineer can design road drainage with ease. For completeness of hydrological and hydraulic design, the designer should refer to Part 1 of this Manual.

1.4 Organisation of the Manual

This Manual is organised as follows:

- **Chapter 1**: Manual Introduction.
- **Chapter 2**: Hydraulic Design of Open Channels.
- **Chapter 3**: Drifts and Low-level crossings.
- **Chapter 4**: Hydraulic Design of Culverts.
- **Chapter 5**: Bridge Design.
- **Chapter 6**: River Channel Training & Erosion Control.
- **Chapter 7**: Urban Drainage.

The chapters are aimed at providing information on the methods and procedures for design of the drainage facilities. Each chapter will cover types of facilities, design considerations and design criteria, hydraulic design (and design limitations where necessary). The common types of drainage facilities are considered and the basis for selection provided. Design procedures and application examples are provided. In addition to nomographs and basic equation solutions, application examples using excel spreadsheet and computer software are provided. Check lists, design charts and tables, and calculation forms provide the designer with necessary tools to perform a wide range of hydraulic analysis and design. Other factors necessary in design include proper location and alignment, debris loading, channel stability and sediment movement, minimisation of long-term maintenance requirements, outlet channel protection, safety, structural, economic and project life cycle are included.

1.5 Design Process

The following section describes the hydraulic design activities typically expected to occur in each phase of a project.

1.5.1 Planning and Programming

One of the objectives of the planning and programming phase is to develop a planning-level cost estimate. Certain projects involving significant drainage-related challenges may require some initial hydrological and hydraulic investigation in order to appropriately estimate the nature and approximate size of required drainage structures for estimating purposes. Adequate consultation with relevant authorities should be done during the planning and programming phase to assess whether drainage issues will pose significant challenges to the project.

1.5.2 Preliminary Design

In the preliminary design phase, general background information on hydrology and hydraulics is required to identify major drainage features and regulatory constraints.

The locations and sizes of proposed cross-drainage structures (bridges and culverts) must be determined early in the preliminary design phase because of their potential to affect the roadway profile and other elements of the preliminary design of the project. Preliminary hydraulics analyses for bridges will enable the determination of the bridge limits, span/girder type, span lengths, bent locations, and bent orientation.

Since many of the design parameters for drainage structures are to be established during the preliminary design phase, it is necessary to conduct the bulk of the hydrological and hydraulic analysis during this phase. These analyses will usually include, but may not be limited to:

1. Field reconnaissance.
2. Collection of relevant data on the stream and watershed.
3. Gathering of relevant previous hydrological studies previously carried out.
4. Conducting required hydraulic surveys of existing structures and streams.
5. Obtaining available topographic mapping of the streams and floodplains.
6. Determining stream flood profiles for existing conditions through hydraulic modelling.
7. Determining required sizes of drainage structures to meet design criteria:
   a. For bridges this includes establishing preliminary opening size, span lengths, pier locations and girder elevation.
   b. For culverts preliminary design of opening size and profile is performed.
   c. For storm drains this includes preliminary design of trunk alignment, size and profile.
8. Estimating stream flood profiles under proposed project conditions (potentially for multiple design alternatives) to determine project impacts.
9. Adjusting proposed structure designs as necessary to mitigate project impacts.

1.5.3 Environmental

Preliminary hydraulic studies are needed in the preparation of environmental documentation to evaluate the impacts of the proposed project on waterways and floodplains. Changes in water surface elevation, construction in channels, bridge construction methods, etc, commonly impact water resources. The identification of appropriate temporary and permanent stormwater quality best management practices will be required during the environmental documentation phase.

1.5.4 Design Development

As part of the detailed design process, stream crossing hydrology and hydraulics should be refined and finalised. Refinement is usually needed to reflect detailed field survey data, changes in basic design conditions or assumptions, or to reflect revised methodology if there has been a significant delay between schematic development and detailed hydraulic design.

Bridge scour evaluation during the hydraulic design process for all bridges should be available.

In addition to bridge hydraulic design and scour evaluations, a number of other hydrology and hydraulic tasks are required as a project design is being finalised. These tasks include, but are not limited to:

i. Refining the hydraulic design of culverts to finalise sizes, invert profile, end treatment and outlet protection;

ii. Preparing final storm drain details including design of appropriately sized inlets at the proper spacing and lateral sizing;

iii. Preparing pump station details for projects involving pump facility construction; and

iv. Preparing or contributing to the development of Stormwater Pollution Prevention Plans after the roadway drainage design is completed.

Finalised hydraulic calculation sheets and hydraulic reports should be reviewed and approved for implementation.

2 Hydraulic Design of Open Channels

2.1 General

An open channel is defined as a conduit or conveyance (artificial or natural) in which water flows with a free surface. A free surface means that the surface is open to the atmosphere and/or there is no additional pressure on the flow other than atmospheric pressure. Flow is caused by gravity and streams tend to follow the path of least resistance.

There are various types of open channels encountered by the designer of transportation facilities including:

1. Stream channels;
2. Chutes;
3. Roadside channels or ditches;
4. Irrigation channels; and
5. Drainage ditches.

The principles of open channel flow hydraulics are applicable to all drainage facilities including culverts.

Stream channels are usually:

1. Natural channels with their size and shape determined by natural forces;
2. Compound in cross section with a main channel for conveying low flows and a floodplain to transport flood flows; and
3. 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:

1. Man-made with regular geometric cross sections, such as Invert block drains (IBDs) and Slotted Drains; and
2. 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 as appropriate.

2.2 Flow Types in Channels

2.2.1 Flow Characteristics

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 principal 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.

1. Specific Energy (E): is defined as the energy head relative to the channel bottom. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy E becomes the sum of the depth and velocity head:

$$E = y + \alpha\left(\frac{V^2}{2g}\right)$$

Equation 2.1

Where,

y = Depth.

α = Velocity distribution coefficient (see Equation 2.2).

V = Mean velocity, m/s.

g = Gravitational acceleration, 9.81 m/s^2.

2. Velocity Distribution Coefficient (α): Due to the presence of a free surface and also to friction along the channel boundary, the velocities are not uniformly distributed in the channel section. As a result, the velocity head of an open channel is usually greater than the average velocity head computed as $(Q/A_t)^2/2g$. A weighted average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient, defined as:

$$\alpha = \sum_{i=1}^{n}\left(\frac{K_i^{\,3}}{A_i^2}\right) \Big/ \left(\frac{K_t^{\,3}}{A_t^2}\right)$$

Equation 2.2

Where,

α = Velocity distribution coefficient.

K_i = Conveyance in subsection (see Equation 2.13).

K_t = Total conveyance in section = average velocity in subsection, m/s (see Equation 2.13).

A_i = Cross-sectional area of subsection, m^2.

A_t = Total cross-sectional area of section, m^2.

n = Number of subsections.

3. Total Energy Head: The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. The locus of the energy head from one cross section to the next defines the energy grade line. See Figure 2.1 for a plot of the specific energy diagram.

   Written between an upstream cross section designated 1 and a downstream cross section designated 2, the energy equation becomes.

$$(z_1 + d_1) + \left(\frac{v_1^2}{2g}\right) = (z_2 + d_2) + \left(\frac{v_2^2}{2g}\right) + h_L$$

Equation 2.3

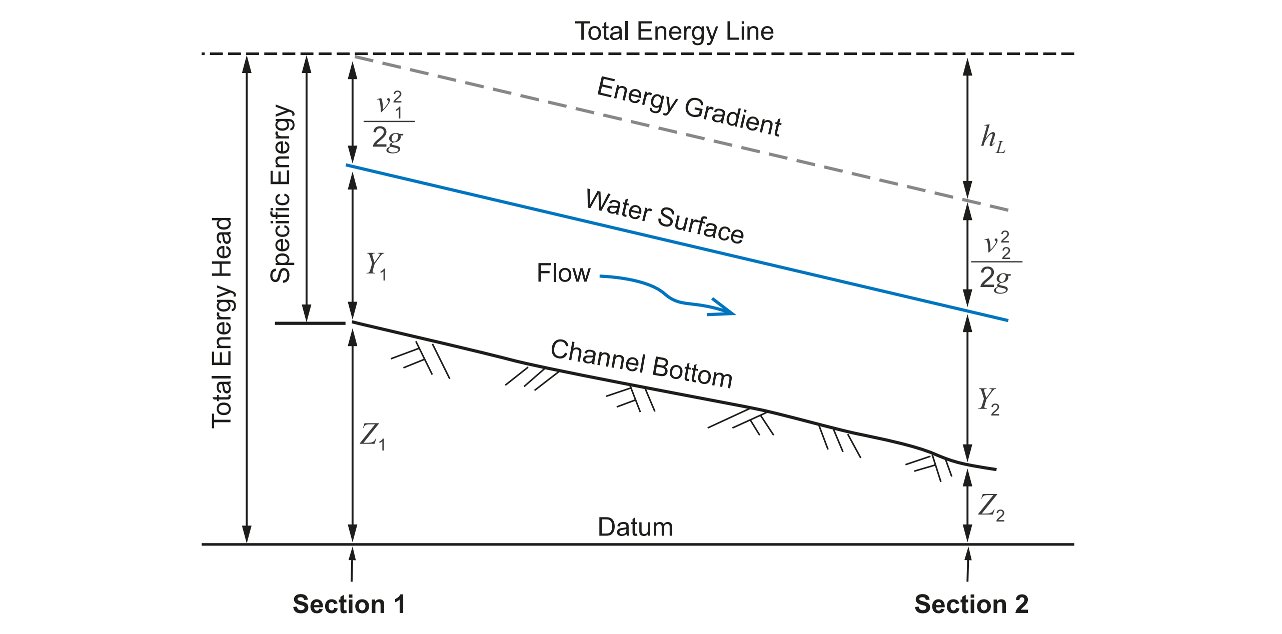
Where,

h_L = Head loss (m)

4. Steady and Unsteady Flow: A steady flow is one in which the discharge passing a given cross-section is constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. When the discharge varies with time, the flow is unsteady.

5. Uniform Flow and Non-Uniform Flow: A non-uniform flow is one in which the velocity and depth vary in the direction of motion, while they remain constant in uniform flow. Uniform flow can only

Figure 2.1 Specific Energy and Discharge Diagram for Rectangular Channels



occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope in the flow direction. However, non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

6. Gradually-Varied and Rapidly-Varied: A non-uniform flow in which the depth and velocity change gradually in the flow direction, such that vertical accelerations can be neglected, is referred to as a gradually-varied flow; otherwise, it is considered to be rapidly-varied.
7. Froude Number (F_r): The Froude number is an important dimensionless parameter in open channel flow. It represents the ratio of inertia forces to gravity forces and is defined by:
8. Subcritical Flow: is distinguished from **supercritical flow** by the Froude number (F_r).

   This expression for Froude number applies to any single section channel of non-rectangular shape. It is recommended that design achieves a Froude Number less than 0.9 (subcritical flow).

9. Critical Flow: The flow in open channels at which the energy content of the fluid is at a minimum. Also, that flow at which the variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called **Critical Depth** at which the Froude number has a value of one. Critical depth is also the depth of maximum discharge when the specific energy is held constant.

$$\frac{\alpha Q^2}{g} = \frac{A^3}{T}$$

Equation 2.5

Where,

α = velocity distribution coefficient.

Q = total discharge, m³/s.

g = gravitational acceleration, 9.81 m/s².

A = cross-sectional area of flow, m².

T = channel top width at the water surface, m.

10. Subcritical Flow: Depths greater than critical occur in subcritical flow and the Froude number is less than one. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is always located downstream.
11. Supercritical Flow: Depths less than critical-depth occur in supercritical flow and the Froude number is greater than one. Small water-surface disturbances are always swept downstream in supercritical flow, and the location of the flow control is always upstream.

12. Hydraulic Jump: A hydraulic jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump, and energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures.

13. Flow in Bends: Flow around a bend in an open channel induces centrifugal forces because of the change in flow direction. This results in a super-elevation of the water surface at the outside of bends and can cause the flow to splash over the side of the channel if adequate freeboard is not provided. This super-elevation can be estimated by the following equation.

$$\Delta d = \frac{V^2\,T}{g\,R_c}$$

Equation 2.6

Where,

Δd = Difference in water surface elevation between the inner and outer banks of the channel in the bend, m.

T = Surface width of the channel, m.

g = Gravitational acceleration, 9.81 m/s^2.

R_c = Radius to the centreline of the channel m.

14. Shear Stress: In a uniform flow, shear stress is equal to the effective component of the gravitational force acting on the body of water parallel to the channel bottom. The average shear stress is equal to:

$$\tau = \gamma\,R\,S$$

Equation 2.7

Where,

τ = Average shear stress, Pa.

γ = Unit weight of water, 9810 N/m^3 (at 15.6°C).

R = Hydraulic radius, m.

S = Average bed slope or energy slope, m/m.

The maximum shear stress for a straight channel occurs on the channel bed and is less than or equal to the shear stress at maximum depth. The maximum shear stress is computed as follows:

$$\tau d = \gamma\,d\,S$$

Equation 2.8

Where,

τd = Maximum shear stress, Pa.

d = Maximum depth of flow, m.

2.2.2 Continuity Velocity

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the following form:

$$Q = A_1\,V_1 = A_2\,V_2$$

Equation 2.9

1
2
Hydraulic Design of Open Channels

Where,

Q = discharge (m³/s)

A = flow cross-sectional area (m²)

V = mean cross-sectional velocity (m/s, perpendicular to the flow area).

The subscripts $_1$ and $_2$ refer to successive cross sections along the flow path.

As indicated by the Continuity Equation, the average velocity in a channel cross-section, (v) is the total discharge divided by the cross-sectional area of flow perpendicular to the cross-section. It is only a general indicator and does not reflect the horizontal and vertical variation in velocity.

Velocity varies horizontally and vertically across a section. Velocities near the ground approach zero. Highest velocities typically occur some depth below the water surface near the station where the deepest flow exists. As a result, the velocity head of an open channel is usually greater than the average velocity head computed as $(Q/A_t)^2/2g$. A weighted average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient, (α), defined as in sub-section 2.2.1.

For one-dimensional analysis techniques such as the Slope Conveyance Method and (Standard) Step Backwater Method (see Chapter 5), ignore the vertical distribution, and estimate the horizontal velocity distribution by subdividing the channel cross section and computing average velocities for each subsection. The resulting velocities represent a velocity distribution.

2.2.3 Energy Principles

Conservation of energy is a basic principle in open channel flow. The total energy at given channel cross section can be represented as:

$$H = (z + d) + \frac{v^2}{2g}$$

Equation 2.10

Where,

H = Total energy head (m).

z = Height of channel bed above some reference datum (m), known as elevation head.

d = Depth of flow normal to the direction of flow (m), known as hydrostatic head.

V = Velocity of flow.

g = Acceleration due to gravity (9.81 m/s²).

The term $V^2/2g$ within this equation is known as the velocity head.

Written between an upstream cross section designated 1 and a downstream cross section designated 2, the energy equation becomes,

$$(z_1 + d_1) + \frac{v_1^2}{2g} = (z_2 + d_2) + \frac{v_2^2}{2g} + h_L$$

Equation 2.11

Where,

h_L = Head loss (m)

2.3 Channel Flow Parameters by Manning's Equation

2.3.1 Manning's Equation

Manning's equation which is commonly used for the computation of steady flow in natural and constructed channels is a single section analysis or slope conveyance method. This method is justified to standard roadway ditches, culverts and storm drain outfalls.

1

2

Hydraulic Design of Open Channels

Manning's equation: $Q = \left(\frac{1}{n}\right) A\, R^{2/3}\, S^{1/2}$

Equation 2.12

Where,

Q = Discharge rate, m^3/s.

A = Cross sectional flow area, m^2.

R = Hydraulic radius A/P.

S = Energy gradient line slope, m/m.

n = Manning's roughness coefficient.

In channel analysis, it is often convenient to group the channel cross-sectional properties in a single term called the channel conveyance (K), shown in Equation 2.13 below.

$$K = \left(\frac{1}{n}\right) A\, R^{2/3}$$

Equation 2.13

Manning's equation can then be written as:

$$Q = K\, S^{1/2}$$

Equation 2.14

and conveyance, $K = Q/\sqrt{S}$

Equation 2.15

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and is independent of the stream bed 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**.

2.3.2 Roughness Coefficient

A critical parameter in solving Manning's equation is the Manning's roughness coefficient, n. The selection of an appropriate Manning's n value for design purposes is often based on observation and experience. 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 streamflow data. Manning's 'n' values for artificial channels are more easily defined than for natural stream channels.

For rigid boundary channels (e.g., concrete lined) the n-value is fairly constant, while in grass-lined channels the value can vary quite dramatically based on the type of vegetation and its height relative to flow depth.

Table 2.1 provides a tabulation of typical Manning's n values for both artificial and natural stream channels.

2.3.3 Subdividing Cross Sections

Coordinates of lateral distance and ground elevation that locate individual ground points define the cross-sectional geometry of streams. The cross section is normally taken where the flow direction is along a single straight line. However, in wide floodplains or bends it may be necessary to use a section along intersecting straight lines such as a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Table 2.1 Typical Channel Manning's Roughness Coefficients *n*

No.	Type of channel and description	Minimum	Normal	Maximum
1	**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	0.030	0.035	0.040
	4. Earth bottom and rubble sides	0.025	0.030	0.035
	5. Stoney 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			
	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
2	**Minor Streams (top width at flood stage < 30m)**			
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	?	?	?
	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 flood-ways with heavy stand of timber and under-brush	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	?	?	?
	2. Bottom: cobbles with large boulders.	0.040	0.050	0.070

3	**Floodplain**			
a	Pasture, no brush			
	1. Short grass	0.025	0.030	0.035
	2. High grass	0.030	0.035	0.050
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	?	?	?
	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,	0.080	0.100	0.120
	5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
4	**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
5	**Various Open Channel Surfaces**			
a	Concrete	0.012-	0.020	--
b	Gravel bottom with			
	1. Concrete	0.020	--	--
	2. Mortared stone	0.023	--	--
	3. Riprap	0.033	--	--
c	Natural Stream Channels			
	1. Clean, straight stream	0.030	--	--
	2. Clean, winding stream	0.040	--	--
	3. Winding with weeds and pools	0.050	--	--
	4. With heavy brush and timber	0.100	--	--
d	Floodplains			
	1. Pasture	0.035	--	--
	2. Field Crops	0.040	--	--
	3. Light Brush and Weeds	0.050	--	--
	4. Dense Brush	0.070	--	--
	5. Dense Trees	0.100	--	--

Cross sections shall be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as in the case of over-bank flows.

The conveyances of each subsection are computed separately to determine the flow distribution and K (conveyance) and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection.

2.3.4 Switchback Phenomenon

If the cross section is improperly subdivided, the solution 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. 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.

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.

2.3.5 Stage-Discharge Relationship for Flow in the Stream

Since Manning's Equation does not allow a direct solution to water depth (given discharge, longitudinal slope, roughness characteristics, and channel dimensions), an indirect solution to channel flow is necessary. This is accomplished by developing a stage-discharge relationship for flow in the stream. This involves:

1. Apply a range of incremental water surface elevations to the cross-section,
2. 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
3. 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.

2.3.6 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.

2.3.7 Other Formulae

Other formulae used to analyse flow in channels and conduits include:

Chézy: $V = C(RS)^{1/2}$

Equation 2.16

Darcy Weisberch: $V = \{\left(\frac{2g}{f}\right)^{1/2} (RS)^{1/2}\}^{1/2}$

Equation 2.17

$$\text{Colebrook: White } V = -(32gRS)^{½} log_{10} \{ \frac{k_s}{14.8R} + \frac{1.225v}{R} (32gRS)^{½} \}$$

Equation 2.18

Where,

V = Cross-sectional mean velocity (m/s).

R = Hydraulic radius (m).

S_f = Friction gradient (dimensionless).

C = Chézy coefficient ($m^{½}$ /s).

n = Manning coefficient ($s/m^{1/3}$).

f = Darcy-Weisbach friction factor (dimensionless).

k_s = Surface roughness (m).

v = Kinematic viscosity (m^2/s).

g = Acceleration due to gravity (m/s^2).

2.4 Hydraulic Analysis

The hydraulic design process for open channels consists of establishing criteria, developing, and evaluating alternatives, and selecting the alternative that best satisfies the criteria. The analysis is aimed at determining the depth and velocity at which a given discharge will flow in a given geometry, roughness and slope. Depth and velocity of flow are necessary to design channel linings and highway drainage structures.

There are two methods of analysis used to analyse open channel flow regime flow, single section analysis and step backwater method.

2.4.1 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 for a given discharge and cross-section, 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. Specific application of this method will be covered in design of roadside channels (covered in this chapter), side ditches and drains in Chapter 6 and urban storm drainage covered in Chapter 7 of this Manual.

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:

a. Select the typical cross-section at or near the location where the stage-discharge curve is needed.

b. Assign a roughness coefficient (Manning's n-value) as described above.

c. Compute the area, wetted perimeter and weighted n-value for each submerged subsection.

d. Compute the subsection discharges with Manning's Equation. Use the subsection values for roughness, area, wetted perimeter, and slope. The sum of all of the incremental discharges represents the total discharge for each assumed water surface elevation.

Note: Compute the average velocity for the section by substituting the total section area and total discharge into the continuity equation $V = Q/A$.

e. Tabulate or plot the water surface elevation and resulting discharge (stage versus discharge).

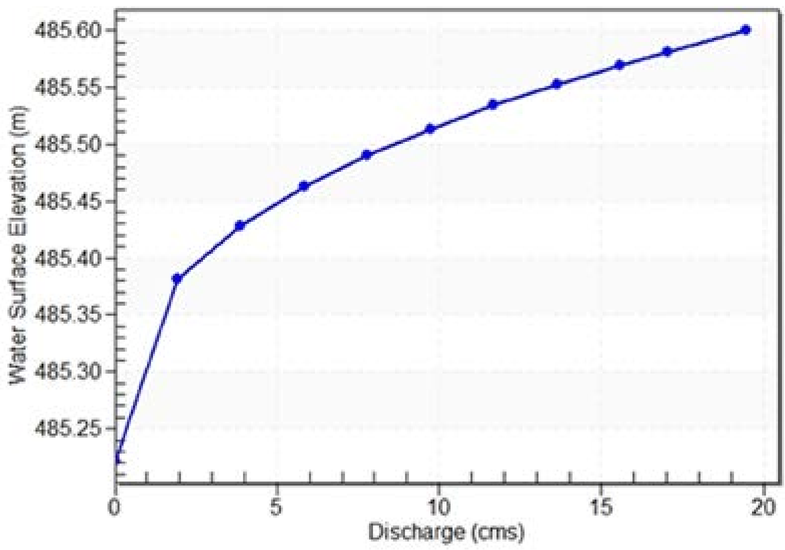
f. Repeat the above steps with a new channel depth or add a depth increment to the trial depth. The choice of elevation increment is somewhat subjective. However, if the increments are less than about 0.075 m) considerable calculation is required. On the other hand, if the increments are greater than 0.5 m. the resulting stage-discharge relationship may not be detailed enough for use in design.

g. Determine the depth for a given discharge by interpolation of the stage versus discharge table or plot.

2.4.2 Graphical Technique and Use of Nomographs

Graphical techniques or nomographs given as Charts B.1 to B.4 in the Appendix and Table 2.2 can be used for trapezoidal and prismatic channels. Chart B.1 is reproduced here below for ease of reference. A worked example with solution is given in Appendix A.2. The best approach, especially in the case of stream channels, is to use a computer program such as WSPRO or HEC-2 to obtain the normal depth.

Figure 2.2 Stage Discharge Curve for Slope Conveyance



In the charts, tables and nomographs, the symbols represent:

d or y = depth of flow

B = Bottom width

S = Channel slope

Z:1 = horizontal to vertical channel sides slope

Other symbols are defined in the main text.

Other nomographs and charts that simplify design by these procedures are reproduced in the appendices.

2.4.3 Step-Backwater (Standard Step Method) Analysis

Step backwater method is used to compute the complete water surface profile in a stream reach or analyse other gradually varied flow situations in open channels. This method is the most appropriate for evaluating surface water elevations, bridge hydraulic design and major channel design. This

Chart B.1 Solution to Manning's Equation for Channels of Various Side Slopes

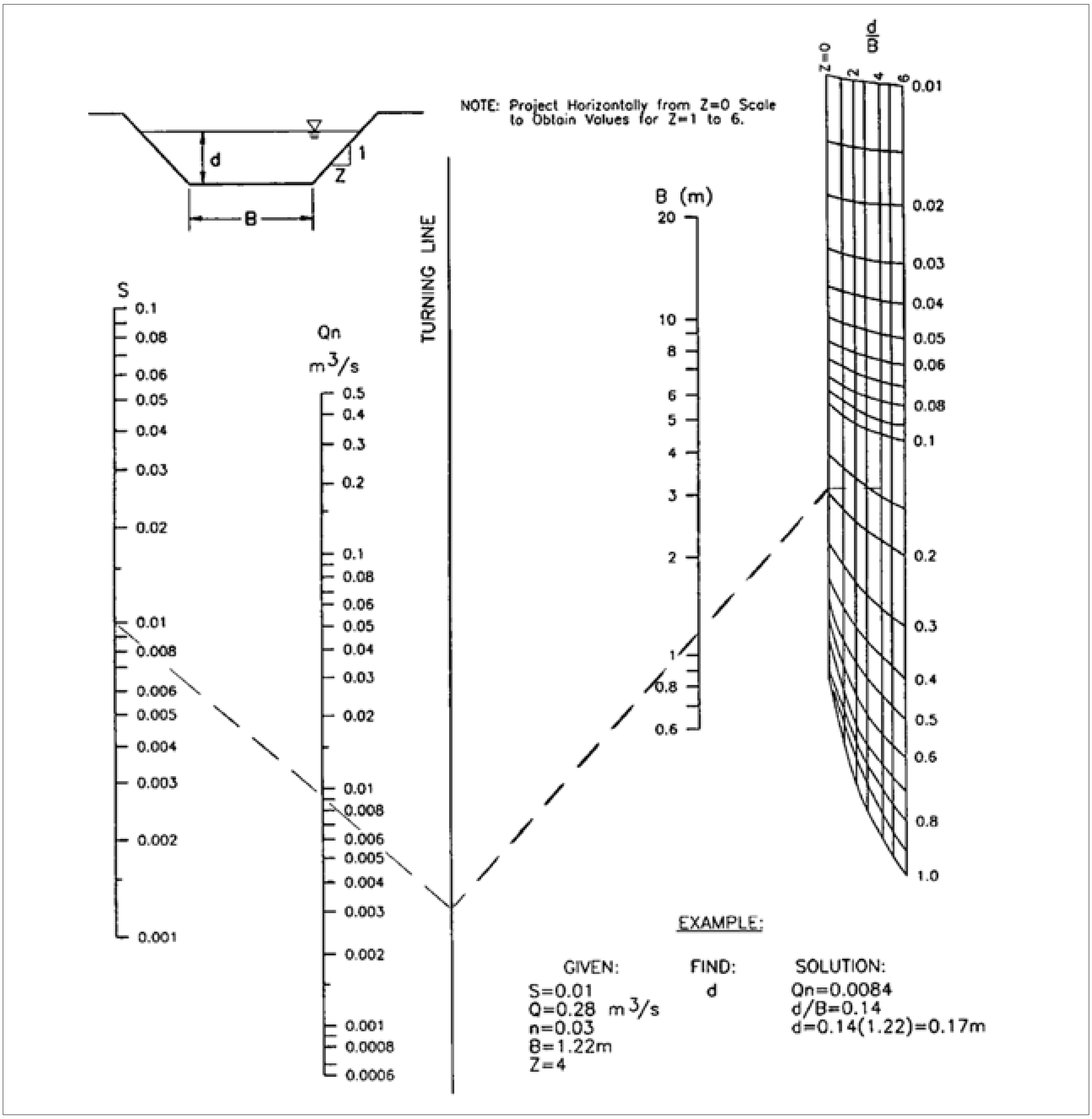


Table 2.2 Flow in Trapezoidal Channels by Manning's Equation Values of $Q_n/(B^{8/3}/S^{1/2})$

y/B	Z = 0	Z = 1/4	Z = 0.5	Z = 0.75	Z = 1	Z = 1.25	Z = 1.5	Z = 1.75	Z = 2	Z = 3
0.05	0.00638	0.00649	0.0066	0.00667	0.00673	0.0068	0.00687	0.00693	0.00693	0.00714
0.10	0.0191	0.0198	0.0206	0.021	0.0125	0.0129	0.0222	0.0225	0.0229	0.0241
0.15	0.0356	0.0377	0.0394	0.0409	0.0423	0.0435	0.0446	0.0459	0.0466	0.0505
0.20	0.0548	0.059	0.0628	0.066	0.0687	0.0714	0.0741	0.0761	0.0781	0.0869
0.25	0.0761	0.0835	0.0896	0.0956	0.101	0.106	0.11	0.114	0.118	0.134
0.30	0.0983	0.11	0.121	0.13	0.138	0.147	0.153	0.161	0.167	0.193
0.35	0.122	0.14	0.155	0.169	0.182	0.193	0.204	0.214	0.225	0.264
0.40	0.147	0.171	0.193	0.211	0.23	0.246	0.262	0.277	0.292	0.349
0.45	0.172	0.204	0.233	0.259	0.284	0.306	0.328	0.349	0.369	0.448
0.50	0.199	0.24	0.277	0.312	0.345	0.374	0.403	0.43	0.457	0.561
0.60	0.252	0.315	0.374	0.431	0.483	0.531	0.578	0.822	0.665	0.835
0.70	0.308	0.398	0.486	0.567	0.645	0.72	0.788	0.855	0.922	1.18
0.80	0.365	0.488	0.61	0.727	0.835	0.943	1.04	1.14	1.23	1.60
0.90	0.422	0.586	0.747	0.902	1.05	1.19	1.33	1.46	1.59	2.09
1.00	0.481	0.687	0.895	1.10	1.30	1.49	1.66	1.84	2.01	2.67
1.25	0.63	0.976	1.34	1.70	2.05	2.38	2.71	3.02	3.33	4.53
1.50	0.78	1.31	1.87	2.44	3.00	3.54	4.07	4.59	5.08	7.00
1.75	0.936	1.68	2.51	3.35	4.18	4.99	5.77	6.54	7.34	10.20
2.00	1.08	2.10	3.25	4.43	5.6	6.73	7.88	8.95	10	14.20
2.50	1.39	3.08	6.06	7.14	9.22	11.3	13.3	15.3	17.2	24.90
3.00	1.70	4.26	7.41	10.71	14.1	17.4	20.6	23.8	27	39.30
4.00	2.33	7.27	13.6	20.54	27.7	34.7	41.7	48.5	55.3	82.10

aspect of bridge design will be covered in Chapter 5. It is especially useful for determining unrestricted water surface profiles where a highway crossing is planned, and for analysing how far upstream the

Table 2.3 Recommended Value for Manning's n and Maximum Velocity of Flow for Channels

Type of Channel	Mannin's n	Maximum Velocity (m/s)
Closed Conduits		
Concrete	0.010 - 0.012	6
Corrugated Metal	0.024	6
Vitrified Clay	0.01	6
Brick	0.014	3 - 4.5
Open Paved Channels		
Concrete	0.012	6
Dressed Stone in Mortar	0.015	6
Random Stone in Mortar	0.015	5.5 - 6
Dry Rumble Riprap	0.025	3 - 4.5
Brick	0.014	3 - 4.5
Asphalt	0.014	5.5 - 6
Open Unpaved Channels		
Soft Sedimentary Rock, smooth and uniform	0.040	2.5 - 3
Soft Sedimentary Rock, Jagged and irregular.	0.045	1.5 - 2.5
Hard rock, smooth and uniform	0.040	6
Coarse gravels	0.025	1.5
Stiff clay	0.02	1.1
Sand, loam, fine gravel, volcanic clay	0.022	0.6
Open Channel, Vegetated		
Clear, straight, no deep pools.	0.028	
Straight but some weeds	0.034	
Sluggish with weeds and deep pools.	0.065	
Very weedy reaches.	0.11	
Shorter than 150mm long		1 - 1.5
Length 150 - 250mm		1.5 - 2
Length 250 - 600mm		2 - 2.5
Longer than 600mm		2.5 - 3

water surface elevations are affected by a culvert or bridge.

At least four cross sections are required to complete this procedure, but you often need more than three cross sections. The number and frequency of cross sections required is a direct function of the irregularity of the stream reach

The Standard Step Method uses the Energy Balance Equation, which allows the water surface elevation at the upstream section (2) to be found from a known water surface elevation at the downstream section. Because the calculations involved in this analysis are tedious and repetitive, a computer program such as the FHWA/USGS program WSPRO or U.S. Army Corps of Engineers HEC-2 can be used to assist with the equations.

The WSPRO program has been designed to provide a water surface profile for six major types of open channel flow situations:

1. Unrestricted flow;
2. Single opening bridge;
3. Bridge opening(s) with spur dikes;

4. Single opening embankment overflow; and
5. Multiple alternatives for a single site, and multiple openings.

The HEC-2 program developed by the U.S. Army Corps of Engineers is widely used for calculating water surface profiles for steady gradually varied flow in a natural or man- made channel. Both subcritical and supercritical flow profiles can be calculated. The effects of bridges, culverts, weirs, and structures in the floodplain may also be considered in the computations. This program is also designed for application in flood plain management.

The computation of water surface profiles by both WSPRO and HEC-2 is based on the standard step method in which the stream reach is divided into sub reaches by cross sections spaced so that the flow is gradually varied in each sub reach. The energy equation is then solved in a systematic fashion for the stage at one cross section based on the stage at the previous cross section.

The method requires definition of the geometry and roughness of each cross section as discussed at the beginning of Section 2.4. 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. Refer to HEC-15 for more information on channel design techniques and considerations

To develop the methodology, the energy equation is repeated from Section 2.4.

$$h_1 + \alpha_1 \frac{v_1^2}{2g} = h_2 + \alpha_2 \frac{v_2^2}{2g} + h$$

Equation 2.19

Where,

h_1, h_2 = The upstream and downstream stages, respectively, m.

V = Mean velocity, m/s.

α = Velocity distribution coefficient.

H_l = Head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, m.

g = Acceleration of gravity, 9.81 m/s^2.

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:

$$h_L = K_m\left[\frac{\alpha_1 V_1{}^2}{2g} - \frac{\alpha_2 V_2{}^2}{2g}\right] + S_f L$$

Equation 2.20

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.

α = Velocity distribution coefficient.

V = Mean velocity, m/s.

g = Acceleration of gravity, 9.81 m/s^2.

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.

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 (Figure 2.3). Data collection upstream and downstream shall be rounded up to the nearest 50 meters.

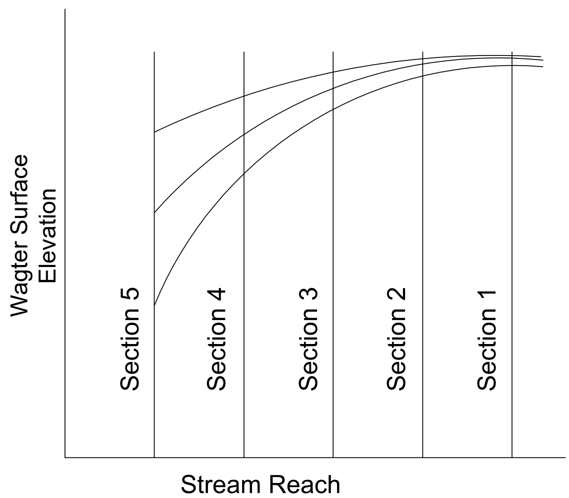
Equations for determining upstream and downstream reach lengths are as follows: -

$$L_{dn} = 1.2\,\frac{HD^{0.8}}{S}$$

Equation 2.21

$$L_u = 2.0\,\frac{HD^{0.6} \text{ x } HL^{0.5}}{S}$$

Figure 2.3 Water Surface Profile Convergence



Equation 2.22

Where,

L_{dn} = Downstream study length (along main channel), m (for normal depth starting conditions).

L_u = Estimated upstream study length (along main channel), m (required for convergence of the modified profile to within 30 mm of the base profile).

HD = Average hydraulic depth (1- percent chance event flow area divided by the top width), m.

S = Average reach slope, m/m.

H_L = Head loss ranging between 0.15 m and 1.5 m at the channel crossing structure for the 1-percent chance flood, m.

Computation Procedure

A sample procedure form for use in calculating water surface profiles is shown as Figure 2.4.

Column 1 Cross Section No. is the identification number. Kilometres upstream from the mouth are recommended.

Figure 2.4 Sample Computation Sheet

Cross Section No.	Water Surface Elevation		Area	Hydraulic Radius (R)	$R^{2/3}$	n	K	K_t	S_t	L	h_t	$\frac{K^3}{A^2}$	α	V	$\frac{\alpha V^2}{2g}$	$\Delta\frac{\alpha V^2}{2g}$	h_o	Δ Water Surface Elevation
	Assumed	Computed																
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

Column 2 Assumed water surface elevation, which must agree with the resulting computed water surface elevation within +/-0.015m, for trial calculations to be successful.

Column 3 Computed, the rating curve value for the first section, but thereafter, the value calculated by adding WS to the computed water surface elevation for the previous section.

Column 4 A, cross-section area: If the section is complex and has been subdivided into several parts, use one line on the form for each subsection and sum to get A.

Column 5 R, hydraulic radius.

Column 6 $R^{2/3}$.

Column 7 Manning's roughness coefficient 'n'.

Column 8 K, conveyance, defined as $C_m AR^{2/3}/n$, where C_m is 1.

Column 9 K_t, average conveyance for reach, calculated by 0.5(K_{td} + K_{tu}), where subscripts D and U refer to downstream and upstream ends of the reach, respectively.

Column 10 S_f, average slope through the reach determined by $(Q/K_t)^2$.

Column 11 L, the discharge-weighted reach length.

Column 12 h_f, energy loss due to friction through the reach, calculated by $h_f = (Q/K_t)^2 L = S_f L$

Column 13 $\Sigma(K^3/A^2)$, part of the expression relating distributed flow velocity to an average value.

Column 14 α, the velocity distribution coefficient, calculated by $\Sigma(K^3/A^2)/\ (K^3/A^2)$ where the numerator is the sum of values in column 13 and the denominator is calculated from K_t and A_t.

Column 15 V, the average velocity calculated by Q/A.

Column 16 $\alpha V^2/2g$, the average velocity corrected for flow distribution.

Column 17 $\Delta(\alpha V^2/2g)$, the difference between velocity heads at the downstream and upstream sections. A positive value indicates velocity is increasing, therefore, use a contraction coefficient for 'other losses'. A negative value indicates the expansion coefficient should be used in calculating 'other losses'.

Column 18 ho is "other losses," and is calculated by multiplying either the expansion or contraction coefficient, K_m, by the absolute value of column 17.

Column 19 ΔWS, the change is water surface elevation from the previous cross section. It is the algebraic sum of columns 12, 17 and 18.

2.4.4 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 scour or deposits in each stream tube, determined by sediment routing, will give the variation of channel geometry in the vertical direction.

2.5 General Channel Design Considerations

2.5.1 Objective

It is necessary to carry out channel analysis in order to evaluate:

1. Channels capacity and the potential for flooding of the highway and adjacent properties.

2. The potential for disturbance of the channel upstream or downstream of the highway right-of way.
3. Changes in lateral flow distribution.
4. Changes in velocity, depth and direction of flow.
5. The need for additional conveyance in case of excess runoff.
6. The need for channel lining or other bed or bank protection to prevent erosion.

2.5.2 General Design Considerations

1. Establishing the ecological requirements for the channel such as – determining the specific or generic riparian management requirements.
2. Design capacity should consider safety and welfare of road users and adjacent property owners.
3. Type and frequency of maintenance and accessibility of equipment.
4. Potential to become unstable for use due to high flows, velocities, etc during its service life.
5. Environmental impacts.
6. Design discharge to be consistent with requirements.
7. Freeboard - will depend on the consequence of overflow of the channel bank. At a minimum, the freeboard should be sufficient to prevent waves, superelevation changes, or fluctuations in water surface from overflowing the sides.
8. Side slopes should be based on the stability of the material the channel is to be constructed in (channel is shaped before any lining is applied or grass grows).

2.5.3 Channel Hydraulic Design

Assess the following when designing transportation drainage systems:

1. Potential flooding caused by changes in water surface profiles
2. Disturbance of the river system upstream or downstream of the road right-of-way
3. Changes in lateral flow distributions
4. Changes in velocity or direction of flow
5. Need for conveyance and disposal of excess runoff
6. Need for channel linings to prevent erosion.

2.5.3.1 Design Criteria

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.

1. **Stream Channels**

The following criterion applies to natural channels and may be revised if necessary:

a. The hydraulic effects of floodplain 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.

b. 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.

c. Stream bank stabilisation shall be provided, when appropriate, to any stream disturbance such as encroachment and should include both upstream and downstream banks and the local site.

d. 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.

e. Grass channels generally provide better habitat than hardened channel sections. Velocities should be carefully checked at design flows and the outer banks at bends should be specifically designed for increased shear stress.

f. Where open channels pass close to residential areas and schools, consideration must be given to the safety of children when designing for upper velocity limits or deep drains.

2. Roadside Channels

Adherence to the following criteria and guidelines is recommended:

a. Roadside channels should be lined to minimise or prevent erosion. Flexible linings are preferred. Rigid linings are acceptable when conditions warrant their use (i.e, when increased channel capacity is needed and channel geometry is fixed).

b. The flow in roadside channels should be subcritical whenever possible to avoid the adverse characteristics of supercritical flow (erosion, etc,). An exception to this is chutes. The flow in chutes is commonly supercritical because of the normally steep channel slope.

c. When possible, roadside channels should be located so that peak water surface elevation during passage of the design flow is outside the clear zone, unless a roadside barrier is provided.

d. The channel's slope will be dictated by adjacent terrain and right of way constraints. Where possible, roadside channels should be designed to have self-cleansing velocities and to avoid standing water in the roadway. (See Table 2.3). To prevent deposition of sediment, the minimum slope for turf lined roadside channels should be 0.5%.

e. If the underlying channel material is susceptible to erosion, either a flexible or rigid lining should be provided to minimise erosion. (Table 2.3 gives recommended maximum velocity above which it will necessitate provision of lining):

   i. Flexible Linings include turf, stone filling (fine, medium, and heavy), and dry rip-rap.

   ii. Rigid linings include grouted rip-rap, gabions, asphalt concrete, and Portland cement concrete.

f. Roadside channels within busy streets shall be covered for safety purposes. Slotted drains may be formed from a rectangular reinforced concrete or brick lined channel covered with a precast slotted reinforced concrete slab. Their use should be specified with caution as the cover slabs are susceptible to damage from traffic and impact from heavy loads and the drains are subject to clogging up with debris and other detritus. Because the drain is not normally visible to the cursory inspection, they seldom receive the regular maintenance and cleaning they require.

g. The following two maintenance activities should be considered during a field trip for inclusion as part of the project.
   i. Cleaning and reshaping turf lined channels.
   ii. Repairing or replacing failed or inadequate linings.

2.6 Channel Design Technique

The design procedures for all types of channels have many similar elements, however each type of channel will require unique inputs. Sub-section 2.6.1 will outline a process for assessing a natural stream channel and sub-section 2.6.2 will offer a more specific design procedure for roadside channels.

2.6.1 Stream Channels

The analysis of a stream channel, in most cases, takes place in conjunction with the design of a road hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the road in such a manner that will not cause damage to the road, 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 floodplain.

Step-by-Step Procedure

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

A. Data Collection.
   i. Topographic, site, and location maps.
   ii. Roadway profile.
   iii. Photographs.
   iv. Field reviews.
   v. Design data at nearby structures.
   vi. Gauging records.

B. Historical Data Available.
   i. Ministry of Water Resources studies.
   ii. Floodplain studies.
   iii. Catchment area studies.

C. Environmental constraints.
   i. Floodplain encroachment
   ii. Floodway designation
   iii. Fish habitat
   iv. Commitments in review documents

D. Design criteria.
   i. See Section 2.5.

Step 2 Determine the Project Scope

A. Determine level of assessment.
   i. Stability of existing channel.
   ii. Potential for damage.
   iii. Sensitivity of the stream.

B. Determine type of hydraulic analysis.
- **i.** Qualitative assessment.
- **ii.** Single-section analysis.
- **iii.** Step-backwater analysis.

C. Determine additional survey information.
- **i.** Extent of streambed profiles.
- **ii.** Locations of cross sections.
- **iii.** Elevations of flood-prone property.
- **iv.** Details of existing structures.
- **v.** Properties of bed and bank materials.

Step 3 Evaluate Hydrological Variables

A. Compute discharges for selected frequencies.

B. Hydrology - Consult Part 1 of this Manual.

Step 4 Perform Hydraulic Analysis
- **i.** Single-section analysis (2.4.1).
- **ii.** Select representative cross section .
- **iii.** Select appropriate n values (Table 2.2).
- **iv.** Compute stage-discharge relationship.

A. Step-backwater analysis.

B. Calibrate with known high water.

Step 5 Perform Stability Analysis

A. Geomorphic factors.

B. Hydraulic factors.

C. Stream response to change.

Step 6 Design Countermeasures

A. Criteria for selection.
- **i.** Erosion mechanism.
- **ii.** Stream characteristics.
- **iii.** Construction and maintenance requirements.
- **iv.** Vandalism considerations.
- **v.** Cost.

B. Types of countermeasures.
- **i.** Meander migration countermeasures.
- **ii.** Bank stabilisation.
- **iii.** Bend control countermeasures.
- **iv.** Channel braiding countermeasures.
- **v.** Degradation countermeasures.
- **vi.** Aggradation countermeasures.

C. For additional information.
- **i.** HEC-20 Stream Stability.
- **ii.** Road in the River Environment.
- **iii.** See Reference List.

Step 7 Documentation

i. Prepare report and file with background information.

2.6.2 Roadside Channels

The primary function of roadside channels is to collect surface runoff from the road and areas that drain to the right-of-way and convey the accumulated runoff to acceptable outlet points. A secondary function of a roadside channel is to drain subsurface water from the base of the roadway. This will prevent saturation and loss of support for the pavement and provide a positive outlet for subsurface drainage systems such as pipe underdrains.

The alignment, cross section, and grade of roadside channels is constrained to a large extent by the geometric and safety standards applicable to the project. These channels should accommodate the design runoff in a manner that assures the safety of motorists, and minimises future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects.

The rate of flow or discharge (Q), the depth of flow (d), and the velocity of flow (V) depend on the channel's geometry (shape and size), roughness (n), and slope (S).

In order to obtain the optimum roadside channel system design, it may be necessary to make several trials of the previous procedure before a final design is achieved.

More details on channel lining design may be found in HEC-15 including consideration of channel bends, steep slopes, and composite linings.

Step-by-Step Procedure
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.

Table 2.4 Classification of Vegetal Covers as to Degrees of Retardancy

Retardance	Cover	Condition
A	Native Grass	Excellent stand, tall >750mm
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

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Hydraulic Design of Open Channels

Table 2.5 Summary of Shear Stress for Various Protection Measures

Protective Cover	Category		τ_p (Pa)
Vegetation			
	Class A		177
	Class B		101
	Class C		48
	Class D		29
	Class E		17
Temporary			
	Woven Paper		7
	Jute Net		22
	Curled Wood Mat		74
	Synthetic Mat		96
	Gravel:	D_{50} = 25 mm	19
		D_{50} = 50 mm	38
	Rock:	D_{50} = 150 mm	120
		D_{50} = 300 mm	239
Gabions			1676
Geoweb			479
Soil Cement			>2155
Concrete construction blocks, granular filter under layer			>958
Wedge-shaped blocks with drainage slot			>1197

Table 2.6 Manning's Roughness Coefficients (HEC-15)

Lining Category	Lining Type	0 – 0.15 m	0.15 – 0.6 m	> 0.6 m
Rigid				
	Grouted Riprap	0.015	0.013	0.013
	Stone Masonry	0.040	0.03	0.028
	Soil Cement	0.042	0.032	0.03
	Asphalt	0.025	0.022	0.02
	Brick	0.018	0.016	0.016
Unlined				
	Bare Soil	0.023	0.020	0.02
	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
	Fibre Glass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.041	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap				
	25 mm D_{50}	0.044	0.033	0.034
	50 mm D_{50}	0.066	0.041	0.035
Rock Riprap				
	150 mm D_{50}	0.104	0.069	0.035
	300 mm D_{50}	--	0.078	0.04

Note: *Values listed are representative values for the respective coefficients, n, vary with the flow depth. * Some 'temporary' linings become permanent when buried.*

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.

D. Perform the layout of the proposed roadside channels to minimise diversion flow lengths.

Step 2 Obtain or Establish Cross Section Data

A. Provide channel depth adequate to drain the subbase.

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:

- **i.** Right-of-way limits.
- **ii.** Trees or environmentally-sensitive areas.
- **iii.** Utilities.
- **iv.** 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 road grades)

B. Provide a 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.

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:

- **i.** Increase bottom width,
- **ii.** Make channel side slopes flatter,
- **iii.** Make channel slope steeper,
- **iv.** Provide smoother channel lining, and
- **v.** Install drop inlets and a parallel storm drainpipe 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.

H. 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 2.4 and/or Table 2.5.

B. Estimate the flow depth and choose an initial Manning's n value from Table 2.6.

C. Calculate normal flow depth y_o (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:

$$\tau_d \text{ (Pa)} = 2990\, y_o S$$

Where, S = channel slope, m/m.

E. If $\tau_d < \tau_p$ then lining is acceptable, otherwise consider the following options:

- **i.** Choose a more resistant lining.
- **ii.** Use concrete, gabions, or other more rigid lining either as full lining or composite.
- **iii.** Decrease channel slope.
- **iv.** Decrease slope in combination with drop structures.
- **v.** Increase channel width and/or flatten side slopes.

Step 6 Analyse 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:

- **i.** Increase or decrease in discharge.
- **ii.** Increase in velocity of flow.
- **iii.** Confinement of sheet flow.
- **iv.** Change in outlet water quality.
- **v.** Diversion of flow from another catchment area.

B. Mitigate any adverse impacts identified in Step 6(A) above, possibilities include:

- **i.** Enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel.
- **ii.** Install velocity control structures.
- **iii.** Increase capacity and/or improve lining of downstream channel.
- **iv.** Install sedimentation/infiltration basins.
- **v.** Install sophisticated weirs or other outlet devices to redistribute concentrated channel flow.
- **vi.** Eliminate diversions that result in downstream damage, and which cannot be mitigated in a less expensive manner.

3 Drifts and Low-Level Crossings

3.1 General

Design flows at river crossings often substantially exceed the capacity of small to medium type culvert structures as described in Chapter 4. However, due to economic constraints, it is not always possible to construct high level bridges at these locations. Low level crossings, which also come under the name of drifts, fords, Irish crossing or vented drifts, can offer a cost-effective alternative.

It is the aim of this chapter to provide assistance to the engineer concerning:

a. The choice of type of low-level crossing;

b. The technical and economic aspects to be taken into consideration; and

c. The hydrological and hydraulic design process.

3.2 Definition and Terminology

A Low-Level River Crossings (LLRC) is a submersible road structure, designed in such way as to experience no or limited damage when overtopped. This type of structure is appropriate when the inundation of a road for short periods is acceptable.

Different names of Low-Level Crossings are used in many parts of the world, such as:

a. Low Level River Crossing.

b. Low Level Bridge.

c. Submersible Crossing/bridge.

d. Irish Crossing.

e. Causeway.

f. Vented Causeway.

g. Drift.

h. Ford.

i. Submersible bridge.

In order to clearly define the different type of structures the following classification of LLRC is proposed:

3.2.1 Drifts

A drift is defined as a specifically prepared surface for vehicles to drive over when crossing a river. A drift does not contain any openings underneath the surface for allowing passing water through. The surface layer may consist of gravel, concrete, grouted stone concrete blocks held together longitudinally with polyester, galvanised steel or stainless-steel cables. Drifts are also referred to in the literature as Fords.

3.2.2 Causeway

A vented causeway (also referred to as a causeway) in essence also consists of a suitable surface layer over which vehicles may drive but contains openings underneath allowing water to pass through the structure.

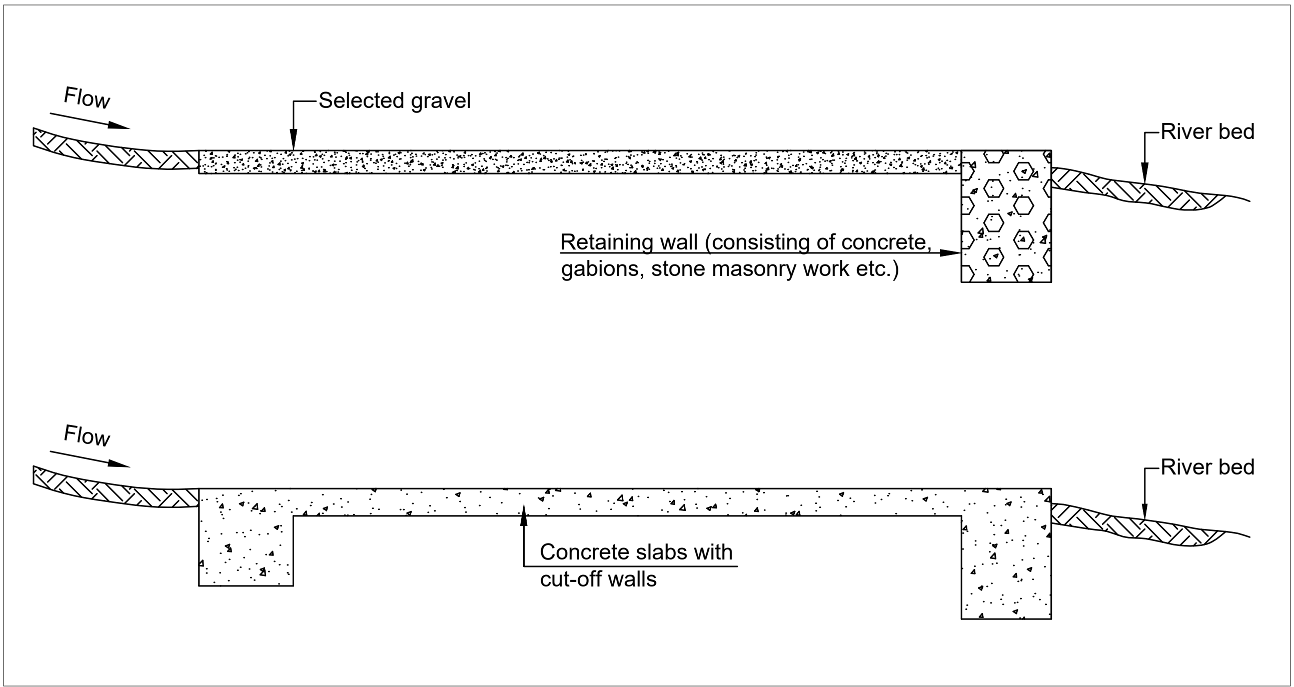
These openings may be of circular or rectangular shape and can be formed by means of pre-cast pipes or portal culverts, corrugated iron void formers, short spun decks etc. Vented causeways are also referred to in the literature as Vented Fords.

Vented causeways with several openings under the actual roadway could have the following functions:

a. To reduce the water level upstream of the structure.

b. To raise the tailwater level, so less embankment protection is required on the downstream side.

Figure 3.1 Example of Drifts

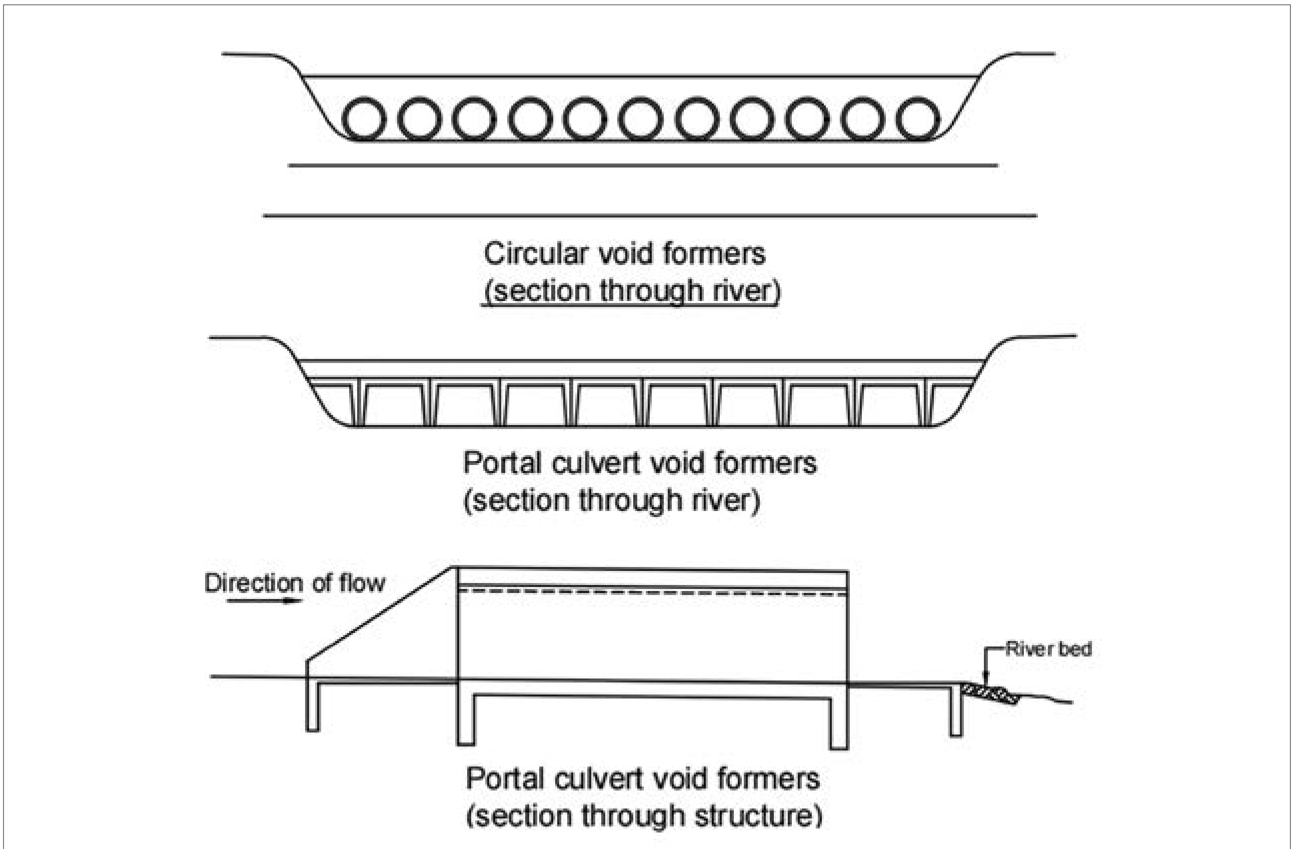


c. To act as anti – ponding structures.

3.2.3 Submersible Bridges

A submersible bridge is defined as a structure consisting of a short – span deck (typically between 4

Figure 3.2 Example of vented causeways



and 7.5 m) supported by a sub structure consisting of two abutments and any number of piers. The height of the deck above the riverbed is usually less than 2m.

3.3 Application Characteristics

3.3.1 Basic Characteristics

1. LLRC are designed to be inundated from time to time.
2. Construction costs are considerably lower than those for a conventional bridge.

LLRC are appropriate under the following circumstances:

1. When short-term disruption of traffic is acceptable.
2. When alternative routes exist which can be taken during flood periods.
3. Where high level crossings are not economically justified.

The following limitations of LLRC have to be taken into consideration:

1. The fact that the crossing might not be usable from time to time.
2. The risk that drivers might try to use the crossings during flood flow and of being washed away.
3. The level of maintenance, which might be required after flooding events.

3.3.2 Road Network Considerations

If a particular community has only one access road and the access road crosses a river without a structure, the decision whether to construct a LLRC or a high – level bridge depends on the acceptability of short periods of inaccessibility, the construction costs and the economic justification of the options.

With large rivers, attention should be paid to the total road network in the area, the number and locations of river crossing structures, as well as the levels of these structures in terms of design return period. Rather than designing all river crossing structures for the same return period, variations in the return periods used for design can be considered. In this way the number of accessible structures during flooding will be reduced, whilst alternatives remain available. In contrast with the first option a situation may occur where all the structures under consideration are overtopped at the same time.

3.4 Design Considerations

3.4.1 Site Selection

As with all river crossing structures LLRC should be located within a straight section of the river where the river flow is as uniform as possible. Riverbanks on the outside of bends tend to erode which might lead to the floodwater by-passing the structure during flooding.

Where the width of a river channel varies, the advantages of locating the structure in a narrower section should be compared to those associated with location in a wider section. Benefits of a narrower section are shorter length and, therefore, lower construction costs. Benefits of narrower sections also cover the possibility that the narrower section is associated with less weathered in-situ material, which may offer better foundation conditions.

Benefits of a wider section in the river are that flow velocity is relatively low with shallower depth. These two benefits reduce the risk that the structure may be damaged and increases the safety of vehicles crossing the structure.

Crossing the river at a skew should be avoided. A skew approach, coupled with the possible blocking of opening with debris tends to direct the full force of the river towards on of the riverbanks, which

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3
Drifts and Low-Level Crossings

increases the possibility of the approach being washed away.

The structure should be straight. A horizontally curved structure will be subject to similar problems of undesirable concentration of flow.

3.4.2 Hydrological Considerations

Design flow return periods are to be chosen from Table 5.1 in Chapter 5 of this Manual.

The recommended methods for design flood calculations are explained in Chapters 4 and 5 of this Manual. Generally, is it not required to achieve high accuracy of flood water estimation as the structure is designed to occasionally be overtopped.

In many cases it is thus sufficient to apply the Rational Method for flood calculation. If the theoretical submergence period is of concern, the TRRL Method must be applied in order to estimate the duration of flood flow above a certain flood level, associated with a specific flood return period.

3.4.3 Hydraulic Design

In this section the design procedure for both, standard drifts and vented drifts are discussed.

The capacity of a structure is determined as the sum of the discharge that could be accommodated over the structure within acceptable depth, and the discharge to be accommodated underneath the structure. The sum is then compared to the design discharge, Q_{design}, in order to evaluate the adequacy of the structure.

a. Flow Over the Structure

Decide on the maximum flow depth over the structure through which a vehicle will still be able to pass safely (100 mm) for supercritical flow due to the high momentum transfer associated with the velocities, and 150 mm for subcritical flow over the structure.

Determine the discharge that could be accommodated over the structure. As a first assumption, especially if the slope in the direction of flow is 2 to 3% as recommended elsewhere, assume this flow to be supercritical. For supercritical channel flow over the structure:

$$Q_{over} = \frac{A_{over}{}^{5/3} * S_o{}^{0.5}}{n_{conc}\, P_{over}{}^{2/3}}$$

Equation 3.1

Where,

Q_{over} = The discharge that could be accommodated over the structure within the selected flow depth (m³/s).

A_{over} = Area of flow over structure at the flow depth selected m².

S_o = Slope in direction of flow, for example 0.02 or 0.03 m/m.

n_{conc} = Manning n-value for a concrete deck, n_{conc} can be taken as 0.016 s/m$\frac{1}{3}$.

P_{over} = Wetted perimeter at the flow depth selected (m).

A_{over} and P_{over} are calculated as follows:

$$A_{over} = A_1 + A_2 + A_3 \text{ or } A_{over} = \frac{1}{3} d\sqrt{800K_1}\, d + dL_2 + \frac{1}{3}\, d\sqrt{800K_3}\, d$$

Equation 3.2

$$P_{over} = P_1 + P_2 + P_3 \text{ or } P_{over} = \frac{1}{2}\sqrt{800K_1}\, d + L_2 + \frac{1}{2}\sqrt{800K_3}\, d$$

Equation 3.3

Where,

Ai, A_2, A_3 = areas defined in Figure 3.3.
d = depth of flow over the structure (m).
K_1 = the K value for vertical curve 1.
K_3 = the K value for vertical curve 3.

With K being a vertical road alignment parameter, defined as the horizontal length of road required for a 1 % change in the gradient of the road.

Figure 3.3 Cross Section Through River at Drift

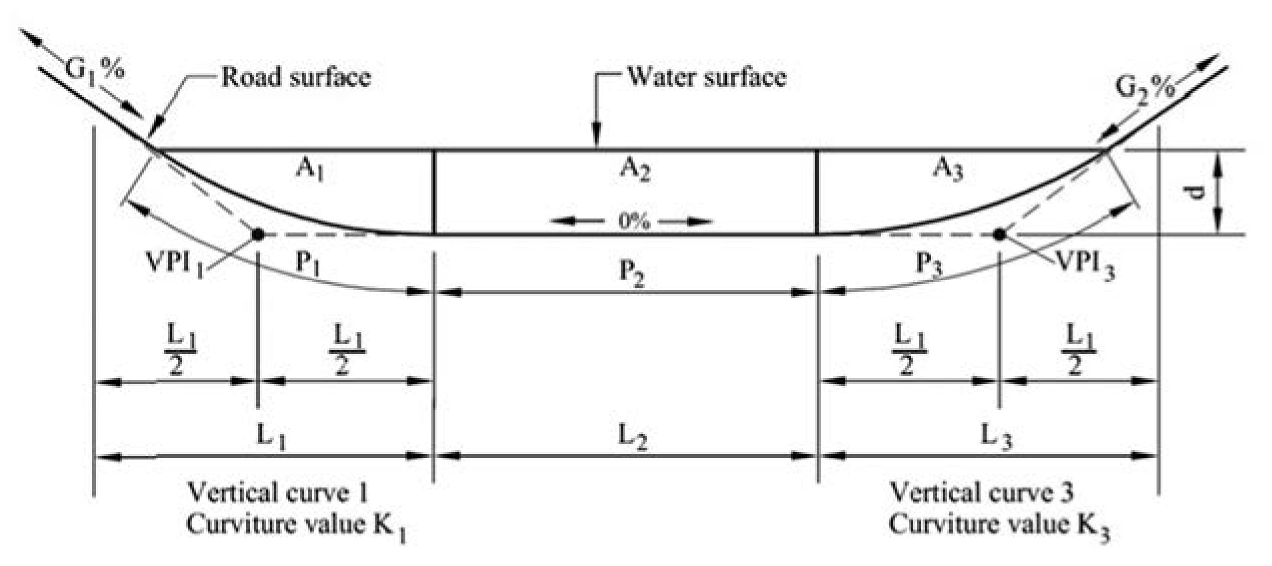
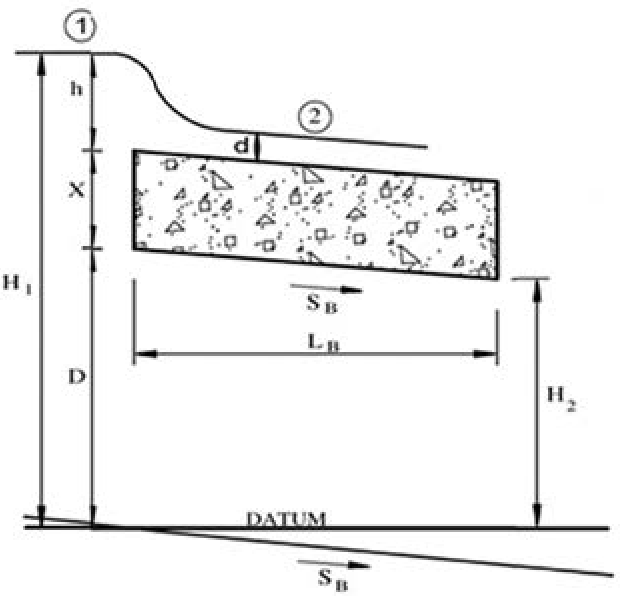


Figure 3.4 Cross Section Through Structure at Drift



The vertical road alignment, K (K_1 and K_3) should not be confused with K_{Inl} and K_{out} following below. The symbol K is used because it is the symbol used in vertical road design methodology.

Note: *In the calculation of the flow over the structure the effect of guide-blocks are for simplicity reasons ignored.*

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Drifts and Low-Level Crossings

b. Flow Through the Structure

Flow under the structure is in essence flow through a culvert opening as discussed in Chapter 4. However, as the bridge deck represents an obstacle to flow over the structure a slightly adapted way of culvert calculation is recommended for the flow through the structure.

Assume outlet control

The flow passing through the structure is defined as Q_{under},

Where,

$$Q_{under} = V_{under} * A_{eff}$$

Equation 3.4

$$V_{under} = \sqrt{\frac{H_1 - H_2}{\frac{C}{2g} + \frac{n_{eff}^2 L_B}{R^{4/3}}}}$$

Equation 3.5

Where,

A_{eff} = The effective inlet area through the structure (m²).

ΣA_{cell} = The effective inlet area through the structure.

L_B = The total width of the deck of the structure (m).

V_{under} = The average velocity of flow through the structure (m/s).

C = Factor that reflects the transition losses.

Determine the total energy height (H_j) upstream of the structure and the water level at the outlet of the structure:

Assumption: Since the water is dammed by the structure, the velocity v_1 = 0 m/s

$$H_1 = h + x + D$$

Equation 3.6

Where,

X = The thickness of the deck (depending on the structural design) (m).

D = The height of the soffit of the deck above the river invert level (m).

By applying the conservation of energy principle, determine the depth upstream of the structure, h, that is required to pass the flow rate, Q_{over}:

$$h = \frac{V_2^2}{2g} + d$$

$$\text{With } V_2 = \frac{Q_{over}}{A_{over}}$$

$$H_2 = D - L_b S_o$$

Equation 3.7

Where,

L_B = the total width of the deck of the structure (m).

S_o = slope of the conduit underneath the structure (m/m)

C is a factor representing the local or transition losses due to flow convergence/divergence at the inlet/outlet:

$$C = \sum (K_{inl.} + K_{outl.})_{each\ cell}$$

Equation 3.8

K_{inl} and K_{outl}, are determined as followed for rectangular sections:

K_{inl}, at outlet	sudden transition K_{inl}	= 0.5
	gradual transition K_{inl}	= 0.25
K_{outl} at outlet control	sudden transition K_{outl}	= 1.0
	gradual transition K_{outl}	= 1.0 for 45° < θ < 80°
		= 0.7 for θ = 300
		= 0.2 for θ = 150

Where,

θ = The diversion angle

n_{eff} = The effective Manning n-value for flow through the structure

$$n_{eff} = \sum \frac{n_{cell} * P_{cell}}{P_{eff}}$$

Equation 3.9

and

$$n_{cell} = \frac{P_{concrete} * n_{concrete}}{P_{cell}} + \frac{P_{river} * n_{river}}{P_{cell}}$$

Equation 3.10

Where,

$P_{concrete}$ = The part of the wetted perimeter that has concrete surface per cell (m).

P_{river} = The part of the wetted perimeter that is made up by the riverbed per cell (m).

P_{cell} = The total wetted perimeter of each cell (m).

P_{eff} = Effective wetted perimeter for the flow passing through the structure (m).

$n_{concrete}$ = The Manning coefficient of concrete (s/m$^{1/3}$) = 0.016/m$^{1/3}$.

n_{river} = The Manning roughness coefficient of the riverbed (s/m$^{1/3}$).

= 0.03 s/ml/^{3}

$$R = \frac{A_{eff}}{P_{eff}} \text{ (hydraulic radius (m))}$$

Equation 3.11

c. Total Flow

The total discharge capacity needs to exceed the design discharge:

Determine if $Q_{over} + Q_{under} > Q_{design}$

If so, the capacity of the structure meets the design capacity. If not, the design height or the length of the structure would have to be increased, and the flow checked again.

A worked example is given in Appendix A.4.

4 Hydraulic Design of Culverts

4.1 Types and Shapes

A culvert is a conduit which conveys stream flow through a roadway embankment or past some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life. Culvert may be described either by their shapes, material and functions.

Typically, several shapes provide hydraulically adequate design alternatives:

a. **Circular** – The most common shape used for culverts, this shape is available in various strengths and sizes. The need for cast-in-place construction is generally limited to culvert end treatments and appurtenances.

b. **Pipe-arch and elliptical** – Generally used in lieu of circular pipe where there is limited cover or overfill, structural strength characteristics usually limit the height of fill over these shapes when the major axis of the elliptical shape is laid in the horizontal plane. These shapes are typically more expensive than circular shapes for equal hydraulic capacity.

c. **Box (or rectangular)** – A rectangular culvert lends itself more readily than other shapes to low allowable headwater situations. The height may be lowered and the span increased to satisfy hydraulic capacity with a low headwater. In addition, multiple barrel box culverts accommodate large flow rates with a low profile.

d. **Modified box** – Economical under certain construction situations, the longer construction time required for cast-in-place boxes can be an important consideration in the selection of this type of culvert. Pre-cast concrete and metal box sections have been used to overcome this disadvantage.

e. **Arch** – Arch culverts span a stream using the natural streambed as the bottom. As a result, arch culverts serve well in situations where the designer wishes to maintain the natural stream bottom for reasons such as fish passage. Nevertheless, the scour potential and the structural stability of the streambed must be carefully evaluated. Structural plate metal arches are limited to use in low cover situations.

The terrain often dictates the need for a low profile due to limited fill height or potential debris clogging. The following absolute minimum sizes shall be used to avoid maintenance problems and clogging:

1. **Urban areas:** Cross drainage minimum, 450 mm
2. **Rural areas:** Cross drainage minimum, 900 mm, 600 mm for access culverts.

Land use requirements can dictate a larger or different barrel geometry than required for hydraulic considerations.

4.1.1 Embedded and Open-Bottom Culverts

An embedded culvert can be any shape, but is most often a circular, box or pipe arch that has been buried into the ground typically 20 - 40% of its height. An open-bottom culvert is often a box or arch shape built on a vertical wall foundation.

4.1.2 Low Water Crossings (Drifts)

Low water crossings may either be vented or unvented. A vented crossing has a hydraulic opening beneath the road for low flows, while an unvented crossing has no opening. A vented low-water crossing is essentially a culvert sized to allow low flow passage but overtops the roadway at high flows. Refer to Chapter 3 of this Manual for detailed information on low water crossings and drifts.

4.1.3 Long Span Culverts

Long span culverts are better defined based on structural design aspects than on the basis of hydraulic considerations. Special shapes include vertical and horizontal ellipses, underpasses, and low- and high-profile arches. Generally, the spans of long span culverts range from 7m to 14m. (In areas of high fill, length of culverts may exceed these measurements).

4.1.4 Materials

The most commonly used culvert materials are concrete (both reinforced and non-reinforced), corrugated metal (aluminium or steel) and plastic (high-density polyethylene (HDPE) or polyvinyl chloride (PVC). Less commonly used materials include clay, stone and wood, as might be found in historic culvert structures. Materials for culverts continue to be developed, and in the future could include various types of plastics, fiberglass, and composite materials. Culverts may also be lined with other materials to inhibit corrosion and abrasion, or to reduce hydraulic resistance. For example, corrugated metal culverts may be lined with asphaltic concrete or a polymer material. Culverts may also be lined with other materials to inhibit corrosion and abrasion, or to reduce hydraulic resistance.

Some commonly used combinations materials are as follows:

1. **Pipe** (concrete, steel, aluminium, plastic): circular or pipe-arch and elliptical (CMP only).
2. **Structural-plate** (steel or aluminium): circular, pipe-arch, elliptical, or arch.
3. **Box** (or rectangular) (single or multiple barrel boxes or multiple boxes): concrete box culvert or steel or aluminium box culvert.
4. **Long span** (structural-plate, steel or aluminium): low-profile arch, high profile arch, elliptical, or pear.

4.1.5 Inlets and Outlets

Different inlet and outlet configurations are utilised on culvert barrels. Commonly used inlet configurations include projecting culvert barrels, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitred to conform to the fill slope. Hydraulic performance, structural stability, aesthetics, erosion control, and fill retention are considerations in the selection of various inlet configurations. The designer should bear the following in mind in developing his design:

1. **Projecting Inlets or Outlets:** These are box or pipe culvert extensions beyond the embankment of the roadway. They have low construction cost, eliminating headwalls and wingwalls. However, they are susceptible to damage during roadway maintenance. They are also considered unsafe. They have poor inlet hydraulic efficiency, and they may require anchoring of the inlet to strengthen the weak leading edge. They are recommended for only low standard roads,(or rectangular) (single or multiple barrel boxes or multiple boxes): concrete box culvert or steel or aluminium box culvert.
2. **End Walls with Bevels:** They are used with metal pipe, whereby the pipe is cut to conform to the side-slope. They improve the inlet coefficient. They provide embankment stability, embankment erosion protection, and protection from buoyancy. They shorten the required structure length and reduce maintenance damage.
3. **End walls:** They provide embankment stability, embankment erosion protection, and protection from buoyancy. They shorten the required structure length and reduce maintenance damage.

   Type A. Conveys the flow away and perpendicular to the roadway.

   Type B. Conveys the flow into longitudinal drains parallel to the roadway.

4. **Improved Inlets:** These may be considered for culverts that will operate in inlet control. They can increase the hydraulic performance of the culvert but may also add to the total culvert cost. Therefore, they are usually beneficial only for long culverts under multi-lane highways with medians.

   Type C: Drop-type inlet increases performance, (or rectangular) (single or multiple barrel boxes or multiple boxes): concrete box culvert or steel or aluminium box culvert.

   Kerb Drop Inlet: For use in urban sections where kerbs and/or kerbs and gutters are prevalent.

5. **Wingwalls:** These are used to retain the roadway embankment to avoid a projecting culvert barrel. They are also used where the side slopes of the channel are unstable, and where the culvert is skewed to the normal channel flow. They can affect hydraulic efficiency if the flare angle is < 30 or > 60.
6. **Aprons:** They are used to reduce scour from high headwater depths or from approach velocity in the channel. They should extend at least 1.5 pipe diameters upstream, and should not protrude above the normal streambed elevation.
7. **Outlet Protection:** In general scour holes at culvert outlets provide efficient energy dissipators. Outlet protection for the selected culvert design flood shall be provided where the outlet scour hole depth computations indicate:
   a. The scour hole will undermine the culvert outlet.
   b. The expected scour hole may cause costly property damage.
   c. Scour hole causes a nuisance effect (most common in urban areas).
8. **Safety Considerations:** Culvert ends sometimes present a hazard to traffic that runs off the road increasing the severity of the accident and injuries. It may be advisable to provide protection in high hazard locations.

4.2 Design Criteria and Considerations

Culvert's basic function is to provide cross drainage for a stream channel. Other hydraulic functions include floodplain relief, where a culvert might be placed in the over-bank of a wide floodplain to provide drainage of the over-bank area during large flood events. Smaller culvert structures often function to provide ditch relief for drainage ditches along a roadway, diverting some of the discharge from the ditch.

4.2.1 Waterway Data

The installation of a culvert often changes channel and waterway conditions upstream and downstream. Accurate pre-construction waterway data must be collected in order to predict these changes. These data must include:

a. **Cross Section:** Stream cross sectional data acquired from a field survey at the site are highly desirable to supplement available topographic mapping. Ideally, a minimum of two cross sections should be taken, one upstream and one downstream.
b. **Slope:** The longitudinal slope of the existing channel in the vicinity of the proposed culvert should be defined in order to properly position the culvert in vertical profile and to define flow characteristics in the natural stream.
c. **Resistance:** The hydraulic resistance coefficient of the natural channel must be evaluated in order to calculate channel flow conditions.
d. **Channel Stability and Sediment Transport:** Evidence of existing channel instability, both laterally and vertically, should be documented and considered in the design of a culvert crossing. This can include observations of eroding bank-lines, channel shifting, scour holes, and sediment depositional areas.

e. **Tailwater:** Culvert performance is likely to be affected by the downstream water surface elevation or tailwater. Defining the tailwater condition is an open channel flow calculation procedure and is an important initial step in designing any culvert.

f. **Upstream Storage:** The storage capacity available upstream from a culvert may have an impact upon its design. Upstream storage capacity can be obtained from large scale contour maps of the upstream area, but a 0.5 m contour interval map is desirable.

g. **Culvert Skew:** the culvert skew shall not exceed 45 degrees as measured from a line perpendicular to the roadway centreline.

Photographs of site conditions are often beneficial.

4.2.2 Roadway Data

The proposed or existing roadway affects the culvert cost, hydraulic capacity, and alignment. The data required must include:

a. **Cross Section:** The roadway cross section normal to the centreline is typically available from highway plans.

b. **Culvert Length:** The estimated dimensions are obtained by superimposing the proposed culvert barrel on the roadway cross section and the stream-bed profile. This establishes the inlet and outlet invert elevations

c. **Longitudinal Roadway Profile:** For cross drainage culverts the roadway profile represents an obstruction encountered by the flowing stream. The profile contained in road plans generally represents the roadway centreline profile.

4.2.3 Cultural and Historical Sites

In some locations, cultural or historic sites may impact culvert design. This could be the result of a cultural or historic site in the vicinity of a culvert, or it could be that the culvert appurtenances, typically headwalls and wing-walls, are classified as historic. The Kayas of the Miji Kenda of the coast is a typical example of a cultural site.

4.2.4 Economics

The design of a culvert installation should always include an economic evaluation. The benefits of constructing a large capacity culvert to accommodate all of these events with no detrimental flooding effects are normally outweighed by the initial construction costs. Thus, an economic analysis of the trade-offs is necessary. For example, although metal pipe culverts are usually less expensive than concrete pipe culverts, a cost estimate may indicate that this is not the case. There are local concrete pipe culvert manufacturers producing pipes of varying quality; presently all metal pipes need to be imported.

Evaluating process shall be based on a comparison of the total cost of alternate materials over the design life of the structure. The analysis should consider:

1. Durability (service life).
2. Cost and availability.
3. Ease of construction and maintenance.
4. Structural strength.
5. Effect on traffic flows.
6. Abrasion and corrosion resistance.
7. Water tightness requirements.

4.2.5 Risk Analysis

Risk analysis should be performed for large culvert installations or for locations with high potential for flood damages. The objective of the risk analysis is to find the optimum culvert capacity based on a comparison of benefits and costs.

4.2.6 Debris Control

Experience and evidence indicate that watercourse will transport a heavy volume of controllable debris in the following areas.

1. For culverts located in mountainous or steep regions.
2. For culverts that are under high fills.
3. Where clean out access is limited.

In such cases, access must be available to clean out the debris control device.

4.2.7 Allowable Headwater

Allowable Headwater (AHW) elevation based on the selected Flood Frequency and headwater considerations. Existing field conditions, channel geometry, and culvert hydraulics will control the allowable headwater depth. The stream terrain upstream of the culvert should be considered when determining AHW. In selecting an Allowable Headwater Elevation (AHW), the following should be considered:

1. Damage to culvert and roadway.
2. Traffic interruption due to culvert or roadway damage.
3. Hazard to human life and safety.
4. Allowable Headwater (AHW) / Culvert Depth (D) Ratio.
5. Low point in the roadway grade line.
6. Roadway elevation above structure.
7. Elevation at which flood waters will flow to adjacent water courses.
8. Stability of the roadway embankment above the culvert.
9. Current and future land use that may be impacted.

The design headwater should not exceed the allowable headwater. If it does, additional culvert configurations must be selected and evaluated until a configuration is found that produces a design headwater equal to or less than the allowable headwater.

4.2.8 Multiple Barrels

Multiple barrel culverts should fit within the natural dominant channel with only minor widening of the channel to avoid conveyance loss through sediment deposition in some of the barrels. When the approach flow is supercritical, either a single barrel or special inlet treatment is required to avoid adverse hydraulic jump effects. It is good practice to install one barrel at the flow line of the stream while other barrels are set slightly higher to reduce sedimentation.

4.2.9 Stream Stability Assessment

A stream stability assessment is critical to the long-term performance of a culvert. A stream stability assessment can provide insight and understanding of potential problems. Natural channel systems are very dynamic and always changing. At a minimum before any culvert is designed, the designer should visually confirm that the stream reach is both vertically and horizontally stable. For detailed stream stability assessment see Chapter 6 of this Manual.

4.2.10 Peak Design Flow

Large and expensive culvert installations may warrant extensive hydrological analysis. The magnitude of the peak flow is dependent upon the selection of a return period. The assignment of a return period is generally based on the importance of the roadway and flood damage potential. Culvert operation should be evaluated for flows other than the peak design flow because:

1. It is good design practice to check culvert performance through a range of discharges to determine acceptable operating conditions.
2. Regulations may require analysis at a larger discharge than that used to design the culvert, such as the 50-year discharge commonly used to define the regulatory floodplain
3. In performing flood risk analyses, estimates of the damages caused by headwater levels due to floods of various frequencies are required

For guidelines on choice of design peak flow refer to Chapter 4 of Part 1 of this Manual. The guidelines for culverts are reproduced in Table 4.1 below for ease of reference:

Table 4.1 Guidelines on Design Flood Frequencies (Return Periods) for Culverts

No.	Structure Type	Design Flood Frequency (years)	Check Frequency (years)
1	**Pipe Culverts**	10	25
2	**Box Culverts:** Area of opening/s < 6 m^2	25	50
3	**Box Culverts:** Area of opening/s > 6 m^2	50	100

4.3 Inlet and Outlet Control

An exact theoretical analysis of culvert flow is extremely complex because the following is required:

1. Analysing non-uniform flow with regions of both gradually varying and rapidly varying flow;
2. Determining how the flow type changes as the flow rate and tailwater elevations change;
3. Applying backwater and draw-down calculations, energy, and momentum balance;
4. Applying the results of hydraulic model studies; and
5. Determining if hydraulic jumps occur and if they are inside or downstream of the culvert barrel.

The procedures in this chapter use the following:

1. **Control Section** − The location where there is a unique relationship between the flow rate and the upstream water surface elevation. Inlet control is governed by the inlet geometry. Outlet control is governed by a combination of the culvert inlet geometry, the barrel characteristics, and the tailwater.
2. **Minimum Performance** − Is assumed by analysing both inlet and outlet control and using the highest headwater. The culvert may operate more efficiently at times (more flow for a given headwater level), but it will not operate at a lower level of performance than calculated.

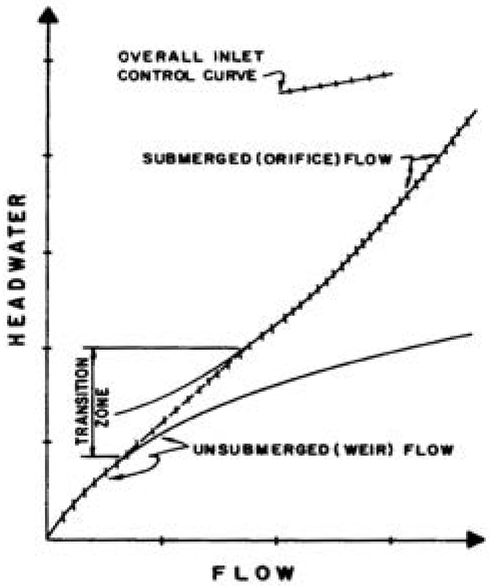
4.3.1 Inlet Control

For inlet control, the control section is at the upstream end of the barrel (the inlet). The flow passes through critical depth near the inlet and becomes shallow, high velocity (supercritical) flow in the culvert barrel. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.

a. **Headwater Factors** − Headwater depth is measured from the inlet invert of the inlet control section to the surface of the upstream pool. The inlet area is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area. Inlet edge configuration describes the entrance type. Some typical inlet edge configurations include thin edge projecting, mitred edges, square edges in a headwall, and bevelled edges. Inlet shape is usually the same as the shape of the culvert barrel. Typical shapes are rectangular, circular, elliptical, and arch. Check for an additional control section, if different from the barrel.

b. **Hydraulics** − Three regions of flow are shown in Figure 4.1 below: un-submerged, transition, and submerged:

Figure 4.1 Inlet Control Curves



c. **Inlet Control Curves and Design Equations**

- Unsubmerged

$$\text{Form (1)}\ \frac{HW_1}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M - 0.5S^2$$

Equation 4.1

$$\text{Form (2)}\ \frac{HW_1}{D} = K\left[\frac{K_u Q}{AD^{0.5}}\right]^M$$

Equation 4.2

- Submerged

$$\frac{HW_1}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M + Y - 0.5S^2$$

Equation 4.3

Definitions

HW_1 = Headwaters depth above inlet control section invert, m.

D = Interior height of culvert barrel, m.

H_c = Specific head at critical depth ($d_c + \frac{V_c^2}{2g}$), m.

Q = Discharge, m³/s.

