The South African National Roads Agency Limited

DRAINAGE MANUAL

5th Edition - Fully Revised

THE SOUTH AFRICAN NATIONAL ROADS AGENCY LTD

Creating wealth through infrastructure

WEB2P

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2nd print

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FOREWORD

Water is often thought of as the source of civilization; hence the hypothesis that the development of hydraulics is related to the evolution of ancient societies such as those of Mesopotamia and Egypt. Owing to the structure of the early states, which entailed a closed system of absolute monarchy and monopoly with only a small number of literate scholars, the pace of technological advancement was cumbersome. The basket remained the only water-lifting device in Egypt until the sheduf was introduced during the time of the New Kingdom – almost 3 500 years after the commencement of agriculture in Egypt and 1 500 years after the rise of the nation-state. The development of the waterwheel and the Archimedes clean water screw followed 1 000 years later in Alexandria.

Founded by the Egyptian ruler, during the Ptolemies dynasty (323 BC to AD 30), the Mouseion in Alexandria hosted scholars such as Euclid and Archimedes (287 to 212 BC) who made significant advances in mathematics of cones and cylinders as well as differential equations leading to major advances in hydraulic engineering. These Alexandrian scholars laid the foundations of theoretical hydrology in connection with practical applications. Around the same time the Persians too had already made an ingenious contribution to hydraulic engineering by developing a water delivery system known as qanats – a subterranean system of tunnels connecting wells. However, it is the Romans who were instrumental in expanding the science of hydraulic engineering to various parts of their empire.

Through the ages, civil engineers have always had to cope with unforeseen natural forces. The external forces created by climatic change, and further exacerbated by human induced variables, can unexpectedly and significantly influence the hydrological cycle with serious socio-economic effects. Although mathematical analysis and modelling cannot cater for every eventuality, we can certainly attempt to scientifically predict the behaviour of these natural forces and minimise their impacts on our environment.

South Africa, for instance, is known for its low average annual rainfalls and large seasonal variations. Despite the latter, abnormal rainfalls have historically had disastrous consequences. Although our problems are not without precedent, societies are always inter-linked and local catastrophes could have serious regional and national repercussions.

The channelling of water by societies for usage and development has remained an ongoing challenge since the days of the early mathematicians through to modern-day engineers. We trust that this manual, published as a guide to both students and practitioners will assist in meeting these challenges. It must, however, be emphasised that it is merely an aid and should ultimately not replace sound engineering analysis and judgement.

Nazir Alli
Chief Executive Officer
The South African National Roads Agency Limited

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Edwin Kruger
Editor
The South African National Roads Agency Limited

Feedback:

Any positive feedback for possible incorporation into future editions will be appreciated. Please email such comments/feedback to the Editor at bridges@nra.co.za

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LIST OF SYMBOLS

<u>Chapter 2</u>

AF	=	the damage cost of abnormal floods
AF^x_n	=	damage cost of an abnormal flood to design alternative X in year n
AFQ_D	=	damage cost of a design flood, Q_D
AF_{RMF}	=	damage cost of an RMF
B/C	=	benefit/cost ratio
C_A	=	all costs incurred in establishing a facility
C_A	=	investment (capital) cost that is required to implement the alternative A
C_A	=	the construction cost of alternative x
F	=	future value
FA	=	future amount at the end of year n
i	=	annual discount rate as a decimal fraction
IRR	=	internal rate of return technique
M	=	the normal annual cost of maintenance and operation of alternative x
n	=	discount period in years

NPV	=	net present value
PQ_D	=	probability of an SDF in year n, i.e. $(1/T_{SDF})$, where T_{SDF} is the return period of the SDF
P_{RMF}	=	probability of an RMF in year n, i.e. $(1/T_{RMF})$, where T_{RMF} is the return period of the RMF
PW	=	present worth (value in year zero)
PWOC	=	present worth of cost
$PWOC^x$	=	present Worth of Cost of alternative x
$PW(M_0+U_0)$	=	the present worth of facility maintenance costs and user costs of the null alternative
$PW(M_A+U_A)$	=	the present worth of facility maintenance costs and user costs of a proposed alternative
$PW(CS_A)$	=	consumer surplus gained through additional usage induced by the proposed alternative
r	=	rate at which the left-hand and right-hand sides of the equation are equal, resulting in a NPV of zero

Chapter 3

a	=	constant
A	=	area of catchment (km²)
ARF	=	area reduction factor (%)
ARF_{iT}	=	area reduction factor (%)
b	=	constant
C	=	run-off coefficient (dimensionless)
C	=	catchment parameter with regard to reaction time
C_1	=	run-off coefficient for rural area with a value between zero and one
C_{1D}	=	rural run-off coefficient incorporating the effect of dolomites
C_{1T}	=	rural run-off coefficient incorporating the effect of dolomites and initial saturation factor
C_{100}	=	calibration coefficient (SDF method)
C_2	=	calibration coefficient (SDF method)
C_2	=	run-off coefficient for urban area with a value between zero and one
C_3	=	run-off coefficient for lakes with a value between zero and one
C_P	=	run-off coefficient according to average soil permeability
C_S	=	run-off coefficient according to average catchment slope
C_T	=	combined run-off coefficient for T-year return period (dimensionless)
C_V	=	run-off coefficient according to average vegetal growth
D	=	storm duration (hours)
F	=	lag coefficient
F_T	=	adjustment factor for initial saturation for return period T
f_{iT}	=	flood run-off factor (%)
H	=	height (m)
H	=	height of most remote point above outlet of catchment (m)
$H_{0,10L}$	=	elevation height at 10% of the length of the watercourse (m)
$H_{0,85L}$	=	elevation height at 85% of the length of the watercourse (m)
he_{iT}	=	effective rainfall (mm)
I	=	rainfall intensity (mm/hour)
I_T	=	average rainfall intensity for return period T (mm/h)
K	=	regional constant
K_{RP}	=	constant for T-year return period
K_T	=	constant for T-year return period
K_u	=	dimensionless factor
L	=	hydraulic length of catchment (watercourse length) (km)
L_C	=	distance from outlet to centroid of catchment area (km)
m	=	order number
m	=	number, in descending order, of the ranked annual peak floods

M = 2-year return period daily rainfall from TR102 [3.16]
M = mean of the annual daily maxima
MAR = mean annual rainfall (mm/a)
n = design life (years)
n = length of record (years)
P = mean annual rainfall (mm/annum)
P = probability (%)
P_1 = probability of at least one exceedence during the design life
P_{AvgT} = average rainfall over the catchment for the T-year return period (mm)
P_{AvgiT} = average rainfall for T-year storm duration (mm)
P_{iT} = point intensity for the return period T (mm/h)
P_T = point rainfall for the return period T (mm)
P_x = probability of x exceedances over the design life (n)
$P_{t,T}$ = the precipitation depth for a duration of *t* minutes and a return period of T years
Q = peak discharge (m³/s)
Q_e = peak discharge of unit hydrograph (m³/s)
Q_{iT} = peak discharge for T-year return period (m³/s)
Q_p = unit hydrograph peak discharge (m³/s)
Q_{RMF} = regional maximum flood peak flow rate (m³/s)
Q_T = peak discharge for T-year return period (m³/s)
r = roughness coefficient
R = average number of days per year on which thunder was heard (days/year)
S_{av} = average slope (m/m)
T = time (hours)
T = return period (years)
T_C = time of concentration (hours)
T_L = lag time L (hours)
T_p = time to peak (hours)
T_{SD} = storm duration (hours)
t = duration (minutes)
x = number of exceedences
α = area distribution factor
β = area distribution factor
γ = area distribution factor

Chapter 4

A = sectional area (m²)
A_1 = upstream sectional area (m²)
A_2 = downstream sectional area (m²)
B = channel width (m)
C_c = contraction coefficient (≈ 0,6)
C_L = loss coefficient for sudden transitions
E = specific energy (m)
f = roughness coefficient
F_x and F_y = force components exerted by the solid boundary on the water
g = gravitational acceleration (m/s²)
h_f = energy loss over distance L (m)
k_s = roughness coefficient, representing the size of irregularities on bed and sides (m)
h_l = transition loss (m)
h_f = friction losses (m)
h_l = transition losses occurring where the flow velocity changes in magnitude or direction (m)
k_s = measure of absolute roughness (m)
L = distance (m)
L = pipe length (m)

Mg	=	weight of the enclosed fluid mass (N)
p	=	intensity of pressure at centre-line (Pa)
p_1 and p_2	=	intensities of pressure on either side of the bend (N/m^2)
q	=	discharge per unit width ($m^3/s/m$)
Q	=	discharge (m^3/s)
r_c	=	centre line radius (m)
R	=	hydraulic radius i.e. area divided by wetted perimeter (m)
S	=	energy slope, which is equal to bed slope only when flow is uniform (m/m)
v	=	uniform channel velocity (m/s)
V	=	volume stored (m^3)
$\overline{v}$	=	average velocity (m/s)
$\overline{v}_1$ and $\overline{v}_2$	=	upstream and downstream average velocities (m/s)
$\overline{v}_x$	=	average velocity component in x direction (m/s)
w	=	vertical sluice opening (m)
y	=	depth of flow measured perpendicular to the streambed (m)
$\overline{y}$	=	distance between water surface and centre of gravity of section (m)
z	=	bed level at point where depth of flow = y (m)
z	=	centre line elevation (m)
α	=	coefficient compensating for variations in velocity across a section
β_1 and β_2	=	angles of direction (°)
$\left(\gamma\overline{y}A\right)_x$	=	force component in x direction
γ	=	specific weight (value for water 9,8 x 10^3 N/m^3)
θ	=	longitudinal bed slope angle, (°)
ρ	=	mass density = 1 000 kg/m^3 for water
υ	=	kinematic viscosity (≈ 1,14 x 10^{-6} m^2/s for water)

Chapter 5

A	=	effective cross-sectional plan area of the opening (m^2)
B	=	total flow width (m)
C	=	inlet coefficient (0,6 for sharp edges or 0,8 for rounded edges)
C_D	=	discharge coefficient
D	=	depth of flow (m)
E	=	specific energy (m)
F	=	blockage factor (say, 0,5)
Fr	=	Froude number
H	=	total energy head above grid (m)
H	=	energy head ≈ flow depth for upstream conditions (m)
H	=	head (m)
K_L	=	discharge coefficient
s	=	energy gradient (m/m)
$\overline{v}$	=	average velocity (m/s)
V_s	=	settling velocity (m/s)
x	=	distance between obstructions (m)
y	=	depth of flow at deepest point (m)
y_2	=	equivalent sequent jump depth with a horizontal bed (m)
z	=	height of obstructions (m)

Chapter 6

A_{over}	=	area of flow over structure at the flow depth selected (m^2)
A_{eff}	=	the effective inlet area through the structure (m^2)
B	=	the width of the channel (or the length of the structure) (m)
d	=	depth of flow over the structure (m)

D	=	the height of the soffit of the deck above the river invert level (m)
f_i	=	a dimensionless factor related to the design level
Fr	=	Froude number
g	=	gravitational acceleration (9,81 m/s²)
L_B	=	the total width of the deck of the structure (m)
n	=	Manning n-value ($s/m^{1/3}$)
$n_{concrete}$	=	Manning roughness coefficient of concrete ($s/m^{1/3}$)
n_{river}	=	Manning roughness coefficient of the river bed ($s/m^{1/3}$)
P_{cell}	=	the total wetted perimeter of each cell (m)
$P_{concrete}$	=	the part of the wetted perimeter that has a concrete surface per cell (m)
P_{eff}	=	ΣP_{cell} (effective wetted perimeter for the flow passing through the structure) (m)
P_{over}	=	wetted perimeter at the flow depth selected (m)
P_{river}	=	the part of the wetted perimeter that is made up by the riverbed per cell (m)
Q_2	=	discharge with a 1:2 year return period (m³/s)
Q_{design}	=	design discharge (m³/s)
Q_{over}	=	discharge over the structure within the selected flow depth (m³/s)
Q_{under}	=	discharge capacity of the openings through the structure (m³/s)
R	=	hydraulic radius (m)
S_0	=	slope in direction of flow (m/m)
$\bar{v}_{under}$	=	average velocity of flow through the structure (m/s)
x	=	thickness of the deck (depending on the structural design outcome) (m)

<u>Chapter 7</u>

B	=	width (inside of culvert) (m)
C_B	=	inlet coefficient for culverts
C_h	=	inlet coefficient for culverts
D	=	inside diameter (m)
D	=	height (inside of culvert) (m)
h_{f1-2}	=	friction losses between cross-section 1 and 2 (m)
$\sum h_{l1-2}$	=	transition losses between cross-section 1 and 2 (m)
H_1	=	upstream energy level, relative to the invert level (m)
H_2	=	downstream energy level, relative to the invert level (m)
$\bar{I}$	=	average inflow (m^3/s)
I_1	=	inflow to the ponded area at time t_1 (m³/s)
I_2	=	inflow to the ponded area at time t_2 (m³/s)
K_{in}	=	inlet secondary loss coefficients
K_{out}	=	outlet secondary loss coefficients
O	=	outflow through culvert (m³/s)
O_1	=	outflow at t_1 (beginning of time step Δt) (m³/s)
$\bar{O}$	=	average outflow (m³/s)
S	=	temporal storage or ponding volume (m³)
S_0	=	natural slope (m/m)
S_c	=	critical slope (m/m), where Fr = 1
$\bar{v}$	=	average velocity (m/s)
y_n	=	normal flow depth (m)
y_c	=	critical flow depth (m)
ΔS	=	change in storage volume (m³)
Δt	=	time step that is used (s)
$\frac{I_1 + I_2}{2}\Delta t$	=	average volumetric inflow (m³)
$\frac{O_1 + O_2}{2}\Delta t$	=	average volumetric outflow (m³)

Chapter 8

A_1	=	flow area at section 1 (m²)
A_4	=	flow area at section 4 (m²)
A_n	=	flow area at for normal flow conditions (m²)
A_{n2}	=	projected flow area at constricted section 2 below normal water level of the river section (m²)
B	=	mean channel width (m)
B_n	=	mean channel width for normal flow conditions (m)
B_n	=	total flow width for the normal stage (m)
b	=	pier width (m)
C	=	Chézy coefficient
C	=	a constant
C	=	coefficient for specific gravity and stability factors
C_b	=	backwater coefficient
D	=	flow depth (m)
d_{50}	=	average particle diameter (m)
D_{50}	=	median size of bed material (m)
D_{50}	=	riprap size (m)
D_{50}	=	median riprap size (m)
d_{avg}	=	average depth in the main channel
D_c	=	critical particle size for the critical velocity V_c (m)
D_m	=	effective mean bed material size (1,25D_{50}) (m)
d_s	=	local scour depth at pier (m)
e	=	voids ratio of soil mass
Es	=	specific energy (m)
F	=	constant obtained from measured data
F_D	=	distance of the design flood, Q_T, below a deck soffit (underside of deck) (m)
Fr	=	Froude number
Fr_1	=	Froude number directly upstream of the pier
F_s	=	side factor to describe bank resistance to scour
F_{SBP}	=	freeboard to shoulder breakpoint (m)
g	=	gravitational acceleration (9,81 m/s^2)
h*	=	backwater damming height, afflux (m)
h^*_{1A}	=	backwater damming height abnormal stage conditions (m)
k_s	=	absolute roughness of river bed (m)
K^*	=	secondary energy loss coefficient
K	=	pier shape coefficient (1,5 for round-nosed and 1,7 for rectangular piers)
K	=	factor applied for abutments
K_1	=	a factor defined
K_1	=	correction for pier nose shape
K_2	=	correction factor for angle of attack of flow
K_3	=	correction factor for bed condition
K_4	=	correction factor for armouring due to bed material size
K_u	=	0,0059 (SI units)
L	=	pier length (m)
L_s	=	spacing between spurs (m)
n	=	Manning's coefficient of roughness (s/m$^{1/3}$)
Q	=	total discharge (m³/s)
Q	=	equivalent steady discharge which would generate the channel geometry (m³/s)
Q_T	=	design flood (m³/s)
Q_{2T}	=	twice the recurrence interval design flood (m³/s)
S	=	energy slope (m/m)
s	=	specific gravity of soil particles
S_s	=	specific gravity of riprap
SF	=	required stability factor to be applied
q	=	discharge per unit width (m³/s.m)

q	=	discharge through the sub-channel
V	=	velocity on pier (m/s)
$\bar{V}_i$	=	average velocity through sub-channel (a, b or c)
$\bar{V}_1$	=	average velocity through Section 1 (m/s)
$\bar{V}_1$	=	mean velocity upstream of the pier (m/s)
$\bar{V}_1$	=	average approach velocity (m/s)
$\bar{V}_{2A}$	=	average velocity in constriction during abnormal stage conditions (m/s)
$\bar{V}_{2c}$	=	average critical velocity in constriction (m/s)
$\bar{V}_{n2}$	=	average flow velocity at section 2 based on A_{n2}
$\bar{V}_a$	=	average velocity in the main channel
V_*	=	shear velocity (m/s)
V_{*c}	=	critical shear velocity (m/s)
V_i	=	approach velocity when particles at pier begin to move (m/s)
V	=	characteristic average velocity in the contracted section (m/s)
V_{ss}	=	particle settling velocity (m/s)
V_R	=	velocity ratio
V_{c50}	=	critical velocity for D_{50} bed material size (m/s)
V_{c90}	=	critical velocity for D_{90} bed material size (m/s)
y	=	mean depth of flow (m)
y	=	depth of flow in the contracted bridge opening (m)
$\bar{y}$	=	projected normal flow depth in the constriction (m)
y_0	=	depth upstream of pier (m)
y_1	=	flow depth directly upstream of pier (m)
y_2	=	flow depth under bridge (m)
y_{2c}	=	critical depth in constriction (m)
y_s	=	scour depth (m)
Y_t	=	total maximum scour depth (m)
Y_0	=	maximum general scour depth (m)
Y_s	=	local scour depth (m)
α_1	=	velocity coefficient
α_2	=	velocity head coefficient for the constriction
θ	=	bank angle with the horizontal (°)
ρ	=	density of water (kg/m^3)
ρ_d	=	dry bulk density (kg/m^3)
ρ_s	=	saturated bulk density (kg/m^3)
φ	=	riprap angle of repose (°)
τ_c	=	critical tractive stress for scour to occur (N/m^2)
ν	=	kinematic fluid viscosity (m^2/s)

Chapter 9

A	=	surface area (m^2)
A_g	=	geotextile area available for flow (m^2)
A_t	=	total geotextile area (m^2)
AOS	=	apparent opening size (mm)
B	=	a coefficient (dimensionless)
B	=	width of collector drain (m)
C_u	=	the uniformity coefficient
d	=	diameter of pipe (m)
D_{85}	=	soil particle size for which 85% of openings are smaller (mm)
D_x	=	the sieve size through which x% of the material passes (mm)
g	=	longitudinal slope of the road (m/m)

i	=	hydraulic gradient (m/m)
I	=	design infiltration rate (mm/h)
k	=	Darcy coefficient of permeability (m/s) and
k_s	=	permeability of material (m/day)
k_b	=	permeability of an open-graded layer (m/day)
k_t	=	permeability of the channel backfill (m/day)
L	=	length of the pipe (m)
L	=	length of paving (1 m wide) subject to infiltration (m)
n	=	Manning's n ($s/m^{1/3}$)
n_b	=	porosity of an open-graded layer
O_{95}	=	opening size in geotextile for which 95% of openings are smaller (mm)
P	=	1h duration/1 year return period rainfall intensity (mm/h)
q	=	drainage rate (mm/day)
q	=	discharge per meter width ($m^3/s.m$)
S	=	spacing (m)
S	=	cross-slope of a drainage layer (m/m)
S_o	=	slope of the pipe (m/m)
t	=	depth of flow in material (mm)
T	=	drainage period for layer (h)
t_b	=	thickness of drainage layer (mm)
t_b	=	effective thickness of drain layer (mm)
W	=	width of the drainage layer (m)
ψ	=	geotextile permittivity

CHAPTER 1 – INTRODUCTION

1.1 HISTORICAL OVERVIEW

When the *Road Drainage Manual* (RDM) was first published in Afrikaans in 1981[(1.9)], its purpose was defined as:

"*om nuttige inligting aangaande paddreinering in 'n bruikbare vorm saam te vat*" (to combine useful information on road drainage in a usable format).