A = Full cross-sectional area of the culvert barrel, m².

S = Culvert barrel slope, m/m.

K, M, c, and Y = Are constants as defined in Table 4.2.

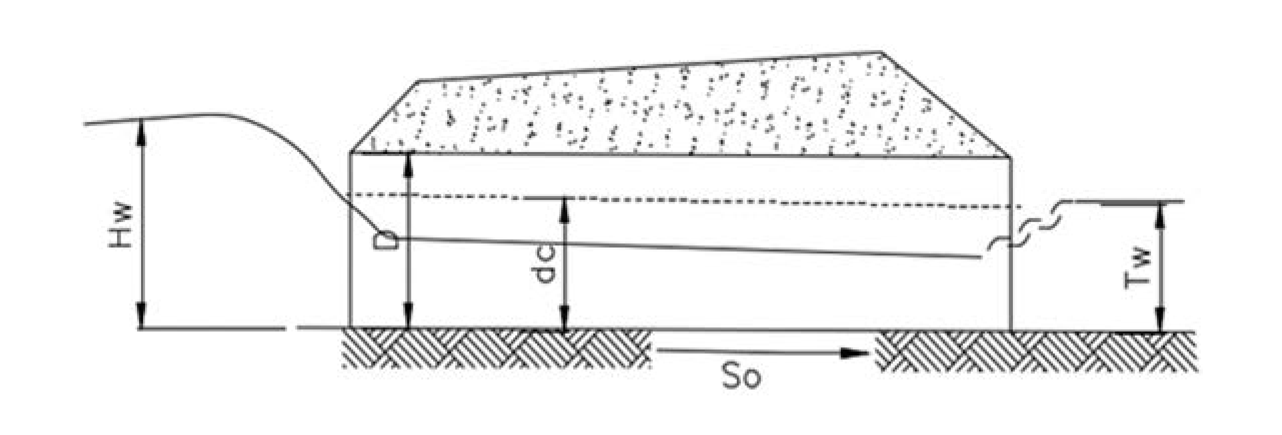
K_u = 1.811

Table 4.2 Constants for Inlet Control Design Equation

Shape and Material	Nomo-graph Scale	Inlet Edge Description	Unsubmerged		Submerged	
			K	M	c	Y
Circular Concrete	1	Square edge w/headwall	0.0096	2.000	0.0398	0.67
	2	Groove end w/headwall	0.0018	2.000	0.0292	0.74
	3	Groove end projecting	0.0045	2.000	0.0317	0.69
Circular CMP	1	Headwall	0.0078	2.000	0.0379	0.69
	2	Mitred to slope	0.0210	1.330	0.0463	0.75
	3	Projecting	0.0340	1.500	0.0553	0.54
Circular	A	Bevelled ring 45° bevels	0.0018	2.500	0.0300	0.74
	B	Bevelled ring 33.7° bevels	0.0018	2.500	0.0243	0.83
Rectangular Box	1	30° to 75° wingwall flares	0.026	1.000	0.0347	0.81
	2	90° and 15° wingwall flares	0.061	0.750	0.0400	0.80
	3	0° wingwall flares	0.061	0.750	0.0423	0.82
Rectangular Box	1	45° wingwall flare, d = 0.043D	0.510	0.667	0.0309	0.80
	2	18° to 33.7° wingwall flare, d=0.08D	0.486	0.667	0.0249	0.83
Rectangular Box	1	90° headwall w/19mm chamfers	0.515	0.667	0.0375	0.79
	2	90° headwall w/45° bevels	0.495	0.667	0.0314	0.82
	3	90° headwall w/33.7° bevels	0.486	0.667	0.0252	0.865
Rectangular Box	1	19mm chamfers; 45° skewed headwall	0.545	0.667	0.0505	0.73
	2	19mm chamfers; 30° skewed headwall	0.533	0.667	0.0425	0.705
	3	19mm chamfers; 15° skewed headwall	0.522	0.667	0.0402	0.66
	4	45° bevels; 10° - 45° skewed headwall	0.498	0.667	0.0327	0.75
Rectangular Box 19mm chamfers.	1	45° non-offset wingwall flares	0.497	0.667	0.0339	0.803
	2	18.4° non-offset wingwall flares	0.493	0.667	0.0361	0.806
	3	18.4° non-offset wingwall flares; 30° skewed barrel.	0.495	0.667	0.0386	0.71
Rectangular Box Top Bevels	1	45° wingwall flares - offset	0.497	0.667	0.302	0.835
	2	33.7° wingwall flares - offset	0.495	0.667	0.0252	0.881
	3	18.4° wingwall flares - offset	0.493	0.667	0.0227	0.887
C M Boxes	2	90° headwall	0.0083	2.000	0.0379	0.69
	3	Thick wall projecting	0.0145	1.75	0.0419	0.64
	5	Thick wall projecting	0.0340	1.500	0.0496	0.57`

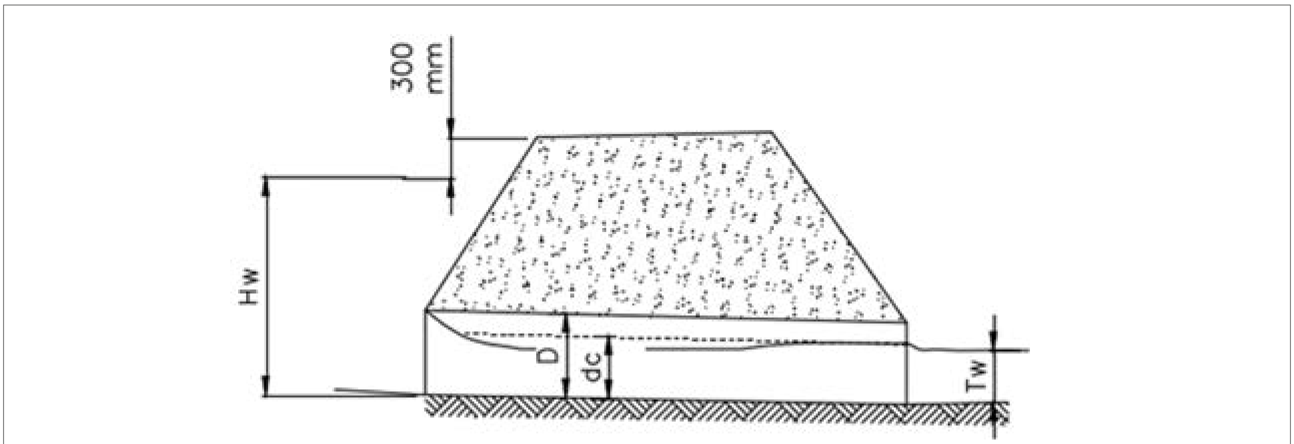
i. **Unsubmerged** − For headwater below the inlet crown, the entrance operates as a weir (see Figure 4.2). A weir is a flow control section where the upstream water surface elevation can be predicted for a given flow rate. The relationship between flow and water surface elevation can be determined by model tests of the weir geometry or by measuring prototype discharges.

Figure 4.2 Unsubmerged Flow, Type I, Inlet Control



ii. **Submerged** − For headwaters above the inlet, the culvert operates as an orifice (see Figure 4.3). An orifice is an opening, submerged on the upstream side and flowing freely on the downstream side, which functions as a control section.

Figure 4.3 Submerged Flow, Type V, Inlet Control



Use of Nominal Structures

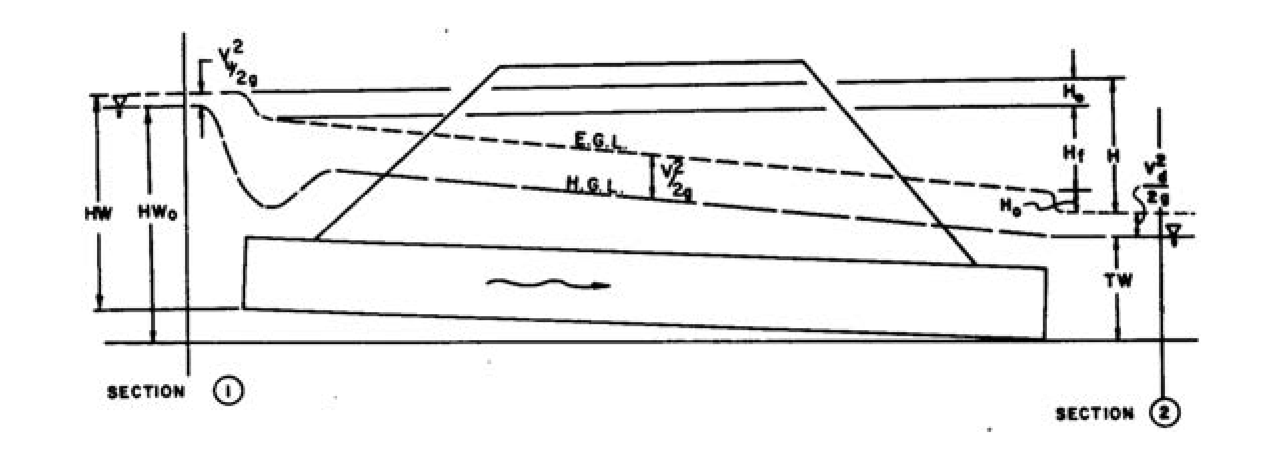
d. **Nomographs** − The inlet control flow versus headwater curves, which are established using the above procedure, are the basis for constructing the inlet control design nomographs. Note that in the inlet control nomographs, HW is measured to the total upstream energy grade line including the approach velocity head. Inlet control nomographs are shown in Charts 4.1, 4.2, and 4.3 for concrete pipe culverts, box culverts and corrugated metal culverts, respectively.

4.3.2 Outlet Control

Outlet control has depths and velocity that are subcritical. The control of the flow is at the downstream end of the culvert (the outlet). The tailwater depth is assumed to be critical depth near the culvert outlet or in the downstream channel, whichever is higher. In a given culvert, the type of flow is dependent on all of the barrel factors. All of the inlet control factors also influence culverts in outlet control. Outlet control flow is illustrated in Figure 4.4.

a. **Barrel Roughness** − a function of the material used to fabricate the barrel. Typical materials

Figure 4.4 Flow Type IV Outlet Control



include concrete and corrugated metal. The roughness is represented by a hydraulic resistance coefficient such as the Manning's '*n*' value. Typical Manning's '*n*' values are presented in Table 4.3.

b. **Barrel Area** − measured perpendicular to the flow

c. **Barrel Length**—the total culvert length from the entrance to the exit of the culvert. Because the design height of the barrel and the slope influence the actual length, an approximation of barrel length is usually necessary to begin the design process.

d. **Barrel Slope** − the actual slope of the culvert barrel and is often the same as the natural stream slope. However, when the culvert inlet or outlet is raised or lowered, the barrel slope is different from the stream slope.

e. **Transition Zone** − The transition zone is located between the unsubmerged and the submerged flow conditions where the flow is poorly-defined. This zone is approximated by plotting the unsubmerged and submerged flow equations and connecting them with a line tangent to both curves, as indicated in Figure 4.6.

f. **Tailwater Elevation** − based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation, or field observations are used to define the tailwater elevation.

g. **Hydraulics** − Full flow in the culvert barrel is assumed for the analysis of outlet control hydraulics. Outlet control flow conditions can be calculated based on an energy balance from the tailwater pool to the headwater pool.

Table 4.3 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	68 mm x 13 mm corrugations	0.022 - 0.027
Annular or Helical Pipe	150 mm x 25 mm corrugations	0.022 - 0.025
	125 mm x 25 mm corrugations	0.025 - 0.026
	75 mm x 25 mm corrugations	0.027 - 0.028
	150 mm x 50 mm structural plate	0.033 - 0.035
	230 mm x 64 mm structural plate	0.033 - 0.037
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow Spiral Rib Metal	68 mm x 13 mm corrugations	0.012 - 0.024
	Smooth Walls	0.012 - 0.013
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow Spiral Rib Metal	68 mm x 13 mm corrugations	0.012 - 0.024
Spiral Rib Metal	Smooth Walls	0.012 - 0.013

4.3.3 Design Equations and Definitions

i. Losses

$$H_L = H_e + H_f + H_v + H_b + H_j + H_g$$

Equation 4.4

Where,

H_L = Total energy loss, m.
H_e = Entrance loss, m.
H_f = Friction losses, m.
H_v = Exit loss (velocity head), m.
H_b = Bend losses, m.
H_j = Losses at junctions, m.
H_g = Losses at grates, m.

ii. Velocity

$$V = \frac{Q}{A}$$

Equation 4.5

Where,

V = Average barrel velocity, m/s.
Q = Flow rate, m^3/s.
A = Cross sectional area of flow with the barrel full, m^2.

iii. Velocity Head

$$H_v = \frac{V^2}{2g}$$

Equation 4.6

Where,

g = Acceleration due to gravity, 9.8 m/s^2.

iv. Entrance Loss

$$H_e = K_e \frac{V^2}{2g}$$

Equation 4.7

Where,

K_e = Entrance loss coefficient (see Table 4.4).

v. Friction Loss

$$H_f = \frac{19.63n^2L}{R^{1.33}} \frac{V^2}{2g}$$

Equation 4.8

Where,

n = Manning's roughness coefficient, (see Table 4.3).

L = length of the culvert barrel, m.

R = hydraulic radius of the full culvert barrel = A/P, m.

P = wetted perimeter of the barrel, m.

vi. Exit Loss

$$H_o = 1.0 \frac{V^2}{2g} - \frac{V_d^2}{2}$$

Equation 4.9

Where,

V_d = Channel velocity downstream of the culvert, m/s (usually neglected, resulting in Equation 4.10).

$$H_o = H_v = \frac{V^2}{2g}$$

Equation 4.10

vii. Barrel Losses

$$H = H_e + H_o + H_f$$

$$H = 1 + K_e + \frac{19.63n^2L}{R^{1.33}} \frac{V^2}{2g}$$

Equation 4.11

a. **Energy Grade Line** − the energy grade line represents the total energy at any point along the culvert barrel. Equating the total energy at sections 1 and 2, upstream and downstream of the culvert barrel in Figure 4.6: Flow Type VII, the following relationship results:

$$HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_L$$

Equation 4.12

Where,

HW_o = Headwater depth above the outlet invert, m.

V_u = Approach velocity, m/s.

TW = Tailwater depth above the outlet invert (m).

V_d = Downstream velocity, m/s.

H_L = Sum of all losses (Equation 4.4).

b. **Hydraulic Grade Line** − The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are parallel lines separated by the velocity head except at the inlet and the outlet.

c. **Nomographs (full flow)** − The nomographs were developed assuming that the culvert barrel is flowing full and;

   i. TW > D, Flow Type IV Outlet Control (see Figure 4.6); or

   ii. d_c > D, Flow Type VI Inlet Control (see Figure 4.7);

   iii. V_u is small and its velocity head can be considered a part of the available headwater (HW) used to convey the flow through the culvert;

   iv. V_d is small and its velocity head can be neglected;

   Equation 4.9 becomes:

$$HW = TW + H - S_oL$$

Equation 4.13

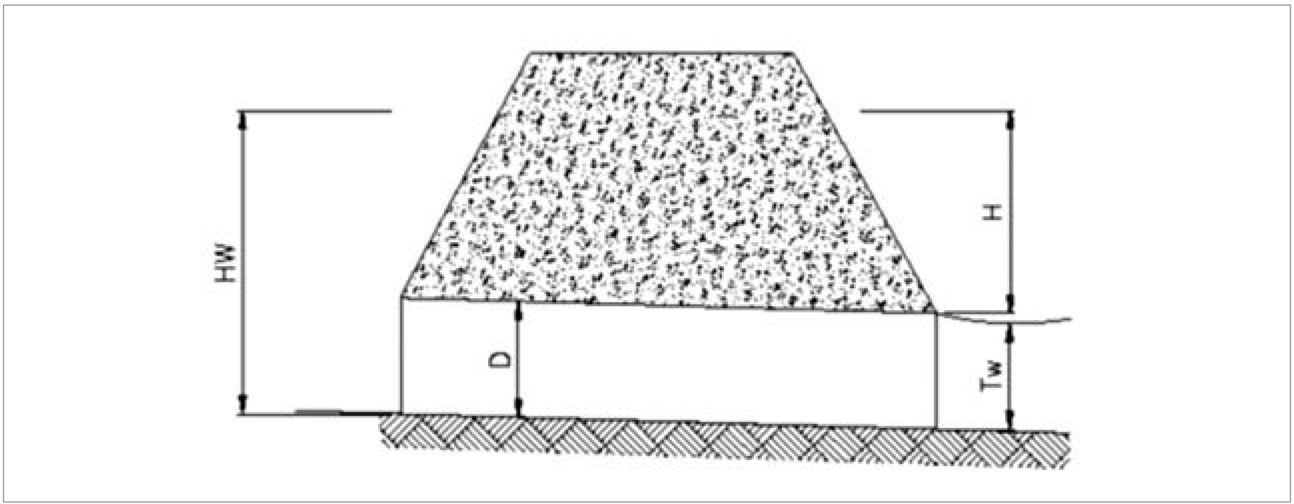
Where,

HW = Depth from the inlet invert to the energy grade line, m.

H = The value read from the nomographs (Equation 4.8), m.

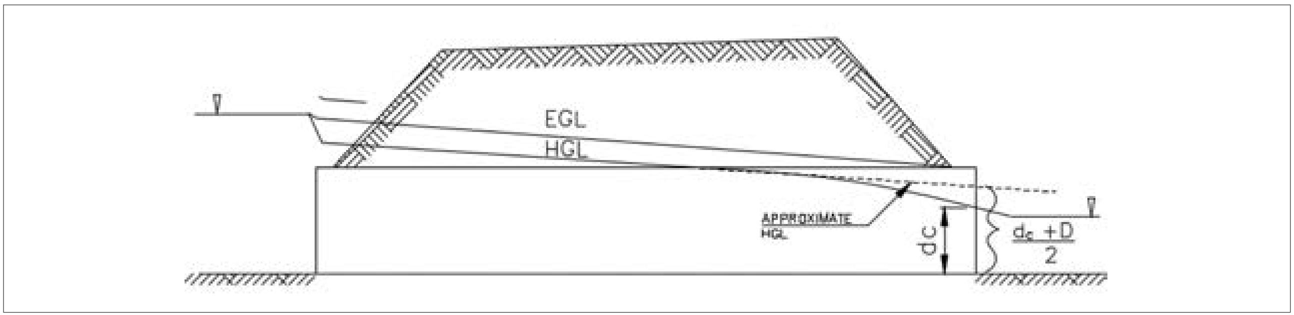
S_oL = Drop from inlet to outlet invert, m.

Figure 4.5 Submerged Pipe Flowing Full, Inlet Control, Flow Type VI



d. **Nomographs (Partly full flow)** − Equations 4.4 through 4.9 were developed for full barrel flow. The equations also apply to the flow situations which are effectively full flow conditions. if TW < d_c. (Figure 4.6)

Figure 4.6 Flow Type VII



1
2
3
4
Hydraulic Design of Culverts

i. **Backwater Calculations** – Backwater calculations may be required that begin at the downstream water surface and proceed upstream. If the depth intersects the top of the barrel, a full flow extends from that point upstream to the culvert entrance.

ii. **Approximate method** – It has been found that the hydraulic grade line pierces the plane of the culvert outlet at a point one-halfway between critical depth and the top of the barrel or $(d_c + D)/2$ above the outlet invert. TW should be used if higher than $(d_c + D)/2$. The following equation should be used:

$$HW = h_o + H - S_oL$$

Equation 4.14

Where,

h_o = the larger of TW or $(d_c + D)/2$, m

Adequate results are obtained down to a $HW = 0.75D$. For lower headwaters, backwater calculations are required. (See Figure 4.7 if $TW < d_c$ and Figure 4.8 if $TW > d_c$).

Figure 4.7 Flow Type II. $TW < d_c$

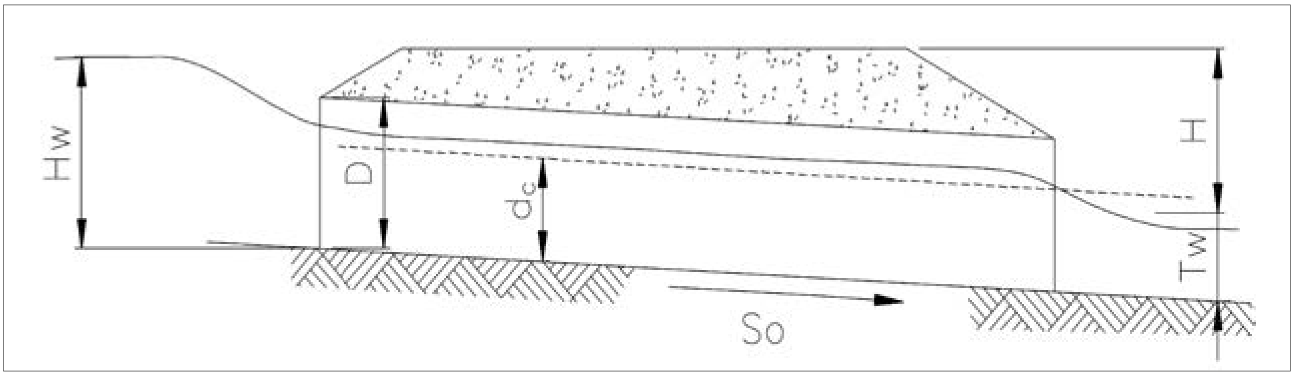
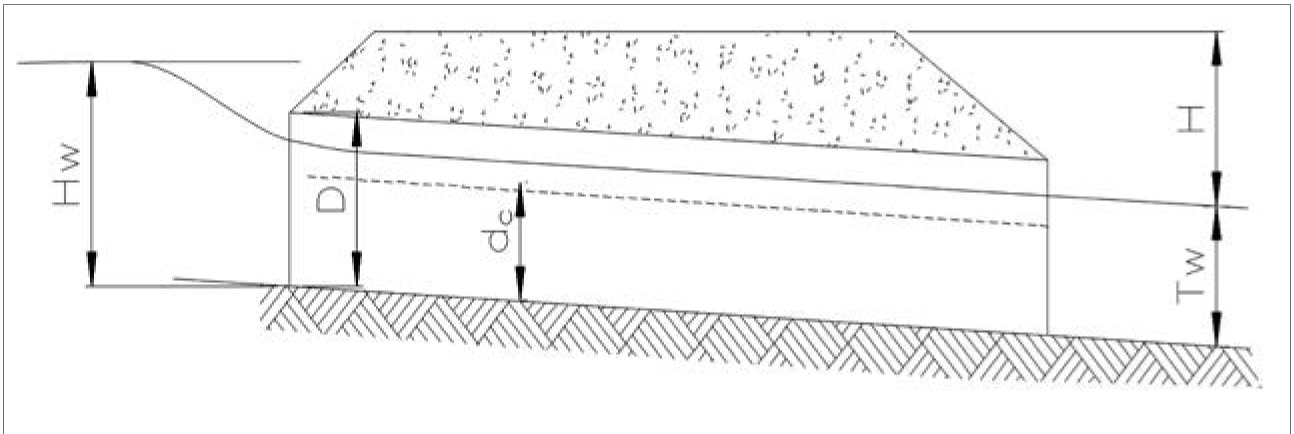


Figure 4.8 Flow Type III, $TW > d_c$



e. **Outlet Velocity** – Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit. Culverts usually give outlet velocities that are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion. If outlet erosion protection is necessary, the flow depths and Froude number may also be needed.

f. **Water Surface Profile** – If water surface profile (drawdown) calculations are necessary, begin at dc at the entrance and proceed downstream to the exit. Determine at the exit the depth and flow area. Use normal depth and velocity. This approximation may be used since the water surface profile converges towards normal depth if the culvert is of adequate length. The outlet velocity may be slightly higher than the actual velocity at the outlet.

g. **In Outlet Control** – The cross-sectional area of the flow is defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the conduit:

   i. Critical depth is used when the tailwater level is less than critical depth.

   ii. Tailwater depth is used when tailwater is greater than critical depth, but below the top of the barrel.

   iii. The total barrel area is used when the tailwater level exceeds the top of the barrel.

Table 4.4 Entrance Loss Coefficient (Outlet Control, Full or Partially Full) $H_e = K_e\,(V^2/2g)$

Type of Structure and Design of Entrance		Coefficient ke
Pipe, concrete		
Mitered to conform to fill slope		0.7
End-section conforming to fill slope*		0.5
Projecting from fill, square cut end		0.5
Headwall or headwall and wingwalls	*Square-edge*	0.5
	Rounded (radius = 1/12D)	0.2
	Rounded (radius = 1/12D)	0.2
	Socket end of pipe (groove-end)	0.2
	Projecting from fill, socket end (groove-end)	0.2
	Bevelled edges, 33.7˚ or 45˚ bevels	0.2
	Side- or slope-tapered inlet	0.2
Pipe, or pipe-arch, corrugated metal		
Projecting from fill (no headwall)		0.9
Mitered to conform to fill slope, paved or unpaved slope		0.7
Headwall or headwall and wingwalls square-edge		0.5
End-section conforming to fill slope*		0.5
Bevelled edges, 33.7˚ or 45˚ bevels		0.2
Side- or slope-tapered inlet		0.2
Box, Reinforced Concrete		
Wingwalls parallel (extension of sides) square-edged at crown		0.7
Wingwalls, 10˚ to 25˚ or 30˚ to 75˚ to barrel, square-edged at crown		0.5
Headwall parallel to embankment (no wingwalls)	*Square-edged on 3 edges*	0.5
	Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
	Bevelled edges on 3 sides	0.2
Wingwalls at 30˚ to 75˚ to barrel, crown edge rounded to radius		0.2
of 1/12 barrel dimension, or bevelled top edge		0.2
Side- or slope-tapered inlet		0.2

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4.3.4 Roadway Overtopping

Roadway overtopping will begin when the headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway.

The flow will be similar to flow over a broad crested weir.

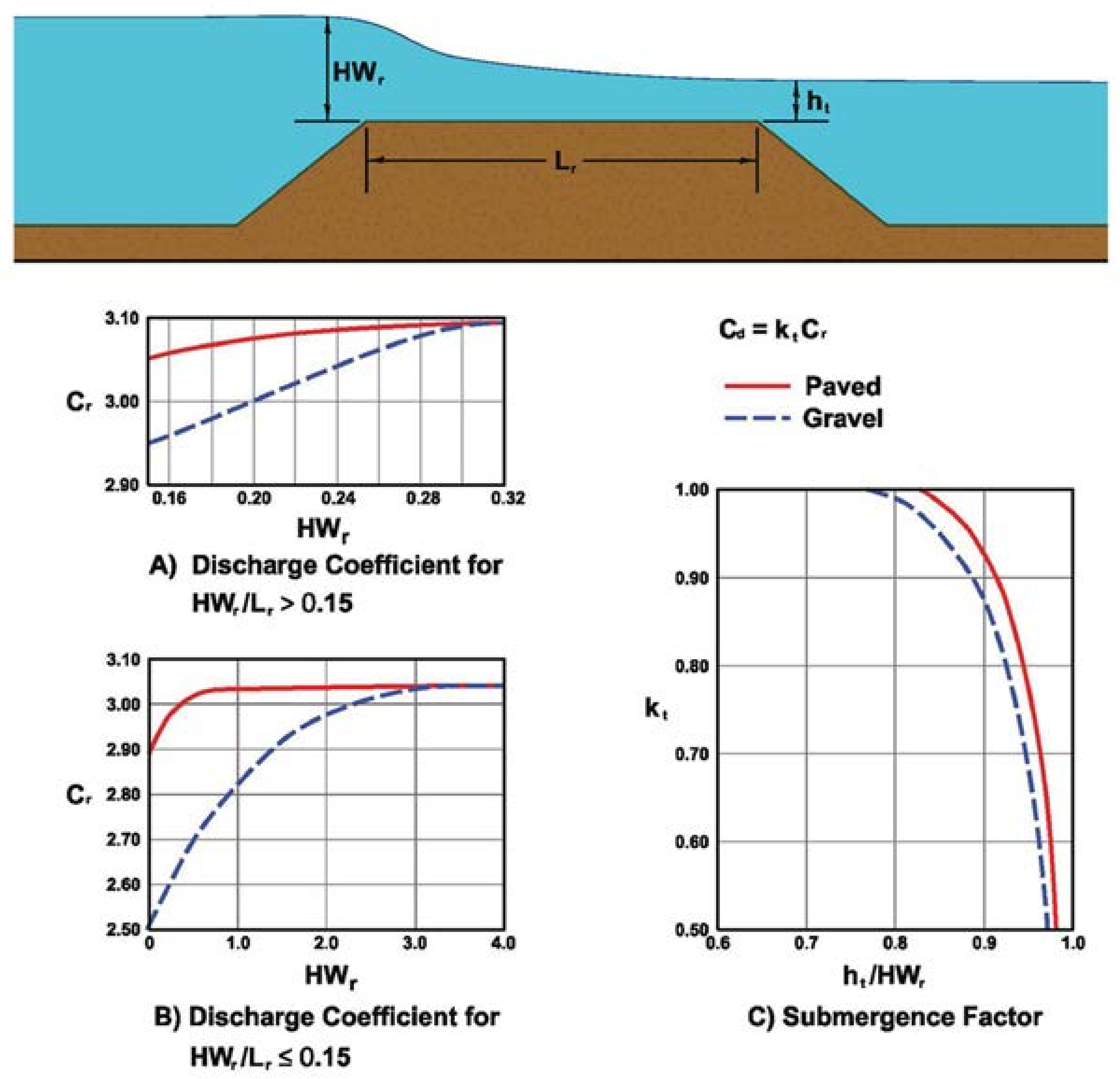
$$Q_r = C_d L HW_r^{1.5}$$

Equation 4.15

Where,

Q_r = Overtopping flow rate, m³/s.
C_d = Overtopping discharge coefficient (weir coefficient), = $k_f C_r$.
K_t = Submergence coefficient.
C_r = Coefficient of free discharge.
L = Length of the roadway crest, m.
HW_r = The upstream depth, measured above the roadway crest, m.

Figure 4.9 Discharge Coefficients For Roadway Overtopping



Source FHWA-HDS 5-2012

Total Flow − calculated for a given upstream water surface elevation using Equation 4.15. In this equation, roadway overflow plus culvert flow must equal total design flow. A trial-and-error process is necessary to determine the flow passing through the culvert and the amount flowing across the roadway. Performance curves for the culvert and the road overflow may be summed to yield an overall performance.

4.3.5 Performance Curves

A performance curve is a plot of flow rate versus headwater depth or elevation, velocity, or outlet scour. The culvert performance curve is made up of the controlling portions of the individual performance curves for each of the following control sections (See Figure 4.10).

Inlet − the inlet performance curve is developed using the inlet control nomographs (see Charts 4.1, 4.2 and 4.3).

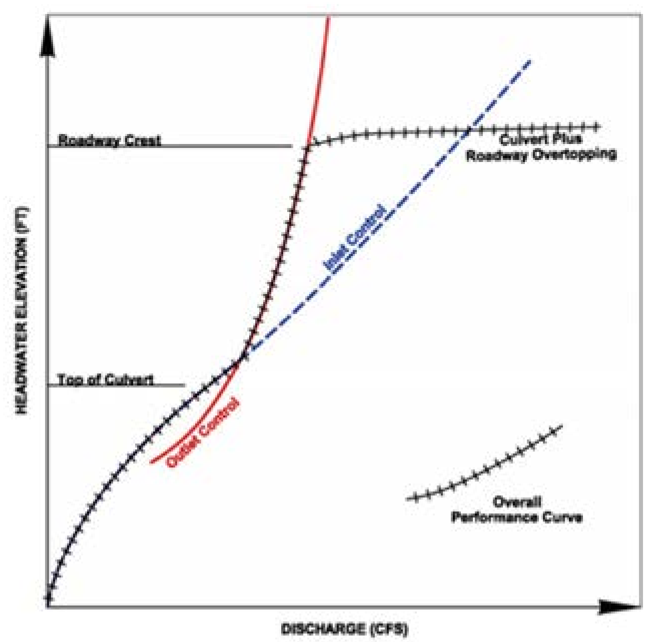
Outlet − the outlet performance curve is developed using Equations 4.4 through 4.10, the outlet control nomographs, or backwater calculations.

Roadway − roadway performance curve is developed using Equation 4.15.

Overall − the overall performance curve is the sum of the flow through the culvert and the flow across the roadway and can be determined by performing the following steps.

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters must be calculated.
2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.
3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and Equation 4.12 to calculate flow rates across the roadway
4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve as shown in Figure 4.10 below.

Figure 4.10 Overall Performance Curve



4.3.6 Design Procedure

The following design procedure provides a convenient and organised method for designing culverts for a constant discharge, considering inlet and outlet control. A sample computation form has been provided as Figure 4.11 to guide the user. It contains blocks for the project description, designer's identification, hydrological data, culvert dimensions and elevations, trial culvert description, inlet and outlet control HW, culvert barrel selected, and comments.

The overall procedure is summarised in Flowchart 4.1 at the end of this section.

Step 1: Assemble Site Data and Project File

- **a.** Data include:
    - Topographic, site, and location maps.
    - Embankment cross section.
    - Roadway profile.
    - Field visit (sediment, debris).
    - Design data of nearby structures.
- **b.** Studies by other agencies including Ministry of Water Resources.
- **c.** Environmental constraints contained in environmental review documents.
- **d.** Design criteria: Review applicable criteria, and prepare risk assessment, if necessary.

Step 2: Determine Hydrology, Minimum data required − drainage area maps and discharge-frequency plots.

Step 3: Design Downstream Channel, Minimum data are cross section of channel and the rating curve for channel.

Step 4: Summarise Data on Design Form (see Form Figure 4.11 at the end of this chapter). Use data from Steps 1-3.

Step 5: Select Design Alternative.

- **a.** See Section 4.3.
- **b.** Choose culvert material, shape, size, and entrance type.

Step 6: Select Design Discharge Q_d.

- **a.** See Section 4.4: subchapter Design Limitations.
- **b.** Determine flood frequency, (Refer to Table 4.1).
- **c.** Determine Q from plot (Step 2).
- **d.** Divide Q by the number of barrels.

Step 7: Determine Inlet Control Headwater Depth (HW_i).

Use the inlet control nomograph (Charts 4.1, 4.2 or 4.3 and Charts B.5 to B.16).

Note: *A plastic sheet with a matte finish can be used for marking such that the nomographs can be preserved.*

- **a.** Locate the size or height on the scale.
- **b.** Locate the discharge.
- **c.** Locate the size or height on the scale.
- **d.** Locate the discharge.
    - For a circular shape use discharge
    - For a box shape use Q per foot of width.
- **e.** Locate HW/D ratio using a straight-edge.
- **f.** Extend a straight line from the culvert size through the flow rate.
- **g.** Mark the first HW/D scale. Extend a horizontal line to the desired scale, read HW/D, and note on Charts.

h. Calculate headwater depth (HW)

i. Multiply HW/D by D to obtain HW to energy grade line.

ii. Neglecting the approach velocity $HW_i = HW$ including the approach velocity $HW_i = HW$ - approach velocity head.

Step 8: Determine Outlet Control Headwater Depth at Inlet (HW_{oi}).

a. Calculate the tailwater depth (TW) using the design flow rate and normal depth (single section) or using a water surface profile

b. Calculate critical depth (d_c) using appropriate chart (Charts 4-1, 4.2 or 4.3 and Charts B-5 to B-16) locate flow rate and read d_c. d_c cannot exceed D.

c. Calculate $(d_c + D)/2$

d. Determine (h_o)

$$h_o = \text{the larger of } TW \text{ or } d_c + \frac{D}{2}$$

e. Determine entrance loss coefficient (Ke) from Table 4.4.

f. Determine losses through the culvert barrel (H).

i. Use nomograph charts or equation 4.8 or 4.9 if outside range.

ii. Locate appropriate Ke scale.

iii. Locate culvert length (L) or (L_1).

iv. Use (L) if Manning's n matches the n value of the culvert and - use (L_1) to adjust for a different culvert n value.

$$L_1 = L\left(\frac{n_1}{n}\right)^2$$

Equation 4.16

Where,

L_1 = adjusted culvert length, m.

L = actual culvert length, m.

n_1 = desired Manning n value.

n = Manning n value on chart.

1. Mark point on turning line;

i. Use a straight edge; and

ii. Connect size with the length.

2. Read (H);

i. Use a straight-edge;

ii. Connect Q and turning point;

iii. Read (H) on Head Loss scale.

g. Calculate outlet control headwater (HW)

i. Use Equation 4.14. if V_u and V_d are neglected

ii. Use Equation 4.4, 4.7c. and 4.9 to include V_u and V_d.

If HW_{oi} is less than $1.2D$ and control is outlet control:

1. The barrel may flow partly full.

2. The approximate method of using the greater tailwater or $(d_c + D)/2$ may not be applicable.

iii. Backwater calculations should be used to check the result; and

iv. If the headwater depth falls below $0.75D$, the approximate method shall not be used.

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Step 9: Determine Controlling Headwater (HW_c)

a. Compare HW_i and HW_{oi}, use the higher
   i. $HW_c = HW_i$, if $HW_i > HW_{oi}$ the culvert is in inlet control.
   ii. $HW_c = HW_{oi}$, if $HW_{oi} > HW_i$ the culvert is in outlet control.

Step 10: Compute Discharge over the Roadway (Q_r)

a. Calculate depth above the roadway (HW_r), $HW_r = HW_c - HW_{ov}$.
   HW_{ov} = height of road above inlet invert.
b. If $HW_r \leq 0$, $Q_r = 0$. If $HW_r > 0$, determine Q_r.

Step 11: Compute Total Discharge (Q_t)

$$Q_t = Q_d + Q_r$$

Equation 4.18

Step 12: Calculate Outlet Velocity (V_o) and Depth (d_n)

a. If inlet control is the controlling headwater, calculate flow depth at culvert exit:
   i. Use normal depth (d_n);
   ii. Use water surface profile.
b. Calculate flow area (A).
c. Calculate exit velocity (V_o) = Q/A.

If outlet control is the controlling headwater

a. Calculate flow depth at culvert exit:
   i. Use (d_c) if $d_c > TW$;
   ii. Use (TW) if $d_c < TW < D$;
   iii. Use (D) if $D < TW$.
b. Calculate flow area (A).
c. Calculate exit velocity (V_o) = Q/A.

Step 13: Review Results

Compare alternative design with constraints and assumptions, if any of the following are exceeded, repeat Steps 5 through 12:

a. The barrel must have adequate cover;
b. The length should be close to the approximate length;
c. The headwalls and wingwalls must fit site conditions;
d. The allowable headwater should not be exceeded; and
e. The allowable overtopping flood frequency should not be exceeded.

Step 14: Plot Performance Curve

a. Repeat Steps 6 through 12 with a range of discharges.
b. Use the following upper limit for discharge:
   i. Q_{100} if $Q_d < Q_{100}$;
   ii. Q_{500} if $Q_d > Q_{100}$;
   iii. Q_{max} if no overtopping is possible;
   iv. Q_{max} = largest flood that can be estimated.

Step 15: Related Designs

Consider the following options: (See Section 4.1; Design Features, and Related Design)

a. Tapered inlets if culvert is in inlet control and has limited available headwater.
b. Flow routing if a large upstream headwater pool exits.
c. Energy dissipators if V_o is larger than the normal V in the downstream channel.
d. Sediment control storage for sites with sediment concerns such as alluvial fans.

Step 16: Documentation

Prepare report and file with background information

4.4 Design Limitations

4.4.1 Allowable Headwater

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:

1. Will not damage upstream property.
2. Not higher than 300 mm below the edge of the shoulder.
3. Equal to an HW/D not greater than 1.5.
4. No higher than the low point in the road grade.
5. Equal to the elevation where flow can be diverted around the culvert.

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

1. Does not exceed 500 mm increase over the existing 100-year flood in the vicinity of buildings or habitations; and
2. Has a level of inundation that is tolerable to upstream property and roadways for the review discharge.

4.4.2 Tailwater Relationship of Channel

The hydraulic conditions downstream of the channel determine the tailwater depth relationship for different discharges.

1. Backwater curves at sensitive locations or single cross sections should be used. For important structures several downstream cross sections are required.
2. Critical depth and equivalent hydraulic grade line can be used if the culvert outlet is operating with a free outfall.
3. The high-water elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent) should be used to evaluate the influence of confluences.
4. Evaluate the hydraulic conditions of the downstream channel to determine a tailwater depth for a range of discharges which includes the review discharge.

4.4.3 Tailwater Relationship ~ Confluence or Large Water Body

1. Use the high-water elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent); and
2. If statistically independent, evaluate the joint probability of flood magnitudes and use a likely combination resulting in the greater tailwater depth.

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4.4.4 Maximum Velocity and Minimum Velocity

The maximum velocity at the culvert exit shall be consistent with the velocity in the natural channel or shall be mitigated with channel stabilisation and energy dissipation. Where the outflow velocity is greater than can be accommodated by the receiving waters, a form of energy dissipater will be required. (See section 4.5).

The minimum velocity in the culvert barrel should result in a tractive force ($\tau = dS$) greater than critical of the transported streambed material at low flow rates. When streambed material size is not known 0.8 meters per second should be used as an approximation.

If clogging is probable, consider installation of a sediment trap or size culvert to facilitate cleaning.

4.5 Use of Nomographs for Culvert Design

Culvert design can be accomplished using design aids in this Manual to manually determine the appropriate culvert size, shape (box or circle) and material that will accommodate a design flood at a given road crossing. Nomographs use standard hydraulic equations that take into account the key variables required for the design of peak flood discharge for the design return period and the maximum permissible headwater.

Typical examples of nomographs are illustrated below as Charts 4.1 to 4.3. Other monographs for design of culverts are given in Charts B.5 to B.16 in the Appendix. The nomographs apply to culverts with inlet control where there is no restriction to the downstream flow of the water. An illustrative example on design using nomographs is given in Appendix A.3. The computations are best carried using a computation Form (See Figure 4.11).

Chart 4.1 Headwater Depth and Capacity for Corrugated Metal Pipe Culverts with Inlet Control

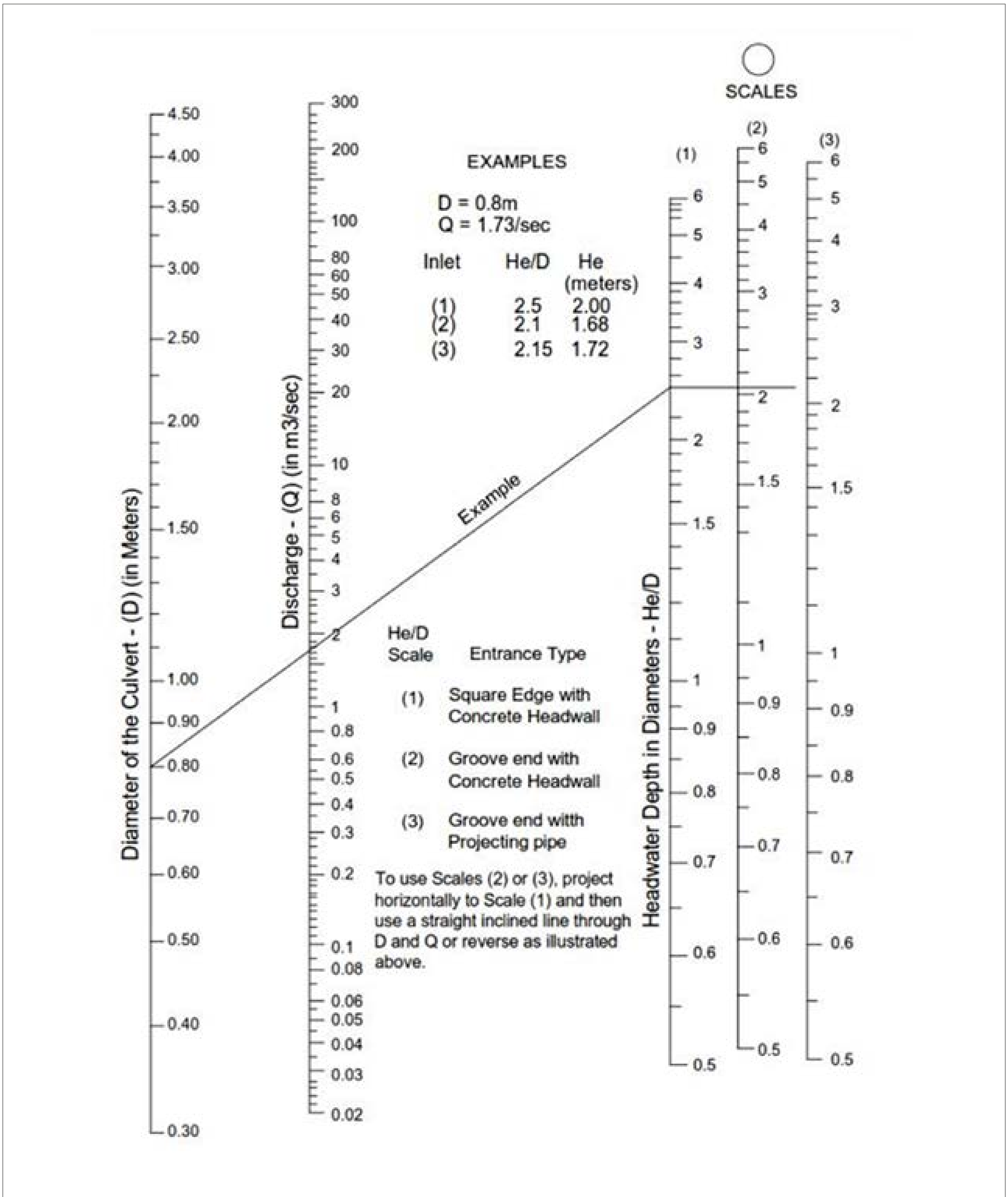


Chart 4.2 Headwater Depth and Capacity for Concrete Box Culverts with Inlet Control

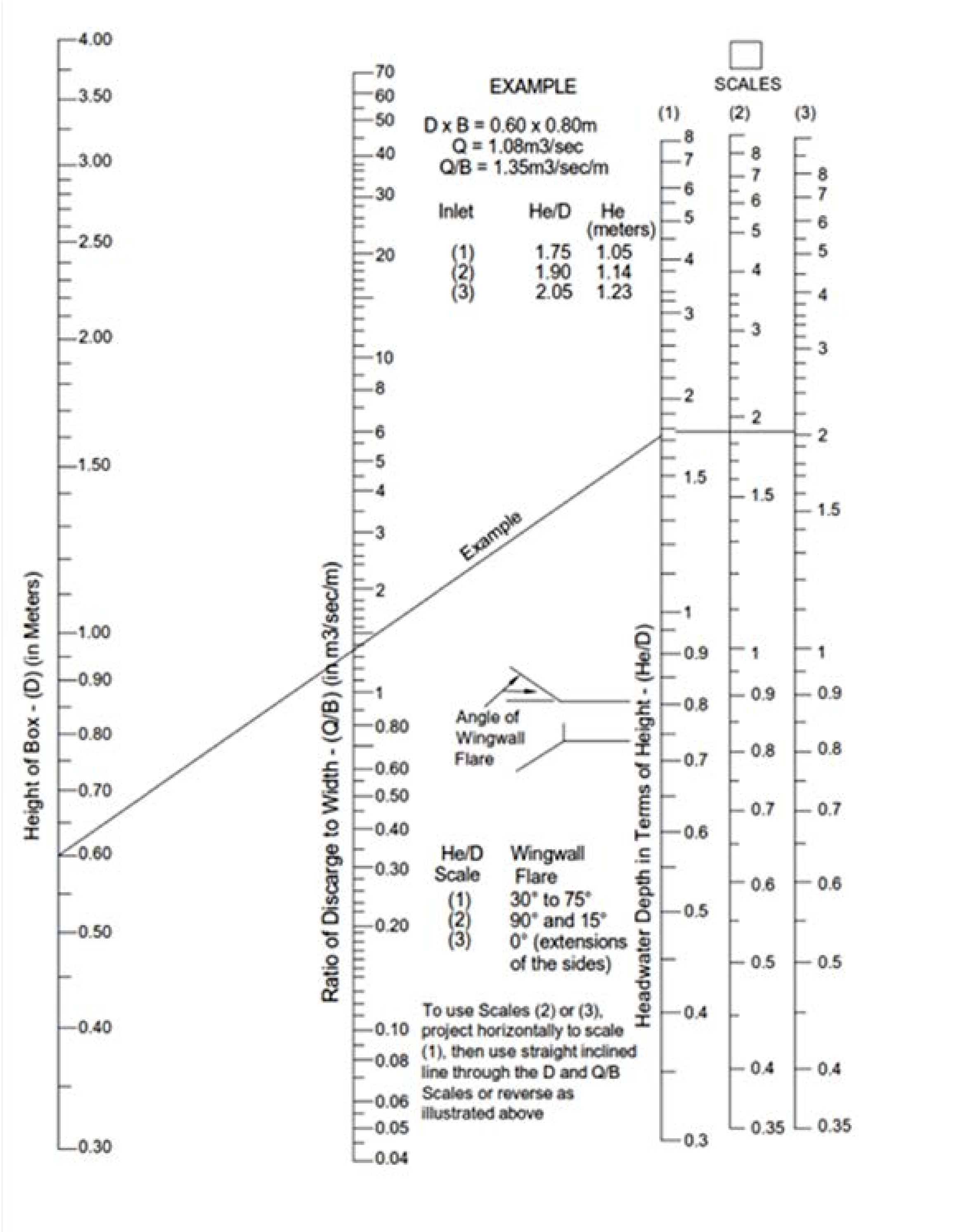


Chart 4.3 Headwater Depth and Capacity for C. M. Pipe Culverts with Inlet Control

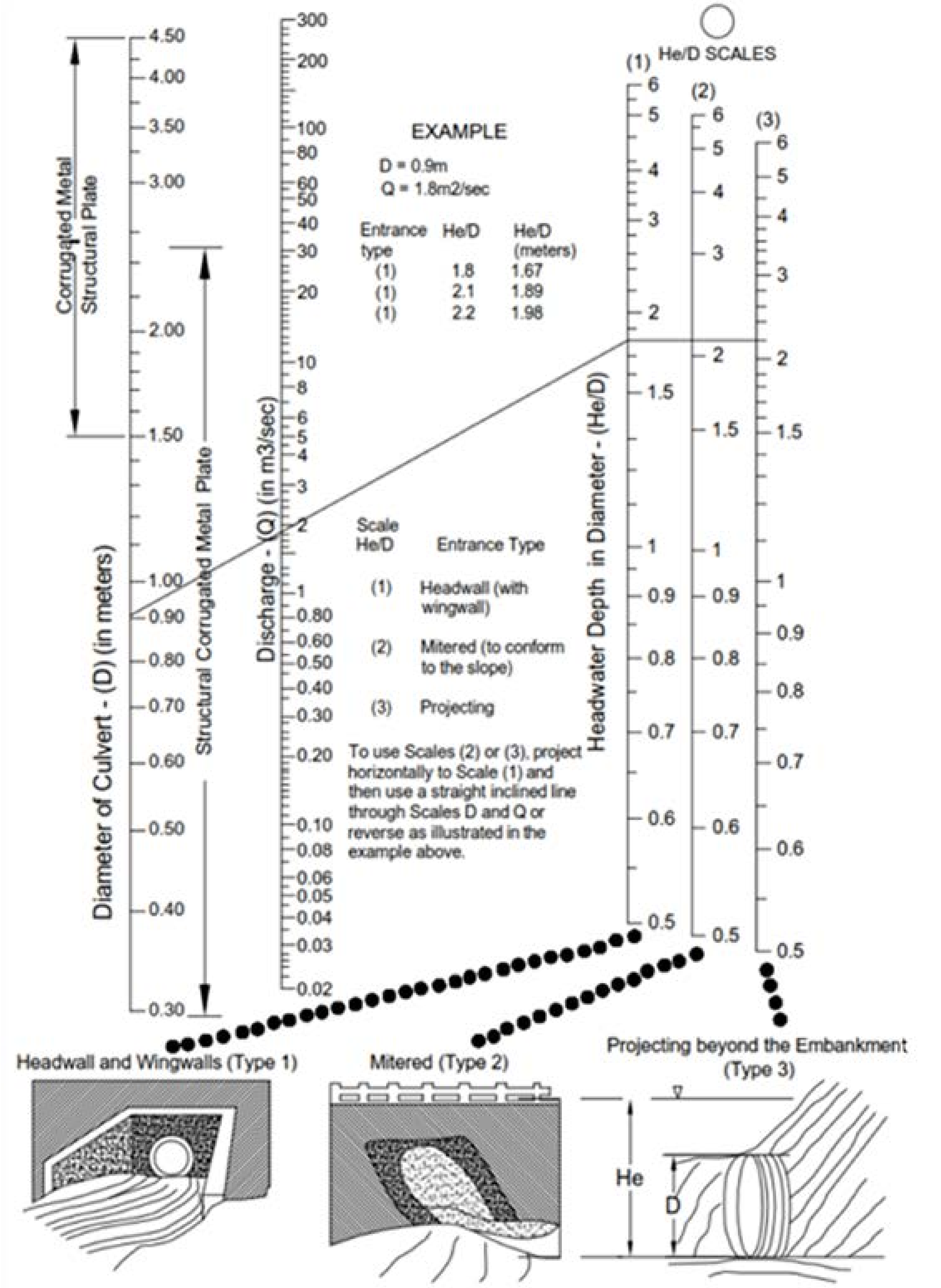


Figure 4.11 Computation Form

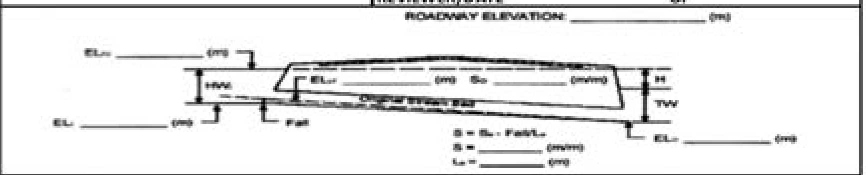
PROJECT .... | STATION .... | CULVERT DESIGN FORM
SHEET .... OF .... | DESIGNER/DATE .... OF ....
REVIEWER/DATE OF

HYDROLOGICAL DATA

0 METHOD: 0 STREAM SHAPE
0 DRAINAGE AREA: 0 OTHER
0 CHANNEL SHAPE:
0 ROUTING:

DESIGN FLOWS/TAILWATER

RI (Years)..... FLOW (m^3 ....TW (m)........



HEADWATER CALCULATIONS

CULVERT DESCRIPTION MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW, Q (m^3/s)	FLOW PER BARREL, Q/N (m^3/s)	INLET CONTROL HW/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	K_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
RCB - 2135 x 1830 mm - Bevel	*11.33*	*5.32*	*1.27*	*2.32*	*0.00*	*59.47*	*0.85*	*1.43*	*1.63*	*1.63*	*0.20*	*0.85*	*55.06*	*59.47*	*9.68*	*59.47 < 59.47 OK*
	14.16	*6.69*	*1.56*	*2.87*	*0.00*	*60.02*	*0.93*	*1.65*	*1.74*	*1.74*	*0.20*	*1.31*	*55.63*	*60.02*	*10.36*	*> 59.47 Calc Q.*
Performance Curve	*2.83*	*1.30*	*0.46*	*0.85*	*0.00*	*58.00*										
	5.65	*2.69*	*0.78*	*1.43*	*0.00*	*58.58*										
	8.50	*3.99*	*1.01*	*1.86*	*0.00*	*59.01*										

TECHNICAL FOOTNOTES

(1) USE Q/NS FOR BOX CULVERTS
(2) HW/D = HW/D OR HW/D FROM DESIGN CHARTS
(3) FALL = HW_1 . (EL_{hd} . EL_{sf}); FALL IS ZERO IF CULVERT IS ON GRADE
(4) EL_{hi} = HW_1 + ELi (INVERT OF INLET CONTROL SECTION)
(5) TW BASED ON DOWNSTREAM CONTROL ON FLOW DEPTH IN CHANNEL
(6) h_o = TW OR (dc + D)/2 WHICHEVER IS GREATER
(7) H = $(1+k_e+(1.63\ n^2L)/R^{1.33})V^2/2g$
(8) EL_{ho} = H + h_o

SUBSCRIPT DEFINITIONS:

a. Approximate
f. Culvert Face
h_d. Headwater in Inlet Control
h_o. Headwater in Outlet Control
i. Inlet Control Section.
o. Outlet
s_f. Stream bed at Culvert Section

COMMENTS AND DISCUSSIONS

CULVERT BARREL SELECTED

SIZE:
SHAPE:
MATERIAL:
ENTRANCE:

4.6 Use of Empirical Formulae for Culvert Design

The hydraulic capacities of drainage structures can be calculated using the following formulae that assume critical flow conditions in the respective structures:

4.6.1 For Pipe Culverts

The hydraulic design of pipe culverts is based on the method put forwards by A. Delorme and published in 'Annales des Ponts Chausees' where the maximum capacity of the circular pipe culvert is given by:

$$Q = 2.8\ rH^{3/2}$$

Equation 4.19

Where,

Q = The discharge in m³/s.
r = The radius of the pipe in metres.
H = Upstream water depth in metres.

4.6.2 For Box Culverts

The flow through a box culvert is similar to that existing over a broad crested weir except for the lateral restrictions provided by the culvert wall. Corrective coefficient may be applied to the discharge to allow for these restrictions. An experimental study made by A. Delorme and published in 'Annales des Pontsas Chausees' checked on the theoretical relationships between discharge and upstream water level and proposes for practical use of the formula below for sizing of box culverts. It further proposes the longitudinal gradient of the box culvert be at 2% to ensure self-cleansing velocities.

$$Q = 1.7LH^{3/2}$$

Equation 4.20

Where,

Q = Discharge in m³/s.

L = Box culvert width in metres.

H = Upstream water depth in metres.

4.7 Software Applications for Culvert Design

As culvert hydraulics are extremely complicated various software products are freely available to assist the lengthy process of culvert design as described above. The most widely use applications have been developed by the American Federal Highway Authority (HY8) or the Hydraulic Engineering Corps (HEC). HEC RAS 4 can be applied for water level calculations concerning floodplain crossings, multiple culvert calculations and bridge hydraulics.

The application of these software tools is highly recommended for checking the design results especially reached by the Nomograph Methods.

4.7.1 Application of HY-8

HY-8 is structured to be a culvert design tool. Input data are design discharge range, tailwater channel geometry, a roadway cross section and an embankment template. Any commercially available culvert alternative material and size can easily be selected, and a performance curve produced that is compared to design targets. (See worked example Appendix A.6).

4.7.2 Application of HEC-RAS

HEC- RAS is structured to be an analysis tool of a stream reach using water surface profiles. Input data are design discharge range, a series of channel cross sections, roadway geometry, bridges and/or culvert descriptions. HEC- RAS has the same commercially available culvert alternative material and size ranges as HY-8. (See worked example Appendix A.9).

4.8 Design of Energy Dissipators

Energy dissipators are any device designed to protect downstream areas from erosion by reducing the velocity of flow to acceptable limits.

The failure or damage of many culverts on detention basin outlet structures can be traced to unchecked erosion. Erosive forces which are at work in the natural drainage network are often exacerbated by the construction of a road or by other urban development. Interception and concentration of overland flow and constriction of natural waterways inevitably results in an increased erosion potential. To protect the culvert and adjacent areas, it is sometimes necessary to employ an energy dissipator.

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4.8.1 Guidelines Design of Energy Dissipation

The following guidelines should be taken into account in designing for water energy dissipation.

1. Culvert inlets should be lined.
2. Where gradients allow, drop inlets shall be designed to accommodate the inflow.
3. All culverts should have gradients of 1.5% to 3% and the outlet drains should have gradients 3%-5% to ensure self-cleansing and prevention of erosion.
4. Where such criteria given in (3) are satisfied the outlet drain shall be lined with wet stone masonry or concrete for a minimum length of 3 m starting from the mouth of the culvert.
5. Erosion protection should be designed and placed at all culvert outlets. This includes apron, rip rap, stone pitching on drain side slopes and a key wall.
6. Where there is a drop or steep gradient at the outlet of the culvert and high flow velocity is likely, box type stilling basins should be placed at the mouth of the culvert.
7. Where moderate flow velocity/energy is anticipated, and standards of culvert outlet drain gradients are exceeded then tumble or impact energy dissipators should be considered. If there is adequate drop stilling basin dissipators may be used.

4.8.2 Design of Mitigating of Scour Hole at Culvert Outlets

The first step is to determine the depth and length of the scour hole at the culvert outlet. The following parameters must be determined:

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 6 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:

$$V_0 = \frac{Q}{A_0}$$

Where,

Q = Culvert discharge, (m^3/s)

(Under normal circumstances Q would be the design capacity of the culvert, which would be full flow. Ao would be the outlet area of the culvert pipe.)

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:

$$d_e = \left(\frac{A_0}{2}\right)^{0.5}$$

Froude Number (F_r): is an important factor in the design of energy dissipators. For rectangular shapes, it is calculated as follows:

$$Fr = \frac{V_o}{(gd_o)^{0.5}}$$

Equation 4.21

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.

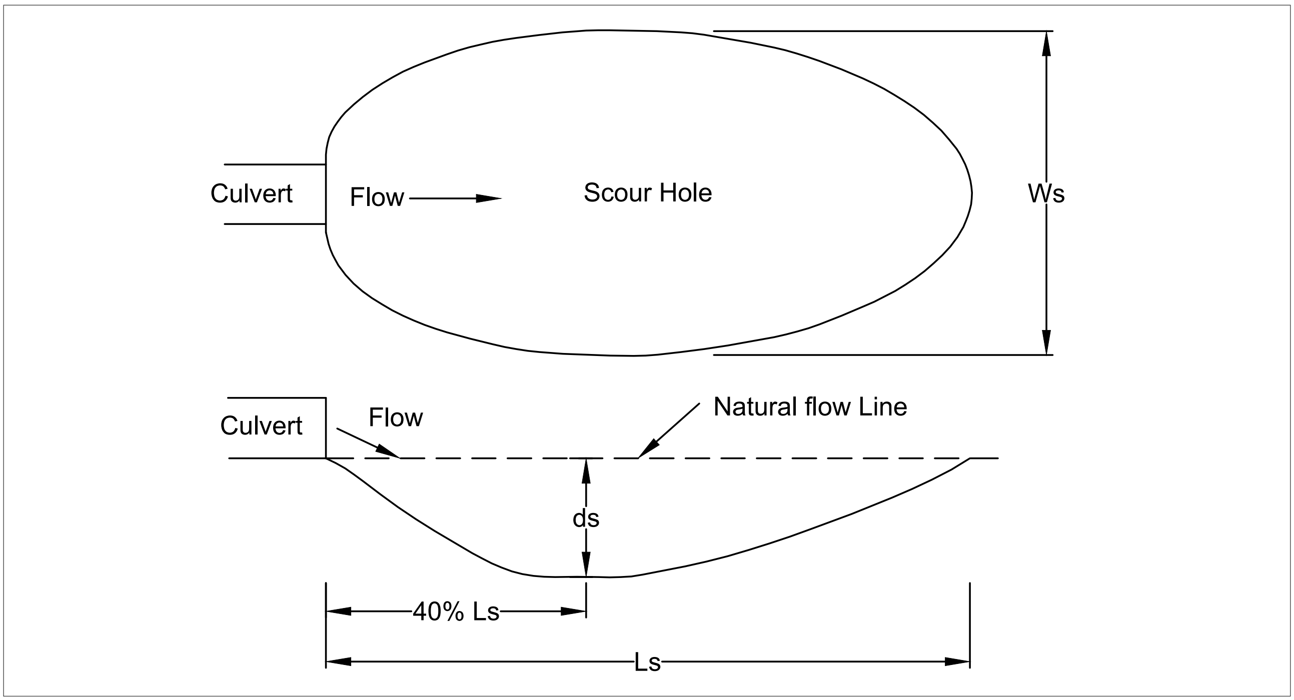
4.8.3 Scour Hole Estimation

The scour prediction procedure is used to estimate scour hole geometry base on the soil, flow data and culvert geometry. It serves together with maintenance history and site reconnaissance information to determine energy dissipator needs.

Only scour holes on cohesionless material are discussed herein. On cohesive soils, refer to Ref. HEC-14, section V, Appendix H.

Scour hole geometry varies with the tailwater conditions. The maximum scour geometry occurring at tailwater depths is less than half the culvert height. The maximum depth of scour, d_s, occurs at a location approximately 0.4 L_s downstream of the culvert, where L_s is the length of the scour.

Figure 4.12 Computation Form



The equation below 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 4.5.

$$\left[\frac{d_s}{R_c}, \frac{W_s}{R_c}, \frac{L_s}{R_c}, \frac{V_s}{R_c}\right] = C_s\, C_h \left(\frac{\alpha}{\sigma^{1/3}}\right)\left(\frac{Q}{\sqrt{gR_c^{2.5}}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta}$$

Equation 4.22

$$d_s.\ W_s.\ L_s = [F_1]\ [F_2]\ [F_3]\ R_c$$

Equation 4.23

$$d_s.\ W_s.\ L_s = C_s\ C_h\, F_1\ F_2\ F_3\ R_c$$

Equation 4.24

and,

$$F_1 = \frac{\alpha}{\beta^{0.33}}$$

Equation 4.25

$$F_2 = \left(\frac{Q}{g^{0.5}R^{2.5}}\right)^{\beta}$$

Equation 4.26

$$F_3 = \left(\frac{t}{316}\right)^{\theta}$$

Equation 4.27

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, (m³/s).
g = Acceleration due to gravity, (9.81 m/sec²).
t = Duration of the peak flow, (minutes). Use 30 minutes if unknown.
R_c = 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 Table 4.5 to Table 4.7.

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:

$$\sigma = \left(\frac{d_{84}}{d_{16}}\right)^{0.5}$$

Equation 4.28

Where,

d_{84} = mean particle diameter at the 84th percentile of the distribution.
d_{16} = mean particle diameter at the 16th 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 4.5 Coefficients for Computing Scour Hole (Dimensions Using Equation 4.22)

	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 4.6 Coefficients, C_s for Culvert Slope (Using Equation 4.23)

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 4.7 Coefficients, C_h for Culvert (Outlets Above the Stream Bed$_1$)

Drop Height (H_d)[1]	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

The coefficients have been derived from experiments with sand-bed materials.

A worked example on the calculation of the depth, length and width of scour at culvert outlet is given in the Appendix A.7.

A summary of the procedure is given in flowchart Figure 4.13 at the end of this section.

Step 1: Assemble Site Data and Project File

- **a.** See culvert design file for site survey.
- **b.** Review Section 4.2 for applicable criteria.

Step 2: Determine Hydrology

See culvert design file.

Step 3: Select Design

- **a.** See Section 4.4 Design Limitations.
- **b.** See culvert design file.
- **c.** Select flood frequency.
- **d.** Determine Q from frequency plot (step 2).

Step 4: Design Downstream Channel

- **a.** See culvert design file.
- **b.** Determine channel slope, cross-section, normal depth and velocity.
- **c.** Check bed and bank materials stability.

Step 5: Design Culvert

See culvert design file and obtain design discharge, outlet flow conditions (velocity and depth), culvert type (size, shape, and roughness), culvert s slope, and performance curve.

Step 6: Summarise Data on Design Form

- **a.** See Form A7.1 'Energy Dissipator Checklist' in the appendix.
- **b.** Enter data from steps 1-5 into Form A7.1.

Step 7: Estimate Scour Hole Size

- **a.** Enter input for scour equation on form A7.1.
- **b.** Calculate d_s. W_s. L_s

Step 8: Determine Need for Dissipator

An energy dissipator is needed if:

- **a.** The estimated scour hole dimensions exceed the allowable right-of- way, undermine the culvert cut-off wall, or present a safety or aesthetic problem;
- **b.** Downstream property is threatened; or
- **c.** V_o is much greater than V_d.

Step 9: Select Design Alternative

- **a.** See HEC 14 for Design Options
- **b.** Calculate Froude number, Fr.
- **c.** Choose energy dissipators types:
  - **i.** If $Fr > 3$, design a stilling basin according to HEC 14, section X.
  - **ii.** If $Fr < 3$, design a riprap basin, and if $Q < 12$ m^3/s for each barrel. If these are not acceptable or economical, try other dissipator types in HEC 14.

Step 10: Design Dissipators

For design of appropriate dissipator refer to HEC 14.

Step 11: Design Riprap Transition

a. Most dissipators require some protection adjacent to the basin exit.
b. The length of protection can be judged based on the difference between V_o and V_d. The riprap shall be designed using HEC 11.

Step 12: Review Results

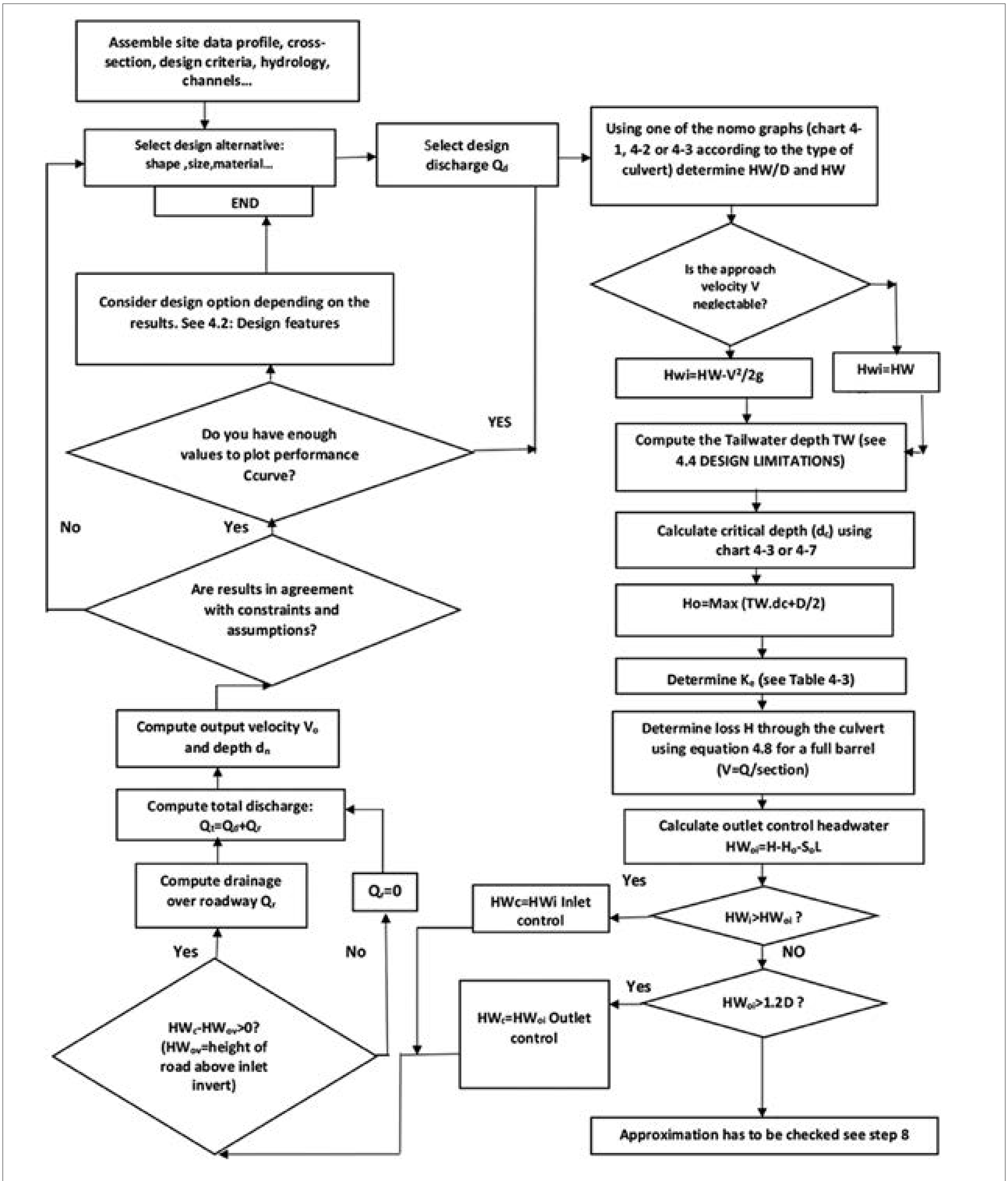
a. If downstream channel conditions (velocity, depth and stability) are exceeded, either design riprap for channel, step 4, or select another dissipator, step 9.
b. If preferred energy dissipator affects culvert hydraulics, return to step 5 and calculate culvert performance.
c. If debris-control structures are required upstream, consult HEC 9.
d. If a check Q was used for the culvert design, assess the dissipator performance with this discharge.

Step 13: Documentation

a. On hydrology; See Part 1 of this Manual.
b. Include computations in culvert report or file.

A worked example in determination of scour depth is given in the Appendix A.7.

Figure 4.13 Flowchart: Design of Culverts



See Procedure in section 4.3.6

5 Hydraulic Design of Bridges

5.1 General Definitions and Principles

5.1.1 Definition

Bridges are defined as:

a. Structures that transport traffic over waterways or other obstructions.

b. Part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure.

c. Structures with a centreline span of 6 meters or more; however, structures such as large box culverts of multi-plate arches, designed hydraulically as bridges as described above are treated in this chapter, regardless of length.

5.1.2 Standards and Policies

Standards are a set of goals that establish a definite course of action or method of action and that are selected to guide and determine present and future decisions. Principles that are unique to bridge crossings are presented in this section. The hydraulic analysis should consider various stream-crossing system designs to determine the most cost-effective proposal consistent with design constraints.

These policies identify specific areas for which quantifiable criteria can be developed:

a. The final design selection should consider the maximum backwater allowed (0.5 m) unless exceeding the limit can be justified by special hydraulic conditions.

b. The final design should not significantly alter the flow distribution in the floodplain.

c. The 'crest-vertical curve profile' shall be considered as the preferred road crossing profile when allowing for embankment overtopping.

d. A specified clearance shall be established to allow for passage of debris; a vertical clearance shall be established based on normally expected flows and to allow for the passage of small boats where necessary.

e. Degradation or aggradation of the river as well as contraction and local scour shall be estimated as part of the final design; the design should either eliminate scour or provide scour protection.

f. Foundation level shall be positioned below the total scour depth wheneverpractical.

5.2 Hydraulic Performance of Bridges

The stream-crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. The hydraulic variables and flow types are defined in Figure 5.1 and Figure 5.2 as discussed below:

1. Backwater (h_1) is measured relative to the normal water surface elevation without the effect of the bridge at the approach cross-section (Section 1). It is the result of contraction and re-expansion head losses and head losses due to bridge piers. Backwater can also be the result of a 'choking condition' in which critical depth is forced to occur in the contracted opening with a resultant increase in depth and specific energy upstream of the contraction. This is illustrated in Figure 5.2.

2. Type I consists of subcritical flow throughout the approach, bridge, and exit cross sections and is the most common condition encountered in practice (see Figure 5.2)

3. Type IIA and IIB (see Figure 5.2) both represent subcritical approach flows which have been choked by the contraction resulting in the occurrence of critical depth in the bridge opening. In Type IIA the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In the Type IIB it is higher than the normal water surface elevation and a weak hydraulic jump immediately downstream of the bridge contraction is possible.

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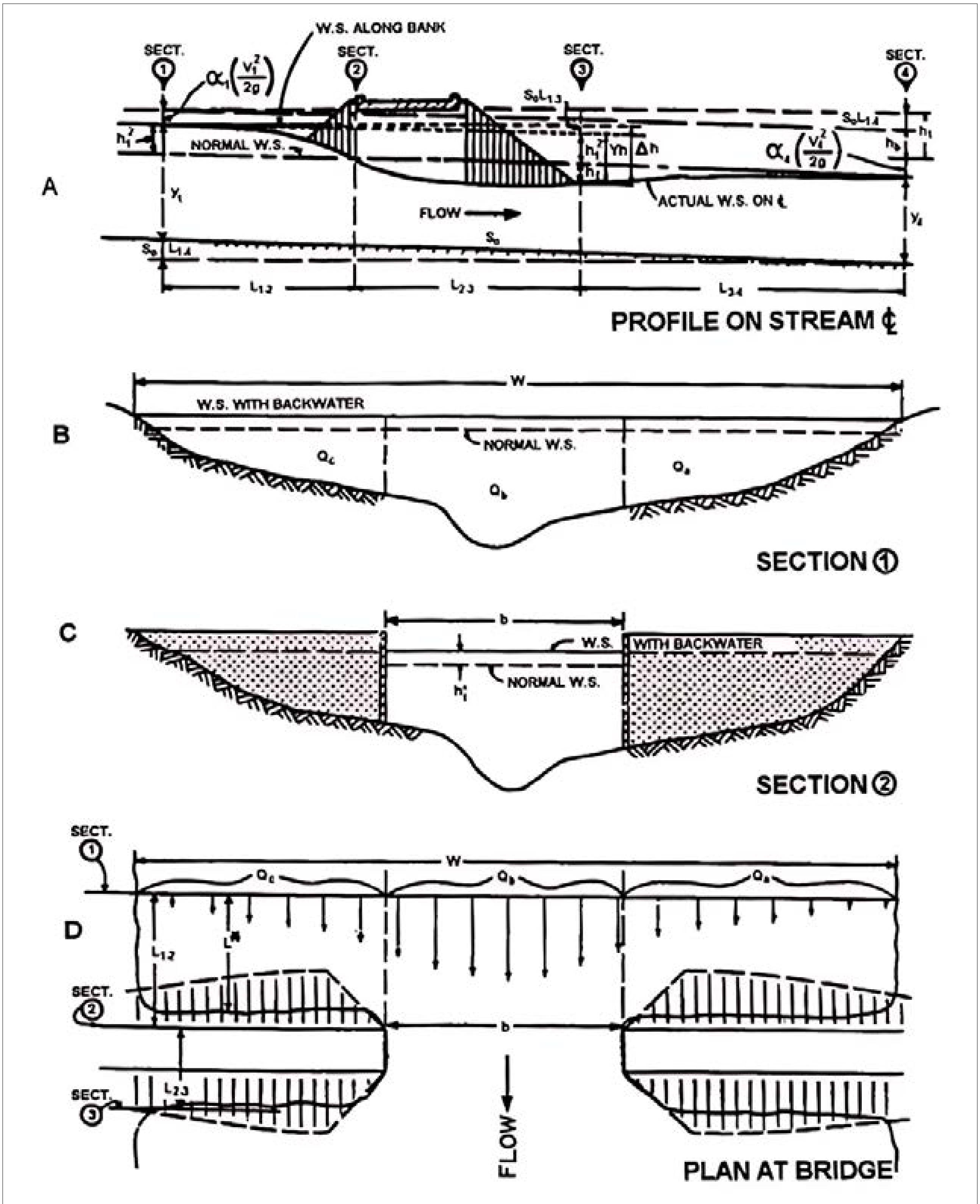
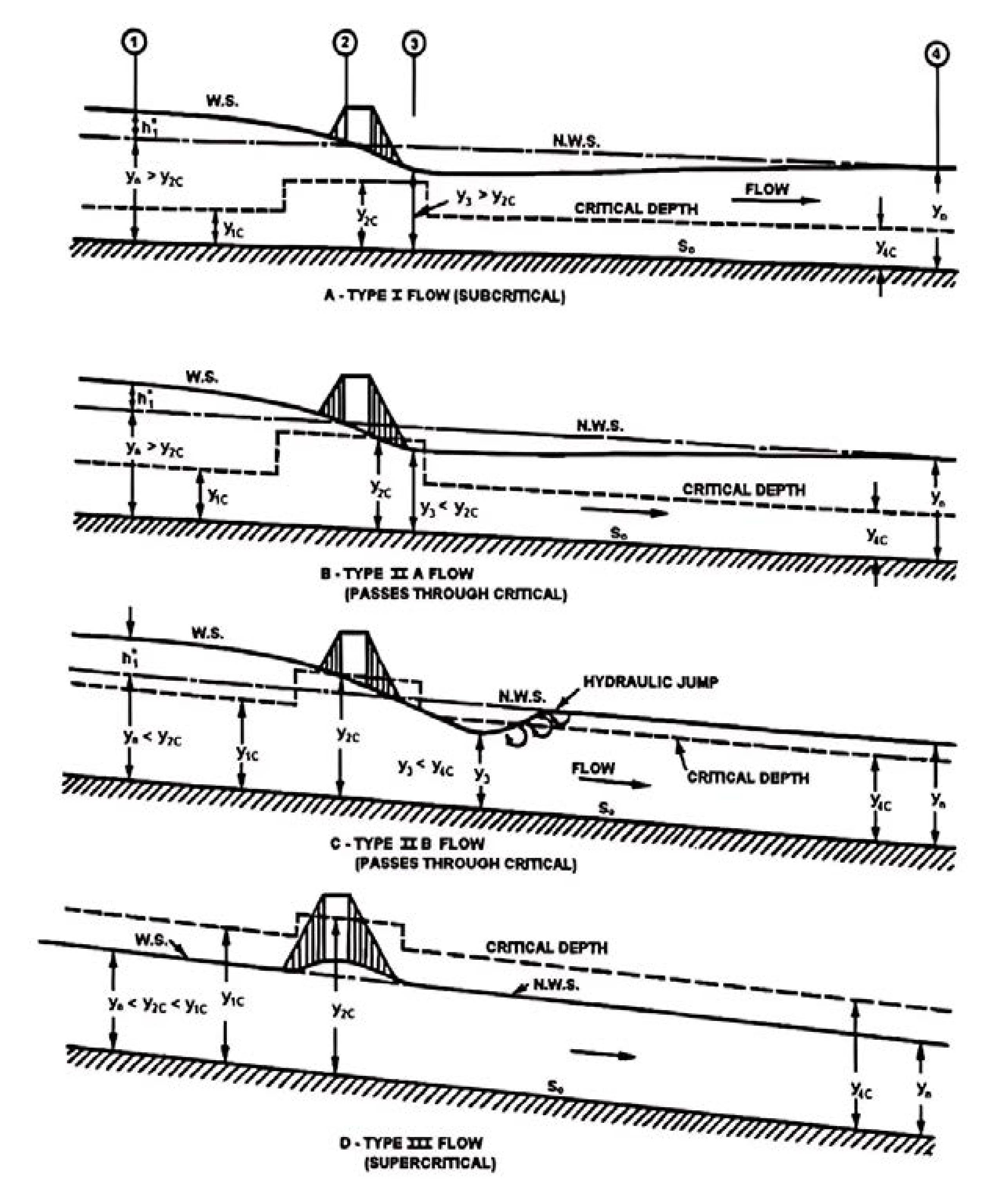
4. Type III flow (see Figure 5.2) is supercritical approach flow and remains supercritical through the bridge contraction. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

Figure 5.1 Bridge Hydraulics Definition Sketch



Source: HDS-1

Figure 5.2 Bridge Flow Types



Source: HDS-1

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5.3 Design Criteria and Consideration

Bridge design criteria discussed here only cover the hydraulic aspects. For detailed hydrological design the designer is referred to Part 1 of this Manual and Structural design of bridges covered in 'Road Design Manual Volume 4; Part 2: Bridge and Culvert Design and Part 3: Retaining Structures Design'.

5.3.1 Key Elements of Bridge Hydraulic Design

The key elements of bridge hydraulics design include:

1. Hydrology – this is covered in detail in Part 1 of this Manual.
2. Bridge locations – the different scenarios and their pros and cons.
3. Stream hydraulics – understanding the nature of the waterway and stream flow mechanics.
4. Topography of the stream characteristic upstream and downstream of the water crossing.
5. Bridge orientation – selection of the best position and orientation of the structure.
6. General arrangement – selection of the most hydraulically efficient bridge configuration i.e, of the superstructure and substructure.
7. Hydraulic head and flow velocities upstream and downstream of structure – these determine the water profile for the design maximum flood discharge. It also covers the contraction of the channel or waterway because of obstruction caused by the bridge structure and approaches.
8. Determination of the freeboard – which is the height of the underside of the bridge (usually bottom of the main beams) above the height of the free surface of the water at the design flood levels.
9. Bridge scour/scour depth – the calculation of potential scour and remediation.
10. Sedimentation, the impacts and remediation – sedimentation changes the stream characteristics and can affect the capacity of bridges significantly.
11. Protection works – these are determined based on the potential for scour and critical water attack areas.
12. River training – alteration for the geometrics of the stream including sloping and benching and slope protection both upstream and downstream of the bridge.

There are different types of bridges, and each type affects the hydraulics differently:

1. Small bridges – these have short spans (up to 15 m) on relatively narrow and low volume streams.
2. Medium bridges – these have medium size spans (up to 50 m) on a well-defined river channel.
3. Large bridges – these have spans greater than 50 m and include cable stay bridges built on very wide rivers and floodplains. The configuration may consist of multiple or a series of bridges close to each other. Excessive backwater may result in the river bursting its banks and inundating the floodplains.

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5.3.2 General Considerations

The following are the general criteria related to the hydraulic analyses for the location and design of bridges:

1. Backwater shall not significantly increase flood damage to property upstream of the crossing.
2. Velocities through the structure(s) will not either damage the road facility or increase damages to adjacent or downstream property.
3. Maintaining the existing flow distribution to the extent practicable, is recommended.
4. Pier spacing, orientation, and abutment are to be designed to minimise flow disruption and potential scour; spill-through type abutments using side slopes are preferred over deep abutments to minimise scour and backwater effects.
5. Select foundation design and/or scour countermeasures to avoid failure by scour.
6. Allow freeboard at structure(s) designed to pass anticipated debris and minimise blockage of bridge openings.
7. Acceptable risks of damage or viable measures to counter the unpredictability of alluvial streams.
8. 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 minimise expansion or contraction of flow between the two bridges.
9. Minimal disruption of ecosystems and values unique to the floodplain and stream.
10. The choice of bridge structure and size will depend on the level of service required and life-cycle costs perspective.
11. Adequate right-of-way shall be provided upstream and downstream of the structure for maintenance operation.
12. Compliance with statutes.

5.3.3 Specific Criteria

These criteria augment the general consideration. They provide specific, quantifiable values that relate to local site conditions. Evaluation of various alternatives according to these criteria can be accomplished by using the water surface profile programs such as HEC-RAS.

a. **Inundation**
Inundation (or submergence) of the carriageway dictates the level of traffic services provided by the facility. The carriageway overtopping flood level identifies the limit of serviceability.

b. **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 road.

c. Design Floods

Design floods for such purposes as the evaluation of backwater, clearance, and overtopping shall be established on risk-based assessment of local site conditions. They should reflect consideration of traffic service, environmental impact, property damage, hazard to human life, and floodplain management criteria. Many times, the historical flood is so large that a structure to handle the flow becomes uneconomical and is not warranted. It is the engineer's responsibility to determine the design discharge. The overtopping discharge is calculated at the site but may overtop the roadway some distance away from the site.

For guidelines on choice of design peak flows, refer to Chapter 4 of Part 1 of this Manual. The guidelines for bridges are reproduced in Table 5.1 below for ease of reference:

Table 5.1 Guidelines on Design Return Periods for Bridges

	*Return Period (years)	
	Design Flood	Check Flood
Short bridges (6 m < L ≤ 15 m)	50	100
Medium Bridges (15 m < L ≤ 50 m)	50	100
Long bridge (L > 50 m)	100	200

Where L is the length of the bridge opening from the abutments to abutment.

***Note:** Wherever there is a need to adjust the design frequencies to accommodate adverse effects such as that of climate change, it is recommended to increase the design return periods by 50% or 100% in the high-risk areas.*

d. Backwater

Backwater levels – the final bridge design selection should consider the maximum backwater allowed (0.5 m) unless exceeding of the limit can be justified by special hydraulic conditions and low risk of inundation and damage to valuable land and properties.

e. Freeboard

A minimum clearance 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. The specified clearance shall be based on site-specific conditions. This includes bridges with crest curves. This should apply even after several years of sedimentation under or downstream of the bridge. Clearance shall conform to the following guidelines:

Table 5.2 Clearance (m) Under Bridge Superstructure

Structure Type	Minimum Freeboard (m)
Short and medium bridges (6m < L ≤ 50 m)	*1.0
Long span bridge (L > 50m)	1.5

f. Embankment Overtopping

Where the maximum predictable flood cannot be accommodated through the bridge opening the design shall include embankment overtopping. Consideration shall be made of the disruption of traffic, inundation, and hydraulics of the weir.

g. Flow Distribution

The movement of the proposed stream-crossing location shall be calculated to determine the flow distribution and to establish the location of the bridge opening(s). The proposed facility should not cause any significant change in the existing flow distribution. Relief openings in the approach roadway embankment shall be investigated if there is more than a 20% redistribution of flow.

h. Relief Openings

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.

i. The flow line of the flood channel shall be set above the stage elevation of the dominant discharge

ii. The flood channel must extend far enough up and downstream of the bridge to establish the desired flow regime through the affected reach.

iii. The flood channel must be stabilised to prevent erosion and scour.

iv. Foundation levels shall be positioned below the total scour depth whenever practical.

v. Relative positioning of the bridge axis - with respect to the orientation of the flow

i. Magnitude of Flood for Scour (Super Flood)

Design bridge foundation scour considering the magnitude of flood, through the 1% event (100 year), which generates the maximum scour depth. The designer should use a safety factor of three. It is recommended that the super flood 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 (see Table 5.5 in section 5.6) For larger structures the modelling of scour with appropriate hydraulic modelling software such as HEC RAS 3 is strongly recommended.

j. Environmental Considerations

Environmental criteria must be met in the design of stream-crossing systems. Such considerations might require the expertise of a biologist on the design team. Water quality considerations should also be included in the design process insofar as the stream- crossing system affects the water quality relative to beneficial uses. As a practical matter with bridges, the hydraulic design criteria related to scour, degradation, aggradation, flow velocities, and lateral distribution of flow, for example, are important criteria for evaluation of environmental impacts as well as the safety of the stream-crossing structures.

k. 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, aquatic animals and normal flow distribution must be considered.

All borrow areas existing within the flood plain shall be chosen so as to minimise the potential for scour and adverse environmental effects within the limits of the bridge and its approaches on the floodplain.

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. (See Chapter 6).

5.4 Design Procedure

5.4.1 Bridge Location Selection and Orientation Guidelines

The guidelines, which should be followed in locating and orienting bridges, preparatory works, and design processes, are given below:

1. Bridge location – bridges should be located and centred 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.

2. Bridge openings – 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.

3. Orientation of structural elements - orient headers and interior bends to conform to the streamlines at design flood level. 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.

4. Minimising scour – locate and orient the bridge headers and piers to minimise the potential for excessive scour.

5. Re-mediation for excessive intrusion of headers/approaches into the stream – if the intrusion of either or both roadway headers/approaches into the stream floodplains is more than about 240 m, consider including either relief openings or guide banks.

6. Incorporate existing vegetation in the overall bridge plan - where practicable, leave trees and shrubs intact even within the right-of-way. Minimising 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.

7. Allowing for overtopping of bridge approaches – check the critical factors of disruption of traffic and the related impacts when allowing for overtopping of the bridge approaches. The flow over the top should be designed to take the excess flood discharge, which cannot be accommodated through the bridge structure. The overtopped section behaves as a weir and the depth of water should generally not exceed 0.7 m. The design should include protection against scour.

5.4.2 Design Procedure Outline

The following design procedure outline shall be used. Although the scope of the project and individual site characteristics make each design a unique one, this procedure shall be applied:

5.4.2.1 Data Collection

a. **Survey**
   1. Topography, including several river cross sections up to 100 m upstream and downstream of planned bridge, depending on size of planned bridge.
   2. Geology.
   3. Water marks:
      i. Highest-known flood level (HFL), is the highest historical flood level.
      ii. Ordinary flood level (OFL), is the level to which the river normally rises during the wettest part of the year.
      iii. Low water level (LWL), is the level prevailing in the river during dry weather.
   4. History of debris accumulation and scour.
   5. Hydraulic performance of existing structures.

6. Maps, aerial photographs.
7. Rainfall and stream gauge records.
8. Field reconnaissance.
9. Existing Structures.

b. Other Relevant Information

1. Water Resources and Irrigation studies.
2. River basin studies.
3. 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 risk assessment.
2. Application of agency criteria.

5.4.2.2 Hydrological Analysis

a. Catchment Area Morphology

1. Drainage area (attach map)
2. Catchment area and stream slope
3. Channel geometry

b. Hydrological Computations

1. Discharge for historical flood that complements the high-water marks used for calibration.
2. Discharges for specified frequencies.

5.4.2.3 Hydraulic Analysis

a. Manual analytical methods;
b. Computer hydraulic model construction;
c. Computer model calibration and verification
d. Computer model sensitivity analysis (roughness, structural coefficients. future climate change and blockage scenarios)
e. Hydraulic performance for existing conditions
f. Hydraulic performance of proposed designs
g. Scour depth calculations and mitigation;
h. Mitigation measures;
i. Design details such as for protection works and river training.

5.4.2.4 Final Design

a. Risk assessment/least-cost alternative {(Least Total Expected Cost -(LTEC)}.
b. Measure of compliance with established hydraulic criteria.
c. Consideration of environmental and social criteria.
d. Design details such as riprap, scour abatement, and river training.

5.4.2.5 Documentation

a. Complete project records, etc.

Standard form for drainage surveys is shown in Appendix D.

5.4.3 Hydraulic Design for Small to Medium Size Bridges

The basic design process is given below:

1. Determine freeboard – establish the height of the structures considering the specifications for the minimum freeboard (see Table 5.2) and any technically justifiable adjustments to the freeboard based on the prevailing site conditions;
2. General arrangement – prepare the general arrangement for the bridge substructure and superstructure making adjustments on the positions of the abutments and piers to optimise the hydraulic capacity and costs;
3. General and local scour – calculate the general scour and local scour and assess the worst-case scenario of the profiles;
4. Calculate backwater elevations – check that the backwater resulting from the constriction of the waterway area caused by the abutments and piers does not cause excessive flooding or damage upstream and that it does not affect the height of the structure and freeboard.
5. Finalising general arrangement – finalise the general arrangement of the bridge structure and estimate the sizing of piers and abutments;
6. Checking effects of backwater and scour – check the effect of the final general arrangement to the backwater and local scour. The backwater effect (afflux) should be calculated for all bridges with a design water velocity exceeding 1.0 m/s.
7. Cost evaluation – using known unit costs, estimate cost of the superstructure, the substructure, river training and protection works including any relief culverts which may be required;
8. Iteration – repeat the process using alternative crossing configurations in an iterative process to determine the optimum bridge hydraulic design.

5.4.3.1 Design Methods

There are two main bridge hydraulic design methods:

1. Manual analytical methods – these methods are suitable for small to medium bridges of relatively small catchment areas of up to 300km^2.
2. Computer based hydraulics design methods – these involve tailor-made software for modelling streamflow and flood discharges and are suitable for all situations including large catchment areas and wide flood plains.

Manual Analytical Methods

Determination of freeboard (height of bridge structure)

The design of the height of the structure is dependent on the assessment of the water levels. There are 3 different levels of water, which can be considered, and they include:

1. High flood level (HFL) – this is the highest known flood level or that which is estimated through the consideration of much higher flood return period, say 200-year return. It is usual not to design bridge hydraulics based on HFL, but this can be used as a check.
2. Design flood level (DFL) – this is the water level which is based on the design flood return period, 50 years for short to medium bridges and 100 years for large bridges.
3. Low water level (LWL) – this is the base-flow or water passing through the bridge during the dry season.

Freeboard is distance from the free water surface of the DFL to the bottom of the lowest part of the superstructure measured on the upstream side of the structure.

Table 5.2 provides specifications for minimum allowable freeboard for different flood discharge scenarios for bridges.

The calculation of bridge discharge is carried out based on the design flood discharge of the river. Two manual methods are applicable and are covered in more detail in Chapter 2 which includes natural channels (streams and rivers) and artificial channels (canals, drains, etc,).

It is assumed that the design flood discharge/flow (Q) which is computed for the stream or the river at design peak flow would be the same upstream of the structure, through the structure and immediately downstream of the structure. The parameters, which vary are:

1. Depth of water – the depth of the water immediately upstream (Headwater depth, yH) and the depth at the structure and the depth of the water immediately downstream of the structure (Tailwater depth, yT) would be different and decreasing in that order.
2. Flow velocity – the velocity of the flow of water immediately upstream of the structure (V_1) and immediately downstream if the structure (V_2) would be different and increasing in that order.

Manning's equations are used to estimate the peak flood discharge and are given below:

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

Equation 5.1

Where,

V = Flow velocity in m.

n = Manning's roughness coefficient.

R = Hydraulic radius (m); defined as A/P.

P = The wetted perimeter in m.

A = Waterway area, m^2.

S = The slope which is the riverbed slope.

The discharge (Q) is then calculated from the velocity and waterway area in m^3/sec (or cumecs).

$$Q = AV$$

Equation 5.2

This is applicable in conditions where flow is uniform, and the stream cross-section is relatively regular longitudinally.

Where the waterway is non-uniform the stream cross-section should be divided into several sections as explained in subsection 2.6.1 in Chapter 2 of this Manual.

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Where there is an existing bridge on the same river the orifice method could be used to determine the hydraulics of the bridge.

The following equation is used for estimating the flow discharge and flood levels.

$$Q = C_o L y_1 \left[(y_1 - y_2) + (1 + e)\frac{V^2}{2g}\right]^{1/2}$$

Equation 5.3

Where,

Q = Discharge in m³/sec.

C_o and e = coefficients to account for the effect of the structure on the flow of water.

L = the effective length of the waterway i.e, the distance between the abutments minus the widths of the piers.

y_1 = the depth of the water immediately upstream of the bridge measured from marks left by the river in flood.

y_2 = the depth of water immediately downstream of the bridge measure from the marks of water levels on piers and abutments.

V = mean velocity of approach (m/s) g = acceleration by gravity (9.81m/s²).

The values of C_o and e are given in Table 5.3. The intermediate values are derived by interpolation. Backwater, $h = y_1 - y_2$.

The discharge, Q, must be increased by 5% when $h = y_1 - y_2 > \frac{y^2}{4}$

Table 5.3 Values of C_o and e in the Orifice Formula

L/W	C_o	e
0.50	0.892	1.050
0.55	0.880	1.030
0.60	0.870	1.000
0.65	0.867	0.975
0.70	0.865	0.925
0.75	0.868	0.860
0.80	0.875	0.720
0.85	0.897	0.510
0.90	0.923	0.285
0.95	0.960	0.125

Discharge for bridges can also be estimated using the procedure explained in sub-section 4.6.

$Q = 1.833 * W * H^{3/2}$ for a single structure opening

Equation 5.4a

$Q = 1.833 + 1.655(n'-1) * W * H^{3/2}$ for a multiple structure opening

Equation 5.4b

Where,

W = Width of opening (m).

H = Depth of flow (m).

n' = Number of openings.

Computer Aided Design Software

Computer aided design software such as HEC RAS are the most used approach in hydraulic design of bridges.

5.4.4 Hydraulic Design for Medium to Large Bridges

The design of large bridges is complex and requires large amounts of data. As a result, computer aided design is the most preferred.

In designing large bridges, it is necessary to carry out preliminary design using estimated values of key parameters.

The following key information is required for the preliminary hydraulic design.

1. Preliminary channel width.
2. Elevation at excavated channel width.
3. Skew, station at centreline of channel.
4. Recurrence interval for design event.
5. Drainage area.
6. Design discharge.
7. 50- and 100-year discharges.
8. 200-year discharge for check and bridge scour (this should be discussed with local drainage experts and agreed upon).
9. Minimum low girder elevation.
10. Thalweg elevation.
11. Low water level (LWL).
12. Design flood level (DFL).
13. 50- and 100-year flood levels.
14. 200-year flood levels.
15. Design velocity (V).
16. 50- and 100-year velocities.
17. 200-year velocity and riprap dimensions for the bridge design.

Once the preliminary design is completed and the appropriate choice of bridge, orientation and general arrangement is determined then a detailed site investigation should be carried out, including determination of stream reach, flow patterns, geotechnical surveys of bed and bank materials and in-situ conditions. The data and information should be collected bearing in mind that it is mainly required as inputs for the hydraulic design software e.g., HEC-RAS.

The general procedure for the design of hydraulics of large bridges is given in the following step-by-step process.

1. Determine watershed hydrology (Refer to Chapters 4 and 5 of Part 1 of this Manual).
2. Obtain information on:
   a. Flood history of the river or flood plain – this information can be obtained from local residents and local authorities and the road authorities who may have relevant knowledge.
   b. Investigate any existing bridge structure(s) close to the proposed crossing point preferably on the same river. Check both upstream and downstream of the structure for scour, siltation, etc.
3. Obtain hydraulic information from existing studies for road projects near the site and use the data from the study to relate the information to the proposed crossing.

4. Complete a water surface profile analysis through the bridge reach. This should include the analysis of the natural situation without the proposed bridge and an analysis with the proposed bridge. The water surface profile may be determined using the program HEC-RAS, ISIS or Mike 11 or any other approved software. The HEC-RAS program and ISIS are freely available to download and are recommended to be used as standard software for bridge crossing analysis.
5. The return period and design discharge for the profile analysis shall be computed as discussed in Chapter 4 Part 1 of this Manual. Factors which contribute to the selection of the return period include the capacity and size of the road, whether it is located in a rural or urban area, and the expected traffic levels.
6. A range of bridge opening sizes smaller and larger than the existing channel should be analysed and then compared with the existing and natural conditions to choose the optimum bridge channel width for the design flow.
7. Locate the bridge within the floodplain and select a skew, which best fits 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.
8. Assess the impacts to the surrounding property and roadway for the overtopping and check 100-year flood for the various alternatives identified in Step 4. Any increase in the floodplain should be avoided if possible other mitigation measures may be put in place (e.g., compensation storage areas).
9. Make preliminary calculations for aggradation/degradation, contraction scour and local scour (Section 5.6).
10. Select the necessary revetment protection (i.e., riprap, guide banks, spur dikes, etc,) for the bridge and channel. These are detailed in Chapter 6.

5.5 Software Design Methodologies

The HEC-2 software model uses a variation of the momentum method in its special bridge routine when there are bridge piers. The momentum equation between cross- sections 1 and 3 is used to detect Type II flow and solve for the upstream depth, in this case with critical depth in the bridge contraction. The bridge analysis routines in HDS-1 and WSPRO software may yield a better definition of actual hydraulic performance.

5.5.1 HDS-1

The method in HDS-1 (see worked example in Appendix A.8) is an energy approach with the energy equation written between cross sections 1 and 4 as shown in Figure 5.1 for Type I flow. The backwater is defined in this case as the increase in the approach water surface elevation relative to the normal water surface elevation without the bridge.

This model uses a single typical cross section to represent the stream reach from points 1 to 4 on Figure 5.1. It also requires the use of a single energy gradient. **This method is no longer recommended for final design analysis of bridges due to its inherent limitations, but it may be useful for preliminary analysis**. Previous studies show the need to use a multiple cross section method of analysis to achieve reasonable stage- discharge relationships at a bridge.

5.5.2 Energy (WSPRO)

WSPRO combines step-backwater analysis with bridge backwater calculations. This method allows for pressure flow through the bridge, embankment overtopping, and flow through multiple openings and culverts. The bridge hydraulics still rely on the energy principle, but there is an improved technique for determining approach flow lengths and the introduction of an expansion loss coefficient. The flow-length improvement was found necessary when approach flows occur on very wide, heavily-vegetated floodplains. The program also greatly facilitates the hydraulic analysis required to determine the least-cost alternative.

WSPRO can be used for both preliminary and final analyses of bridge hydraulics. Even if only a single surveyed cross section is available, the input-data propagation features of WSPRO make it easy to apply, with more comprehensive output available than with HDS-1.

5.5.3 HEC-1 and WSP-2

HEC-1 and WSP-2 are recognised methods for computing water surface profiles. A worked example in HEC RAS is shown in Appendix A.9.

5.5.4 Modelling Methods

Other modelling methods include the following:

1. **2-Dimensional Modelling**

   The water surface profile and velocities in a section of river are often predicted using a computer model. In practice, most analysis is performed using one-dimensional methods such as the standard step method found in WSPRO. While one-dimensional methods are adequate for many applications, these methods cannot provide a detailed determination of the cross-stream water surface elevations, flow velocities or flow distribution.

   Two-dimensional models are more complex and require more time to set up and calibrate. They require essentially the same field data as a one-dimensional model and, depending on complexity, may require a little more computer time.

   The FESWMS model has been developed to analyse flow at bridge crossings where complicated hydraulic conditions exist. This two-dimensional modelling system is flexible and may be applied to many types of steady and unsteady flow problems including multiple opening bridge crossings, spur dikes, flood plain encroachments, multiple channels, flow around islands and flow in estuaries. Where the flow is essentially two- dimensional in the horizontal plane a one-dimensional analysis may lead to costly over- design or possibly improper design of hydraulic structures and improvements.

2. **Physical Modelling**

   Complex hydrodynamic situations defy accurate or practicable mathematical modelling. Physical models shall be considered when:

   a. Hydraulic performance data is needed that cannot be reliably obtained from mathematical modelling.
   b. Risk of failure or excessive over-design is unacceptable, and research is needed,
   c. The constraints on physical modelling are:
      i. Size (scale).
      ii. Cost.
      iii. Time.

3. **WSPRO Modelling**

   The water surface profile used in the hydraulic analysis of a bridge should tend from a point downstream of the bridge that is beyond the influence of the constriction to a point upstream that is beyond the extent of the bridge backwater. The cross sections that are necessary for the energy analysis through the bridge opening for a single opening bridge without spur dikes are shown in Figure 5.3. The additional cross sections that are necessary for computing the entire profile are not shown in this figure. Cross sections 1, 3, and 4 are required for a Type I flow analysis, and are referred to as the approach section, bridge section, and exit section, respectively. In addition, cross section 3F, which is called the full-valley section, is needed for the water surface profile computation without the presence of the bridge contraction. Cross-section 2 is used as a control point in Type II flow but requires no input data. Two more cross sections must be defined if spur dikes, and a roadway profile are specified.

Pressure flow through the bridge opening is assumed to occur when the depth immediately upstream of the bridge opening exceeds 1.1 times the hydraulic depth of the opening. The flow is then calculated as orifice flow with the discharge proportional to the square root of the effective head. Submerged orifice flow is treated similarly with the head redefined. WSPRO can also simultaneously consider embankment overflow as a weir discharge. This leads to flow classes I through 6 as given in Table 5.4.

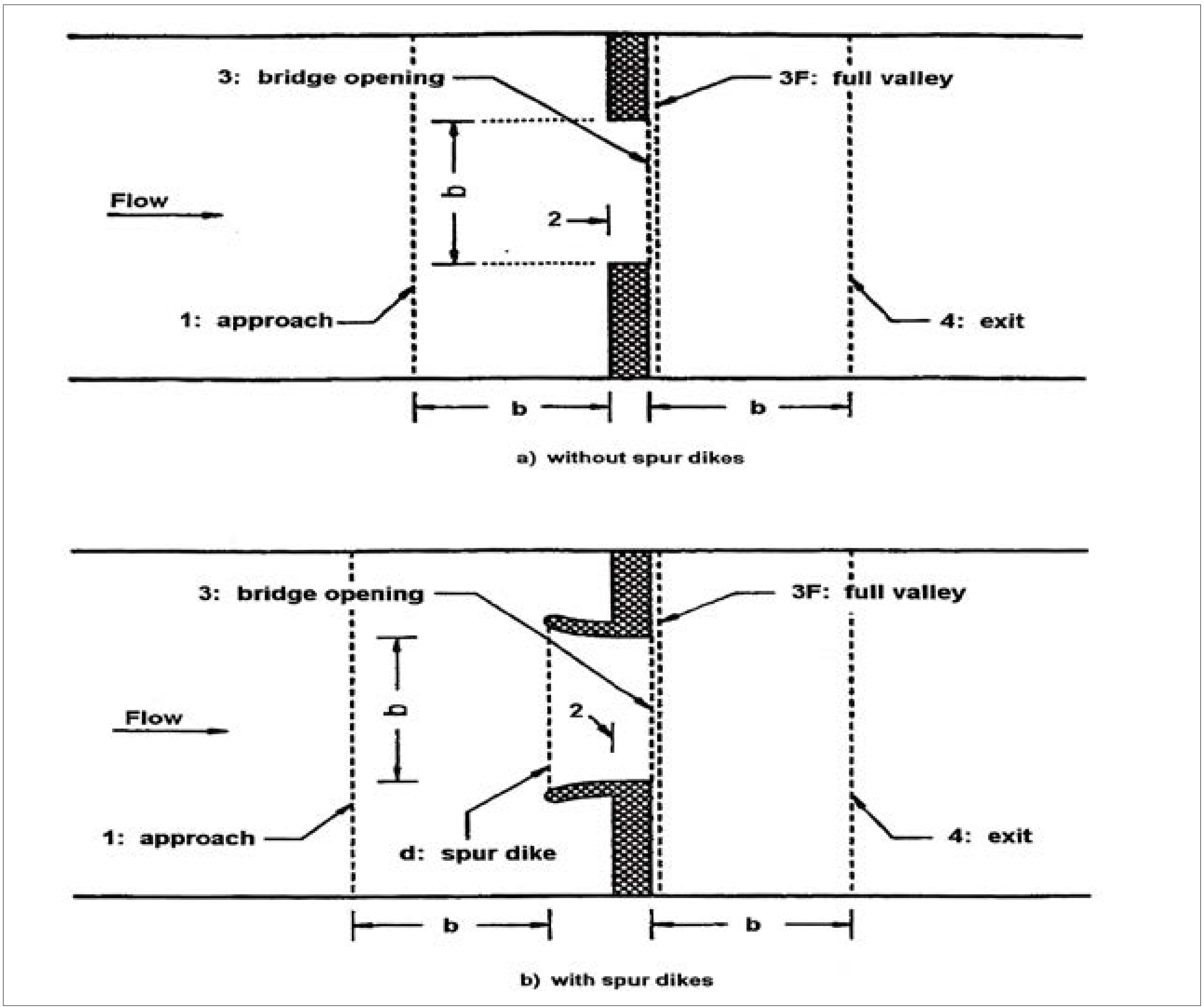
Table 5.4 Flow Classification According to Submergence Conditions

Flow Through Bridge Opening Only	Flow Through Bridge Opening and Over Road Grade
Class 1 - Free surface flow	Class 4 - Free surface flow
Class 2 - Orifice flow	Class 5 - Orifice flow
Class 3 - Submerged orifice flow	Class 6 - Submerged orifice flow

In orifice flow, only the upstream girder is submerged, while in submerged orifice flow both the upstream and downstream girders are submerged. A total of four different bridge types can be treated.

A user's instruction manual for WSPRO should serve as a source for more detailed information on using the computer model.

Figure 5.3 Cross-section Locations for Stream Crossing with a Single Waterway Opening

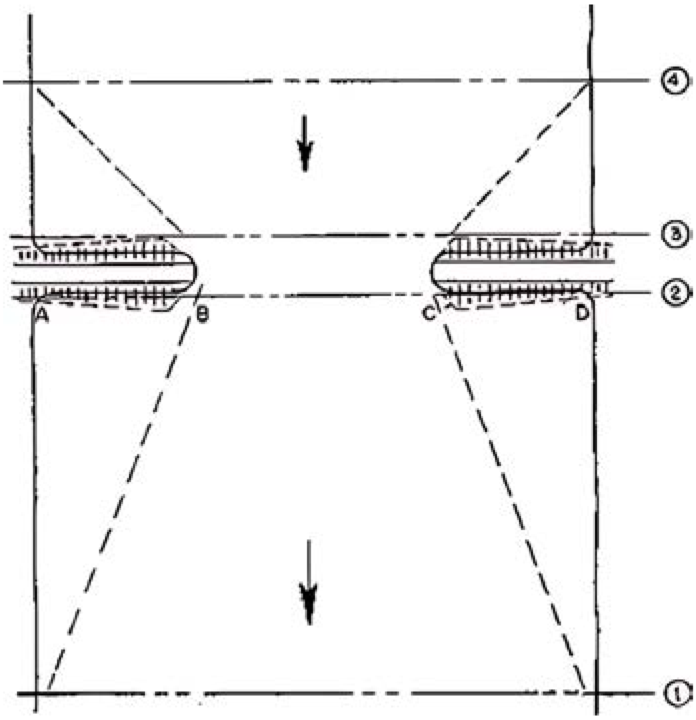


4. HEC-2 Modelling

The water surface profile used in the hydraulic analysis of a bridge should extend from a point downstream of the bridge that is beyond the influence of the constriction to a point upstream that is beyond the extent of the bridge backwater. The cross sections that are necessary for the energy analysis through the bridge opening for a single opening bridge using the special bridge option are shown in Figure 5.4.

Energy losses caused by structures such as bridges and culverts are computed in two parts. First, the losses due to expansion and contraction of the cross section on the upstream and downstream sides of the structure are computed in the standard step calculations. Secondly, the loss through the structure itself is computed by either the normal bridge or the special bridge methods. (See worked example Appendix A.9).

Figure 5.4 Cross-section Locations in the Vicinity of Bridges



5.6 Bridge Scour and Aggradation

5.6.1 General

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. The designer shall use the most appropriate scour forecasting methods.

Users of this Manual should consult HEC-18 for a more thorough treatise on scour and scour prediction methodology. A companion FHWA document to HEC-18 is HEC-20 'Stream Stability at Highway Structures'.

The inherent complexities of stream stability, further complicated by road stream crossings, requires a multilevel solution procedure. The evaluation and design of a road stream crossing or encroachment shall begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis shall be followed with quantitative analysis using basic hydrological, hydraulic and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses such as evaluation of catchment area sediment yield, incipient motion analysis and scour calculations. This analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained.

If not, a more complex quantitative analysis based on detailed mathematical modelling and/or physical hydraulic models shall be considered. This multilevel approach is presented in HEC-20.

Less hazardous perhaps are problems associated with aggradation. Where freeboard is limited, problems associated with increased flood hazards to upstream property or to the travelling public due to more frequent overtopping may occur. Where aggradation is expected, it may be necessary to evaluate these consequences. In addition, aggradation in a stream reach may serve to moderate potential scour depths. Aggradation is sometimes referred to as negative scour.

5.6.2 Scour Types

Present technology dictates that bridge scour be evaluated as interrelated components:

1. Long term profile changes (aggradation/degradation).
2. Plan form change (lateral channel movement).
3. Contraction scour/deposition.
4. Local scour.

5.6.2.1 Long Term Profile Changes

Long-term profile changes can result from streambed profile changes that occur from aggradation and/or degradation.

1. Aggradation is the deposition of bedload due to a decrease in the energy gradient.
2. Degradation is the scouring of bed material due to 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.

5.6.2.2 Plan Form Changes

Plan form changes are morphological changes such as meander migration or bank widening. The lateral movement of meanders can threaten bridge approaches as well as increase scour by changing flow patterns approaching a bridge opening. Bank widening can cause significant changes in the flow distribution and thus the bridge's flow contraction ratio.

It is difficult to anticipate when a change in planform may occur. It may be gradual or the result of a single major flood event. Also, 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 planform; however, it is important to incorporate potential planform changes into the design of new bridges and design of countermeasures for existing bridges.

Assessing the significance of planform changes, such as the shifting location of meanders, the formation of islands, and the overall pattern of streams, usually cannot be accomplished without field observations. Records and photographs taken by bridge inspectors and maintenance personnel may provide some insight into the nature of the stream for the initial assessments. Historical aerial photographs of the stream can be extremely valuable in this analysis. Ultimately, an engineering judgement must be made as to whether possible future or existing planform changes represent a hazard to the bridge, and the extent of field work required to evaluate this condition.

For a detailed discussion of this subject refer to HEC 20 'Stream stability at highway structures'.

5.6.2.3 Contraction

A constriction of the channel, which may be caused, in part, by bridge piers in the waterway, can result in channel contraction scour. Deposition results from an expansion of the channel or the bridge site being positioned immediately downstream of a steeper reach of the stream. Highways, bridges, and natural channel contractions are the most encountered cause of constriction scour. Two practices are provided in this Manual for estimating deposition or contraction scour.

1. 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.
2. 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.

The same empirical practice algorithms used in this Manual to evaluate a naturally contracting reach may also be used to evaluate deposition in an expanding reach provided armoring is not expected to occur. With deposition the practice of applying the empirical equations 'in reverse' is required; i.e., the narrower cross section is upstream which results in the need to manipulate the use of the empirical 'contraction scour' equation. This need to manipulate the intended use of an equation does not occur with the sediment routing practice, which is why it may be more reliable in an expanding reach.

5.6.2.4 Local Scour

The potential scour hazard at a bridge site is exacerbated by abutments or piers located within the flood flow prism. The amount of potential scour caused by these features is termed local scour. Local scour is a function of the geometry of these features as they relate to the flow geometry. However, the importance of these geometric variables will vary. As an example, increasing the pier or cofferdam width either through design or debris accumulation will increase the amount of local scour, but only up to a point in subcritical flow streams. After reaching this point, pier scour should not be expected to increase measurably with increased stream velocity or depth. This threshold has not been defined in the more rare, supercritical flowing streams.

5.6.2.5 Natural Armoring

Armoring occurs because a stream or river is unable, during a particular flood, to move the coarser material comprising either the bed or, if some bed scour occurs, its underlying material. Scour may occur initially but later become arrested by armoring before the full scour potential is reached again for a given flood magnitude. When armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit to form a layer of riprap like armor on the stream bed or in the scour holes and thus limit further scour for a particular discharge. This armoring effect can decrease scour hole depths which were predicted based on formulae developed for sand or other fine material channels for a particular flood magnitude. When a larger flood occurs than used to define the probable scour hole depths, scour will probably penetrate deeper until armoring again occurs at some lower threshold.

Armoring may also cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage further plan form changes that are difficult to assess. Bank widening also spreads the approach flow distribution, which in turn results in a more severe bridge opening contraction.

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5.6.2.6 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 realised 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 bedrock streams, and streams with gravel and boulder beds.

5.6.3 Scour Analysis Methods

Before the various scour forecasting methods for contraction and local scour can be applied, it is first necessary to:

1. Obtain the fixed bed channel hydraulics.
2. Estimate the profile and plan form scour or aggradation.
3. Adjust the fixed bed hydraulics to reflect these changes.
4. Compute the bridge hydraulics.

Two methods are provided in this Manual for combining the contraction and local scour components to obtain total scour. The first method identified as Method 1 shall be used when streambed armoring is of concern, more precise contraction scour estimates are deemed necessary, or deposition is expected and is a primary concern. The second method, Method 2, should have applications where armoring is not a concern or insufficient information is available to permit its evaluation, or where more precise scour estimates are not deemed necessary.

5.6.3.1 Method 1

This analysis method is based on the premise that the contraction and local scour components do not develop independently. As such, the local scour estimated with this method is determined based on the expected changes in the hydraulic variables and parameters due to contraction scour or deposition; i.e, through what may prove to be an iterative process, the contraction scour and channel hydraulics are brought into balance before these hydraulics are used to compute local scour. Additionally, with this method the effects of any armoring may also be considered.

The general approach for this method is as follows:

1. Estimate the hydraulics of the natural channel for a fixed bed condition based on existing site conditions.
2. Estimate the expected profile and plan form changes, (See sub-section 5.6.2(b), (Plan Form Changes).
3. Adjust the hydraulics of the natural channel based on the expected profile and plan form changes.
4. Select a trial bridge opening and compute the bridge hydraulics.
5. Estimate contraction scour or deposition, (See subsection 5.6.3.2).
6. Once again revise the natural channel's geometry to reflect the contraction scour or deposition changes and then again revise the channel's hydraulics (repeat this iteration until there is no significant change in either the revised channel hydraulics or bed elevation changes- a significant change would be a 5% or greater variation in velocity, flow depth or bed elevation).
7. Use the revised bridge and channel hydraulic variables and parameters, considering the contraction scour or deposition to calculate the local scour,
8. Extend the local scour assessment below the predicted contractions scour depths in order to obtain the total scour.

5.6.3.2 Method 2

This is considered a conservative practice as it assumes that the scour components develop independently. Thus, as indicated with Method 1 the potential local scour to be calculated using this method would be added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics.

The general approach with this method is as follows.

1. Estimate the hydraulics of the natural channel for a fixed bed condition based on existing conditions.
2. Assess the expected profile and plan form change, (See sub-section Plan Form Changes).
3. Adjust the fixed bed hydraulics to reflect any expected profile or plan form changes.
4. Estimate contraction scour using the empirical contraction (See subsection 5.6.3.1) and the adjusted fixed bed hydraulics assuming no bed armoring.
5. If the reach is expanding, estimate the deposition by 'reversing' the empirical equation application and considering deposition as 'negative' scour.
6. Estimate local scour using the adjusted fixed bed channel and bridge hydraulics assuming no bed armoring.
7. Add the local scour to the contraction scour or aggradation ("negative" scour) to obtain the total scour.

5.6.4 Scour Assessment Procedure

Bridge scour assessment should normally be accomplished by collecting the data and applying the general procedure outlined in this section. It is generally accomplished using computer software, and worked examples are given as a portion of the manuals for HEC- 2, HEC RAS 3, and other software.

5.6.4.1 Site Data

The data to be obtained from the site include the following:

i. **Bed Material**

Obtain bed material samples for all channel cross sections when armoring is to be evaluated. If armoring is not being evaluated, this information need only be obtained at the site. From these samples try to identify historical scour and associate it with a discharge. Also, determine the bed material size-weight distribution curve in the bridge reach and from this distribution determine D_{16}, D_{50}, D_{84}, and D_{90}.

ii. **Geometry**

Obtain existing stream and floodplain cross sections, stream profile, site plan and the stream's present, and where possible, historic geomorphic plan form. Also, locate the bridge site with respect to such things as other bridges in the area, tributaries to the stream or close to the site, bedrock controls, manmade controls (dams, old check structures, river training works, etc.), and downstream confluence with other streams. Locate (distance and height) any “headcuts” due to natural causes or such things as gravel mining operations. Upstream gravel mining operations may absorb the bed material discharge resulting in the more adverse clear water scour case discussed later. Any data related to plan form changes such as meander migration and the rate at which they may be occurring are useful.

iii. **Historic Scour**

Any scour data on other bridges or similar facilities along the stream.

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iv. Hydrology

Identify the character of the stream hydrology, i.e., perennial, ephemeral, intermittent as well as whether it is subject to broad hydrographic peaks resulting from gradual flow increases such as occur with general thunderstorms.

v. Geomorphology

Classify the geomorphology of the site, i.e., such things as whether it is a floodplain stream, crosses a delta, or crosses an alluvial fan; youthful, mature or old age.

5.6.4.2 Design Procedure Step by Step

The steps in the design procedure are as follows:

Step 1: Choice of Method

Decide which analysis method is applicable. Method 1 shall be used to evaluate bridges where armoring or an expanding reach are of concern as well as where Method 2 indicates a significant potential scour hazard may exist. Method 2 shall be used to quickly evaluate existing bridges to identify significant potential scour hazards or, where armoring or an expanding reach are obviously not of concern, on a proposed bridge.

Step 2 : Flood Determination

Determine the magnitude of the base flood as well as the magnitude of the incipient overtopping flood, or relief opening flood. Accomplish steps 3 through 12 using the discharge that places the greatest stress on the bed material in the bridge opening.

Step 3: Determination of Bed Material Size

Determine the bed material size that will resist movement and cause armoring to occur.

Step 4: Definition of Water Surface Profile

Develop a water surface profile through the site's reach for fixed bed conditions using, HEC RAS-4, or other software.

Step 5: Assessment of Expected Changes in The Vertical Stream Profile

Assess the bridge crossing reach of the stream for profile bed scour changes to be expected from degradation or aggradation. Again, take into account past, present and future conditions of the stream and catchment area in order to forecast what the elevation of the bed might be in the future. Certain plan form changes such as migrating meanders causing channel cut-offs would be important in assessing future streambed profile elevations. The possibility of downstream mining operations inducing 'headcuts' shall be considered. The quickest way to assess streambed elevation changes due to 'headcuts' (degradation) is by obtaining a vertical measurement of the downstream 'headcut(s)' and projecting that measurement(s) to the bridge site using the existing stream profile assuming the stream is in regime; if it is not, then it may be necessary to estimate the regime slope. A more time-consuming way to assess elevation changes would be to use some form of sediment routing practice in conjunction with a synthetic flood history.

Step 6 : Assessment of Expected Changes in Horizontal Stream Profile

Assess the bridge crossing reach of the stream for plan form scour changes. Attempt to forecast whether an encroaching meander will cause future problems within the expected service life of the road or bridge. Take into account past, present and expected future conditions of the stream and catchment area in order to forecast how such meanders might influence the approach flow direction in the future. The sediment routing practice discussed later for computing channel contraction scour or aggradation may prove useful in making such assessments - particularly if coupled to a synthetic flood history. This forensic analysis on a site's past geomorphologic history to forecast the future may prove useful. Otherwise, this assessment will be largely subjective in nature.

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Step 7: Adjustment of Parameters

Based on the expected profile and plan form scour changes, adjust the fixed bed hydraulic variables and parameters.

Step 8: Assessment of Scour Magnitude

Assess the magnitude of channel or bridge contraction scour using Method I or Method 2 based on the fixed bed hydraulics of Step 7.

Step 9: Assessment of local Scour

Assess the magnitude of local scour at abutments and piers using Method 1 or 2.

Step 10: Plotting of Results

Plot the scour and aggradation depths from foregoing steps on a cross section of the stream channel and flood plain at the bridge site. Treat any aggradation as a negative scour. Enlarge any overlapping scour holes. The top width of a local scour hole ranges from 1.0 to 2.8 times the depth of scour. A top width value of 2.0 times the depth of scour is suggested for practical applications.

Step 11: Evaluation of Findings

Evaluate the findings of Step 10. If the scour is unacceptable, consider the use of scour countermeasures or revise the trial bridge opening and repeat the foregoing steps.

Step 12: Preliminary Foundation Design

Once an acceptable scour threshold is determined, the geotechnical engineer can make a preliminary foundation design for the bridge based on the scour information obtained from the foregoing procedure using commonly accepted safety factors. The structural engineer should evaluate the lateral stability of the bridge based on the foregoing scour.

Step 13: Design Test on 'Super flood'

Repeat the foregoing assessment procedures using the greatest bridge opening flood discharge associated with Table 5.5 below. These findings are again for the geotechnical engineer to use in evaluating the foundation design obtained in Step 12. A foundation design safety factor of 1.1 is commonly used to ensure that the bridge is marginally stable for a flood associated with the 'super flood'.

Table 5.5 Guidelines for Scour Design Flood Frequencies

Hydraulic Design Flood Frequency, Q_D	Scour Design Flood Frequency, Q_S	Scour Design Check Flood Frequency, Q_C
Q_{10}	Q_{25}	Q_{50}
Q_{25}	Q_{50}	Q_{100}
Q_{50}	Q_{100}	Q_{200}
Q_{100}	Q_{200}	Q_{500}

The procedure is summarised in flowchart Figure 5.20 at the end of this section.

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5.6.5 Scour Assessment Equations

5.6.5.1 Contraction Scour

Four conditions (cases) of contraction scour are commonly encountered:

Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river.

b. No contraction of the main channel, but the overbank flow area is completely obstructed by the road embankments.

c. Abutments are set back away from the main channel.

Case 2. Flow is confined to the main channel (i.e., there is no overbank flow). The normal river channel width becomes narrower due to the bridge itself or the bridge site is located at a narrowing reach of the river.

Case 3. A relief bridge in the overbank area with little or no bed material transport in the overbank area (i.e., clear-water scour).

Case 4. A relief bridge over a secondary stream in the overbank area with bed material transport (like case one).

Contraction scour can be evaluated using two basic equations: (1) **clear-water** equation (Equation 5.6), and (2) the **live-bed** scour equation (Equation 5.7).

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material being considered for movement and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening.

If $V_c > V$, then a clear-water contraction scour will exist. If $V_c < V$), then live-bed contraction scour will exist.

$$\text{Critical velocity, } V_c = K_u\, y^{1/6} D^{1/3}$$

Equation 5.5

Where,

V_c = Critical velocity above which bed material of size D and smaller, will be transported, (m/s).

y = Average depth of flow upstream of the bridge, (m).

D = Particle size for V_c (m).

D_{50} = Particle size in a mixture of which 50 percent are smaller, (m).

K_u = 6.19.

Where Live-bed contraction scour depths are limited by armoring of the bed by large sediment particles in the bed material or by sediment transport of the bed material into the bridge cross-section, live-bed contraction scour at a bridge can be determined by calculating the scour depths using both the clear-water and live-bed contraction scour equations and using the smaller of the two depths.

a. Clear-water Contraction scour

The recommended clear-water contraction scour equation is based on a development suggested by Laursen (1963).

$$y_2 = \left[\frac{K_u\, Q^2}{D_m^{\,2/3} W^2}\right]^{3/7}$$

Equation 5.6

$y_s = y_2 - y_o$ = (average contraction scour depth).

Where,

y_2 = Average equilibrium depth in the contracted section after contraction scour, (m).

Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, m³/s.

D_m = Diameter of the smallest non-transportable particle in the bed material (1.25 D_{50}) in the contracted section, (m).

D_{50} = Median diameter of bed material, (m) (a reasonable lower limit of D_{50} equal to 0.2 mm can be applied to this equation).

W = Bottom width of the contracted section less pier widths, (m).

y_o = Average existing depth in the contracted section, (m).

K_u = 0.025.

b. Live-bed Contraction Scour

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section (Laursen 1960). The equation assumes that bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left[\frac{Q_2}{Q_1}\right]^{6/7}\left[\frac{W_1}{W_2}\right]^{k_1}$$

Equation 5.7

$y_s = y_2 - y_o$ (average contraction scour depth)

Equation 5.8

Where,

y_1 = Average depth in the upstream main channel, m.

y_2 = Average depth in the contracted section, m.

y_0 = Existing depth in the contracted section before scour, m (see Note 7).

Q_1 = Flow in the upstream channel transporting sediment (not including overbank flows), associated with the width W, m³/s.

Q_2 = Flow in the contracted channel, (m³/s).

D_m = Diameter of the smallest non transportable particle in the bed material (1.25 D_{50}) in the contracted section, (m).

D_{50} = Median diameter of bed material, (m).

W_1 = Bottom width of the contracted section less pier widths, (m).

W_2 = Bottom width of main channel in contracted section less pier width(s), (m).

k_1 = Exponent determined below:

V^*/T	k_1	Mode of Bed Transport
< 0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

Where,

V^* = $(\partial o/\gamma w)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section, (m/s).

T = Fall velocity of bed material based on the D50, m/s (Figure 5.5).

g = Acceleration of gravity 9.81 m/s².

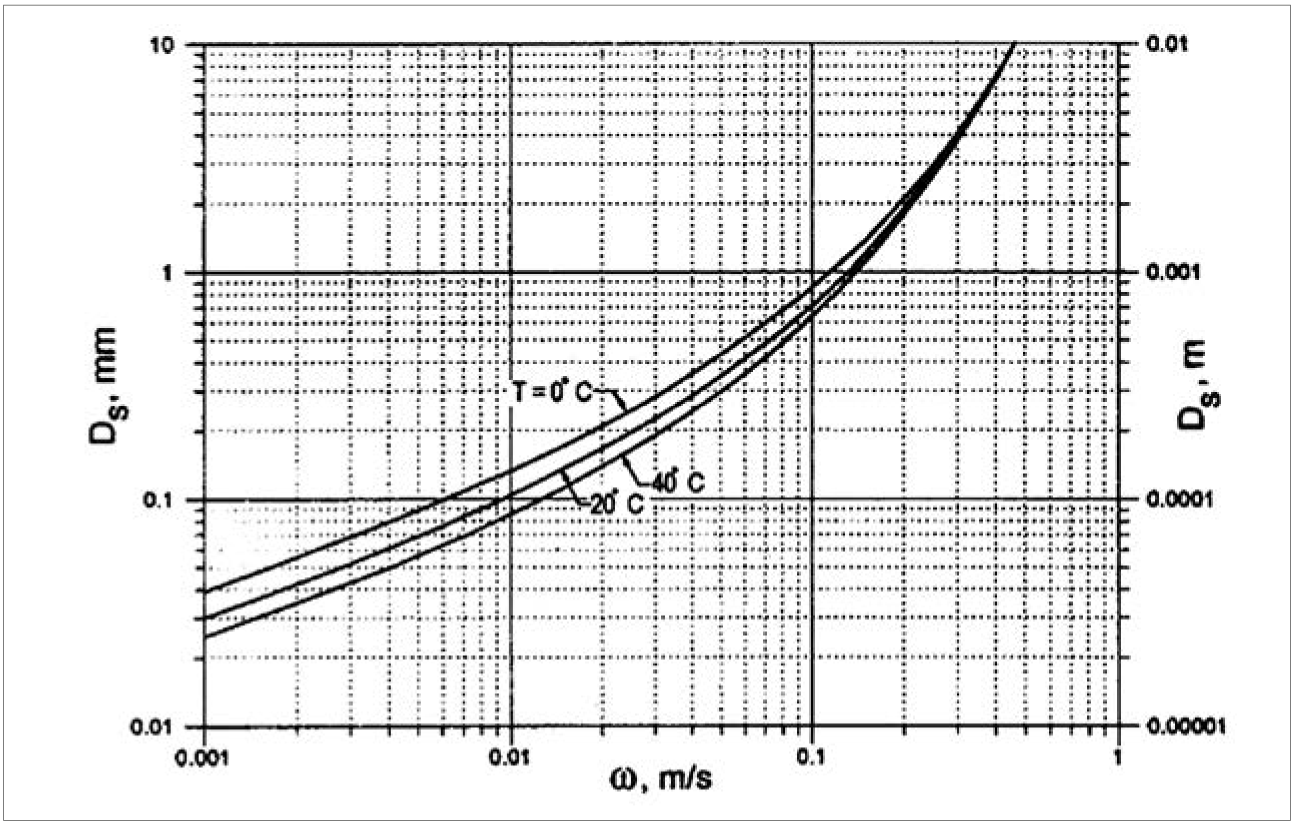
S_1 = Slope of energy grade line of main channel, m/m.

∂o = Shear stress on the bed, Pa N/m².

γ_w = Density of water (1,000 kg/m³).

ω = Fall velocity of the bed material of a given size, m/s.

Figure 5.5 Fall Velocity of Sand-Sized Particles with Specific Gravity of 2.65.



c. **Contraction Scour with Backwater**

Both equations calculate contraction scour depth assuming a level water surface ($y_s = y_2 - y_o$). A more consistent computation would be to write an energy balance before and after the scour. For live-bed the energy balance would be between the approach section (1) and the contracted section (2). Whereas, for clear-water scour it would be the energy at the same section before (1) and after (2) the contraction scour.

d. **Contraction Scour In Cohesive Materials**

The live-bed and clear-water contraction scour equations presented in sub-section 5.6.5.1 are developed for cohesionless sediments and provide estimates of scour for a hydraulic condition sufficient to produce ultimate scour. For silts and clays the critical shear stress (V_c) increases due to cohesion. Figure 5.6 shows that grain size and critical shear are well correlated for sand and gravel sizes.

Ultimate Scour (Equation 5.9), is based on analysis on laboratory data (Briaud et al. 2011):

Ultimate scour can be calculated for a particular hydraulic condition once the critical shear is known.

$$y_s\text{ - }ult = 0.94y_1\left[\frac{1.83V_2}{\sqrt{gy_1}} - \frac{K_u\sqrt{\frac{\tau_c}{\rho_w}}}{gny_1^{1/3}}\right]$$

Equation 5.9

Where,

y_1 = Upstream average flow depth (m).

V = Average flow velocity in the contracted section (m/s).

τ_c = Critical shear stress (N/m^2).

n = Manning n.

K_u = 1.

ρ_w = Specific gravity of water.

This equation computes the centreline scour downstream of the entrance. The centreline scour in the vicinity of the entrance is 35 percent greater.

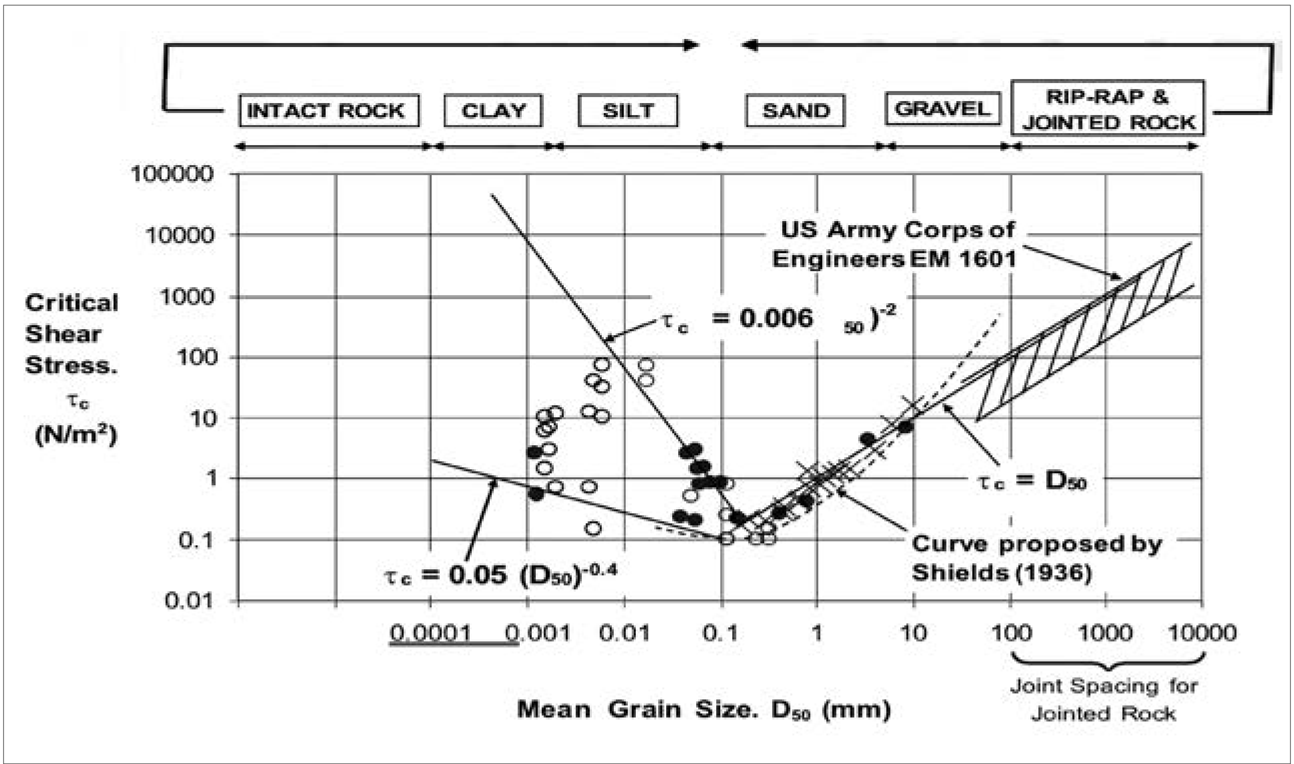
For a specific flow, the initial shear stress can be computed from:

$$\tau = \gamma \left(\frac{nV_2}{K_u}\right) y_0^{-1/3}$$

Equation 5.10

Figure 5.6 shows that grain size and critical shear are well correlated for sand and gravels sizes.

Figure 5.6 Critical Shear Stress Versus Particle Size (Briaud et al. 2011)



(The only reliable way of determining critical shear for silt and clay particles is to perform materials testing)

e. Estimating Pressure Flow Scour (Vertical Contraction Scour)

Discharge under the superstructure can be conservatively assumed to be all approach flow below the top of the superstructure at height $h_b + T$, where h_b is the vertical size of the bridge opening prior to scour and T is the height of the obstruction including girders, deck. and parapet.

The depth at the location of maximum scour is comprised of three components:

1. h_c, the vertically contracted flow height from the streamline bounding the separation zone under the superstructure at the maximum scour depth.
2. y_s, the scour depth, and
3. t, the maximum thickness of the flow separation zone. The separation zone does not convey any net mass from the upstream opening of the bridge to the downstream exit.

The pressure scour depth ys is determined by using the horizontal contraction scour equations to calculate the height, $y_s + h_c$, required to convey flow through the bridge opening at the critical velocity. This height is equivalent to y_2 (the average depth in the contracted section) in the clear-water contraction scour Equation 5.6 and the live-bed contraction scour Equation 5.8. Combining this relation with the definitions of t and hb:

$$y_s = y_2 + t - h_b$$

Equation 5.11

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Use Equation 5.6 to calculate y_2 for clear-water conditions and Equation 5.8 to calculate y_2 for live-bed conditions. When flow overtops the bridge or approach roadway, the value of Q_2 (flow in the contracted channel) in the live-bed equation (Equation 5.8) or Q (discharge through the bridge) in the clear-water equation (Equation 5.6) should include only the flow through the bridge opening. This discharge is obtained from hydraulic models such as HEC-RAS or FST2DH.

For live-bed applications, the upstream channel discharge, Q_1 and channel flow depth, y_1, used in Equation 5.8 may also need to be adjusted.

For non-overtopping flows Q_1 is not adjusted and $y_1 = h_{ue} = h_u$.

For overtopping flows illustrated in Figure 5.7, Q_1 is adjusted and y_1 = hue = $h_b + T$, where T is the height of the obstruction including girders, deck, and parapet.

If the bridge consists of railing with openings, the blockage height T extends up to the lower edge of the opening under the railing. The potential for debris blocking openings in the railing should be considered when determining T. For overtopping flows in live-bed conditions, Q_{ue} is used for Q_1 in Equation 5.8 and is calculated from the total channel discharge at the approach, Q_1, from:

$$Q_{ue} = Q_1\left(\frac{h_{ue}}{h_u}\right)^{8/7}$$

Equation 5.12

Where,

Q_{ue} = Effective channel discharge for live-bed conditions and bridge overtopping flow, m³/s.

Q_1 = Upstream channel discharge (m³/s) as defined for Equation 5.8.

H_u = Upstream channel flow depth (m) as defined for Equation 5.8.

H_{ue} = Effective upstream channel flow depth for live-bed conditions and bridge overtopping, m.

Where,

The separation zone thickness, t, is calculated using Equation 5.13:

$$\frac{t}{h_b} = 0.5\left(\frac{h_b.h_t}{h_u^2}\right)^{0.2}\left(1 - \frac{h_w}{h_t}\right)^{-0.1}$$

Equation 5.13

Where,

h_b = Vertical size of the bridge opening prior to scour, m.

h_t = Distance from the water surface to the lower face of the bridge girders, equals h_u - h_b m.

h_w = Weir flow height = h_t - T for $h_t > T$, $h_w = 0$ for $h_t \leq T$.

t = The height of the obstruction including girders, deck, and parapet.

Figure 5.7 Vertical Contraction and Definition for Geometric Parameters

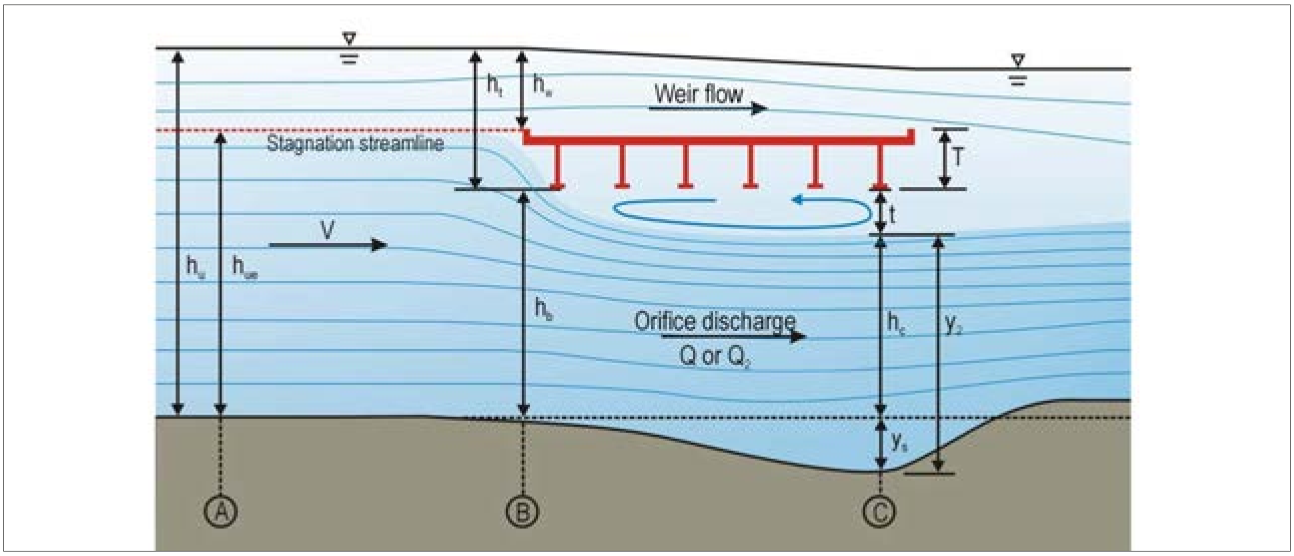


Figure 5.8 Definition Sketch for Pier Scour

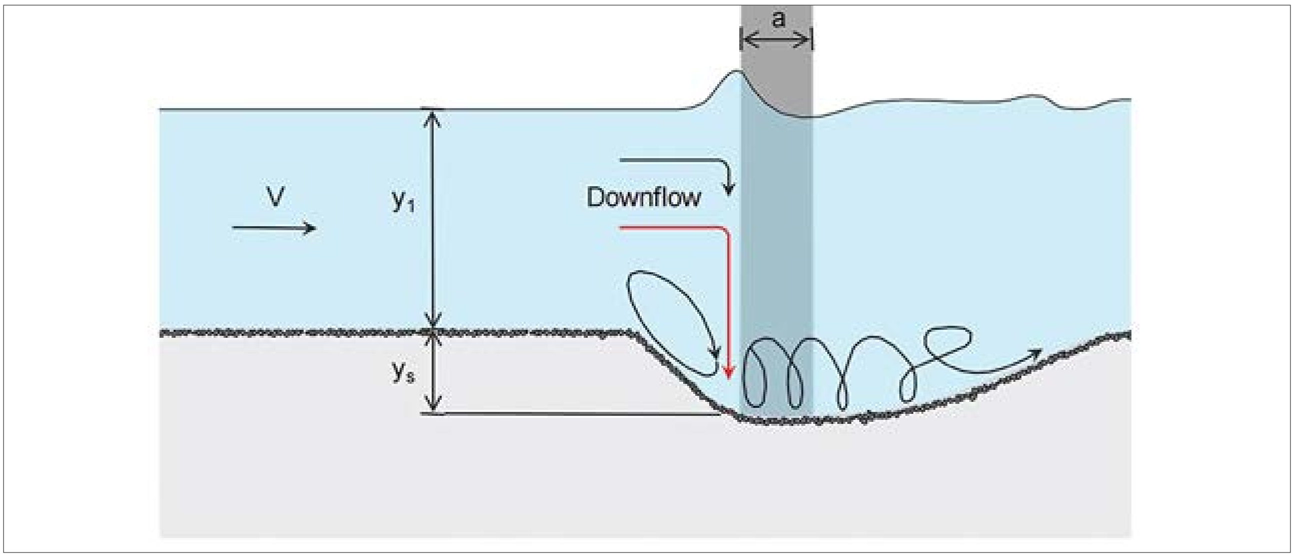
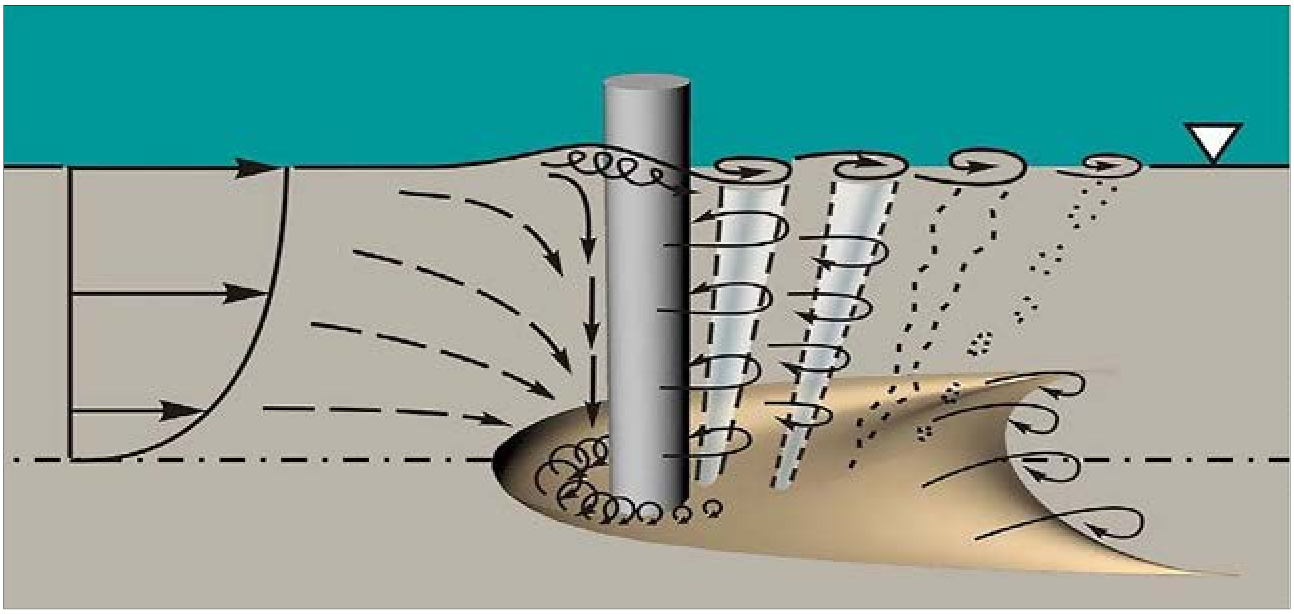


Figure 5.9 The Main Flow Features Forming the Flow Field at a Narrow Pier of Circular Cylindrical Form



Source: NCHRP 2011a

The factors that affect the depth of local scour at a pier are: velocity of the flow just upstream of the pier; depth of flow; width of the pier; length of the pier if skewed to the flow; size and gradation of bed material; angle of attack of approach flow; shape of the pier; bed configuration; and the formation of ice jams and debris.

a. The HEC-18 Pier Scour Equation

The HEC-18 pier scour equation (based on the CSU equation) is recommended for both live- bed and clear-water pier scour. The equation predicts maximum pier scour depths.

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{\,0.43}$$

Equation 5.14

The maximum scour depth for round nose piers aligned with the flow is:

$y_s \leq 2.4$ times the pier width (a) for $Fr \leq 0.8$

$y_s \leq 3.0$ times the pier width (a) for $Fr > 0.8$

In terms of y_s/a, Equation 5.14 is:

$$\frac{y_s}{a} = 2.0K_1 K_2 K_3 \left(\frac{a}{y_1}\right)^{0.35} Fr_1^{\ 0.43}$$

Equation 5.15

$$y_s = 2.0K_1 K_2 K_3 K_4 \, a^{0.65} y_1^{\ 0.35} Fr_1^{\ 0.65}$$

Equation 5.16

Where,

y_s = Depth of scour in metres.
K_1 = Correction factor for pier nose shape.
K_2 = Correction factor for angle of attack of flow of flow.
K_3 = Correction factor for bed condition.
K_4 = Correction factor for armoring of bed material.
a = Pier width in metres.
y_1 = Flow depth directly upstream of the pier in metres.
Fr_1 = Froude Number directly upstream of the pier, = $V_1/(gy_1)^{1/2}$.
L = Length of pier, m.
V_1 = Mean velocity of flow directly upstream of the pier, m/s.
g = Acceleration of gravity 9.81 m/s^2.

The correction factors K_1 and K_2 are given in Table 5.6 and Table 5.7 respectively.

Table 5.6 Correction Factor K_1, for Pier Nose Shape

Shape of Pier in Plan	Length/Width Ration (L/b)	K_1
Circular	1	1
Lenticular	2	0.91
	3	0.76
	4	0.67 - 0.73
	7	0.41
Parabolic Nose		0.8
Triangular 60		0.75
Triangular 90		1.25
Elliptic	2	0.91
	3	0.83
Oval	4	0.86 - 0.92
Rectangular	2	1.11
	4	1.11 (HEC 18) - 1.40 (F&C)
	6	1.11

Table 5.7 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	1	1
15	1.5	2	2.5
30	2	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5

The correction factor, K_2, for angle of attack of the flow, K, is calculated using the following equation:

$$K = (Cos\ \theta + \frac{L}{a} Sin\ \theta)^{0.65}$$

Equation 5.17

Table 5.8 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	0.6m-3m	1.1
Medium dunes	3m - 9m	1.1 - 1.2
large dunes	≥ 9m	1.3

The value of correction factor for armouring, K_4, can be determined from the following sets of equations:

$$K_4 = \{1 - 0.89(1-V_R)^2\}^{0.5}$$

Equation 5.18

Where,

$$V_R = \frac{V_1 - V_i}{V_{c90} - V_i}$$

Equation 5.19

and,

$$V_i = 0.645 \left(\frac{D_{50}}{b}\right)^{0.053} V_{c50}$$

Equation 5.20

with,

VR = 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 D90 bed material size (m/s).

V_{c50} = Critical velocity for D50 bed material size (m/s).

b = Pier width (m).

and,

$$V_i = 0.645 \left(\frac{D_{50}}{b}\right)^{0.053} V_{c50}$$

Equation 5.21

Where,

D_c = Critical particle size for the critical velocity V_c (m).

V_c = 6.19 $y^{1/6}$ $D_c^{1/3}$

Equation 5.22

Correction Factor for Scour at Wide Piers

K_w factor should be used to correct Equation 5.14 or 5.15 for wide piers in shallow flow. The correction factor should be applied when the ratio of depth of flow (y) to pier width (a) is less than 0.8 (y/a < 0.8); the ratio of pier width (a) to the median diameter of the bed material (D_{50}) is greater than 50 (a/D_{50} > 50); and the Froude Number of the flow is subcritical.

$$K_w = 2.58 \left(\frac{y}{a}\right)^{0.34} F_{r1}^{0.65} \qquad \text{for } V/V_c < 1$$

Equation 5.23

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$$K_w = 1.0 \left(\frac{y}{a}\right)^{0.13} F_{r1}^{\ 0.25} \qquad \text{for } V/V_c \geq 1$$

Equation 5.24

Where,

K_w = Correction factor to Equation 5.14 or 5.15 for wide piers in shallow flow. The other variables as previously defined.

b. Scour for Complex Pier Foundations

Complex pier foundations include:

1. Pile groups.
2. Pile groups and pile caps.
3. Pile groups, pile caps and solid piers.

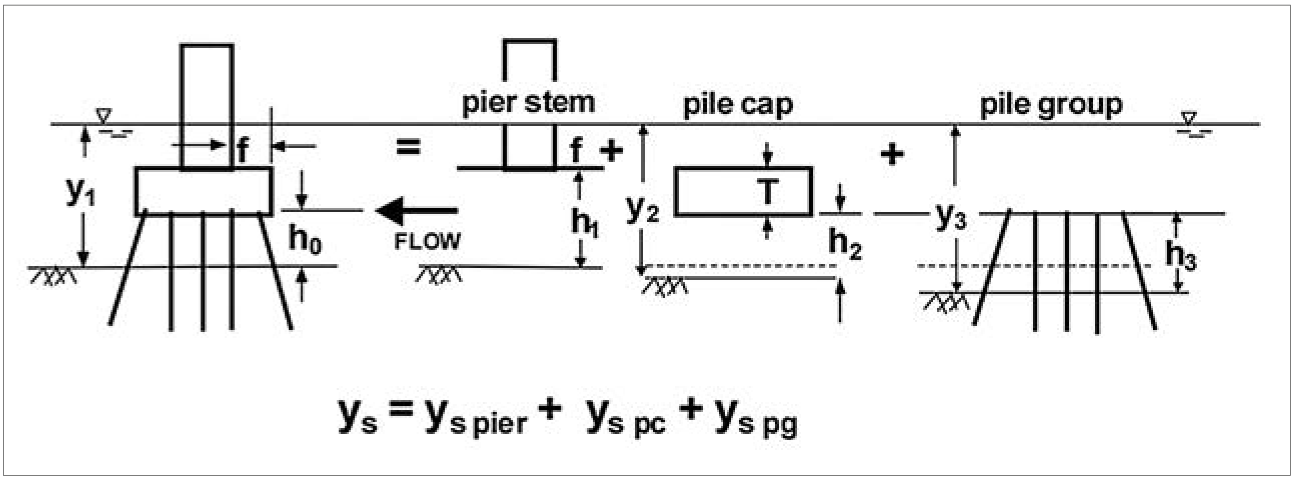
The exposure of these components to the flow would be:

1. By design – in this case, the scour is incorporated in the design and the exposure of the piles is expected and catered for in the design.
2. Due to scour mechanisms – these include long-term scour, general (contraction) scour and local scour.
3. Due to stream migration – this is a result of aggradation and degradation of the stream laterally.

Scour is caused by components of the substructure, which include the pier stem, the footing/pile cap and the pile group.

The steps listed below are recommended for determining the depth of scour for any combination of the three sub-structural elements exposed to the flow. However, engineering judgement is an essential element in applying the design graphs and equations presented in this section as well as in deciding when a more rigorous level of evaluation is warranted.

Figure 5.10 Scour Components of a Complex Pier



The configuration of these sub-structural elements or components and the parameters necessary to determine the scour are given in Figure 5.10.

Where,

y_1 = Approach flow depth at the beginning of the calculations (m), i.e, before scour.

y_2 = $y_1 + y_{s\ pier}/2$ = the adjusted flow depth for pier cap calculations (m).

y_3 = $y_1 + y_{s\ pier}/2 + y_{s\ pc}/2$ = adjusted flow depth for pile group calculations (m).

v_1 = Approach velocity used at the beginning of calculations (m/s).

v_2 = $v_1(y_1/y_2)$ = adjusted velocity for pile cap calculations (m/s).

v_3 = $v_1(y_1/y_3)$ = adjusted velocity for pile group calculations (m/s).
f = Distance between front edge of footing/pile cap and pier (m).
h_0 = Height of pile cap above bed at beginning of calculations (m).
h_1 = $h_0 + T$ = height of pier stem above the bed before scour (m).
h_2 = $h_0 + y_{s\ pier}/2$ = height if pile cap after pier stem scour component has been calculated (m).
h_3 = $h_0 + y_{s\ pier}/2 + y_{s\ pc}/2$ = height of pile group after pier stem and pile cap scour components have been calculated (m).
S = Spacing of piles (centre to centre) (m).
T = Thickness of pile cap or footing (m).

Total scour from superposition of components is given by:

$$y_s = y_{s\,pier} + y_{s\,pc} + y_{s\,pg}$$

Equation 5.25

Where,

y_s = Total scour depth (m).
$y_{s\ pier}$ = Scour component for the pier stem in the flow (m).
$y_{s\ pc}$ = Scour component for the pier cap/footing in the flow (m).
$y_{s\ pg}$ = Scour component for the pile group in the flow (m).

The configuration of these substructural elements or components and the parameters necessary to determine the scour are given in Figure 5.10.

Where,

y_1 = Approach flow depth at the beginning of the calculations (m), i.e, before scour.
y_2 = $y_1 + y_{s\ pier}/2$ = the adjusted flow depth for pier cap calculations (m).
y_3 = $y_1 + y_{s\ pier}/2 + y_{s\ pc}/2$ = adjusted flow depth for pile group calculations (m) .
v_1 = approach velocity used at the beginning of calculations (m/s).
v_2 = $v_1(y_1/y_2)$ = adjusted velocity for pile cap calculations (m/s).
v_3 = $v_1(y_1/y_3)$ = adjusted velocity for pile group calculations (m/s).
f = distance between front edge of footing/pile cap and pier (m).
h_0 = height of pile cap above bed at beginning of calculations (m).
h_1 = h_0 + T = height of pier stem above the bed before scour (m).
h_2 = $h_0 + y_{s\ pier}/2$ = height if pile cap after pier stem scour component has been calculated (m)
h_3 = $h_0 + y_{s\ pier}/2 + y_{s\ pc}/2$ = height of pile group after pier stem and pile cap scour components have been calculated (m).
S = Spacing of piles (centre to centre) (m).
T = Thickness of pile cap or footing (m).

Total scour from superposition of components is given by:

$$y_s = y_{s\,pier} + y_{s\,pc} + y_{s\,pg}$$

Equation 5.26

Where,

y_s = Total scour depth (m).
$y_{s\ pier}$ = Scour component for the pier stem in the flow (m).
$y_{s\ pc}$ = Scour component for the pier cap/footing in the flow (m).
$y_{s\ pg}$ = Scour component for the pile group in the flow (m).

c. Determination of the Pier Stem Scour Depth Component

The pier stem scour component, y_s pier is given by:

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0K_1\,K_2\,K_3\left(\frac{a_{\,pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{gy_1}}\right)^{0.43}\right]$$

Equation 5.27

The quantity in the square brackets in Equation 5.27 is the basic pier scour ratio as if the pier stem were full depth and extended below the scour.

Where,

$K_{h\,pier}$ = Coefficient to account for height of pier stem above bed and shielding effect by pier cap/ footing overhanging distance (f) in front of the pier stem.

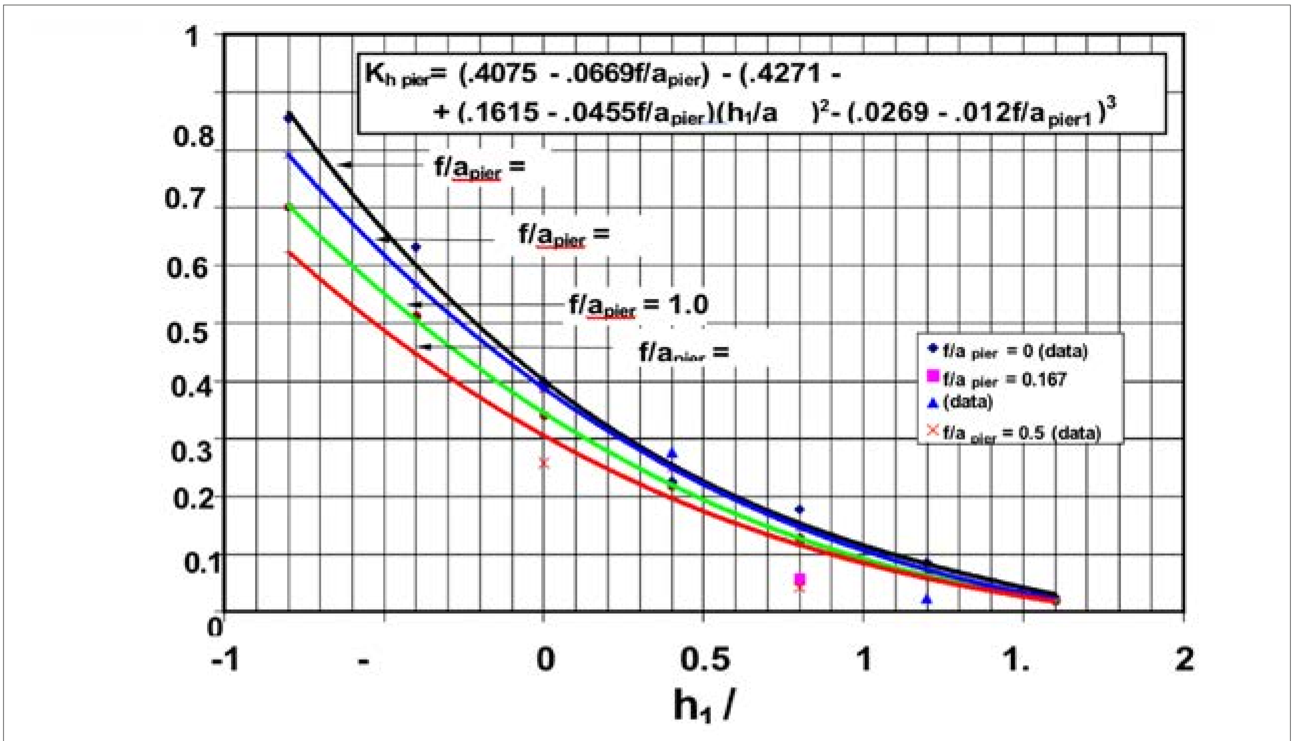
a_{pier} = Effective pier width (m).

Values of $K_{h\,pier}$ are calculated from Equation 5.28 and can also be read from Figure 5.11.

$$K_{h\,pier} = \frac{0.4075\text{-}0.0669f}{a_{\,pier}} - \frac{0.4271\text{-}0.0778f}{a_{\,pier}}\;\frac{h_1}{a_{\,pier}} + \frac{0.1615\text{-}0.0455f}{a_{\,pier}}\left(\frac{h}{a_{\,pier}}\right)^2 - \frac{0.0269\text{-}0.012f}{a_{\,pier}}\left(\frac{h}{a_{\,pier}}\right)^3$$

Equation 5.28

Figure 5.11 Suspended Pier Scour Ratio



d. Determination of the Pile Cap (Footing) Scour Depth Component

In determining the scour depth component 2 scenarios should be considered.

Case 1: The bottom of pier cap/footing is above riverbed by design or long-term scour – the approach in calculating scour is to reduce the width of the pile cap, b_{pc} to an equivalent full depth solid pier width b^*_{pc}. Once this is done, the pier scour Equation 5.29 can be used. The value of b^*_{pc} is obtained from Figure 5.12 or calculated using Equation 5.30. Apply b^*_{pc} in Equation 5.27 to determine pier cap scour depth component.

$$\frac{a^*_{pc}}{a_{pc}} = EXP\left[\text{-}2.705 + 0.51Ln\,\frac{T}{y_2} - 2.783\left(\frac{h_2}{y_2}\right)^3 + \frac{1.751}{EXP\,\dfrac{h_2}{y_2}}\right]$$

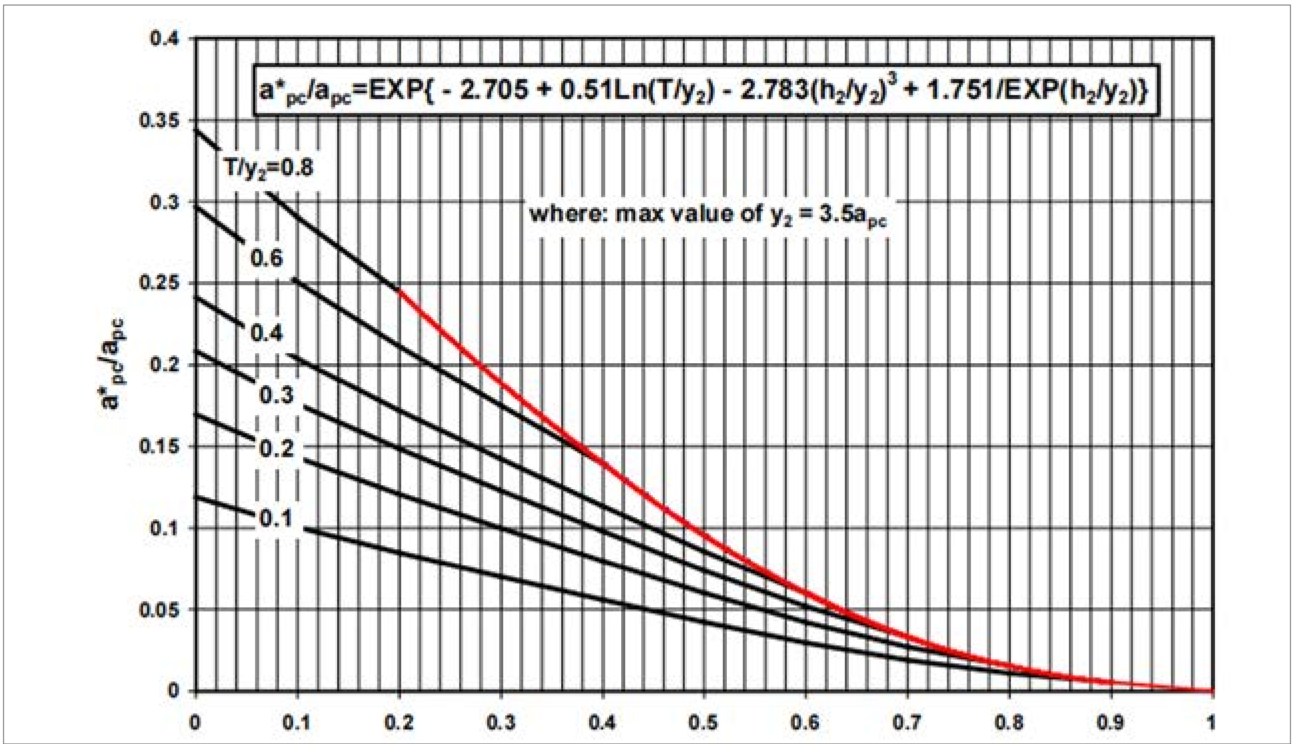
Equation 5.29

$$\frac{y_{s\,pc}}{y_2} = 2.0\,K_1\,K_2\,K_3\,K_W\left(\frac{b^*_{pc}}{y_2}\right)^{0.65}\left(\frac{V_2}{\sqrt{g y_2}}\right)^{0.43}$$

Equation 5.30

K_1, K_2 and K_3 can be obtained from Table 5.6, Table 5.7, and Table 5.8 respectively. For skewed flow use L/b for the pier cap as the L/b for the equivalent pier values of K_w refer to Equations 5.23 and 5.24.

Figure 5.12 Pier Cap (Footing) Equivalent Width



As mentioned earlier, engineering judgement is required in determining K_w because it was developed from field data.

Case 2: The bottom of the pile cap/footing is below the riverbed.

Determine the flow velocity at the footing or pile cap (V_f), which is given by Equation 5.31. The configuration of flow is given in Figure 5.13.

$$\frac{V_f}{V_2} = \frac{L_n\left(10.93\,\frac{y_f}{k_s} + 1\right)}{L_n\left(10.93\,\frac{y_2}{k_s} + 1\right)}$$

Equation 5.31

Where,

V_f = The average flow velocity in the flow zone below the top of the footing/pile cap (m/s).

V_2 = Average adjusted velocity in the vertical of flow approaching the pier (m/s).

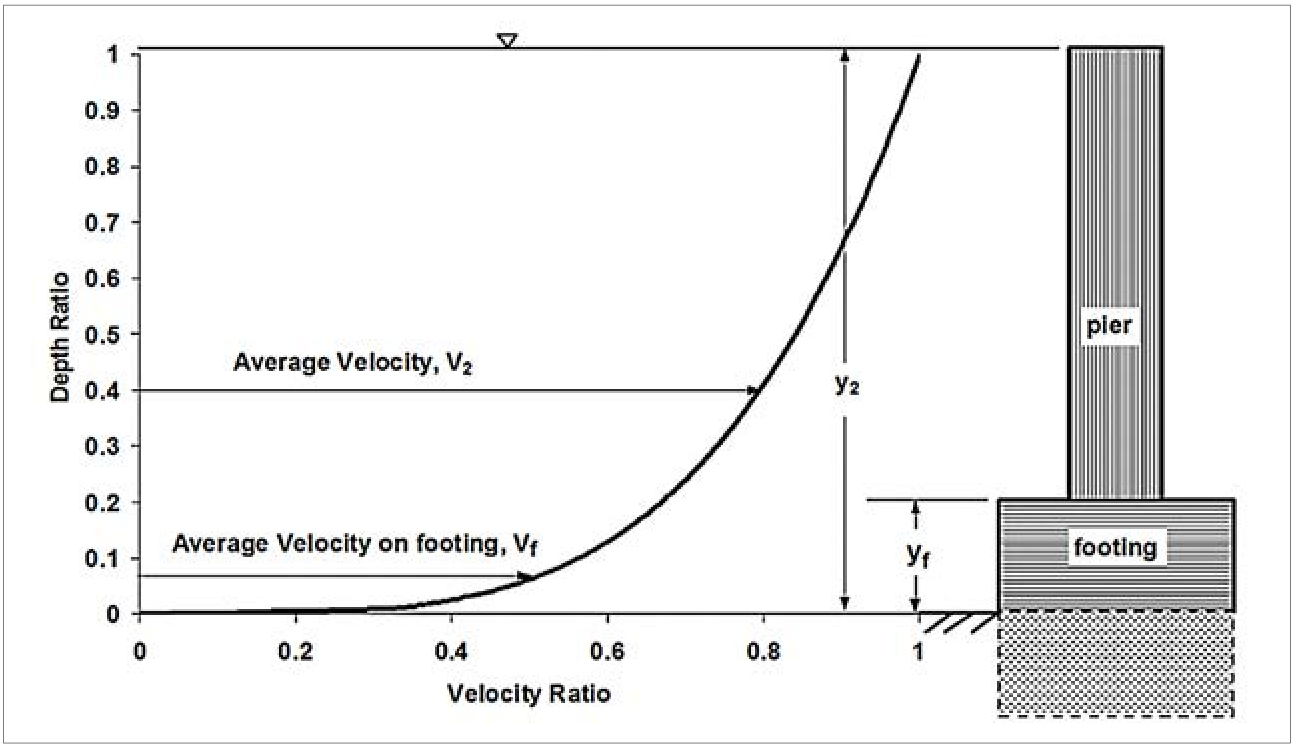
L_n = Natural log to base e.

Y_f = $h1 + y_{s\,pier}/2$ = distance from bed after scour to the top of the footing/ pier cap (m).

K_s = Grain roughness of the bed normally taken as D_{85} of the sand size bed material and $3.5D_{85}$.

Y_2 = Adjusted depth of flow upstream of the pier after scour (m).

Figure 5.13 Velocity and Depth Ratios for Exposed Footing/Pile Cap



e. Determination of the Pile Group Scour Depth Component

Guidelines are given for analysing the following typical cases:

i. Special case of piles aligned with each other and with the flow. No angle of attack.

ii. General case of the pile group skewed to the flow, with an angle of attack, or pile groups with staggered rows of piles.

The calculation involves the following steps:

i. Determine the projected width of the piles onto the plane perpendicular to the flow.

ii. Determine the effective width of a pier that would cause the same scour if the piles were exposed to the flow.

iii. Adjust the flow depth, velocity and exposed height of the pile group to take into consideration the pier stem and pile cap/footing scour components already calculated.

iv. Determine the pile group height factor based on exposed height of pile group above the riverbed.

v. Calculate the pile group scour component.

The effective width of an equivalent full depth pier, b^*_{pg}, is given by

$$b^*_{pg} = b_{proj} K_{sp} K_m$$

Equation 5.32

Where,

b_{proj} = Sum of non-overlapping projected widths of piles, Figure 5.14 and Figure 5.15.

K_{sp} = Coefficient of pile spacing, Figure 5.16.

K_m = Coefficient of a number of aligned rows m, where m is the number of rows in the pile group, Figure 5.17.

The value of K_m is constant for all S/b values when there are more than 6 rows of piles and is 1.0 for skewed or staggered pile groups.