It was realised at that stage that many road drainage components were not being designed optimally. This could be attributed largely to the fact that it typically took a designer a while to accumulate enough separate pieces of information, which were required to undertake road drainage design. The large numbers of copies of the RDM that have been sold since then, as well as the large number of persons who have attended courses on the application of the manual, were proof of the need for such a document.

It is especially necessary in the southern African context to provide manuals for the local designers since they have fewer opportunities to consult with specialists. Whilst a Drainage Manual must cater for this reality, it remains true that only persons with the necessary insight and experience may deal with major drainage structures and complex scour phenomena at these structures.

As in the original RDM, recommendations are made regarding methods of calculation, coefficients and design criteria for general use. In contrast with textbooks, theories are only quoted briefly with the emphasis rather on the application of the theories. Wherever possible, available information has been sorted through so that only the most useful information giving reliable answers, is provided in this manual.

1.2 CURRENT POSITION OF SANRAL

The South African National Roads Agency Limited (SANRAL), generally known as SANRAL, was formed on 1 April 1998, as an independent, statutory company registered in terms of the Companies Act.

The South African Government, represented by the Minister of Transport, is the sole shareholder and owner of SANRAL. Its mandate is to develop, maintain and manage South Africa's proclaimed national roads. [(1.1)]

In striving to reach the key objectives, SANRAL is being, and has been, pro-active in compiling relevant information, developing manuals and participating in capacitating the role players in the roads industry to ensure a sustainable road infrastructure network.

Compiled SANRAL manuals and standards that have been well accepted and which are widely used include:

- *Procedure for road planning and design*, (2003) [(1.2)]
- *Policy in terms of road planning and design*, (2003) [(1.3)]
- *Geometric Design Manual*, (2002) [(1.4)]
- *Code of procedure for the planning and design of highway and road structures in South Africa*, (2002) [(1.5)]

As a result of the core value to be pro-active to the needs of the customers and other stakeholders and the wide use of the earlier *Road Drainage Manual* (1981 – Compiled by the South African Roads Board), SANRAL decided to revise and upgrade this document as a user tool for all persons involved in the design of drainage structures and systems. This document is titled: *Drainage Manual*.

The purpose of the *Drainage Manual* is to provide a reference document with regard to drainage and to demonstrate and reference some software for the modelling and analysis of drainage problems. The *Drainage Manual* covers road drainage in particular detail. Revisions and additions to the original RDM, include:

- Clarifying points, which have proven to be open to misinterpretation.
- Adding new useful local information, which has become available since the publication of the RDM during 1981.
- Addressing identified gaps in information.
- Deleting information which has become obsolete due to changes in policy or practise, and replacing it with material in accordance with new requirements [(1.5)].
- Illustrating hand calculations for verification and reinforcing of the analysis procedures and theories.
- Referring to appropriate computer software.

The *Drainage Manual* therefore contains the following:

- A Road Map at the beginning of each chapter to guide the user to specific sections to indicate typical problems and to provide references to other chapters.
- Theoretical description of the applicable calculations for drainage problems.
- Hand calculations of typical problems.
- Reference to applicable software utilities and user manuals for the programs.

The *Drainage Manual* should be read in conjunction with the *Code of Procedure for the Planning and Design of Highway and Road Structures in South Africa* (February 2002 or later editions). The Code covers the procedures to be followed and design requirements to be met in the planning and design of structures for The South African National Roads Agency Limited (SANRAL), including structures being designed for other authorities, but funded by SANRAL, or structures built for other authorities that are proclaimed national roads.

The reader is also referred to the *Guidelines for the Hydraulic Design and Maintenance of River Crossings* Volumes I to VII [(1.6)], compiled by the Committee of State Road Authorities during the period 1992 – 1994 and published by the Department of Transport, as well as the *Design Manual for Standard Box Culverts* [(1.7)], published by the National Transport Commission and the Natal Provincial Administration in 1981. The content of this *Drainage Manual* takes precedence over the documents indicated above.

1.3 LAYOUT OF THE DRAINAGE MANUAL

The *Drainage Manual* contains ten chapters. These chapters have been structured to provide the user with the relevant theoretical bases, worked examples as reinforcement of the application of the theory, as well as design procedures and reference to utility software. Photographs of different aspects of drainage and drainage structures have also been included to provide further background to the topics covered in this document. The focuses in the different chapters are:

Chapter 1: (This chapter): Provides an introduction to the *Drainage Manual*.

Chapter 2: Reviews the planning, economic and legal considerations.

Chapter 3: Discusses the methods that can be used to calculate floods for different recurrence intervals.

Chapter 4: Reflects procedures for hydraulic calculations.

Chapter 5: Discusses surface drainage design.

Chapter 6: Includes details of low-level crossings.

Chapter 7: Contains design information for lesser culverts and storm water pipes.

Chapter 8: Focuses on bridges and major culverts and scour at these structures.

Chapter 9: Discusses sub-surface drainage.

Chapter 10: Reflects relevant web links and refers to supporting software for drainage.

Each chapter ends with analytical and calculation procedures for different typical drainage problems.

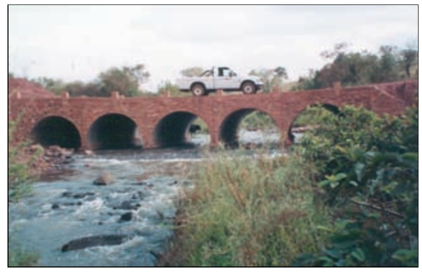
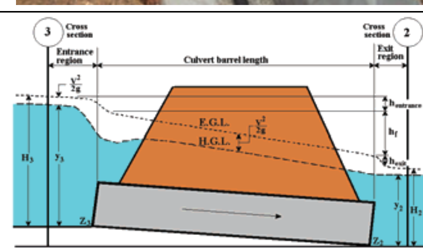
At the start of each chapter a “Road Map” is included. This Road Map reflects the topics covered in the chapter, provides a quick reference to other topics, refers to typical problems and hand calculations and contains reference to utility programs. **Table 1.1** (Road Map 1) provides the road map for the *Drainage Manual* and reflects the different aspects covered in this publication.

It is trusted that the document will provide valuable assistance in the design of drainage systems. Users are invited to comment on any aspect that they feel should be extended or reviewed.

Supporting drainage utility software and user manuals for this software are being developed. These will be contained in a separate document [(1.8)].

Table 1.1: Overview of the Road Maps

ROAD MAP 1		
Road Map	**Topic**	**Schematic**
1	Chapter 1 - Introduction	
2	Chapter 2 - Economic analysis procedures and legal aspects with regard to road drainage	
3	Chapter 3 - Flood hydrology	
4	Chapter 4 - Hydraulic calculations	
5	Chapter 5 - Surface drainage	

Road Map	Topic	Schematic
6	Chapter 6 - Low-level crossings and causeways	
7	Chapter 7 - Lesser culverts and storm water pipes	
8	Chapter 8 - Bridges and major culverts	
9	Chapter 9 - Sub-surface drainage	
10	Chapter 10 - Web-based links and supporting software	

1.4 REFERENCES

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1.4 SANRAL. *Geometric Design Manual*. (2002). [Online]. Available: http://www.nra.co.za/downloads.html [30 November 2006]

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1.9 Rooseboom, A., Basson, M.S., Loots, C.H., Wiggett, J.H. and Bosman, J. (1983). *National Transport Commission road drainage manual*. Second edition. Pretoria: Director-General: Transport. Chief Directorate: National Roads.

CHAPTER 2 - ECONOMIC EVALUATION OF DRAINAGE SYSTEMS AND LEGAL ASPECTS PERTAINING TO DRAINAGE SYSTEMS

WJ Pienaar and SJ van Vuuren #

2.1 INTRODUCTION

The aim of this chapter is to provide comprehensive guidelines for the economic evaluation of drainage systems and hydraulic structures for road projects by practising transport engineers and transport economists. In this respect the guidelines should achieve the following objectives:

- Ensure that the results of evaluations conform to an acceptable degree of accuracy;
- Standardise the approaches without prescribing the extent of investigations and the levels of detail of reports thereon; and
- Offer systematic techniques for testing the proposed projects in terms of quantified technical and economic efficiency criteria.

As is the case with any other infrastructure improvement project, the economic evaluation of drainage systems and hydraulic structures measures costs and benefits from the point of view of society as a whole. It therefore requires a precise identification, quantification and evaluation of all the costs and benefits associated with a project. It further requires ascertaining whether the scale, the scope and the timing of project implementation are adequate to ensure the most appropriate use of the country's resources. Another aspect of the economic evaluation is that it can provide an input during the design phase of a project and thus contribute to optimisation of the project design.

The project evaluation process as conceptualised in this chapter is primarily concerned with methodological and quantitative consistency and standards of the analysis. At the same time it addresses, to the maximum possible extent, the basic practical aspects with the aim of achieving an acceptable degree of uniformity in approach and comparability of results when the project evaluation is carried out by different professional entities and/or in different regions of the country.

Given the serious consequences that inadequate drainage designs might have, the legal issues are of major importance. Designers need to be aware of the basic legal principles, and should seek expert legal advice when justified. It is appropriate that the legal aspects are covered, together with economic evaluations, as the risk that is accepted in design, is increasingly being determined by budget constraints.

Aspects, which are covered, include:

- Legislation;
- Wrongfulness and negligence;
- Court judgements; and
- Professional liability.

Table 2.1 contains the Road Map that provides a reference to the aspects that are covered in this chapter.

 # An earlier document on Legal Aspects by TJ Scott, has been appended electronically on supporting CD

Table 2.1: Road Map for economic evaluation of drainage systems and legal aspects

ROAD MAP 2			
Typical problems	**Par**	**Input variables**	**Hand calculations ***
Determination of the economic value of a project	2.2.1	Economic value of the resource, future inflation, base date, time value of money, discounting procedures	Par 2.9
Economic evaluation techniques	2.3	Present worth Net present value Benefit/Cost ratio Internal rate of return	Par 2.9 Example problems
Risk of flood damage	2.5	Design standard, maintenance and operation costs and other additional costs relating to the reinstatement of the project	
Economic selection of project alternatives	2.7	Mutually exclusive projects	Par 2.7.2
		Independent projects	Par 2.7.3
		Mutually exclusive and Independent projects	Par 2.7.4

Note: * Supporting utility software could be used to determine the NPV, IRR and conduct a Life Cycle Cost Analysis (LCCA)

2.2 BASIC PRINCIPLES OF ECONOMIC EVALUATION

2.2.1 Economic value of resources

2.2.1.1 Need for shadow pricing

In order to assess the economic efficiency of the application of resources, an economic evaluation, based on the consideration of social cost, has to be conducted. Social cost is the *opportunity cost* to society of the resources it uses. It is equal to the value of whatever society forgoes by using resources for a certain purpose instead of another. This involves determining the *scarcity value* of inputs in a project and of the outputs produced as a consequence.

The prices obtained under perfectly competitive market forces are indicators of peoples' willingness to pay for different goods and services and thus serve as a measure of value to society. Competitive market prices are signals between producers and consumers that, in the absence of any market failure, mediate the efficient (i.e. optimal) allocation of resources. In reality, markets are of course different: governments (acting on behalf of societies) distort prices for all kinds of reasons, and market failure does occur.

When goods and services, appearing in a *cost-benefit analysis*, are traded in a competitive market with few distortions and failures, their market prices are a relatively good guide of the willingness to pay for the goods and services, and are usually accepted and used in the evaluation as they stand. When they are not so traded, the economic price can either be estimated directly by finding out what people would be willing to pay if there was an effectively working market, or by taking the distorted market price and adjusting it by *shadow pricing*.

An economic evaluation of transport projects must be based on shadow prices, due to the following:

- Whenever (i) transaction prices in the market do not reflect the social opportunity cost of resources exchanged, and (ii) non-market items (such as life, comfort, convenience and the external effects of activities) are assessed, shadow prices need to be determined that represent, or act as proxy for, their scarcity value. (In economics literature shadow prices of market items are sometimes referred to as factor costs or accounting costs or economic costs, while the shadow prices of non-market items are sometimes labelled *surrogate prices*).

- Double counting will occur if those government levies included in the price of vehicle operating inputs, and used by the government for investment in and maintenance of infrastructure, are treated as if they form part of vehicle running costs.

- Cost exaggeration will occur if indirect tax serving as general state revenue (e.g. value-added tax and customs and excise duties that the government might opt to spend on non-infrastructure related items) is included in the economic cost of a transport project.

- Indirect taxation will lead to a distorted cost comparison if modes or projects are compared and their inputs are not taxed proportionately, or if the inputs of some are not taxed at all.

2.2.1.2 Shadow pricing of market items

The shadow price of a market item is equal to its transaction price, minus indirect taxes and user charges, plus subsidies and other refinements that reflect the effect of price distortions in order to represent the item's economic (or resource) cost. The *transaction price* includes the normal profit, and not above-normal profit that will be realised in a competitive market. The *distortions* result from market and government regulatory failures.

Taxes and subsidies do not represent economic resources but simply involve transfers of funds between the public and private sectors. Benefits and/or costs may be under- or over-emphasised if user charges, indirect taxes and subsidies are not excluded from an analysis of public projects.

Transfer payments are financial payments between different parties within the defined entity on whose behalf an evaluation is performed, for example, a nation in the case of a cost-benefit analysis. From the point of view of a nation as a whole, domestic transfer payments have no significance. Individuals, households, firms and other sub-groups may experience these costs and benefits, but for the national economy they are like funds transferred internally from one accounting category to another within a firm. So transfer payments within a nation, whether linked with costs or with benefits, should be omitted from any cost-benefit analysis. One could compare it with cash being moved from one's left pocket to one's right pocket.

In a financial evaluation for an entity that pays taxes or duties or receives subsidies, **the income and payments should be considered as inflows and outflows** like any other revenues and expenditures, and should be included in the financial evaluation. In a cost-benefit analysis, however, where the defined entity is a nation, or society as a whole, taxes and subsidies and other international transfer payments should not be counted. The government is only a sub-group within the nation or society as a whole, and these transfers between the government and particular citizens do not increase or decrease the welfare of the nation, because they do not represent the *opportunity cost* of specific resources sacrificed or gained.

2.2.1.3 Shadow pricing of non-market items

The *shadow price of a non-market item* (or an intangible item, such as comfort, convenience and time) represents the utility gained through its beneficial effects and/or the disutility suffered in the form of external costs created by its supply and use. This is defined by what consumers are willing to pay or sacrifice for its benefits, and what they are willing to pay not to suffer its disutility. (As mentioned above this type of shadow price is sometimes called a surrogate price.)

2.2.2 Future inflation

Based on the assumption that the relative scarcity of the resources used in the supply and operation of a facility will not change in the long run, it is customary to disregard inflation in cost-benefit analysis and to base all future values in real value on a specific base date.

2.2.3 The evaluation base date

In economic evaluation, the principle is to express the value of all costs and benefits in a common unit, usually referred to as *constant pricing*. To avoid problems the analyst should use the price levels pertaining to a defined base date, which should be stated, for example "rands at mid-2004 prices" or "rands at the prices of 1 January 2005". All costs and benefits should be expressed in real terms with respect to a base date, whatever the future periods they are expected to occur. The defined base date itself can be fixed according to the analyst's convenience, for instance the date on which most of the cost estimating was finalised. The base date for inflation is independent of "year zero" (the present time when the analysis is performed) for discounting. However, for easy value interpretation and present relevance it is desirable that the base date should be set as close to year zero as is possible (i.e. as recently as possible). Setting the base date at some point in the future, i.e. after year zero, should be avoided. Firstly, estimating future values is more prone to error than estimating present or recent values of resources. Secondly, the convention of assuming that no differential inflation will occur in the future makes the choice of a future base date meaningless.

2.2.4 The time value of money

In order to evaluate and compare different projects on an economic basis, it is necessary that benefits and costs be assessed on a common time basis, since money has a time value. An amount *X* would be more valuable now than say in a year's time if it is assumed that inflation or other factors will result in a change of the monetary value of goods, resources and services. This greater relevance of present power of disposal over funds, compared with eventual power of disposal over the same amount, is referred to as the *time preference propensity*.

Time preference is the tendency to prefer desirable things to happen sooner rather than later and undesirable things later rather than sooner. This may be ascribed to three aspects:

- uncertainty;
- decreasing *marginal utility*; and
- greed.

Goods now may be preferred to goods in future decades, because of uncertainty as to whether one will be alive to enjoy them; because one expects one’s total income to be higher than at present so that the expected addition to utility from an equal addition to consumption is less; or through impatience.

Although the prevalence of inflation influences the intensity of individuals' time preference propensity for money, inflation itself is not the only reason why money has a time value. Even in non-inflationary periods, people value present consumption more highly than identical consumption in the future. The intensity of this time preference is related to a society's average income that it receives on its capital (i.e. all savings and investments). The average time preference of an amount may, therefore, be equated with its opportunity or alternative cost, as reflected in the average or expected return on capital over a period.

One can observe people's behaviour and deduce approximately what their individual time preference rate, or unconscious discount rate, is. When decisions are made on behalf of a large number of people, as a society, there is again an observable time preference rate: this collective societal rate is called the social time preference rate.

Because of the time preference attached to the power of disposal over money, the present worth of benefits achieved and costs incurred several years into the future become increasingly smaller as the evaluation process incorporates values that lie further and further in the future. An economic assessment is meaningful only after all future values have been expressed in equivalent terms; in other words after being reduced to their worth at a common point in time by means of a *social discount rate*.

There are three broad approaches toward the determination of the social discount rate, i.e.:

- The first approach emphasises the social time preference rate.
- The second takes cognisance of the social opportunity cost of capital.
- The third is a synthesis of the above two.

The social time preference rate is not equal to the social opportunity cost of capital, because of imperfections in the capital market and because individuals’ collective behaviour toward the future differs from their behaviour as individuals.

The social time preference rate is generally lower than the time preference rate of individuals acting in a market situation. Society as a whole, or the government as its agent, is more aware of desirable goods and services that a free market cannot provide without government mediation, and is conscious, although perhaps only vaguely, of the ethical rights of future generations.

Governments are responsible firstly for reflecting this difference between individual rates and collective rates, and secondly for representing the interests of future generations. This implies taking a long-term view. Therefore, the social time preference rate is generally lower than the time preference rate of individuals.

By applying the opportunity cost principle to discounting; it may be argued that the social discount rate ought to reflect what society forgoes elsewhere if it uses resources for a period of time in a certain project. In theory, this is the return on the last (i.e. marginal) investment that consumes the last of society's available capital. This is the real rate that would reflect the choice made by society between present and future investment returns. The real interest is also known as the social opportunity cost of capital.

The specific rate of discount used in economic evaluation will influence the balance between the cost of investment, which takes place early in the analysis period, and the value of the benefits achieved in future. For example, the present worth of benefits may exceed the present worth of an investment using a low rate of discount and vice versa using a high rate. The rate of discount used is thus significant in the outcome of an economic evaluation. Although economic efficiency requires that the rate of discount should reflect the social opportunity cost, the choice is usually a political one. Therefore, the decision maker normally prescribes the social discount rate.

2.2.5 The discounting procedure

The basic formula for discounting a single future amount to its present worth becomes more comprehensible if discounting is regarded as the reverse of compound interest computation; in other words the conversion of present worth into future worth by making use of a specific rate of interest.

If, for instance, the time value of an amount = i percent per annum, its present worth (PW) in a year's time will increase to PW(1 + i/100).

After two years its worth would equal $PW(1 + i/100)(1 + i/100) = PW(1 + i/100)^2$.

After three years the PW would have increased to an amount equalling $PW(1 + i/100)^3$, and so on, until after n years it would equal $PW(1 + i/100)^n$.

Since discounting is the reciprocal of interest or future amount computation, the calculation applied to determine the future amount $PW(1 + i/100)^n$ for a number of years (n) at an interest rate of i percent, would have to be inverted. In other words, one would have to multiply the future value by $1/(1 + i/100)^n$ to obtain its present worth.

The term $(1 + i/100)^n$ is known as the interest function, whereas its reciprocal, namely $1/(1 + i/100)^n$, is known as the discount function or the present worth factor.

The following formula is used to compute the PW of a future amount (FA) at the end of year n at a discount rate of i percent per annum:

$$PW = \frac{FA}{(1+i)^n} \qquad \ldots(2.1)$$

where:

PW	=	present worth (value in year zero)
FA	=	future amount at the end of year n
i	=	annual discount rate as a decimal fraction
n	=	discount period in years

2.3 ECONOMIC EVALUATION TECHNIQUES

Various techniques, all based on the principle of discounted cash flows, can be used for cost-benefit analysis. The four most commonly applied techniques are:

- Present worth of cost (*PW*OC) technique;
- Net present value (*NPV*) technique;
- Benefit/cost ratio (*B/C*) technique; and
- Internal rate of return (*IRR*) technique.

Use of these four techniques in the economic choice of projects is discussed in Section 2.7 below.

2.3.1 Present Worth of Cost (PWOC) technique

This technique selects the lowest cost alternative among mutually exclusive projects. All economic costs (i.e. the opportunity costs) associated with the provision, management, maintenance and use of each possible alternative project are discounted to their present worth. Given the objective of economic efficiency the alternative that yields the lowest *PWOC* is regarded as the most cost-effective (beneficial) proposal. This method can be expressed as follows:

$$\mathrm{PWOC} = \mathrm{C_A} + \mathrm{PW(M + U)} \qquad \ldots(2.2)$$

where:

PWOC	=	present worth of cost
C_A	=	all costs incurred in establishing a facility (i.e. the opportunity cost of the investment)
PW(M + U)	=	present worth of all facility maintenance costs and user costs.

[Note that in the case of the null alternative (i.e. the existing facility whose possible replacement or upgrading is being investigated, and against which the other mutually exclusive alternatives are measured), PWOC = PW(M + U)]

When a proposed project will, because of lower user cost, induce additional traffic over and above normal-growth traffic, the abovementioned criterion of lowest total transport cost presents a contradiction in terms. Any induced traffic will inflate the facility's *PWOC*, thus defeating the objective of minimum cost.

2.3.2 Net Present Value (NPV) technique

This technique provides an economic performance measure that is used to:

- select the best alternative among the mutually exclusive projects; and
- to help establish an overall economic viability of independent projects.

Net present value (*NPV*) is a technique whereby the present worth of investment cost is subtracted from the present worth of all the future project benefits. The present worth of both costs and benefits is calculated by using an official (social) discount rate. All projects reflecting a positive *NPV* are economically viable, while the project alternative with the highest such value is most suitable for implementation, as this will maximise the net benefit for the society as a whole.

The technique may be expressed thus:

$$\mathrm{NPV} = \mathrm{PW(M_0 + U_0) - PW(M_A + U_A) + PW(CS_A) - C_A} \qquad \ldots(2.3)$$

where:

NPV	=	net present value of benefits
$PW(M_0+U_0)$	=	the present worth of facility maintenance costs and user costs of the null alternative
$PW(M_A + U_A)$	=	the present worth of facility maintenance costs and user costs of a proposed alternative
$PW(CS_A)$	=	consumer surplus gained through additional usage induced by the proposed alternative. This is equal to one-half of the benefit accruing to each existing journey multiplied by the number of induced trips.
C_A	=	investment (capital) cost that is required to implement the alternative A

If the NPV > 0 then the project should be implemented, but should the NPV be < 0 the investment does not provide an acceptable return.

2.3.3 Benefit/Cost Ratio (B/C) technique

This technique provides an economic performance measure used for the selection of the most advantageous independent project(s) by determining the ratio between the present worth of the future project benefits and the present worth of the project investment costs. The future project benefits are associated with the annual savings, relative to the null alternative, plus the consumer surplus gained through additional usage induced by the proposed facility.

The ratio between the sum of the discounted benefits and the sum of the project investment costs is obtained by dividing the former by the latter. All proposals with a ratio value greater than one are viable, while the one with the highest ratio value is economically the most advantageous. However, when mutually exclusive projects are compared, incremental analysis should be used to identify the best alternative.

The method may be expressed as follows:

$$B/C = \frac{PW(M_0 + U_0) - PW(M_A + U_A) + PW(CS_A)}{C_A} \qquad \ldots(2.4)$$

where:

B/C = benefit/cost ratio

2.3.4 Internal Rate of Return (IRR) technique

This technique provides an economic performance measure used for the selection of the most advantageous independent project(s). The distinctive feature of this technique is that its application does not entail a singular discounting procedure with one official rate only. Future benefits ("returns") for the period under review are discounted to the beginning of the period. The benefit of each year is calculated in the same manner as with the *NPV* and *B/C* techniques. The sum of these discounted amounts is compared with the discounted project investment cost. Different rates of discount are selected iteratively and applied until at a certain rate the sum of the annual discounted returns equals discounted investment costs. This rate is then referred to as the internal rate of return.

The project with the highest internal rate of return can be regarded as the most advantageous, although the actual criterion is to compare the rate thus obtained with the opportunity cost of capital as represented by the prevailing real discount rate.

If the prevailing real discount rate exceeds the prevailing social discount rate, the alternative is economically viable. However, when mutually exclusive projects are compared, incremental analysis should be used to identify the best alternative.

The method could be expressed as follows:

IRR = r

When $PW(M_0 + U_0) - PW(M_A + U_A) + PW(CS_A) = C_A$ …(2.5)

where:

IRR	=	internal rate of return
r	=	rate at which the left-hand and right-hand sides of the equation are equal, resulting in a NPV of zero.

2.4 DESCRIPTION OF THE EVALUATION PROCESS

2.4.1 Overview of the evaluation process

The economic evaluation process of drainage systems and hydraulic structures is a process of investigation and reasoning designed to assist decision makers in reaching an informed and rational choice. The process involves engineering and economic considerations, organised in a number of defined activities. The activities involved in the project evaluation process can be broadly divided into the following tasks:

1. Define the purpose of the economic analysis (Section 2.4.2).
2. Select the appropriate economic evaluation techniques (Section 2.4.3).
3. Identify project alternatives (Section 2.4.4).
4. Calculate recurring user costs/project benefits (Section 2.4.5).
5. Estimate project implementation and maintenance costs (Section 2.4.6).
6. Incorporate the cost of abnormal flood damage (Section 2.5).
7. Calculate the economic performance by means of one or more of the economic evaluation techniques (Section 2.3).
8. Undertake sensitivity and risk analyses (Section 2.6).
9. Identify other considerations (Section 2.4.6.5).
10. Choose between mutually exclusive alternatives and independent projects (Section 2.7).
11. Presentation of the results.

2.4.2 Establish the purpose of the economic evaluation

The economic evaluation of drainage systems and hydraulic structures could be performed at the following levels:

- LEVEL 1: To determine the economic viability.
- LEVEL 2: To establish specific design criteria/details of a specific project.

2.4.2.1 LEVEL 1: To determine the economic viability

The objectives of this type of evaluation are twofold:

- To inform the decision maker whether or not to proceed with detailed design and/or implementation of a project
- To provide a basis for prioritising independent projects.

This type of evaluation is usually undertaken for a specified road section, including ancillary services and structures. Drainage systems and hydraulic structures are in most cases considered to be an integral component of the road, and would thus not be evaluated separately from the rest of the road.

However, there may be instances where major hydraulic structures, such as bridges and viaducts, may be evaluated as stand-alone projects. There are cases where an existing bridge, for example, is considered for replacement because of structural failure, capacity constraints or improved road alignment. In such instances the bridge and the corresponding road realignment should be economically justified in its own right.

2.4.2.2 LEVEL 2: To establish specific design criteria/details of a specific project

This type of evaluation is usually undertaken to compare different design and route alignment options in order to select the most appropriate route alignment, or to optimise the efficiency of the design. Drainage systems and hydraulic structures may be subjected to this type of evaluation to ensure selection of the most appropriate design criteria that will in turn ensure the greatest total net benefit.

2.4.3 Selection of appropriate economic evaluation technique(s)

The individual economic evaluation techniques establish the economic viability of prospective projects on the basis of either:

- Minimum total costs, which can be determined through the present worth of cost (*PWOC*) technique. This technique is suitable for a LEVEL 2 evaluation.
- Net benefit, which is determined by the net present value (*NPV*) technique. This technique is suitable for a LEVEL 1 evaluation.
- Relative benefit, which is usually determined either by (i) the benefit/cost ratio (*B/C*) technique or (ii) the internal rate of return (*IRR*) technique. This technique is suitable for a LEVEL 1 evaluation.

These techniques could be classified into two groups on the basis of their underlying philosophy. For the first group (first bullet above), only the cost of each alternative is calculated; the argument being that the alternative with the lowest cost would be superior. The *PWOC* technique falls into this group. In working with the second group of techniques (the second and third bullet above), both benefits and investment costs of alternatives are calculated.

2.4.4 Definition of project alternatives

2.4.4.1 Define the null alternative (Alternative 0)

The null alternative is usually defined as the do-nothing alternative, which represents the existing facility that road users currently use. In the definition of this alternative one usually assumes that no further improvements other than normal routine and periodic maintenance are performed over the evaluation period.

Sometimes the do-minimum option, instead of using the do-nothing option, is used as the basis for evaluating the proposed project. In such cases the do-minimum option becomes the null alternative. The do-minimum option is used when a certain minimum level of expenditure is required for the facility to continue to perform its function to an acceptable standard. It is not compulsory to define an Alternative 0 when a Level 2 evaluation is undertaken.

2.4.4.2 Define the project alternative(s) (Alternative 1 through n)

For the definition of project alternatives, a range of possible solutions should be examined to ensure that the best possible solution is arrived at, given the various applicable physical, economic and other constraints. Preliminary screening may be necessary, with a limited number of alternatives evaluated in depth.

In a climate of economic stringency, this part of the process is particularly important. Creative low-cost solutions, even where these depart from ideal standards, may facilitate the improvement of traffic conditions where full-scale road construction may have to be deferred. Such low-cost solutions could be considered as separate alternatives.

Staging and phasing of project implementation, and also its appropriate timing on grounds of both technical and economic considerations, should be carefully assessed.

Staging of implementation or delayed implementation of a project may commend itself as a possible solution worth evaluating only after the initial project evaluation has been carried out. Whilst it is obviously preferable to attempt to identify staging or deferring of implementation as a project alternative at an early stage in the evaluation process, an iterative process may well assist in arriving at the most suitable approach to implementation from an economic perspective.

2.4.5 Calculate recurring user costs / project benefits

2.4.5.1 General

Benefits are defined as savings in user costs of existing, normal-growth, diverted and transferred traffic, and facility recurring costs relative to the null alternative (i.e. the existing situation or present facility of which the improvement or replacement is being investigated), plus the consumer surplus gained through additional (i.e. induced) usage generated and/or developed by the proposed facility. The underlying philosophy of techniques falling into this group is that an alternative will be economically viable if benefits exceed its investment costs. The investment cost of a project can be defined as the opportunity cost of the economic resources sacrificed in establishing the project.

Transport infrastructure provides access and facilitates mobility. In areas that lack adequate accessibility and mobility, the supply of new transport facilities is consequently often used as an instrument of development by activating investment in and business interaction between economically dormant areas or regions. However, when transport facilities are built primarily with a view to improving existing mobility (such as in already developed areas) the prime objective is to reduce user costs (i.e. vehicle running costs, accident costs and travel time, including any other disutility resulting from poor quality of service when travelling). In order to determine how efficiently proposed facilities will fulfil their intended role, it is necessary that their provision should be subject to economic evaluation.

The total costs of transport infrastructure supply and usage comprise one-off and recurring costs. One-off costs comprise the investment or initial costs. Recurring costs are incurred continuously throughout the service life of a facility, and consist of the facility user costs and facility maintenance costs. An increase in one-off costs (i.e. greater investment) generally gives rise to a decrease in recurring costs, and vice versa. Determining the optimum trade-off between investments and recurring costs could, therefore, achieve minimisation of total societal cost per trip undertaken.

In a societal sense transport cost is composed of the total costs involved in transport operations and the provision of infrastructure that makes transport possible. The word "cost" is used here in its widest possible sense, and includes all the negative side-effects of transport operations, whether or not they are measurable in monetary terms.

2.4.5.2 User cost classification

User costs could be categorised as follows:

- Vehicle running costs in relation to:
    - existing traffic; and

  - additional traffic that would be the result of:
    - normal-growth traffic;
    - diverted traffic;
    - transferred traffic;
    - development traffic; and
    - generated traffic.
- Travel time cost in relation to:
  - individuals; and
  - vehicles
- Accident cost suffered:
  - by people;
  - to vehicles; and
  - to goods and other property.
- Cost of abnormal floods.

To determine the benefits accruing to the various categories of users under vehicle running costs, it is necessary to differentiate between the following components of the anticipated traffic:

Existing traffic is the current traffic on a facility that is due to be replaced or improved. When a new transport facility is to be introduced, without any changes to the existing transport network in the area, the existing traffic equals zero. When a new facility replaces an existing one, the current traffic of the existing facility is taken as the existing traffic of the new facility.

Normal-growth traffic is traffic growth that would have occurred in spite of the creation of the new facility. The growth may be attributable to the following factors:

- general population growth;
- an increase in the per capita ownership of vehicles; and
- an increase in average use per vehicle.

Diverted traffic is traffic of the same transport mode diverted from other facilities because of the opening of a new facility. When a new facility is introduced without any changes to existing facilities in the network, all the traffic transferred from the existing facilities to the new facility is regarded as diverted traffic.