Figure 5.14 Projected Width of Piles in Special Case (Projected Width is Perpendicular to the Flow)

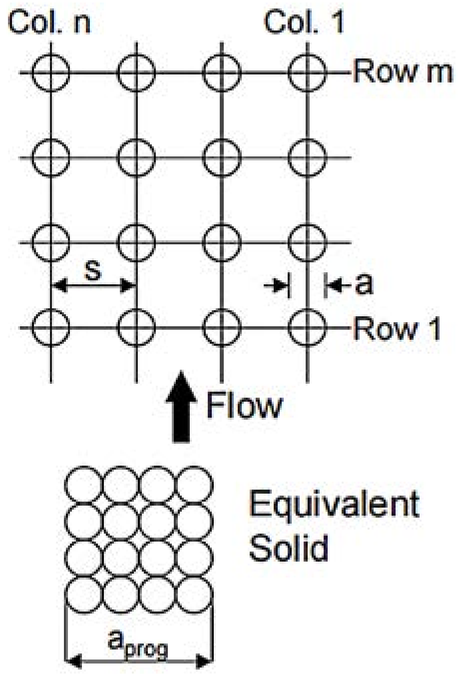
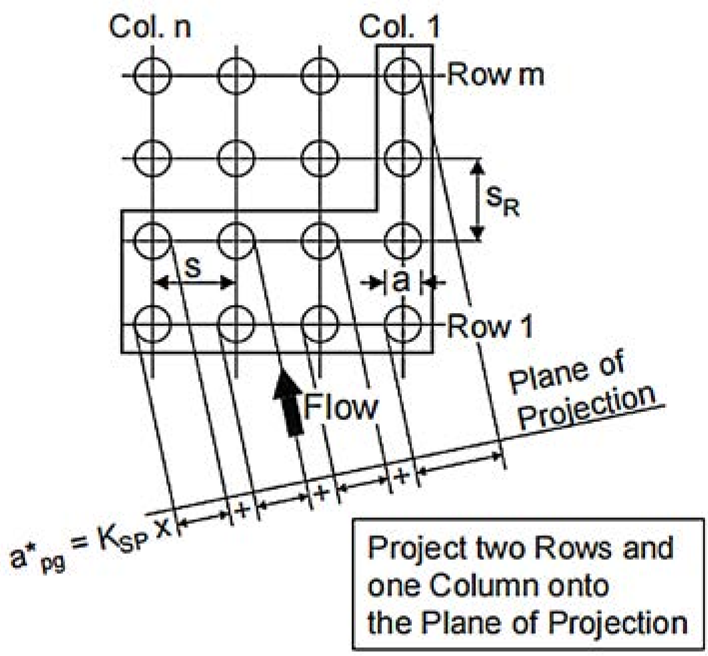


Figure 5.15 Projected Width of Piles for the General Case of Skewed Flow



$\frac{a_{proj}}{a}$

The coefficient of pile spacing, K_{sp} is determined from Figure 5.16 or Equation 5.33.

$$K_{sp} = 1 - \frac{4}{3}\left[1 - \frac{1}{\left(\frac{a_{proj}}{a}\right)}\right]\left[1 - \left(\frac{S}{a}\right)^{-0.6}\right]$$

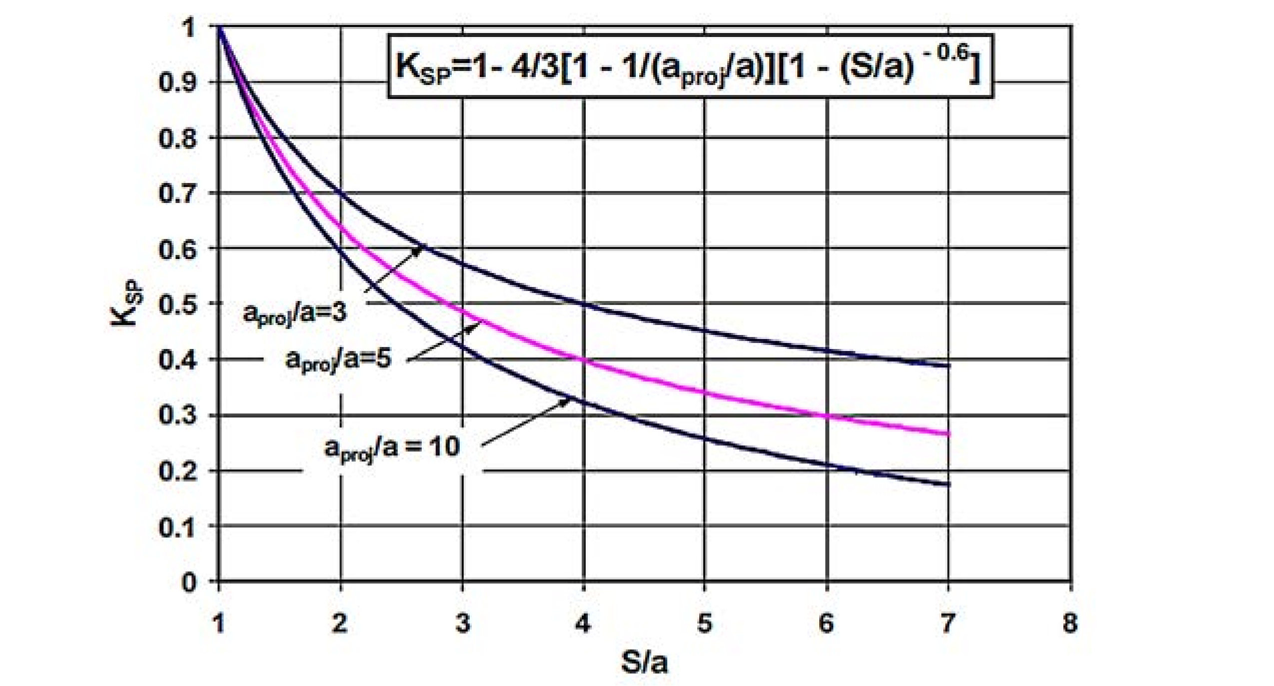
Equation 5.33

Where,

S = Spacing of piles, centre to centre (m).

a = Width of piles.

Figure 5.16 Pile Spacing Factor



The coefficient for the number of rows, K_m, is given by Equation 5.34 and can also be obtained from Figure 5.17.

$$K_m = 0.9 + 0.10m - 0.0714(m - 1)[2.4 - 1.1\frac{S}{a} + 0.1\left(\frac{S}{a}\right)^2]$$

Equation 5.34

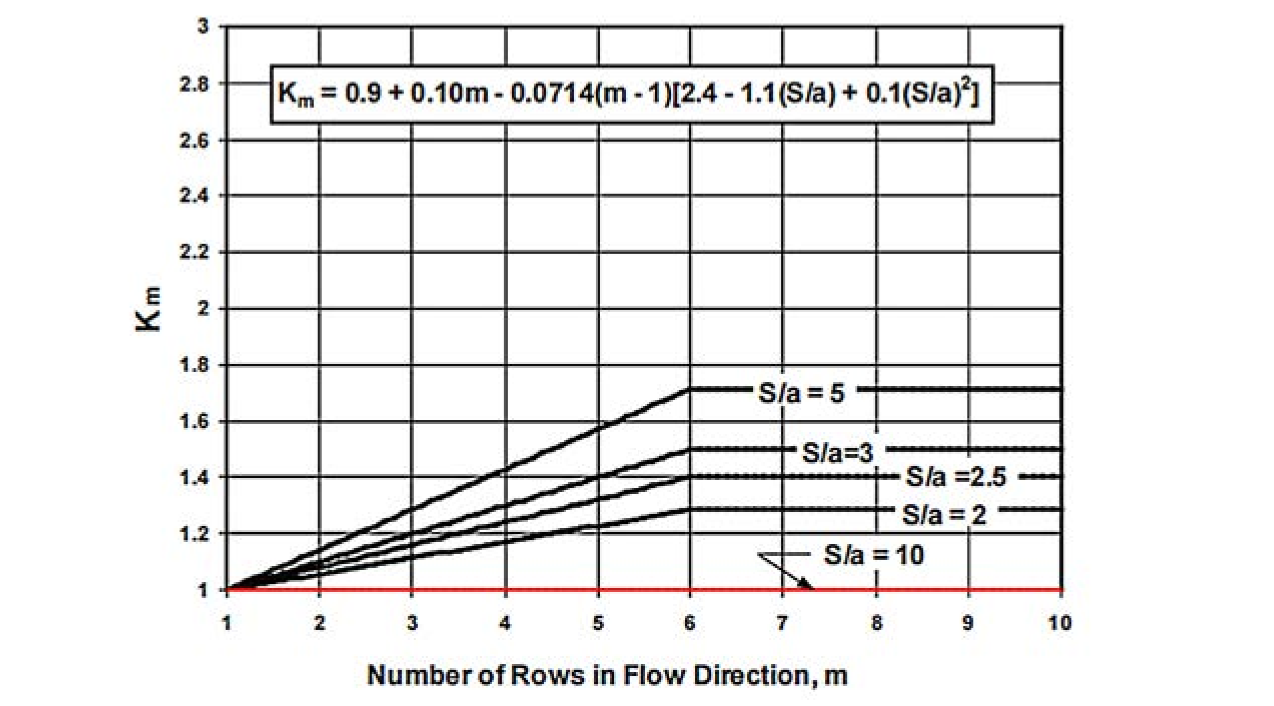
Where,

m = Number of pile rows.

S = Distance between rows centre to centre (m).

a = Width of rows (m).

Figure 5.17 Adjustment Factor for Number of Aligned Rows of Piles



Calculate the pile group scour component using Equation 5.35.

$$\frac{Y_{spg}}{y_3} = K_{hpg}\left[2.0K_1\,K_3\left(\frac{a^*_{pg}}{y_3}\right)^{0.65}\left(\frac{v^3}{\sqrt{gy_3}}\right)^{0.43}\right]$$

Equation 5.35

Where,

K_{hpg} = Pile group height factor as a function of $\frac{h_3}{y_3}$.

The maximum value of $y_3 = 3.5b^*_{pg}$

$H_3 = h_0 + y_{s\ pier}/2 + y_{s\ pc}/2$ = height of pier group above the lowered riverbed after pier and pile cap scour components have been computed (m).

The adjusted flow depth (y_3) and velocity (V_3) are given by Equations 5.36 and Equation 5.37 respectively.

$$y_3 = y_1 + \frac{y_{s\,pier}}{2} + \frac{y_{s\,pc}}{2}$$

Equation 5.36

$$V_3 = V_1\left(\frac{y_1}{y_2}\right)$$

Equation 5.37

Where,

y_3 = Adjusted flow depth (m).

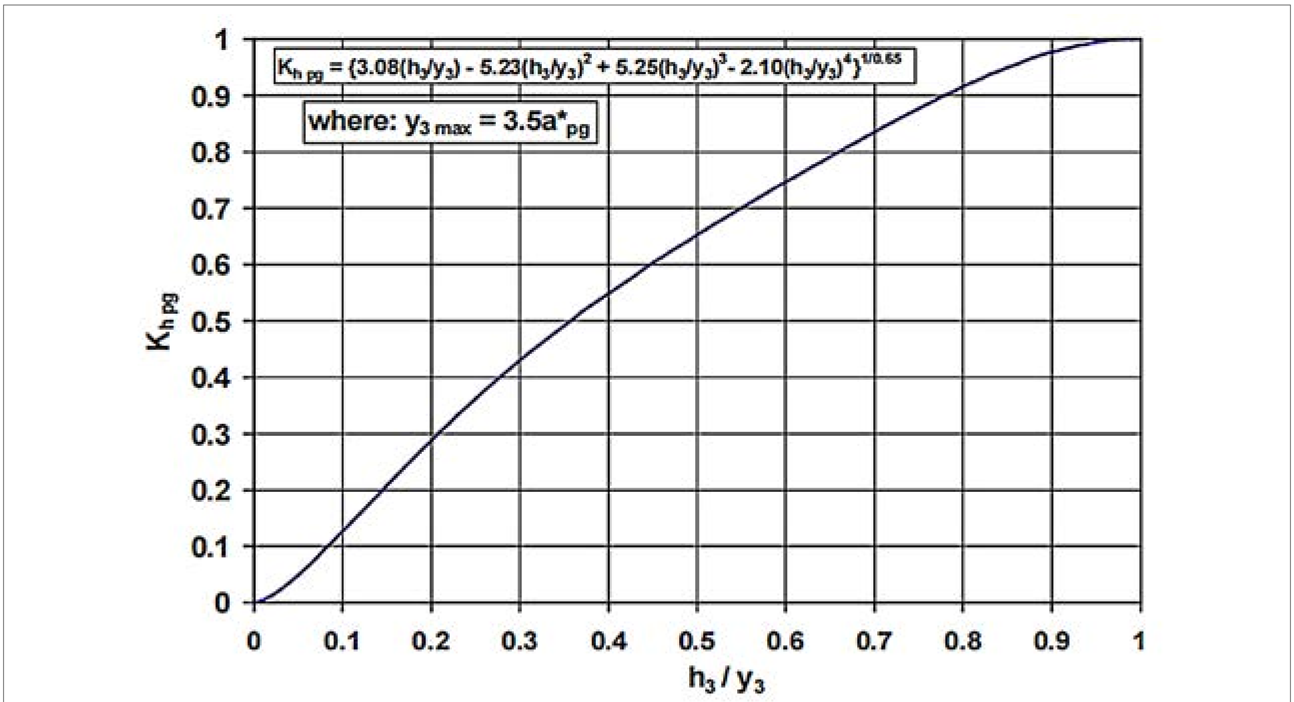
V_3 = Adjusted velocity m/s).

K_{hpg} is computed using Equation 5.38 or can be obtained from Figure 5.18.

$$K_{hpg} = \left[3.08\,\frac{h_3}{y_3} - 5.23\left(\frac{h_3}{y_3}\right)^2 + 5.25\left(\frac{h_3}{y_3}\right)^3 - 2.10\left(\frac{h_3}{y_3}\right)^4\right]^{\frac{1}{0.65}}$$

Equation 5.38

Figure 5.18 Pile Group Height Adjustment Factor



Source: Adapted from Sheppard 2001

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Hydraulic Design of Bridges

f. Determination of Total Scour Depth for the Complex Pier

The total scour for the complex pier from Equation 5.39 is:

$$y_s = y_{s\,pier} + y_{s\,pc} + y_{s\,pg}$$

Equation 5.39

Recommendation:

The guidelines described in this section provide a first estimate and a good indication of what can be anticipated from a physical model study. Physical model studies are recommended in cases of complex pile configurations where costs are a major concern, where significant savings are anticipated, and/or for major bridge crossings.

5.6.5.3 Computing Local Scour at Abutments

Two equations for computation of live-bed abutment scour are used. When the wetted embankment length (L) divided by the approach flow depth (y_1) is greater than 25, the HIRE equation (Richardson. 1990) is recommended. When the wetted embankment length divided by the approach depth is less than or equal to 25, equation by Froehlich (Froehlich. 1989) is used.

The HIRE equation is: $y_s = 4y_1 \dfrac{K_1}{0.55} K_2 F_{r1}^{0.33}$

Equation 5.40

Where,

y_s = Scour depth in feet (m).

y_1 = Depth of flow at the toe of the abutment on the overbank or in the main channel, (m), taken at the cross section just upstream of the bridge.

K_1 = Correction factor for abutment shape, Table 5.9.

K_2 = Correction factor for angle of attack (θ) of flow with abutment.

K_2 = $(\theta/90)^{0.13}$.

θ = 90 when abutments are perpendicular to the flow.

θ < 90 if embankment points downstream.

θ > 90 if embankment points upstream.

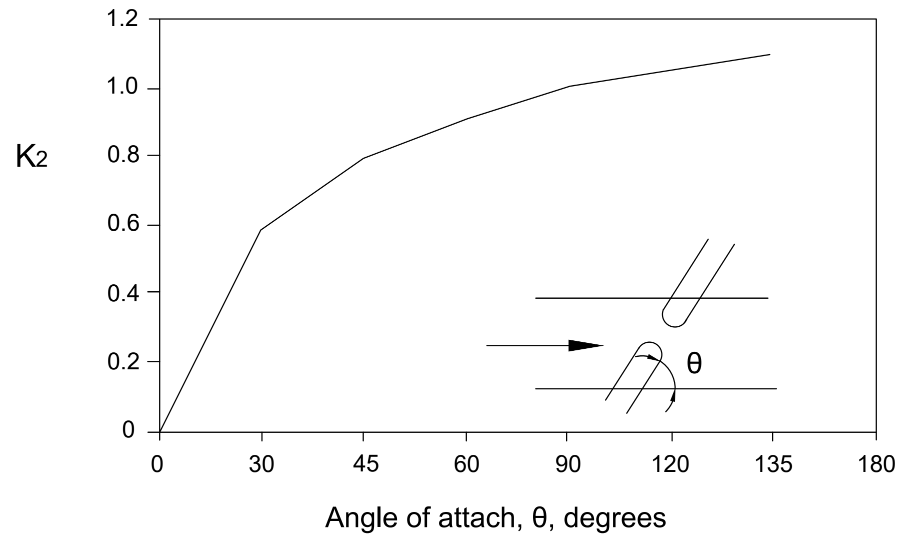
F_{r1} = Froude number based on velocity and depth adjacent and just upstream of the abutment toe.

Table 5.9 Correction Factor for Abutment Shape, K_1

Description	K_1
Vertical Abutment Wall	1.00
Vertical - Wall Abutment with Wingwalls	0.82
Spill-through Abutments	0.55

The correction factor, K_2, for angle of attack can be taken from Figure 5.19.

Figure 5.19 Correction Factor for Abutment Skew, K_2



Froehlich's Equation

Froehlich live-bed scour depth is obtained from the following equation:

$$y_s = 2.27K_1\,K_2\,(L')^{0.43}\,y_a F_{r1}^{\ 0.61} + y_a$$

Equation 5.41

Where,

y_s = Scour depth in metres.

K_1 = Correction factor for abutment shape, Table 5.9.

K_2 = Correction factor for angle of attack.

$K_2 = (\theta/90)^{0.13}$

(θ) is angle of flow with abutment.

θ = 90 when abutments are perpendicular to the flow.

θ < 90 if embankment points downstream.

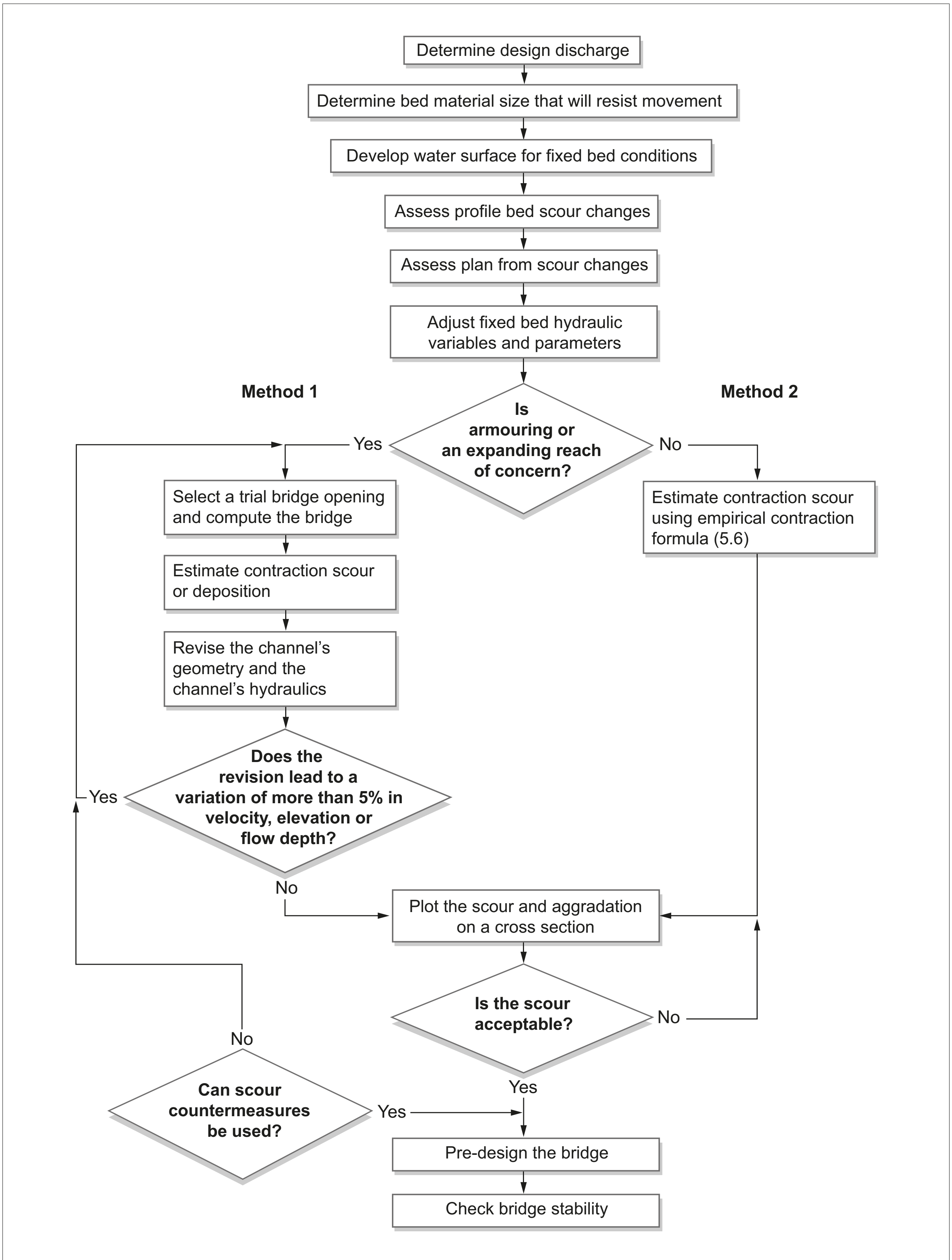
θ > 90° if embankment points upstream.

L' = Length of abutment (embankment) projected normal to flow, (m)

y_a = Average depth of flow on the floodplain at the approach section.

F_r = Froude number of the floodplain flow at the approach section, $F_r = V_e/(gy_a)^{1/2}$

Figure 5.20 Flow Chart for Scour Assessment Procedure



6 River Channel Training & Erosion Control

6.1 General

Structures constructed across any flood plain or near a river stream channel will limit the natural stream function and will create conflict with the environment. Water flow along the stream channel will cause adverse effects including scour and riverbank instability, undermining of approach embankments and erosion of drainage structures outlets. Similarly, road embankments are subjected to soil erosion from natural erosive forces and climatic conditions, particularly torrential rains, and strong winds.

It is therefore important to design and carry out appropriate countermeasures to protect the road embankment and the waterways against these adverse effects. The extent of the protection is to a large degree determined by the materials used in the construction, exposure to damage and the severity of attack.

6.2 River Channel Training and Revetment Measures

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. Their selection, location and design are dependent on hydraulic and geomorphic factors that contribute to stream instability as well as costs, construction, and maintenance consideration.

Key difficulties associated with river behaviour include:

1. Bank erosion and channel course change.
2. Flood protection.
3. Aggradation and degradation of riverbed.
4. River flow extraction or diversion.
5. Bridging the river channel.
6. Maintaining navigable channel.

6.2.1 Probable Problems

Potential problems are essentially the adverse effects of high flood on the river and drainage structures. They are categorised into natural and anthropogenic problems.

6.2.1.1 Natural Problems

Natural problems include the following:

1. Frequent changes in river course.
2. Heavy shoal formation – diversion of main current towards the banks.
3. Landslides in catchment leading to rise in silt load.
4. Avulsion of one river into another.
5. Development of natural cut-off.
6. Heavy erosion of banks by hill streams during flash flood.
7. Aggradation of riverbed leading to high flood levels thus aggravating flooding.
8. River instability due to changes in bed slope which may also be due to seismic activity.
9. Changes in river channels due to changes in rainfall patterns.
10. Changes in a river due to changes in its base level.
11. Erratic behaviour of rivers in deltaic area.
12. Erratic behaviour of braided rivers.
13. Formation of sand bars at river outfalls into the sea.
14. Navigational problems due to shoal formation.

6.2.1.2 Anthropogenic Problems

Anthropogenic problems are a result of the artificial interventions and/or features. Some of the problems are listed below:

1. Degradation of riverbed downstream of a structure (dam, barrage, bridge).
2. Effect of dredging/channelisation of riverbed.
3. Effects of constriction of river width.
4. Effects of flood embankment on the regime of rivers.
5. Effects of spurs and bed bars of different types in river behaviour.
6. Effects of extraction of boulders and sand.
7. Effects of inter-basin transfers of water on the regime of rivers.
8. Effects of riverbed cultivation and construction by farmers.
9. Effects of heavy urbanisation along the river.

6.2.2 Criteria for Choice of Countermeasures

The natural and anthropogenic problems need to be assessed properly in order to understand the difficulties, which may be encountered in relation to the behaviour of rivers. The design of river training or revetments or any other engineering countermeasures should consider river behaviour. The criteria for choice of appropriate river training measures include:

i. Quick and safe passage of high flood.
ii. Efficient transport of sediment load.
iii. Make river course stable and prevent or minimise bank erosion.
iv. Provide sufficient draft for navigation.
v. Prevent outflanking of a structure by directing flow in a defined stretch of a river.

6.2.3 Design of River Training and Revetment Structures

Design of river training and bank protection works is aimed at providing effective and sustainable countermeasures in the form of river training structures and revetments and bed armouring. The design should adequately mitigate the problems and resulting behavioural challenges of rivers.

6.2.3.1 Objectives

To achieve the above, the design should meet the following objectives:

1. Quick and safe passage of high flood.
2. Efficient transport of sediment load.
3. Make river course stable and prevent or minimise bank erosion.
4. Provide sufficient draft for navigation.
5. Prevent outflanking of a structure by directing flow in a defined stretch of a river.

6.2.3.2 Principles

The design and construction of bank protection works are governed by the following principles:

1. Cost benefit analysis is necessary in determining the most optimal option of scour countermeasures. The cost of the protective measures should not exceed the cost of the consequences of the anticipated stream action.
2. Base designs on studies of channel morphology and processes and on experience with compatible situations. Consider the ultimate effects of the work on the natural channel (both upstream and downstream).

3. Understand that the objective of installing bank stabilisation and river training measures is to protect the road. The protective measures themselves are expendable.
4. The designers should personally undertake a field inspection trip of the site and the river and catchment upstream and downstream of the bridge site after construction and undertake inspection with the aid of surveys to check results and to modify the design, if necessary.
5. The environmental impacts of scour countermeasures should be addressed through the design.
6. Any previous evidence of dynamic changes in the vicinity of the site (such as early photographs) should be considered.
7. Geotechnical and soil characteristics that may impact on the design of countermeasures should be determined and taken into account.
8. Many of the counter measures impact on river flow characteristics and its environment hence physical hydraulic modelling may be necessary to determine the effects.
9. Scour processes are not entirely predictable; hence, a monitoring and maintenance plan should be put in place.

6.2.3.3 Design Procedures

Once the problems and difficulties have been identified, the next step is to determine their impact through investigations of the characteristics of the river and the engineering parameters and quantify the impacts of the problems. This information is obtained from channel and bridge hydraulics, which are covered in Chapter 2 and Chapter 5 respectively. The parameters include:

i. Design flood flow characteristics – this provides information on the quantity and velocity of flow.
ii. The determination of various types of scours is covered in Chapter 5. This includes information on scour potential resulting from bed and bank materials, shear resistance of the materials (τp), shear stress (τd), scour depth and lateral scour calculations. Information can be obtained from topographical survey reports, geotechnical and materials investigation reports.
iii. The determination of adverse impacts under extreme events using the 'check' flood return periods.

6.2.4 Typical River Training and Revetment Structures

Broadly, there are two categories of river training structures:

i. Transverse protection structures – designed and constructed perpendicular to the flow/water course. These include check dams (scour checks), spurs or groynes, sills, screens, bandals, bank protection as a bar, etc.
ii. Longitudinal protection structures – designed and constructed on banks parallel to the flow/water course. These include revetments and rock riprap, levees or earth fill embankments, sheet piles, concrete embankments, etc.

Other common protection works include:

1. Gabions –are specially designed rock crates, which can be placed on banks, slopes, and to surround structural elements of the drainage structures such as piers and abutments.
2. Channel lining – these include dry and wet stone pitching, stone sets, bamboo piles and sandbags.

6.2.4.1 Spurs or Dykes

Spurs are obstruction dykes formed in the watercourse to:

1. Arrest meander migration of the stream.
2. Channel a poorly defined stream into a definite channel.
3. Direct the flow along a less erosive path.

4. Lessen the skew angle of flow through a structure.
5. Reduce the required flow opening under a structure.
6. Protect the banks of a watercourse by improving their stability.

The primary purpose of guide banks, however, is to reduce local scour at abutments.

Spurs can comprise either permeable or impermeable structures depending on their intended function. Deflector spurs, diverting the primary flow currents away from the bank in a more desirable direction, should be of impermeable construction, while retarder spurs, intended to retard the flow velocity at the bank and diverting flow away from the bank, could be of the semi-permeable type.

Spurs materials can consist of riprap, sheet piles, gabions, concrete, dolosses, jetties with the jacks being either treated timber or steel, etc. The choice of the material eventually used, is dependent upon many factors and the designer should exercise sound engineering judgement in selecting the most suited material for a particular application.

Spur Specifications

Spur design includes setting the limits of bank protection required; selection of the spur type to be used; and design of the spur installation including spur length, orientation, permeability, height, profile, and spacing.

Below are general specifications, which can be used in the design of spurs:

1. They should be designed in batches of 2 to 5.
2. Spurs can be submerged or non-submerged.
3. Spurs can be permeable or impermeable.
4. The height of spurs < bank height.
5. For submerged spurs, ⅓ ≤ height ≤ ½ of depth of water.
6. Maximum flow constriction = 20% or length of spur should be 0.2 x river width, and 2-2.5 x scour depth on concave bank and 2.5-3.0 x scour depth on convex bank.
7. Spacing is calculated using Equation 6.1 or generally 4-5 x length.
8. Nose, upstream and downstream side of the shank needs erosion protection.

Figure 6.1 Extent of Protection Required at a Channel Bend

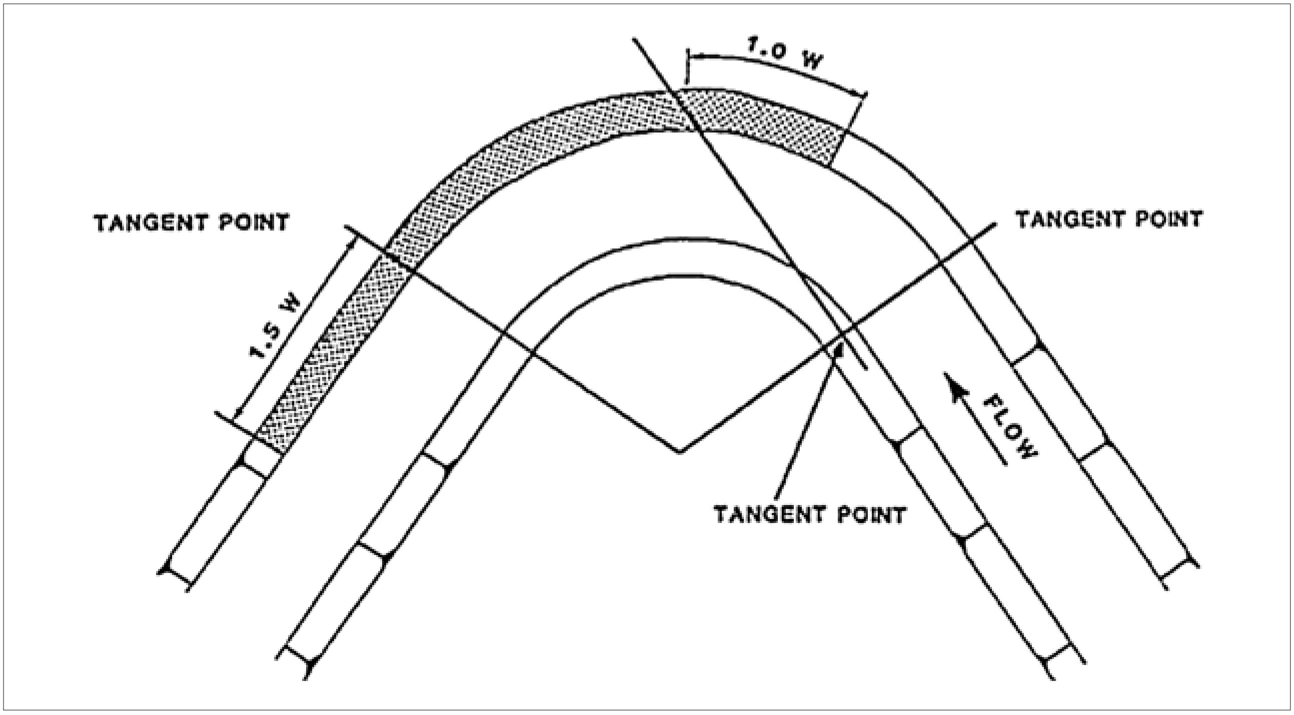
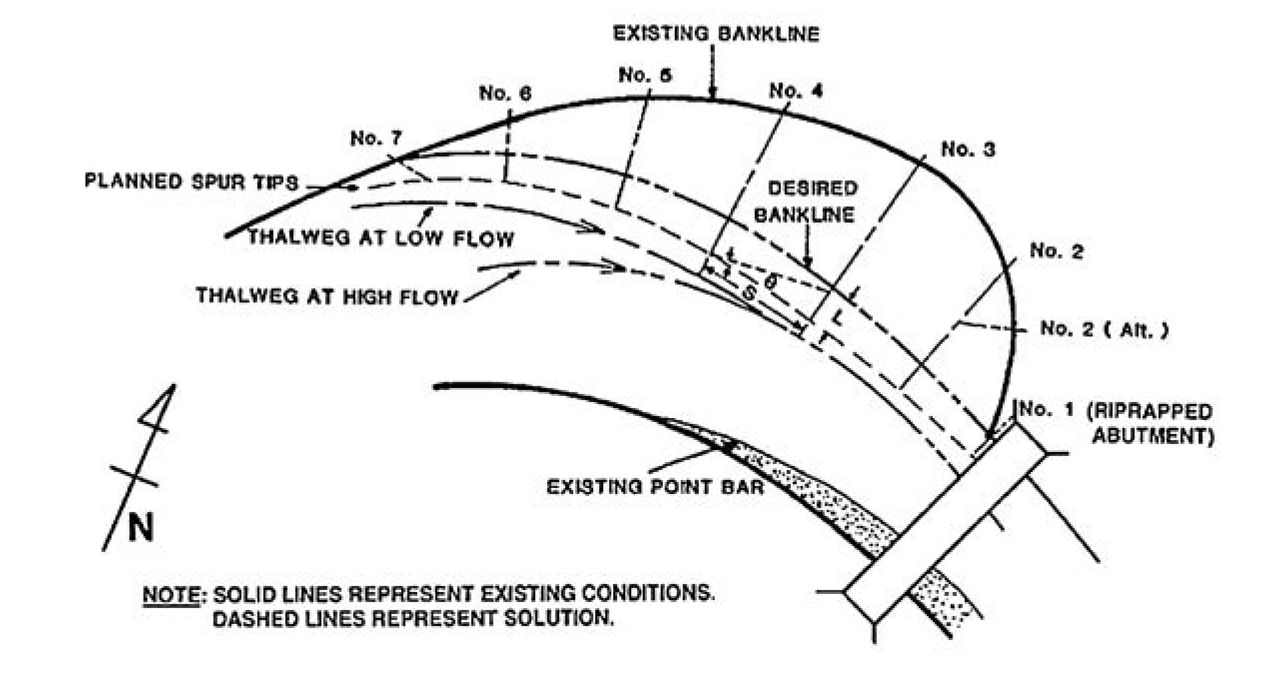


Figure 6.2 Illustration of Spur Spacing and Expansion Angle



$$S = L_e \cot \theta$$

Equation 6.1

Where,

S = the spacing between spur toes (m).

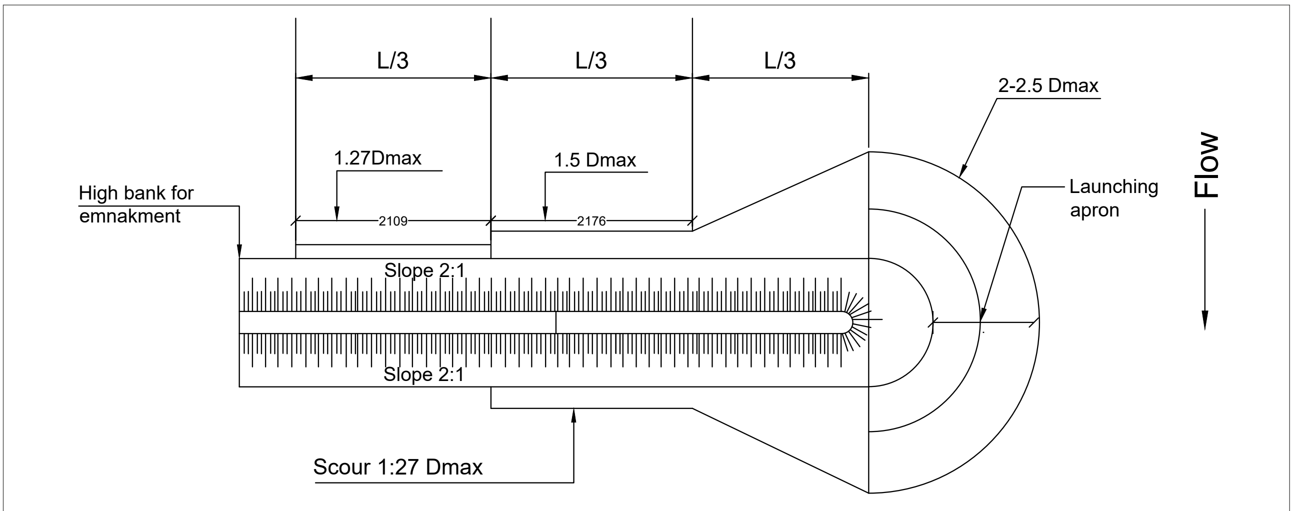
L_e = effective length of spur from the bank line (m).

θ = expansion angle downstream of spur tips (degrees) shown in Figure 6.2 above.

The flow expansion angle θ, is the angle at which flow expands towards a bank downstream of a spur. This angle is a function of the permeability of the spur and the ratio of the spur length to the channel width (FHWA 1990), (See Figure 6.2).

The angle between the spur centre line and the direction of flow also influences the expansion angle, but to a lesser extent. It is recommended that the spurs be orientated at an angle of approximately 90 degrees to the desired bank line. The leading spurs should, however, be angled downstream to provide a smoother transition of the natural flow lines and to minimise scour at the nose.

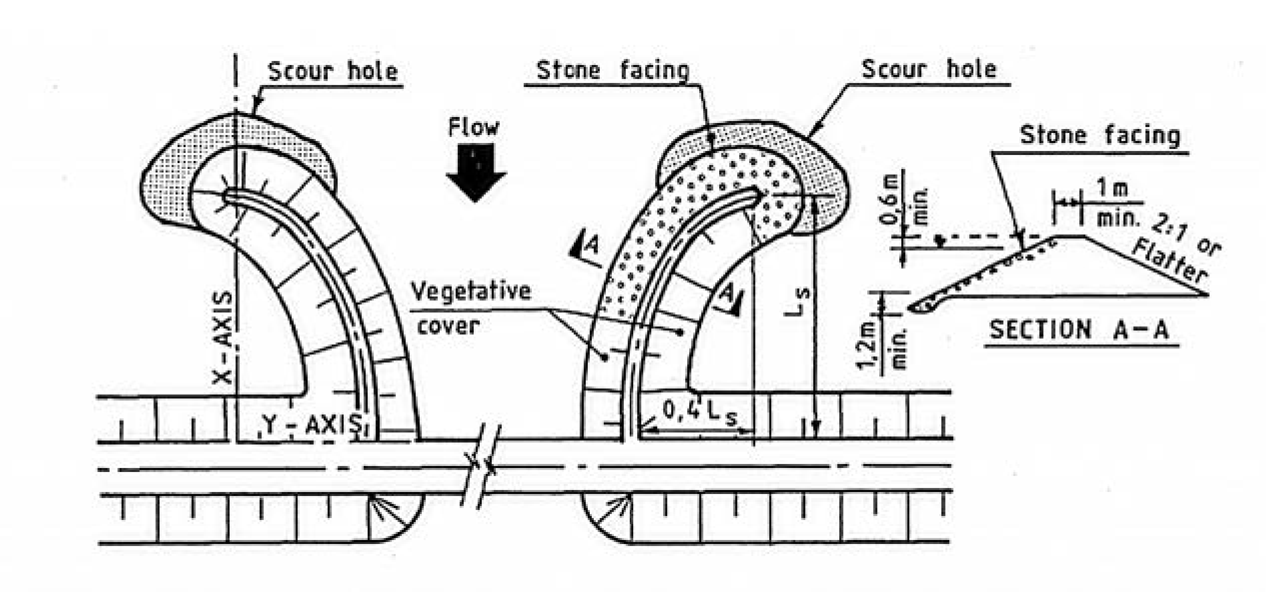
Figure 6.3 Plan Showing Depth of Scour of Spurs



6.2.4.2 Guide Banks (Berms)

Guide banks are used to guide stream flow through the bridge opening and thereby prevent erosion of the approach embankment and scour at the abutments. The two major enhancements guide banks bring to bridge design are (1) reduce the separation of flow at the upstream abutment face and thereby maximise the use of the total bridge waterway area, and (2) reduce the abutment scour due to lessening turbulence at the abutment face. Guide banks can be used on both sand- and gravel-bed streams.

Figure 6.4 Typical Guide Bank Layout



Guide Banks Specifications

Principal factors to be considered when designing guide banks, are their orientation to the bridge opening, plan shape, upstream and downstream length, cross-sectional shape, and crest elevation.

In the design of guide banks preference should be given to orient the guide banks at the junction with the bridge parallel to the flood flow direction and to align them with the abutments of the bridge so that the distance between the banks at the bridge is equal to the bridge opening.

Guide bank design specifications are illustrated in Figure 6.5 and Figure 6.6 below:

Figure 6.5 Specification of Guide Bank (Berm)

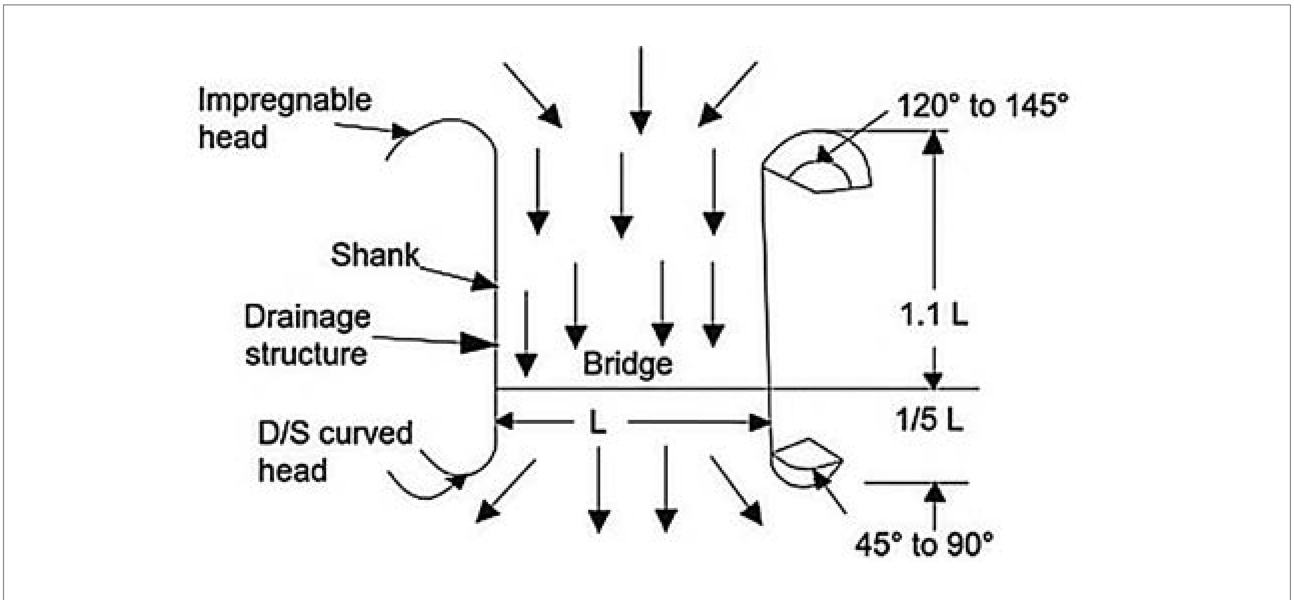
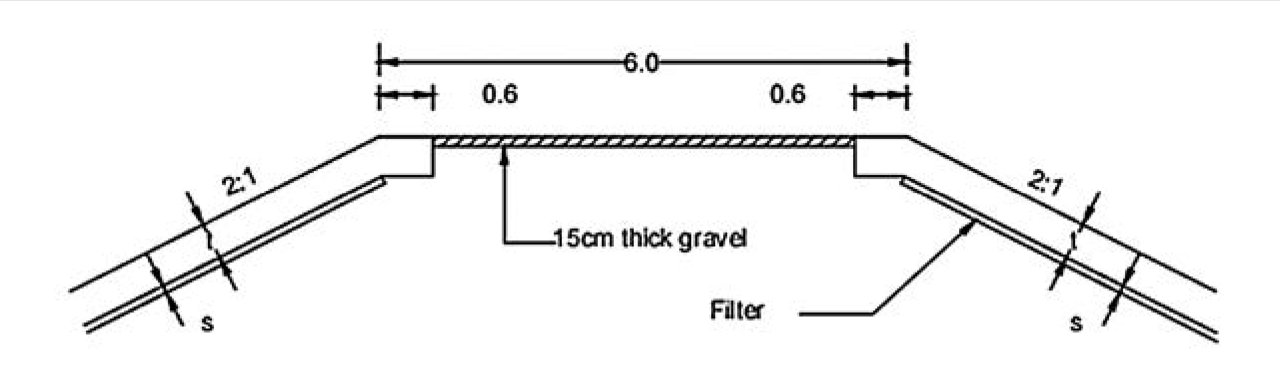


Figure 6.6 Typical Cross-section of Guide Bank



6.2.4.3 Check Dams/Drop Structures

Check dams, consisting of channel drops or weirs, are used downstream of road structures to control vertical instabilities by arresting the progression of head cutting and general channel bed degradation. They are usually built of rock riprap, concrete, sheet piles, gabions, or treated timber piles. However, the material used to construct the structure will depend on their availability, the height of drop required, and the width of the channel.

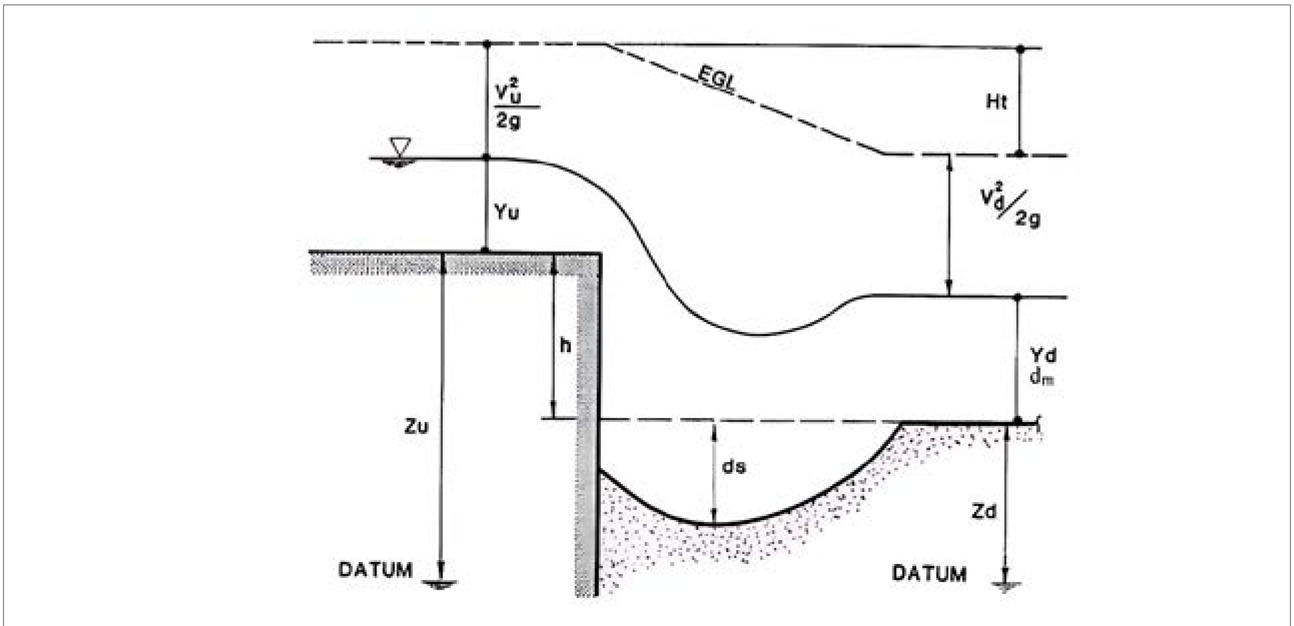
The check dam must be designed structurally to withstand the forces of water and soil, assuming that the scour hole is as deep as estimated. As a result of the vertical drop and concomitant turbulences and dissipation of energy, check banks can initiate erosion of the banks and channel bed downstream thereof. For this reason, it may prove more economical to construct a series of check dams of lower height. Additionally, introducing energy breakers downstream of the check dam, maintenance can be reduced considerably.

To prevent lateral scour of the banks of a stream downstream of check dams, the banks of the stream must be adequately protected using riprap or other revetments. Riprap should be sized and placed in a similar fashion as for spurs and guide banks.

For the purposes of design, the maximum expected scour can be assumed to be equal to the scour for a vertical, unsubmerged drop, regardless of whether the drop is actually sloped or is submerged.

A typical vertical drop resulting from a check dam is diagrammatically illustrated in Figure 6.7.

Figure 6.7 Typical Cross-section of Guide Bank



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The recommended equation (Pemberton and Lara 1984) to estimate the depth of scour downstream of a vertical drop:

$$d_s = K_u H_t^{0.225} q^{0.54} - d_m$$

Equation 6.2

Where,

d_s = Local scour depth for a free overfall, measured from the streambed downstream of the drop, (m).

q = Discharge per unit width, (m³/s).

H_t = Total drop in head, measured from the upstream to downstream energy grade line (m).

d_m, Y_d = Tail water depth (m).

K_u = 1.9

H_t is computed using Bernoulli's equation for steady uniform flow.

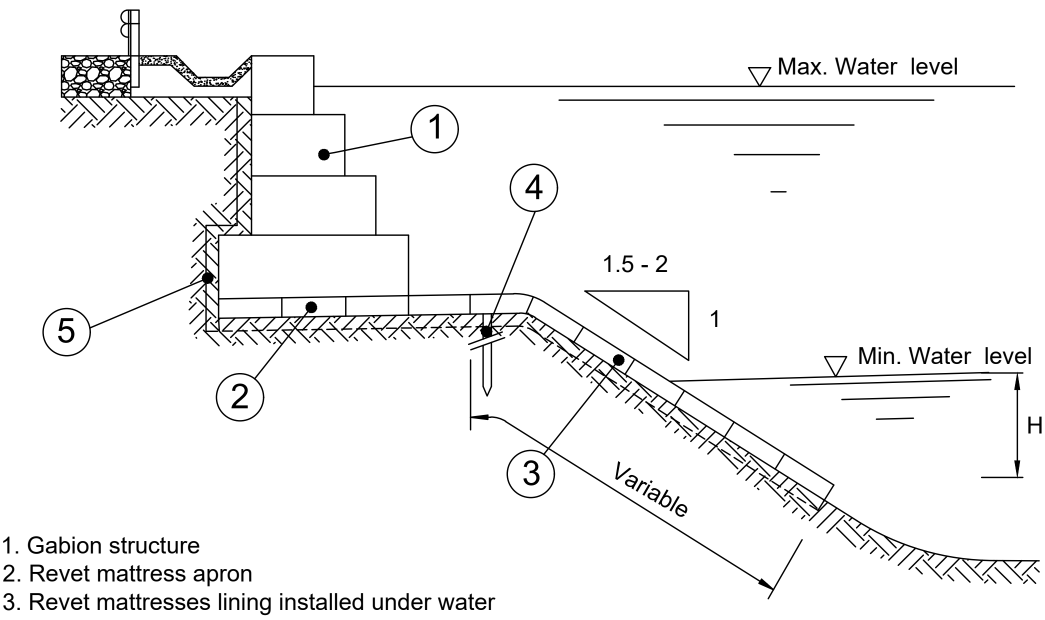
The subscripts u and d refer to upstream and downstream of the channel drop, respectively.

(For design of stilling basins for drop structures the reader is referred to FHWA Hydraulic Engineering Circular Number 14, 'Hydraulic Design of Energy Dissipators for Culverts and Channels' (FHWA).

6.2.4.4 Bank Sloping and Protection

Steep and unstable slope require treatment, which involves reducing the inclination of the slope and applying protection works. The design may also include benching or design of berms as shown in Figure 6.8.

Figure 6.8 Bank Sloping and Protection



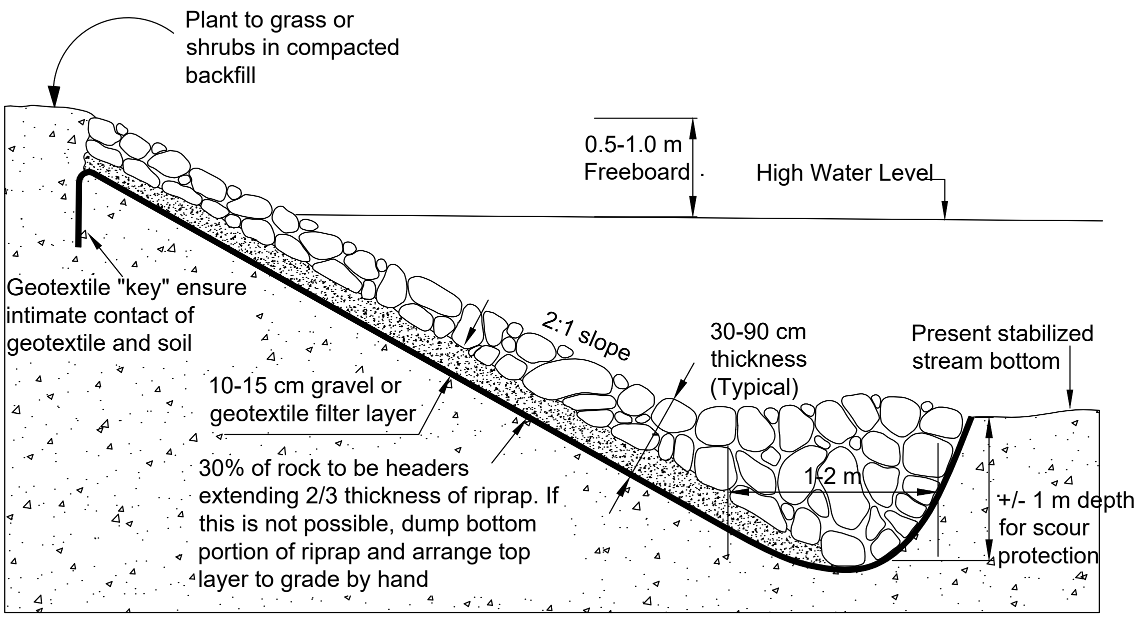
1. Gabion structure
2. Revet mattress apron
3. Revet mattresses lining installed under water
4. Timber pile for temporary anchorage of the mattresses during the launch
5. Non-woven Geotextile

6.2.4.5 Riprap Revetment

Riprap revetments are used to provide protection for embankments, streambanks, and streambeds. They may be flexible or rigid and can be used to counter virtually all erosion mechanisms. Flexible revetments include rock riprap, partially grouted rock riprap, rock-and-wire mattresses, gabions, pre-cast articulating concrete blocks (also known as Articulated Concrete Blocks ACBs), rock-filled trenches, windrow revetments, used tire revetments, geomats, geosynthetics, concrete canvas and vegetation. The system is not recommended at abutments where highly erosive flow conditions tend to be induced.

a. **Specifications for Riprap**: Sizes of stones for riprap may be estimated by either the Factor of Safety Formulae or by Stream Flow Velocity Method.

Figure 6.9 Bank Sloping and Protection



Sizes of stones for riprap may be estimated by either the Factor of Safety Formulae or by Stream Flow Velocity Method.

b. **Safety Factors Method (SFM) formulae**: The sizes of stone can be estimated from the Safety Factors Method (SFM) formulae:

$$D_{50} = 0.503 \text{ x } i^{0.43} \text{ x } q_f^{0.58}$$

Equation 6.3

Where,

D_{50} = Median size of angular stone (m).

i = Embankment slope (m/m).

q_f = Failure unit discharge (m^3/s).

i. **Size of Riprap by Stream Flow Velocity Method**

The Isbash Curve indicates the maximum size rock that might be considered in a critical application. If suitably large rock is not available then the use of cement grouted rock, masonry, or gabions should be considered. Riprap installation details are shown in Figure 6.9.

It is recommended that the riprap layer have a thickness of 3 x D_{50}. All stones should be contained reasonably well within the riprap layer thickness, with little or no oversize stones protruding above the surface of the riprap matrix. Maximum allowable value of shape factor (ratio of length to thickness) for the riprap stone should be 3. Rounded stone are not recommended for use in the armored layers.

Layer thickness should not be less than 0.30 m for practical placement. By increasing the layer thickness, the stability of the protection, particularly in the case of smaller stone will be increased.

ii. Filters

Geotextile and geosynthetic filters and granular filters may be used in conjunction with riprap bank protection. The filter layer serves as a transitional layer and prevents the movement of soil. It also allows groundwater to drain from the soil without building up pressure.

Criteria for matching the filter material to the in-situ/base material to be filtered are given below:

For uniformly graded filter material:

$$R_{50} = \frac{D_{50\ (of\ filter\ material)}}{D_{50\ (of\ base\ material)}} = 5 \text{ to } 10 \qquad \text{Equation 6.4}$$

$$R_{50} = \frac{D_{50\ (of\ filter\ material)}}{D_{50\ (of\ base\ material)}} = 12 \text{ to } 58 \qquad \text{Equation 6.5}$$

$$R_{15} = \frac{D_{15\ (of\ filter\ material)}}{D_{15\ (of\ base\ material)}} = 12 \text{ to } 40 \qquad \text{Equation 6.6}$$

Geotextile filters can be a woven monofilament, or a needle punched non-woven geotextile that must be permeable. The geotextile needs to have an apparent opening size of 0.25 to 0.5 mm. In the absence of other information, a 200 g/m^2 needle-punched non- woven geotextile is commonly used for many soils filtration and separation applications.

Thickness of filter should be between 200mm and 300mm.

6.3 Erosion Control and Road Surface Drainage

6.3.1 General

Erosion problems may occur on the side slopes of embankments or cuttings, grade shoulders or at any other point where surface run off is concentrated or a spring occurs. The obvious palliatives are therefore well-designed surface and subsurface drainage features and appropriate slope angles for the soils and the rocks present.

Identifying and assessing potential erosion problems:

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

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

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

Various surface protection systems can be used in conjunction with the above:

1. Stone pitching.
2. Riprap.
3. Gabion mattresses.
4. Precast concrete blocks.
5. Sand-cement bags.
6. Vegetation.
7. Container cells.

6.3.2 Stone Pitching

There are primarily three methods of stone pitching, each with its specific application, viz,:

1. Plain stone pitching.
2. Grouted stone pitching.
3. Wired-and-grouted stone pitching.

Stone pitching is generally used to protect only the toe of the bank or embankment below the design flood level, with vegetational protection at higher levels. It does not offer any protection against scour, unless a rock toe is provided. However, due to its flexibility, it retains a degree of effectiveness under limited subsidence.

Plain stone pitching systems will eventually permit some growth of vegetation when exposed to fresh water. To promote growth, it is desirable to cover the stone with topsoil. The vegetation has a binding effect and tends to restore the natural roughness of the channel. It may, however, give an untidy appearance.

Stone pitching requires routine inspection as well as repairs, to counter progressive failure occasioned by dislodged stones.

Specifications for Stone Sizes:

For velocity below 4 m/s and on slopes of lower than 1:1; the following guidelines apply in selecting the stones for pitching.

Table 6.1 Stone Size for Various Surface Velocities

Surface flow velocity in m/s	Minimum mass of stone pitching in kgs			
	Bank Slope			
	1 : 1	1: 1.5	1 : 2	1 : 3
1	0.09	0.03	0.02	0.01
2	5.7	2.0	1.3	0.89
3	64	23	15	10
4	360	130	84	57
5	1400	500	320	220

6.3.3 Gabions

Gabions are rectangular wire cages used to retain rockfill to serve as retaining structures or surface protection against erosion and scour. The baskets can be filled with smaller rocks than would be required for mass rock retaining walls or rip rap protection. Typically, the gabion baskets are 0.5 or 1.0 m thick. Smaller sized rocks can be used, because the wire basket surrounding the rock in the mattress, or gabion, tends to make the mass act as a unit while retaining flexibility. Figure 6.10 illustrates the application gabions in the design of protection works.

Figure 6.10 Gabion Protection for Structure Outlet

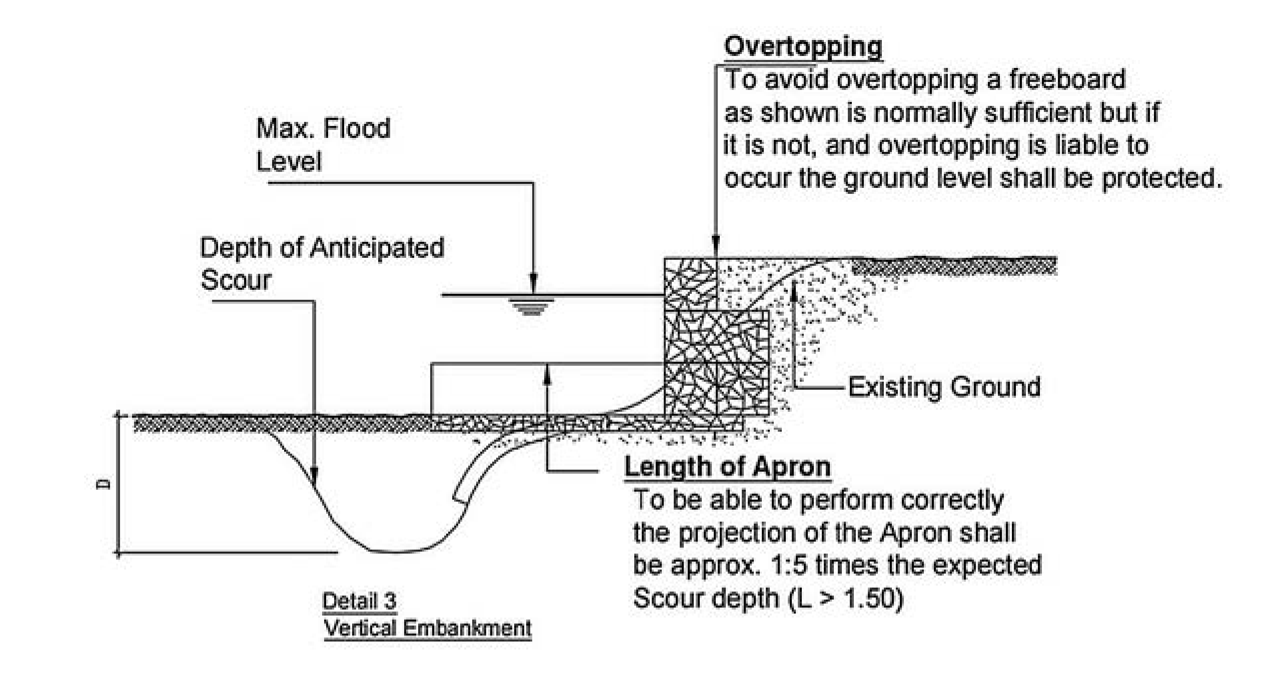


Table 6.2 Specifications for Gabions

	Standard dimensions
Gabion Box	
Length	1, 2, 3 or 4 m
Width	1.0 m
Height	0.3, 0.5 or 1.0 m
Diaphragm (Cells Separation)	1.0 m
Gabion Mattress	
Length	6 m
Width	2.0 m
Height	0.2, 0.3 to 0.5 m
Diaphragm (Cells Separation)	0.6 to 1.0 m

Gabions provide a flexible protective layer, and this property may be critical in situations where significant geotechnical movements or consolidation is likely to occur.

Filter Blankets should be applied to the back of the gabions or beneath the riprap if the riverbank consists of fine, non-cohesive material, to prevent such material from being washed away through the voids in the riprap or gabion lining. The filter blanket can consist of a 0.5 mm thick polyester non- woven textile carpet (minimum weight 250 g/m^2) or multiple layers of stones with the finest layer closest to the riverbank and the coarsest layer towards the water. The polyester carpet should be protected from sunshine, and should be placed, overlapped, and anchored according to the manufacturer.

6.3.4 Protection of Road Embankments

Embankment slopes above the design water level (DWL) should be protected from scour/erosion with at least 0.5 m of stones sized 0 - 100 mm (d_{50} > 70 mm), if no calculation of scour is made. The maximum allowable slope inclinations for some sorted friction soil materials are shown in Table 6.3.

Table 6.3 The Maximum Allowable Slope and Velocity

Slope material	Max. slope H:V (angle)	Design Water Velocity (m/s)
Gravel ($d_{50} \geq$ 70 mm)	1.7 : 1 (30°)	$\leq$ 2 m/s
Boulders ($d_{50} \geq$ 300 mm)	1.4 : 1 (35°)	$\leq$ 2 m/s
Boulders ($d_{50} \geq$ 300 mm)	1.7 : 1 (30°)	> 2 m/s

Where overtopping of road embankment is anticipated, protection can be achieved by the placement of a rock layer on the downstream face of the embankment. The objective in the design should be the prevention of stone movement and progressive riprap layer failure. Where necessary, a filter layer should be placed between the embankment fill and the rock protection to prevent fine material from being washed out through the voids of the face stones. Geotextile fabrics have generally replaced sand/gravel filters in road works.

To arrest further erosion at the toe of embankments, energy dissipators may be installed.

6.3.5 Lined Channels or Cascades

Lined channels or cascades are provided where a watercourse or gully is a direct cause of slope instability. The lining may be impermeable (mortared masonry and/or concrete) or permeable (gabion). The structure may comprise cascades, chutes and check dams. Generally, gabion structures are preferred since they are flexible and allow water ingress provided, they are located below the wet season groundwater table.

6.3.6 Bio-engineering Procedures for Erosion Protection

Bio-engineering can be broadly defined as the use of vegetation, either alone or in conjunction with engineering structures, and non-living plant material, to reduce erosion and shallow-seated instability on slopes. Table 6.4 are guidelines on choosing bio-engineering procedures. Table 6.5 guidelines on choice common plant species available in Kenya.

Table 6.4 Guidelines on Choice of Bio-engineering Procedures

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

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Table 6.4 Guidelines on Choice of Bio-engineering Procedures *(continued...)*

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

Table 6.5 Recommended Guidelines on Available Plant Species

Botanical Name	English Name	Uses	Suited Zones
Bracharia humidicola	Creeping Signal Grass	Terrace banks, water ways, gullies	1.500 – 2.500 m
Cloris guayana	Rhodes grass	Pasture, terrace banks, gullies, road banks	1.000 – 2.250 m
Cynodon spp.	Star grass	Water ways gullies, road embankments	< 2.500 m
Eragrostis curvula	Love grass	Grass strips, embankments	< 2.500 m
Panicum maximum	Guinea grass	Terrace banks, grass strips	< 2.000 m
Panicum coloratum, var. Makarikari	Makarikari grass	Terrace banks	< 1.500 m
Paspalum notatum	Bahia grass	Terrace banks, water ways	1.500 – 2.000 m
Pennisetum clandestinum	Kikuyu grass	Water ways, gullies	1.500 – 2.000 m
Pennisetum purpureum	Napier grass	Terrace banks grass strips, and along embankments of drains and water ways	< 2.000 m
Setaria anceps	Nandi Setaria	Terrace banks grass strips and along embankments of drains and water ways	1.500 – 2.250 m
Setaria splendida	Giant Setaria	Terrace banks grass strips and along embankments of drains and water ways	1.500 – 2.250 m
Tripsacum laxum	Guatemala grass	Terrace banks grass strips and along embankments of drains and water ways	< 1.750 m
Vetiveria zizanoides	Vetiver grass	Road embankments, gullies	< 2.500 m

Source: Soil and Water Conservation Manual for Kenya

6.4 Design of Side Drains

Moisture is the single most important factor affecting pavement performance and long-term maintenance costs. Side drains serve two main functions, namely, to collect and remove surface water from the immediate vicinity of the road, and, to prevent any sub-surface water from adversely affecting the road pavement structure.

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

i. Water collected by the drain must be discharged safely in a manner that will not initiate erosion elsewhere.

ii. Construction of masonry-lined drains should be limited to undisturbed slope materials. Differential settlement, which frequently occurs in made ground and particularly at the interface between natural ground and fill, will lead to rupture.

iii. Drain gradients should not exceed 15 %.

iv. Where people must cross the drain, easy side slopes should be provided so that the people will not fill the drain to cross it.

v. Stepped drain outlets should be provided with a cascade down to the collection point.

vi. Drains should discharge into a stream channel wherever possible, and preferably into channels that already convey a sizeable flow in comparison to the drain discharge.

vii. Low points in the drain system should be designed against overtopping by widening or raising the side walls.

viii. Lengths of drain should be kept short by the construction of frequent outlets to reduce erosion potential should drain failure occur.

ix. Open drains are best located outside the clear zone.

x. The drain should be cleaned and maintained easily. For ease of maintenance and to minimise erosion they should be wide and have sloped sides.

xi. There should be sufficient discharge points and culverts to ensure that the drain never gets very deep.

6.4.1 Types of Drains

Typical drains used in road construction include the following:

1. **Table Drains**

   Table drains are located along the outer edge of the road shoulder in cuts, and beside shallow raised carriageways in fill. They collect run-off from the pavement, shoulders and cut batters and convey the flows to a suitable outfall, which could be via a diversion drain or to a culvert.

2. **Interceptor, Cut-off or Catch-water Drains**

   Catch water drains (sometimes known as cut-off drains) intercept the surface water at the top of cut batters in order to prevent riling (the forming of small channels across the surface), erosion or scouring of the batter slope. They may also be placed at the bottom of fill slopes to intercept water from adjacent properties as well as convey road drainage to an outlet.

   This type of drain has a flat bottom from 2.0 to 2.5 m wide and 0.3 m in depth. However, the depth should be sufficient to carry the design flow.

In erodible soils, the catch drain may take the form of a low levee bank along the top of the batter. In such soils, a drain cut into the surface may rapidly erode and enlarge itself or cause local slips in the batter by piping. These issues need to be addressed in the location and design of catch drains. V-shaped drains are not preferred and should not be used in erodible soils.

3. Median Drains

Median drains collect run-off from the roadway pavement and median and direct the flow to the pavement drainage system. The main limitation on median drains relates to safe slopes for errant vehicles. A desirable side slope of one in 10 or flatter severely restricts the capacity of such drains unless the median is very wide. Road safety requirements may result in the median drain being augmented by grated pits and underground pipes.

4. Inlet or Outlet Drains

These drains direct water towards the culvert inlet on the upstream side and convey water away from the culvert outlet and surrounding area on the downstream side. They are commonly used where the inlet or outlet invert levels of the culvert are below the natural surface. In such cases the length of drain required is determined by 'daylighting' the drain, i.e, construction of a drain at a set slope until it breaks out (daylights) at the natural surface.

5. Bench Drains

A bench drain is provided for the bench (i.e., ledge) that is constructed on a batter, or natural slope. The purpose of benches is to reduce erosion to the batter faces, reduce the amount of water in cuttings to be carried by table drains and in some cases to also improve sight distance on horizontal curves. Bench drains carry water from the bench to suitable drainage outlets. (See Figure 6.14)

Build berms only as wide as required, as adding a berm will increase the load on the outside road edge and may create additional instability in highly erodible soils (exceed shear strength). If necessary, it is recommended to grass or hydroseed berms to protect them in sensitive areas. Consider armoring berms where cut-out spacing is restricted by the terrain or cannot easily be addressed with flumes or drainage socks.

6. Contour Drain

A contour drain, also known as a contour bank, is a surface drain designed to slow the rate of run-off by diverting water along a gently sloped path, from a site to nearby stable areas, at a discharge velocity that will not cause erosion.

7. Swales

Vegetation can be used to provide the filtering surface area to spread and reduce flow velocities that allows sedimentation as well as providing a substrate for biofilm growth and hence biological uptake of soluble pollutants.

8. Discharge Channels

Depending on topographic condition, it is sometimes necessary to collect water at the top of either a cutting or an embankment and discharge it down the slope. For this purpose, discharge channels shall be constructed and lined with masonry, concrete or metal. The usual dimensions are 400 mm deep. If half rounded channel elements are used the diameter should normally be 500 mm. Other sizes will be determined on the basis of procedure given in subsection 6.4.3.

For safety reasons these features should be placed outside the edge of the surfacing. Where a crash barrier is installed the kerbing or the channel should be installed immediately in front of the supports, on the traffic side.

At the base of the embankment, toe ditches may be necessary to remove water from the vicinity of the embankment or to prevent erosion of the fill.

Figure 6.11 Roadside Ditch /Vertical Ditch on High Embankment

Figure 6.12 Ribbed Flume Drain on High Embankment

9. Herringbone (or Chevron) Drains:

These are type of discharge channels. They are constructed in herringbone fashion on slope faces to collect surface seepages and surface runoff. They are often quite shallow (about 1m deep) but can be much deeper. Care needs to be taken to ensure that the construction of the drain does not lead to further instability and to ensure that the drain can still function in the event of minor down slope movements. Vegetative material may be included to improve slope stability.

10. Counterfort Drains

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

11. Horizontal Drains

Horizontal drains are used to intercept groundwater and seepage at depth. They require the use of plastic pipes and specialist drilling equipment that may not always be available, and they are not easy to install. They are expensive and justified only along urban sections where the pavement is confined between footpaths.

12. Flumes

Flumes help to protect earthworks from erosion by conveying water over vulnerable areas, such as fill and batter slopes, to more stable ground. Most roads in hill country require sections to be flumed. Flumes, like drainage culverts, require fall to reduce silting up.

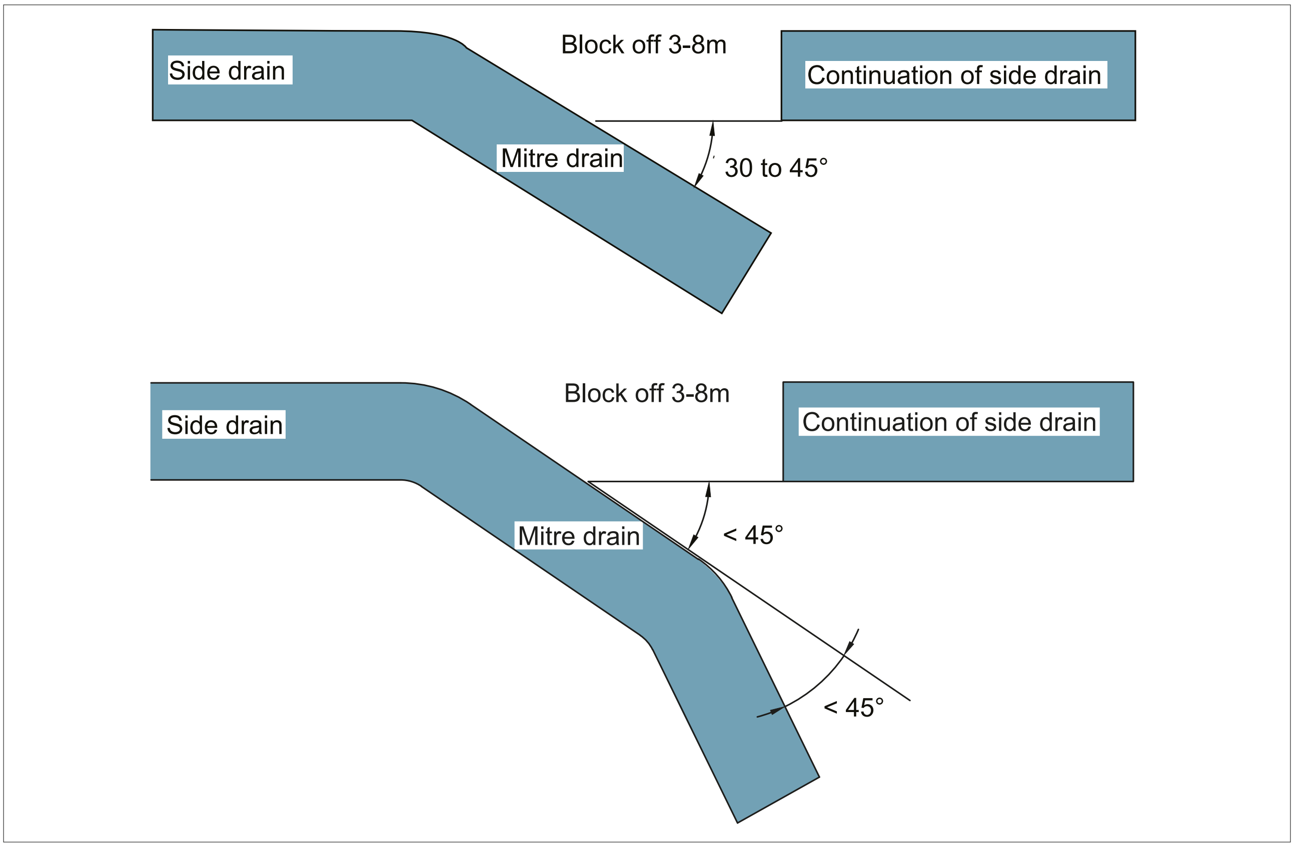
13. Mitre Drains or Turnouts or Diversion Drains

Mitre drains or turnouts are used to drain off the water from side drain onto adjacent land, mainly in low volume roads. The spacing of diversion drains should be such that the flow in a table drain does not exceed the capacity of the table drain. As a standard, mitre drains should be spaced at 50m and not more than 100m intervals unless otherwise constrained by adjacent properties and land use.

A block-off is required to ensure that water flows out of the side drain into the mitre drain. The angle between the mitre drain and the side drain should not be greater than 45 degrees. An angle of 30 degrees is ideal. The desirable slope of the mitre drains is 2 to 5%. In flat terrain, a small gradient of 1% or even 0.5% may be necessary to discharge water, or to avoid very long drains. The drain should lead gradually across the land, getting shallower and wider. Stones may need to be laid at the end of the drain to help prevent erosion.

If it is necessary to take water off at an angle greater than 45 degrees, it should be done in two or more bends so that each bend is not greater than 45 degrees (Figure 6.13).

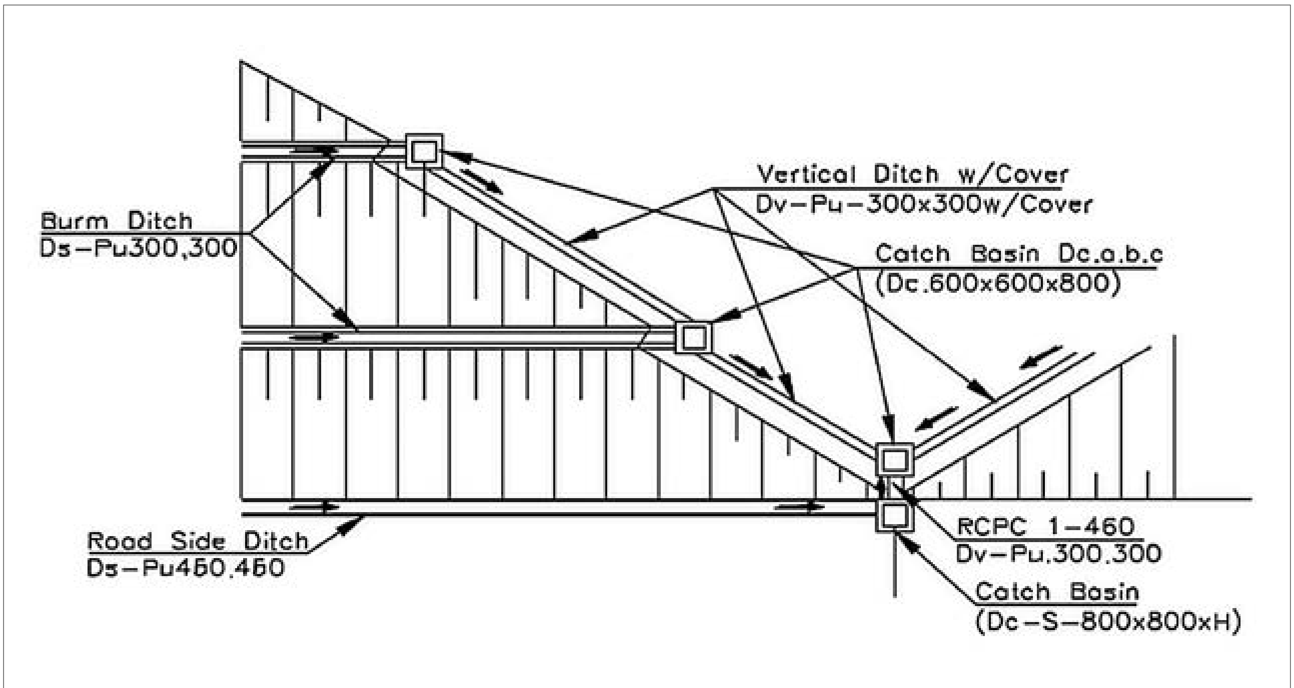
Figure 6.13 Angle of Mitre Drain



14. Chutes

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

Figure 6.14 Typical Drainage Arrangement for Cut and Embankment Sections



6.4.2 Guidelines on Choice of Side Ditches

The types of side ditches, which will generally be used, are shown in Figure 6.16 below. The figures cover side drains with 1:2 slopes and bottom widths up to 1 m. The largest pre-calculated drain has a width of 5 meters and a capacity of 15m^3/s at 1% slope. For wider and flatter drains individual calculation based on the Manning - Strickler formula are recommended.

The guidelines given in the tables below are based upon general economic and aesthetic considerations. However, the type of side ditch selected must be checked to ensure that it will carry the expected flow without running so deep as to wet the road pavement nor so fast as to cause scour.

Figure 6.15 Guidelines for Side Ditches

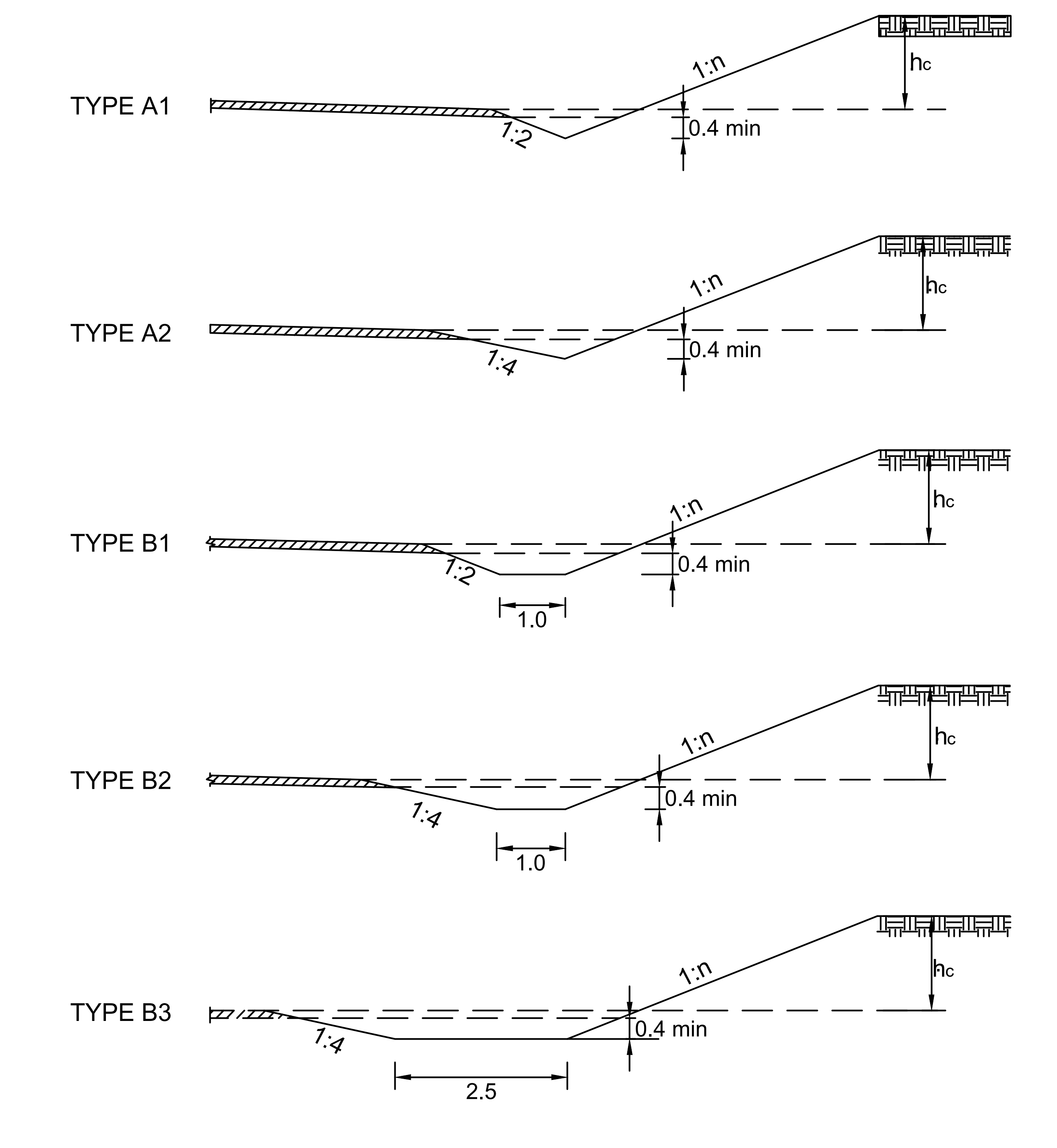


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Table 6.6 Guidelines on Choice of Side Ditches

Side Ditch Type	To be used under the following conditions	Remark
A_1	Mountainous terrain with heavy earthwork	Backslope to be varied according to stability of cut material. Slope should be stable and enable vegetation to establish
A_2	Rolling terrain with moderate earthworks. Mountainous terrain where flatter ditch than A_1 is required due to capacity or velocity limitations.	As for A_1
B_1	Mountainous terrain where flatter ditch than A_1 is required due to capacity or velocity limitations.	As for A_1. Width may be increased if fill material is required.
B_2	Rolling terrain with moderate earthworks where flatter ditch than A_2 is required due to capacity or velocity limitations.	As for B_1
B_3	Flat terrain with little earthworks. Rolling terrain with moderate earthworks where flatter ditch than B_2 is required due to capacity or velocity limitations.	As for B_2

Figure 6.16 Guidelines for Cut-off Ditches

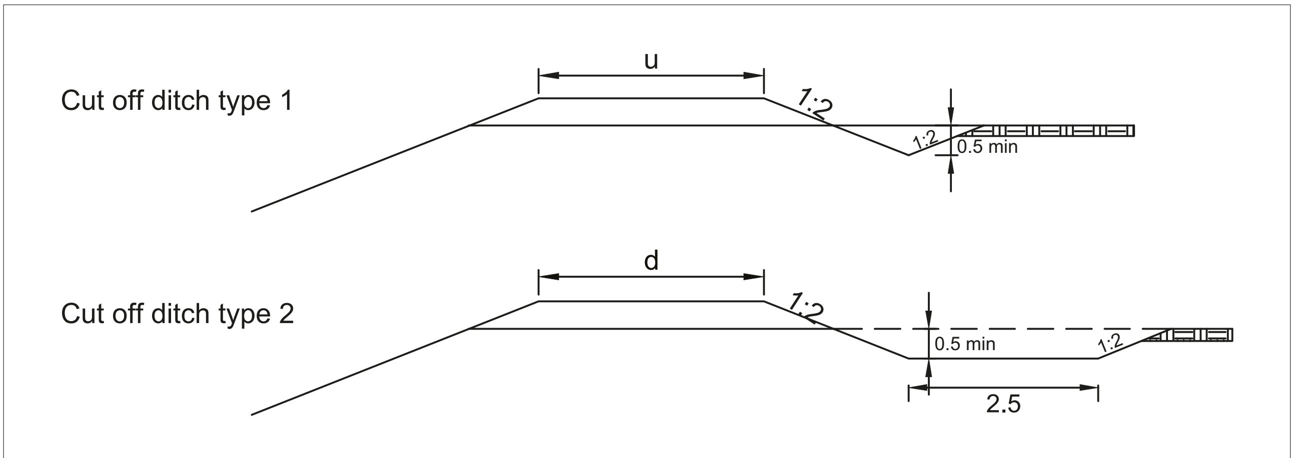


Table 6.7 Guidelines on Cut-off Ditches

Cut-off Ditch Type	To be used under the following conditions
1	Moderate catchment area and little chance of siltation.
2	Large catchment area and little in areas liable to silting and/or damage to ditch profile by pedestrians, cattle, etc.

Economic and Aesthetics

The side ditch type should match the adjacent terrain, e.g, wide ditches with gentle side slope in flat open country and generally be achieved.

6.4.3 Dimensioning of Side Drains

6.4.3.1 Discharge Calculation

Side ditches must be designed to carry the stormwater run-off originating from the carriageway, shoulder drain and cut-slope. Where cut – off ditches are not provided, any run-off from beyond the cut must also be included.

6.4.3.2 Capacity of Side Ditches

The capacity of side ditches should be determined using the Manning – Strickler Formula (metric).

$$Q = \frac{1}{n} A\, R^{2/3}\, S^{0.5} = V A$$

Equation 6.7

$$\text{Or } V = \frac{1}{n} R^{2/3}\, S^{0.5}$$

Equation 6.8

Where,

Q = Capacity in m^3/s.

A = Cross sectional are narrow ditches with steeper side slopes in hilly and mountainous terrain. If this principle is followed an economic and aesthetic design will a of water.

V = Velocity of flow (m/s).

n = Mannings Coefficient.

R = Hydraulic Radius, A/P where P is the wetted perimeter, m.

S = Longitudinal slope (m/m).

Capacity of V-shaped side ditches with sides of different slopes may be determined using Equation 7.9 or 7.10 in Chapter 7 of this Manual.

Spreadsheets or other acceptable format should be used to calculate the capacity, flow speed and water depth in the drainage channel at different slopes.

Limiting value for the velocity of flow (v) to prevent scour, together with the corresponding Roughness Coefficients, are given in Table 6.8 for the different types of ditch material which will normally be encountered.

Table 6.8 Maximum Permissible Velocities in Erodible Ditches and Corresponding Roughness Coefficients

Material in Ditch	Maximum Permissible Velocity (m/s)	Roughness Coefficient (Manning's n)
Sand, Loam, Fine Gravel, Volcanic Ash	0.6	0.033
Stiff Clay	1.1	0.020
Course Gravel	1.5	0.025
Conglomerate, Hard Rock and Soft Rock	3.0	0.040
Hard Rock	3.0	0.040
Masonry	3.0	0.025
Concrete	3.0	0.015

In areas where good grass cover is guaranteed, these values may be increased up to a maximum of 1.5 m/s. In such cases a Roughness Coefficient of 0.03 should be used. Where grass cover is expected but not guaranteed a maximum velocity of 1.1 m/s should be used with a Roughness Coefficient of 0.033.

Where velocities are greater than those shown, erosion protection measures such as scour checks will be required. Alternatively, the channels width can be altered. Ditches can also be lined with gabion, dry stone pitching, riprap or vegetation.

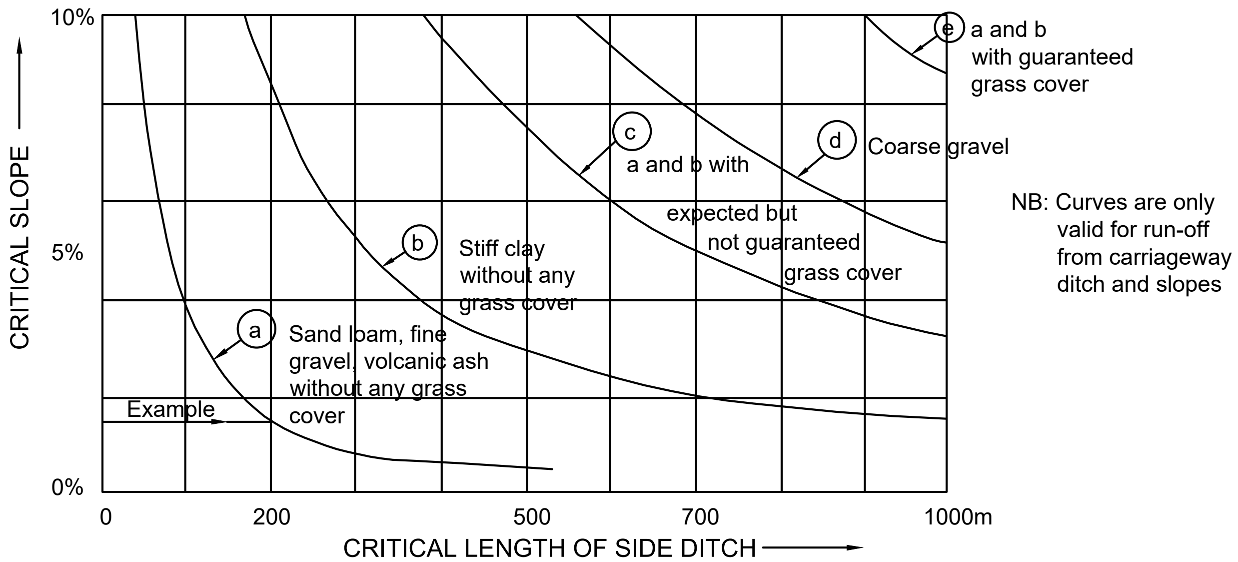
6.4.4 Protection of Ditches and Channels from Erosion and Sedimentation

The critical length of unlined ditches must be determined with regard to erosion control.

This critical length is defined as the maximum length of ditch, in which water velocities do not give rise to erosion. The maximum length of ditch in a particular material can then be determined from the maximum velocity calculated from Equation 6.9.

A single maximum length diagram for all of Kenya cannot be established, as the rainfall intensity of the projects area, which defines the volume of water to be discharged, varies from place to place.

Figure 6.17 Example for Critical Length Diagram for Side Drains



EXAMPLE: - If a side ditch with a surface of type (a) is longer than 200m, and the slope at that point is more than 1.5%, scour checks are necessary and shall, begin there.

6.4.4.1 Scour Checks

Scour checks should be designed as control sections and will thus match the side ditch cross sections so as not to cause an obstruction, which would raise the water level.

Careful consideration must be given to the spacing of scour checks; such spacing will not be constant but should reduce as the flow increases.

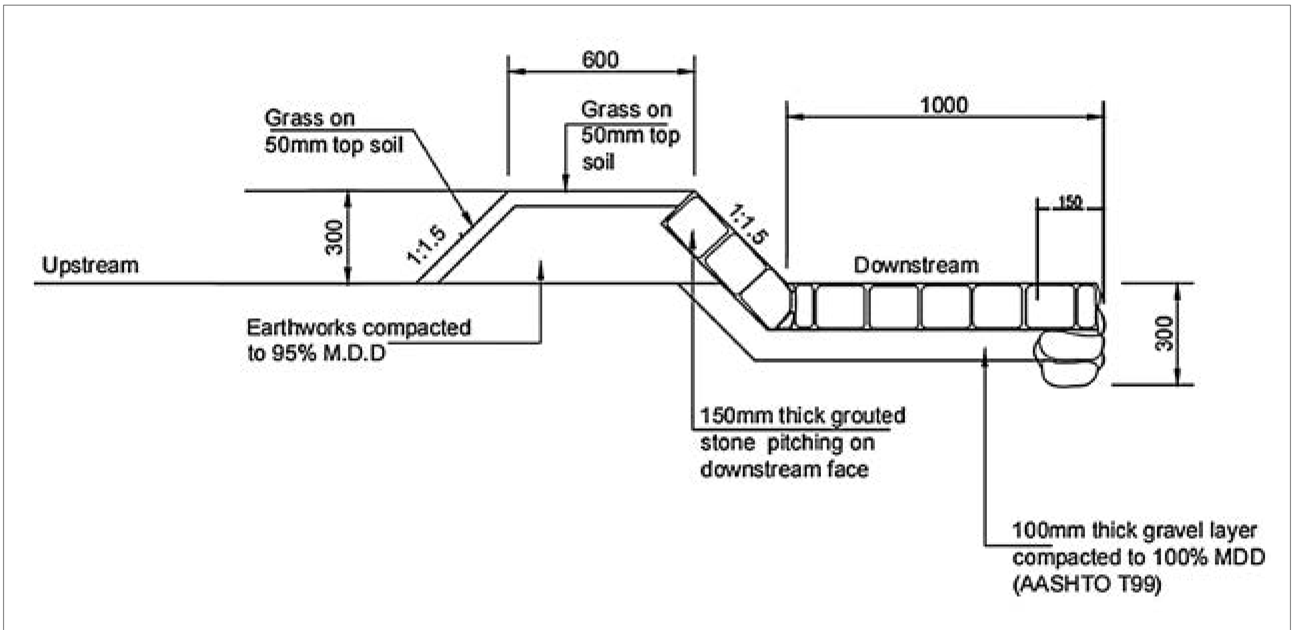
The distance between the scour checks depends on the gradient and the soil condition of the road. The steeper the slope or the softer the soil, the closer the spacing of the scour checks. Table 6.9 below gives guidelines on spacing of scour checks with respect to road gradients.

Table 6.9 Spacing Between Scour Checks

Road gradient (%)	Scour Check Spacing (m)
3	Not required.
4	17
5	13
6	10
7	8
8	7
9	6
10	5
11	4
12	4

Scour checks should only be made from concrete or grouted stone pitching. Other materials are not acceptable. A typical example of a grouted stone pitched scour check profile is illustrated in Figure 6.18.

Figure 6.18 Specification for a Typical Grouted Stone Pitched Scour Check



6.4.4.2 Sedimentation Control

If water velocities are too low sedimentation may occur. Ditches and drains should therefore be given sufficient gradient everywhere, in so far as topography and erosion control will permit. Sedimentation velocities for a few types of material are approximately the following:

Table 6.10 Sedimentation Velocities for Different Material

Material	Sedimentation Speed in m/s
Silt	0.08
Fine sand	0.15
Coarse sand	0.20
Fine gravel	0.30
Gravel	0.65

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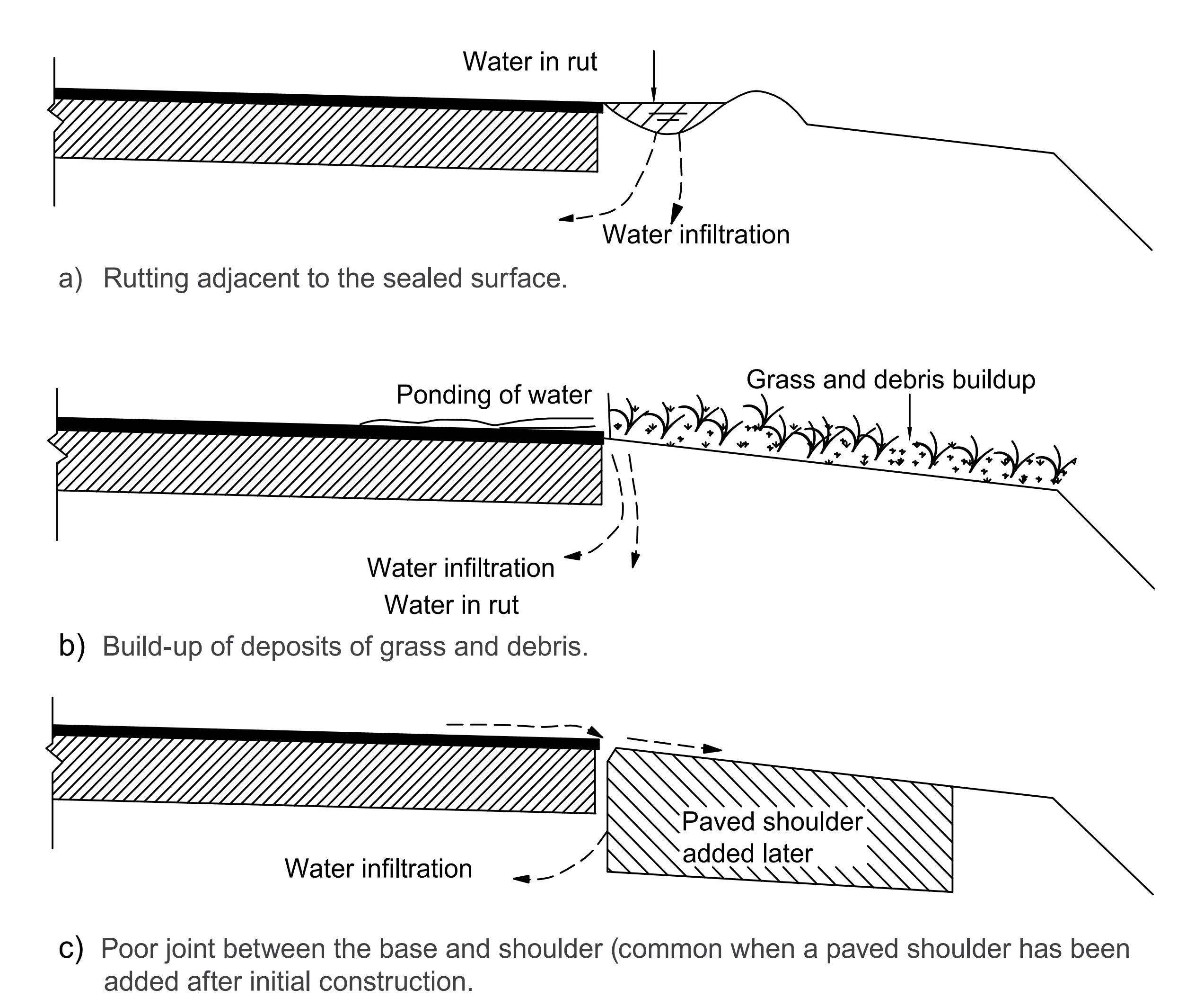
6.5 Drainage of Road Surface

Rain falling on the road surface can penetrate the pavement layers in several ways as illustrated below:

Figure 6.19 Rainfall Penetration to Pavement Layers



Drainage within the pavement layers is an essential element of structural design because the strength of the subgrade in service depends critically on the moisture content during the most likely adverse conditions. It is therefore critical to ensure that water can drain away quickly from within the pavement. This can be achieved by several measures as follows:

6.5.1 Proper Drainage of Pavement Layers

A permeability inversion exists when the permeability of the pavement and subgrade layers decreases with depth. It often occurs at the interface between sub-base and subgrade since many subgrades are cohesive fine- grained materials. It is therefore desirable that the permeability of the pavement and subgrade layers are at least equal or are increasing with depth. For example, the permeability of the base must be less than or equal to the permeability of the sub-base in a three-layered system.

Where permeability inversion is unavoidable, the road shoulder should be sealed to an appropriate width to ensure that a lateral wetting front does not extend under the outer wheel track of the pavement. In addition, an extra width of the base must be primed and sealed, and the seal should cover about 200 mm of the shoulder.

When permeable base material is used, particular attention must be given to the drainage of the base layer. The 'trench' type of cross-section in which the pavement layers are confined between continuous impermeable shoulders must be avoided. This applies principally to stone bases.

Drainage grips through the shoulders have proved to be generally ineffective in the long run as they are blocked by silt and clay. The only effective way is to provide a continuous drainage layer through the shoulders.

The main cases to consider are as follows:

i. **Pervious Base - Impervious Subbase**

   It would be ideal to extend base and subbase across the shoulders. However, except for narrow shoulders, this solution is extremely costly.

   In most cases, the drainage of the base shall be ensured by a continuous drainage layer placed underside of the base. This drainage layer shall consist of proper filter material (graded stone, sand, etc) and should be at least 75mm thick.

ii. **Pervious Base - Pervious Subbase**

   Although costly, the only effective solution is to extend the subbase across the shoulders to the drainage ditches.

iii. **Impervious Base**

   Provided that the shoulders are sufficiently impervious and the joint between base and shoulder is properly sealed, internal drainage should not be necessary. However, if this cannot be assured, then drainage must be considered.

In cases (1) and (2) above, it is essential that the surfaces of the subbase and the drainage layer be given adequate crossfall to ensure the egress of water under any circumstances. In the case of a superelevated section, the drainage layer should be omitted.

Note: Longitudinal pipe-drains may also be used, but their cost is such that they will be justified only along urban sections where the pavement is confined between footpaths.

6.5.2 Proper Designed Cross and Longitudinal Slopes

Well-designed cross and longitudinal slopes are needed to assist the shedding of water from the road surface into the side drains.

The following guidelines are recommended:

6.5.2.1 Cross-falls

Table 6.11 Normal Pavement Cross Slopes for Effective Drainage

Surface Type	Range in Rate of Surface Slope (%)
High-Type Surface	2.5 to 6.0
Intermediate Surface	2.5 to 3.0
Low-Type Surface	2.5 to 6.0
Shoulders	
Bituminous or Concrete	2.5 to 6.0
With Kerbs	4.0

Additional Guidelines

1. For normal road section, 2.5% should be adequate. Use of a cross slope steeper than this value on pavements with a central crown line is not desirable
2. In areas of intense rainfall, a somewhat steeper cross slope (2.5%) may be used to facilitate drainage and the maximum governed by the requirements of geometric design manual.
3. Although not widely encouraged, inside lanes can be sloped toward the median if conditions warrant.
4. Median areas should not be drained across travel lanes.
5. Shoulders should be sloped to drain away from the pavement, except with raised, narrow medians and super-elevations.

6.5.2.2 Longitudinal Slopes

1. Experience has shown that the recommended minimum values of roadway longitudinal slope given in the geometric design manuals will provides safe, acceptable pavement drainage.
2. For kerbed pavements, longitudinal slope should not be less than 0.5% with an absolute minimum of 0.3%. (Guidelines on provision of longitudinal slopes in the design of road refer to chapter 5 of Road Design Manual - Volume 1: Part 3).
3. 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.

For detailed design of drainage for pavement and its components, see Chapter 7. Sub-section 7.4.6 gives recommendations on mitigation against hydroplaning.

7 Urban Drainage

7.1 General

7.1.1 Design Objectives

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 designed for shorter return periods ranging from 2 to 10 years.

The major system provides overland relief for stormwater flows exceeding the capacity of the minor system. The major system is composed of pathways that are provided for the runoff to flow to natural or manmade receiving channels such as streams, creeks, or rivers. They are normally designed for longer return periods in excess of 10 years. These design facilities are covered in more detail in Chapters 2 to 5 of this Manual.

This chapter covers drainage systems and elements, which intercept or capture runoff and the conveyance system, namely:

1. Kerb and channel.
2. Chutes.
3. Cascades.
4. Edge and median drainage.
5. Drainage pits.
6. Pipe networks (including pump stations).
7. Grated inlets.

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 stormwater runoff from the roadway 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. The systems will usually require detention or retention basins, and/or other best management practices for the control of discharge quantity and quality.

The design of a drainage system should address the needs of the travelling public and those of the local community through which it passes. The drainage system for a roadway traversing an urban area is more complex than for roadways traversing sparsely settled rural areas. This is due to:

1. The wide roadway sections, flat grades (both in longitudinal and transverse directions), shallow water courses and absence of side channels.
2. The more costly property damage that may occur from ponding of water or from flow of water through built-up areas.
3. The roadway section must carry traffic but also act as a channel to convey the water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway may interfere with or possibly halt the passage of highway traffic.

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The most serious effects of an inadequate storm drainage system are:

i. Damage to adjacent property, resulting from water overflowing the roadway kerbs and entering such property.

ii. Risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway.

iii. Weakening of the base and subgrade due to saturation from frequent ponding of long duration.

The provisions made in this Manual will apply to both rural and urban set ups.

7.2 Urban Hydrology

Hydrological design involves the calculation of discharge or runoff. The design parameters, which should be determined include:

7.2.1 Catchment Area

The Rational Formula is best used in urban areas due to the relatively small catchment areas, high runoff coefficients and large sections of impermeable surface. It is also well suited for the design of storm drains, where the momentary peak flow rate is desired. This method is described in RDM 2.1. Larger drainage areas require a different method, as do drainage systems which include detention storage.

Design of urban storm drainage facilities (storm drainage systems) comprise of two separate and distinct systems, namely the Primary System (terrain or grassed area) and the Secondary System (road surface). Runoff is calculated for different road lengths (100 m, 200 m, 500 m etc,) separately for the road surface and the adjacent terrain (width 10-50 m).

7.2.2 Rainfall Intensity

Intensity-Duration-Frequency curves (IDF curves) have been developed for several regions in this country. Refer to appendix A of Part 1 of this Manual. However, for small watersheds involved in urban drainage, specific IDF curves may have to be developed for the particular site.

7.2.3 Volume of Runoff (or Discharge)

Volume of runoff (or discharge)– at any particular point in the drainage channel the runoff from catchment area upstream at design flow is the value of discharge, which should be determined in order to specify the capacity of the drainage structure or system.

7.2.4 The Design Return Period

Guidelines for design return periods which are appropriate for the different drainage structures and systems are given in Table 4.1 in Chapter 4 of Part 1 of this Manual. The part relevant for urban drainage is reproduced hereunder as Table 7.1 for ease of reference.

Table 7.1 Guidelines on Design Return Period

Material	**Return Period** (years)	
	Design Flood	**Check Flood**
Gutters and inlets	2	5
Side ditches	5	10

The design flow is determined using the Rational Method:

$$Q = 0.0278\ \text{CIA}$$

Equation 7.1

Where,

Q = maximum rate of runoff, m³/s

C = runoff coefficient representing a ratio of runoff to rainfall (see Table 4.4 through to Table 4.8 in Part 1 of this Manual.

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, h_a.

(Worked examples are given in Appendices A.14 and A.15)

7.2.5 Time of Concentration

The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of interest. For urban drainage, the time of concentration is generally made up of two components:

1. The time for overland and gutter flow to reach the first inlet.
2. The time to flow through the storm drainage system to the point of interest (if applicable). For inlet spacing- Inlet time is the time for water to move across the pavement plus length of gutter to the inlet. For the first inlet, it is estimated at not less than 7 minutes.

The commonly used formula for estimating flow time, t_c is Kirpich;

$$t_c = 0.0663\ L^{0.77}\ S^{-0.385}$$

Equation 7.2

Where,

t_c = time of concentration (min).

S = overall catchment slope (m/m).

L = main stream length (m).

(See worked examples Appendices A.14 and A.15).

7.3 Urban Drain System

7.3.1 Types of Urban Drains and Components.

A storm drain is defined as that portion of the storm drainage system that receives runoff from inlets and conveys the runoff to some point where it is then discharged into a channel, water body or piped system. There are two components of urban storm drainage system:

1. Those which collect stormwater runoff from the roadway surface and right-of-way and convey it along and through the right-of-way.
2. Those which 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.

7.3.1.1 Stormwater Collection Facilities

1. **Roadside and Median Ditches** are used to intercept runoff and carry it to an adequate storm drain. These ditches should have adequate capacity for the design runoff 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.
2. **Gutters** are used to intercept pavement runoff and carry it along the roadway shoulder to an adequate storm drain inlet. Kerbs 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 kerbed in urban settings. Parabolic gutters without kerbs are used in some areas.
3. **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 roadway, 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, kerb inlets, slotted inlets, etc.

Drainage inlet locations are often established by the roadway geometries as well as by the intent to reduce the spread of water onto the roadway 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 roadway surface.

The spacing of grate inlets on continuous grades is determined by the allowable water spread on the pavement and the efficiency of the inlets.

7.3.1.2 Stormwater Conveyance System

Upon reaching the main storm drainage system, stormwater is conveyed along and through the right-of-way to its discharge point via storm drains connected by access holes or other access structures. These include:

1. **Storm Drains** receive 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.
2. **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).
3. **Stormwater Pump Stations** are required as a part of storm drainage systems in areas where gravity drainage is impossible or not economically justifiable. Stormwater pump stations are often required to drain depressed sections of roadways.

7.3.1.3 Stormwater Discharge Controls

1. **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 runoff in detention/ retention basins, storm drainage pipes, swales and channels, or other storage facilities. These facilities 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.
2. **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.

Detention/retention facilities, water quality control systems and pumps are functions of water department, municipal or city authorities and hence are outside the scope of this roads Manual. However, design procedure for detention storage is included in appendix C for reference.

7.4 Design of Urban Drainage Systems

7.4.1 Urban Drainage System Types

An urban drainage system is generally defined as a runoff collection and transportation system, which is responsible for quickly removing stormwater runoff only from urban areas to prevent any flooding. The system comprises of storm drains, curbs (kerbs), gutters, ditches, inlets, access holes, pipes and other conduits, open channels, pumps, detention basins and water quality control facilities. An efficient urban drainage system must satisfy the following general guidelines and considerations.

7.4.1.1 Storm Drain Pipe Systems

1. A higher design frequency (or return interval) should be used for storm drain systems located in a major sag vertical curve to decrease the depth of ponding on the roadway and bridges and potential inundation of adjacent property. Where feasible, the storm drains should be designed to avoid existing utilities.
2. Shorter and smaller conveyances can be designed to carry runoff to nearby holding areas, natural conservation areas, or filter strips (with spreaders at the end of the pipe).
3. Ensure that storms in excess of pipe design flows can be safely conveyed through a development without damaging structures or flooding major roadways. This is often done through design of both a major and minor drainage system. The minor (piped) system carries the mid-frequency design flows while larger runoff events may flow across lots and along streets as long as it will not cause property damage or impact public safety.
4. The maximum high water at any point should not interfere with the intended functioning of an inlet opening, or reach an access hole cover. Maximum allowable high-water levels should be established along the storm drainage system prior to initiating hydraulic evaluations.
5. It is desirable to maintain a self-cleaning velocity in the storm drain to prevent deposition of sediments and subsequent loss of capacity. For this reason, storm drains should be designed to maintain full-flow pipe velocities.
6. Both minimum and maximum cover limits must be considered in the design of storm drainage systems. Minimum cover limits are established to ensure the conduit's structural stability under live and impact loads. With increasing fill heights, dead load becomes the controlling factor.

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7. National and local road agencies should maintain standards for storm drain location. They may be located a short distance behind the kerb or in the roadway near the kerb. It is preferable to locate storm drains on public property. Acquisition of required easements can be costly and should be avoided wherever possible.
8. Where possible, storm drains should be straight between access holes. However, curved storm drains are permitted where necessary to conform to street layout or avoid obstructions.
9. Attention should be provided to the storm drain outfalls to ensure that the potential for erosion is minimised. Drainage system design should be coordinated with the proposed staging of large construction projects to maintain an outlet throughout the construction project.
10. Stormwater systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage paths within a drainage basin. These natural drainage paths should be modified as necessary to contain and safely convey the peak flows generated by the development.
11. In establishing the layout of stormwater networks, it is essential to ensure that flows will not discharge onto private property during flows up to the major system design capacity. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets.

7.4.1.2 Street and Roadway Gutters/Kerbs

1. Most common gutters are triangular or semi-circular in cross section. The gutter's cross-sectional dimensions will depend on the presence of parking lanes, shoulders, kerb type and height, and whether or not a formed gutter.
2. Gutters are efficient flow conveyance structures. Impervious surfaces should be disconnected hydrologically where possible and runoff should be allowed to flow across pervious surfaces or through grass channels. Gutters should be used only after other options have been investigated and only after runoff has had as much chance as possible to infiltrate and filter through vegetated areas.
3. It may be possible not to use gutters at all, or to modify them to channel runoff to off- road pervious areas or open channels. For example, kerb opening type designs take roadway runoff to smaller feeder grass channels. Care should be taken not to create erosion problems in off-road areas. Protection during construction, establishment of strong stands of grass, and active maintenance may be necessary in some areas.
4. Desirable gutter grades should not be less than 0.5 percent for kerbed pavements, and with a minimum 0.2 percent in very flat terrain. Minimum grades can be maintained in very flat terrain by use of a sawtooth profile.
5. To provide adequate drainage in sag vertical curves, a minimum slope of 0.5 percent shall be maintained within 15 meters of the level point in the curve. Special gutter profiles shall be developed to maintain a minimum slope of 0.2 percent up to the inlet. Although ponding is not usually a problem at crest vertical curves, on extremely flat curves a similar minimum gradient shall be provided to facilitate drainage. (See also sub-section 6.5.2).
6. The design of pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. Cross slopes of 2 percent have little effect on driver effort in steering, especially with power steering, or on friction demand for vehicle stability. Use of a cross slope steeper than 2.5 percent on pavements with a central crown line is not desirable. In areas of intense rainfall, a somewhat steeper cross slope may be necessary to facilitate drainage. In such areas, the cross slope may be increased to 2.5 percent. Shoulders should generally be sloped to drain away from the pavement, except with raised, narrow medians. (See also sub-section 6.5.2)

7. Kerbing at the outside edge of pavements is normal practice for low-speed urban road facilities. Gutters may be 0.3 to 1.0 meters wide. A standard kerb and gutter have a width of 0.6 meters is integral with the kerb. Gutters are on the same cross slope as the pavement on the high side and depressed with a steeper cross slope on the low side, usually 10 percent. Typical practice is to place kerbs at the outside edge of shoulders or parking lanes on low-speed facilities. The gutter width may be included as a part of the parking lane. A shoulder gutter is not required adjacent to barrier walls on high fills.
8. A shoulder gutter may be appropriate to protect fill slopes from erosion caused by water from the carriageway pavement. A shoulder gutter shall be considered on fill slopes higher than 5 meters at a side slope of 2:1, or elsewhere where slopes may be subject to erosion. In areas where permanent vegetation cannot be established, shoulder gutter shall be provided on fill slopes higher than 3 meters. Inspection of the existing/proposed site conditions and contact with maintenance and construction personnel shall be made by the designer to determine if vegetation will survive.
9. A shoulder gutter may be appropriate at bridge ends where concentrated flow from the bridge deck would otherwise run down the fill slope. This section of gutter shall be long enough to include the transitions. Shoulder gutters are not required on the high side of super-elevated sections or adjacent to barrier walls on high fills.

7.4.1.3 Roadside Median Channels and Barriers

1. Due to right-of-way limitations, roadside channels cannot be used on most urban arterials. 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.
2. Kerbed road sections are relatively inefficient at conveying water, and the area tributary to the gutter section shall be kept to a minimum to reduce the hazard from water on the pavement. Where practicable, the flow from major areas draining toward kerbed road pavements shall be intercepted by channels as appropriate.
3. It is preferable to slope median areas and inside shoulders to a centre depression (swale), to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities, and for facilities with more than two lanes of traffic in each direction. If used, temporary storage in shallow medians must be carefully engineered to handle high intensity rainfall.
4. Where median barriers are used and, particularly on horizontal curves with associated super-elevations, it is necessary to provide for some relief for the water that accumulates against the barrier. This can be done with weep holes in the barrier. In order to minimise flow across travelled lanes, a more preferred method of relief is to collect the water into a subsurface system that ultimately connects with the main storm drainage system. Slotted drains may be used adjacent to median barriers.
5. Median swales and natural channels should be trapezoidal. The geometrics of a median swale will depend on the width and side slopes (embankment slope or cut slope) adopted for the median.

7.4.1.4 Inlet Structures

1. Inlets should be located to maximise overland flow path, take advantage of pervious areas, and seek to maximise vegetative filtering and infiltration.
2. Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic but should take advantage of natural depression storage where possible.
3. Inlets should be located to serve as overflows for structural stormwater controls. For example, a bioretention device in a commercial area could be designed to overflow to a catch basin for larger storm events.
4. The choice of inlet type should match its intended use. A slumped inlet may be more effective supporting water quality objectives.
5. Use several smaller inlets instead of one large inlet to:
   i. Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.
   ii. Provide a safety factor: If a drain inlet clogs, the other surface drains may pick up the water.
   iii. Improve aesthetics: Several smaller drains will be less obvious than one large drain.
   iv. Spacing smaller drain inlets will give surface runoff a better chance of reaching the drain. Water will have further to travel to reach one large drain inlet.

7.4.1.5 Bridge Deck Design Guidelines and Considerations

1. Drainage of bridge decks is similar to other kerbed carriageway sections. It is often less efficient, because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets or scuppers have a higher potential for clogging by debris. Bridge deck construction usually requires a constant cross slope. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from carriageways shall be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.
2. Zero gradients and sag vertical curves shall be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage shall be 0.2 percent. When bridges are placed at a vertical curve and the longitudinal slope is less than 0.2 percent, the gutter spread shall be checked to ensure a safe, reasonable design.
3. Scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. They also have a low initial cost and are relatively easy to maintain. However, the use of scuppers shall be evaluated for site-specific concerns.
4. Scuppers should not be located over embankments, slope pavement, slope protection, navigation channels, driving lanes, or railroad tracks. Runoff collected and transported to the end of the bridge shall be collected by inlets and down drains. Runoff should also be handled in compliance with applicable storm water quality regulations.
5. The standard scupper drain is four 100 mm in diameter and spaced on 3 m centres, unless spread calculations indicate closer spacing is required. Design using a factor of safety of 2. Scuppers will not be directly discharging onto railroads, roadway travel lanes, shared-use paths, or sidewalks. Provide erosion protection, which could include splash pads or rubble, for scuppers discharging onto erodible surfaces.

7.4.2 Key Design Criteria.

In designing storm drainage, it is important to take into consideration the following:

1. **Capacity** – inadequate capacity is detrimental to both the drainage system and the road structure as water not disposed of correctly and sufficiently leads to flooding and often costly damages.
2. **Blockages** – areas with sandy materials or where runoff carries a lot of debris will experience frequent blockages of storm water drains, which in turn results in flooding of the road structure.
3. **Maintenance** – this involves mainly drain cleaning hence accessibility is of paramount importance. In sandy areas, closed drains tend to clog up quickly and drain clearing is usually difficult or impossible. This leads to poor maintenance of the storm drain, which in turn affects its functionality. In such areas, it is recommended to design open drains or box drains with discrete slabs covering the top of the drain, which can be lifted off to allow easy access for drain clearing.

7.4.3 Design Procedure of Storm Drainage

The design of storm drainage systems involves several steps to derive both the hydraulic and structural parameters. The steps include:

1. **Step 1:** Review relevant information particularly the geometric design – The geometric design provides information on:
   a. **The longitudinal profile** – the longitudinal profile provides information on the slope of the channels. These data are important in the calculation of discharge and flow velocities. It is also used in the calculation of the drainage areas for hydrological calculation.
   b. **Transverse profiles** – the transverse profiles provide information on the cross-sectional profiles for the drainage structures such as the kerbs and channels, ditch edges and median drains.
3. **Step 2:** Hydrological design – The hydrological design involves the calculation of discharge based on rainfall intensity, time of concentration, catchment area and catchment area characteristics (see section 7.2).
4. **Step 3:** Hydraulics design – this involves the determination flow characteristic within the drainage channels. The results of these calculations provide information on discharge at any particular point along the drainage channels and this is checked against their maximum discharge capacities.
5. **Step 4:** Locating outlets and inlets such as chutes, cascades, drainage pits, grated inlets, etc. The main purpose of these outlets is to ensure that the capacity of the drainage structures such as kerbs and channels is not exceeded. If these capacities are exceeded, there could be overflow onto embankments, which may cause erosion, and the flow may encroach excessively on the carriageway posing a safety hazard to traffic such as excessive hydroplaning.
6. **Step 5:** Determining discharge in drainage pipes - This involves the calculation of flow discharges in pipes and calculation of head losses along the conduits and at manholes and junctions. The purpose is to ensure that the drainage system is adequate to accommodate the flow generated by run-off from the road and surrounding area within the drainage catchment area.

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7.4.3.1 Relevant Information for Design

The following information should be obtained by the designer:

1. **The geometric design of the road** – this includes the design report and design drawings. From this, the drainage systems and structures, which are assumed in the geometric design, their location and the longitudinal profile are determined. The Geometric Design Manual should also be referred to for and transverse profiles, specifications of the geometric design elements.
2. **Pavement design of the road** – including specifications for the pavement and surfacing and materials for the kerbs, channels and shoulders. This information is required for the determination of 'n' values for hydraulic design calculations (refer to the Pavement Design Manual for materials specifications).
3. **The hydrological information of the area** i.e, the IDF curves (for rational method); curve numbers, catchment area maps and topographic surveys (refer to RDM 2.1).

7.4.4. Hydraulic Capacity of Storm Drains

7.4.4.1 Manning Equation

The most widely used formula for determining the hydraulic capacity of storm drains of gravity, pressure gravity and pressure flows is the Manning Formula, and it is expressed by the following equation.

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

Equation 7.3

Where,

V = Mean velocity of flow, m/s.

R = Hydraulic radius A/P.

S = Energy gradient line slope, m/m.

n = Manning's roughness coefficient.

In terms of discharge, the above formula become:

$$Q = A\,\frac{R^{2/3}\, S^{1/2}}{n}$$

Equation 7.4

Where,

Q = Discharge rate, m^3/s.

A = Cross sectional flow area, m^2.

R = Hydraulic radius A/P.

S = Energy gradient line slope, m/m.

n = Manning's roughness coefficient.

For storm drains flowing full, the above equations become:

$$V = 0.397\,\frac{D^{2/3}\, S^{1/2}}{n}$$

Equation 7.5

$$Q = 0.312\,\frac{D^{8/3}\, S^{1/2}}{n}$$

Equation 7.6

Where,

D = diameter of pipe, m.

The nomograph solutions of Manning's formula for full flow in circular storm drains is shown in Chart B.14 in the Appendix. Table 7.3 gives representative values of the Manning's coefficient for various storm drain materials.

Table 7.2 Manning's n – values for Urban Drainage Channels

Material	Manning's n	Maximum Flow Speed
Closed conduits		
Concrete	0.01 - 0.012	6
Corrugated metal	0.024	6
Vitrified clay	0.010	6
Brick	0.014	3 - 4.5
Open Paved Channels		
Concrete	0.012	6
Dressed stone in mortar	0.015	5.5 - 6
Random stone in mortar	0.017	5 -5.5
Dry rubble riprap	0.025	3 - 4.5
Brick	0.014	3 - 4.5

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 Manning's formula as shown in Equation 7.7 or by using values given in Table 7.2.

$$S = \frac{(nV)^2}{R^{\frac{4}{3}}}$$

Equation 7.7

Where,

V = Mean velocity of flow, m/s.

R = Hydraulic radius A/P.

S = Energy gradient line slope, m/m.

n = Manning's roughness coefficient.

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Table 7.3 Minimum Slopes Necessary to Ensure 0.9 m/m Storm Drains Flowing Full

Pipe Size (mm)	Full Pipe (m^3/s)	Minimum Slopes m/m					
		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

7.4.4.2 Cole-brook White Formula

Manning's equation only provides an estimate of pipe capacity. For that reason the Colebrook – White formula should be used for more exact calculations in urban areas.

The Cole-brook White Formula calculates the flow velocity as:

$$V = -\sqrt{32gRS}\,\log_{10}\left\{\frac{k}{14888R} + \frac{1.225v}{R\sqrt{32gRS}}\right\}$$

Equation 7.8

Where,

V = Velocity of flow in m/s.

R = Hydraulic Radius (m).

S = Energy gradient (m/m).

v = Kinematic viscosity (m^2/s).

K = Equivalent sand roughness (mm).

g = Gravitational acceleration (m/s^2).

Since the Colebrook White Formula is rather complex, charts are normally used.

7.4.5 Sizing of Drainage Channels and Spacing of Outlets and Grated Inlets

7.4.5.1 Sizing of Drainage Channels

Drainage channels such as kerbs, L-channels and gutters are usually of predetermined dimensions depending on the geometric design or standard specifications. In addition, precast kerbs and L-channels would have predetermined sizes. It follows therefore that the dimensions of these channels and the slope are fixed. The maximum discharge capacities are therefore also fixed.

7.4.5.2 Flow in Gutters/Kerbs, L-Channels and Medians

Runoff from the carriageway and shoulders flows onto the kerbs, gutters or a combination of kerbs and gutters or L-channels or a combination of L-channels and gutters. The cross-slope of the kerbs may be different from the cross-slope of the gutters. The gutters may have a different or similar cross-slope as the shoulders. Hence, various combinations are possible. Medians run in the middle of the carriageway and may be V-shaped or trapezoidal. Their cross-slopes should not pose any hazard to traffic. Figure 7.1 illustrates the differences and the parameters, which are essential for design. The following additional guidelines should be observed.

1. The spread of water in a gutter and onto the travelled way should not exceed the spread and puddle depth (if kerb is used to create the gutter).
2. On long steep embankments where kerbing is warranted to minimise embankment erosion, a grate inlet, drainage structure, and closed chute is preferred to intercept and convey gutter flow down the embankment and ensure that the water will be contained within the channel.
3. In a permanent roadside or median channel, about 150 mm of freeboard is generally considered adequate.
4. Gutters: - Slope will be the same as the vertical alignment of the travelled way.
5. Chutes: - Slope should be parallel to the embankment.
6. Roadway ditches should be parallel to the travelled way. The slope may be greater than the grade of the travelled way to outlet the roadway ditch to a natural channel or another channel (culvert, etc,).
7. Toe-of-Slope ditches should be parallel to the toe-of-slope. The slope may increase in the vicinity of the outlet to a natural channel or another channel (culvert, etc,).
8. Intercepting ditches should be parallel to the top of the cut slope.
9. Median swales should be parallel to the travelled way. The slope may be greater than the grade of the travelled way to increase channel capacity or to outlet the swale to a natural channel or another channel (culvert, etc,).
10. Relocated natural channels should maintain the existing channel slope. Abrupt changes in slope will result in the deposition of transported material or scouring.

Figure 7.1 Typical Gutter Section

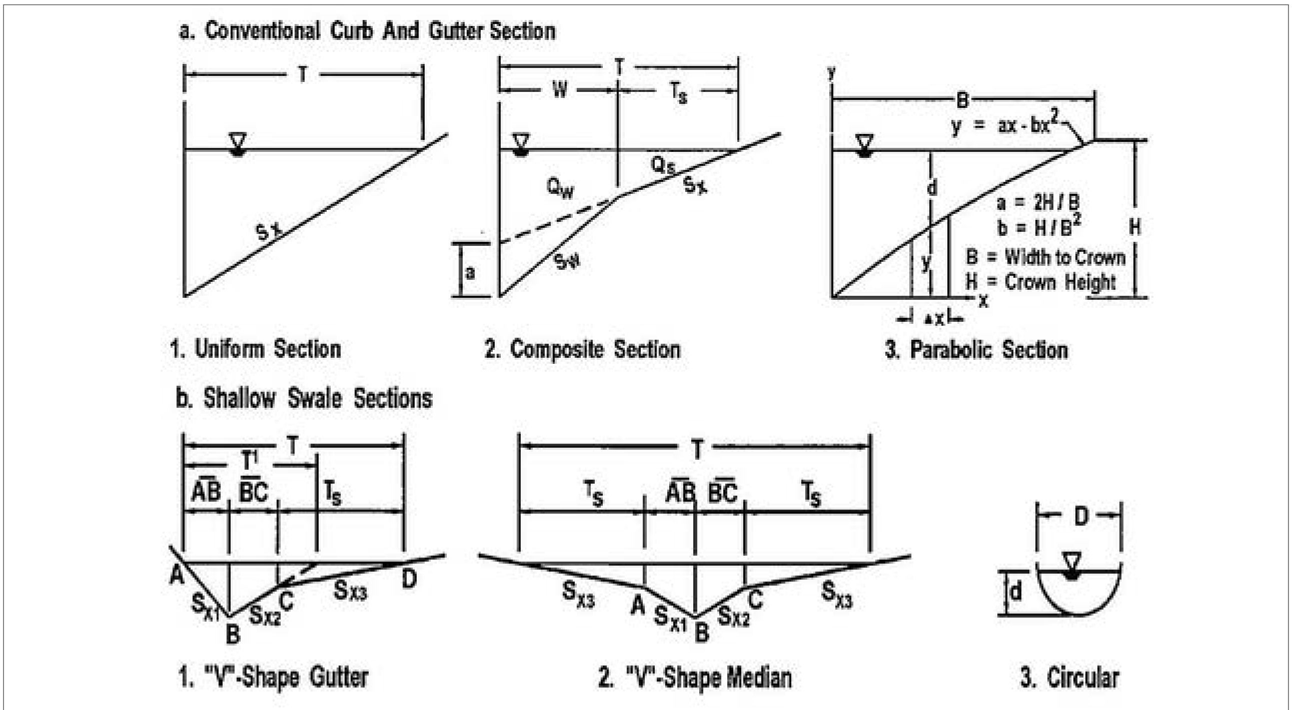
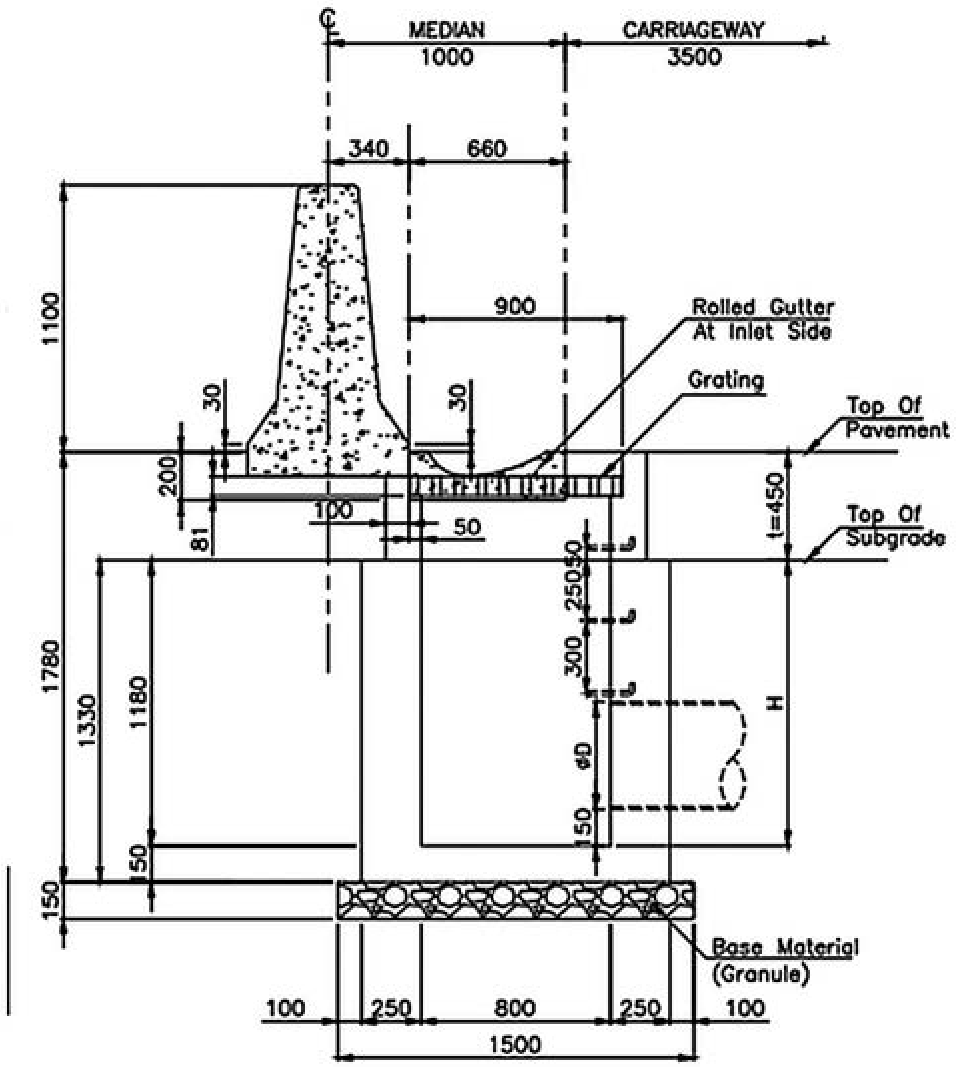


Figure 7.2 Typical Median Drainage Arrangement



Gutter flow calculations are necessary to establish the spread of water on the shoulder and pavement section. A modification of Manning's equation can be used for computing flow in triangular channels. The modification is necessary because 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 increment of width across the section.

7.4.5.3 Calculation of Kerb and Gutter Flow

1. **Spread**

   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 parking lanes or shoulders, and portions of the travelled surface.

   Spread distance, T, is measured perpendicular to the kerb face to the extent of the water on the roadway and is shown in Figure 7.1 above.

Table 7.4 Recommended Minimum Design Frequency and Spread

Road Classification		Design Frequency	Design Spread
High Volume	< 70 km/hr	5-year	Shoulder + 1 m
Medium Volume	> 70 km/hr	5-year	Shoulder
Directional	Sag Point	10-year	Shoulder + 1 m
Collector	< 70 km/hr	5-year	1/2 Driving Lane
	> 70 km/hr	5-year	Shoulder
	Sag Point	5-year	1/2 Driving Lane
Local Streets	All classes	2-year	1/2 Driving Lane

2. Flow in Gutters

Gutter Flow calculations are necessary to establish the spread of water on the shoulder, parking lane, or pavement section. To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. The resulting equation is:

$$Q = \frac{K_u}{n} S_x^{1.67} S_L^{0.5} T^{2.67}$$

Equation 7.9

or in terms of T

$$T = \left[\frac{Qn}{K_u S_x^{1.67} S_L^{0.5}}\right]^{0.375}$$

Equation 7.10

Where,

K_u = 0.376.

n = Manning's coefficient (Table 7.2).

Q = Flow rate, m³/s.

T = Width of flow (spread), m.

S_L = Longitudinal slope, m/m.

S_x = Cross slope, m/m.

Equation 7.9 neglects the resistance of the kerb face since this resistance is negligible.

Spread on the pavement and flow depth at the kerb are often used as criteria for spacing pavement drainage inlets.

Nomographs are used to solve gutter flow equations. Chart B.17 is a nomograph for solving Equation 7.9. The chart can be used for either criterion with the relationship:

$$d = T S_x$$

Equation 7.11

Where,

d = Depth of flow, m.

S_x = Cross slope, m/m.

Nomograph in Chart B.21 also solves Equation 7.9 for gutters having triangular cross sections.

Worked examples to illustrate use of nomograph for the analysis of conventional gutters with uniform cross slope are given in Appendices A.16 to A.18.

3. Gutter Spacing

Spacing of the gutters is determined either by rule of thumb: every 45 m, or one gulley for every 200m^2 of catchment area.

Alternatively, the following formula is used:

$$\text{Gulley Spacing } y = \frac{328 * S^{0.5}}{W}$$

Equation 7.12

Where,

S = Slope of Pavement in %.

W = width of Pavement in metres.

4. Composite Gutter Sections

Chart B.17 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.

***Note:** Chart B.19 can also be used to calculate the flow in a composite gutter section. Similarly Charts B.17 and B.18 are used for design of triangular gutters and circular respectively.*

7.4.5.4 Inlet and Bridge Deck Drainage Design

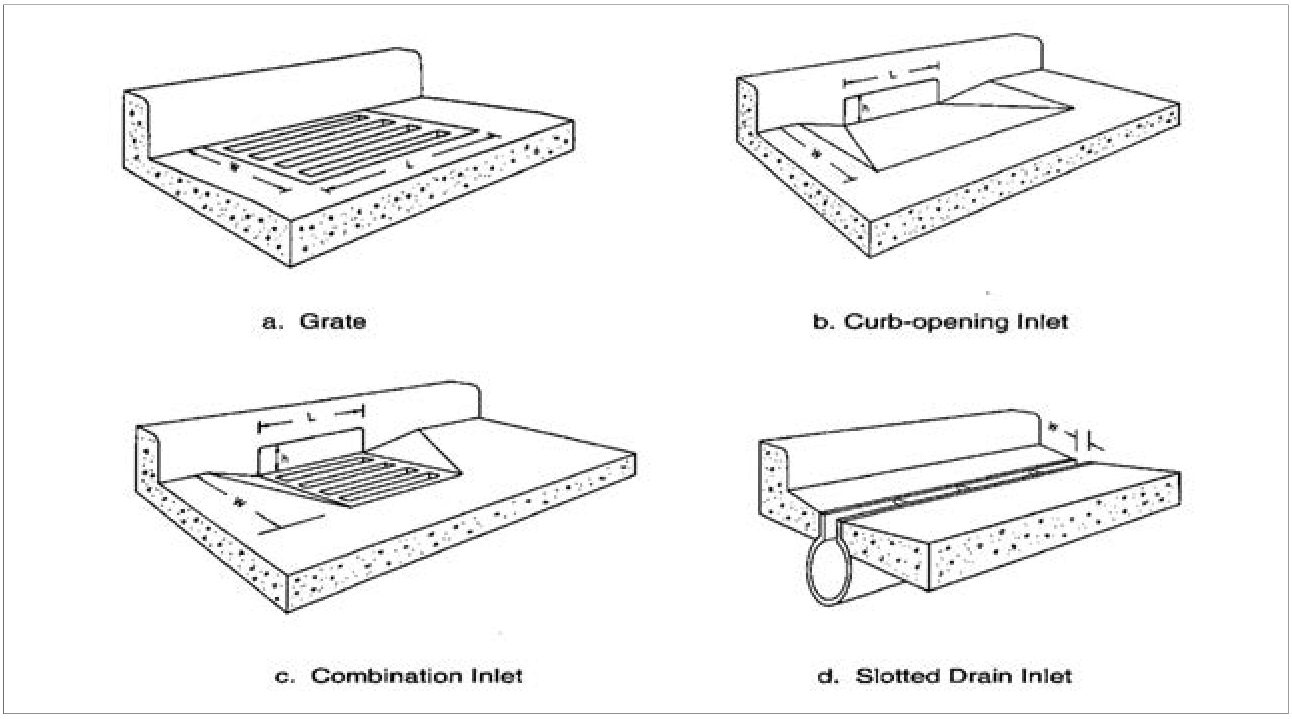
The hydraulic capacity of a storm drain inlet and bridge deck depends upon their geometry as well as the characteristics of the gutter flow.

1. Inlet Types

Inlets used for the drainage of road surfaces can be divided into the following four classes:

a. Grate inlets
b. Kerb-opening inlets
c. Combination inlets
d. Slotted inlets

Figure 7.3 Classes of Storm Drain Inlets



2. **Locating and Spacing of Outlets and Grated Inlets**

There are a number of locations where inlets may be necessary with little regard to contributing drainage area.

a. At all low points in the gutter grade.
b. Immediately upstream of median breaks, entrance/exit ramp gores, cross walks, and street.
c. Intersections, i.e, at any location where water could flow onto the travel way
d. Immediately upgrade of bridges (to prevent pavement drainage from flowing onto bridge decks).
e. Immediately downstream of bridges (to intercept bridge deck drainage).
f. Immediately upgrade of cross slope reversals.
g. Immediately upgrade from pedestrian cross walks.
h. At the end of channels in cut sections.
i. On side streets immediately upgrade from intersections.
j. Behind kerbs, shoulders or sidewalks to drain low area.

In order to avoid discharge from exceeding the capacity of the drainage channels or structures outlets shall be placed along the drainage channels to remove excess flow.

3. **Process of Determining Inlet Locations:**

Start at a high point on the roadway grade, and work towards the low point. Then begin at the next high point and work down.

a. Carry out the hydrological design and determine the design discharge from the catchment area.
b. Select a trial drainage area approximately 90 to 150 m long below the high point and outline the area on the plan Include any area that may drain over the kerb, onto the roadway. However, where practical, drainage from large areas behind the kerb should be intercepted before it reaches the roadway or gutter.
c. Determine inlet sub catchments and calculate the flow widths along roads and streets.
d. Compare the design discharge with the maximum allowable discharge. If the design discharge is lower than the maximum allowable discharge, move the position of the outlet or grated inlet further down-slope of the channel and repeat steps (2), (3) and (4). If the design discharge is greater than maximum allowable discharge, then move the position of the outlet or grated inlet up-slope along the channel to reduce spacing.
e. Determine of amount of discharge, which is diverted into the outlets or grated inlets. Consider the efficiency of the outlets and grated inlets in diverting part of the flow out of the drainage channels.
f. Once the position of the first outlet or grated inlet is determined and the outflow and discharge that continues down the drainage channel are calculated then distance to the next outlet or grated inlet can be determined.
    i. The position of the outlet or grated inlet is then selected between 90m and 150m.
    ii. Flow that passes the first outlet or grated inlet should be added to the discharge calculated for the drainage area of the next outlet or grated inlet.
    iii. The flow that passes the first outlet or grated inlet is calculated by subtracting the intercepted flow from the design capacity of the drainage channel at the point where the outlet of grated inlet is positioned. Use the appropriate nomographs to determine intercepted flow for given depth of flow in the channels (See Charts B.21 to B.27 in Appendix B).
g. The total discharge is then computed for the second outlet or grated inlet through the iterative process described in (3) and (4) above to design the spacing between the first and the second outlet or grated inlet.
h. Repeat the iterative process for subsequent outlets or grated inlets.

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4. Grate 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. Chart B.29 shows splash-over velocities for various grate configurations, and the portion of frontal flow intercepted by the grate.

The ratio of frontal flow to total gutter flow, E_o for straight cross slope is expressed by the following equation:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67}$$

Equation 7.13

Where,

Q = Total gutter flow, m³/s.

Q_w = flow in width W, m³/s.

W = width of depressed gutter or grate, m.

T = total spread of water in the gutter, m.

Chart B.18 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:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o$$

Equation 7.14

Where,

Q = Total gutter flow, m³/s.

E_o = The ratio of frontal flow to total gutter flow.

Q_w = Flow in width W, m³/s.

Q_s = Side flow, m³/s.

The ratio of frontal flow intercepted to total frontal flow, Rf, is expressed by the following equation:

$$R_f = 1 - 0.295\,(V - V_o)$$

Equation 7.15

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. Chart B.29 provides a solution of Equation 7.15 which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The average gutter velocity (total gutter flow divided by the area of flow) is needed to use Chart.29. This velocity can also be obtained from Chart B.21.

The gutter velocity is needed to use Chart B.29 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:

$$R_s = 1 / \left[1 + \frac{0.0828V^{1.8}}{S_x L^{2.3}}\right]$$

Equation 7.16

Where,

V = velocity of flow in gutter, m/s.

L = length of the grate, m.

S_x = cross slope, m/m.

Chart B.30 provides a solution to Equation 7.16, with $Q = AV$.

The efficiency, E, of a grate is expressed as:

$$E = R_f E_o + R_s(1 - E_o)$$

Equation 7.17

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q[R_f E_o + R_s(1 - E_o)]$$

Equation 7.18

5. 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:

$$Q_i = CPd^{1.5}$$

Equation 7.19

Where,

P = perimeter of grate excluding bar widths and the side against the kerb, m.

C = 1.66.

d = depth of water at kerb measured from the normal cross slope gutter- flow line, m.

The capacity of a grate inlet operating as an orifice is:

$$Q_i = CA(2gd)^{0.5}$$

Equation 7.20

Where,

C = 0.67 orifice coefficient.

A = clear opening area of the grate, m^2.

g = gravitational acceleration, 9.81 m/s^2.

Chart B.22 is a plot of Equations 7.19 and 7.20 for various grate sizes.

6. Kerb Inlet on Grades

Kerb-opening inlets are effective in the drainage of road pavements where flow depth at the kerb is sufficient for the inlet to perform efficiently.

The length of kerb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}\left(\frac{1}{nS_x}\right)^{0.6}$$

Equation 7.21

Where,

K = 0.817.

L_T = Kerb opening length required to intercept 100% of the gutter flow, m.

S_x = Cross slope, m/m.

n = Manning's coefficient (Table 7.2)

S = Slope of Pavement in %.

Q = Total interception gutter flow.

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The efficiency of kerb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}$$

Equation 7.22

Where,

L = Kerb-opening length, m.

L_T = Kerb opening length required to intercept 100% of the gutter flow, m.

Chart B.24 is a nomograph for the solution of Equation 7.21 and Chart B.25 provides a solution of Equation 7.22.

The length of inlet required for total interception by depressed kerb-opening inlets or kerb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, Se, in Equation 7.21.

$$S_e = S_x + S'_w E_o$$

Equation 7.23

Where,

S'_w = Cross slope of the gutter measured from the cross slope of the pavement.

S'_w = (a/1000$_W$) W, or S_w - S_x

S_x = Cross slope, m/m

a = Gutter depression, mm

W = Width of depression, m

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 calculate the length of kerb-opening by substituting S_e for S_x in Equation 7.21. The prefix 'w' denotes the width of depression for curb opening inlets. (See Figure 7.1)*

7. Kerb Inlet in Sag

The capacity of a kerb-opening inlet in a sag depends on water depth at the kerb, the kerb opening length and the height of the kerb opening. The inlet operates as a weir to depths equal to the kerb 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 kerb-opening inlet operating as a weir is:

$$Q_I = C_w(L + 1.8W)d^{1.5}$$

Equation 7.24

Where,

C_w = 1.27.

L = Length of kerb opening, m.

W = Width of depression, m.

d = Depth of water at kerb measured from normal cross slope gutter flow line, m.

The weir equation for kerb-opening inlets without depression becomes:

$$Q_I = C_w L d^{1.5}$$

Equation 7.25

The depth limitation for operation as a weir becomes: $d \leq h$

Kerb-opening inlets operate as orifices at depths greater than approximately 1.4 x height of kerb-opening.

The interception capacity can be computed by:

$$Q_i = C_o A \left[2g\,(d_i - \frac{h}{2})\right]^{0.5}$$

Equation 7.26

Where,

C_o = Orifice coefficient (0.67).

h = Height of kerb-opening orifice, m.

A = Clear area of opening, m^2.

d_i = Depth at lip of kerb opening, m.

g = Gravitational acceleration, 9.81 m/s^2.

Note: *Equation 7.26 is applicable to depressed and un-depressed kerb-opening inlets and the depth at the inlet includes any gutter depression.*

8. Slotted Inlets on Grade

a. Longitudinal Placement

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:

$$L_T = KQ^{0.42}S^{0.3}\left(\frac{1}{nS^x}\right)^{0.6}$$

Equation 7.27

Where,

K = 0.817

L_T = Slotted inlet length required to intercept 100% of the gutter flow, m.

Sx = Cross slope, m/m.

n = Manning's coefficient (Table 7.2).

S = Slope of Pavement in %

The slot width must be at least 44 mm for Equation 7.27 to be valid.

The efficiency of slotted inlets shorter than the length required for total interception is expressed by:

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}$$

Equation 7.28

Where,

L = slotted inlet length, m.

L_T = slotted inlet length required to intercept 100% of the gutter flow, m.

Chart B.24 is a nomograph for the solution of Equation 7.27, and Chart B.25 provides a solution of Equation 7.28.

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 7.29:

$$S_e = S_x + S'_w E_o$$

Equation 7.29

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 Chart B.18)

Note: *The same equations are used for both slotted inlets and kerb-opening inlets.*

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b. Transverse Placement

At locations where it is desirable to capture virtually all of the flow in the kerbed 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 lineal meter of drain on longitudinal slopes of 0% of 6%.

Capacity curves are available from the manufacturer. The ideal installation would utilise 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.

9. Slotted Inlets in Sag

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:

$$Q_i = 0.8LW(2gd)^{0.5}$$

Equation 7.30

Where,

W = width of slot, m.

L = length of slot, m.

d = depth of water at slot, m.

g = gravitational acceleration, 9.81 m/s^2.

For a slot width of 44 mm the above equation becomes:

$$Q_i = 0.156Ld^{0.5}$$

Equation 7.31

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. Chart B.28 provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

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 7.5 shows the spacing required for various depths at kerb 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 kerbed storm drainage.

Table 7.5 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 d = depth at kerb (m)
3 L = length of the vertical curve
4 A = algebraic difference in approach grades
5 Drainage maximum K = 74m/% (K is ratio L/A)

11. Bridge Deck:

Bridge deck drainage is similar to that of kerbed roadway sections.

7.4.6 Hydroplaning

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:

a. Design the road geometries including cross-fall greater than 2.5% to reduce the drainage path lengths of the water flowing over the pavement. This will prevent flow build-up.

b. 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.

c. 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 tire. This releases any hydrodynamic pressures that are created and reduces the potential for the tire to hydroplane.

d. 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.

7.4.7 Computer Programs

Hand computations for design of storm drain are lengthy and time consuming. 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 analyse the performance of a storm drain system under assumed inflow conditions. It includes ditch design procedure and recommended shape, type in comparison to various factors. Others include Storm Drainage Systems - Storm & Sanitary, Hydro CAD. WMS – Watershed Modelling System (WMS), among others.

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Appendices

Appendix A: Worked Examples

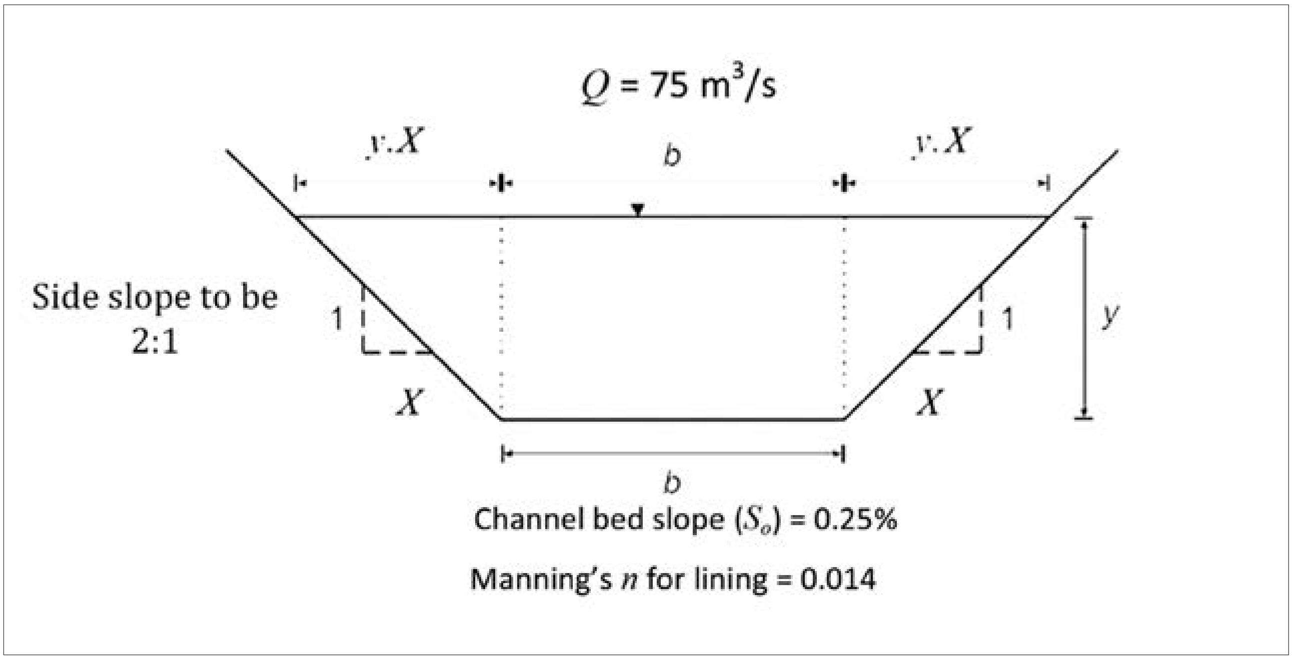
A.1 Determine Design Discharge (Q), Bed Slope (So). Manning's n Value and Channel Side Slope

This example describes the process to design an open channel based on flow rate, selected channel side slopes and other key properties as discussed in sub section 2.3 to 2.5

Given the following channel cross-section and data, and using the channel bed slope to approximate the Friction Slope (S_f), design a concrete-lined, trapezoidal channel.

Given Channel cross-section:

Figure A1-1 Cross-section



Solution:

For best hydraulic section;

Hydraulic radius $R = \dfrac{by + Xy^2}{2\left\{y^2 + (2y)^2\right\}^{½} + b} = \dfrac{y}{2}$

Equation A1

Step 1

Substitute $X = 2$ and resolve for b as a function of y.

$$R = \frac{by + Xy^2}{2\left\{(y^2 + (Xy)^2\right\}^{½} + b} = \frac{y}{2}$$

$$b = y(2\sqrt{5}-4)$$

Step 2

Combine Manning's Equation 2.15 and Equation A1 above, then substitute in Q = 75 m³/s, $A = by + Xy2$, $R = y/2$, So = 0.0025 (remember; So estimates Sf) and n = 0.014:

$$75 = \frac{(b + 2y^2)\left(\frac{y}{2}\right)^{2/3} 0.0025^{1/2}}{0.014}$$

$$21 = (by + 2y^2)\left(\frac{y}{2}\right)^{2/3}$$

Now substitute:

$$b = y(2\sqrt{5}-4)$$

The expression becomes:

$$21 = (y(2\sqrt{5}-4))y + 2y^2)\left(\frac{y}{2}\right)^{2/3}$$

and y = 2.65 m, depth of flow.

Step 3

Calculate b using y:

$$b = y(2\sqrt{5}-4)$$

b = 2.65 x $(2\sqrt{5}-4)$ = 1.25 m, width of bed.

Assume freeboard of 950 mm

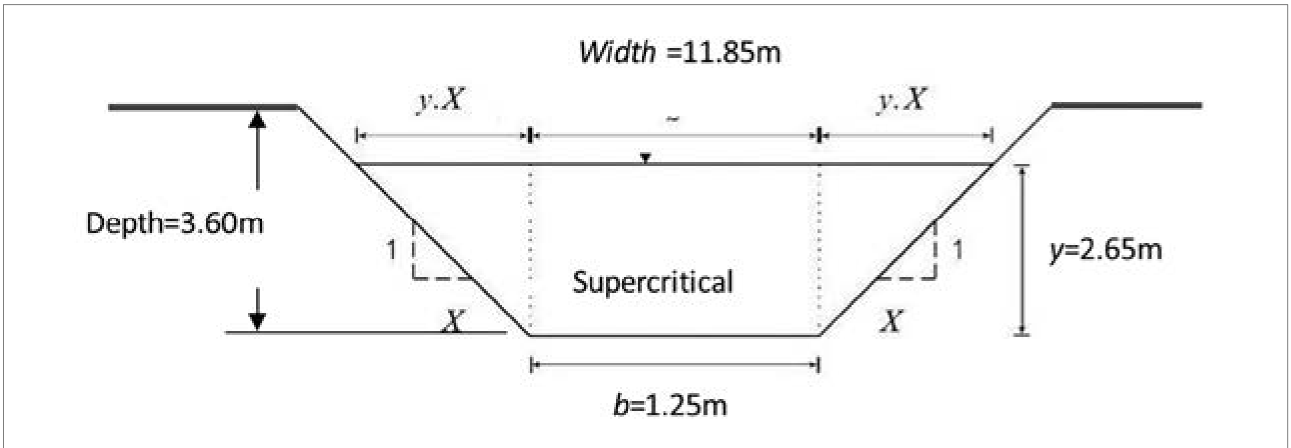
Step 4

Check the Froude Number (Equation 2.7: $F_r = 75\sqrt{\frac{11.85}{g\ 17.36^3}}$ = 1.14 (Supercritical))

Step 5

Determine final channel geometry:

Figure A1.2 Cross-section



For trapezoidal channels, value of flows plotted against channel geometrics and depth of flow are given in Table 2.2.

A.2 Analysis of Channels Using Nomographs

Given: A trapezoidal channel with the following characteristics:

S_o = 0.01
B = 0.8 m
z = 3
d = 0.5 m

Using Nomographs find: The channel capacity and flow velocity if the channel is lined with a turf reinforcement mat with an n-value of 0.030.

Solution

Using Chart B.1;

d/B	= 0.5/0.8
	= 0.63
Q_n	= 0.05 m³/s - Chart B.1; $Q = Q_n /n$
	= 0.05/0.030
Therefore channel Capacity:	Q_n = 1.67 m³/s
Flow velocity:	$V = Q/A$
	= 1.67/1.15
Therefore flow velocity,	V = 1.67 m³/s

A.3 Gradually Varying River Flow (Backwater Calculation – Simple Sectional Details)

Determine the flood level at section 3 for a river of trapezoidal section with side slopes 1:2 and varying bed width. The characteristics of the cross-sections are reflected in Table A3.1.

Table A3.1 Characteristics of the River Cross-sections

Section	Base width (m)	Bed level (m)	Chainage (m)	Manning, n (s/m$^{1/3}$)	Remark
1	6	1203.02	0	0.032	Downstream
2	4.8	1203.24	65	0.026	
3	5.6	1203.75	147	0.024	Site
4	5.4	1203.99	214	0.028	
5	5.6	1204.42	280	0.024	Upstream

Using the principle of conservation of mass and energy would solve this problem. It is assumed that the flow rate is constant at 4.33 m³/s. It is necessary to determine the type of flow to establish the control, and then to work away from the control.

Although the assumption that uniform flow will be present at the cross-sections is incorrect, calculation of the *normal flow depth* at each section will give an indication of the type of flow.

In Table A3.2 the *normal flow depths* have been calculated. This is not the solution to the problem but merely a way to establish the type of flow.

Solution

From Table A3.2 it could be concluded (Fr < 1) that the flow will be subcritical and hence that the control will be downstream.

Table A3.2 Flow Characteristics

Section ID	Position ID	Invert (m)	Calculation based on uniform flow assumption						
			Slope (local) (m/m)	Y_n (m)	A (m)	P (m)	R (m)	Cal Q (mt/s)	Fr
a*	b	c	d	e	f	g	h	i	j
1	Downstream	3.02	0.003	1.980	19.718	4.854	1.327	43.3	0.589
2		3.24	0.006	1.653	13.403	2.194	0.973	43.3	0.952
3	Site	3.75	0.004	1.725	15.614	3.316	1.027	43.3	0.792
4		3.99	0.007	1.622	14.026	2.656	0.958	43.3	0.908
5	Upstream	4.42							

Note: ** Refer to the legend table (Table A3.4)*

Table A3.3 Flood Level Calculations

Section ID	ΔX (m)	H Total Energy (m)	Fr (m)	Area (m^2)	P (m)	Velocity (m/s)	E_1 (m)	S_1 (m/m)	(S_0-S_1) Avg (m)	E_2 (m)	$\Delta(E_2-E_2)$ (m)	Water (m)
a*	k	l	m	n	o	p	q	r	s	t	u	v
1		1205.246	0.589	19.718	14.854	2.196	1205.246	0.00338				1205
	65								0.000198	1205.245	-0.001	
2		1205.544	0.675	17.499	13.709	2.474	1205.544	0.00299				1205.23
	82								0.002349	1205.545	0.00	
3		1205.832	0.904	14.105	12.765	3.070	1205.832	0.00475				1205.35
	67								-0.00062	1205.833	0.001	
4		1206.195	0.692	17.282	13.829	2.506	1206.195	0.00366				1205.87
	66								0.002394	1206.195	0.00	
5		1206.505	0.889	14.286	12.832	3.031	1206.505	0.00459				1206.04

Note: ** Refer to the legend table (Table A3.4)*

1 2 3 4 5 6 7 8 9 Appendices

Now start with the assumption of a flow depth at section 1 (downstream) and work upstream by applying the continuity of energy as shown in Table A3.3. Assume that the secondary losses will be negligible.

The flow depth at section 1 is assumed to be 2.258 m. The total energy level at this section is then equal to 2.427 m.

Notes: *A brief description of the columns content for the above tables is listed in Table A3.4 below:*

Table A3.4 Legend Table

Column ID	Description of the variable
a	Section identification
b	Description of the position
c	Invert level (m)
d	Local slope calculated from the level difference between that of the section and the upstream section divided by the distance between the sections
e	Y_n is the calculated flow depth assuming that uniform flow characteristics will occur
f	Calculated area for the given Y_n
g	Calculated wetted perimeter for the given Y_n
h	Calculated hydraulic radius for the given Y_n
i	Calculated flow rate
j, m	Froude number
k	Distance between the sections
n	Calculated area
o	Calculated wetted perimeter
p	Calculated velocity
q	Calculated total energy
r	Calculated energy slope
s	Calculated difference of the energy slope and the channel slope
t	Specific energy
u	Difference in the specific energy
v	Calculated water level

A.4 Hydraulic Design of a Drift Crossing

Problem:

A **river crossing structure is to be provided** for a tertiary road linking rural settlements on both banks of a river. No structure exists and vehicles such as tractors, four-wheel drive vehicles, LDVs and donkey carts cross via the sandy riverbed. The route is not accessible for motorcars. Motorcars use an alternative route via the main road with a length of 45 km.

Test pits were excavated in the sandy riverbed. Solid rock was encountered at depths varying between 1.2 m and 2.0 m.

The approach gradients of the road are moderate and there is no horizontal curvature. The preliminary design of the vertical alignment of the road across the structure to be provided has also been done. The straight section in the middle (L_z) has a length of 20 m. and K_j and K_3 are both 4 m (refer to Figure 3.4). The slope of the road on the southern bank is - 5.6%. and on the northern bank 7.0%.

The deck thickness is taken as 500 mm, and the soffit of the deck is on average 1400 mm above the riverbed.

Solution:

Design flow rate

On the basis of the hydrological investigations (see Chapter see Chapter 4 and 5) a design flow of 60m³/s has been determined. As the structure is meant to overflow on a regular basis this is the 1 year storm water runoff.

Cross section

Due to economic reasons a single lane structure is chosen. The cross fall in the direction of flow is taken as 2%.

Selection of structure

Because of good, but uneven founding conditions a low-level bridge is opted for. Six spans of 6 m each fit the river cross-section well. Piers are 300 mm thick.

The capacity of the structure is determined as the sum of the flow that can be accommodated over the structure and through the structure .

Flow over the structure

Assume supercritical flow and decide on a maximum flow depth of 0.1 m (d). The flow that can be accommodated over the structure is determined from Equation 3.1 in the main text. So is 0.02 (2% as above) and Manning n for concrete is 0.016 s/m1l3. The cross-section area of flow is determined as follows (Equation 3.2):

$$A_{over} = \frac{1}{3}\, d\sqrt{800K_1 d} + dL_2 + \frac{1}{3}\, d\sqrt{800K_3 d}$$

Where K_1 and K_3 are both 4 and L_2 is 20m.

A_{over} = 3.19m^2

$$P_{over} = \frac{1}{2}\sqrt{800K_1} + L_2\,\frac{1}{2}\sqrt{800K_3 d} = 37.89$$

From equation 3.

$$Q_{over} = \frac{A_{over}\frac{5}{3}(S_0)^{\frac{1}{2}}}{nP_{over}^{\frac{2}{3}}} = \frac{3\ 3.19^{\frac{5}{3}}(0.02)^{\frac{1}{2}}}{(0.016)(37.89)^{\frac{2}{3}}}$$

$$Q_{over} = 5.42\ \text{m}^3/\text{s}$$

Flow passing through the structure

Assume outlet control, then $Q_{under} = V_{under} * A_{eff}$

V under is determined from Equation 3.5 for which the following is required:

$$h = \frac{V_2^{\,2}}{2g} + d$$

Where,

$$V_2 = \frac{Q_{over}}{A_{over}} = 1.7\ \text{m/s}$$

h = 0.247 m

H_1 = $h + x + D$, where h is as above, x = 0.5 m and D = 1.4 m

H_1 = 2.147 m

H_2 = D- $L_B\, S_o$, where L_B = (4.0) +(2)(0.25) = 4.5 m (for the guide blocks)

H_2 = 1.4 (4.5)(0.02)

H_2 = 1.31 m

Assume $K_{inl.}$ = 0.5 and $K_{outl.}$ = 1.0 (both sudden transition), then

C = 6 * (0.5 + 1.0), see Equation 7.10

C = 9

P_{cell} is the total wetted perimeter of each cell (m)

P_{cell} (5.7)*(2) + (1.4)*(2) = 14.2m

$$n_{cell} = \frac{P_{concrete}{}^{*}\, n_{concrete}}{P_{cell}} + \frac{P_{river}{}^{*}\, n_{river}}{P_{cell}}$$ and assume n_{over} = 0.03s/m$^{1/3}$

n_{cell} = 0.022 s/m$^{1/3}$

$P_{eff} = P_{cell}$ = (6)(14.2) = 85.2 m

$$n_{eff} = \frac{(6)(0.02)(14.2)}{85.2} = 0.022 \text{ s/m}^{1/3}$$

$$R = \frac{A_{eff}}{P_{eff}}$$; where A_{eff} = (6)(5.7)(1.4) = 47.88 m^2

R = 0.562

V_{under} from Equation 3.5 is:

$$V_{under} = \sqrt{\frac{H_1 - H_2}{\frac{C}{2g} + \frac{n_{eff}{}^2 L_B}{R^{4/3}}}}$$

$$V_{under} = \sqrt{\frac{2.147 - 1.31}{\frac{9}{2\text{x}9.81} + \frac{0.022^2\,(4.5)}{0.562^{4/3}}}}$$

V_{under} = 1.34 m/s

Also from Equation 3.4 : $Q_{under} = V_{under}\, A_{eff}$ = 64.16m^3/s

Design Discharge

The capacity of the structure at design level $Q_{over} + Q_{under}$ = 69.6 m^3/s

As $Q_{over} + Q_{under}$ is larger than Q design (60m^3/s) the design is complete as the structure is adequate. If this was not the case, the level of the deck would have to be adjusted, and the calculation be redone.

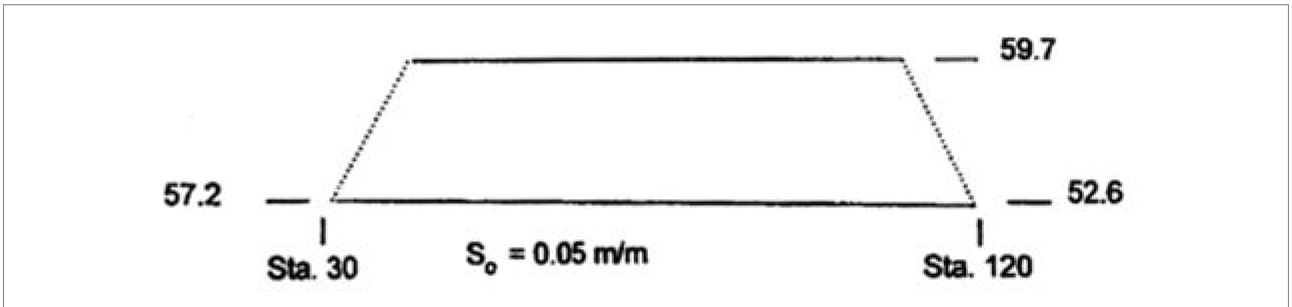
A.5 Design Procedure For a Culvert

The following example problem follows the design procedure steps described in Section 4.3.

Step 1: Assemble Site Data and Project File

A: Site survey project file contains:

Figure A5.1 Cross-section



i. Roadway profile; and
ii. Embankment cross section (see Figure A5.1 above).

B: Site visit notes indicate:

i. No sediment or debris problems; and
ii. No nearby structures.
iii. Studies by other agencies – none.

C: Environmental risk assessment shows:

- **i.** No buildings near floodplain;
- **ii.** No sensitive floodplain values; and
- **iii.** Convenient detours exist.

D: Design criteria:

- **i.** 50-year frequency for design; and
- **ii.** 100-year frequency for check.

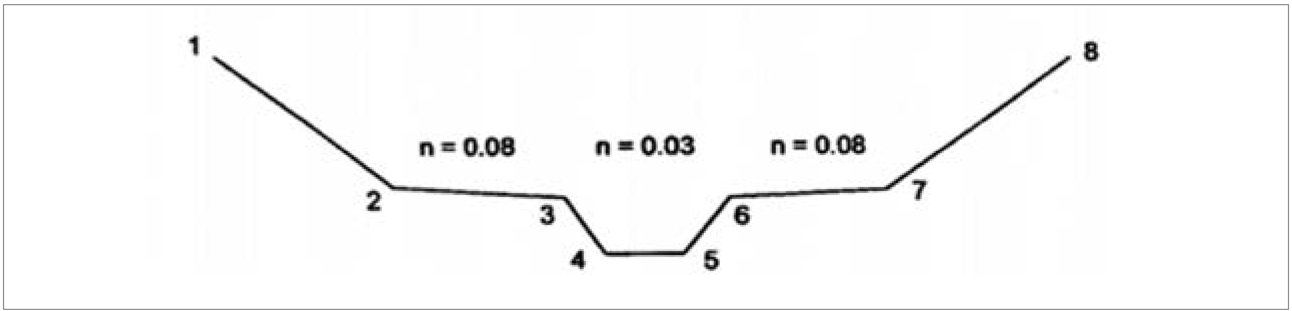
Step 2: Determine Hydrology

A: TRRL equations yield:

- **i.** Q_{50} = 11.3 m³/s
- **ii.** Q_{100} = 14.16m³/s

Step 3: Design Downstream Channel (see Figure A5.2 below)

Figure A5.2 Cross-Section of Channel (Slope = 0.05 m/m)



Step 4: Summarise the Design Form (Figure 4.11)

Table A5.1 River Profile

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

Table A5.2 Hydraulic Analysis Results

Q (m³/s)	TW (m)
2.83	0.43
5.66	0.63
8.50	0.76
11.33	0.85
14.16	0.93

Step 5: Select Design Alternative

Shape - Box

Size - 2135 mm by 1830 mm Material – Concrete

Entrance Wingwalls. 45° bevel, rounded

Step 6: Select Design Discharge

(Q_d = Q_{50} = 11.33 m³/s)

Step 7: Determine Inlet Control Headwater Depth (HW_i)
Use inlet control nomograph - Charts B.10, D = 1.83 m.

a. $\frac{Q}{B} = \frac{11.33}{2.13} = 5.32.$

b. $\frac{HW}{D} = 1.27$ for 45° bevel.

c. $HW_i = \left(\frac{HW}{D}\right)D = (1.27)1.83 = 2.32$ 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 m³/s.

b. d_c = 1.43 m from Chart B.11.

c. $\frac{d_c + D}{2} = \frac{1.43 + 1.83}{2} = 1.63$ m.

d. h_o = the larger of TW or ($d_c + \frac{D}{2}$), $h_o = \frac{d_c + D}{2} = 1.63$ m.

e. K_e = 0.2 from Table 4.4.

f. Determine (H) - use B.12.

- **i.** Ke scale = 0.2.
- **ii.** Culvert length (L) = 90 m.
- **iii.** n = 0.012 same as on chart.
- **iv.** Area = 3.90 m².
- **v.** 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: $HW_r = HW_c - HW_{ov} = 2.32 – 2.59 = -0.27$m

b. If HW_r 0, $Q_r = 0$

Step 11: Compute Total Discharge (Q_t)
$Q_t = Q_d + Q_r$ = 11.33 m³/s + 0 = 11.33 m³/s

Step 12: Calculate Outlet Velocity (V_o) and Depth (d_n)

Inlet Control

a. Calculate normal depth (d_n):

$$Q = \left(\frac{1}{n}\right)A\,R^{2/3}\,S^{1/2} = 11.33 \text{ m}^3/\text{s}$$
$$= \left(\frac{1}{0.012}\right)(2.13{*}d_n)\left(\frac{2.13{*}d_n}{2.13 + 2d_n}\right)^{2/3}(0.05)^{0.5}$$
$$= (2.13{*}d_n)[2.13{*}d_n/(2.13 + 2d_n)]^{2/3} = 0.608$$

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

c. $V_o = \frac{Q}{A} = \frac{11.33}{1.17} = 9.68$ m/s

1 2 3 4 5 6 7 8 9 Appendices

Step 13: Review Results

Compare alternative design with constraints and assumptions, if any of the following are exceeded repeat Steps 5 through 12.

a. Barrel has (2.59 m – 1.83 m) = 0.76 m of cover.

b. L = 90 is OK, since inlet control.

c. Headwalls and wingwalls fit site.

d. Allowable headwater (2.59 m) > 2.32 m is ok.

e. 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 A5.4 that follows this example.

Step 15: Related Designs

Consider the following options (Subsection 4.1)

a. Consider tapered inlets, culvert is in inlet control and has limited available headwater

  i. No flow routing, a small upstream headwater pool exists

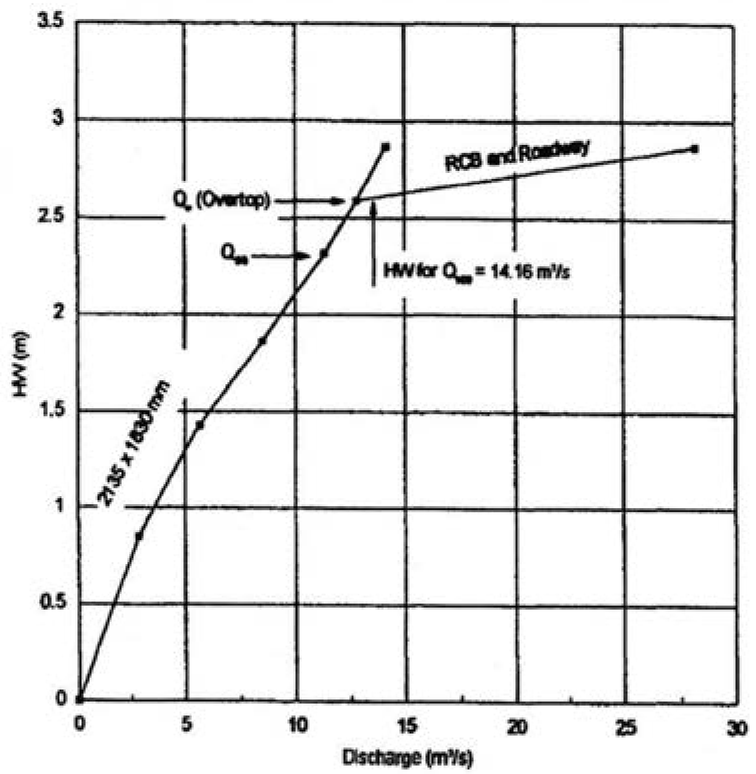
  ii. Consider energy dissipators since V_o = 9.5 m/s > 6 m/s in the downstream channel

  iii. No sediment problem

Step 16: Documentation

Report prepared and background filed.