Transferred traffic is traffic attracted from other modes to a new or improved facility.

Development traffic is traffic developed after implementation of a new facility because of any changing use of the land served by the facility. The supply of new transport facilities in a developing region usually stimulates economic development and settlement as a result of greater accessibility. Improved access generally results in changed and more intensive land use, which in turn attracts and "develops" more traffic to the corridor through which the facility passes. The volume of development traffic can be estimated by means of appropriate trip attraction and trip development indices for the new land uses.

Generated traffic is traffic that did not exist previously and is generated solely through a reduction of the disutility of travel, brought about by lower user costs and higher service quality offered by the improvement or provision of the transport facility.
Generated traffic consists of previously potential transport users who have been encouraged to join the traffic as a result of the reduction of the generalised cost of travel to below the price they are willing to pay for travelling. This puts new destinations within their reach.

(**Development traffic** and **generated traffic** are collectively known as **induced traffic**. Whereas development traffic is attributable to improved accessibility, generated traffic is attributable to the easing of mobility).

2.4.5.3 Reduction in vehicle running costs

Where new transport facilities will not induce a significant volume of additional traffic, savings in running costs attributable to new facilities could normally be measured with a fair degree of accuracy by calculating the difference between vehicle running costs with and without a new or improved facility. Vehicle running costs consist of:

- energy/fuel consumption;
- tyre wear (in the case of road vehicles);
- oil consumption;
- vehicle capital costs; and
- vehicle maintenance costs.

2.4.5.4 Time savings

Time savings mean that trip (journey) times using a new facility are shorter compared with trips using the existing facility between the same origin and destination points. When evaluating the effects of facility improvements, users' travel time savings should be assessed in terms of their alternative value; i.e. the utility that is sacrificed by not being in a position to perform something else than travelling. The question arises whether short time-savings (for example, single minutes or fractions of minutes) have any value. Do sixty savings of one minute each have the same value as a saving of one hour? It is generally accepted that all time-savings, irrespective of whether they are "useful" or not are added up. The reasons for this are firstly that each person would have a different definition of what a useful amount of time is. Secondly, seconds saved whilst travelling on a particular road section under investigation may be insignificant, but the hours saved if the entire route is upgraded would be useful. The fact that time savings on a particular section are disregarded could mean that all time savings are disregarded, which is an underestimation of road improvement benefits.

In recognition:

- that short time-saving incidences may lead to substantially longer and utilisable time opportunities later (as cited in the example above), and
- that a series of short time-saving incidences during a single trip may add up to a long enough period to utilise productively after completion of the trip, it has become customary to assign an economic value to very short time savings.

Time savings in relation to vehicles may be measured from two angles:

- whether the same trip could be covered in less time, or
- whether a longer distance could be covered or more journeys can be made in the same time.

Both have the potential of generating greater income and/or saving money, provided of course that the time saving is used by scheduling additional or longer trips (i.e. market expansion). If time savings are not enough to expand vehicle operations, a benefit can still be gained by using the idle time for maintenance work on the vehicles. These benefits are realised in the form of decreasing vehicle capital cost per kilometre travelled.

2.4.5.5 Reduction in accident cost

One of the main objectives in the planning of a new facility or a facility improvement may be to reduce the number and severity of accidents. This particular benefit may even be the deciding consideration in creating a facility, such as a freeway.

To calculate the benefits of accident prevention it is necessary to predict the accident rate and the severity of the accidents (usually on the basis of similar/comparable facility standards and traffic conditions). The cost of the anticipated benefits is then deducted from the current cost of accidents.

The cost of the anticipated benefits poses two problems:

- to what extent are accidents (with respect to number and severity) in fact attributable to poor infrastructure conditions and quality; and
- what value is placed on loss of human life and personal injury?

2.4.5.6 Cost of abnormal floods

The cost of abnormal floods is dealt with in Section 2.5.

2.4.5.7 Calculation of recurring transport infrastructure cost savings

The establishment of a new or improved transport facility may lead to an increase (negative saving) or a decrease (positive saving) of recurring supply costs. Recurring facility supply costs (notably maintenance cost) will increase when:

- the new facility is an addition to the existing network, and
- when the new facility replaces an old facility but the new one is much more elaborate; for example, when more traffic lanes are added to a road, or a single-carriageway road is replaced by a dual-carriageway road.

An example of obtaining a positive recurring cost saving is when the surface of an unpaved road, which suffers from the onslaught of severe and chronic rainy weather, is replaced with durable weather-proof paving material so that its maintenance cost decreases. (Note that a negative saving is deducted from project benefits and not added to project cost.)

2.4.5.8 Total project benefit projection for use in cost-benefit analysis

The benefits of a new facility are estimated by means of a “with” and “without" comparison. Recurring cost is first projected for a continuation of the existing situation. In other words, the analyst assumes that no expenditure will be incurred on improvements other than those related to routine and periodic maintenance.

The potential savings in recurring costs, which may be affected by the introduction of a new facility, can be estimated in the following two ways:

- If the proposed project will replace some existing facilities completely, a projection is made of recurring infrastructure and user costs, assuming (i) a continuation of the existing situation, and (ii) the introduction of the new facility. Savings in recurring costs are estimated by subtracting the proposed facility's projected recurring cost from the existing situation's projected recurring cost.

- If the addition of a new facility to an existing transport network is investigated (in other words, existing routes and transport services are retained), savings in recurring infrastructure and user costs are determined by projecting these costs for a network "with" the additional facility and subtracting the result from projected cost for the existing network "without" the additional facility.

In the cases above, the benefit accruing to induced traffic (i.e. generated plus developed traffic) is not considered. On the assumption that demand for a transport facility is represented by a linear demand schedule, half of the benefit accruing to each existing journey is added to the project benefits for each induced trip. This is based on the additional or extended consumer surplus accruing to existing users (i.e. their saving) and the new consumer surplus created with respect to induced traffic.

The consumer surplus is the difference between the price a consumer or user actually pays for a product (i.e. a good or a service) and the amount that he/she would be willing to pay for the product; i.e. the value he/she attaches to the product. (A user's willingness to pay for a product is reflected by the satisfaction or utility he/she derives from the product, which is taken to be greater than the price actually paid; i.e. the utility sacrificed to obtain the product). In **Figure 2.1** the willingness of users to make use of a transport facility at different prices is represented by the linear demand curve reflected as line UABD. At a generalised user cost per journey (the utility sacrificed plus disutility suffered) of U_0 the traffic volume equals T_0 with a resultant consumer surplus represented by area UAU_0.

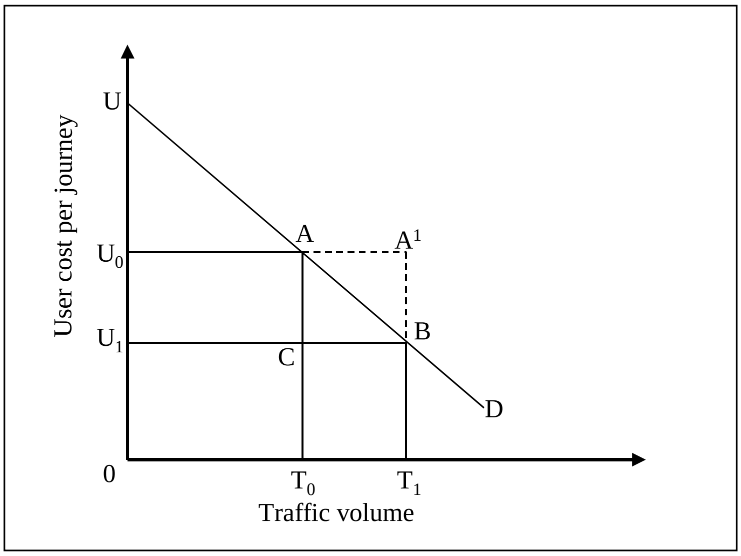

Figure 2.1: Potential user benefits over a specific period in respect of a new or improved facility

If, for example, the facility referred to above is improved so that the user cost for existing traffic is reduced to U_1 and the existing traffic volume is T_0, the saving for existing users is represented by rectangle U_0ACU_1 in **Figure 2.1**. This rectangle is the area by which the consumer surplus or net utility of the existing users increases. If T_1 - T_0 represents the induced traffic volume, the "saving" achieved by induced traffic is not represented by rectangle AA^1BC; instead, the demand curve halves the "saving" to the area of triangle ABC, which represents the consumer surplus or net utility gained by the induced traffic.

The benefits achieved by each individual traffic component are calculated as follows:

- The savings accruing to existing traffic, normal-growth traffic and diverted traffic are computed by subtracting each traffic component's user cost between origin and destination with the new facility from what it would have been without the new facility.

- The benefit of traffic transferred from other transport modes is calculated by subtracting the cost of transferred journeys from the cost saving effected by those transport modes from which traffic was transferred (This is often a negative saving. For example, in the case of a new freeway between two cities, some commuter rail users may transfer to freeway use, thus adding to road user costs, while the volume of commuter rail patronage lost may not be enough to reduce incrementally the supply of rail commuter services - thus additional costs accrue).

- The benefit accruing to induce traffic is equal to the consumer surplus, created with respect to this traffic component.

2.4.5.9 Treatment of non-user benefits in economic evaluation

Care should be taken not to regard user benefits (savings accruing to users) as total transport benefits. Non-user benefits are an important consideration in many decisions about proposed transport facility construction or improvement, especially when a facility has the potential of stimulating economic activity and development. Non-user benefits do not involve savings as user benefits do, but represent a group of plus factors or returns that are partly the consequence of incentives and investments in other sectors of the economy. They can be seen as general economic benefits, above and beyond the direct user benefits, that contribute to the welfare of everyone within the geographical sphere of influence of the facility.

The developmental or regional economic benefits of a transport facility often amount to a distribution or spatial transfer of economic activities to the location or vicinity of the transport facility. These apparent benefits are not credited to a transport facility in cost-benefit analyses. Even in cases where a transport facility will be instrumental in helping to develop and generate new economic activity (i.e. induced activity that has not been distributed or transferred from elsewhere), the general regional economic returns of such activities are not credited to the facility in a cost-benefit analysis. It should be remembered that additional investment (above and beyond that necessary for the transport facility) is a prerequisite for the realisation of general economic and community benefits.

A road that opens up a water-rich and fertile area for agricultural development, for example, will not be credited in terms of the economic benefit of the agricultural returns in a cost-benefit analysis - it will be regarded as a benefit of the agricultural investment whereby it came into being (above and beyond the investment in the transport facility). Similarly, the additional investment in the non-transport-related sections of a transport facility (such as commercial areas where goods and services not related to transport are traded) will receive the credit for returns from commercial activities that may occur there, and not the investment in the transport facility itself.

2.4.6 Determination of project investment costs

2.4.6.1 General

In economic evaluation, enlargement of the existing consumer surplus through a lowering of user costs, plus the generation and development of new consumer surplus, plus the savings in recurring infrastructure maintenance costs, form the basis for calculating benefits. As opposed to this, the opportunity cost of the investment needed to establish a facility (the so-called one-off cost) is the cost component in economic evaluation.

2.4.6.2 Opportunity cost of investment

The opportunity costs of an investment in transport infrastructure include:

- the costs incurred in direct planning and design (traffic surveys, studies of use and establishing a facility, environmental impact studies, compilation and recording of the construction details, etc.);

- the opportunity costs of the land reserve and preparation of the site for development (demolishing, levelling, reinforcement, etc.); and

- the construction of the facility (including the construction of access links, installation of traffic control devices and other appurtenances as well as landscaping). These costs in their totality are often referred to as project implementation costs.

Each of the above-mentioned items includes the opportunity costs for materials and equipment used, labour, contractor operating and overhead costs and normal profit as well as the costs of the needed project management. The actual scarcity value of all inputs that are inevitably or unavoidably needed to create the facility and to link it to the existing transport network (that is, to supply it complete and ready for use).

Care should be taken to include in the analysis only the costs of that part of the facility that is necessary for the functioning of the facility for transport purposes. Constructions that are used for commercial activities and entertainment, for example, are non-transport economic considerations and the costs associated with them are omitted from the evaluation.

The shadow price of existing constructions/buildings used as part of the evaluated project should be calculated on an opportunity-cost (i.e. alternative-use) basis and that of new constructions/buildings on the basis of construction/building costs. Where building costs serve as a basis for these calculations, adjustments have to be made for indirect taxes and distorted labour prices that serve as an input.

In developed countries the labour market is often fairly unrestricted, and shadow pricing of the labour component of costs is seldom considered necessary. However, in developing countries, such as in South Africa and neighbouring countries, the market price for labour, especially unskilled labour, is usually a poor guide to the economic price because of minimum wage legislation and restrictive practices. A more reliable guide to the economic cost of labour may then be obtained from the opportunity-cost principle, based on its use in alternative revenue- earning opportunities. At times of high unemployment this value would be low, probably equal to the minimum living level wage at which people will find it worthwhile to endure the disutility of working for a subsistence wage.

The shadow wage of semi-skilled and skilled workers (excluding professionals and managers) should be based on the minimum living level wage determined for the lowest paid workers in the study area concerned. In general it is unlikely that a lower shadow wage will apply, so that the possibility of over-estimating the opportunity cost of the labour involved is small. Even under conditions of unemployment the labour of professionals and managers should still be valued at market prices.

Under conditions of full employment the market price of labour is used, although full employment remains a theoretical possibility only.

2.4.6.3 Sunk cost

All costs related to a project's implementation incurred in the past should be taken into account at the value of their respective present alternative application possibilities, should these exist.

Costs incurred prior to the evaluation of items that have no alternative use are regarded as sunk costs, and are disregarded in the economic analysis. If work on a project was abandoned and the question is whether the project should be completed, only future costs - the cost of resources needed to complete the project - are relevant for the economic analysis, if the assets reflected in the work already done have no alternative uses. For example, in an economic evaluation of a road of which the location falls within a road reserve which has already been purchased and that is denied alternative application, the opportunity cost of such land is zero, and thus it is excluded from the analysis.

Similarly, planning, design and other project development costs incurred prior to the time of evaluation (i.e. before year zero) are excluded from the analysis as their dedication to the project is irreversible - they cannot be avoided or salvaged by trading them or using them elsewhere. This is also the reason why no initial cost is assigned to the null alternative in a cost-benefit analysis.

2.4.6.4 Analysis period and service life

For each cost-benefit analysis a choice has to be made regarding the period to be analysed. In principle this is related to the expected lifetime of the project being evaluated. In practice this principle may not always be of much help, since many transport facilities can last almost indefinitely. With effective maintenance (the cost of which is counted in an evaluation) many bridges, viaducts, tunnels, retaining walls and such like might physically be able to serve for many decades, though their use may cease earlier for economic reasons.

The opportunity cost principle excludes the possibility of a transport facility, such as an arched bridge spanning a ravine, possessing an additional value over and above the best alternative application opportunity of its land reserve during its service life. The reason is that investment in transport facilities is regarded as sunken; i.e. it is taken that the development itself has no possible alternative application. For this reason it is desirable that the analysis period should stretch over the entire design period or planned lifespan of a facility. However, there are practical reasons why the analysis period should sometimes be shorter than the intended service life.

A pragmatic way to decide on an analysis period for most economic evaluations is to note that, unless the discount rate is very low, benefits (and disbenefits) in the distant future will make little difference to the outcome of the analysis. At a discount rate of ten percent per annum, a stream of benefits valued at R1 million per year will have a present value of R8,514 million if discounted for the next 20 years, while the corresponding figure for discounting over 30 years is R9,427 million, for 40 years R9,779 million, for 50 years R9,915 million, for 100 years R9,999 million, or R10 million if discounted indefinitely. This illustrates that at a discount rate of ten percent per annum, for periods longer than 30 years, the diminishing discounted value of costs and benefits becomes negligible. Since the estimates of annual net benefits usually contain uncertainties at least in the order of five percent, the accuracy of an evaluation is hardly affected if the discounting is terminated after 30 years, even if the benefits really continue much longer. For discount rates higher than ten percent, this period will be shorter than 30 years and vice versa for discount rates lower than ten percent.

Because of uncertainties, all forecasts are speculative owing to the difficulty of predicting future traffic volumes, changes in technology, land use, demographic features, etc. The reliability of projections that extend beyond two decades is always doubtful. For this reason decision-makers often request that analysis periods must not exceed 20 years, even though the logical solutions to certain transport problems may be durable and indivisible projects with service lives substantially longer than 20 years. To be fair when considering such projects, a departure is then often made from the opportunity cost rule by introducing a second best valuation convention in the form of a residual value. This value purports to represent an artificial opportunity cost of a project during its service life.

If it is foreseen during the planning of a facility that the site on which it is to be erected will be permanently allocated to accommodating a transport facility, the site naturally has no terminal (or end) value. This is so because the site will be reserved from being used for an alternative application and thus will have no opportunity cost at the end of the facility's service life.

However, as the service periods for roads extend over several decades, it is unlikely that one could establish with certainty at the time of an economic evaluation for what purpose a facility's site will be used after its service period had come to an end. For this reason it is recommended that, in dealing with terminal value in an economic evaluation, two sensitivity analyses be executed: one in which the terminal value is equal to the current opportunity cost of the land occupied, and one in which the terminal value is equal to nil, (i.e. there is no alternative application opportunity).

The end value of other facility remnants on the site should be regarded as nil in all cases because they cannot be expected to have any alternative application possibilities: on the one hand, the structure is a specialised transport facility that excludes alternative non-transport-related use and, on the other hand, at the end of its service life it is by definition functionally completely worn-out, which makes re-use of structural components impractical.

When the analysis period of a proposed project is shorter than its expected lifespan, the estimate of residual value is more complicated than the estimate of terminal value. The reason is that the facility's implementation cost needs to be apportioned between the expired part of the expected service life at the end the analysis period and the unexpired portion of the expected service life.

Assuming that the expected service life of a transport facility, such as an arched bridge, is 50 years and the analysis period is 20 years; its structure will have a residual value at the end of the analysis period of approximately 30/50 x initial cost, since it will still be serviceable for 30 years more. The residual value in this example is based on the assumption that structural deterioration of the bridge will progress linearly as time passes.