Figure A5.3 Performance Curve for Design Example



A.6 Designing a Culvert Using HY – 8

The hand solution in application example 1 are duplicated using HY-8: The procedure and results are tabulated below:

1. Enter Crossing Data.
2. Select the following options: U.S. Customary Units. Outlet Control Full flow and Exit Loss Standard Method.
3. Analyse Crossing brings up Crossing Summary Table that shows there is overtopping. Full overtopping occurs at 59.9m³/s. The total flow is 14.16 m³/s, culvert taking 12.96 m³/s and 1.19 m³/s passing over the roadway.
4. The outlet velocity at 11.33 m³/s is 9.62 m/s which compares well to 9.68m/s obtained by manual computation
5. Other results may vary slightly because different assumptions are made in HY-8. For example, while in the example both irregular channel profile and tailwater depths are given, in HY-8 you choose either the irregular profile or tailwater profile and not both.
6. Tables A6.1 to A6.3 show results of HY-8 hydraulic analysis

Table A6.2 Downstream Channel Rating Curve

Flow (m³/s)	Water Surface Elevation (m)	Depth (m)	Velocity (m/s)	Shear (Pa)	Froude Number
2.83	53.17	0.59	2.31	100.40	1.09
3.96	53.28	0.70	2.32	119.92	1.44
5.10	53.37	0.79	2.15	134.07	1.33
6.23	53.43	0.85	2.09	145.52	1.17
7.36	53.49	0.91	2.08	155.76	1.08
8.50	53.55	0.97	2.09	165.15	1.03
9.63	53.60	1.02	2.11	173.90	0.99
10.76	53.65	1.07	2.13	182.12	0.96
11.33	53.67	1.09	2.15	186.09	0.95
13.03	53.74	1.16	2.19	197.35	0.92
14.16	53.78	1.20	2.22	204.46	0.91

Table A6.1 Culvert Summary Table

Total Discharge (m³/s)	Culvert Discharge (m³/s)	Headwater Elevation (m)	Inlet Control Depth (m)	Outlet Control Depth (m)	Normal Depth (m)	Critical Depth (m)	Outlet Depth (m)	Tailwater Depth (m)	Outlet Velocity (m/s)	Tailwater Velocity (m/s)
2.83	2.83	58.02	0.818	0.0*	0.218	0.564	0.227	0.723	5.849	1.534
3.96	3.96	58.23	1.035	0.0*	0.272	0.705	0.272	0.834	6.824	1.413
5.10	5.10	58.44	1.242	0.0*	0.321	0.834	0.331	0.924	7.204	1.400
6.23	6.23	58.64	1.438	0.0*	0.367	0.954	0.386	1.003	7.564	1.415
7.36	7.36	58.83	1.628	0.0*	0.410	1.066	0.437	1.075	7.895	1.439
8.50	8.50	59.01	1.815	0.0*	0.452	1.173	0.486	1.142	8.181	1.466
9.63	9.63	59.20	2.004	0.0*	0.493	1.275	0.537	1.204	8.402	1.493
10.76	10.76	59.40	2.197	0.0*	0.532	1.373	0.586	1.263	8.607	1.521
11.33	11.33	59.50	2.296	0.0*	0.552	1.421	0.610	1.291	8.700	1.534
13.03	12.68	59.74	2.542	0.0*	0.597	1.532	0.667	1.372	8.898	1.573
14.16	12.96	59.79	2.594	0.0*	0.606	1.554	0.679	1.423	8.935	1.598

** Full Flow Headwater elevation is below inlet invert.*

Table A6.3 Summary of Culvert Flows at Crossing

Headwater Elevation (m)	Total Discharge (m^3/s)	Culvert 1 Discharge (m^3/s)	Roadway Discharge (m^3/s)	Iterations
58.02	2.83	2.83	0.00	1
58.23	3.96	3.96	0.00	1
58.44	5.10	5.10	0.00	1
58.64	6.23	6.23	0.00	1
58.83	7.36	7.36	0.00	1
59.01	8.50	8.50	0.00	1
59.20	9.63	9.63	0.00	1
59.40	10.76	10.76	0.00	1
59.50	11.33	11.33	0.00	1
59.74	13.03	12.68	0.35	6
59.79	14.16	12.96	1.19	5
59.70	12.45	12.45	0.00	Overtopping

Figure A6.1 Water Surface Profile

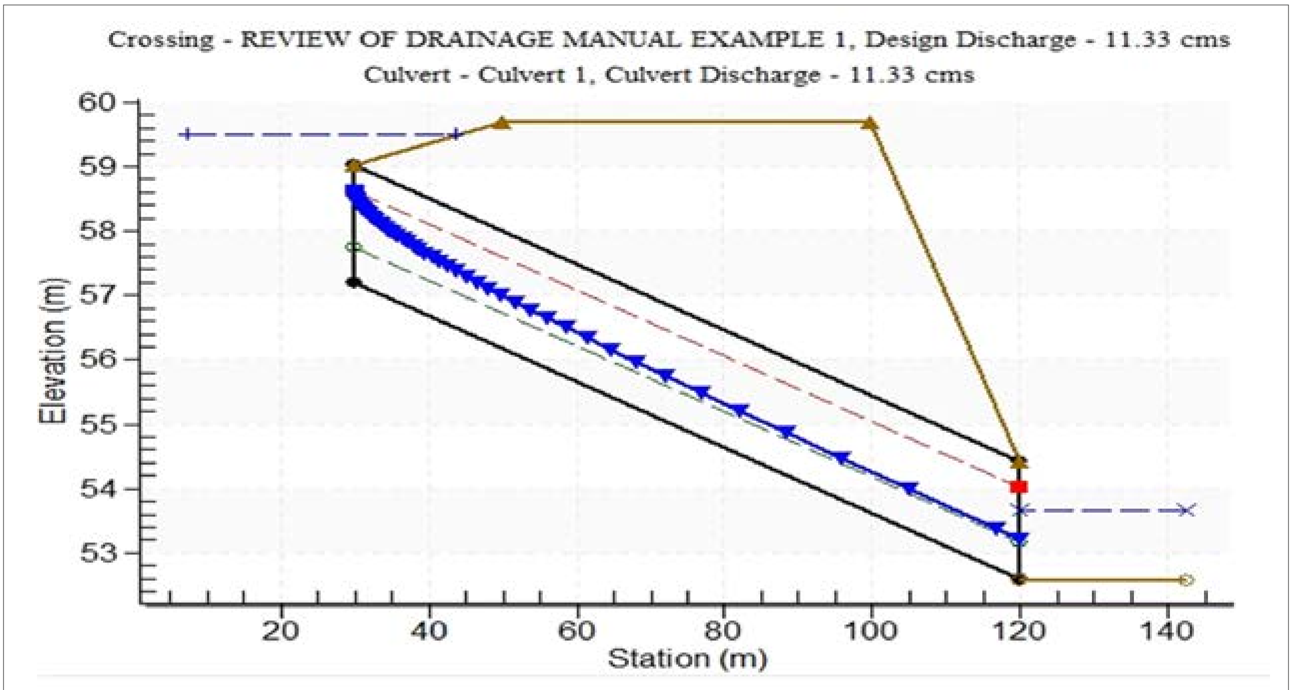


Figure A6.2 Performance Curve

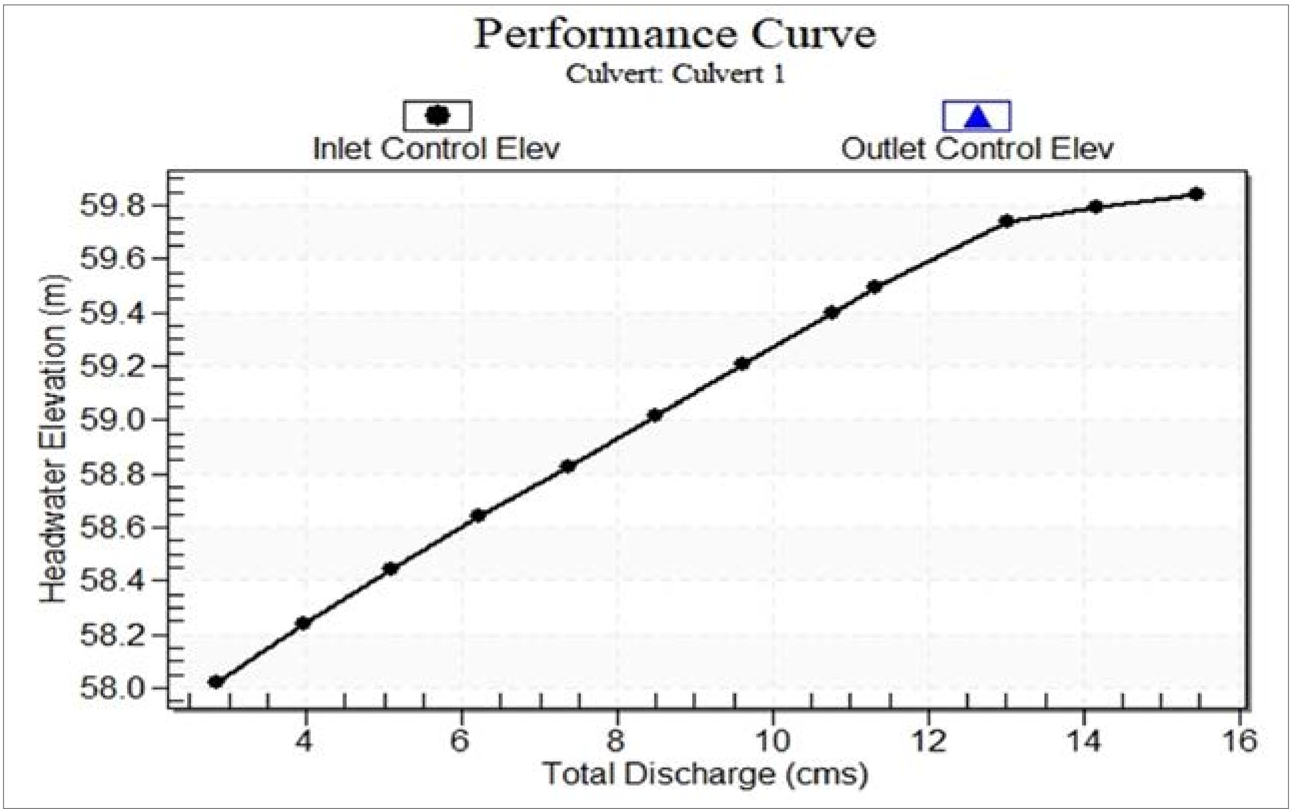
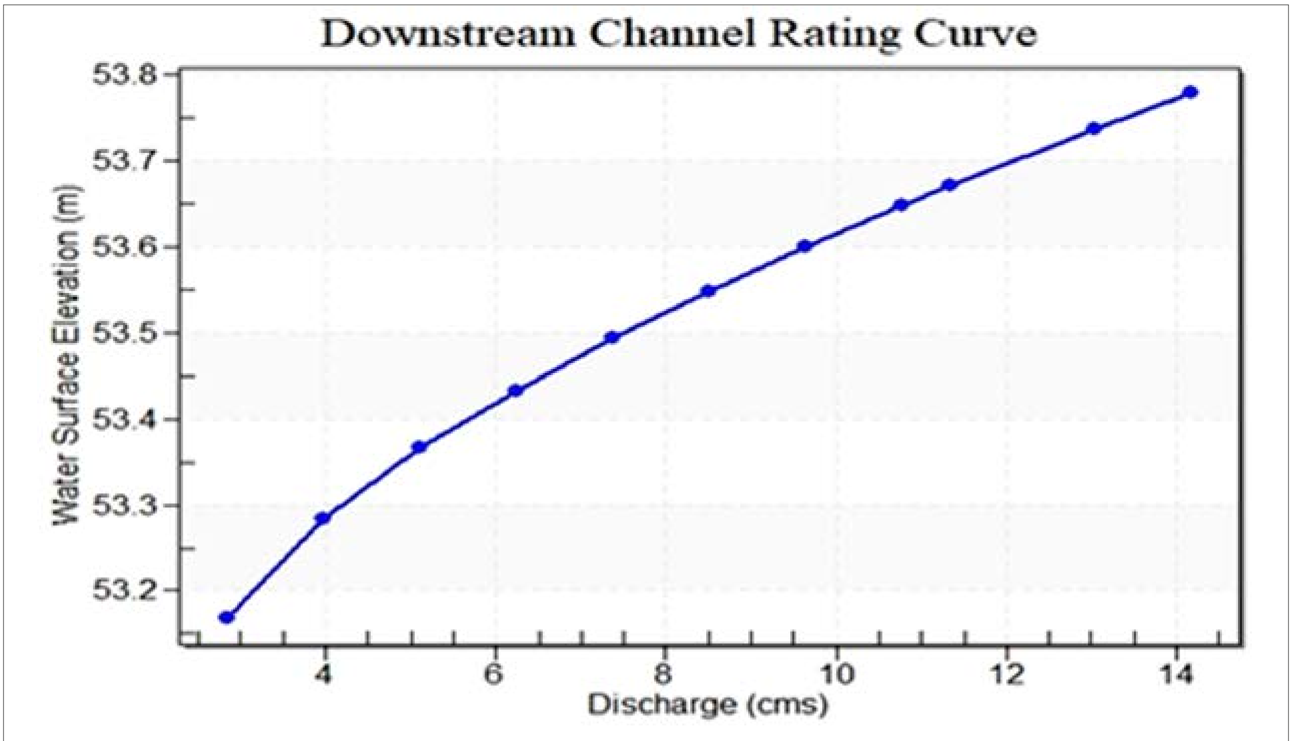


Figure A6.3 Downstream Channel Rating Curve



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A.7 Scour Hole Estimation

A 2135 x 1830 mm Reinforced Concrete Box Culvert (RCB) with a bevelled entrance on a slope of 0.05 m/m was the selected design. Calculations from Example A.5 indicate that this culvert is in inlet control and has the following:

Table A6.3 Summary of Culvert Flows at Crossing

	Q (m^3/s)	HW_i (m)	V_o (m/s)
Q_{50} =	11.33	2.32	8.61
Q_{ot} =	13.00	2.59	8.88
Q_{100} =	14.16	2.62	8.90

Size Scour Hole

The size of the scour hole is determined using Equations 4.21 to 4.27. For a channel with a gravel bed, the standard deviation of the material, σ is 2.10. Table 4.4 shows that the value of C_s = 1.00 and C_h = 1.08. (See Figure A7.1 for a summary of the computations.)

Having determined the sour hole size, the following steps guide in designing appropriate dissipater:

Step 1 Determine Need for Dissipater

The scour hole dimensions are not excessive, but since V_o = 8.67 m/s is much greater than V_d = 5.34 m/s an energy dissipater is needed.

Step 2 Select Design Alternative
Since the Fr > 3, a stilling basin shall be designed.

Step 3 Design Dissipaters
The design of stilling basins is given in HEC 14.

Step 4 Design Riprap Transition
Protection is required (See HEC 11).

Step 5 Review Results
The downstream channel conditions are matched by the dissipater.

Step 6 Documentation

i. See Chapter 4 of Part 1 of this Manual.
ii. Include computations in the culvert report or file.

Results: From the above analysis, the scour hole dimensions are:
Depth = 2.13. Width 8.84m and Length 18.39m

Figure A7.1 - Energy Dissipater Checklist

Project: Review of Road Design Manuals **Plan Sheet:** No. 5 of 45 **Date:** January 19, 2024	**Designer**: Eng. Wkn **Reviewer:** Eng. Jtk

Scour Equations

$$\frac{d_s}{R_c}, \frac{W_s}{R_c}, \frac{L}{R_c}, \frac{V_s}{R_c} = C_s\, C_h \left(\frac{\alpha}{\sigma^{1/3}}\right) \left(\frac{Q}{\sqrt{g}R_c^{2.5}}\right)^{\beta} \left(\frac{t}{316}\right)^{\theta}$$

$$d_s.\ W_s.\ L_s = [F_1]\,[F_2]\,[F_3]\,R_c$$

$$d_s,\ W_s,\ L = C_s\, C_h\, F_1\, F_2\, F_3\, R_c$$

Step 6 - Data Summary

Parameters	Culvert	Channel
Station	30 + 48	121 + 92
Control	Inlet	Super
Type	RCB	Natural
D = Height	1830 mm	2.29 m
B = Width	2135 mm	8.84 m
L = Length	91.45 m	-
Material	Concrete	Gravel
n = Manning's n	0.012	0.03 & 0.08
z : 1. Side slope		1V:1H
Q = Discharge	11.33 m³/s	11.33 m³/s
d = Depth	0.56m	0.86m
$Fr = \frac{V}{(gd)^{0.5}}$	3.54	2.01
A = Flow Area	1.30 m²	2.12 m²
Slope	0.05 m/m	0.05 m/m

Step 7a - Equation Input Data

Factor	Value
Q = Discharge. m³/s	11.33 m³/s
A_c = Culvert Area. m	3.9 m²
P_c = Culvert Perimeter. m	7.92 m
$R_c = (A_c/P_c)$	0.49
DI = Discharge Intensity	21.10
t = Time of concentration	30 min.

Step 7b - Scour Computations

Factor	Depth	Width	Length
α	2.27	6.94	17.10
β	0.39	0.53	0.47
θ	0.06	0.08	0.10
$F_1 = C_s\, C_h \frac{\alpha}{\sigma^{1/3}}$	1.92	6.94	15.62
$F_2 = [DI]^{\beta}$	3.28	5.03	4.19
$F_3 = \left(\frac{t}{316}\right)^{\theta}$	0.87	0.83	0.79
$[F_1]\,[F_2]\,[F_3]\,R_c$	2.68	14.20	25.34
Allowable	2.13	8.84	18.39

If calculated scour > allowable and:
Fr > 3. design a stilling basin
Fr < 3. design a riprap basin

A.8 To Find Bridge Backwater Caused by a Roadway Crossing:

The channel crossing is shown in Figure A8.1 with the following information:

Cross section of river at bridge site showing areas, wetted perimeters, and values of Manning n; normal water surface for design = El 8.5m at bridge; average slope of river in vicinity of bridge S_o = 0.00049; cross section under bridge showing area below normal water surface and width of roadway = 12.2m.

The stream is essentially straight, the cross section relatively constant in the vicinity of the bridge, and the crossing is normal to the general direction of flow.

Under the conditions stated, it is permissible to assume that the cross-sectional area of the stream at section 1 is the same as that at the bridge. The approach section is then divided into subsections at abrupt changes in depth or channel roughness as shown in Figure A8.1. The conveyance of each subsection is computed as shown in columns 1 through 5 of Table A8.1. The summation of the individual values in column 8 represents the overall conveyance of the stream at section 1 or K_1 = 24.889. Note that the water interface between subsections is not included in the wetted perimeter. Table A8.1 is set up in short form to better demonstrate the method. The actual computation would involve many subsections corresponding to breaks in grade or changes in channel roughness.

Since the slope of the stream is known (0.49m/km) and the cross-sectional area is essentially constant throughout the reach under consideration, it is permissible to solve for the discharge by what is known as the slope-area method or:

$$Q = K_1 S_o^{1/2} = 24.889.2(0.00049)^{1/2} = 551.2 \text{ m}^3/\text{s}$$

To compute the kinetic energy coefficient, it is first necessary to complete columns 9, 10, 11 of Table A8.1; then:

$$\alpha_1 = \frac{qv^2}{QV_1^2} = \frac{974.881}{(551/526.137)^2} = 1.61$$

Table A8.1 Calculation Summary

	Sub-Section	Manning n	A (m²)	P (m)	$R = A/P$ (m)	$R^{2/3}$	$k = (1/n)AR^{2/3}$	$q = Qk/k_1$ (m³/s)	$V = q/A$	qv^2
Q_c	0 - 60.96	0.045	58.287	61.021	0.955	0.97	1,256.40	27.80	0.477	6.325
	60.96 - 73.15	0.070	26.496	12.222	2.168	1.675	634.00	14.00	0.528	3.903
Q_b	73.152 - 85.344	0.070	30.147	12.222	2.467	1.826	786.40	17.40	0.577	5.793
	85.344- 128.016	0.035	186.170	44.196	4.213	2.609	13,878.20	307.20	1.650	836.350
	128.016 -135.636	0.050	19.119	7.65	2.499	1.842	704.20	15.60	0.816	6.916
Q_a	135.636 - 152.4	0.050	50.112	16.764	2.989	2.075	2.079.60	46.00	0.918	38.765
	152.4 - 228.6	0.045	155.800	76.505	2.036	1.606	5.560.30	123.10	0.790	76.827
			A_n = 526.13				k_1 = 24.899.20	Q = 551.1		Σqv^2 = 974.88
	S_o = 0.00049									
			A_n^2 = 235.44					Q_b = 340.2		

The sum of the individual discharges in column 9 must equal 551 m³/s. The factor M is the ratio of that portion of the discharge approaching the bridge in width b, to the total discharge of the river:

$M = Q \;\; M = Q_b/Q$ = 340.2/551.2 = 0.62

Entering Figure A8.1 with α_1 = 1.61 and M = 0.62. the value of α_2 is estimated as : 1.40.

Entering Figure A8.2 with M = 0.62, the base curve coefficient is K_b = 0.72 for a bridge waterway of 62.5 m.

Figure A8.1 (Fig. 5 HDS – 1) Estimation of α_2

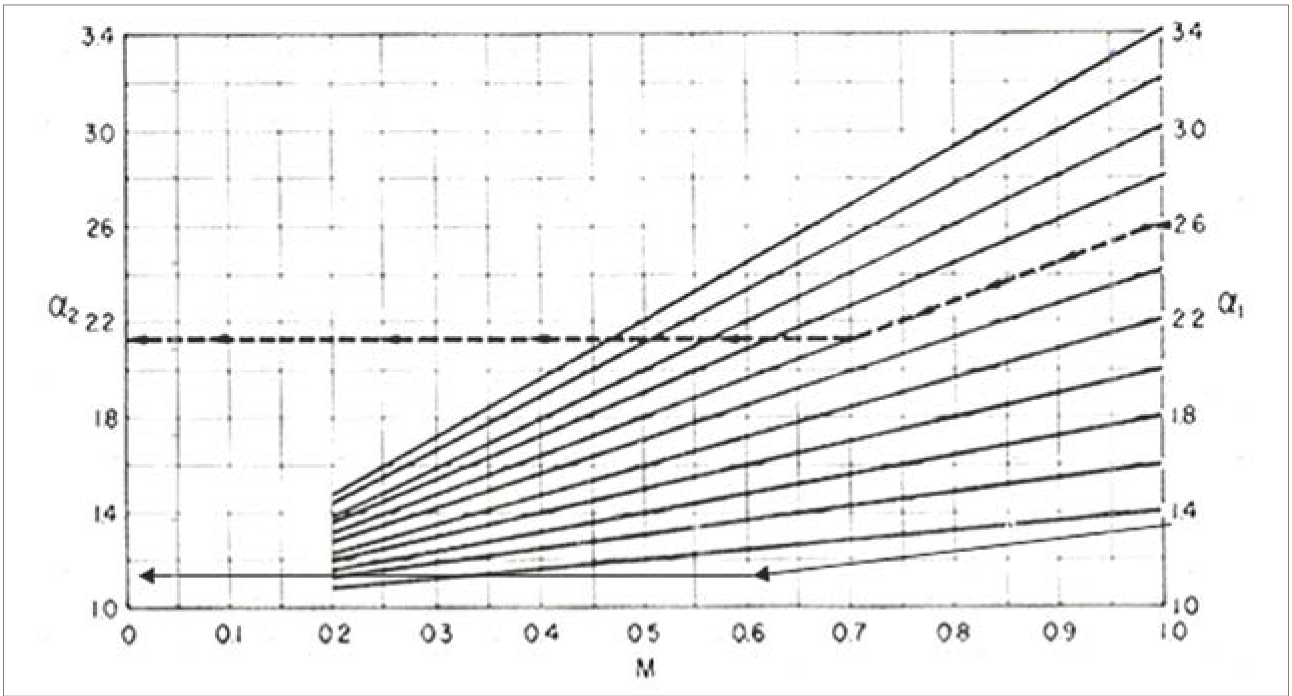
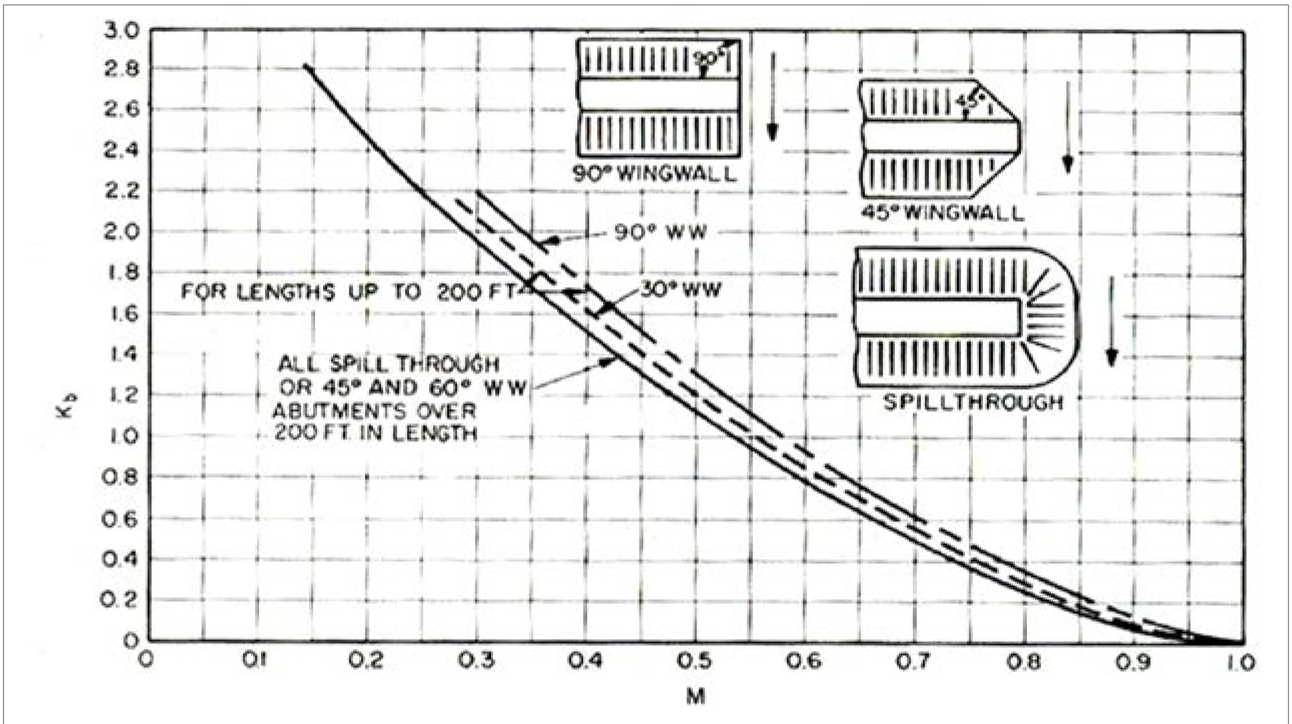


Figure A8.2 (Fig. 6 HDS – 1) Backwater Coefficient Curve



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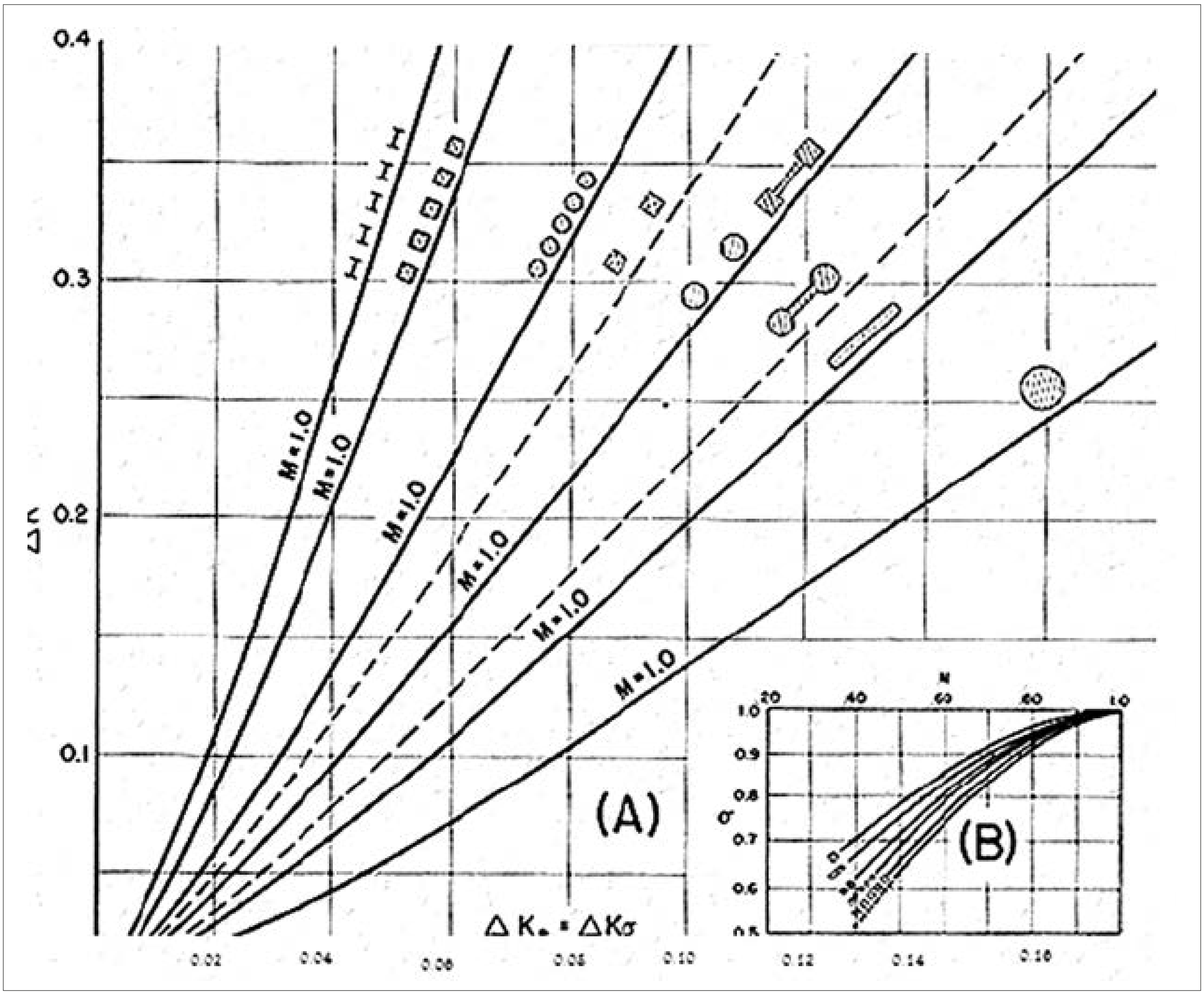
As the bridge is supported by five solid piers, the incremental coefficient (ΔK_p) for this effect is determined. Referring to Figure A8.3 and Table A8.1: the gross water area under the bridge for normal stage, A_n^2, is 235.44 m² and the area obstructed by the piers, A_p, is 16.723 m²; so:

$$J = \frac{Ap}{A_n^2} = 16.723/235.44 = 0.071$$

Entering Figure 8A.4 with J = 0.071 for solid piers, the reading from the ordinate is ΔK = 0.13. This value is for M = 1.0. Now enter A8.4 and obtain the correction factor σ, for M = 0.62 which is 0.84. The incremental backwater coefficient for the five piers, $\Delta K_p = \Delta K\sigma$ = 0.13 x 0.84 = 0.11

Figure A8.4 (Fig. 7a and b of HDS – 1)



The overall backwater coefficient:

$$K^* = K_b + \Delta K_p = 0.72 + 0.11 = 0.83$$

$$V_{n2} = \frac{Q}{A_{n2}} = \frac{551.2}{235.44} = 2.34 \text{ m/s}$$

$$\text{and } \frac{V_{n2}^2}{2g} = \frac{2.342}{2}9.8 = 0.279 \text{ m}$$

The approximate backwater will be:

$$\frac{K^*V^2}{2g} = 0.83 \times 1.40 \times 0.279 = 0.325 \text{ m}$$

Substituting values in the second half of expression for difference in kinetic energy between sections 4 and 1 where A_{n1} = 526.137 m² = A_4.

$$A_1 = 593.1 \text{ m}^2 \text{ and } A_{n2} = 235.44 \text{ m}^2$$

$$\alpha_1 \left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right] \frac{V_{n2}^2}{2g}$$

$$1.61\left(\frac{235.4}{526.2}\right)^2 - \left(\frac{235.4}{593.1}\right)^2] (.279) = (1.61)(.042)(.279) = 0.0189 \text{ m}$$

Then total backwater produced is: $h_1^* = 0.325 + 0.0189 = 0.344$ m

A.9 Hydraulic Assessment of Darathe River Bridge Using HEC RAS

The North Horr – JN Darathe AP Camp (A4) Road in Marsabit County is due for upgrading to bituminous standard. The road catchment is drained by seasonal rivers/streams at various sections, with existing structures including one single span bridge at km 99 + 070, one box culvert 5 m x 2.5 m high at Km 31 + 500, six drifts and several pipe culverts. The Darathe bridge is single span concrete bridge with a span of 15 metres and average height of 4.0 m.

Problem: To carry out hydrological and hydraulic assessment to check whether the existing bridge is safe:

Solution:

A detailed site investigation and analysis were carried out and the following results obtained.

1. Hydrological Data:
   a. Catchment area = 92Km^2
   b. Catchment slope = 3.25%
   c. Channel length = 20 Kms
   d. Average channel slope = 1.25%

From TRL Flood analysis
   a. 25 year flood = 46.8 m^3/s
   b. 50 year flood = 54.02 m^3/s
   c. 100 year flood = 62.33 m^3/s

2. HEC RAS Hydraulic analysis:
   a. The design finished road level at the bridge centreline is 750.00 m
   b. The lowest water level (LWL) at centre-line of the bridge is 745.2 m
   c. The overall depth of bridge deck + beam + upstand beam = 1.2 m
   d. Lowest chord level = 748.8 m; Allowing 1.5 m freeboard, design flood level (DFL) = 750 - (1.2 + 1.5) = 747.3 m.
   e. Several cross-sections were taken, 1,050 metres upstream and 350m downstream of the bridge.

After modelling, a steady flow analysis was carried out and errors corrected. Typical HEC RAS output results are shown below:

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Figure A9.1 Typical Cross-Section Data Editor

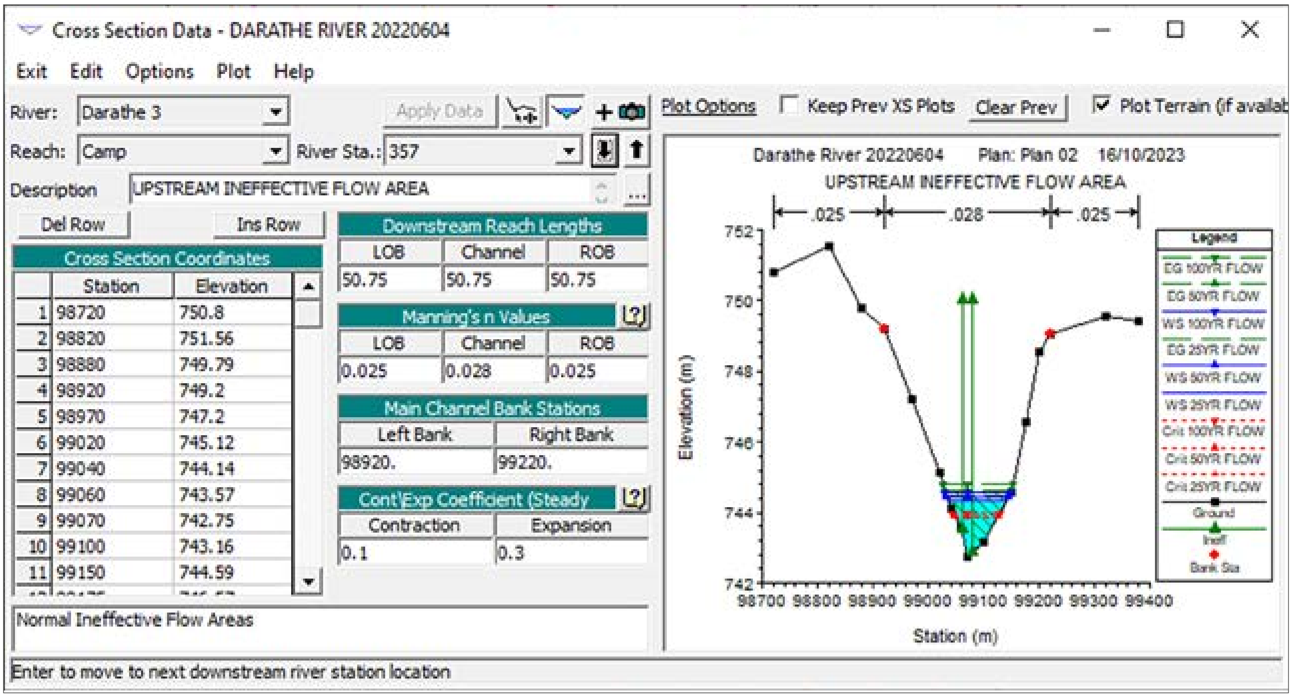

Figure A9.2 Water Surface Profiles For Q = 25Yrs, 50Yrs and 100Yrs

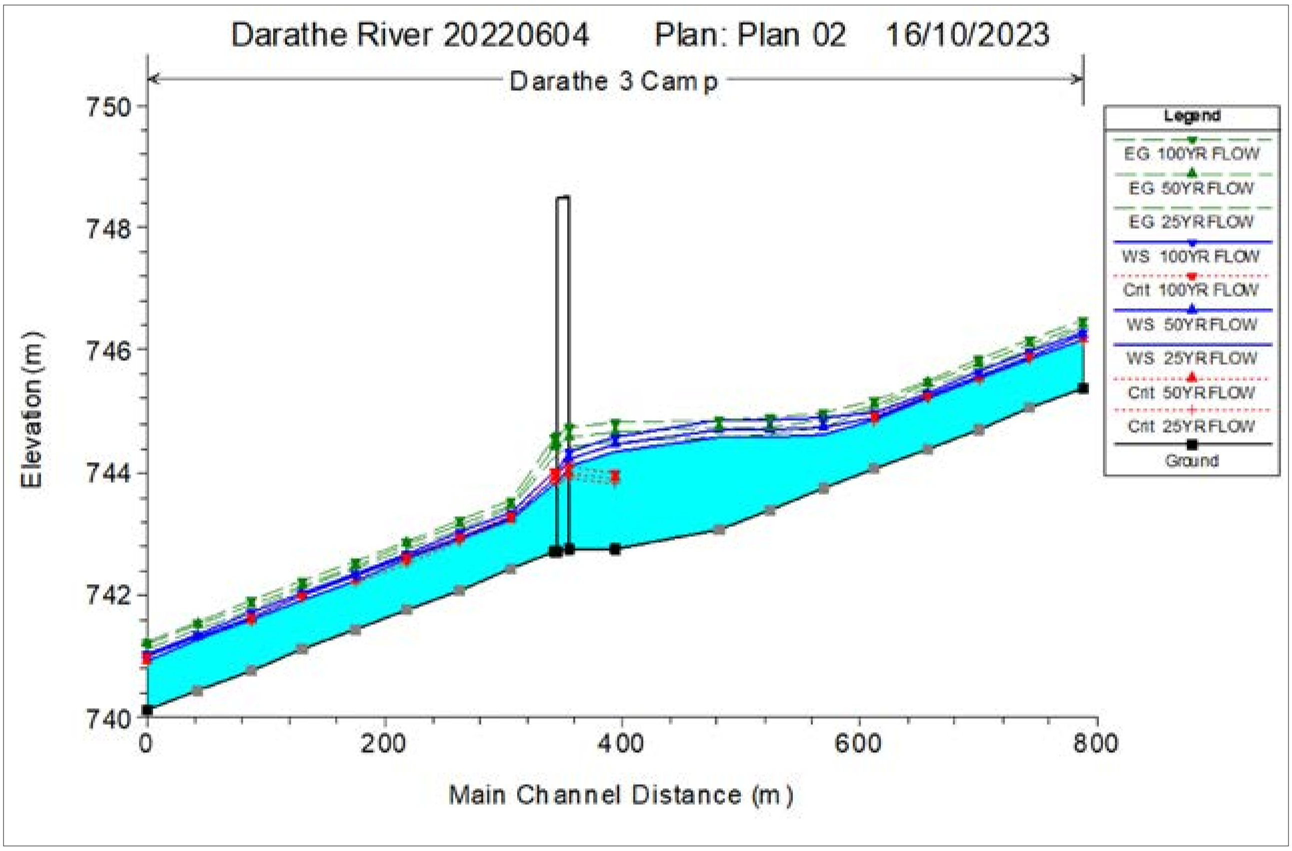

Figure A9.3 Bridge Data at Centreline

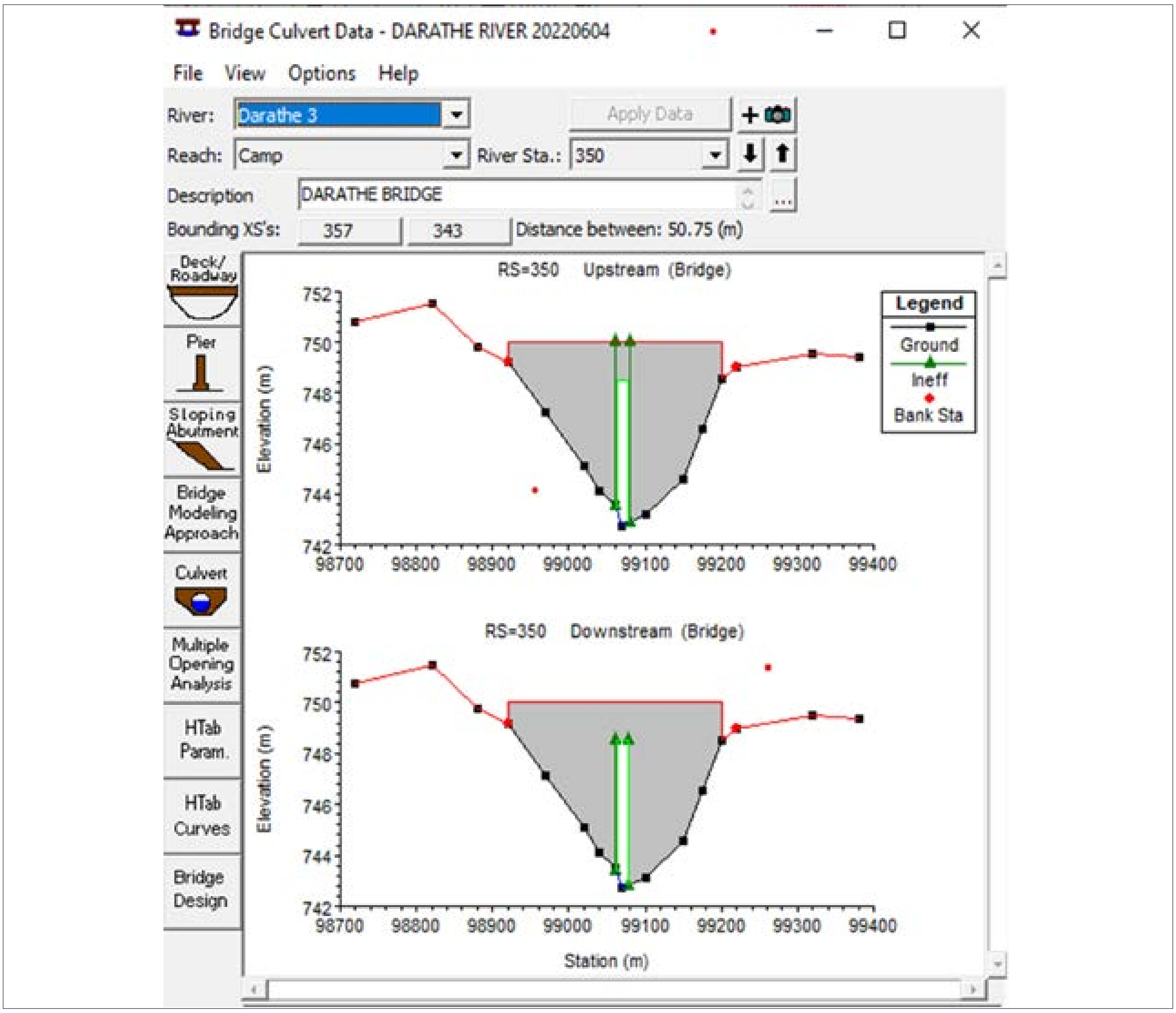

Figure A9.4 Bridge Data at Centreline

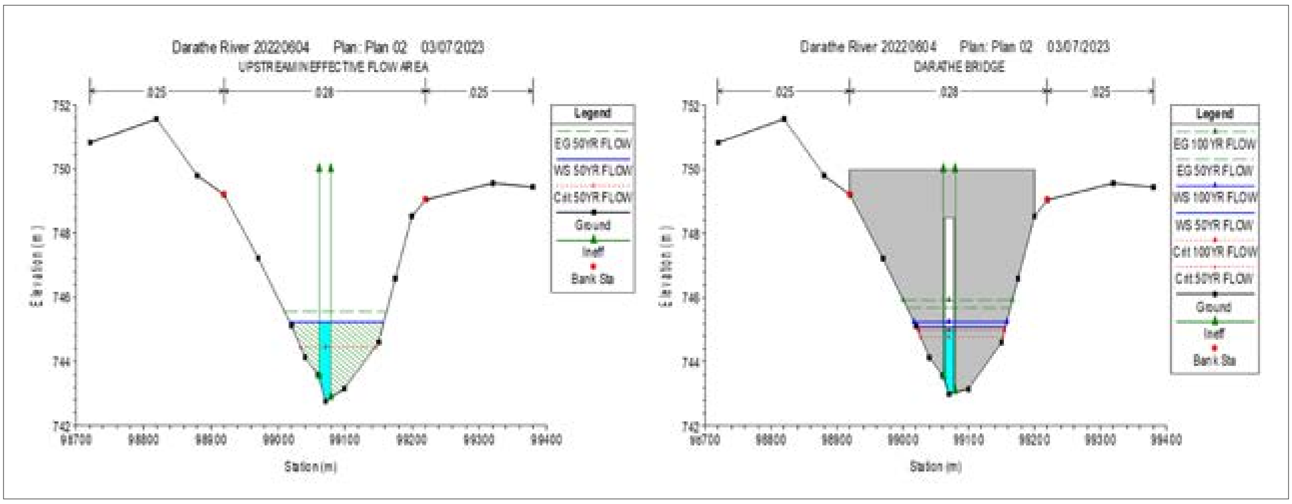

Figure A9.5 Downstream Ineffective Flow Areas

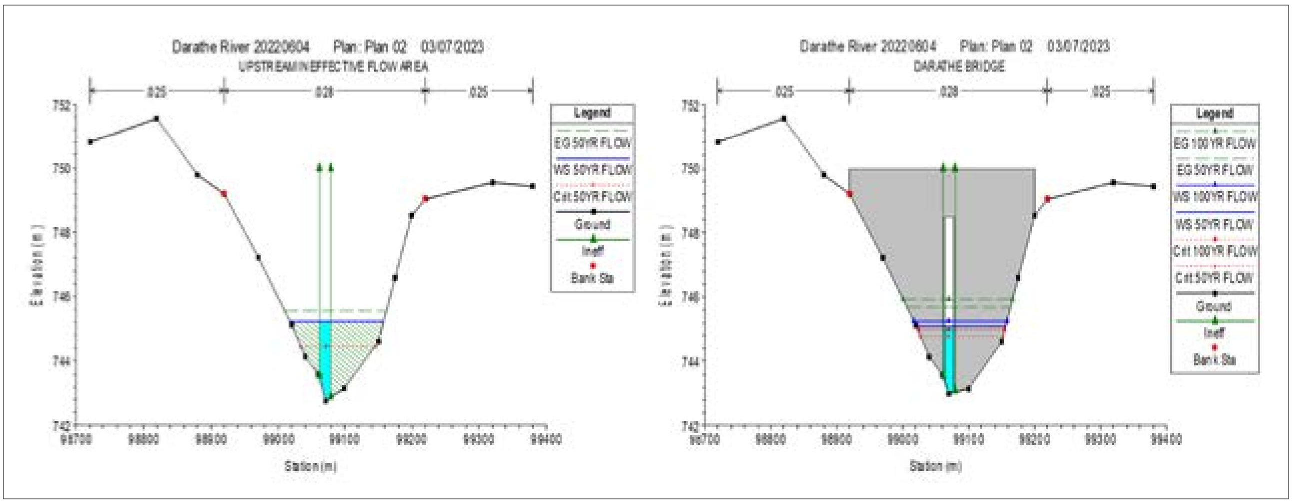

Figure A9.6 Rating Curve At 350 Br U/S

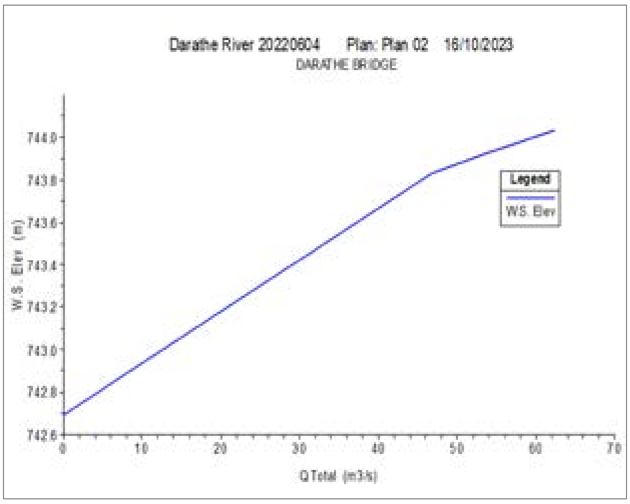

Figure A9.7 General Profile Plot - Velocity

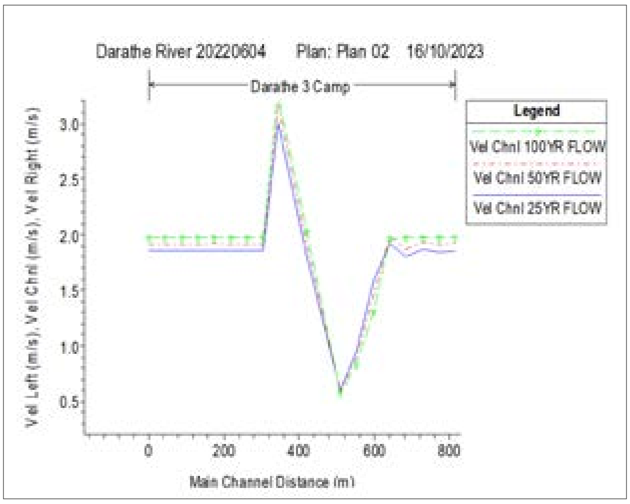

Table A9.1 Results of Analysis Using HEC - RAS Model: Detailed Summary Tables

Cross Section Output

File Type Options Help

River: Darathe 3 Profile: 25YR FLOW

Reach Camp RS: 350 BR U Plan: Plan 02

Plan: Plan 02 Darathe 3 Camp RS: 350 BR U Profile: 25YR FLOW

		Element	Left OB	Channel	Right OB
E.G. Elev (m)	744.42	Element	Left OB	Channel	Right OB
Vel Head (m)	0.31	Wt. n-Val.		0.028	
W.S. Elev (m)	744.11	Reach Len. (m)	10.00	10.00	10.00
Crit W.S. (m)	743.89	Flow Area (m2)		18.94	
E.G. Slope (m/m)	0.004577	Area (m2)		18.94	
Q Total (m3/s)	46.75	Flow (m3/s)		46.75	
Top Width (m)	16.12	Top Width (m)		16.12	
Vel Total (m/s)	2.47	Avg. Vel. (m/s)		2.47	
Max Chl Dpth (m)	1.36	Hydr. Depth (m)		1.18	
Conv. Total (m3/s)	691.1	Conv. (m3/s)		691.1	
Length Wtd. (m)	10.00	Wetted Per. (m)		18.35	
Min Ch El (m)	742.75	Shear (N/m2)		46.33	
Alpha	1.00	Stream Power (N/m s)		114.32	
Frctn Loss (m)		Cum Volume (1000 m3)		9.22	
C & E Loss (m)		Cum SA (1000 m2)		19.18	

Cross Section Output

File Type Options Help

River: Darathe 3 Profile: 50YR FLOW

Reach Camp RS: 350 BR U Plan: Plan 02

Plan: Plan 02 Darathe 3 Camp RS: 350 BR U Profile: 50YR FLOW

E.G. Elev (m)	744.57	Element	Left OB	Channel	Right OB
Vel Head (m)	0.35	Wt. n-Val.		0.028	
W.S. Elev (m)	744.22	Reach Len. (m)	10.00	10.00	10.00
Crit W.S. (m)	743.99	Flow Area (m2)		20.64	
E.G. Slope (m/m)	0.004651	Area (m2)		20.64	
Q Total (m3/s)	54.02	Flow (m3/s)		54.02	
Top Width (m)	16.10	Top Width (m)		16.10	
Vel Total (m/s)	2.62	Avg. Vel. (m/s)		2.62	
Max Chl Dpth (m)	1.47	Hydr. Depth (m)		1.28	
Conv. Total (m3/s)	792.1	Conv. (m3/s)		792.1	
Length Wtd. (m)	10.00	Wetted Per. (m)		18.53	
Min Ch El (m)	742.75	Shear (N/m2)		50.80	
Alpha	1.00	Stream Power (N/m s)		132.96	
Frctn Loss (m)		Cum Volume (1000 m3)		10.36	
C & E Loss (m)		Cum SA (1000 m2)		20.49	

Cross Section Output

File Type Options Help

River: Darathe 3 Profile: 100YR FLOW

Reach Camp RS: 350 BR U Plan: Plan 02

Plan: Plan 02 Darathe 3 Camp RS: 350 BR U Profile: 100YR FLOW

E.G. Elev (m)	744.72	Element	Left OB	Channel	Right OB
Vel Head (m)	0.39	Wt. n-Val.		0.028	
W.S. Elev (m)	744.33	Reach Len. (m)	10.00	10.00	10.00
Crit W.S. (m)	744.09	Flow Area (m2)		22.50	
E.G. Slope (m/m)	0.004727	Area (m2)		22.50	
Q Total (m3/s)	62.33	Flow (m3/s)		62.33	
Top Width (m)	16.07	Top Width (m)		16.07	
Vel Total (m/s)	2.77	Avg. Vel. (m/s)		2.77	
Max Chl Dpth (m)	1.58	Hydr. Depth (m)		1.40	
Conv. Total (m3/s)	906.6	Conv. (m3/s)		906.6	
Length Wtd. (m)	10.00	Wetted Per. (m)		18.77	
Min Ch El (m)	742.75	Shear (N/m2)		55.57	
Alpha	1.00	Stream Power (N/m s)		153.96	
Frctn Loss (m)		Cum Volume (1000 m3)		11.64	
C & E Loss (m)		Cum SA (1000 m2)		21.91	

Table A9.2 Profile Output Tables (Standard Table 1)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Camp	700	25YR FLOW	46.75	745.37	746.18	746.13	746.36	0.007555	1.86	25.11	54.00	0.87
Camp	700	50YR FLOW	54.02	745.37	746.24	746.18	746.43	0.007534	1.92	28.13	57.65	0.88
Camp	700	100YR FLOW	62.33	745.37	746.29		746.49	0.007476	1.97	31.56	61.67	0.88
Camp	656.25*	25YR FLOW	46.75	745.04	745.86		746.03	0.007338	1.84	25.37	54.23	0.86
Camp	656.25*	50YR FLOW	54.02	745.04	745.91		746.10	0.007400	1.91	28.33	57.89	0.87
Camp	656.25*	100YR FLOW	62.33	745.04	745.97	745.91	746.17	0.007524	1.98	31.49	61.60	0.88
Camp	612.50*	25YR FLOW	46.75	744.71	745.52	745.48	745.70	0.007715	1.88	24.92	53.84	0.88
Camp	612.50*	50YR FLOW	54.02	744.71	745.58	745.53	745.77	0.007645	1.93	27.97	57.45	0.88
Camp	612.50*	100YR FLOW	62.33	744.71	745.64		745.84	0.007458	1.97	31.59	61.71	0.88
Camp	568.75*	25YR FLOW	46.75	744.39	745.21		745.38	0.006976	1.81	25.87	54.82	0.84
Camp	568.75*	50YR FLOW	54.02	744.39	745.27		745.44	0.007006	1.87	28.94	58.62	0.85
Camp	568.75*	100YR FLOW	62.33	744.39	745.31	745.25	745.51	0.007564	1.98	31.42	61.50	0.89
Camp	525.00*	25YR FLOW	46.75	744.06	744.85	744.82	745.04	0.008336	1.93	24.23	53.22	0.91
Camp	525.00*	50YR FLOW	54.02	744.06	744.91	744.87	745.11	0.008295	2.00	27.08	56.32	0.92
Camp	525.00*	100YR FLOW	62.33	744.06	744.99	744.93	745.18	0.007328	1.96	31.81	61.93	0.87
Camp	481.25*	25YR FLOW	46.75	743.73	744.62		744.75	0.004988	1.58	29.52	59.31	0.72
Camp	481.25*	50YR FLOW	54.02	743.73	744.74		744.85	0.003645	1.45	37.35	67.93	0.62
Camp	481.25*	100YR FLOW	62.33	743.73	744.88		744.97	0.002588	1.31	47.61	77.80	0.53
Camp	437.50*	25YR FLOW	46.75	743.40	744.58		744.62	0.001342	0.95	49.13	79.15	0.39
Camp	437.50*	50YR FLOW	54.02	743.40	744.71		744.75	0.001036	0.89	60.62	88.74	0.34
Camp	437.50*	100YR FLOW	62.33	743.40	744.86		744.90	0.000785	0.83	74.78	98.26	0.31
Camp	393.75*	25YR FLOW	46.75	743.07	744.57		744.58	0.000396	0.60	77.80	99.96	0.22
Camp	393.75*	50YR FLOW	54.02	743.07	744.70		744.72	0.000331	0.59	92.21	107.64	0.20
Camp	393.75*	100YR FLOW	62.33	743.07	744.86		744.87	0.000277	0.57	109.30	116.11	0.19
Camp	357	25YR FLOW	46.75	742.75	744.34	743.83	744.51	0.001690	1.80	25.93	105.38	0.49
Camp	357	50YR FLOW	54.02	742.75	744.46	743.91	744.65	0.001695	1.91	28.25	112.15	0.50

Total flow in cross section.

Table A9.3 Profile Output Tables (Standard Table 1) Continued....

HEC-RAS Plan: Plan 02 River: Darathe 3 Reach: Camp

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Camp	350		Bridge									
Camp	343	25YR FLOW	46.75	742.69	743.81	743.81	744.27	0.008032	3.01	15.53	75.13	1.01
Camp	343	50YR FLOW	54.02	742.69	743.90	743.90	744.41	0.007797	3.16	17.09	81.55	1.01
Camp	343	100YR FLOW	62.33	742.69	744.01	744.01	744.56	0.007442	3.30	18.88	88.95	1.00
Camp	306.25*	25YR FLOW	46.75	742.42	743.23		743.41	0.007485	1.86	25.19	54.07	0.87
Camp	306.25*	50YR FLOW	54.02	742.42	743.29	743.23	743.47	0.007491	1.92	28.19	57.70	0.88
Camp	306.25*	100YR FLOW	62.33	742.42	743.34	743.29	743.54	0.007490	1.98	31.54	61.64	0.88
Camp	262.50*	25YR FLOW	46.75	742.09	742.90	742.85	743.08	0.007504	1.86	25.17	54.05	0.87
Camp	262.50*	50YR FLOW	54.02	742.09	742.96	742.90	743.14	0.007498	1.92	28.18	57.71	0.88
Camp	262.50*	100YR FLOW	62.33	742.09	743.01	742.96	743.21	0.007497	1.98	31.53	61.65	0.88
Camp	218.75*	25YR FLOW	46.75	741.76	742.58	742.52	742.75	0.007498	1.86	25.18	54.06	0.87
Camp	218.75*	50YR FLOW	54.02	741.76	742.63	742.57	742.82	0.007502	1.92	28.17	57.69	0.88
Camp	218.75*	100YR FLOW	62.33	741.76	742.69	742.63	742.88	0.007505	1.98	31.52	61.63	0.88
Camp	175.00*	25YR FLOW	46.75	741.43	742.25	742.20	742.42	0.007513	1.86	25.15	54.03	0.87
Camp	175.00*	50YR FLOW	54.02	741.43	742.30	742.25	742.49	0.007510	1.92	28.16	57.65	0.88
Camp	175.00*	100YR FLOW	62.33	741.43	742.36		742.56	0.007506	1.98	31.51	61.59	0.88
Camp	131.25*	25YR FLOW	46.75	741.10	741.92		742.09	0.007501	1.86	25.17	54.06	0.87
Camp	131.25*	50YR FLOW	54.02	741.10	741.97		742.16	0.007501	1.92	28.18	57.71	0.88
Camp	131.25*	100YR FLOW	62.33	741.10	742.03	741.97	742.23	0.007505	1.98	31.52	61.64	0.88
Camp	87.50*	25YR FLOW	46.75	740.78	741.59	741.54	741.77	0.007505	1.86	25.16	54.04	0.87
Camp	87.50*	50YR FLOW	54.02	740.78	741.64	741.59	741.83	0.007506	1.92	28.17	57.68	0.88
Camp	87.50*	100YR FLOW	62.33	740.78	741.70	741.64	741.90	0.007504	1.98	31.51	61.61	0.88
Camp	43.75*	25YR FLOW	46.75	740.45	741.26		741.44	0.007485	1.86	25.19	54.07	0.87
Camp	43.75*	50YR FLOW	54.02	740.45	741.32		741.50	0.007482	1.92	28.20	57.72	0.88
Camp	43.75*	100YR FLOW	62.33	740.45	741.37		741.57	0.007480	1.98	31.56	61.66	0.88

Total flow in cross section.

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Results

From the above analysis, it observed that:

For the three flow profiles, 25 year, 50 year and 100 year, the bridge will pass the floods safely (water surface elevations 744.11 m and 744.22 m and 744.33 m all below the bridge design flood level (DFL) - that's lower chord level less freeboard (747.3 m).

Except for 100 year flow, the velocities are within limits, (2.7 to 3.5 m/s). However, there is need to carry out scour assessment.

In the two flow profiles, Froude number is less than 1 – hence the flow is subcritical.

A.10 To find the Magnitude of Live-bed Contraction Scour Depth

Given:

1. The upstream channel width = 98.2 m; depth = 2.62 m
2. The discharge is 773 m³/s and is all contained within the channel. Channel slope = 0.004 m/m
3. The bridge abutments consist of vertical walls with wing walls, width = 37.2 m; with 3 sets of piers consisting of 3 columns 0.38 m in diameter
4. The bed material size: from 0 to 0.9 m depth below the bed the D_{50} is 0.31 mm and below 0.9 m depth below the bed the D_{50} is 0.70 mm with a fall velocity of 0.10 m/s
5. Original depth at bridge is estimated as 2.16 m.

Determine:

The magnitude of the contraction scour depth.

Solution:

1. Determine if it is live-bed or clear-water scour.

   Average velocity in the upstream reach

   V = 773/(2.62 x 98.2) = 3.0 m/s

   For velocities this large and bed material this fine **live-bed** scour will occur. Check by calculating V_c for 0.7 mm bed material size. If live-bed scour occurs for 0.7 mm it would also be live-bed for 0.3 mm.

   $V_c = K_u\, y^{1/6}\, D^{1/3}$

   $V_c = 6.19\,(2.62)^{1/6}\,(0.7)^{1/3} = 0.65$ m/s

   Live-bed contraction scour is verified

2. Calculate contraction scour. Determine K_1 for mode of bed material

   $V^* = (g y_1\, S_1)^{½}$

   $V^* = (9.81 \times 2.62 \times 0.004)^{0.5} = 0.32$ m/s

   $T = 0.1;\quad \dfrac{V^*}{T} = 3.2;\quad K_1 = 0.69$

   Live-bed contraction scour from Equation 5.10

   $$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6.7} \left(\frac{W_1}{W_2}\right)^{K1}$$

   $Q_1 = Q_2$

   $$\frac{y_2}{2.62} = \left(\frac{98.2}{36.06}\right)^{0.69}$$

   y_2 = 2.62 x 2.00 = 5.24 m from water surface

   y_s = 5.24 - 2.16 = 3.08 m from original bed surface

A.11 To Find The Magnitude of Scour Depth of Solid Pier

Given:

1. Solid pier geometry: a = 1.22 m, L = 18 m, round nose
2. Flow variables: y_1 = 3.12 m, V_1 = 3.36 m/s
3. Angle of attack = 0 degrees, g = 9.81 m/s^2
4. Froude No. = 3.36/(9.81 x 3.12)0.5 = 0.61
5. Bed material: D_{50} = 7.3 mm, D_{95} 0.32 mm
6. Bed Configuration: Plane bed

Determine:

The magnitude of pier scour depth.

Solution:

Using the HEC-18 equation (Equation 5.17)

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{043}$$

$$\frac{y_s}{3.12} = 2.0 \times 1.0 \times 1.0 \times 1.1 \times \left(\frac{122}{3.12}\right)^{0.65} \times 0.61^{0.43} = 0.97$$

$$y_s = 0.97 \times 3.12 = 3 \text{ m}$$

Check:

$y_{s\ max}$ = 2.4a

$y_{s\ max}$ = 2.4 x 1.22 = 2.928 m, therefore OK.

A.12 Example Problem A.11 But With Angle of Attack

Given:

Same as Problem 1 but angle of attack is 20 degrees

Solution:

Use Equation 5.20 to compute K_2

$$K_2 = (Cos\ \theta + L\ /\ a\ Sin\ \theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 5.20 (see Table 5.7).

$$L/a = 18/1.22 = 14.8 > 12 \text{ use } 12$$

$$K_2 = (Cos\ 20 + 12\ Sin\ 20)^{0.65} = 2.86$$

$$y_s = 3.01 \text{ X } 2.86 = 8.6 \text{ m}$$

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A.13 Design of Riprap at Channel Bottom and Depth of Flow

Problem:

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.

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.

Solution:

The solution follows the procedure outlined in HEC-15, which is based on the shear stress method.

1. Choose a rounded gravel with D_{50} = 25 mm, then τp = 19 Pa (Table 2.5)
2. Estimate n = 0.033 from Table 2.6 for depth y = 0.15 – 0.6 m
3. Calculate y from Manning's equation:

$$\frac{1.486nQ}{(b^{8/3})(S^{1/2})} = \frac{1.486(0.033)(0.6)}{(1.2^{8/3})(0.005^{1/2})} = 0.256$$

4. Then with Z = 3: $\frac{y}{b}$ = 0.29 and y = 0.35 m
5. Calculate maximum bed shear stress. τd
6. τd = 9800 yS = 9800 x 0.35 x 0.005 = 17 Pa
7. Now because $\tau d < \tau p$. accept D_{50} of approximately 25mm. Otherwise repeat with another riprap diameter.
8. 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.

A.14 Storm Drainage Network Design.

The Rational Method is the traditionally used method for storm drain network design. A peak discharge is calculated for each design point in a storm drainage network.

A comparison of the results of the Summation of Flow Method and Rational Method are shown on the network below:

Given: A hypothetical storm drain network as shown in Figure A14.1:

Figure A14.1 Hypothetical Storm Drain System

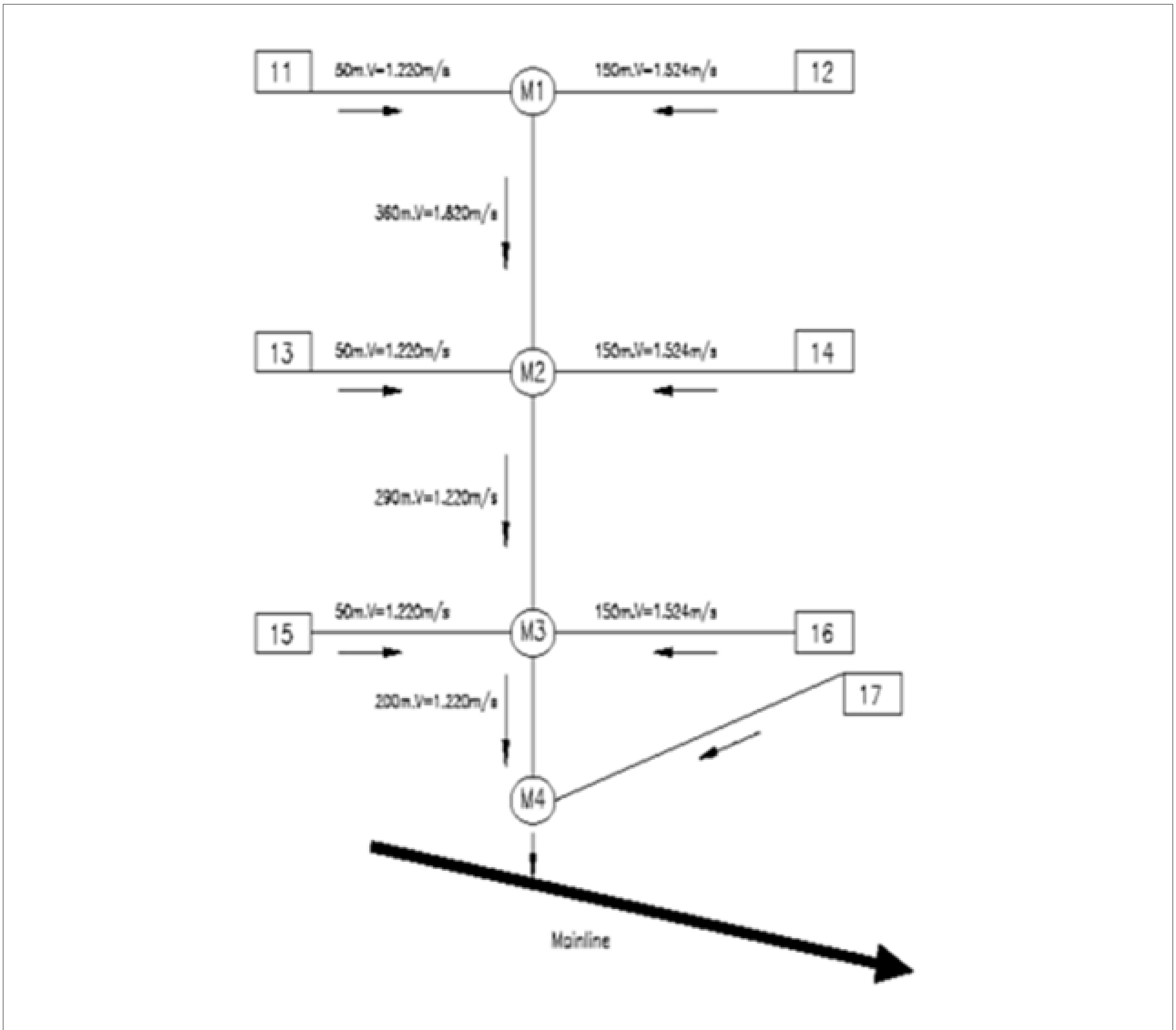

Table A14.1 Data for demonstrating Storm Drain Hydraulic Procedure

Inlet[a]	Drainage Area (ha)	Time of Concentration (min)	Rainfall Intensity (mm/hr)	Runoff Coefficient	Inlet Flow[c] Rate (m^3/s)
1	0.81	10	72	0.9	0.146
2	1.21	15	58	0.9	0.167
3	1.01	10	72	0.9	0.182
4	1.01	10	72	0.9	0.182
5	0.81	12	69	0.9	0.14
6	1.01	15	58	0.9	0.139
7	0.81	10	72	0.9	0.146

a Inlet and storm drain system as shown in Figure A14.1
b Obtained from IDF curves
c Calculated using Rational Formula

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Table A14.2 Results for Summary of Flow Calculation Method

Storm Drain Segment	Tributary Inlets	Summation of Inlet Flow (m3/s)
$I_1 - M_1$	1	0.146
$I_2 - M_1$	2	0.167
$M_1 - M_2$	1 and 2	0.313
$I_3 - M_2$	3	0.182
$I_4 - M_2$	4	0.182
$M_2 - M_3$	1,2,3 and 4	0.677
$I_5 - M_3$	5	0.140
$I_6 - M_4$	6	0.139
$M_3 - M_4$	1,2,3,4,5 and 6	0.956
$I_7 - M_1$	7	0.146
$M_4 - O$	1,2,3,4,5,6 and 7	1.102

100 % inlet intercept has been assumed for data

Table A14.3 Results of Rational Method Calculation for Storm Drain System

Storm Drain Segment	Tributary Area (ha)	Time of Concentration (min)	Rainfall Intensity (mm/h)	Runoff Coefficient	Design Flow Rate (m^3/s)
$I_1 - M_1$	0.81	10	72	0.9	0.146
$I_2 - M_1$	1.21	15	58	0.9	0.167
$M_1 - M_2$	2.02	16.6	55	0.9	0.278
$I_3 - M_2$	1.01	10	72	0.9	0.182
$I_4 - M_2$	1.01	10	72	0.9	0.182
$M_2 - M_3$	4.04	19.9	48	0.9	0.485
$I_5 - M_3$	0.81	12	69	0.9	0.14
$I_6 - M_4$	1.01	15	58	0.9	0.139
$M_3 - M_4$	5.86	26.4	39	0.9	0.572
$I_7 - M_1$	0.81	10	72	0.9	0.146
$M_4 - O$	6.67	30.4	35	0.9	0.584

A.15 Storm Drainage Network Design Using Modified Rational Method

A network drain is to be developed to serve a high-density housing estate close to Nairobi, Kenya.

The catchment has the following characteristics:

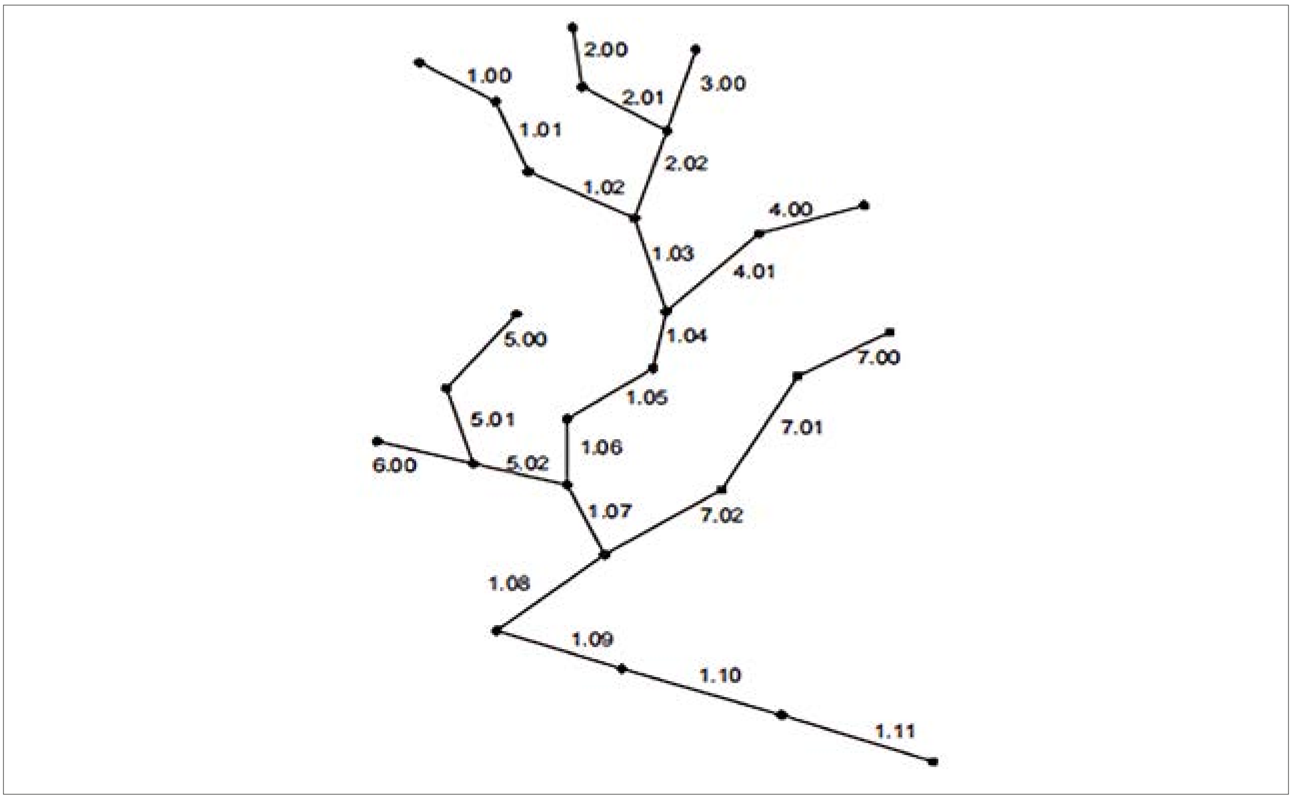
1. Soil is heavy clay and average ground slope is 2%.
2. Unpaved run-off coefficient = 0.75 and overall run-off coefficient for the whole catchment is 0.87.
3. For modified Rational Method, time of entry has been determined to be 2 min.
4. Peak flow factor is 0.5 and unpaved effective run-off factor is 0.38.

Rainfall Intensity:

From the rainfall intensity relationship $i = a/(b + t)^n$; for Nairobi, $b = 0.33$ and $n = 0.85$; the 2 year, 24 hour rainfall = 70mm and the 1 hour rainfall ratio = 0.5. i.e 1 hour. 2 year intensity = 35 mm/hour.

Thus $35 = a/(0.3 + 1)^{0.85}$; whence; $a = 43.74$ or $i = 43.74/(0.3 + t)^{0.85}$

Figure A15.1 Proposed Drain Network



Where t = time of concentration (min)

The flow is given by: $Q = 2.78ciA$

Where Q = design flow (l/s)

C = runoff coefficient

i = rainfall intensity (mm/hr)

a = contributing area (ha)

The calculation to design a system to pass 2 year storm is set out in Table A15.2 which represents the Modified Rational method.

As the time of concentration depends on the diameter chosen, the solution must be iterative by guessing an appropriate diameter in the final column and then checking that this diameter will just pass the design flow.

The pipe travel time are most conveniently obtained from tables based on the Colebrook - White equation or from the formula:

$$V = -\sqrt{32gRS}\, log10 \left(\frac{k}{14888R} + \frac{1.225v}{R\sqrt{32gRS}} \right)$$

Where,

v = velocity (m/s)

R = hydraulic radius

S = energy gradient

γ = kinematic viscosity m^2/s

K = equivalent sand roughness (mm) (see Table A15.1)

g = gravitational acceleration (m/s^2)

Results: The last column shows the recommended pipe diameters of the system.

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Table A15.1 Recommended Equivalent Sand Roughness Value (k)

Storm Drain Material		Roughness (mm)
Concrete		0.6
Asbestos Cement		0.6
Clayware		0.3
uPVC		0.15
Brick	Well pointed	3
	Old, mortar loss	15

***Note:** Where sediment is present in the pipe invert, very high resistance to flow can occur, equivalent to values of up to several hundred millimetres.*

Table A15.2 Storm Frequency, one in 2 years; Roughness Coefficient 0.6 mm; Time of Entry, 2 min.

Pipe No.	Difference in level (m)	Length (m)	Gradient (1 in..)	Velocity (m/s)	Time of Flow (min)	Time of Concentration (min)	Rate of Rainfall (mm/hr)
1.00	1.10	63.10	57.00	1.74	0.60	2.60	108.50
1.01	1.12	66.10	59.00	2.05	0.54	3.14	106.20
1.02	0.73	84.70	116.00	1.46	0.97	4.11	102.20
2.00	1.40	44.80	32.00	2.31	0.32	2.32	109.80
2.01	0.61	49.10	80.00	1.76	0.46	2.78	107.70
3.00	0.98	48.50	49.00	1.44	0.56	2.56	108.70
2.02	1.65	54.30	33.00	2.73	0.33	3.11	106.30
1.03	1.22	27.70	23.00	3.81	0.12	4.23	101.70
1.10	0.37	58.00	156.00	2.24	0.43	7.89	89.40
1.11	0.32	54.00	168.00	2.16	0.42	8.31	88.10

Pipe No.	Paved (m)	Unpaved (m)	Runoff Coefficient	Effective Pervious (m/s)	Total Area	Cummulative (min)	Rate of Flow (l/s)	Pipe Dia. (mm)
1.00	1.10	63.10	57.00	1.74	0.60	2.60	108.50	225
1.01	1.12	66.10	59.00	2.05	0.54	3.14	106.20	225
1.02	0.73	84.70	116.00	1.46	0.97	4.11	102.20	300
2.00	1.40	44.80	32.00	2.31	0.32	2.32	109.80	225
2.01	0.61	49.10	80.00	1.76	0.46	2.78	107.70	300
3.00	0.98	48.50	49.00	1.44	0.56	2.56	108.70	150
2.02	1.65	54.30	33.00	2.73	0.33	3.11	106.30	300
1.03	1.22	27.70	23.00	3.81	0.12	4.23	101.70	375
1.10	0.37	58.00	156.00	2.24	0.43	7.89	89.40	750
1.11	0.32	54.00	168.00	2.16	0.42	8.31	88.10	750

A.16 Design of Gutter

Given: Gutter section illustrated in Figure 7.1: - Typical Gutter Section a.1. with:

$$S_L = 0.01 \text{ m/m}$$

$$S_x = 0.020 \text{ m/m}, \; n = 0.016$$

Find: 1. Spread at a flow of 0.05 m³/s

2. Gutter flow at a spread of 2.5 m

Solution 1:

Step 1. Compute spread, T, using Equation 7.9 or Chart B.17 in appendix B.

$$T = \frac{Q_n}{(K_u \, S_L^{1.67} \, S_x^{0.5})^{0.375}}$$

$$T = \frac{0.05(0.016)}{\{0.376\,(0.020)^{1.67}\,(0.010)^{0.5}\}^{0.375}}$$

$$T = 2.7 \text{ m}$$

Solution 2:

Step 1. Using Equation 7.9 or Chart B.17 with T = 2.5 m and the information given above, determine Q_n.

$$Q = \frac{K_u}{n} S_x^{\;1.67} \, S_L^{\;0.5} \, T^{2.67}$$

$Q_n = (0.376)(0.020)^{1.67}(0.010)^{0.5}(2.5)^{2.67}$

$Q_n = 0.00063$ m³/s

Step 2. Compute Q from Q_n determined in Step 1.

$$Q = \frac{Q_n}{n}$$

Q = 0.00063 / .016

Q = 0.039 m³/s.

A.17 To Determine Spread, Given Gutter Flow.

Step 1: Determine input parameters, including longitudinal slope (S), cross slope (Sx), 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 (Qs). Example: S = 0.01; S_x = 0.02; S_w = 0.06; W = 0.6m; n = 0.016; Q = 0.057 m³/s; try Q_s = 0.020 m³/s.

Step 2: Calculate the gutter flow in W (Q_w), using the equation:

Total flow (Q) = *Gutter flow in the depressed section* (Q_w) + *flow above the depressed section* (Q_s)

$$Q_w = Q - Q_s; \; (Q_w = 0.057 - 0.020 = 0.037 \text{ m}^3\text{/s}) \quad \text{(Equation 7.13)}$$

Step 3: Calculate the ratios Q_w/Q and S_w/S_x and use Chart B.18 to find an appropriate value of W/T,

$$\frac{Q_w}{Q} = \frac{0.037}{0.057} = 0.65; \; \frac{S_w}{S_x} = \frac{0.06}{0.02} = 3 \; \frac{W}{T} = 0.27$$

Step 4: Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

$$T = \frac{0.6}{0.27} = 2.22 \text{ m}$$

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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 Chart B.24, (Q_s = 0.014 m³/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 7.7, W/T = 0.23, then T = 0.6/0.23 = 2.61m and T_s = 2.61 – 0.6 = 2.01m, From Chart B.24. Q_s = 0.023 m³/s OK.