If different alternatives are investigated for an existing facility (referred to as the null alternative), the same period of analysis should be used in each case, since consistency ensures equal treatment of all the alternatives under consideration. A low-cost solution generally has a short service life, soon necessitating additional capital investment in a replacement project, while a costly structure may ensure an extended benefit period before reconstruction or replacement becomes essential. If a number of mutually exclusive alternatives are investigated and the analysis period is (say) 20 years, provision should be made for replacement within the period of analysis of projects with a service life of less than 20 years (necessitating a second set of capital expenditures) to permit comparison with projects that have a service life of more than 20 years.

Note that the **time when the evaluation is carried out is referred to as year zero**. If the project requires a construction period of two years subsequent to year zero and if the period of analysis is 20 years, the service life of the facility under consideration is only 18 years. The shorter the construction period, the sooner benefits can begin to accumulate and the sooner problems created by the existing facility can be eliminated.

2.4.6.5 Unforeseen expenses

In dealing with implementation costs of a project, a provision may have to be made for unforeseen expenses. There are two instances of possible corrections in this regard:

- The actual costs may be higher than those estimated during the design, particularly in view of the complexity of works involved (for example, in the case of cost of drainage systems and hydraulic structures). In order to eliminate the possibility of under-estimating the investment cost in project evaluation, a provision is made in terms of a physical contingency**.** The amount to be added to the project implementation costs in this regard is usually taken as a percentage of design costs, varying between 5 and 15 percent, depending on the level of design (conceptual, preliminary or final) and the related level of confidence in the estimated costs.

- The actual costs may be higher than the estimated costs in cases where the project design has been completed some time before the project evaluation takes place. In such cases estimated costs must be brought up to the date of conducting the project evaluation (the base date) by means of applying an inflation rate for the intervening period. However, it is incorrect to make provision for cost escalation on account of inflation during the analysis period; i.e. for future years (the period beyond the base date).

2.4.6.6 Interest payable on the investment amount

Interest payable on the investment amount (or on any other borrowed funds) should not be included in the economic cost of a project. It is sometimes included when a project is financed by loans, but is excluded when a project is financed from allocations from general state income. However, the manner in which a project is financed has no relation to its economic cost, as the alternative application possibilities of the resources actually used are the same regardless of the method of finance.

2.5 INCORPORATING THE RISK OF FLOOD DAMAGE

2.5.1 Overview of the flood damage evaluation procedure

The optimal design of drainage systems and hydraulic structures could be defined as that which maintains a proper balance between the cost of the project and the cost of potential flood damage (economic risk). In Section 8.2 a discussion is provided on the selection of freeboard for bridges, reflecting the need to conduct a risk assessment. The optimal design would thus be that with the lowest total PWOC. The following object function is proposed for determining the PWOC of a particular design alternative:

$$PWOC^{x} = C_{A} + PW[AF + M] \qquad \ldots(2.6)$$

where:

$PWOC^x$	=	present Worth of Cost of alternative x
C_A	=	the construction cost of alternative x
AF	=	the damage cost of abnormal floods
M	=	the normal annual cost of maintenance and operation of alternative x

The following steps are proposed for calculating $PWOC^x$ in the economic evaluation of drainage systems and hydraulic structures:

1. Establish the minimum design standard of the structure based on the design flood (Q_D).
2. Establish the regional maximum flood (RMF) for the particular area where the structure is planned.
3. Define all technically feasible mutually exclusive alternatives, each representing a technically feasible structure option, ranging from a structure that is designed according to the design flood (Q_D) (Step 1) to a structure that will withstand the RMF.
4. Estimate the implementation cost of each of the alternatives defined in Step 3.
5. Estimate the normal routine and periodic maintenance and operating cost of each of the alternatives defined in Step 3.
6. Calculate the average annual damage cost of an abnormal flood for each of the alternatives identified in Step 3.
7. Perform the economic analysis.

A detailed discussion of each of the above steps follows in Sections 2.5.2 through 2.5.7.

2.5.2 The minimum design standard

It is established practice to design a facility for a specific flood event, commonly referred to as the “standard design flood” or “design flood” (Q_D). The minimum design life of the structure is determined by the choice of Q_D. A structure is designed to withstand a maximum flow expected with a 10-, 20-, 50- or 100-year return period and the expectancy is determined by analysis of records of past rainfalls and flows. From an economic perspective the minimum design standard should be based on the return period that corresponds to the design life of the road.

The reason is that the majority of culverts and bridges become obsolete long before they are worn out by corrosion or erosion. This obsolescence could be ascribed to:

- increased traffic volumes exceeding their traffic carrying capacity,
- road design standards that change, particularly with regard to lane and shoulder width, and
- realignment of the road to provide more favourable geometric characteristics, or to shorten travel distance.

The design flood (Q_D) may be estimated by means of the following methodology. Standard flood frequencies are used for various structures, which from years of experience, more or less represent the optimum design values. Numerous inconsistencies are found if the type and span of the structure, the stream cross-sectional area, or catchment size are used to classify design frequencies. The rate of flow varies very little over short distances along a watercourse, and constitutes the most significant parameter in quantifying the amount of damage done to a road as well as the extent of traffic obstruction.

For this reason the peak flow calculated for a flood with a return period of 20 years (frequency = 1:20) is used as the basis for selection of the appropriate design return period.

Roads are classified into six classes. These definitions correspond to the "Road Infrastructure Strategic Framework for South Africa"[(2.5)]. (Also see paragraph 8.2, and the proposed design return periods in **Table 2.2**.)

Table 2.2: Proposed design return periods, T, for hydraulic structures of different road classes

Road class		**Proposed return period (T) based on the magnitude of the Q_{20} flood**		
		$Q_{20} < 20\ m^3/s$	**$20\ m^3/s < Q_{20} < 150\ m^3/s$**	**$Q_{20} > 150\ m^3/s$**
1	Primary distributor	50	$T = 42{,}31 + 0{,}385Q_{20}$	100
2	Regional distributor	20	$T = 15{,}39 + 0{,}231Q_{20}$	50
3	District distributor	10	$T = 8{,}46 + 0{,}077Q_{20}$	20
4	District collector	5	$T = 4{,}231 + 0{,}0385Q_{20}$	10
5	Access roads	2	$T = 1{,}539 + 0{,}0231Q_{20}$	5
6	Non-motorised/ Access ways	2	$T = 1{,}539 + 0{,}0231Q_{20}$	5

The design return periods given in **Table 2.2** are adapted where necessary in accordance with the Risk Category, as obtained from **Table 2.3**. It remains the responsibility of the designer to motivate the design return period to be used in the planning stage by allowing for the influence of possible future developments, such as urbanisation and afforestation on the flood peak.

If the potential damage and the impact of disruption, due to failure of the structure are high (Category 3 in **Table 2.3**), the design flood given in **Table 2.2** should be enlarged as deemed necessary based on local incidences in the past. In the case of Risk Category 1 consideration might be given to alter the design return period after full consideration of all relevant aspects, and the motivation for alteration is accepted by the appropriate authority.

Where important structures are concerned, the possible influence of particularly large floods (such as the probable maximum flood) should also be taken into consideration. There are far fewer factors influencing the design return period in road surface drainage, and some guidance is given in Sections 5.2.3 and 5.2.5.

Table 2.3: Factors to be considered to determine the risk category of the structure, which is then used to determine the design flood

FACTORS TO BE CONSIDERED	RISK CATEGORY		
	1	2	3
Extent of possible damage			
Potential damage to the road and associated cost of repairs	Low	Medium	High
Potential other damage such as saturation of agricultural land, etc.	Low	Medium	High
Extent of loss of use			
Time needed for repairs to make route trafficable again	Short	Medium	Long
Availability of detours	Good	Medium	None
Obstruction of traffic flow			
Period of flooding	Short	Medium	Long
Traffic density	Low	Medium	High
Depth and velocity of floodwaters	Low	Medium	High
Strategic and economic importance of route			
Strategic and economic importance: military, police, fire brigade, medical services, etc.	Low	Medium	High
Economic importance	Low	Medium	High

2.5.3 Construction cost estimates

The estimate of construction cost (C_A) of each of the alternatives follows the approach described in Section 2.4.6 above.

2.5.4 Maintenance and operation cost

The estimate of normal maintenance and operation cost (*M*) of the structure designed for the specified return period of each of the alternatives follows the approach described in Section 2.4.6 above. These costs may include minor damage to the structure that would occur with expected minor flood damage associated with the design flood, Q_D, of the specified return period. Such damage is regarded as part of the normal routine and periodic maintenance.

2.5.5 Additional costs associated with abnormal floods

The following additional costs are associated with abnormal flood damage (*AF*):

- External costs, such as damage to private property, upstream and downstream of the structure. It should be noted that these costs can be attributed directly or indirectly to the failure of the structure. It consequently excludes costs which would have occurred as a result of the flood if the structure had not been there.

- Capital and maintenance expenses, including damage to road pavement, shoulders, culverts and other roadway items. The capital and maintenance dependent on the discharge past the structure and the capacity of the culvert or span of the bridge (e.g. operational expenses for flagmen, barricades, signage and the marking of traffic detours).

- Road user costs as a result of the disruption, including VOC for travel along the detour and stopping, travel time cost for lower travel speed and longer travel distance, and accident costs as a result of conflict at and along detours, and also caused by unexpected obstructions and hazards.

Some approaches suggest that the average annual damage to a particular structure should be based on the flood damage cost in a particular year and the probability of a flood in that year. However, these respective approaches require that both the risk and damage cost of all sizes of floods (from a RMF down to a two-year design flood, Q_2) in a particular year be determined for each of the mutually exclusive design alternatives. Such a detailed approach is cumbersome and is not practical for the evaluation of a large number of design alternatives.

The approach proposed in these guidelines is much simpler, without sacrificing a significant degree of accuracy. Instead of calculating flood damage for all return periods, all structure designs that do not conform to the SDF of the minimum return period (which corresponds to the design life of the roadway) are discarded, thus reducing the distance between the outer limits of the probability function. For the remaining structure designs, i.e. those that fall within the limits of withstanding a RMF and the Q_D of the minimum return period, the damage cost of the RMF (AF_{RMF}) is determined. The annual expected damage cost is the area **abc** below the Damage-Flood Probability curve in **Figure 2.2**, where P_{RMF} and P_{SDF} reflect the probability of an RMF and design flood, Q_D, respectively in a particular evaluation year. By reducing the distance between the outer limits, this curve can be adequately represented by a straight-line function, as the minimum requirement is for the structure to withstand the design flood, Q_D. A flood of this magnitude would cause no damage to any of the structure alternatives, which are not accounted for under normal routine and periodic maintenance. As mentioned above, all operational and maintenance costs of structures not attributable to abnormal floods are regarded as normal.

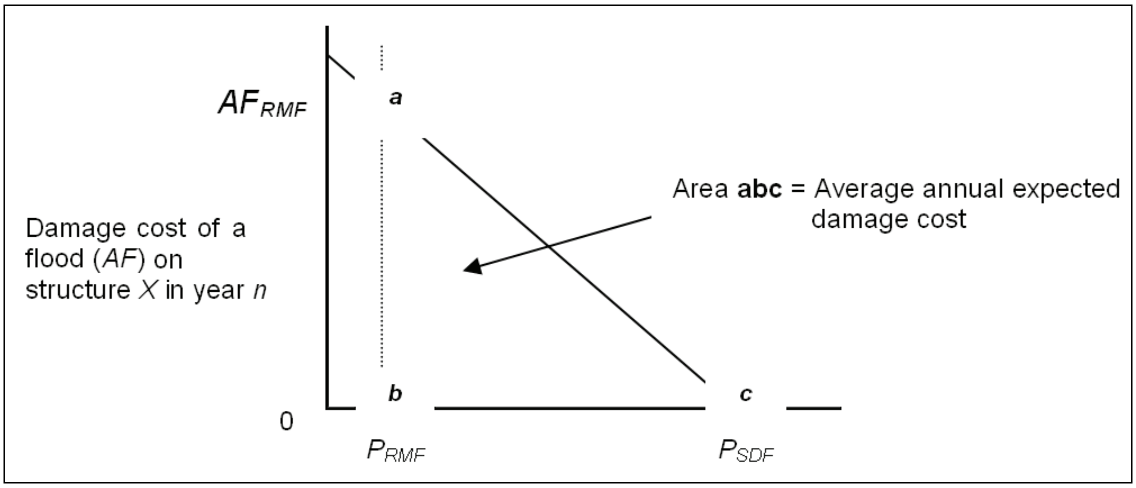

Figure 2.2: Damage-flood probability curve

The damage cost of a structure design alternative in a particular year can be expressed by the following equation:

$$AF^{x}{}_{n} = (PQ_D - P_{RMF})\left[\frac{(AF_{RMF} + AFQ_D)}{2}\right] \qquad \ldots(2.7)$$

where:

$AF^{x}{}_{n}$	=	damage cost of an abnormal flood to design alternative *X* in year n.
PQ_D	=	probability of an SDF in year n, i.e. ($1/T_{SDF}$), where T_{SDF} is the return period of the SDF
P_{RMF}	=	probability of an RMF in year n, i.e. ($1/T_{RMF}$), where T_{RMF} is the return period of the RMF

AF_{RMF}	=	damage cost of an RMF
AFQ_D	=	damage cost of a design flood, Q_D. (This would normally be zero, as the standard structure would be designed according to the return period of Q_D. The return period of the Q_D would be equal to the design life of the road.)

2.5.6 Economic evaluation

The incorporation of the flood damage cost procedure can serve either as an input to a Level 1 economic evaluation in order to optimise the economic viability of a road or major structure, or it can be a stand-alone Level 2 evaluation to refine the design of an already approved project.

In the case of a Level 1 evaluation the additional cost of flood damage (*AF*) for a particular alternative is calculated and added to the recurring cost of that alternative for each year of the evaluation period (Section 2.4.5.6). In this way it is incorporated in the net benefit stream for the purpose of calculating the economic performance measures (NPV, IRR or B/C Ratio).

In the case of a Level 2 evaluation, the PWOC of each alternative is calculated by means of equation 2.6. The alternative with the lowest PWOC is selected as the most efficient design among those that are being evaluated. The economically optimum design could be determined by plotting the PWOC for the evaluated alternatives. A curve may be fitted through the PWOC data points (**Figure 2.3**). The theoretical optimum design is found where the PWOC is at its lowest.

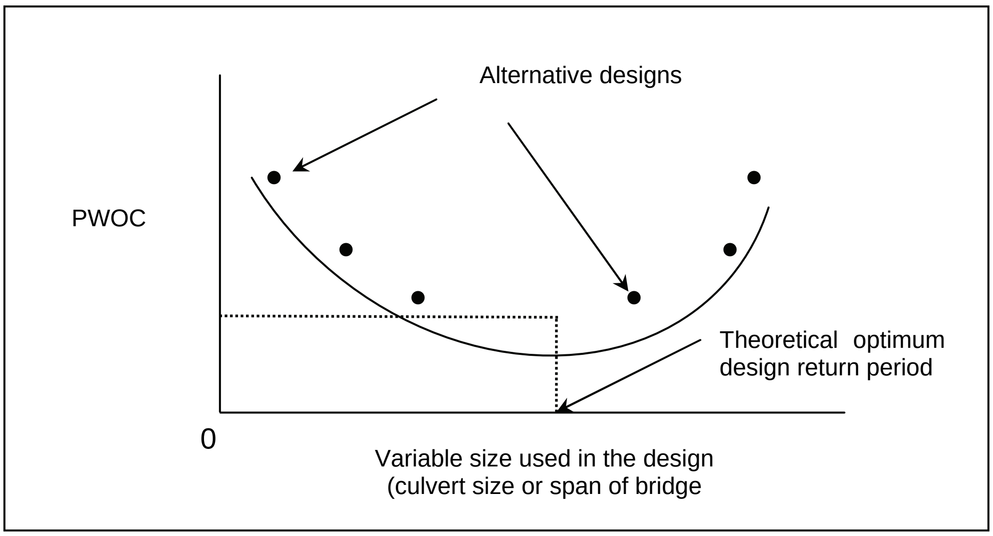

Figure 2.3: Optimum design curve

2.6 RISK AND UNCERTAINTY OF INPUT DATA

All aspects of project evaluation involve uncertainty and risk. Not only are data collected in the field subject to a certain degree of error, but long term projections of 20 years and longer also suggest that unforeseen factors could cause major deviations from the projections.

Projects should consequently not only be appraised with the recognition of uncertainty, but the designs of a project should be approached so as to optimise the total cost of the project. Sensitivity analysis is undertaken to test the sensitivity of the evaluation results for possible deviations in the input parameters. Sensitivity analysis usually considers the variations of input parameters independent from one another. Risk analysis is more specific and involves a formal probability analysis of a likely range of outcomes, where several parameters are identified of which the estimated accuracy is critical to the outcome of the project.

Sensitivity analyses are conducted as found appropriate within the context of the evaluated project. These analyses are carried out for the purpose of testing the sensitivity of the findings and results obtained in the basic evaluation scenario, to changes in some of the initially used basic parameters or assumptions (particularly those for which the level of confidence is lower), or where more than one approach may reasonably be argued and applied. They usually include:

- Combining the VOC savings separately with time cost savings and with accident cost savings.
- Variations in traffic growth rates for the combined savings.
- Variations in the base year AADT.
- Variations in construction costs, depending on the level of design of the investigated project.
- Postponement of project implementation if the basic evaluation scenario indicates that the project is non-viable, or is a marginally viable proposition for immediate implementation.
- Impact of complementary projects.
- Impact of the project on stimulating new developments.

Not all of these sensitivity analyses are always absolutely necessary. It is up to the best judgement of the professional evaluator and/or the client as to what should be included. Care should be taken to limit sensitivity analyses to relevant concerns, in order to inform the decision maker. Attempts to be exhaustive may cause confusion.