Answer: Spread T = 2.61m

A.18 To Determine 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 Chart B.24 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 Chart B.24, Q_s = 0.085 m³/s

Step 3: Calculate the ratios W/T and S_w/S_x, and, from Chart B-18, 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 Chart B.18. E_o = 0.46

Step 4: Calculate the total gutter flow using the equation:

$$Q = Q_s\,(1 - E_o)$$

Where

Q = gutter flow rate, m³/s

Q_s = flow capacity of the gutter section above the depressed section, m³/s

E_o = ratio of frontal flow to total gutter flow (Q_w/Q). Q = 0.085/(1 – 0.46) = 0.157 m³/s

Step 5: Calculate the gutter flow in width (W):

Total flow (Q) = *Gutter flow in the depressed section* (Q_w) + *flow above the depressed section* (Q_s)

Therefore, $Q_w = Q - Q_s$ = 0.157 – 0.085 = **0.072 m³/s**

Appendices

Appendix B: Design Charts

Chart B.1 Solution to Manning's Equation for Channels of Various Side Slopes

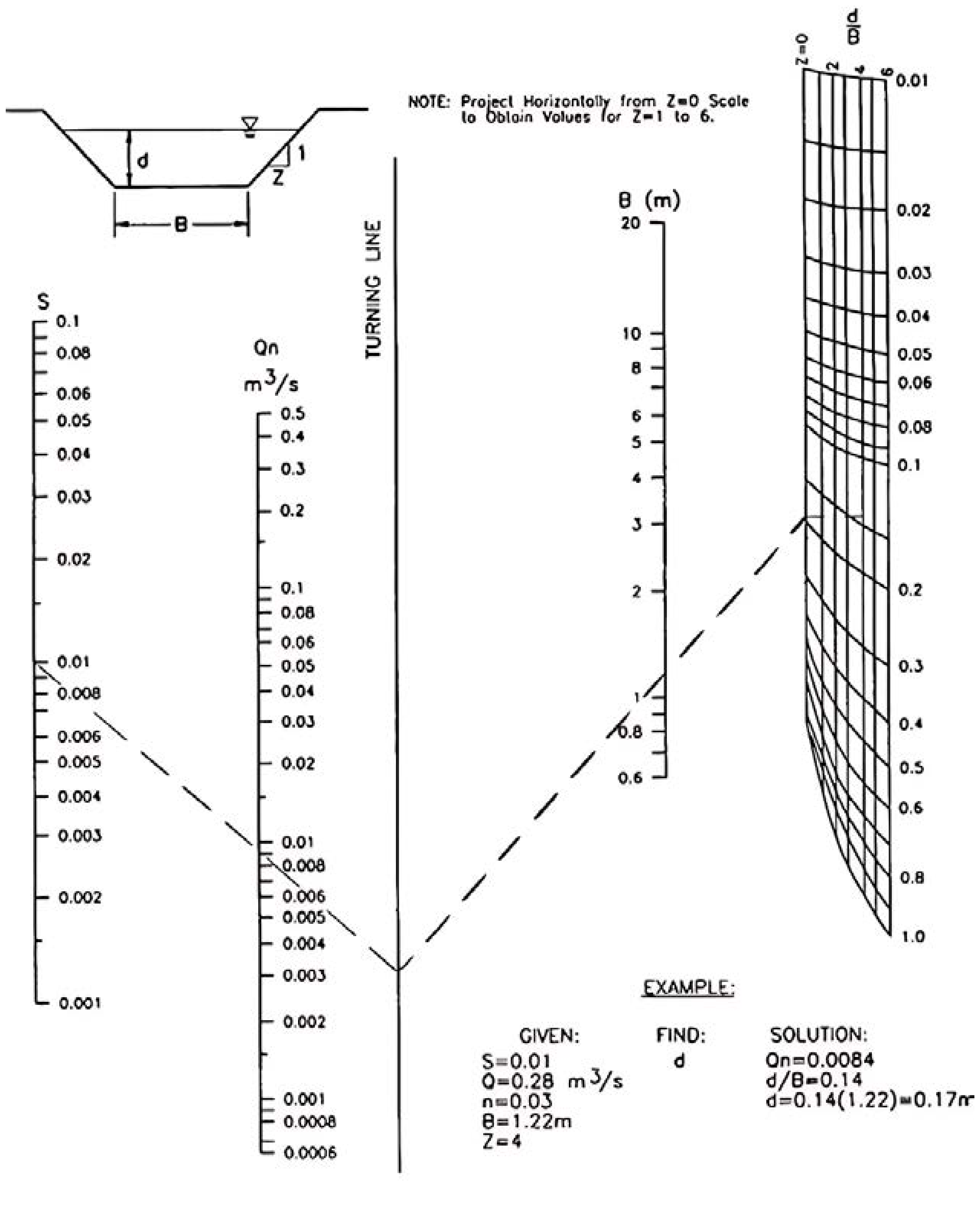

Chart B.2 Trapezoidal Channel Capacity (See Table 2.2)

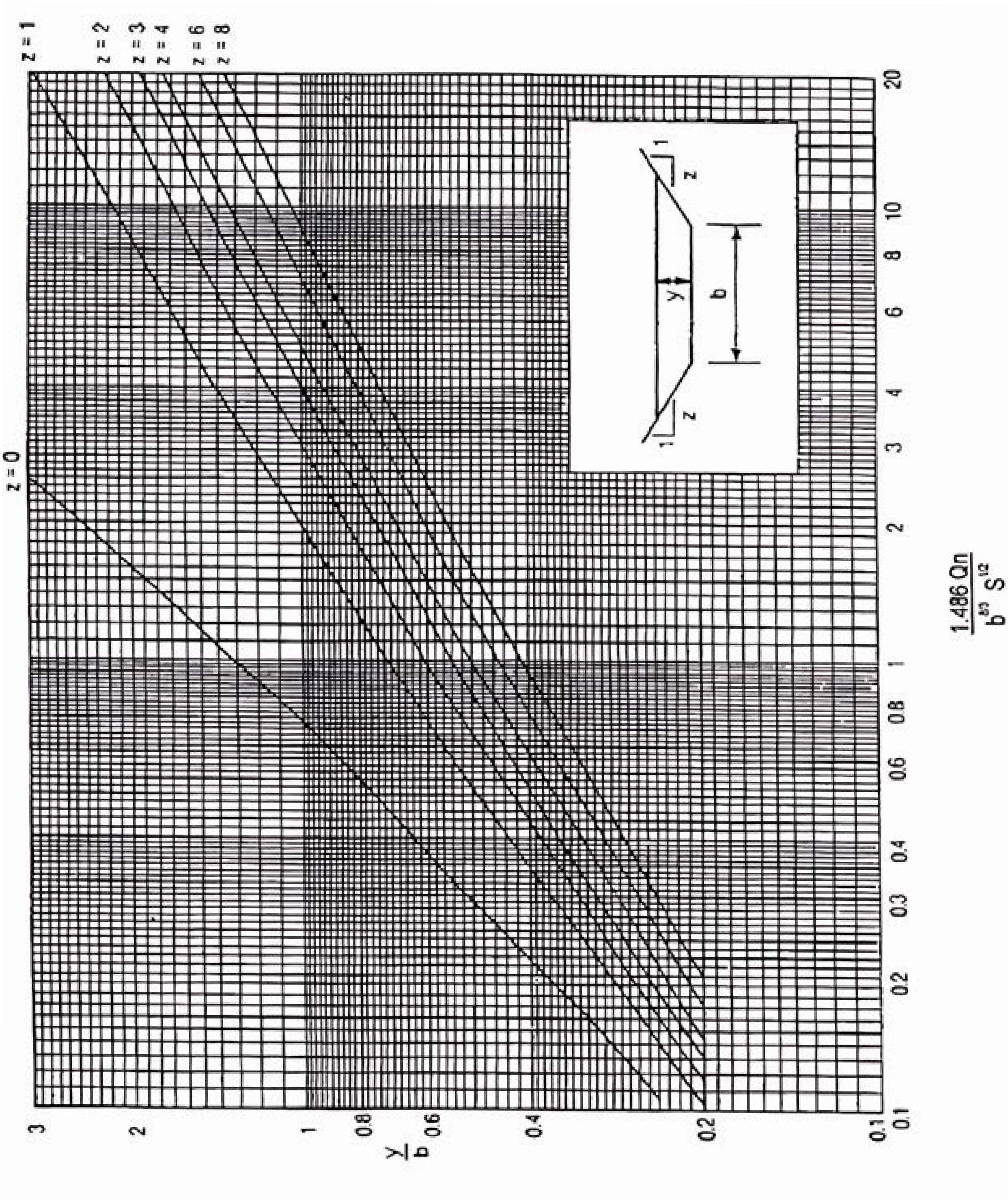

Chart B.3 Geometric Design Chart for Trapezoidal Channels

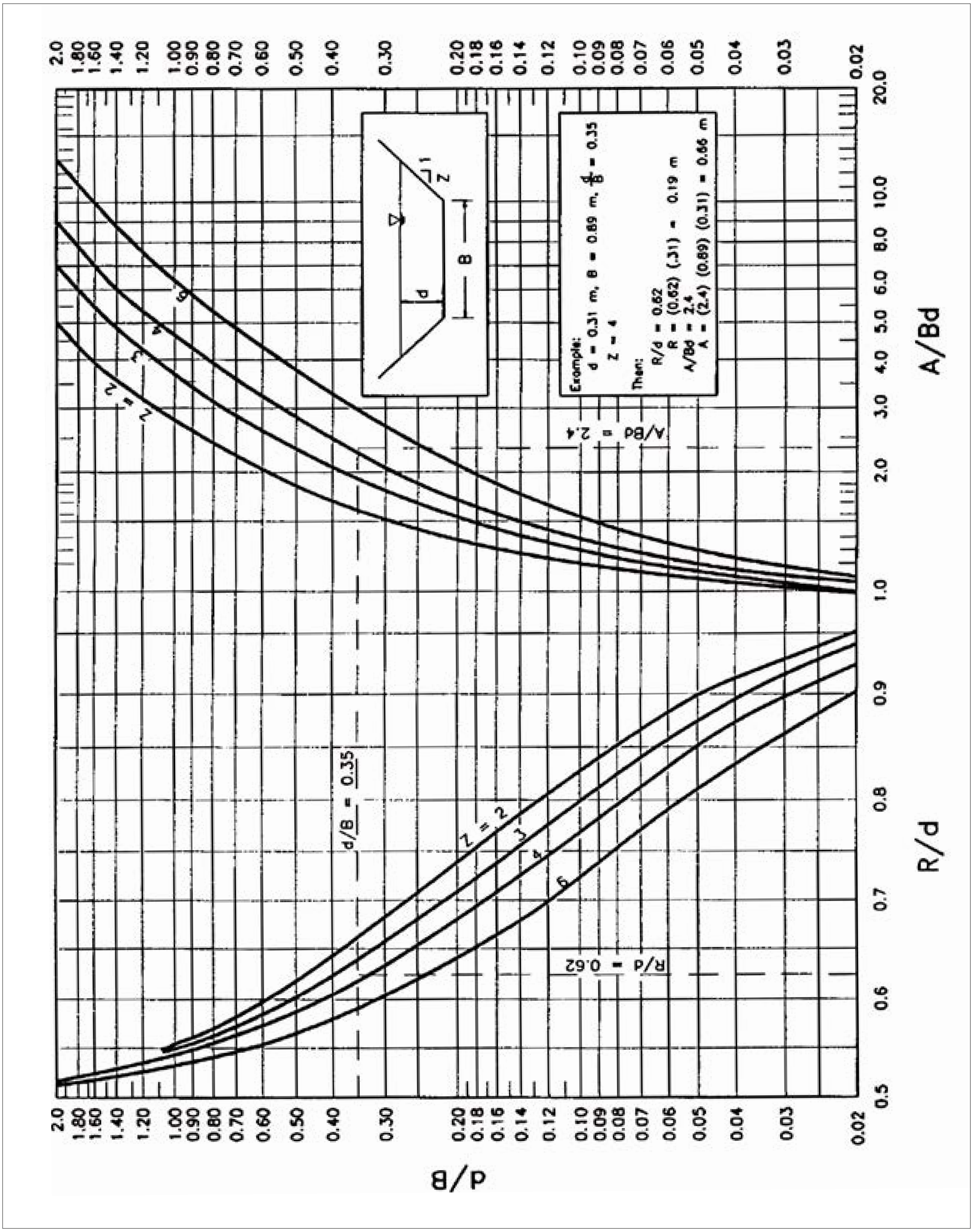

Chart B.4 Ratio of Frontal Flow to Total Flow in Trapezoidal Channels

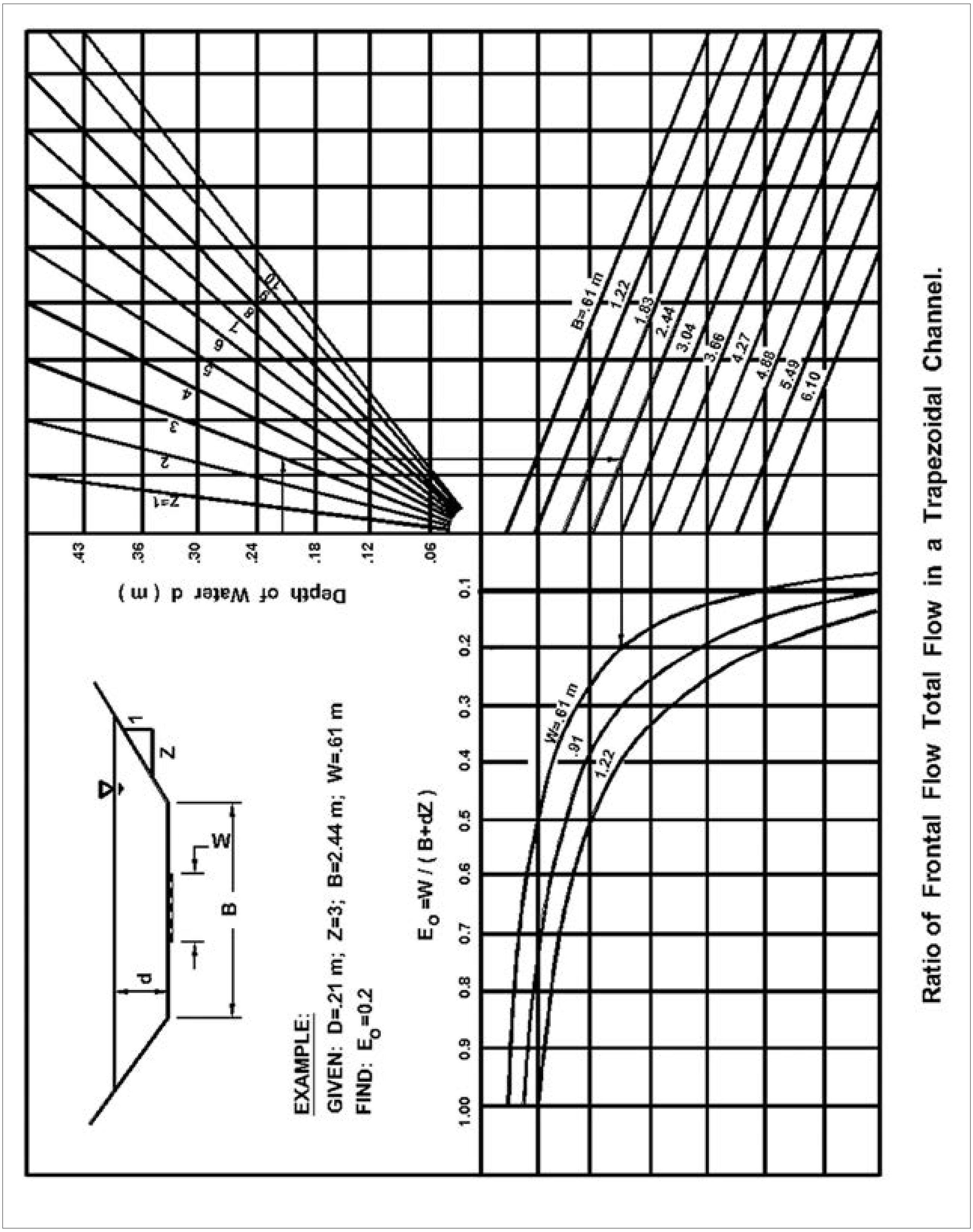

Chart B.5 Headwater Depth and Capacity for Concrete Pipe Culverts with Inlet Control

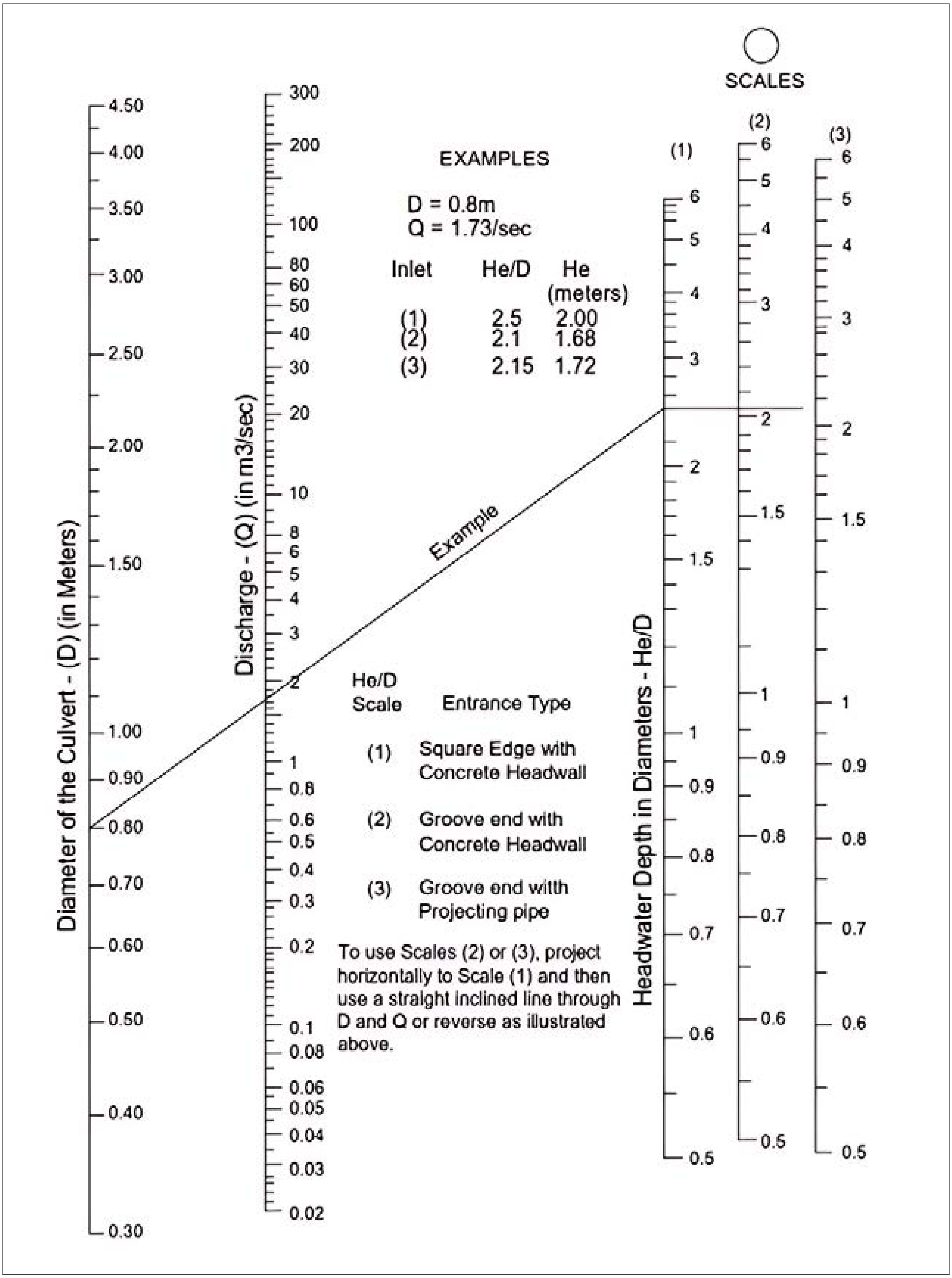

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Chart B.6 Headwater Depth and Capacity for Concrete Box Culverts with Inlet Control

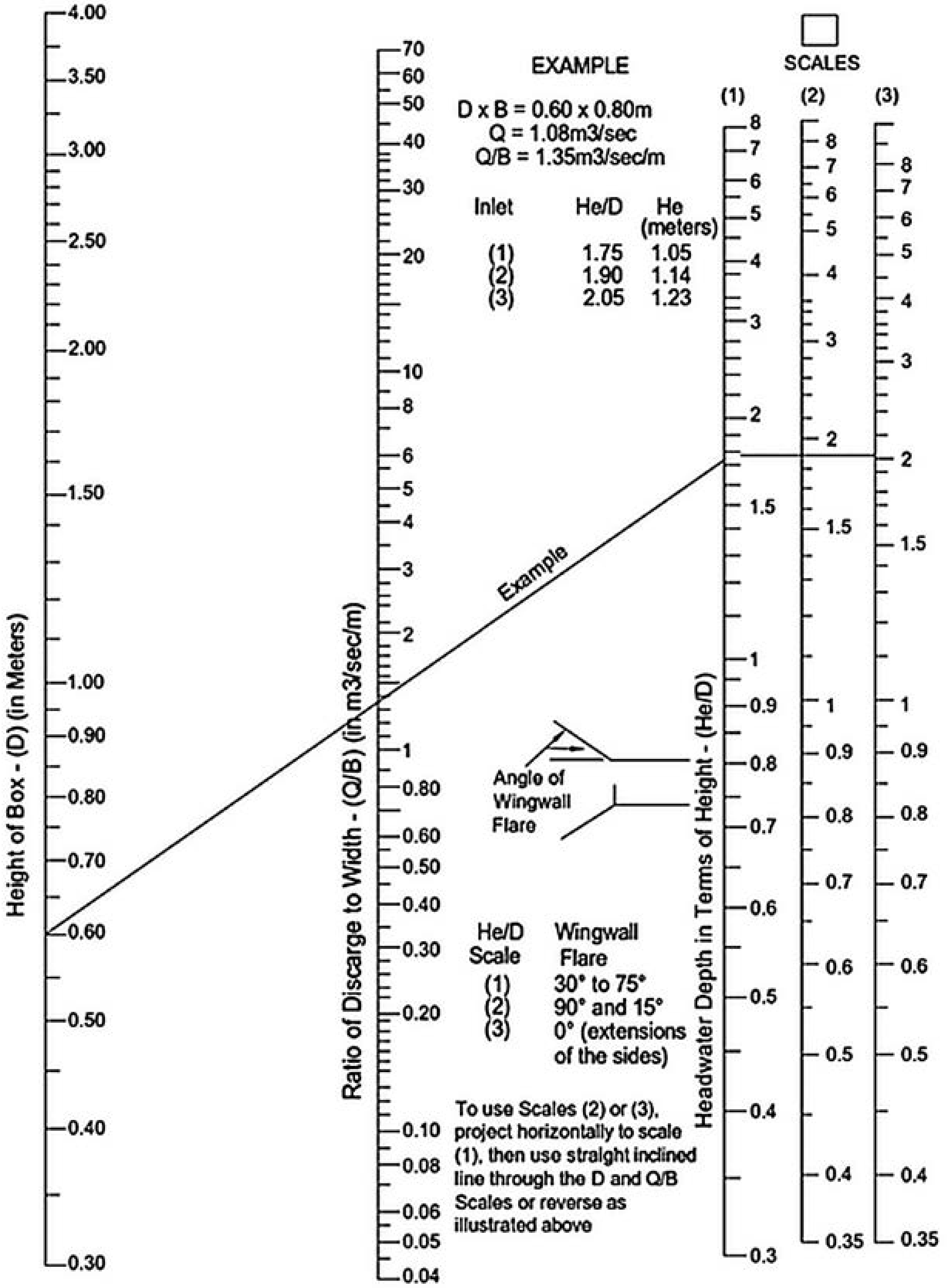


Chart B.7 Headwater Depth and Capacity for C. M. Pipe Culverts with Inlet Control

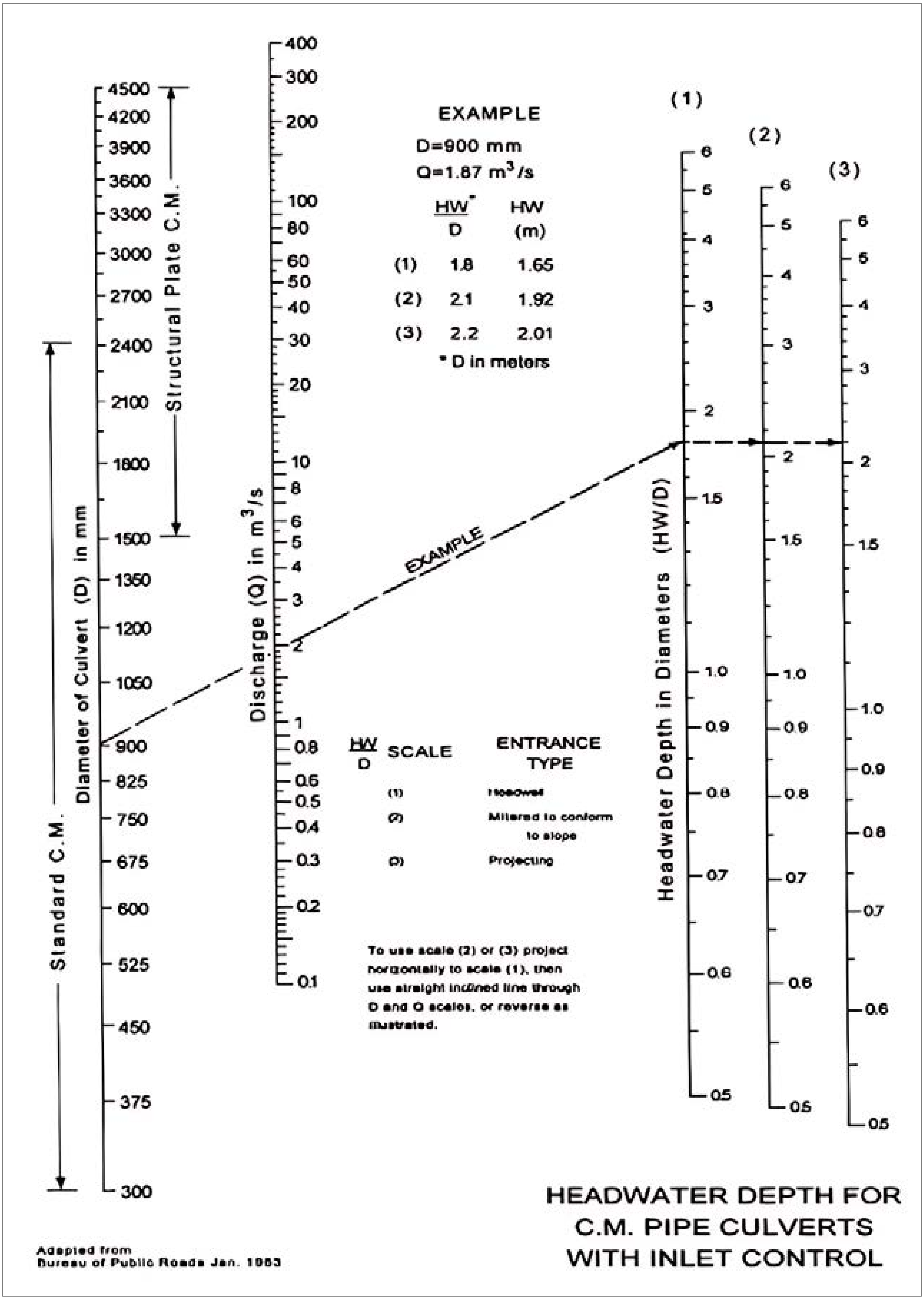
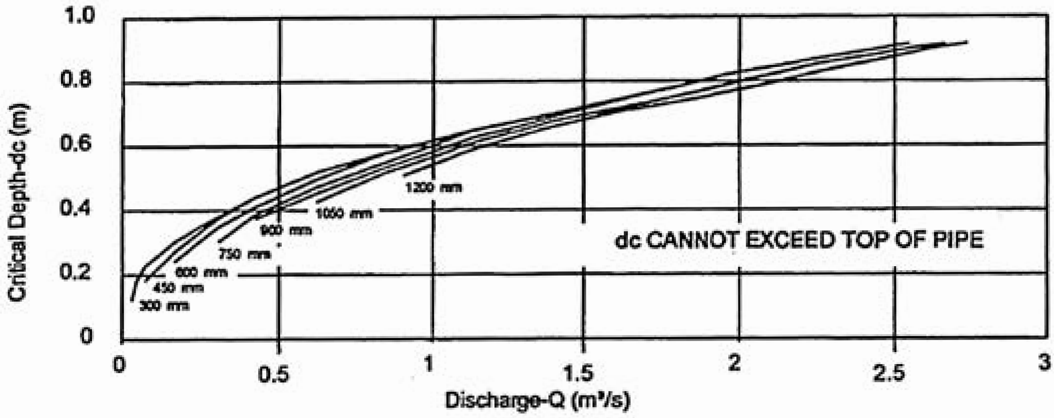
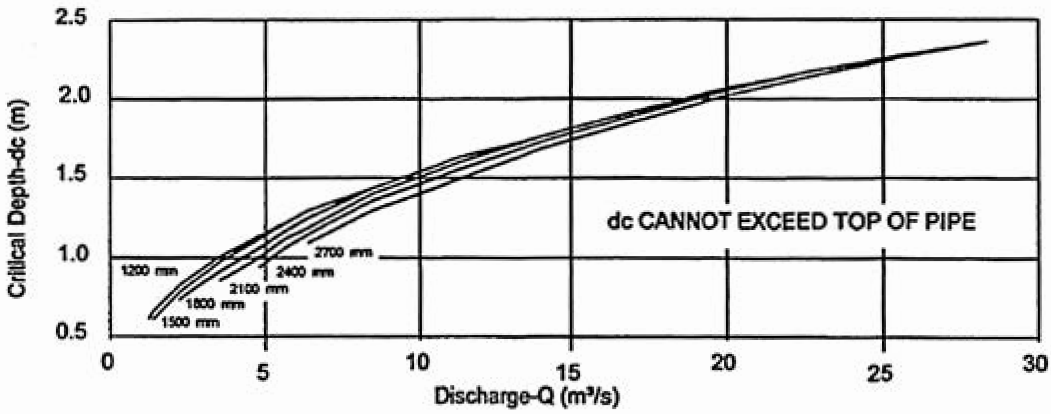
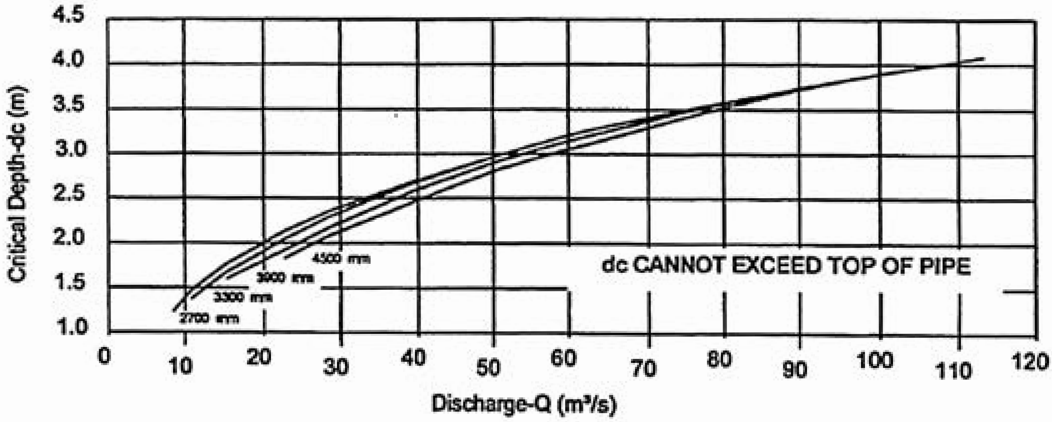


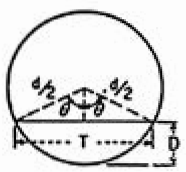
Chart B.8 Critical Depth for Circular Pipes







Adapted from Bureau of Public Roads



$$Q = \frac{2^{5/2} g^{1/2} (\theta_r - \tfrac{1}{2} \sin 2\theta)^{3/2}}{8 (\sin \theta)^{1/2} (1 - \cos \theta)^{5/2}} D_c^{5/2}$$

With :
Q – Discharge (m^3/s)
g - gravity constant (m/s^2)
Dc – Critical depth (m)
θ, θr - See sketch (degree, radian)

Chart B.9 Head Water Depth for Concrete Pipe Flowing Full n = 0.012

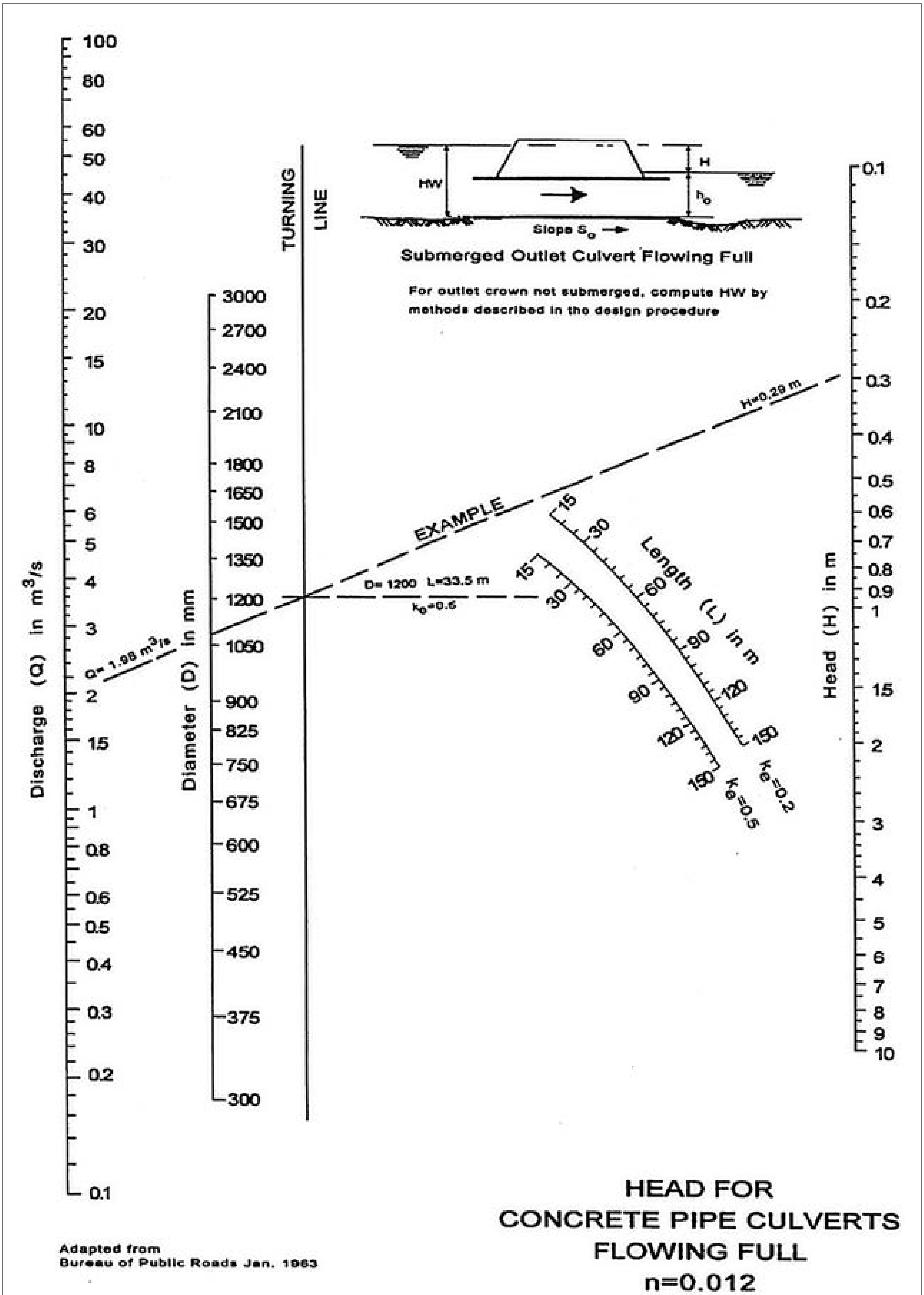


Chart B.10 Head Water Depth for Box Culvert With Inlet Control

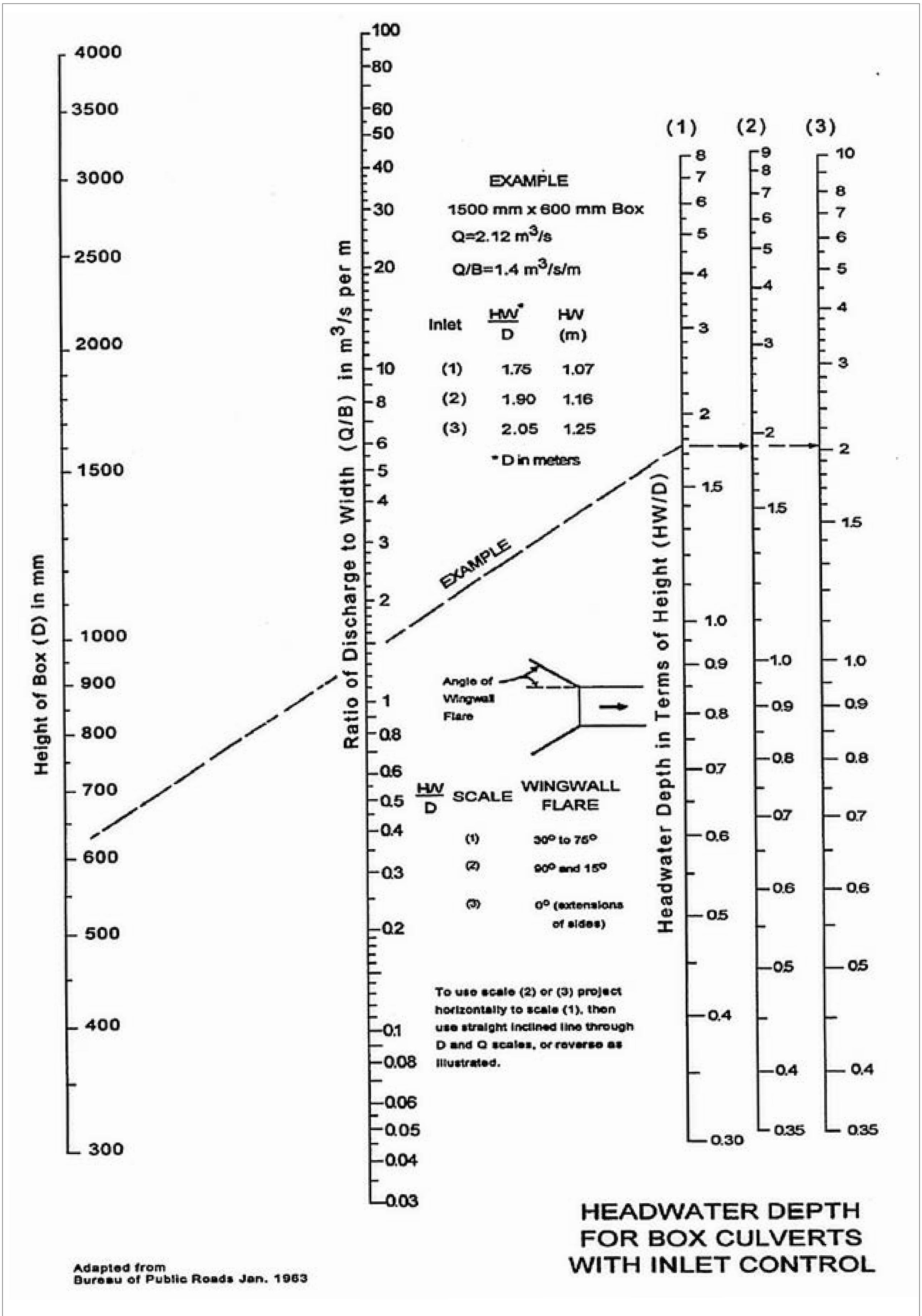
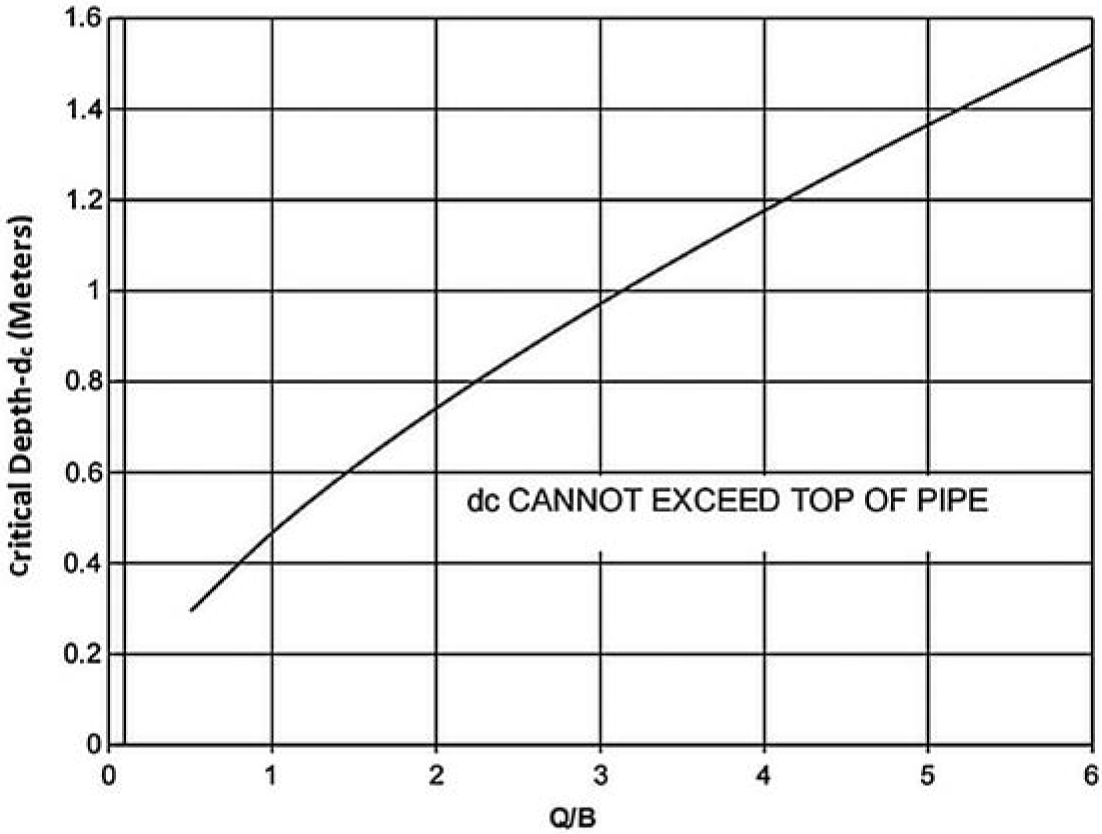


Chart B.11 Critical Depth for Rectangular Section



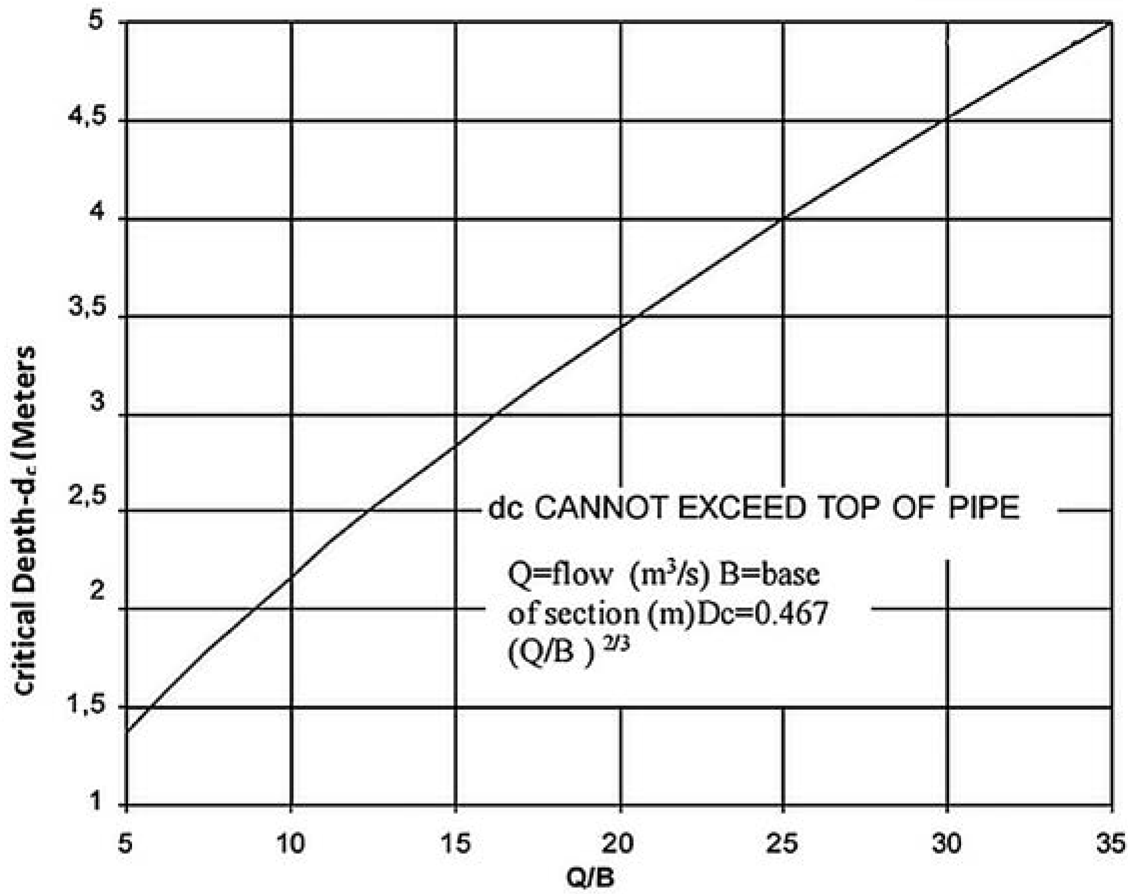


Chart B.12 Headwater Depth for Concrete Box Flowing Full, n = 0.12

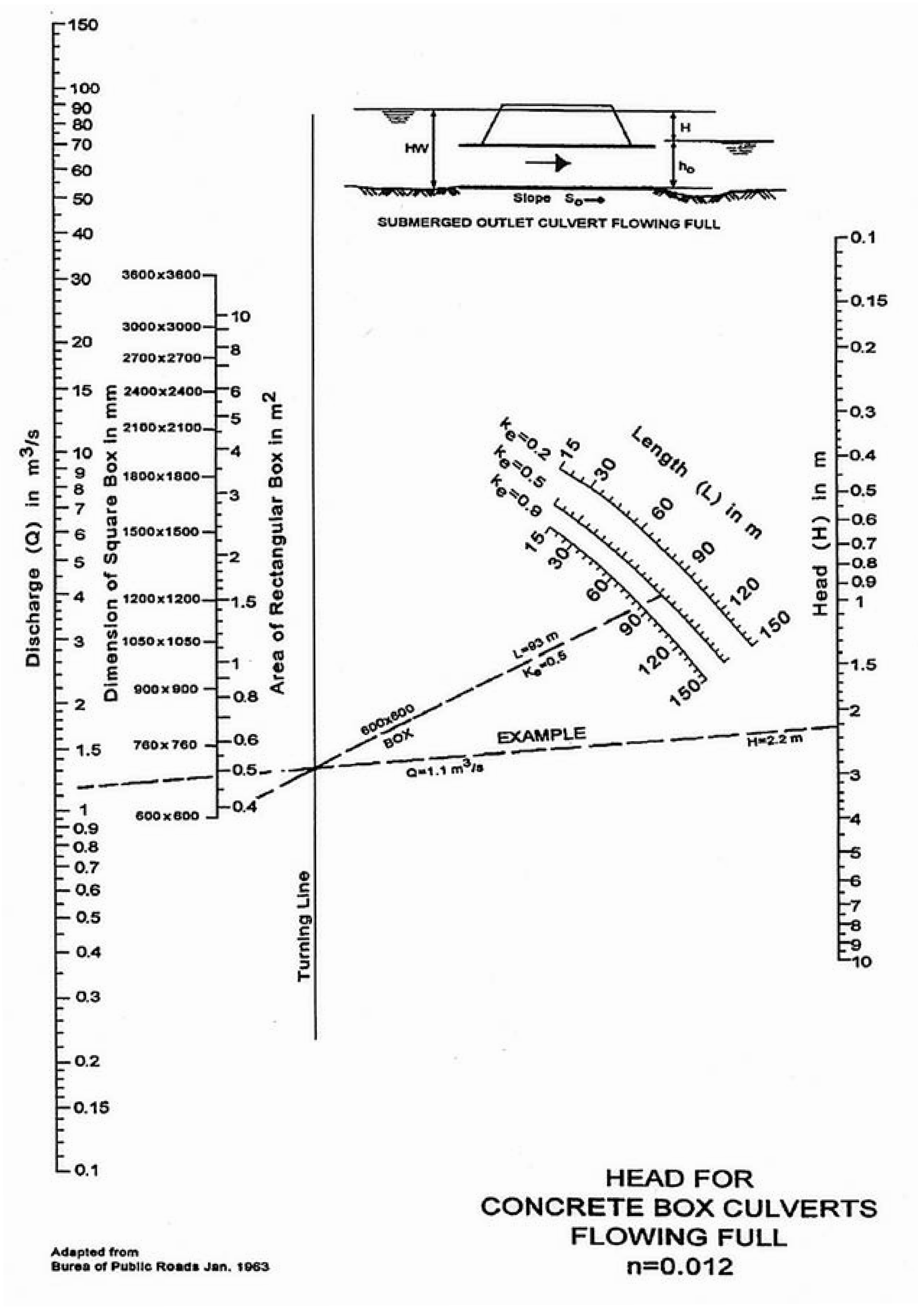


Chart B.13 Solution of Manning's Equation for Flow in Storm Drains.

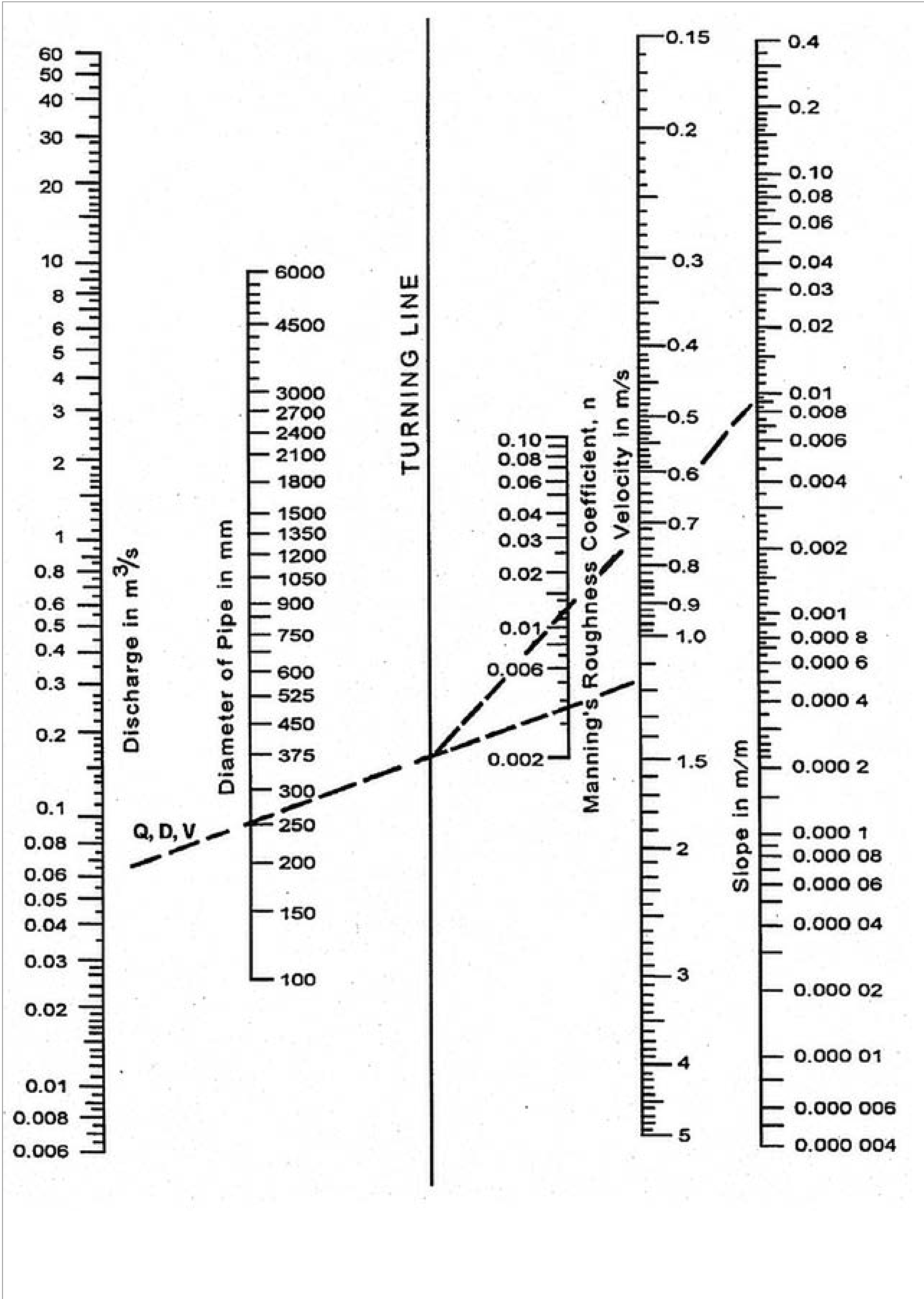


Chart B.14 Nomograph for Computing Required Size of Circular Drain Flowing Full, n = 0.013 or 0.015

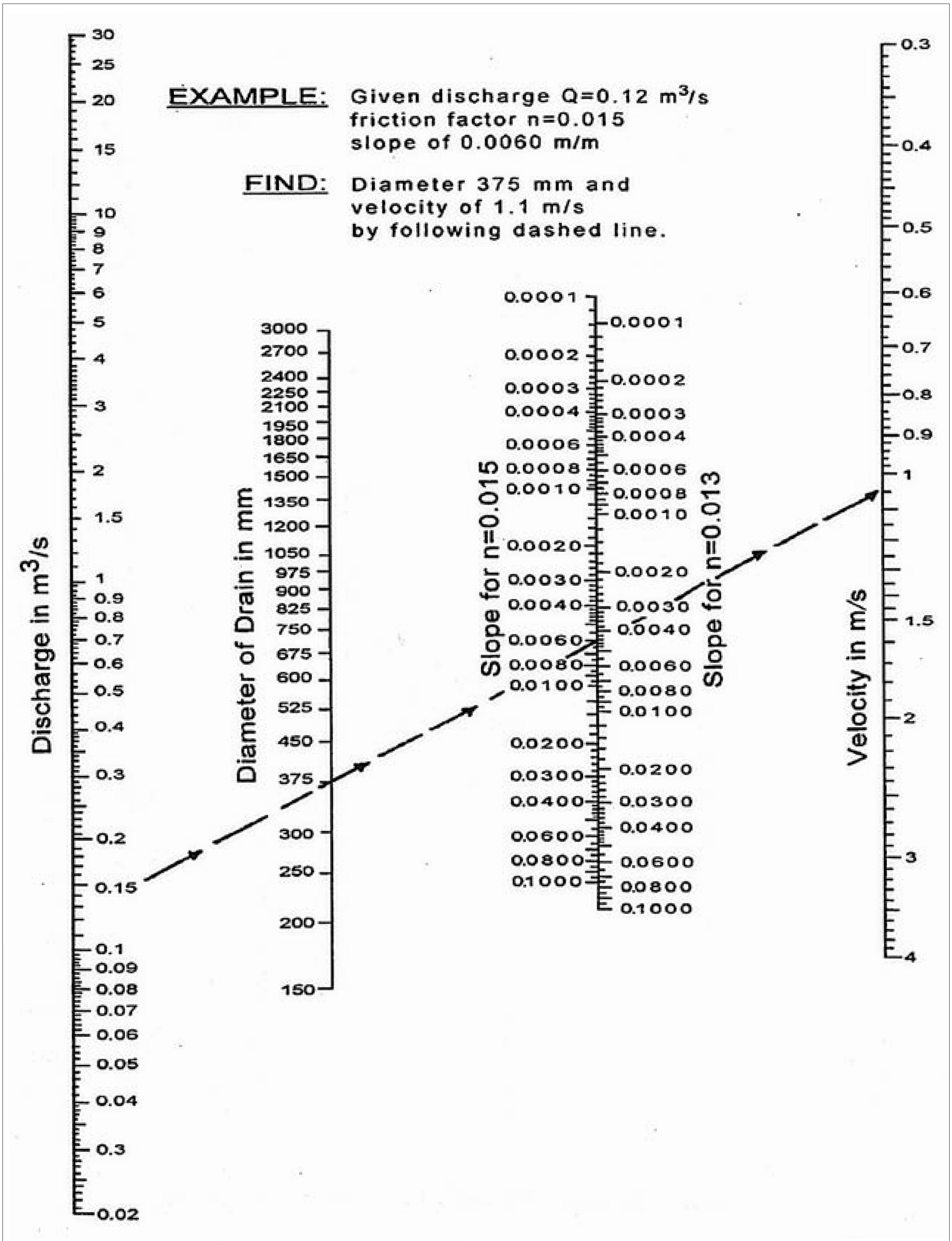


Chart B.15 Solution of Manning's Equation for Flow in Storm Drains.

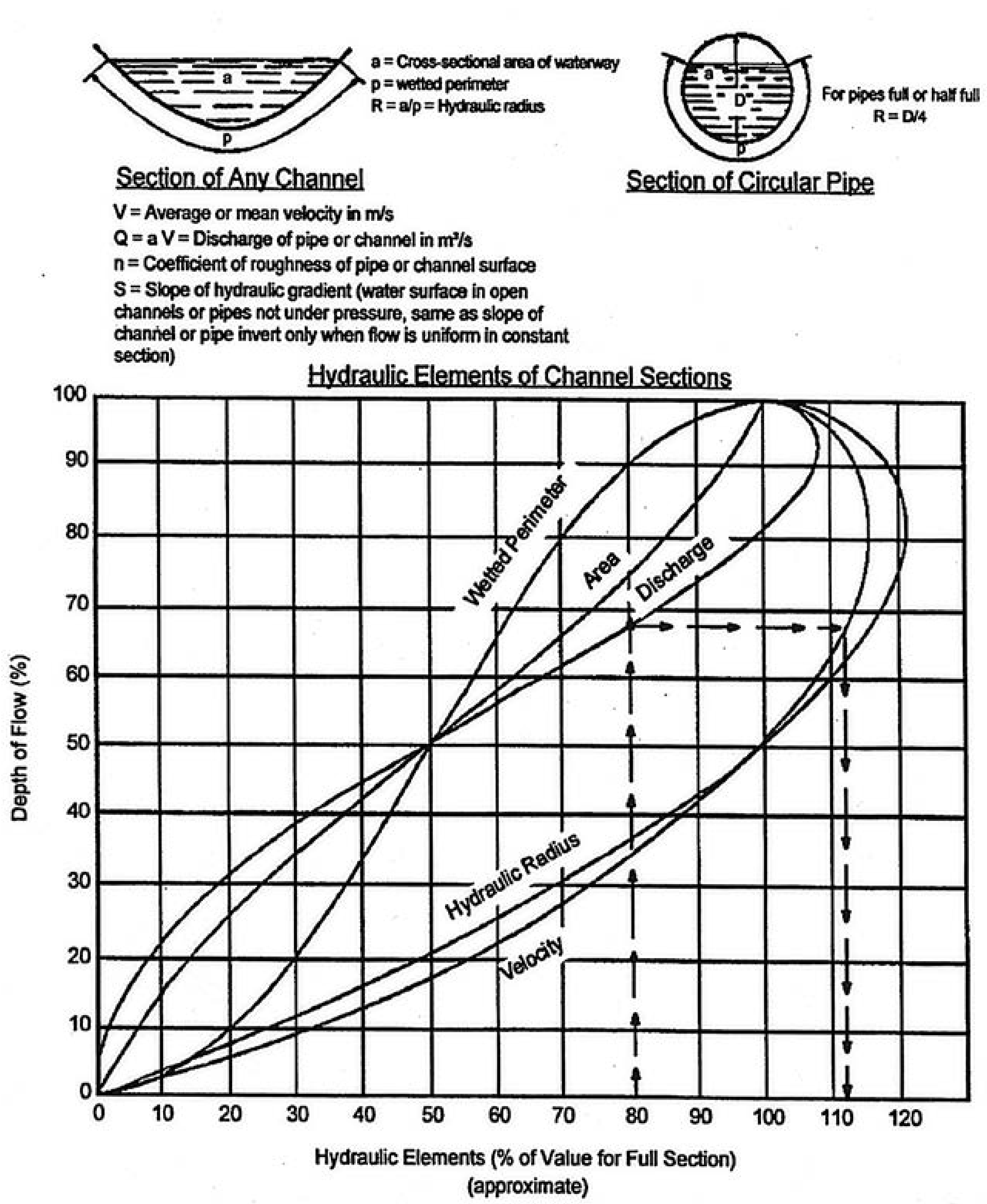


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Chart B.16 Concrete Pipe Flow Nomograph

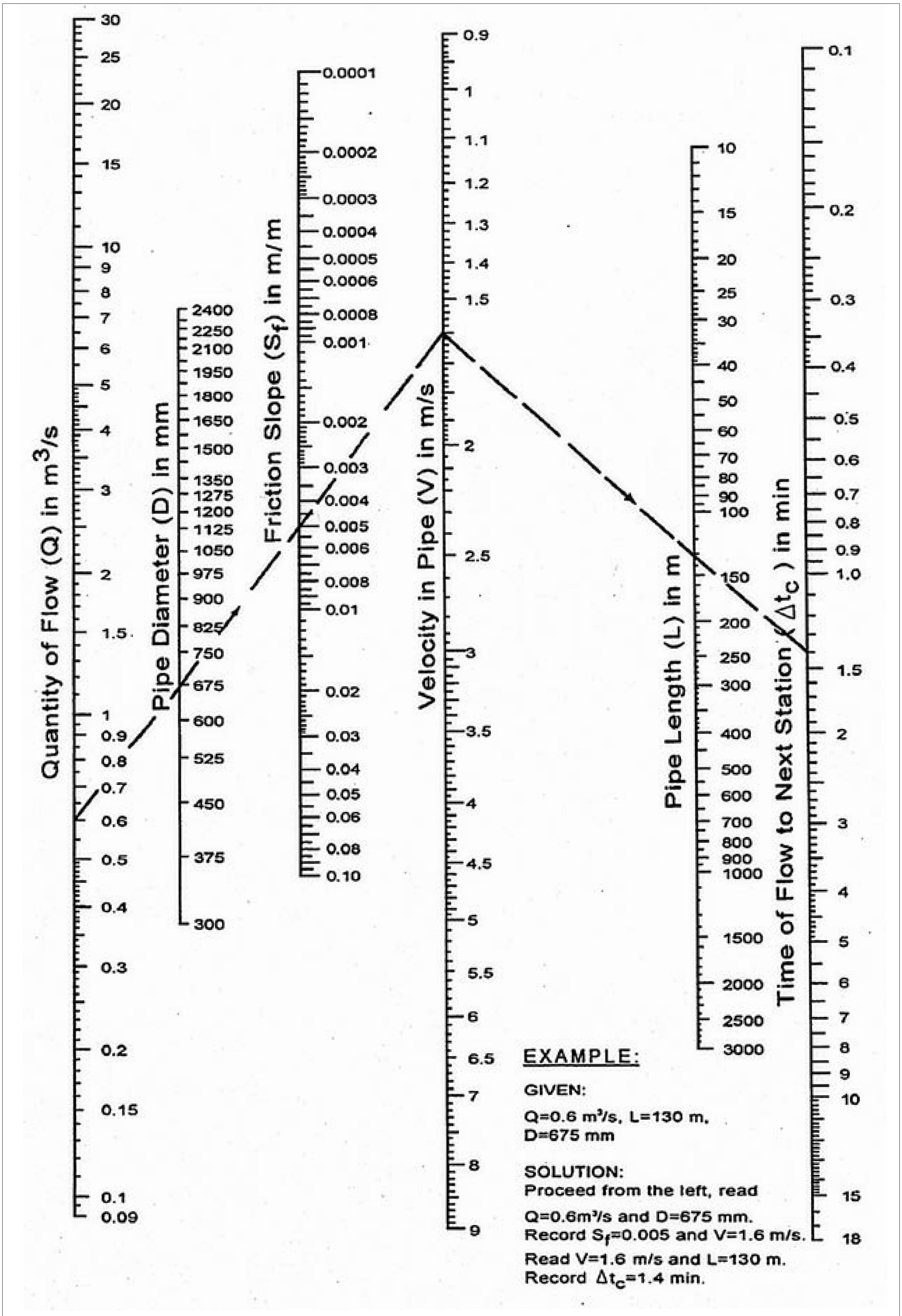

Chart B.17 Flow in Triangular Gutter Sections

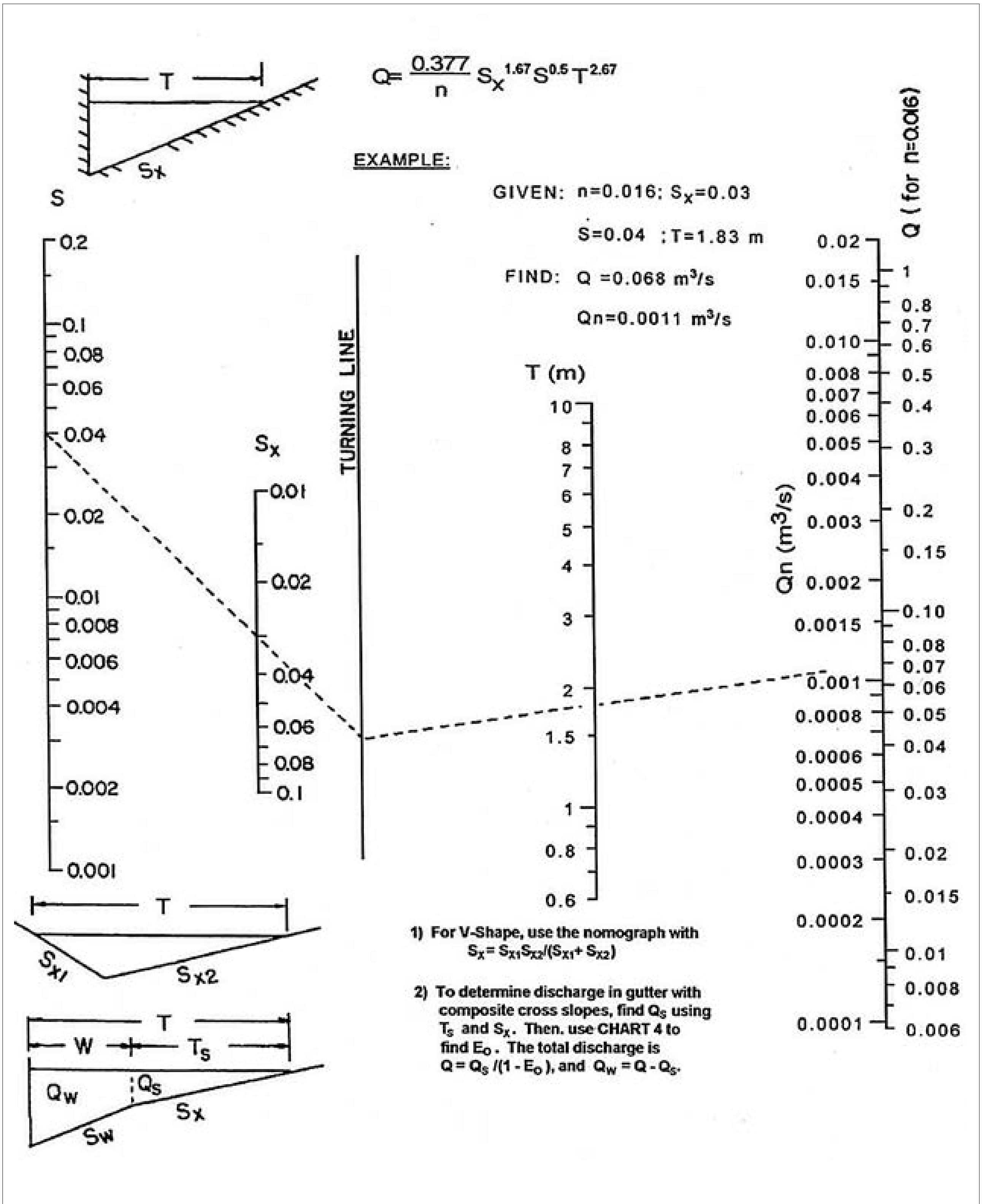

Chart B.18 Ratio of Frontal Flow to Total Gutter Flow

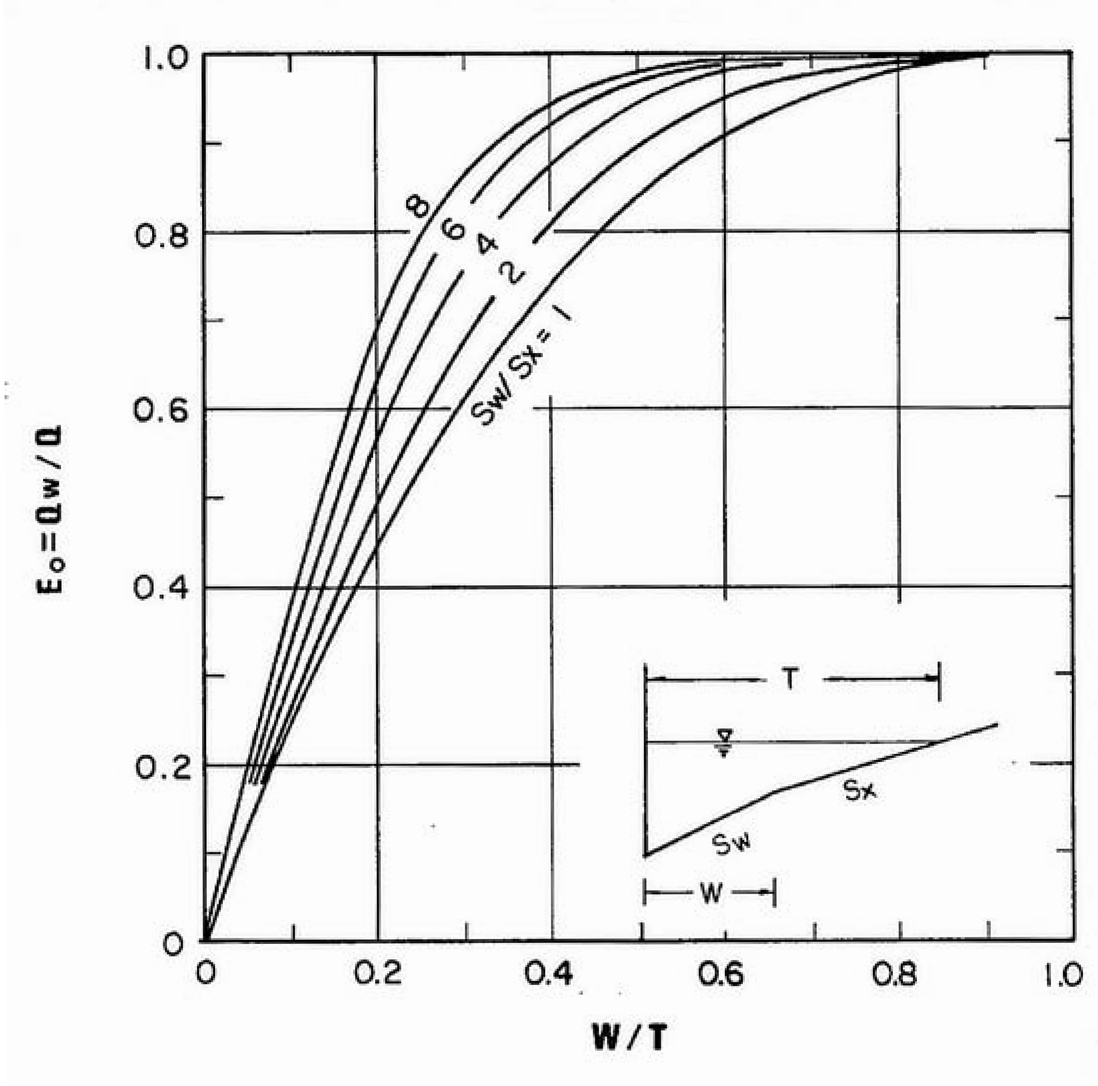

Chart B.19 Flow in Composite Gutter

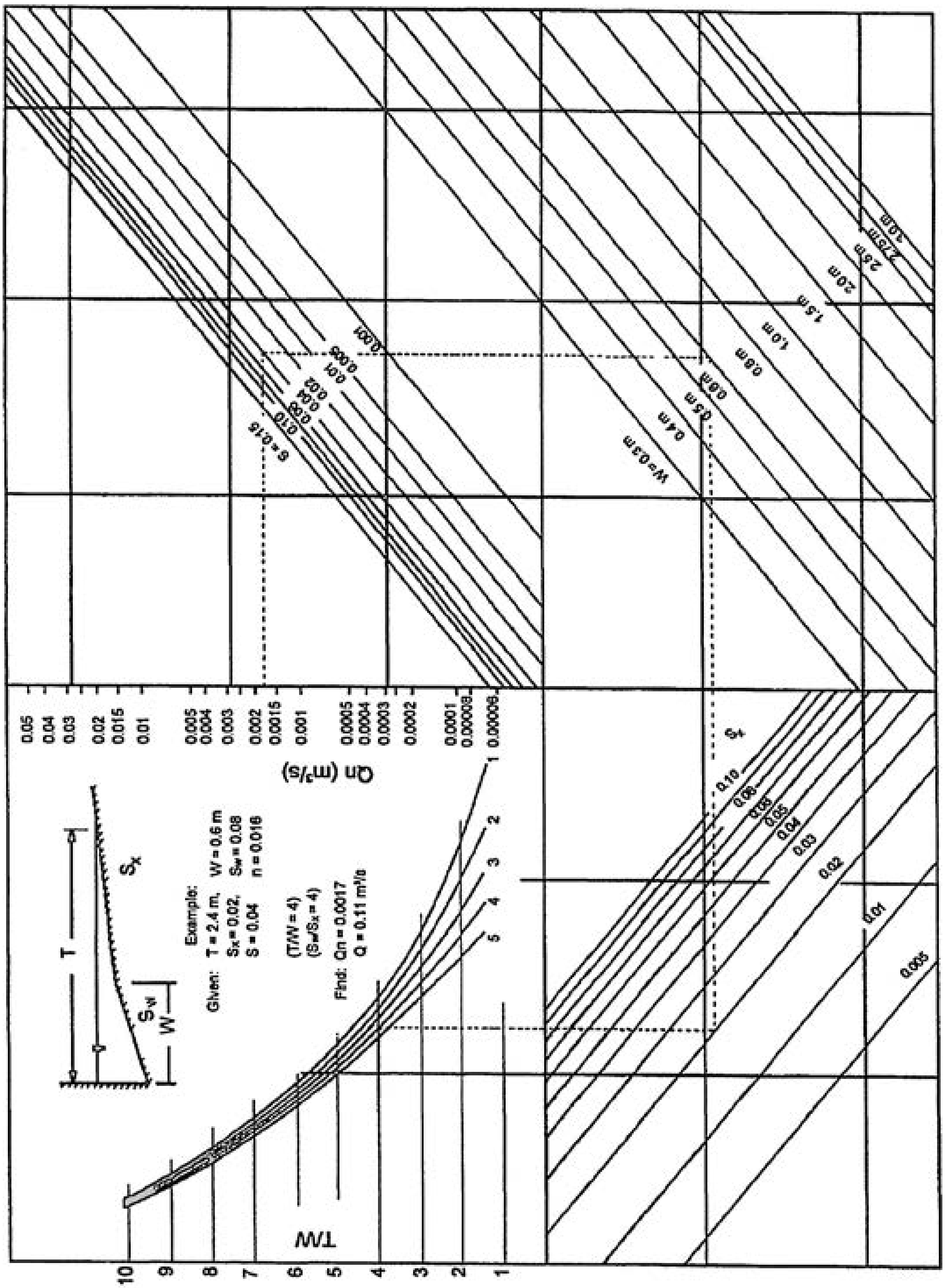

Chart B.20 Conveyance in Circular Channels

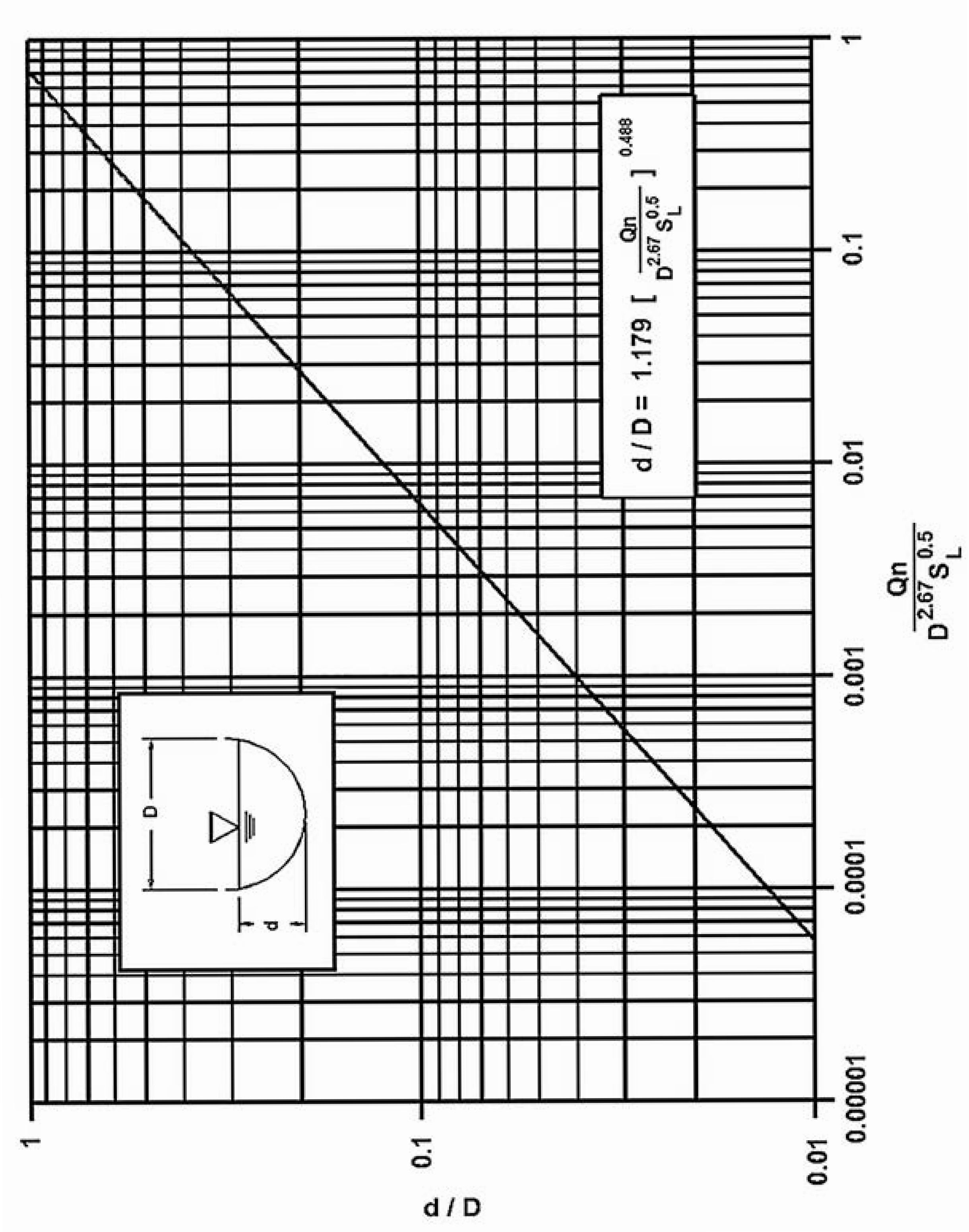

Chart B.21 Velocity in Triangular Gutters

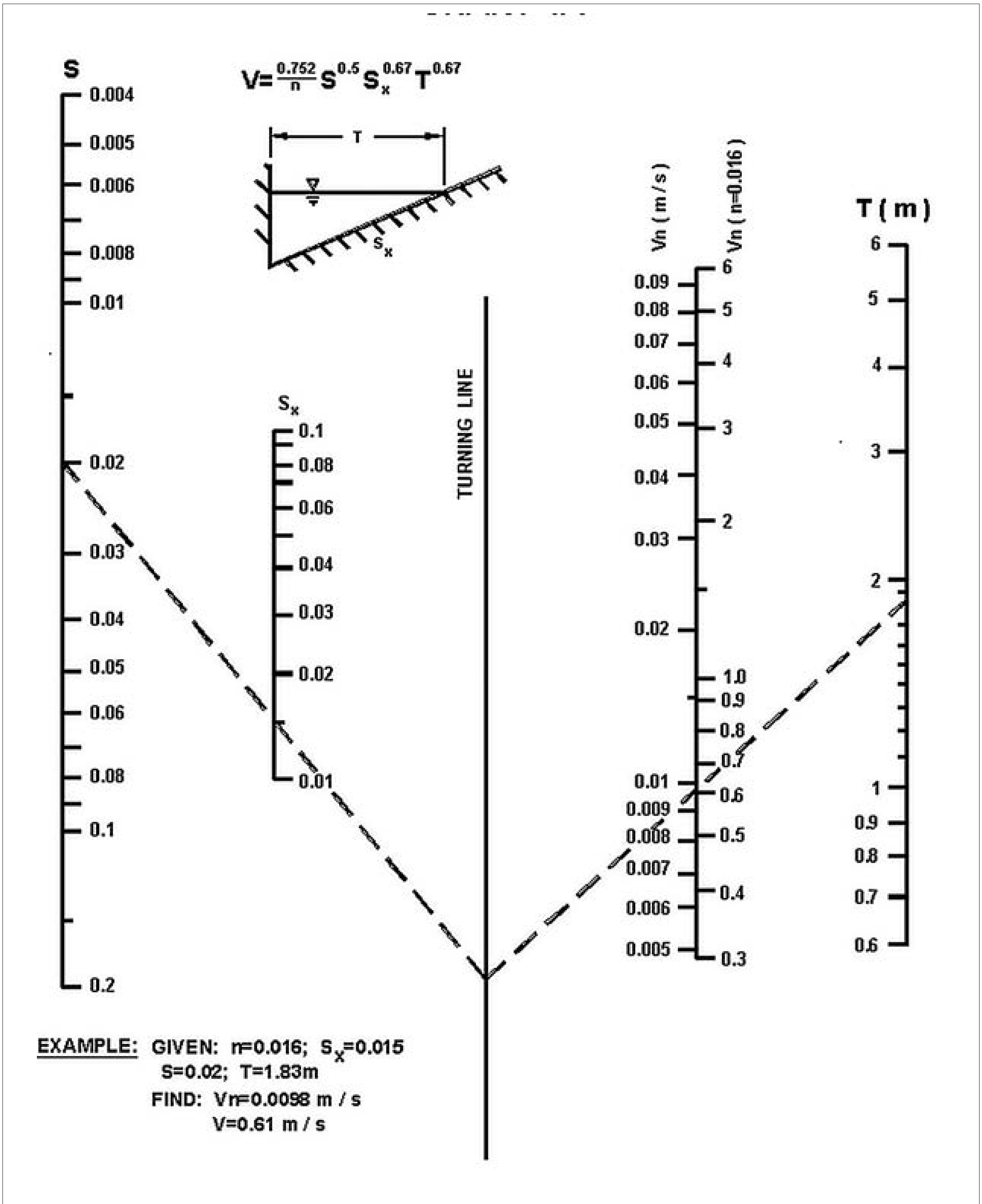

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Chart B.22 Grated Inlet Capacity in Sump Conditions