2.7 ECONOMIC SELECTION OF PROJECTS OR ALTERNATIVES

2.7.1 Selection criteria

Candidate transport projects should not only be evaluated economically, but those chosen for implementation also need to be selected in such a fashion that net social benefit is maximised. The question is how to compile an investment budget of a given size; i.e. how to allocate a fixed sum of funds between alternative and independent projects. There is also the more complex issue of determining the appropriate size of the investment budget when projects are indivisible, as most transport projects are.

The selection and prioritisation of projects on an economic basis usually takes place with reference to the following general criteria:

- The economic principles need to be strictly observed during evaluation.
- All projects must be evaluated in the same manner.
- All alternatives, i.e. the whole range of technically feasible substitute projects, should be evaluated.
- The benefits of a project should exceed its investment cost.
- The financial investment cost of any chosen project must be within the scope of the capital budget.

Proposed projects to be evaluated could be divided into two groups, namely mutually exclusive proposals and independent projects.

Mutually exclusive proposals are substitutes, i.e. alternative methods of fulfilling the same function. The choice of any one of the proposals will, therefore, exclude all the others. The cost-benefit analysis of mutually exclusive proposals involves the selection of the most efficient, i.e. most cost-effective, alternative. Independent projects fulfil different functions and are consequently not alternatives or substitutes for one another. Examples of independent projects are a proposed embankment and retaining wall in area *X*, a proposed bridge in area *Y*, and a proposed viaduct in area *Z*.

More than one independent project may be selected for implementation. In fact, it is possible that all independent projects may be selected if they are all economically justified and sufficient funds are available. The economic evaluation of independent projects involves the ranking of the economically justified projects in terms of their economic merit. The economic choice of a specific project for implementation involves two steps, namely, project selection and project prioritisation.

- Project selection involves the selection of the best (in economic terms) of the mutually exclusive projects, or in other words, the most advantageous way of solving a specific problem. If there are three routes by which to link the two points, the selection of one will exclude the implementation of the other two.

- Project prioritisation is the arrangement of all functionally independent projects in an order of priority according to their respective degrees of economic viability. The choice of one independent project could at most postpone, but not exclude, the choice of another. The projects will be prioritised from most to least attractive up to the point where the capital budget has been exhausted.

A project that yields a *B/C* ratio value greater than one, always has a positive *NPV* and an *IRR* that exceeds its opportunity cost of capital. Provided the initial costs of projects do not differ, any one of the four evaluation techniques discussed may be used to select the best alternative among a number of mutually exclusive projects. When the initial costs of projects do not differ, the alternative with the smallest *PWOC* will have the highest *B/C* ratio, the highest *IRR* and the highest *NPV*. However, if the initial costs differ significantly (which is generally the case), incremental analysis should be used to identify the most suitable alternative.

The *PWOC* and *NPV* techniques cannot be used to prioritise independent projects. The absolute value of a project's benefits depends on its scope. The benefits of a large project may, for instance, have a larger absolute value than the benefits of a smaller project, whereas the relative return of the larger project may be considerably lower than that of the small project. Hence, it is better to use the *IRR* and *B/C* ratio techniques for the prioritisation of independent projects, also taking into account the results of the investment timing analyses. The reduction of generalised travel cost afforded by new and improved transport facilities may induce additional traffic over and above normal-growth traffic. In such cases the criterion of lowest total social transport cost presents a contradiction in terms that complicates the interpretation of the answer indicated by the *PWOC* technique (equation 2.2). Furthermore, this answer does not give an indication of the size of the economic benefit offered by an alternative, unless the answer is subtracted from the *PWOC* of the null alternative. The latter difference is equal to an alternative's *NPV*. However, use of the *PWOC* technique is appropriate in project selection when a chosen alternative:

- will not induce additional usage, and
- a null alternative does not exist.

Examples of the latter situation are when the construction of a new airport on vacant land is contemplated or when the establishment of a new seaport at a presently inaccessible wetland location on the coastline is being considered, and the most economic ways of linking these facilities to an adjacent urban region have to be evaluated. Under these circumstances, the *PWOC* of each alternative transport link may be calculated using equation 2.2, as the complication that occurs with the treatment of induced traffic is not present.

2.7.2 Mutually exclusive projects

Whenever the opportunity occurs to solve a specific problem when the investment timing of the solution project is not challenged by any independent projects elsewhere, the *NPV* measure is the preferred selection criterion.

Suppose, for example that:

- an amount of R5 million has been allocated to rectify a specific problem situation,
- unused funds cannot be transferred to other projects, and
- a choice has to be made among the three economically viable alternatives shown in **Table 2.4**.

Table 2.4: Present value of benefits and investment costs for three alternative projects

Project	Present value of benefits (R'000)	Present value of investment cost (R'000)	Net present value of benefits (NPV) (R'000)	B/C ratio
A	5 400	3 000	2 400	1,80
B	7 000	4 000	3 000	1,75
C	8 100	5 000	3 100	1,62

Regardless of the fact that alternative C shows the smallest relative return, it maximises welfare by having the greatest *NPV*. Incremental *B/C* analysis using **Table 2.5** shows that a move from alternative A to alternative B, as well as a move from alternative B to alternative C will both be beneficial:

$$(B/C)_{B:A} = \frac{(7\,000\,000 - 5\,400\,000)}{(4\,000\,000 - 3\,000\,000)} = 1{,}6\,; \qquad \text{and}$$

$$(B/C)_{C:B} = \frac{(8100\,000 - 7\,000\,000)}{(5\,000\,000 - 4\,000\,000)} = 1{,}1$$

Thus a move from alternative A to alternative C will yield the greatest net benefit. Note that in a mutually exclusive situation, incremental analysis will always indicate that the alternative with the greatest *NPV* is the best project.

2.7.3 Independent projects

When a choice has to be made among a number of independent projects, given a fixed budget, the *B/C* ratio measure is the preferred criterion. Suppose, for example, a roads authority with a fixed budget of R10 million has to make a choice among 14 independent projects, five of which are indicated in **Table 2.5**.

Table 2.5: Present value of benefits and costs for a number of independent projects

Project	Present value of benefits (R'000)	Present value of investment cost (R'000)	Net present value of benefits (NPV) (R'000)	B/C ratio
A	700	300	400	2,33
B	2 700	1 500	1 200	1,80
C	840	450	390	1,87
D	1 280	600	680	2,13
.	.	.	.	.
.	.	.	.	.
N	1 800	900	900	2,00

In this situation the B/C ratio criterion is the preferred measure to apply, because selection of those projects that yield the highest relative return on capital invested will ensure that total benefit is maximised. The project with the highest B/C value is selected first, followed by the one with the second highest B/C value, and so on until the budget is exhausted. Thus the five projects in **Table 2.5** will be chosen in the order A, D, N, C and B. In this way the benefit per rand spent is maximised.

2.7.4 Mutually exclusive and independent projects

Suppose the objective of the decision-maker is to maximise social benefit subject to the restriction of a fixed budget and that both mutually exclusive and independent projects are under consideration. A method of project assessment based on the incremental principle is recommended. The method consists of the following seven steps:

1. Determine the size of the budget. Where there is some degree of latitude as to the total amount available, the amount can be expanded incrementally, and the incremental benefits compared with the incremental expenditure to determine whether any expansion of the budget is justified.
2. Eliminate all projects that exceed the budget limit and all projects that do not satisfy the minimum acceptance criteria as set out above.
3. Determine which project has the highest *B/C* ratio within each group of mutually exclusive alternatives and then leave out the rest of the possible projects in the group.
4. From the projects under consideration choose the one with the highest *B/C* ratio.
5. Review the choice of the best project in each group of mutually exclusive projects by, firstly, reconsidering all the more expensive projects and noting the incremental *B/C* ratios. Within each group of mutually exclusive projects the project with the highest incremental *B/C* ratio is identified and compared with the rest of the independent projects. Secondly, the available budget is adjusted to reflect the effect of the projects already chosen, and all remaining projects that exceed the balance of the budget are omitted.
6. Repeat steps 4 and 5 for as long as possible. The iteration process ends when the budget is exhausted or when no acceptable projects remain for consideration.
7. Consider adjustments to chosen projects when the budget is not completely exhausted and a small adjustment in a chosen project may provide incremental benefits.

The following example demonstrates this procedure. Suppose an agency has R5 million to spend. The projects under consideration are summarised in **Table 2.6**. Projects A_1 to A_6 are six mutually exclusive projects; B_1 and B_2 are mutually exclusive; D_1 to D_3 are mutually exclusive; and F_1 to F_4 are mutually exclusive. Groups A, B, C, D, E and F are independent.

Table 2.6: PVOC, Benefits and cost-benefit ratios of a number of projects

Project	Present value of investment cost (R'000)	Present value of benefits (R'000)	B/C
A_1	500	880	1,76
A_2	1 000	1 920	1,92
A_3	1 500	2 680	1,79
A_4	2 000	3 320	1,66
A_5	2 500	4 120	1,65
A_6	3 000	4 680	1,56
B_1	550	700	1,27
B_2	750	940	1,25
C	750	1 000	1,33
D_1	1 250	1 260	1,01
D_2	1 400	1 620	1,16
D_3	3 000	3 560	1,19
E	300	560	1,87
F_1	675	1 120	1,66
F_2	850	1 480	1,74
F_3	1 050	1 760	1,68
F_4	1 350	2 120	1,57

There is no project that exceeds the budget limit of R5 million and, furthermore, there is no project with a *B/C* ratio of less than one. All projects are, therefore, included in further analyses. In Step 3 the best projects are chosen from groups A, B, D and F, and the projects that enjoy consideration in the next step are reduced to the following:

Project	Present cost	Present benefit	B/C ratio
A_2	1 000	1 920	1,92
B_1	550	700	1,27
C	750	1 000	1,33
D_3	3 000	3 560	1,19
E	300	560	1,87
F_2	850	1 480	1,74

A_2 is chosen from these six projects. The more expensive projects in group A, are now considered in terms of their incremental B/C ratios. The incremental *B/C* ratios of the four projects more expensive than A_2 are as follows:

Project	Incremental cost	Incremental benefit	Incremental B/C ratio
$A_3.A_2$	500	760	1,52
$A_4.A_2$	1 000	1 400	1,40
$A_5.A_2$	1 500	2 200	1,47
$A_6.A_2$	2 000	2 760	1,38

The greatest incremental benefit is achieved by replacing A_2 with A_3. This replacement within group A should now be considered together with the other projects. There is now R4 000 000 left and none of the projects exceeds this limit. The six alternatives now under consideration are as follows:

Project	Present cost	Present benefit	B/C ratio
$A_3.A_2$	500	760	1,52
B_1	550	700	1,27
C	750	1 000	1,33
D_3	3 000	3 560	1,19
E	300	560	1,87
F_2	850	1 480	1,74

Project E is consequently chosen and R3 700 000 of the budget is left. The next project to include is F_2, which immediately places the more expensive projects in group F under the spotlight. The relevant incremental *B/C* ratios are as follows: $F_3.F_2$ = 1,40 and $F_4.F_2$ = 1,28. The former B/C ratio is compared with the remaining projects. There is R2 850 000 left to spend, and this eliminates project D_3, that is more expensive. D_2 takes the place of D_3 on the basis of the *B/C* ratio criterion. The list under consideration is now as follows:

Project	Present cost	Present benefit	B/C ratio
$A_3.A_2$	500	760	1,52
B_1	550	700	1,27
C	750	1 000	1,33
D_2	1 400	1 620	1,16
$F_3.F_2$	200	280	1,40

$A_3.A_2$ has the best *B/C* ratio and A_3 replaces A_2 as chosen project. This costs an additional R500 000, leaving R2 350 000, for spending. The incremental *B/C* ratios within group A are as follows: $A_4.A_3$ = 1,28, $A_5.A_3$ = 1,44 and $A_6.A_3$ = 1,33. The list of competing projects is now as follows:

Project	Present cost	Present benefit	B/C ratio
$A_5.A_3$	1 000	1 440	1,44
B_1	550	700	1,27
C	750	1000	1,33
D_2	1 400	1 620	1,16
$F_3.F_2$	200	280	1,40

Project $A_5.A_3$ has the largest *B/C* ratio, which means that A_3 is replaced at a cost of R1 000 000. This leaves only R1 350 000, and means that D_1 now replaces D_2 on the list of competing projects.

Project	Present cost	Present benefit	B/C ratio
$A_6.A_5$	500	560	1,12
B_1	550	700	1,27
C	750	1000	1,33
D_1	1 250	1 260	1,01
$F_3.F_2$	200	280	1,40

Project F_3 is chosen to replace project F_2, which leaves R1 150 000, and eliminates D_1. The following projects remain for consideration:

Project	Present cost	Present benefit	B/C ratio
$A_6.A_5$	500	560	1,12
B_1	550	700	1,27
C	750	1 000	1,33
$F_4.F_3$	300	360	1,20

Project C is now chosen, leaving R400 000. Since only $F_4.F_3$ falls within this limit, F_4 replaces F_3, leaving another R100 000 in the budget. Therefore, it is decided to fund A_5, C, E and F_4 at a total cost of R4 900 000. Benefits to the value of R7 800 000 are gained in the process.

In the last step small adjustments are made to increase the total benefits. The most attractive project eliminated on the grounds of the budget limit was B_1. Sufficient funds may be acquired to pay for B_1 if F_2 is funded instead of F_4. This leaves R60 000 of additional benefits at R50 000 of additional cost, and the final list of projects is thus A_5, B_1, C, E and F_2.

Underlying this selection procedure is the notion that the decision-maker should try to achieve the greatest possible benefit for every Rand spent.

2.7.5 Timing of project implementation

Unfortunately project viability itself does not reveal the optimum timing of project implementation. For the timing of project implementation, the project should be analysed with a range of investment timings to ascertain which one would yield maximum viability. The question posed here is not "whether" but "when". A project may pass the test of showing a positive net present value, but it may be a better project if it were delayed by one year. Delaying implementation would defer the capital expenditures but lose a year’s benefit.

When benefits are expected to grow continuously in the future, the First-Year Rate of Return (FYRR) could be applied as an investment-timing criterion. If the benefits accruing in the first year of operation exceed the time cost of the investment, i.e. the FYRR is higher than the prescribed discount rate, the project is timely and should go ahead right away. If the FYRR is lower than the prescribed discount rate, but the NPV is positive, commencement of project implementation should be postponed. If budgetary constraints limit the construction programme, the FYRR can be used as an aid to prioritising the projects showing similar degrees of economic viability.

The FYRR is calculated by dividing the sum of the benefits accruing in the first year of operation (i.e. the year subsequent to project completion) by the worth of the project's investment cost at the time of project completion, expressed as a percentage.

Alternatively defined the FYRR is the year-one worth of the benefits of a project accruing in the first year of operation expressed as a percentage of the present worth of its investment costs.

2.8 LEGAL ASPECTS RELATED TO WATER INFRASTRUCTURE

2.8.1 Introduction

The verdict in the leading case of Administrator, Natal v Stanley Motors (1960 1 SA 699 (A)) has been of major concern to designers of bridges and roads. The finding that:

"any one who designs and constructs a bridge and its approaches is under a duty to members of the public to do so in such a manner that it will be capable of resisting all the violence of the weather which may be expected to occur - although perhaps rarely"

has placed a serious question mark behind designs that have been based on design floods with limited return periods.

In order to clarify the legal aspects related to the hydrological and hydraulic design and maintenance of river crossings, Prof TJ Scott prepared a document (Committee of State Road Authorities, TRH 25 : 1994) in which he addresses the typical concerns which confront designers. This useful document titled, "Guidelines for the Hydraulic Design and Maintenance of River Crossings – Volume II: Legal aspects," [(2.4)] reflects an overview of the legal aspects and cases are provided to reflect an understanding of the issues involved. **A copy of the said document is included in an electronic format on the supporting CD, which is attached to this publication**.

The most important developments since 1994 concerning the legal aspects of drainage have been the implementation of the new National Water Act of 1998 as well as the Environmental Conservation Act of 1989.

2.8.2 The National Water Act (Act No 36 of 1998)

The Department of Water Affairs and Forestry has published general authorisations, which establish when licenses are required for activities that may have impacts by [(2.2)]:

- impeding or diverting the flow of water in a watercourse
- altering the bed, banks or characteristics of a watercourse
- removing, discharging or disposing of water found underground if it is necessary for the efficient continuation of an activity or for the safety of people.

2.8.3 The Environmental Conservation Act (Act No 73 of 1989)

This Act [(2.3)] applies *inter alia* to:

The construction and upgrading of roads, railways, airfields and associated structures outside the borders of town planning schemes.

Schedules have been prepared, but not finalised, with respect to:

- identification of geographic areas in which specified activities require environmental authorisation
- activities that require environmental impact assessment

- activities that require initial assessment.

Until the schedules have been finalised provincial governments are dealing with all applications.

Other relevant documents included on the supporting CD are:

The National Water Act (Act No 36 of 1998)
The Environmental Conservation Act of 1989.

2.9 WORKED EXAMPLES

The following three simple examples have been included to illustrate the use of the economic evaluation procedures. The supporting software is capable of determining the NPV, IRR and LCA for more complex income and expenditure streams. It is suggested that the supporting software be used to conduct sensitivity analyses.

2.9.1 Example 2.1 – Net Present Value

Evaluate which of the future income streams S1 or S2 is more favourable if the cost of capital is 10% on a yearly basis and the amounts realize at the beginning of the year.

Year	S1	S2
1	250	100
2	350	400
3	600	350
4	100	600
5	400	250

Solution Example 2.1

If you assume year 1 to be the base year then the NPV's of the two income streams are:
NPV_{S1} = R1 412.39
NPV_{S2} = R1 374.43

These calculations reflect that the income stream S1 is more favourable when comparing the Net Present Values (NPV).

The NPV was calculated using the following formula:

$$NPV = \frac{F}{(1+i)^n} \qquad \ldots(2.8)$$

Where:

F = future value
i = interest rate
n = periods

Each future value was brought back to present values and accumulated to obtain the total NPV for each income stream.

2.9.2 Example 2.2 – Present Value

Determine the current investment that should be made for the replacement of a R1,5 million installation (current cost) after 15 years, if the expected CPIX is 15 % and the return on a fixed investment is 8% p.a.

Solution Example 2.2

Firstly the future value (F) of the investment should be determined. The current installation (P) is worth R1 500 000 and the escalation will be 15 % for a 15-year period.

$$F = P(1 + i)^n \qquad \ldots(2.9)$$

$$F = 1\,500\,000\,(1 + 0{,}15)^{15} = R12\,205\,592$$

Now the current investment (P) should be calculated by discounting the future required value (F) by 8% per annum for the 15-year period.

$$P = \frac{F}{(1 + i)^n} \qquad \ldots(2.10)$$

$$P = \frac{12\,205\,592}{(1 + 0{,}08)^{15}} = R3\,847\,712$$

2.9.3 Example 2.3 – Internal Rate of Return (IRR)

Determine the Internal Rate of Return (IRR) for the following cash flow.

Year	Cash flow
0	-1 300
1	250
2	350
3	600
4	100
5	400

Solution Example 2.3

The internal rate of return is the rate where the $NPV_{income} = NPV_{expenditure}$

$$NPV_{income} = \frac{250}{(1+i)^1} + \frac{350}{(1+i)^2} + \frac{600}{(1+i)^3} + \frac{100}{(1+i)^4} + \frac{400}{(1+i)^5} \qquad \ldots(2.11)$$

$$NPV_{expenditure} = \frac{1300}{(1+i)^0} \qquad \ldots(2.12)$$

Equation (2.11) = Equation (2.12)

$$\frac{250}{(1+i)^1} + \frac{350}{(1+i)^2} + \frac{600}{(1+i)^3} + \frac{100}{(1+i)^4} + \frac{400}{(1+i)^5} = \frac{1300}{(1+i)^0}$$

Solving from this equation for “i”
IRR = 9,525%

2.10 REFERENCES

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2.3 Department of Environmental Affairs and Tourism. (1989). *Environmental Conservation Act.* Act No 73 of 1989.

2.4 CSRA. (1994). Committee of State Road Authority. *Guidelines for the Hydraulic Design and Maintenance of River Crossings – Volume II: Legal aspects.*

2.5 Department of Transport. (2005). *Road Infrastructure Strategic Framework for South Africa*. RISFSA.

CHAPTER 3 - FLOOD CALCULATIONS

M van Dijk and SJ van Vuuren

3.1 OVERVIEW OF THIS CHAPTER

This chapter covers the procedures that could be used to determine the design flood peaks for hydraulic structures and specific return periods. To be able to use these procedures a theoretical introduction is given which is then reinforced by worked examples.

Table 3.1 provides a Road Map for this chapter.

Table 3.1: Road Map for flood calculations

ROAD MAP 3

Typical problems		Input information	Worked examples	Supporting software	Other topics	
Topic	**Par.**		**Example number**		**Topic**	**Par.**
Calculate flood peaks for a small catchment area	3.5.1, 3.5.2, 3.5.3 & 3.5.4	Catchment area, slopes, run-off characteristics, mean annual rainfall	3.1	Utility Programs for Drainage	Design of lesser culverts or storm water conduits Design of surface drainage structures or components Flood line determination Flood routing	7 5 4.4 7.5
Calculate flood peaks for a large catchment area	3.4, 3.5.4 & 3.6	Catchment area, slopes, historical flood records, mean annual rainfall	3.2		Design of bridges and major culverts Scour estimation at major structures Flood line determination	8.2 & 8.3 8.4 4.4
Back calculating of a flood event's return period	3.4, 3.5 & 3.6, 4.2, 4.4	Flood event information, river characteristics, catchment area, slopes, run-off characteristics, mean annual rainfall	-	Utility Programs for Drainage & HEC-RAS	Risk and legal aspects	2.5, 2.8 & 8.2

3.2 INTRODUCTION

3.2.1 Terminology

The run-off that is generated within a catchment through precipitation will depend on the:

- characteristics of the storm event;
- the response characteristics of the catchment; and
- the influence of temporal storage on the run-off.

The temporal distribution of the run-off is reflected in a hydrograph, such as plotted in **Figure 3.1**. The flood peak (Q_P) is reached as soon as the entire catchment contributes to the flood, which is also referred to as the time of concentration (T_C).

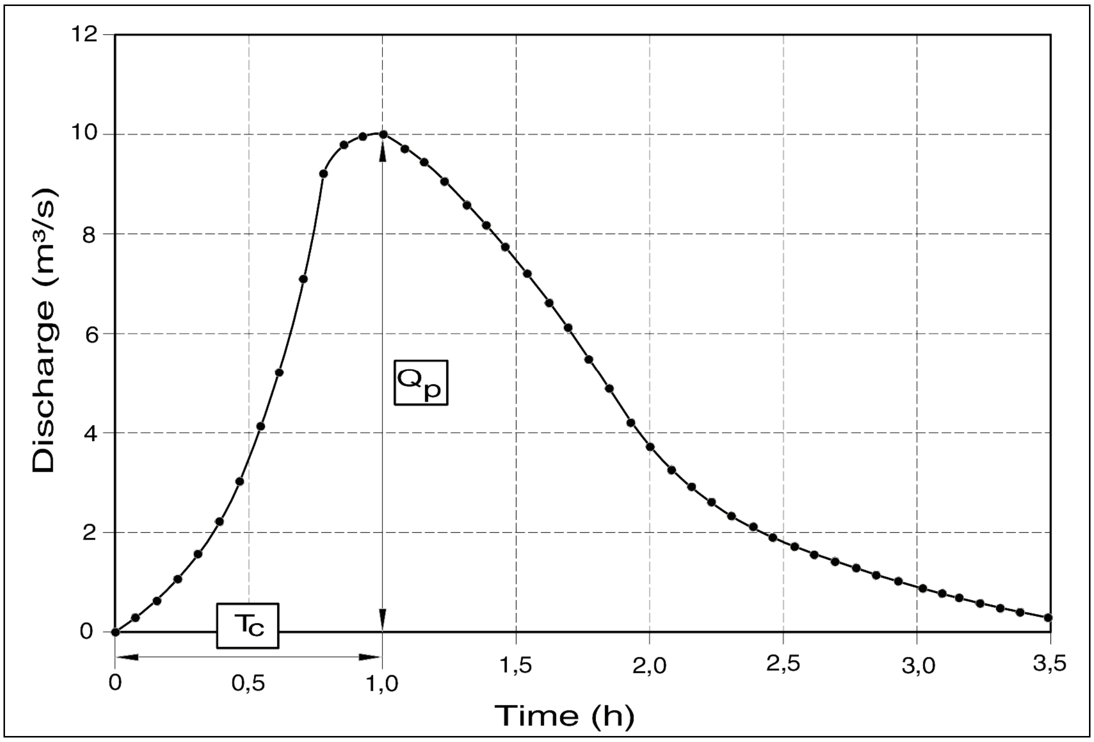


Figure 3.1: Typical flood hydrograph

The potential damage that a flood could cause to human and developments may be related to one or more of the following parameters:

- **High flood level** (HFL) - the maximum water level reached at a given point during the flood.
- **Peak discharge** (Q_P) - the maximum flow rate during the flood.
- **Maximum flow velocity** - the maximum calculated flow velocity associated with a given flow rate.
- **Flood volume** - the volume of water that is released from a catchment, responding to a given storm event and catchment characteristics.
- **Flood duration** - the period of time during which the discharge does not drop below a given limit.

Peak discharge is the most useful parameter in the calculation of the required cross-sectional area to convey a flood and to determine the backwater effect (upstream influence) of any structure that influences the normal flow conditions. The peak discharge is directly related to the characteristics of the storm event and response of the contributing catchment area.

Although the peak discharge does not remain constant as the flood progresses along a watercourse, changes are fairly gradual where there are no tributaries or local temporary storage. It could, therefore, be postulated that the peak discharge is independent of local changes in the watercourse, such as bed slope and cross-sectional shape. With the peak discharge having been determined, the high-flood level (flood line) and associated flow velocities may be determined by means of hydraulic calculations (uniform or gradually varied flow relationships). The flood volume and temporal variance of the flow rate can be derived from the hydrograph.

3.2.2 Different methods in use for the calculation of flood peaks

There are different hydrological calculation methods in use that can be applied to road drainage. The proven and most used methods in Southern Africa have been selected for inclusion in this Manual, and they are:

- Statistical methods
- Rational method
- Alternative Rational method
- Unit Hydrograph method
- Standard Design Flood (SDF) method
- Empirical method

It is good practice in the determination of design floods for bridges or large culverts to use more than one of the above methods, and if historical run-off data is available it should be analysed as well. It is always required to use the SDF method and compare with the other applicable methods. Where large discrepancies occur, an assessment should be conducted to motivate the selected design flood.

These methods have been developed by various institutions, and are either based on measured data (statistical); or on a deterministic basis (Rational, Unit Hydrograph and SDF methods); or are empirical relationships. Except for the statistical method, the methods were "calibrated" for certain regions and flood events, and are limited in terms of the size of the catchment areas on which they could be applied. **Table 3.2** lists the methods, input data requirements, maximum recommended catchment area for which each procedure can be used and references related to the procedures.

Table 3.2: Applications and limitation of flood calculation methods

Method	Input data	Recommended maximum area (km^2)	Return period of floods that could be determined (years)	Reference paragraph
Statistical method	Historical flood peak records	No limitation (larger areas)	2 – 200 (depending on the record length)	3.4
Rational method	Catchment area, watercourse length, average slope, catchment characteristics, rainfall intensity	< 15	2 – 100, PMF	3.5.1
Alternative Rational method		No limitation	2 – 200, PMF	3.5.2
Synthetic Hydrograph method	Catchment area, watercourse length, length to catchment centroid (centre), mean annual rainfall, veld type and synthetic regional unit hydrographs	15 to 5000	2 – 100	3.5.3
Standard Design Flood method	Catchment area, slope and SDF basin number	No limitation	2 – 200	3.5.4
Empirical methods	Catchment area, watercourse length, distance to catchment centroid, mean annual rainfall	No limitation (larger areas)	10 – 100, RMF	3.6

Methods that are not included in this manual are the SCS and Run hydrograph methods (see Section 3.7).

The procedures on which these methods are based are briefly discussed on the next page.

Statistical methods (Section 3.4) involve the use of historical data to determine the flood for a given return period. Their use is thus limited to catchments for which suitable flood records are available, or for which records from adjacent catchments are comparable and may be used, as described in detail in Section 3.4. Where accurate records covering a long period are available, statistical methods are very useful to determine flood peaks for long return periods. The method lends itself to extrapolation of data to determine flood magnitudes for longer return periods.

The **Rational method** (Section 3.5.1) is based on a simplified representation of the law of conservation of mass. Rainfall intensity is an important input in the calculations. Because uniform aerial and time distributions of rainfall have to be assumed, the method is normally only recommended for catchments smaller than about 15 km^2. Only flood peaks and empirical hydrographs can be determined by means of the rational method. Judgement and experience on the part of the user with regard to the run-off coefficient selection is important in this method, but thanks to improved methods, subjective judgement is becoming less important.

The **Alternative Rational method** (Section 3.5.2) is an adaptation of the standard rational method. Where the rational method uses the depth-duration-return period diagram to determine the point precipitation, the alternative method uses the modified recalibrated Hershfield equation as proposed by Alexander [3.1] for storm durations up to 6 hours, and the Department of Water Affairs' technical report TR102 for durations from 1 to 7 days.

The **Unit Hydrograph method** (Section 3.5.3) is suitable for the determination of flood peaks as well as hydrographs for medium-sized rural catchments (15 to 5 000 km^2). The method is based mainly on regional analyses of historical data, and is independent of personal judgement. The results are reliable, although some natural variability in the hydrological occurrences is lost through the broad regional divisions and the averaged form of the hydrographs. This is especially true in the case of catchments smaller than say 100 km^2 in size.

The **Standard Design Flood (SDF) method** (Section 3.5.4) was developed by Alexander [3.14] to provide a uniform approach to flood calculations. The method is based on a calibrated discharge coefficient for a recurrence period of 2 and 100 years. Calibrated discharge parameters are based on historical data and were determined for 29 homogeneous basins in South Africa.

Empirical methods (Section 3.6) requires a combination of experience, historical data and/or the results of other methods. Empirical methods are more suited to check the order of magnitude of the results obtained by means of the other methods.

3.3 FACTORS AFFECTING RUN-OFF

In flood hydrology it is essential to be familiar with and to understand the influence of the various factors affecting run-off before an attempt is made to undertake hydrological calculations. Such factors may be broadly classified as:

- topographical factors;
- antecedent soil moisture conditions;
- developmental influences; and
- climatological variables.

These factors are mutually dependent. Some of these important factors in the above-mentioned classes are discussed hereafter.

3.3.1 Topographical factors

3.3.1.1 Size of catchment

The size of a catchment has an important influence on the rainfall/run-off relationship, and consequently on the suitability of calculation methods. In small catchments for example, the relationship between rainfall intensity and infiltration rate of the soil is very important, whereas in large catchments the quantity of rainfall relative to the water storage capacity of the ground is more important. The peak discharges of small streams within the same geographical area are approximately proportionate to the sizes of the catchments (catchment area < 10 km²). As the catchment becomes large, the peak run-off becomes proportionate to $\sqrt{A}$. The effect of other factors, however, often decreases the influence of catchment size alone.

Topographical maps (1: 50 000) are usually used to determine the area of a catchment. However, for small catchments the accuracy and contour intervals on these maps are not acceptable and topographic detail on a smaller scale, say 1: 10 000, should be obtained. Ortho-photographs should be used, if available. It is considered essential for the designer to visit a catchment personally to obtain an impression of developments and other important characteristics of the catchment.

3.3.1.2 Catchment shape

Even if all other characteristics are the same, fan-shaped catchments (Storm 1) will give rise to higher peak flows than long, narrow catchments (Storm 2). The slope of the main watercourse and other factors may, however, neutralise this influence. When reduction factors are used to adjust rainfall intensities, the movement and intensity of a storm passing over the catchment should be considered, since using only the size of the catchment could be misleading as is illustrated in **Figure 3.2**.

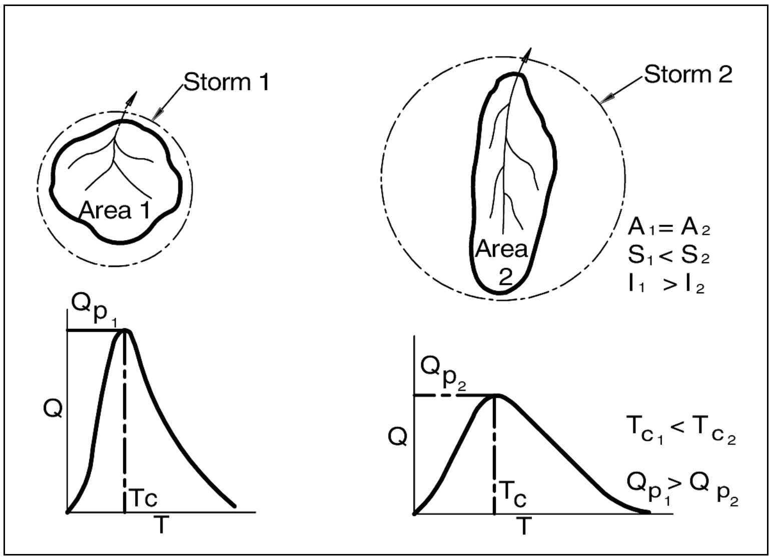


Figure 3.2: Catchments of the same size, but producing different peak discharges

3.3.1.3 Catchment slope

The slope of a catchment is a very important characteristic in the determination of flood peaks. Steep slopes cause water to flow faster and to shorten the critical duration of a flood-causing storm, thus leading to the use of higher rainfall intensities in the run-off formulae. On steep slopes the vegetation is generally less dense, soil layers are shallower, and there are fewer depressions, which cause water to run off more rapidly. The result is that infiltration is reduced and flood peaks are consequently even higher.

Generally there is a good correlation between the slopes of the main watercourse, tributaries and the surrounding landscape. The slope of the main watercourse is usually determined from topographical maps in the manner described in Paragraph 3.5.1.3.

3.3.1.4 Stream patterns

Well-drained catchments have shorter times of concentration and consequently give rise to larger peak flows. The hydraulic effectiveness of a watercourse, whether natural or man-made, affects the flow rates and, therefore, has to be taken into account.

Some streams have numerous tributaries, and others may have only one main watercourse, which receives run-off from overland flow. The meandering of watercourses, marshes and flows outside of river banks affect the flood's progress and increase attenuation of flood peaks.

3.3.1.5 Infiltration

Infiltration is the movement of water through the ground surface into the soil. Usually the infiltration rate is considerably higher at the start of precipitation than a few hours later. Soil moisture tension in the upper layers beneath the surface initially reinforces the effect of gravity to draw water into the soil. In time, however, the soil becomes increasingly saturated so that the tension decreases, capillary spaces become filled with water and infiltration takes place more slowly. Once the soil has become saturated, the surface infiltration rate becomes equal to the deeper infiltration rate to ground water (with interflow and evapotranspiration not considered).

3.3.1.6 Soil type and geology

Soil type has an important influence on the run-off, mainly because of the effect of the infiltration rate. The effect of the soil type also often depends on the volume, duration and intensity of rainfall. The condition of the soil at the onset of a storm will affect run-off. Freshly ploughed or unsaturated soil, for example, will produce a smaller run-off volume and peak discharge than compacted or saturated soil of the same type.

Underlying rock formations and other geological factors, such as riverine deposits, may have significant effects on run-off. Medium to large catchments with underlying dolomite, for instance, result in considerably reduced run-offs.

3.3.1.7 Seasonal effects of vegetation

Seasonal vegetation and falling leaves retard the flow of water and increase infiltration. Normally no provision is made in flood calculations for such seasonal effects.