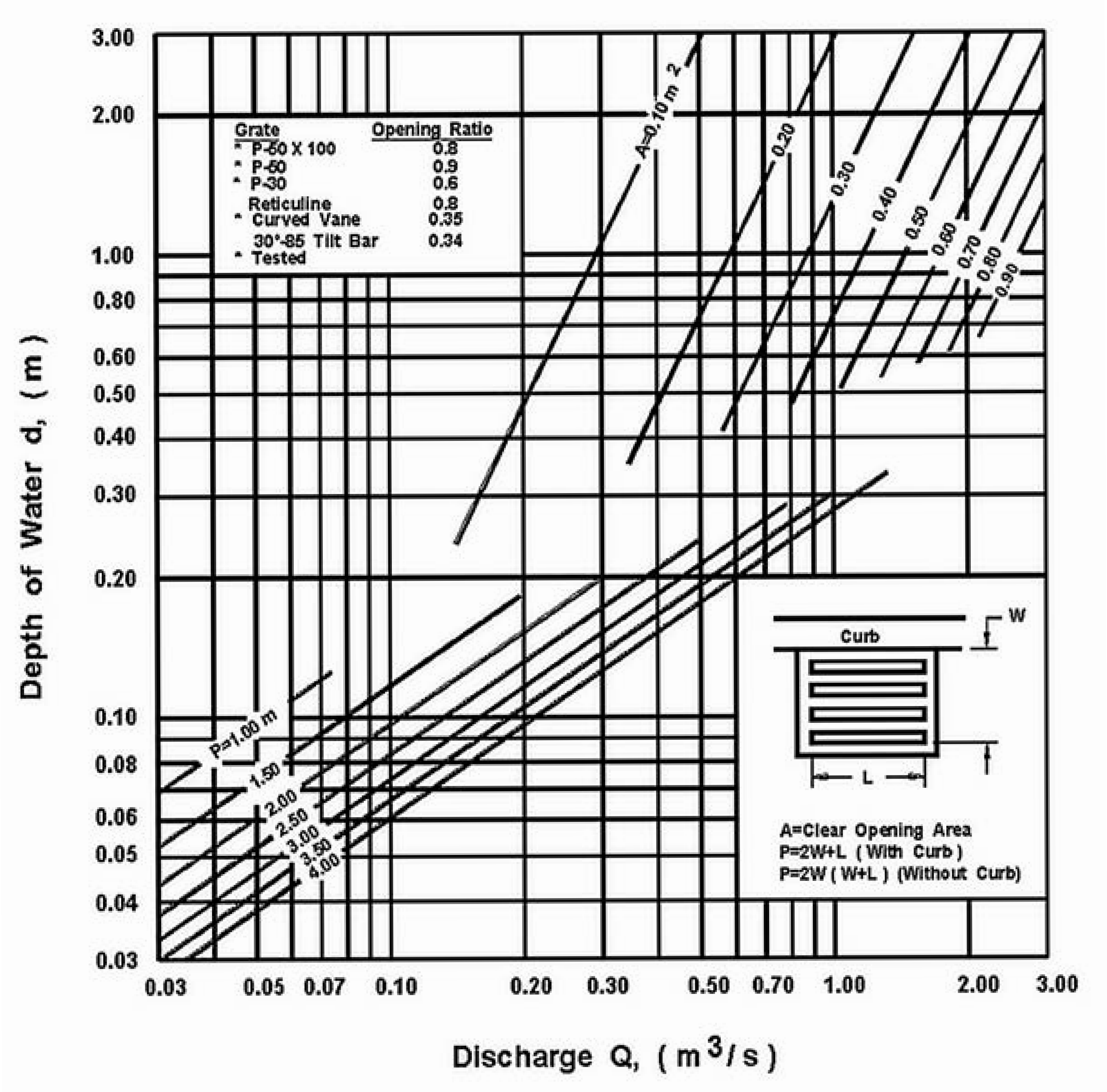


Chart B.23 Velocity in Triangular Gutters

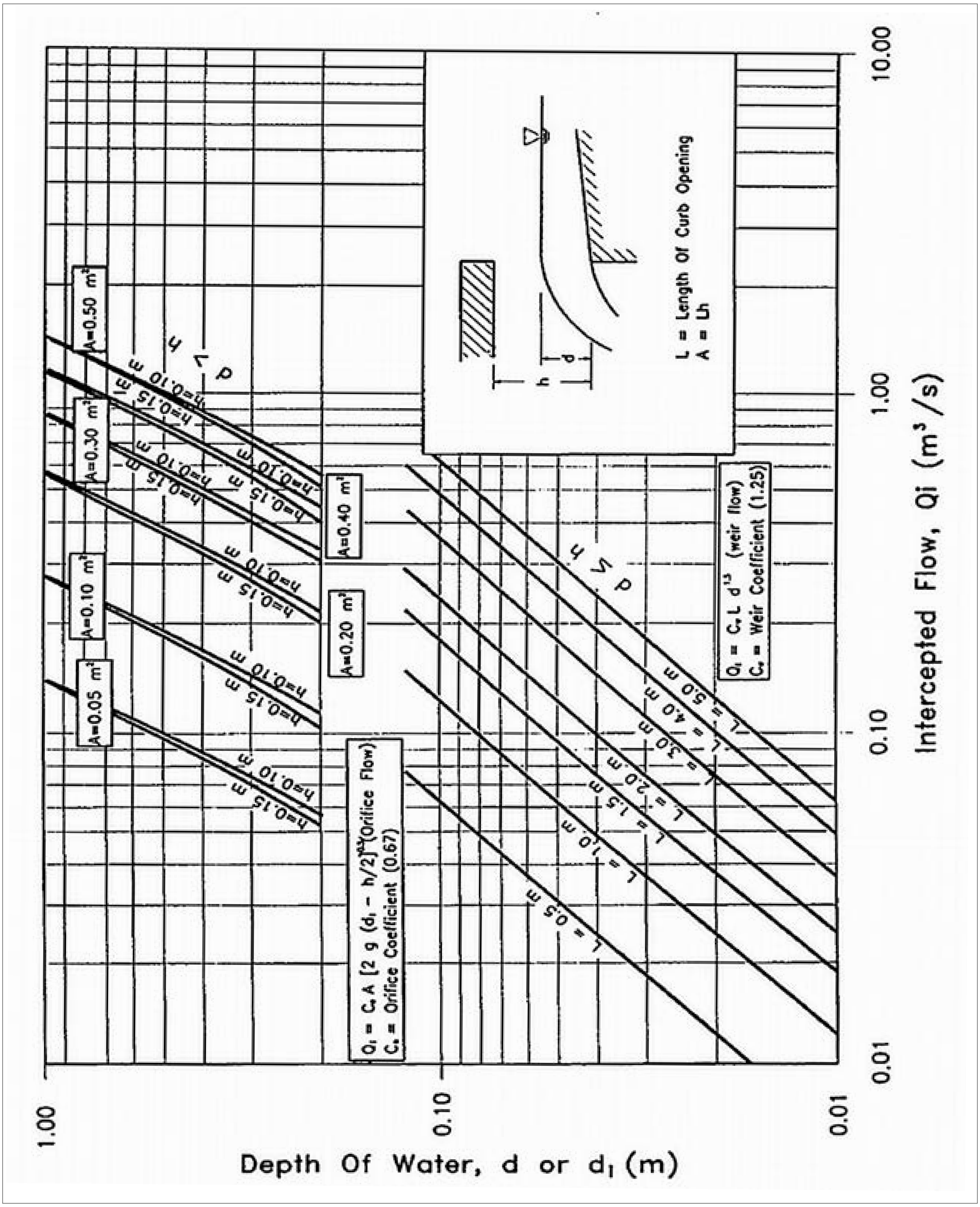


Chart B.24 Kerb and Longitudinal Slotted Drain for Total Interception

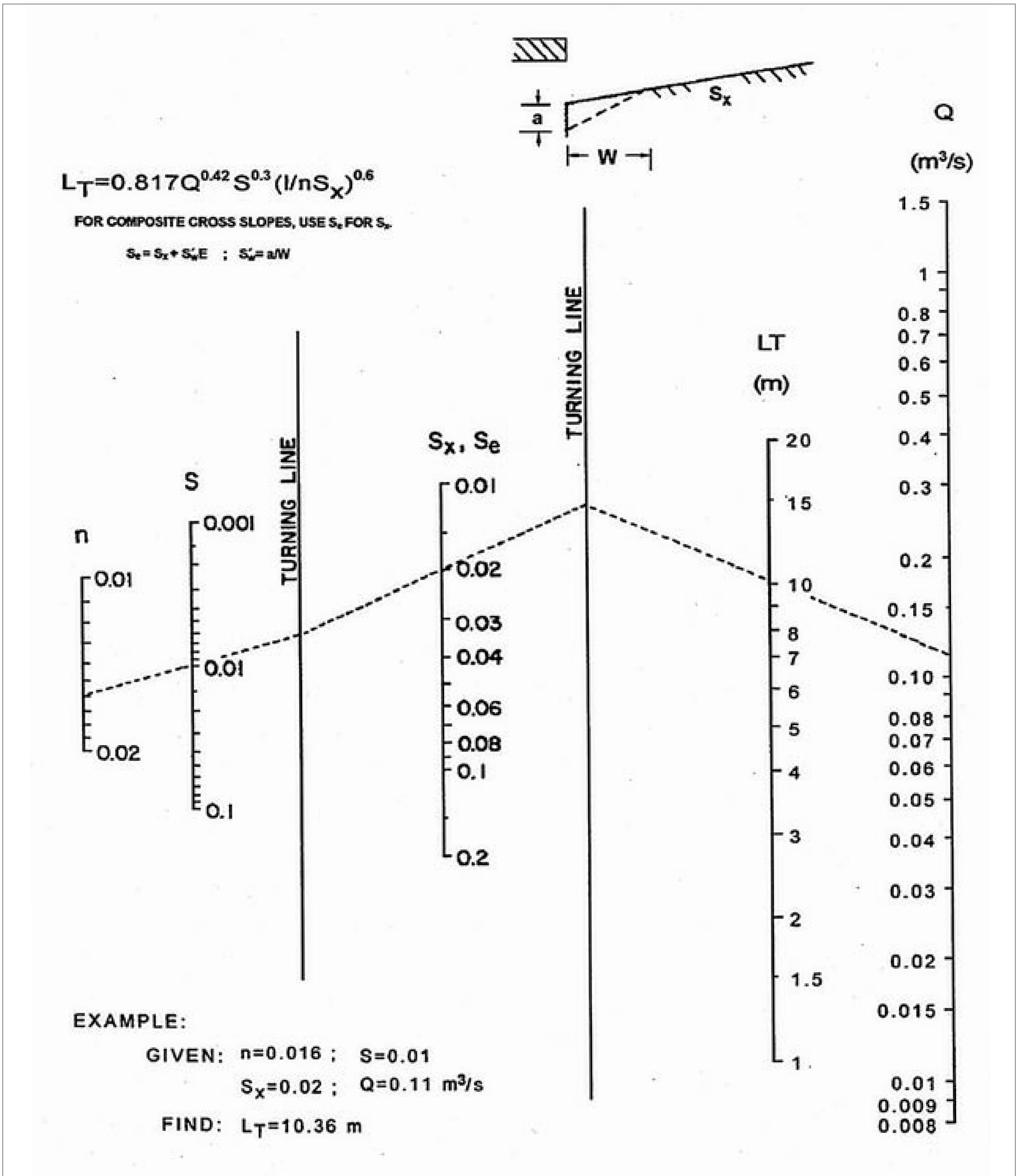


Chart B.25 Velocity in Triangular Gutters

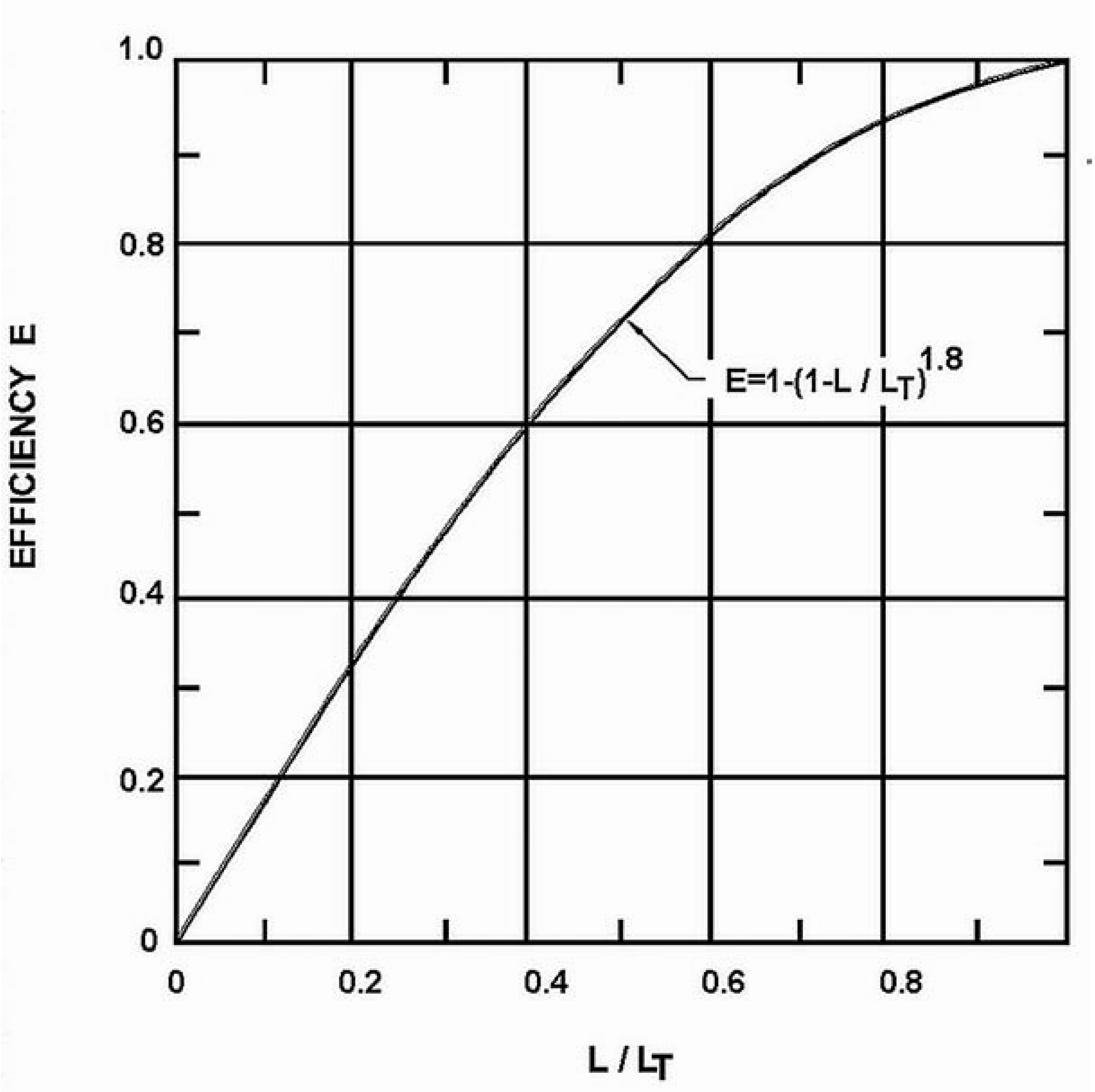


Chart B.26 Depressed Kerb Opening Inlet Capacity in Sump Condition

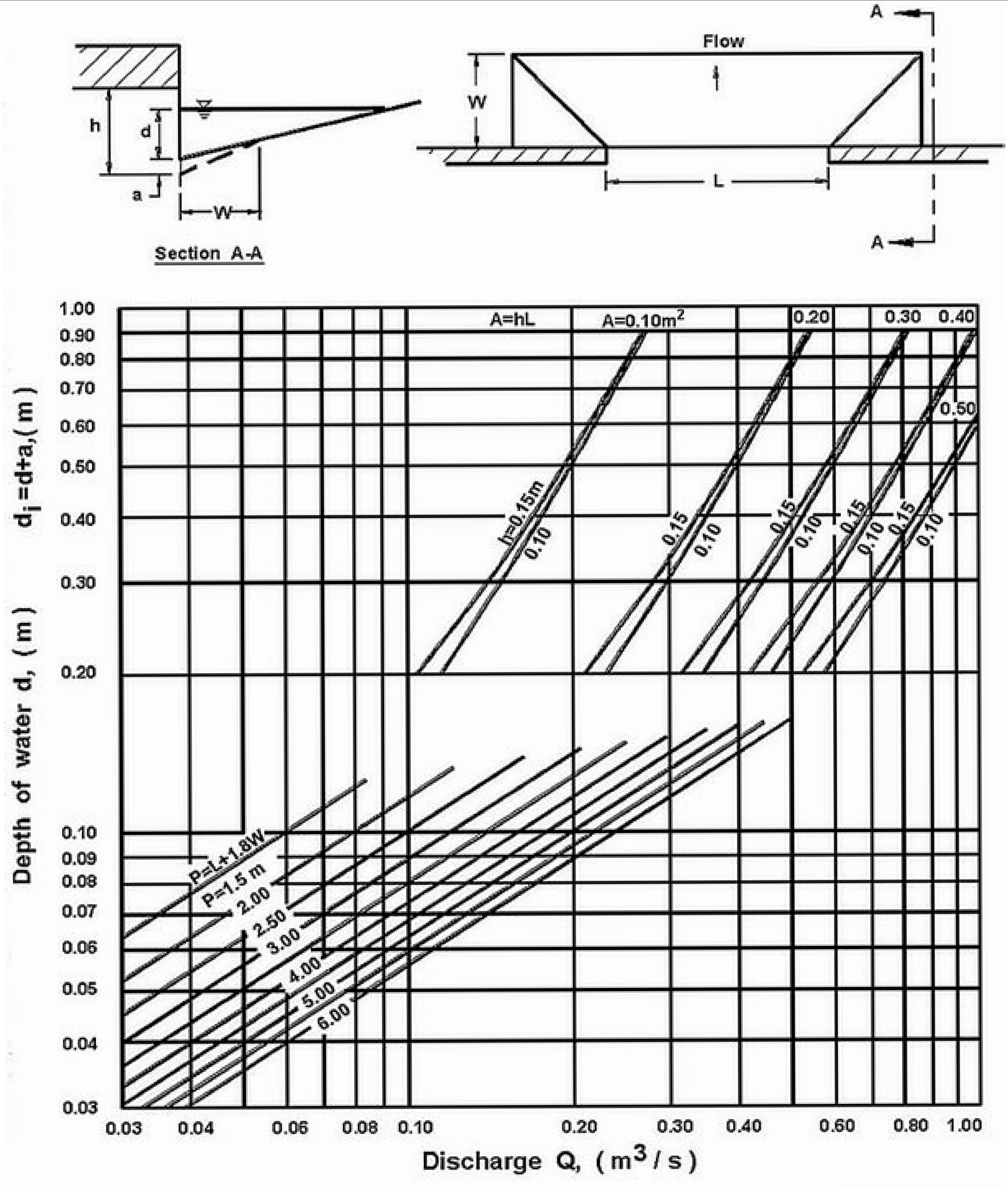


Chart B.27 Kerb Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats

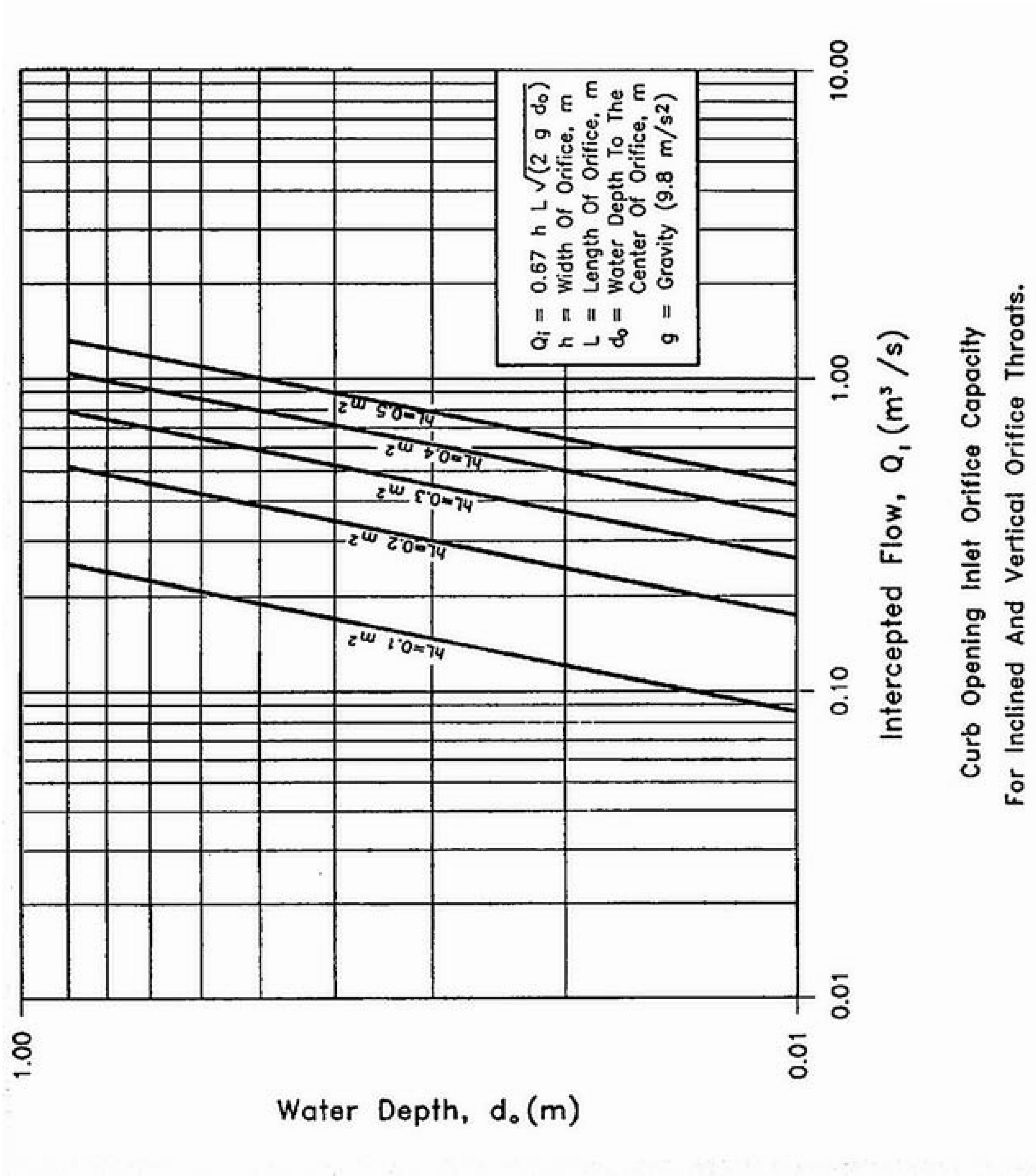


Chart B.28 Slotted Drain Inlet Capacity in Sump Locations

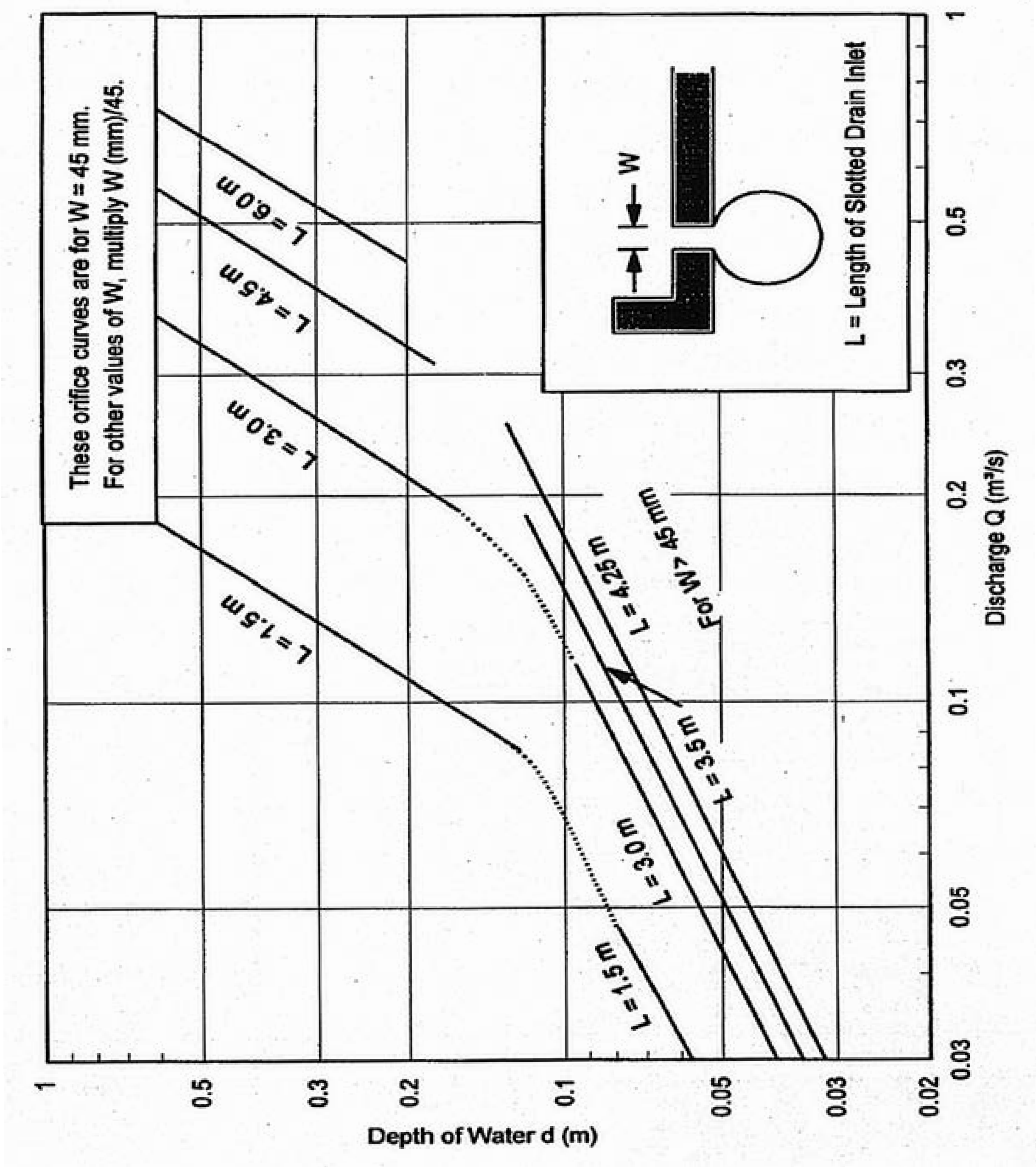


Chart B.29 Grate Inlet Frontal Flow Interception Efficiency

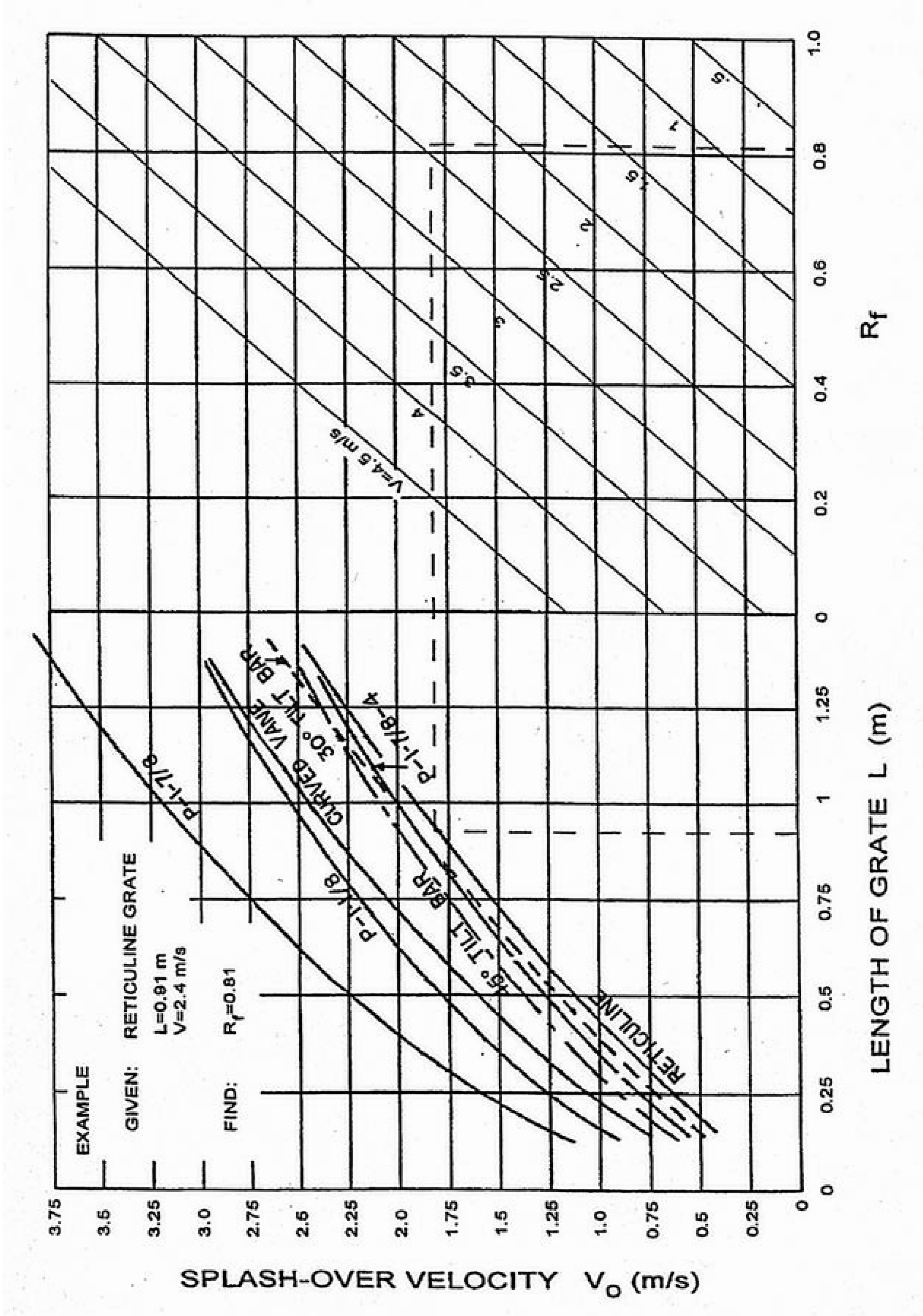
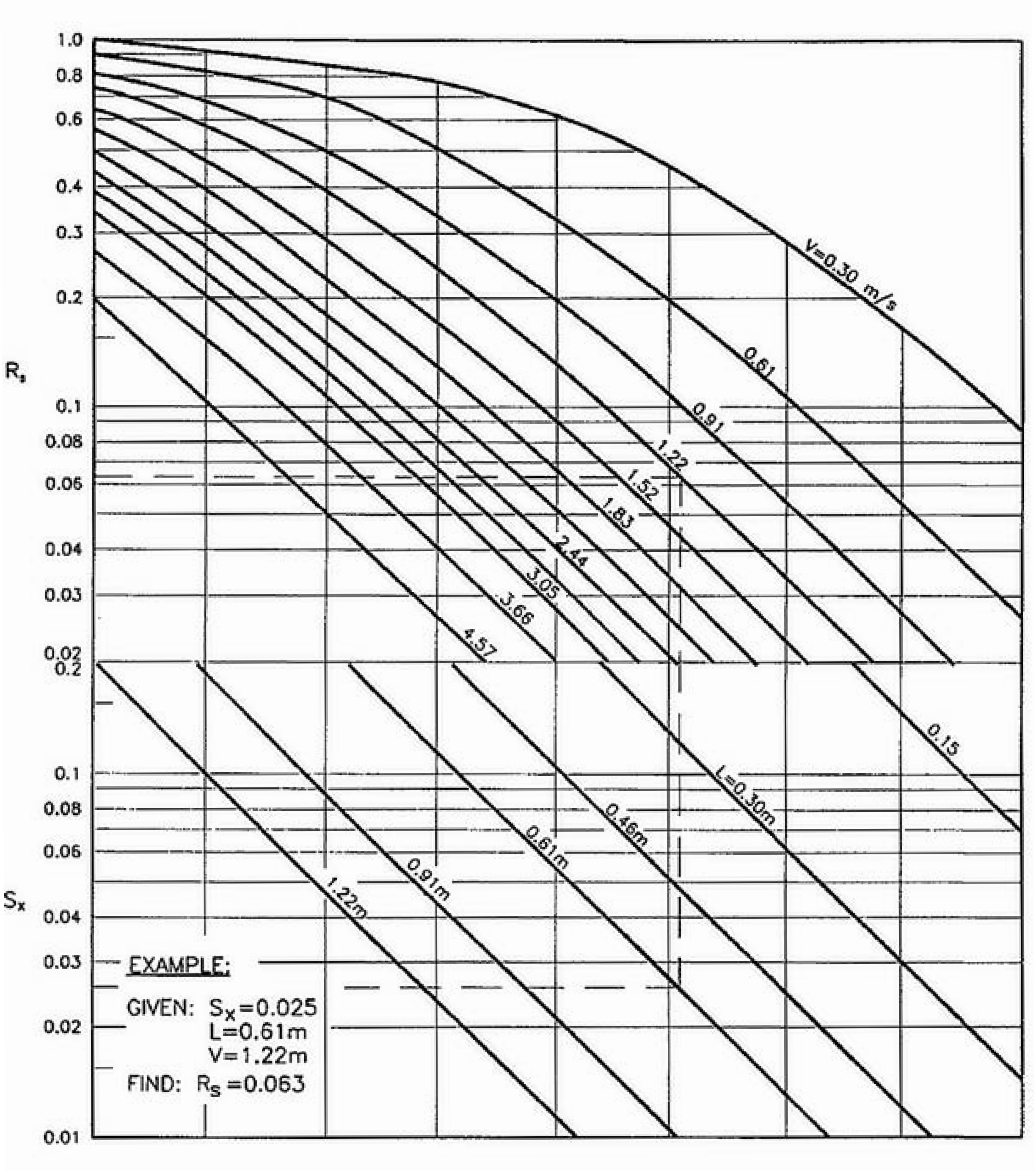


Chart B.30 Grate Side Interception Efficiency



Appendices

Appendix C: Detention Storage

C.1 Introduction

C.1.1 Overview

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanise this type of design may result in major drainage and flooding problems downstream. Under favourable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This chapter provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations. Storage and flood routing associated with culverts is addressed in Chapter 4: Culverts (note: criteria in this chapter does not necessarily apply to routine culvert design).

C.1.2 Location Considerations

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system which could decrease or increase flood peaks in different downstream locations. Thus it is important for the designer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a regional or basin-wide planning basis.

C.1.3 Detention And Retention

Stormwater storage facilities are often referred to as either detention or retention facilities. For the purposes of this chapter, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this Manual. Retention facilities are designed to contain a permanent pool of water. Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities these will be specified.

C.1.4 Computer Programs

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations there are many available reservoir routing computer programs. Also, the storage indicator method can be used which makes calculations simple. All storage facilities shall be designed and analysed using reservoir routing calculations.

C.2 Uses

C.2.1 Introduction

The use of storage facilities for stormwater management has increased dramatically in recent years. The benefits of storage facilities can be divided into two major control categories of quality and quantity.

C.2.2 Quality

Control of stormwater quality using storage facilities offers the following potential benefits:

- Decrease downstream channel erosion.
- Control of sediment deposition, and
- Improved water quality through stormwater filtration (wet ponds only).

C.2.3 Quantity

Controlling the quantity of stormwater using storage facilities can provide the following potential benefits prevention or reduction of peak runoff rate increases caused by urban development.

- Mitigation of downstream drainage capacity problems.
- Recharge of groundwater resources.
- Reduction or elimination of the need for downstream outfall improvements, and
- Maintenance of historic low flow rates by controlled discharge from storage.

C.2.4 Objectives

The objectives for managing stormwater quantity by storage facilities are typically based on limiting peak runoff rates to match one or more of the following values:

- Historic rates for specific design conditions (i.e., post-development peak equals pre- development peak for a particular frequency of occurrence).
- Non-hazardous discharge capacity of the downstream drainage system, and
- A specified value for allowable discharge set by a regulatory jurisdiction.

For a catchment area with no positive outfall, the total volume of runoff is critical and retention storage facilities are used to store the increases in volume and control discharge rates.

C.3 Symbols And Definitions

To provide consistency within this chapter as well as throughout this Manual, the following symbols will be used. These symbols were selected because of their wide use in technical publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

Table C.1 Symbols and Definitions

A	Cross sectional or surface area	m^2
C	Weir coefficient	-
D	Change in elevation	m
D	Depth of basin or diameter of pipe	m
T	Routing time period	sec
G	Acceleration due to gravity	m/s^2
H	Head on structure	m
H_C	Height of weir crest above channel bottom	m
I	Inflow rate	m^3/s
L	Length	m
Q	Flow or outflow rate storage volume	m^3/s
S, V_s		m^3

t_b	Time base on hydrograph	Hrs
TI	Duration of basin inflow	Hrs
t_p	Time to peak	Hrs
W	Width of basin	M
z	Side slope factor	-

C.4 Design Criteria

C. 4.1 General Criteria

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. Possible dispersed or on-site storage may be developed in depressed areas in parking lots, road embankments and highway interchanges, parks and other recreation areas, and small lakes, ponds and depressions within urban developments. The utility of any storage facility depends on the amount of storage. its location within the system, and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility shall be included in the analysis (i.e., 100-year flood). The design criteria for storage facilities should include:

- Release rate,
- Storage volume,
- Grading and depth requirements,
- Outlet works, and
- Location.

C.4.2 Release Rate

Control structure release rates should approximate the peak runoff rates prior to the installation of the detention storage for the 2-year through 10-year storms, with emergency overflow capable of handling the 100-year discharge. Design calculations are required to demonstrate that runoff from the 2- and 10-year design storms are controlled. If so, runoff from intermediate storm return periods can be assumed to be adequately controlled. Multi-stage control structures may be required to control both runoff from the 2- and 10-year storms.

C.4.3 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates and volumes to pre-developed discharge rates for the 2-year through 10-year storms, or other design storms depending on what the design capacity of the downstream system. Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project. For detention basins, all detention volume shall be drained within the average period between storm events (12h).

C.4.4 Grading And Depth

Following is a discussion of the general grading and depth criteria for storage facilities followed by criteria related to detention and retention facilities.

1. General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments shall be less than 6 meters in height and should have side slopes no steeper than 3:1 (horizontal to vertical). Riprap-protected embankments shall be no steeper than 2:1. Geotechnical slope stability analysis is recommended for embankments greater than 3 meters in height and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soil engineering textbooks (54.55).

A minimum freeboard of 0.3 meters above the 100-year design storm high water elevation shall be provided for impoundment depths of up to 6 meters.

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, crop value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements, and required freeboard. Aesthetically pleasing features are also important in urbanising areas.

2. Detention

Areas above the normal high water elevations of storage facilities shall be sloped at a minimum of 5 percent toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities shall be graded toward the outlet to prevent standing water conditions. A minimum 2 percent bottom slope is recommended. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and prevent standing water conditions.

3. Retention

The maximum depth of permanent storage facilities will be determined by site conditions, design constraints, and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions) shall be considered. A depth of 2-2.5 meters is generally reasonable. Where aquatic habitat is required the cognisant wildlife experts shall be contacted for site-specific criteria relating to such things as depth. habitat, and bottom and shore geometry.

C.4.5 Outlet Works

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow, and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of drop inlets, pipes, weirs, and orifices. Slotted riser pipes are discouraged because of clogging problems. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet. For large storage facilities, selecting a flood magnitude for sizing the emergency outlet shall be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency outlet is the 100-year flood. The sizing of a particular outlet works shall be based on results of hydrologic routing calculations.

C.4.6 Location

In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. For all storage facilities, channel routing calculations shall proceed downstream to a confluence point where the drainage area being analysed represents twenty percent of the total drainage area. At this point the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph shall be assessed and shown not to have detrimental effects on downstream areas.

C.5 Dams

C.5.1 Classification

Dams are classified as either new or existing, by hazard potential, and by size. Hazard potential categories are listed below:

Category 1 dams are located where failure would probably result in:

- Loss of human life,
- Excessive economic loss due to damage of downstream properties,
- Public hazard, or
- Public inconvenience due to loss of impoundment and/or damage to roads or any public or private utilities.

Category 2 dams are located where failure may damage downstream private or public property, but such damage would be relatively minor and within the general financial capabilities of the dam owner. Public hazard or inconvenience due to loss of roads or any public or private utilities would be minor and of short duration. Chances of loss of human life would be possible but remote.

Category 3 dams are located where failure may damage uninhabitable structures or land but such damage would probably be confined to the dam owner's property. No loss of human life would be expected.

Table C.2 Size Categories for Dams in Kenya

Category	Storage (m^3 x 1000)	Height (m)
Small	30 to < 5,000	8 to < 12
Intermediate	5,000 to 100,000	12 to 50
Large	> 100,000	> 50

C.6 General Design Procedure

C.6.1 Data Needs

The following data will be needed to complete storage design and routing calculations.

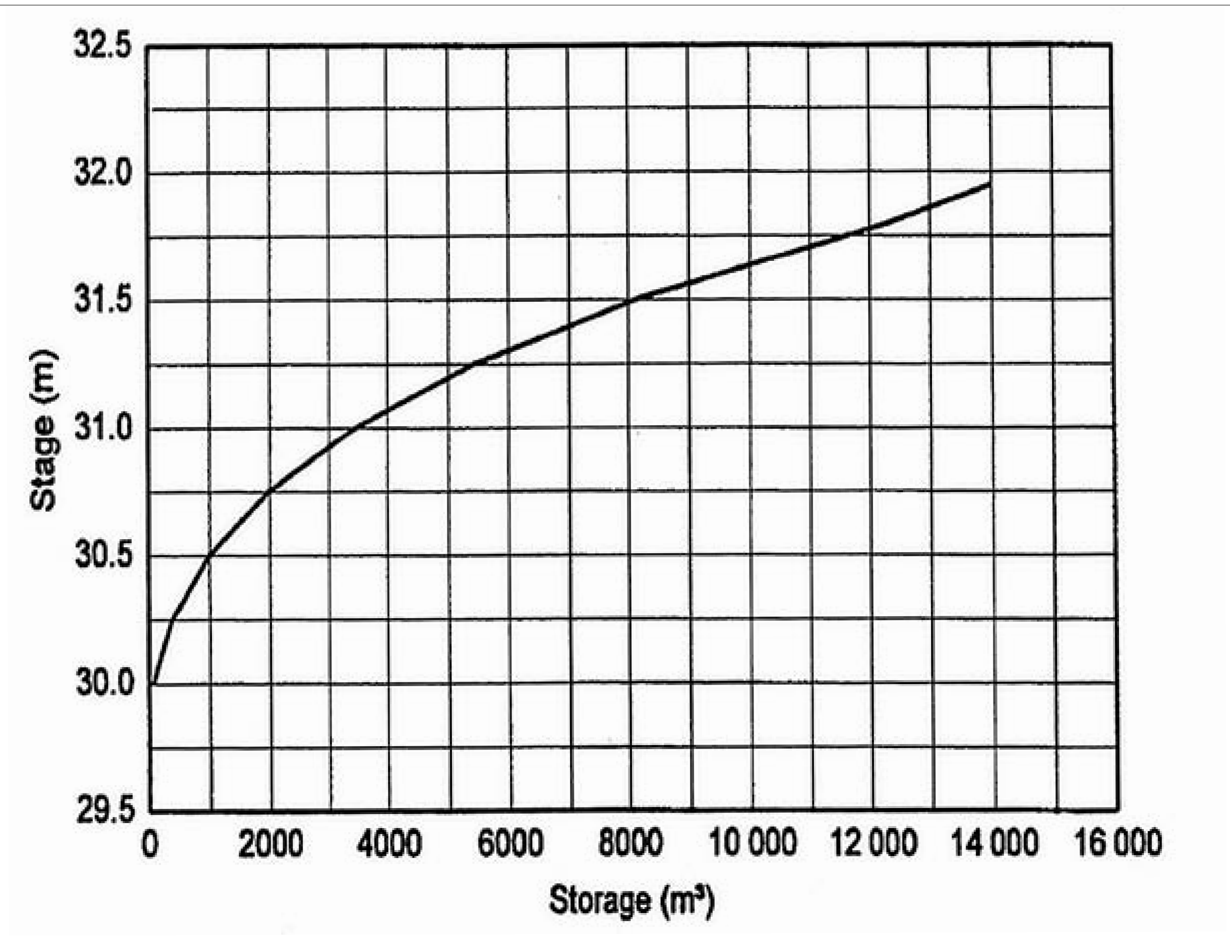
- Inflow hydrograph for all selected design storms,
- Stage-storage curve for proposed storage facility (see Figure C.1 below for an example),
- Stage-discharge curve for all outlet control structures (see Figure C.2 below for an example).

Using these data a design procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet geometry until the desired outflow hydrograph is achieved (see example C.9).

C.6.2 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir.

Figure C.1 Example Stage-Storage Curve for Storage Facility



The data for this type of curve are usually developed using a topographic map and the double-end area frustum of a pyramid or prismoidal formula.

The double-end area formula is expressed as:

$$V_{1,2} = \frac{A_1 + A_2}{2} d$$

Equation C.1

Where,

$V_{1,2}$ = Storage volume, m^3, between elevations 1 and 2.

A_1 = Surface area at elevation 1, m^2.

A_2 = Surface area at elevation 2, m^2.

d = Change in elevation between points 1 and 2, m.

The frustum of a pyramid is expressed as:

$$V = \frac{d}{3} [A_1 + (A_1 A_2)^{0.5} + A_2]$$

Equation C.2

Where,

V = Volume of frustum of a pyramid, m^3.

d = Change in elevation between points 1 and 2, m.

A_1 = Surface area at elevation 1, m^2.

A_2 = Surface area at elevation 2, m^2.

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)\,ZD^2 + \frac{4}{3}\,Z^2\,D^3$$

Equation C.3

Where,

V = Volume of trapezoidal basin, m^3.

L = Length of basin at base, m.

W = Width of basin at base, m.

D = Depth of basin, m.

Z = Side slope factor, ratio of vertical to horizontal.

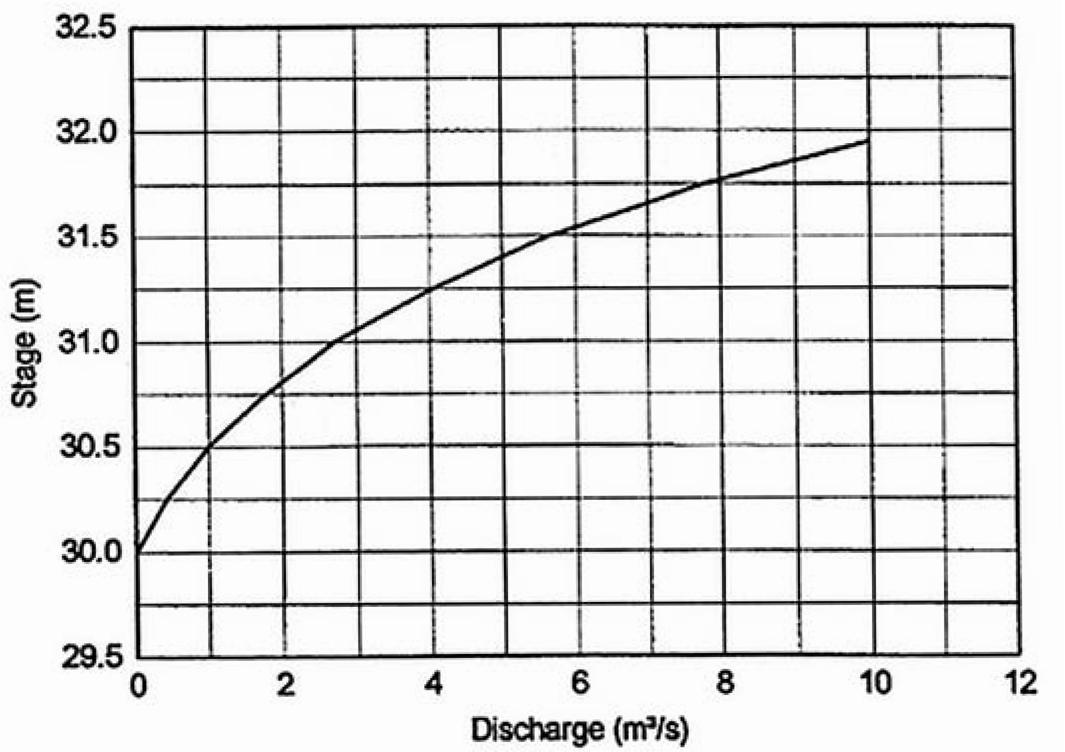
C.6.3 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two spillways: principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway shall be designed taking into account the potential threat to downstream life and property if the storage facility were to fail.

The stage-discharge curve should take into account the discharge characteristics of both the principal and emergency spillways.

Figure C.2 Example of Stage Discharge Curve



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C.6.4 Procedure

A general procedure for using the above data in the design of storage facilities is presented below. See also Figure C.12 at the end of this section for a graphical presentation of the procedure.

Step 1

Compute inflow hydrograph for runoff from the 2-, 10-, and 100-year design storms using the procedures outlined in Part 1 of this Manual. Both pre- and post-development hydrographs are required for the 2- and 10-year design storms. Only the post- development hydrograph is required for runoff from the 100-year design storm.

Step 2

Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1. If storage requirements are satisfied for runoff from the 2- and 10-year design storms, runoff from intermediate storms is assumed to be controlled.

Step 3

Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 shall be used.

Step 4

Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure shall be sized to convey the allowable discharge at this stage.

Step 5

Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If the routed post-development peak discharges from the 2- and 10-year design storms exceed the pre-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to step 3.

Step 6

Consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.

Step 7

Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility shall be routed though the downstream channel system until a confluence point is reached where the drainage area being analysed represents twenty percent of the total drainage area.

Step 8

Evaluate the control structure outlet velocity and provide channel and bank stabilisation if the velocity will cause erosion problems downstream.

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

C.7 Outlet Hydraulics

C.7.1 Outlets

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad- crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in Chapter 4.3 of this Manual shall be used to develop stage-discharge data. Slotted riser pipe outlet facilities shall be avoided.

C.7.2 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure C.3. The discharge equation for this configuration is (Chow. 1959):

$$Q = [(1.805 + 0.221(\frac{H}{H_c})]\ LH^{1.5}$$

Equation C.4

Where,

Q = Discharge, m³/s.

H = Head above weir crest excluding velocity head, m.

H_c = Height of weir crest above channel bottom, m.

L = Horizontal weir length, m.

Figure C.3 Sharp-Crested Weir

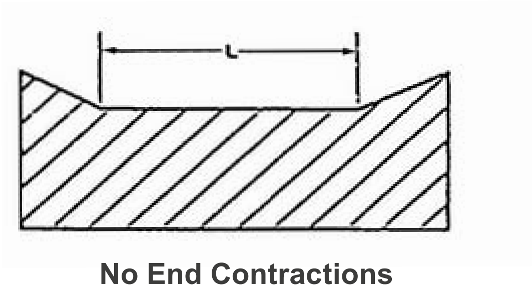
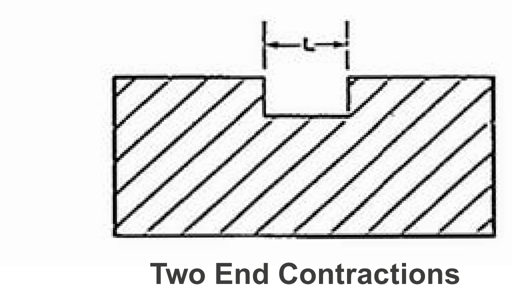


Figure C.4 Sharp-Crested Weir



A sharp-crested weir with two end contractions is illustrated in Figure C.4. The discharge equation for this configuration is (Chow. 1959):

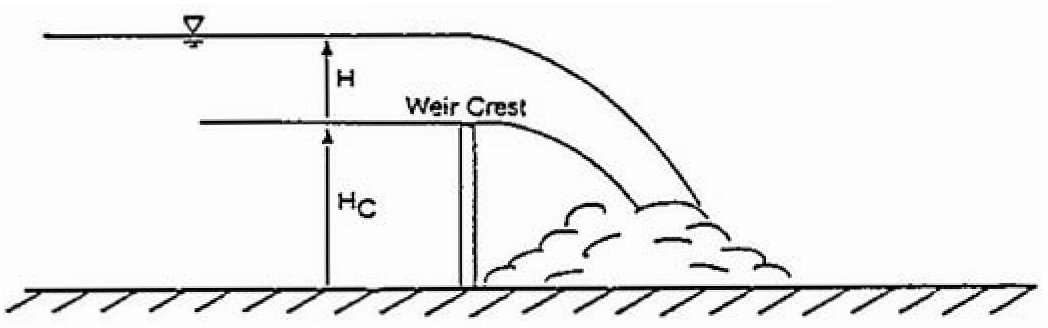
$$Q = [(1.805 + 0.221(\frac{H}{H_c})]\ (L - 0.2H)\ H^{1.5}$$

Equation C.5

Where,

Variables are the same as Equation C.4.

Figure C.5 Sharp-Crested Weir and Head



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A sharp-crested weir will be affected by submergence when the tailwater rim above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King. 1976):

$$Q_s = Q_f\left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385}$$

Equation C.5

Where,

Q_s = Submergence flow, m³/s.
Q_f = Free flow, m³/s.
H_1 = Upstream head above crest, m.
H_2 = Downstream head above crest, m.

C.7.3 Broad-Crested Weirs

The equation generally used for the broad-crested weir is (Brater and King. 1976):

$$Q = CLH^{1.5}$$

Equation C.6

Where,

Q = Discharge, m³/s.
C = Broad-crested weir coefficient.
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 1.740. For sharp corners on the broad-crested weir, a minimum C value of 1.435 shall be used. Additional information on C values as a function of weir crest breadth and head is given in Table C.3.

C.7.4 V-Notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King. 1976).

$$Q = 1.38\ tan\frac{\theta}{2}H^{2.5}$$

Equation C.7

Where,

Q = Discharge, m³/s.
θ = Angle of v-notch, degrees.
H = Head on apex of notch, m.

C.7.5 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary non-linearly with head.

Table C.3 Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head Weir Crest Breadth (m)

(m)	0.15	0.23	0.30	0.46	0.61	0.76	0.91	1.22	1.52	3.05	4.57
0.06	1.55	1.52	1.49	1.45	1.40	1.37	1.35	1.31	1.29	1.37	1.48
0.12	1.61	1.55	1.50	1.46	1.44	1.44	1.42	1.40	1.38	1.41	1.49
0.18	1.70	1.60	1.52	1.46	1.44	1.44	1.48	1.49	1.49	1.49	1.49
0.24	1.82	1.68	1.57	1.48	1.44	1.44	1.47	1.48	1.48	1.49	1.46
0.30	1.83	1.73	1.65	1.52	1.47	1.46	1.46	1.47	1.48	1.48	1.45
0.37	1.83	1.77	1.70	1.58	1.49	1.46	1.46	1.47	1.47	1.49	1.46
0.43	1.83	1.80	1.77	1.61	1.53	1.48	1.46	1.46	1.46	1.47	1.46
0.49	1.83	1.82	1.81	1.69	1.59	1.52	1.48	1.47	1.46	1.46	1.45
0.55	1.83	1.83	1.83	1.69	1.60	1.52	1.48	1.47	1.46	1.46	1.45
0.61	1.83	1.83	1.83	1.69	1.64	1.52	1.55	1.48	1.46	1.46	1.45
0.76	1.83	1.83	1.83	1.81	1.69	1.60	1.55	1.50	1.47	1.46	1.45
0.91	1.83	1.83	1.83	1.83	1.77	1.68	1.60	1.51	1.47	1.46	1.45
1.07	1.83	1.83	1.83	1.83	1.83	1.76	1.68	1.52	1.48	1.46	1.45
1.22	1.83	1.83	1.83	1.83	1.83	1.83	1.76	1.54	1.49	1.46	1.45
1.37	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.59	1.51	1.46	1.45
1.52	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.69	1.54	1.46	1.45
1.68	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.59	1.46	1.45

[1]Measured at least 2.5H upstream of the weir. Reference: Brater and King (1976).

Design equations for proportional weirs are (Sandvik. 1985):

$$Q = 2.74\, a^{0.5}\, b(H - \frac{a}{3})$$

Equation C.9

$$\frac{x}{b} = 1 - \frac{1}{3.17}\left(arctan\left(\frac{y}{a}\right)^{0.5}\right)$$

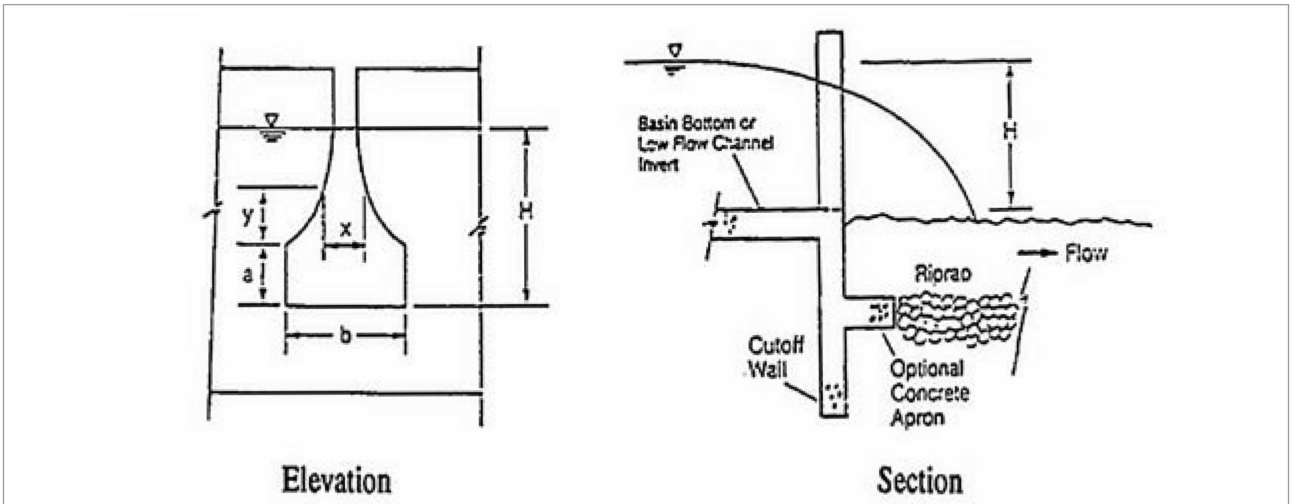
Equation C.10

Where,

Q = discharge, m³/s

Dimensions a, b, h, x, and y are shown below in mm.

Figure C.6 Proportional Weir Dimensions



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C.7.6 Orifices

Pipes smaller than 300 mm may be analysed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions.

$$Q = 0.6A(2gH)^{0.5}$$

Equation C.11

Where,

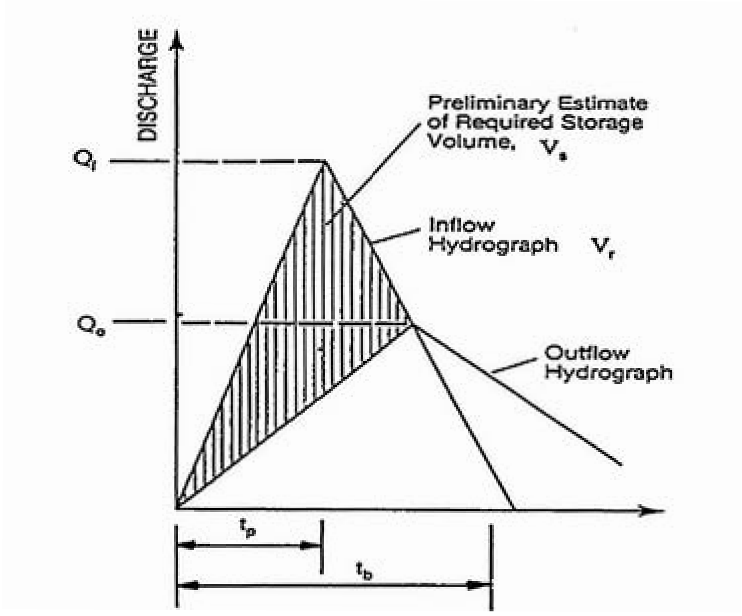
Q = Discharge, m^3/s.
A = Cross-section area of pipe, m^2.
g = Acceleration due to gravity, 9.81 m/s^2.
D = Diameter of pipe, m.
H = Head on pipe, from the center of pipe to the water surface, m.

C.8 Preliminary Detention Calculations

C.8.1 Storage Volume

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 the standard triangular shapes shown in Figure C.7 shown below.

Figure C.7 Triangular Shaped Hydrographs *(For Preliminary Estimate of Required Storage Volume)*



The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_s = 0.5T_i(Q_i - Q_o)$$

Equation C.12

Where,

V_s = Storage volume estimate, m^3.
Q_i = Peak inflow rate, m^3/s.
Q_o = Peak outflow rate, m^3/s.
T_i = Duration of basin inflow, sec.

Any consistent units may be used for Equation C.12.

C.8.2 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff & Singh. 1986).

1. Determine input data, including the allowable peak outflow rate, Q_o, the peak flow rate of the inflow hydrograph, Q_i,- the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.
2. Calculate a preliminary estimate of the ratio V_s/V_r using the input data from Step 1 and the following equation:

$$\frac{V_s}{V_r}\left[1.291\left(1 - \frac{Q_o}{Q_i}\right)^{0.753}\right] / \left(\frac{t_b}{t_p}\right)^{0.411}]$$

Equation C.13

Where,

V_s = volume of storage, m³.

V_r = volume of runoff, m³.

Q_o = outflow peak flow, m³/s.

Q_I = inflow peak flow, m³/s.

t_b = time base of the inflow hydrograph, h (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak).

t_p = time to peak of the inflow hydrograph, h.

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated in Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

C.8.3 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

1. Determine the following:
   - Volume of Determine the following:runoff, V_r.
   - Peak flow rate of the inflow hydrograph, Q_i.
   - Time base of the inflow hydrograph, t_b.
   - Time to peak of the inflow hydrograph, t_p.
   - Storage volume, V_s.
2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Singh. 1976):

$$\frac{Q_o}{Q_i} = 1 - 0.712\left(\frac{V_s}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546}$$

Equation C.14

Where,

Q_o = Outflow peak flow, m³/s.

Q_i = Inflow peak flow, m³/s.

V_s = Volume of storage, m³.

V_r = Volume of runoff, m³.

t_b = Time base of the inflow hydrograph, h (Determined as the time).

t_p = From the beginning of rise to a point on the recession limb, (where the flow is 5 percent of the peak.) time to peak of the inflow hydrograph, in hours

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume (see third example in C.10).

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C.9 Routing Calculations

The following procedure is used to perform routing through a reservoir or storage facility (Puls Method of storage routing). A flowchart presenting this procedure can be found at the end of this section (Flowchart C.2)

Step 1

Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown in Figures C.1 and C.2.

Step 2

Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

Step 3

Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S + (O/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Table C.4.

Table C.4 Storage Characteristics

1. Stage (m)	2. Storage[1] (m³)	3. Discharge[2] (m³/s)	4. S-(O/2)Δt (m³)	5. S+(O/2)Δt (m³)
30.00	62	0	62	62
30.25	370	15	244	496
30.50	987	35	686	1288
30.75	1974	63	1439	2509
31.00	3454	95	2645	4263
31.25	5427	143	4210	6644
31.50	8141	200	6440	9842
31.75	12355	275	9990	14680

[1] Obtained from the Stage-Storage Curve. [2] Obtained from the Stage-Discharge Curve. ***Note:*** *t = 10 minutes = 0.167 hours*

Step 4

For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, S_1-$(O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (example in Figure C.7).

Step 5

Determine the value of $S_2 + (O_2/2)\Delta t$ from the following equation:

$$S_2 + \left(\frac{O_2}{2}\right)\Delta t = [S_1 - (\frac{O_1}{2})\ \Delta t] + [\frac{I_1 + I_2}{2}\Delta t]$$

Equation C.15

Where,

S_2 = Storage volume at time 2 m³.

O_2 = Outflow rate at time 2 m³/s.

Δt = Routing time period. sec.

S_1 = Storage volume at time 1 m³/s.

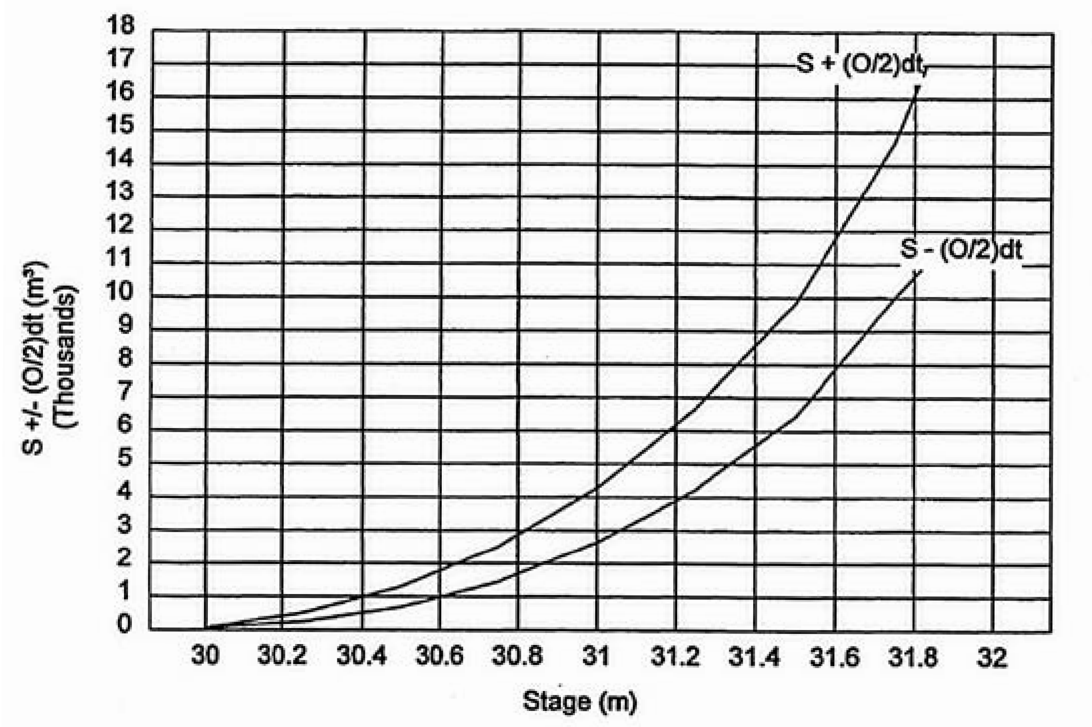
O_1 = Outflow rate at time 1 m³/s.

I_1 = Inflow rate at time 1 m³/s.

I_2 = Inflow rate at time 2 m³/s.

Other consistent units are equally appropriate. Step 6

Figure C.8 Storage Characteristic Curve



Step 6

Enter the storage characteristics curve at the calculated value of S_2 + (O_2/2)Δt determined in Step 5 and read off a new depth of water, H_2.

Step 7

Determine the value of O_2, which corresponds to a stage of $H2$ determined in Step 6, using the stage-discharge curve.

Step 8

Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1, and H_1 equal to the previous I_2, O_2, S_2, and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

C.10 Example Problem

C.10.1 Example

This example demonstrates the application of the methodology presented in this chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions are assumed to have been developed using hydrologic methods covered in Part 1 of this Manual.

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C.10.2 Design Discharge and Hydrographs

Storage facilities shall be designed for runoff from both the 2- and 10-year design storms and an analysis done using the 100-year design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures. Example peak discharges from the 2- and 10-year design storm events are as follows:

- Pre-developed 2-year peak discharge = 4.25 m^3/s.
- Pre-developed 10-year peak discharge = 5.66 m^3/s.
- Post-development 2-year peak discharge = 5.38 m^3/s.
- Post-development 10-year peak discharge = 7.08 m^3/s.

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 4.25 and 5.66 m^3/s for the 2 and 10-year storms, respectively.

Example runoff hydrographs are shown in Table C.5 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 2- and 10-year storms.

Table C.5 Example Runoff Hydrographs

1. Time (hours)	2. 2-year (m^3/s)	3. 10-year (m^3/s)	4. 2-year (m^3/s)	5. 10-year (m^3/s)
0	0	0	0	0
0.1	0.51	0.68	1.08	1.42
0.2	1.73	2.29	3.54	5.04
0.3	3.6	4.81	5.38 > 4.25	7.08 > 5.66
0.4	4.25	5.66	3.54	4.67
0.5	3.17	4.25	1.98	2.55
0.6	2.01	2.69	1.1	1.42
0.7	1.27	1.73	0.62	0.82
0.8	0.85	1.13	0.34	0.45
0.9	0.59	0.79	0.2	0.25
1.0	0.37	0.51	0.11	0.14
1.1	0.28	0.42	0.06	0.08
1.2	0.23	0.37	0	0.03

C.10.3 Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in Section C.8. For runoff from the 2- and 10-year storms, the required storage volumes, V_s, are computed using Equation C.12:

$$V_s = 0.5T_i\,(Q_i - Q_0)$$

2-year storm: V_s = 0.5(1.2 x 3.600)(5.38 – 4.25) = 2441 m^3.

10-year storm: V_s = 0.5(1.25 x 3.600)(7.08 – 5.66) = 3195 m^3.

C.10.4 Design And Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from both the 2- and 10-year design storms are presented in Tables C.7 and C.8. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 2-and 10-year design storms to be provided when the corresponding allowable peak discharges occurred. Storage values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 1.71, a weir length of 1.22 m, and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

Table C.6 Stage-Discharge-Storage Data

1. Stage (m)	2. Q (m^3/s)	3. S (m^3)	4. S_1-$(O/2)\Delta t$ (m^3)	5. S_1+$(O/2)\Delta t$ (m^3)
0.00	0	0	0.0	0.0
0.27	0.3	320	266	1093
0.46	0.6	520	403	2360
0.57	0.9	690	528	3266
0.65	1.1	850	652	3993
0.77	1.4	1000	748	5077
0.87	1.7	1150	844	6162
0.97	2.0	1295	935	7247
1.07	2.3	1445	1031	8332
1.17	2.6	1580	1103	9595
1.25	2.9	1725	1203	10500
1.38	3.4	2010	1398	12310
1.47	3.7	2160	1494	13395
1.54	4.0	2310	1590	14480
1.61	4.2	2441	1676	15384
1.69	4.6	2590	1762	16648
1.77	4.9	2740	1858	17733
1.81	5.1	2885	1967	18458
1.95	5.7	3195	2176	20485
2.07	6.2	3490	2374	22439
2.13	6.5	3640	2470	23524
2.25	7.0	3960	2700	25335

Table C.7 Storage Routing for the 2-Year Storm

1. Time (hrs)	2. Inflow (m^3/s)	3. I_1+I_2)/2 (m^3)	4. H_1 (m^3)	5. S_1-$(O_1/2)\Delta t$ (m^3)	6. S_2+$(O_2/2)\Delta t$ (m^3) 3. + 5.	7. H_2 (m)	8. Outflow (m^3/s)
0.0	0.00	0.0	0.00	0.0	0.0	0.00	0.00
0.1	1.08	194.4	0.00	0.0	194.4	0.10	0.08
0.2	3.54	831.6	0.13	123.3	954.9	0.50	1.02
0.3	5.38	1605.6	0.62	616.7	2222.3	0.99	2.80
0.4	3.54	1605.6	1.22	1221.2	2826.8	1.21	3.68 < 4.25 OK
0.5	1.98	993.6	1.46	1492.5	2486.1	1.12	3.23
0.6	1.10	554.4	1.34	1381.5	1935.9	0.87	2.41
0.7	0.62	309.6	1.10	1073.1	1382.7	0.65	1.56
0.8	0.34	172.8	0.82	801.8	974.6	0.50	1.05
0.9	0.20	97.2	0.63	616.7	713.9	0.42	0.76
1.0	0.11	55.8	0.52	518.1	573.9	0.32	0.51
1.1	0.06	30.6	0.40	394.7	425.3	0.25	0.34
1.2	0.00	10.8	0.30	308.4	319.2	0.15	0.20
1.3	0.00	0.0	0.21	185.0	185.0	0.12	0.08

Table C.8 Storage Routing for the 10-Year Storm

1. Time (hrs)	2. Inflow (m³/s)	3. I_1+I_2)/2 (m³)	4. H_1 (m³)	5. S_1-(O_1/2)Δt (m³)	6. S_2+(O_2/2)Δt (m³) 3. + 5.	7. H_2 (m)	8. Outflow (m³/s)
0.0	0.00	0.0	0.00	0.0	0.0	0.00	0.00
0.1	1.42	255.6	0.00	0.0	255.6	0.12	0.08
0.2	5.04	1162.8	0.12	98.7	1261.5	0.76	1.39
0.3	7.08	2181.6	0.76	740.1	2921.7	1.49	3.79
0.4	4.67	2115.0	1.49	1554.2	3669.2	1.77	4.90 < 5.66 OK
0.5	2.55	1299.6	1.77	1603.5	2903.1	1.51	3.88
0.6	1.42	714.6	1.51	1541.9	2256.5	1.25	2.92
0.7	0.82	403.2	1.25	1233.5	1636.7	0.94	1.93
0.8	0.45	228.6	0.94	925.1	1153.7	0.73	1.30
0.9	0.25	126.0	0.73	727.8	853.8	0.58	0.91
1.0	0.14	70.2	0.58	542.7	612.9	0.43	0.59
1.1	0.08	39.6	0.43	407.0	446.6	0.37	0.45
1.2	0.03	19.8	0.37	345.4	365.2	0.27	0.31
1.3	0.00	0.0	0.27	271.4	271.4	0.18	0.17

Storage routing was conducted for runoff from both the 2- and 10-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the 'Stage Discharge Storage Data' given on the previous page and the Storage Characteristics are shown below for runoff from the 2- and 10- year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-year design storms.

For the routing calculations the following equation was used:

$$S_2 + \frac{O_2}{2}\Delta t = [S_1 - \frac{O_1}{2}\Delta t] + [\frac{I_1 + I_2}{2})\,\Delta t]$$

Also, column 6 = column 3 + column 5

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased, or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, runoff from the 100-year storm shall be routed through the storage facility to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability and maintenance, grading to prevent standing water, and provisions for public safety.

C.10.5 Downstream Effects

An estimate of the potential downstream effects (i.e., increased peak flow rate and recession time) of detention storage facilities may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown below for the 10-year design storms.

Potential effects on downstream facilities shall be minor when the maximum difference between the recession limbs of the pre-developed and routed outflow hydrographs is less than about 20 percent. As shown in Figure C.9, the example results are well below 20 percent, downstream effects can thus be considered negligible and downstream flood routing omitted.

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C.11 Land-Locked Retention

C.11.1 Introduction

Catchment area areas that drain to a central depression with no positive outlet (ie- some eastern lakes) are typical of many topographic areas including karst topography, and can be evaluated using a mass flow routing procedure to estimate flood elevations. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrogeologic 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.

C.11.2 Mass Routing

The steps presented below for the mass routing procedure are illustrated by the example given in Figure C.10 below and resumed in flow Chart C.2 at the end of this section.

Step 1

Obtain cumulative rainfall data for the 100-year frequency, duration design event from Figure C.11.

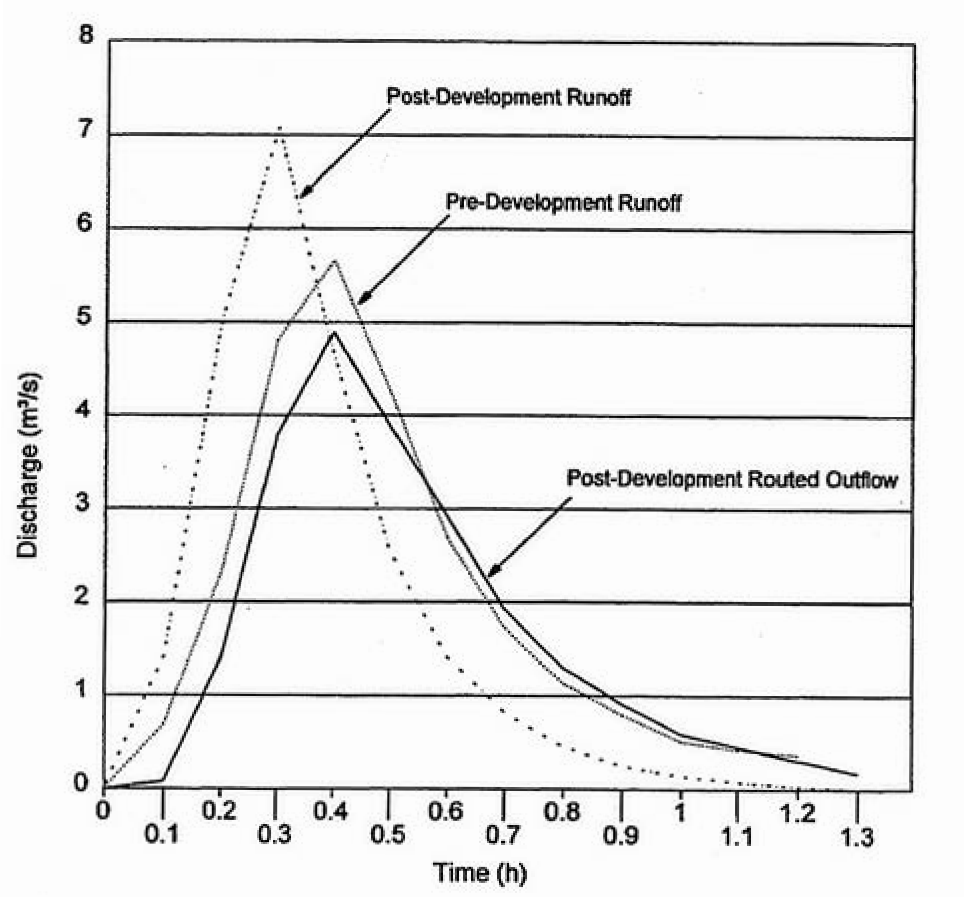
Step 2

Calculate the cumulative inflow to the land-locked retention basin using the rainfall data from Step 1 and runoff procedure from Part 1 of this Manual. Plot the mass inflow to the retention basin.

Step 3

Develop the basin outflow from field measurements of hydraulic conductivity, taking into consideration worst-case water table conditions. Hydraulic conductivity shall be established using in-situ test methods, then results compared to observed performance characteristics of the site. Plot the mass outflow as a straight line with a slope corresponding to worst-case outflow in centimetres/hour.

Figure C.9 Run-Off Hydrographs



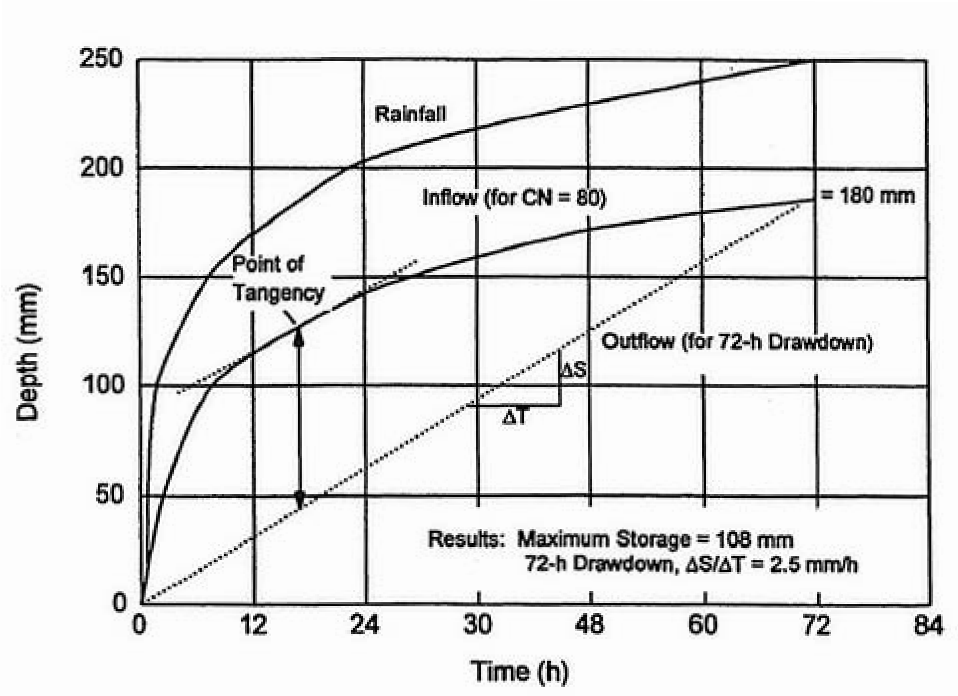
Step 4

Draw a line tangent to the mass inflow curve from Step 2, which has a slope parallel to the mass outflow line from Step 3.

Step 5

Locate the point of tangency between the mass inflow curve of step 2 and the tangent line drawn for Step 4. The distance from this point of tangency and the mass outflow line represents the maximum storage required for the design runoff.

Figure C.10 Mass Routing Curve



Step 6

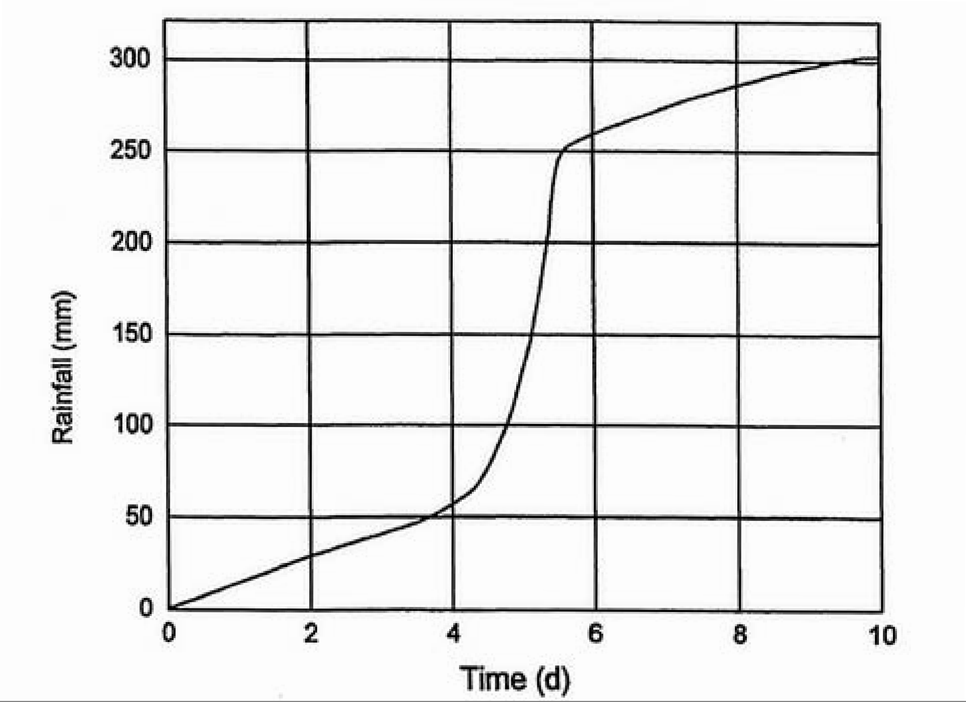
Determine the flood elevation associated with the maximum storage volume determined in Step 5. Use this flood elevation to evaluate flood protection requirements of the project. The zero-volume elevation shall be established as the normal wet season water surface or water table elevation or the pit bottom, whichever is highest.

Step 7

If runoff from the project area discharges into a drainage system tributary to the landlocked depression, detention storage facilities are required to comply with the pre- development discharge requirements for the project.

Unless the storage facility is designed as a retention facility, including water budget calculations, environmental needs and provisions for preventing anaerobic conditions, relief structures shall be provided to prevent standing water conditions.

Figure C.11 Cumulative Rainfall Data for 100-Year 10-d Design Storm



C.12 Retention Storage Facilities

C.12.1 Introduction

The use of retention storage facilities which have a permanent pool (wet ponds), is often discouraged because of the extensive maintenance that is sometimes required. Provisions for weed control and aeration for prevention of anaerobic conditions shall be considered. Also, facilities should not be built that have the potential for becoming nuisances or health hazards. Note, wet ponds are required where water quality problems are to be addressed.

C.12.2 Water Budget

Water budget calculations are required for all permanent pool facilities and should consider performance for average annual conditions. The water budget should consider all significant inflows and outflows including, but not limited to, rainfall, runoff, infiltration, exfiltration, evaporation, and outflow.

Average annual runoff may be computed using a weighted runoff coefficient for the tributary drainage area multiplied by the average annual rainfall volume. Infiltration and exfiltration shall be based on site-specific soils testing data. Evaporation may be approximated using the mean monthly pan evaporation or free water surface evaporation data given below.

C.13 Example Problem

C.13.1 Example

A shallow basin with an average surface area of 1.21 hectares and a bottom area of 0.81 ha is planned for construction at the outlet of a 40.47 ha catchment area. The catchment area is estimated to have a post-development runoff coefficient of 0.3. Site-specific soils testing indicates that the average infiltration rate is about 0.25 cm/h. Determine for average annual conditions if the facility will function as a retention facility with a permanent pool.

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C.13.2 Solution

1. From rainfall records, the average annual rainfall is about 1270 mm,
2. The mean annual evaporation is 890 mm,
3. The average annual runoff is estimated as: **Runoff** = (0.3) (1.27 m) (404.700 m^2) = 154.190 m^3.
4. The average annual evaporation is estimated as: **Evaporation** = (0.89 m) (12 100) = 10 770 m^3.
5. The average annual infiltration is estimated as:
   Infiltration = (0.0025 m/hr) (24 hours/day) (365 days/yr) (8100 m^2)
   Infiltration = 177.390 m^3.
6. Neglecting basin outflow and assuming no change in storage, the runoff (or inflow) less evaporation and infiltration losses is: **Net Budget** = 154.190 - 10.770 – 177 390 = -33. 970 m^3.
   Thus, the proposed facility will not function as a retention facility with a permanent pool.
7. Revise pool design as follows:
   Average surface area = 0.81 ha and bottom area = 0.40 ha
8. Recompute the **evaporation and infiltration Evaporation** = (0.89 m) (8100) = 7210 m^3.
   Infiltration = (0.0025 m)(24h/d)(365d/y) (4000 m^2) = 87 600 m^3.
9. The revised runoff less evaporation and infiltration losses is:
   Net Budget = 154.190 - 7210 - 87600 = +59 380 m^3.

The revised facility is assumed to function as a retention facility with a permanent pool.

C.14 Construction and Maintenance Considerations

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage facilities. To assure acceptable performance and function, storage facilities that require extensive maintenance are discouraged. The following maintenance problems are typical of urban detention facilities and facilities shall be designed to minimise problems:

- Weed growth;
- Grass and vegetation maintenance;
- Sedimentation control;
- Bank deterioration;
- Standing water or soggy surfaces;
- Mosquito control;
- Blockage of outlet structures;
- Litter accumulation; and
- Maintenance of fences and perimeter plantings.

Proper design should focus on the elimination or reduction of maintenance requirements by addressing the potential for problems to develop.

- Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained by hand cutting or by using available power-driven equipment, such as tractor mowers,
- Sedimentation may be controlled by constructing traps to contain sediment for easy removal or low-flow channels to reduce erosion and sediment transport,
- Bank deterioration can be controlled with protective lining or by limiting bank slopes,
- Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or by constructing underdrain facilities to lower water tables,
- In general, when the above problems are addressed, mosquito control will not be a major problem,
- Outlet structures shall be selected to minimise the possibility of blockage (i.e., very small pipes tend to block quite easily and shall be avoided),
- Finally, one way to deal with the maintenance associated with litter and damage to fences and perimeter plantings is to locate the facility for easy access where this maintenance can be conducted on a regular basis.

C.15 Protective Treatment

Protective treatment may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences may be required for detention areas where one or more of the following conditions exist:

- Rapid stage increases would make escape practically impossible where small children frequent the area;
- Water depths either exceed 1.0 meters for more than 24 hours or are permanently wet and have side slopes steeper than 4: 1;
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 1.5 meters or a flow velocity greater than 1.5 m/s; and
- Side slopes equal or exceed 2:1.

Guards or grates may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

Fencing shall be considered for dry retention areas with design depths in excess of 1 meter for 24 hours, unless the area is within a fenced, limited access facility.

Figure C.12 Design Storage Facilities

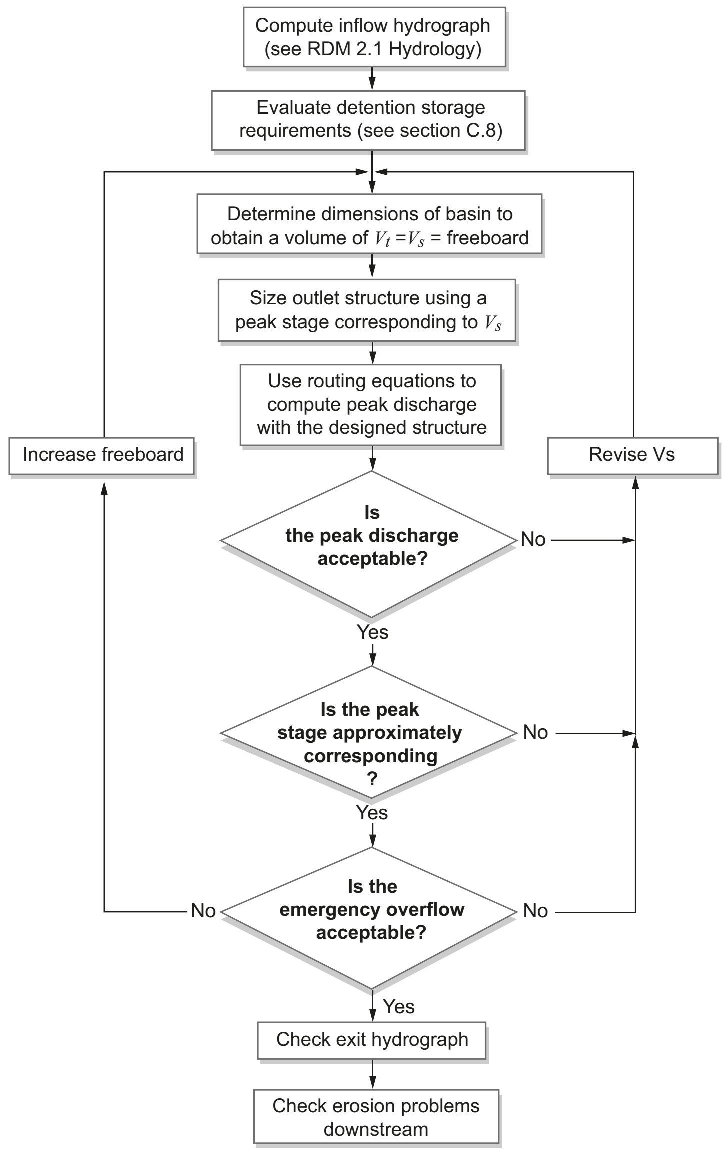
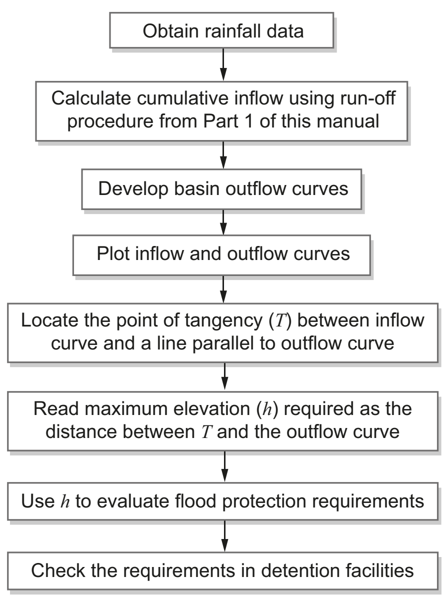


Figure C.13 Flood Elevations Estimation in Land-locked Areas



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Appendix D: Hydrographic Survey Forms

Form 1 Field Visit Investigation Form

Project Name:			
By:		**Date:**	
Structure type:		**Piers Type:**	
Size or span:		**Skew:**	
Skew:		**Inlet:**	
Clear HT:		**Outlet:**	
Abutment Types:		**% ∇ of Road:**	
Inlet Type:		**% ∇ of Stream:**	
Existing waterway cover:			
Overflow begins at:		**Length of overflow:**	
Check for debris:		**Max AHW** (m):	
Reason:			
Side slopes:		**Height of banks** (m):	
Up or downstream restriction:		**Outlet channel base:**	
Type of material in stream:		**Mannings value:**	
Ponding:		**Check bridges up and downstream:**	
Survey required:		**Check land use up and downstream:**	
Remarks:			

Form 2 Hydrographic Survey Field Inspection Check List

GENERAL PROJECT DATA:			
Project number:			
Road name:		**K.P:**	
Site name:		**Station:**	
Site description:	Cross Drain [ ] Irrigation [ ] Storm Drain [ ] Channel Change [ ] Other [ ]		
Survey source:	Field [ ] Aerial [ ] Other [ ]		
Date of Survey:		**Inspected by:**	
OFFICE PREPARATION FOR INSPECTION:			
Reviewed:		**Aerial photos?**	Yes[ ] No [ ]
Site Sketch?	Yes[ ] No [ ] Required? [ ]	**Mapping/maps?**	Yes[ ] No [ ]
Permanent file?	Yes[ ] No [ ] No data found [ ]	**Length of overflow:**	
Special requirements and problems identified for field checking:	[ ] Hydrologic Boundary - obtain hydrologic channel geometry.		
	[ ] Adverse Flood History - obtain HW Marks/dates/eyewitness.		
	[ ] Irrigation Ditch - obtain several water depths.		
	[ ] Other.		
	[ ] Adverse Channel Stability and Alignment History - Check for head cutting, bank caving, braiding, increased meander activity.		
	[ ] Structure Scour - check flow alignment.		
	[ ] Scour at culvert outlet.		
	[ ] Evidence of bridge scour.		
	[ ] Obtain bed/bank material samples at.		

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Form 2 Hydrographic Survey Field Inspection Check List *(continued)*

FIELD INSPECTION: (The following details obtained at the site are annotated on the Drainage Survey)	
Survey appears correct?	Yes[ ] No [ ]
Apparent errors are:	
Which are resolved by:	
Flooding apparent?	No [ ] Yes, HW marks obtained [ ] Yes, but HW marks not obtained because:
Does all floods reach the site?	No [ ] Yes, HW marks obtained [ ] Yes, but HW marks not obtained because:
Does flood waters enter an irrigation ditch?	No [ ] Yes, HW marks obtained [ ] Yes, but HW marks not obtained because:
Hydrologic channel geometry obtained?	No [ ] Yes [ ] Because:
Structure scour evidence?	No [ ] Minor [ ] Yes [ ] Obtained bed/bank samples [ ] Noted any flow alignment problems [ ] Bed/bank material samples not obtained [ ] Flow alignment not noted because:
Irrigation facility?	No [ ] Yes [ ] Describe:
Manning's n obtained?	No [ ] Yes [ ] Because:
Property damage due to BW?	No [ ] Yes [ ] Elevation/property type checked [ ] Yes, but elevation/property type not obtained because [ ]:
Environmental hazards present?	No [ ] Yes [ ] Details obtained [ ] Yes, but details not obtained because [ ]:
Ground photos taken?	Upstream floodplain and all property [ ] Downstream floodplain and all property [ ] Site looking from downstream [ ] Site looking from upstream [ ] Channel Material w/scale [ ] Evidence of channel instability [ ] Evidence of scour [ ] Existing structure inlet/outlet [ ] Other [ ]
Effective drainage area visually verified?	No [ ] Yes [ ] Because:
POST INSPECTION SURVEY ANNOTATION:	
Section II Findings annotated on survey?	Yes [ ] No [ ] See section attached (attach typed explanation by site station and site name, and check list section and number).
Survey originals and check lists forwarded to	Roadway Design Unit [ ] Hydraulic Design Unit for hydraulic design [ ] Designer Making Inspection [ ]

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