Photograph 3.1: Variation in catchment cover

3.3.2 Developmental influences

3.3.2.1 Land use

Since human activities may well have a considerable effect on the run-off characteristics of a catchment, present and future conditions should be properly taken into account, particularly with regard to urbanisation. The effect of urbanisation depends on the percentage of the surface area that is made impermeable and on changes in the drainage pattern caused by storm water systems. Urbanisation usually increases the size of flood peaks by 20 to 50 per cent of those under natural conditions. Where there is industrial or other high-density building development, this figure may rise to 100 per cent or more. Examples of the influence of urbanisation on the peak discharge from a catchment as a function of impermeable surface area, return period and percentage area with storm water drainage are given in **Tables 3.3** and **3.4** [3.2]. The type of urban development in South Africa, where wall boundaries are common could, however, reduce the peak discharge rate while it increases the flow volume. Preliminary results support this conclusion. This is however dependent on the topographical characteristics as discussed in paragraph 3.3.1 and is site specific.

Table 3.3: Possible influence of % of impermeable surface area on peak flow, for different return periods, expressed as multiples [3.2]

Return period (years)	Percentage area consisting of man-made impermeable surfaces				
	1	10	25	50	80
2	1,0	1,8	2,2	2,6	3,0
5	1,0	1,6	2,0	2,4	2,6
10	1,0	1,6	1,9	2,2	2,4
25	1,0	1,5	1,8	2,0	2,2
50	1,0	1,4	1,7	1,9	2,0
100	1,0	1,4	1,6	1,7	1,8

Table 3.4: Possible influences on peak flow of % of impermeable surface area and % area with storm water drainage, expressed as multiples [3.2]

Percentage area with storm water drainage	Percentage of impermeable surface area					
	0	20	40	60	80	100
0	1,0	1,3	1,5	1,8	2,0	2,4
20	1,3	1,5	2,1	2,5	2,9	3,7
40	1,4	2,1	2,5	2,9	3,7	4,7
60	1,5	2,2	2,8	3,6	4,5	5,5
80	1,6	2,3	3,0	4,2	5,0	6,2
100	1,7	2,4	3,2	4,4	5,6	6,8

Photograph 3.2: Developmental influences (Loftus/University of Pretoria area)

3.3.2.2 Storage

Storage in a catchment occurs as detention storage (the filling up of small depressions in the ground surface), storage in overland (sheet) and river flows, as well as in pans, lakes, vleis and marshes. Storage could have a considerable effect on the attenuation and translation of flood peaks. Flood attenuation is discussed in **Chapter 7**.

3.3.2.3 Reservoirs

Reservoirs may intercept large volumes of run-off and thus considerably reduce peak flows. Generally it is realistic to assume that reservoirs would be reasonably full when conditions that favour large floods (large catchments) occur. The effect of interception by reservoirs can be investigated by assuming that the reservoir is full and conducting routing calculations. **Chapter 7** refers to the procedures (flood routing) that can be employed to undertake these calculations. Uncertainty regarding the operation of sluice gates during a flood, or prior to a flood event, to create storage volume for flood attenuation, complicates the assessment. The operational guidelines and policy for the release strategy should be investigated. A general assumption, however, is that the maximum controlled release from a dam should not be higher that the inflow peak.

3.3.3 Climatological variables

3.3.3.1 Climate

Climate has an important influence on many of the factors that affect run-off. Vegetation growth and soil formation, for example, are strongly affected by rainfall and temperature. There is a clear relationship between rainfall intensity and mean annual precipitation in different regions of South Africa. The wetter parts of the country generally experience higher rainfall intensities.

Areas with high rainfall generally also have wetter antecedent soil moisture conditions with correspondingly higher run-off from rainfall.

3.3.3.2 Rainfall as a flood parameter

In South Africa, rainfall is the most important form of precipitation, and together with hail, is mainly responsible for flood run-off. Snow does not contribute significantly to floods in SA, but hydrologically speaking contributes to low flow in certain regions. In large catchments the quantity, intensity and distribution of rainfall are important factors, but in the determination of flood run-off for small catchments, rainfall intensity remains the dominant factor.

The relationship between rainfall and run-off depends on many factors, which will be discussed later, and consequently cannot be simplified. Although the correlation between the rainfall return period and the resulting flood peak is poor, it has been found that when the peak run-off and rainfall are considered separately, the relationship between peak run-off for a given period and the rainfall intensity for the same return period remains reasonably constant for different return periods [(3.3)]. Rainfall could thus be used to determine design floods, although a rainstorm of a given return period very seldom results in a flood peak with the same return period.

3.3.3.3 Time and area distribution of rain storms

The run-off from a catchment depends not only on the intensity and quantity of rain, but is also affected by the duration, size, uniformity, velocity and direction of any storm passing over the catchment. Rain rarely falls evenly over a catchment, with the result that the rainfall inputs and flood run-offs vary across such an area. The point-to-point differences in the area and time distribution of rainfall depend, in turn, on the type of rain, for example, convection, orographic, frontal or cyclonic rain.

Convection rain occurs in the form of thunderstorms and tends to be extremely uneven and unpredictable; orographic rain also shows significant point-to-point differences but the distribution is more predictable; frontal rain is fairly evenly distributed along the longitudinal direction of the front, but there are marked differences in the direction of movement.

In contrast, cyclonic rains show fairly even distributions with the heaviest precipitation and intensity at the centre. The type of rain that would cause floods depends largely on the location and size of the catchment.

In most of the methods of calculation used in road drainage, it is assumed that the flood-causing storm has a precipitation duration just long enough to allow run-off from all parts of the catchment to contribute simultaneously to the flood peak, hence the relationship between the critical duration of a storm and the so-called time of concentration (T_C) as well as other methods used to measure catchment response time. In large catchments heavy rainfall over only a part of the area may also cause flooding, but the design floods for large catchments are mainly obtained via statistical analyses of measured discharges.

Storms that move over a catchment in the downstream direction often cause larger flood peaks than stationary or other storms, since in effect they shorten the time of concentration. Where the prevailing direction of storms is usually downstream, particularly within a long catchment, an indication of the possible effect of such storms could theoretically be obtained by shortening the time of concentration by the time the storm takes to move across the catchment. Whilst storms may move at speeds of up to 50 km/h, it is difficult to determine a design speed. Storms also rarely move in a straight line. Such adjustments are generally not made in practice.

Photograph 3.3: Rainfall over catchment (frontal)

Photograph 3.4: Convection rain

3.4 STATISTICAL METHODS

3.4.1 Applicability of statistical methods

Statistical methods are based on the fitting of probability distributions functions to measured values of maximum annual flood peaks. The accuracy of these methods depends a great deal on the reliability of the measured values, particularly the accuracy with which flow rates are measured, and on the length of the historical record. The latter should preferably be longer than half of the design return period, and should include both wet and dry periods. Statistical methods are reliable only when applied to a catchment of the same stream or at least of the same hydro meteorological region (possessing the same flood-causing characteristics and factors) as those on which the statistical fitting was done. The more compatible the catchment characteristics of the monitored catchment and those of the catchment under consideration, the more reliable the calculations are likely to be.

Since statistical methods are based on the use of historical events to predict future floods, any changes in the flood-causing factors within a catchment will also affect the reliability of the methods. Such changes should consequently be investigated and, where possible, be provided for.

In SA measured flow records for the hydrological years (October to September) are available from the Directorate of Water Affairs and Forestry (www.dwaf.gov.za), some local authorities, bodies such as the Rand Water Board and some universities. However, the measurements usually pertain to large catchments, and the statistical methods are thus only applicable to such areas.

3.4.2 Annual and partial series

An annual series is formed when the largest discharge value for every year during the recorded period is used as a basis for calculation. For a 20-year period there would thus be 20 values, representing the highest peak in every hydrological year, but not necessarily representing the 20 highest peaks.

In a partial series, all the recorded flood peaks are ranked in a descending order and the first number (equal to the number of years of data) of peaks is selected for the calculation. This selection procedure means that some of the annual peaks may not be included in the series, whereas more than one flood peak from some years may be included.

In road design, annual series are usually used that give considerably lower flood peak values for small return periods in comparison with partial series (**Figure 3.3**). The economic considerations are, however, based on annual series so that the prescribed return periods implicitly allow for the differences.

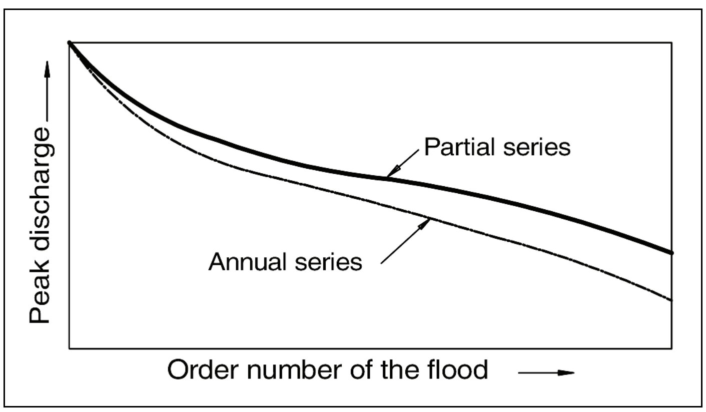


Figure 3.3: Comparison between annual and partial series

3.4.3 Overview of probabilities

The return period (T) is the average period over a large number of years during which an event (peak flow) repeats or exceeds itself. The annual probability of the occurrence of an event having a T-year return period equals:

$$P = \left(\frac{1}{T}\right) \quad \ldots (3.1)$$

The probability of an event with a return period of T to occur over a given design life of n years may be determined as follows [3.4]:

$$P_1 = 1 - \left(1 - \frac{1}{T}\right)^n \quad \ldots (3.2)$$

where:

P_1 = probability of at least one exceedence during the design life
n = design life in years
T = return period in years

P_X is the probability of x exceedences over the design life (n), and can be determined as follows:

$$P_X = n^c x P^x \left(1 - P\right)^{n-x} \quad \ldots (3.3)$$

or

$$P_X = n^c x P^x \left(1 - \frac{1}{T}\right)^{n-x} \quad \ldots (3.4)$$

where:

x = number of exceedences
$n^c x$ = number of combinations of n events taken x at a time

$$n^c x = \frac{n!}{x!\left(n-x\right)!} \quad \ldots (3.5)$$

A summary of the design return periods needed in order not to exceed an allowable risk of occurrence is given in **Table 3.5**.

Table 3.5: Return periods needed in order not to exceed given Probabilities of occurrence for different design lives

Probability of occurrence (%)	Life of project (years)				
	1	10	25	50	100
1	100	910	2 440	5 260	9 100
10	10	95	238	460	940
25	4	35	87	175	345
50	2	15	37	72	145
75	1,3	8	18	37	72
99	1,01	2,7	6	11	22

3.4.4 Methods of calculation used in statistical analysis

There are various statistical distribution functions that could be applied in the analysis of extreme values (flood peaks), but most of these are time-consuming unless performed by means of a computer. The graphic method is consequently recommended for hand calculations. The method aims to represent the distribution functions as a linear relationship on paper by means of specific divisions on the horizontal axis of the graph paper. Special graph paper is available for Normal and Gumbel distributions. The vertical scale may be linear or logarithmic.

Log-normal distribution functions are usually suitable for distributions in most parts of South Africa. However, the Gumbel distribution functions (a special case of the General Extreme Value (GEV) distribution functions)[3.1] yield better results in areas with relatively regular and high rainfall, such as the eastern parts of the country and the South and South-Western Cape.

The position in which every point should be plotted on the graph is calculated using the Weibull formula. The general equation is given below and the values for the constants *a* and *b* are provided in **Table 3.6**.

$$T = \frac{n + a}{m - b} \qquad \text{… (3.6)}$$

where:

T	=	return period in years
n	=	length of record in years
m	=	number, in descending order, of the ranked annual peak floods
a	=	constant (see **Table 3.6**)
b	=	constant (see **Table 3.6**)

If the horizontal axis has a probability classification, the probability (P) is calculated as:

$$P = \frac{1}{T} \qquad \text{… (3.7)}$$

Once the points have been plotted, a straight line is drawn through them to obtain the best fit. If it is not possible to obtain a good fit, another type of probability graph could be attempted. It is recommended that more than one type be tried to determine which type fits the historical data best. Poor fits may occur as a result of changes that have taken place with time in the catchment, or because of the unsuitability of the method being used. If the differences are explainable, correction factors should, where possible, be applied to the data on a deterministic basis. Otherwise alternative methods such as the Log-Pearson Type III (standard in the USA) or the "General Extreme Value" method (UK Flood Studies) [3.23] should be considered. Some of the commonly used plotting positions recommended for use in hydrological analyses are given in **Table 3.6**. If several distributions are plotted on a single graph then the general purpose Cunane plotting position should be used.

Table 3.6: Commonly used plotting positions [3.1]

Type	Plotting position	Distribution
Weibull (1939)	a = 1 & b = 0	Normal, Pearson 3
Blom (1958)	a = 0,25 & b = 0,375	Normal
Gringorten (1963)	a = 0,12 & b = 0,44	Exponential, EV1 & GEV
Cunane (1978) average of above two	a = 0,2 & b = 0,4	General purpose
Beard (1962)	a = 0,4 & b = 0,3	Pearson 3
Greenwood (1979)	a = 0 & b = 0,35	Wakeby, GEV

Detailed descriptions of direct statistical analysis methods are provided in Alexander's book *Flood Risk Reduction Measures* [3.1] (also see **Appendix 3A**). The reliability and accuracy of the historical record is extremely important, as well as the selection of the probability distribution function that best fits the data taking into account the outliers (high and low), zero flows, missing data and trends. **Figure 3.4** represents a typical example of a fitted distribution function; in this example Log Pearson Type 3 using Cunane plotting positions.

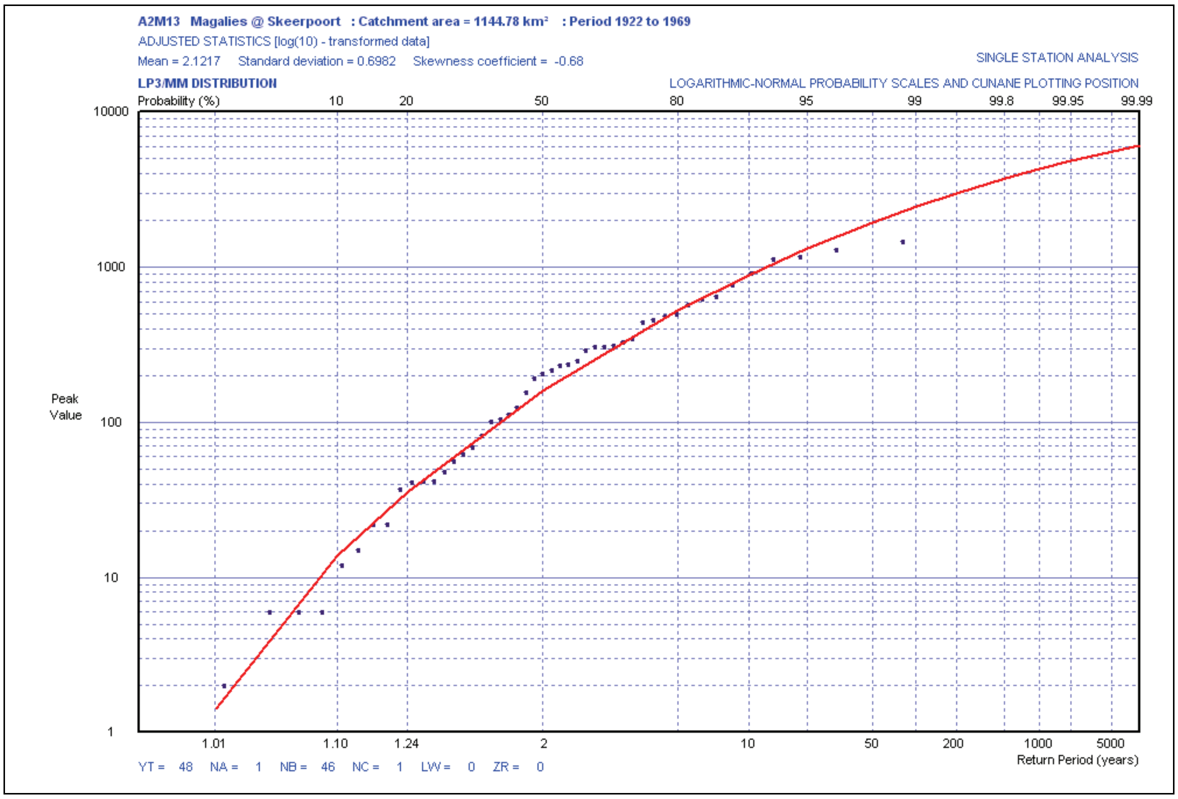


Figure 3.4: Example of graphical frequency analysis plot

If different statistical methods provide conflicting results for medium and large catchments where the impact in terms of the flood size is significant, it is recommended that an independent investigation by an experienced person be undertaken.

3.4.5 Use of short records in statistical analysis

Where the available flow records cover only a short period (± 5 years), these records could be extrapolated if reliable records from neighbouring measuring station(s) with comparable long records are available. It is good practice to limit the extrapolation of flow records to a maximum of two to three times that of the observed period.

For example if two flood records are available, record A with a limited record length and record B with a substantial length, then record A may be extrapolated by considering the partial series of all floods above a prescribed minimum for the short record (A) and for the corresponding period of the long record (B). In order to obtain a larger sample, all floods above the prescribed minimum are used, instead of simply a number equal to the number of years. The values of corresponding ranked order numbers are then plotted against one another on double logarithmic paper, and a straight line is fitted to show the relationship between flood peaks for stations A and B. The peak discharge of the desired return period is then computed for the long record in the usual manner, and is transferred to the station with the short record by using the determined logarithmic relationship between flow record A and B. **Figure 3.5** displays the logarithmic relationship between the two records used to obtain flow peaks for station A for longer recurrence intervals.

For greater reliability, relationships could be determined with more than one long-term record.

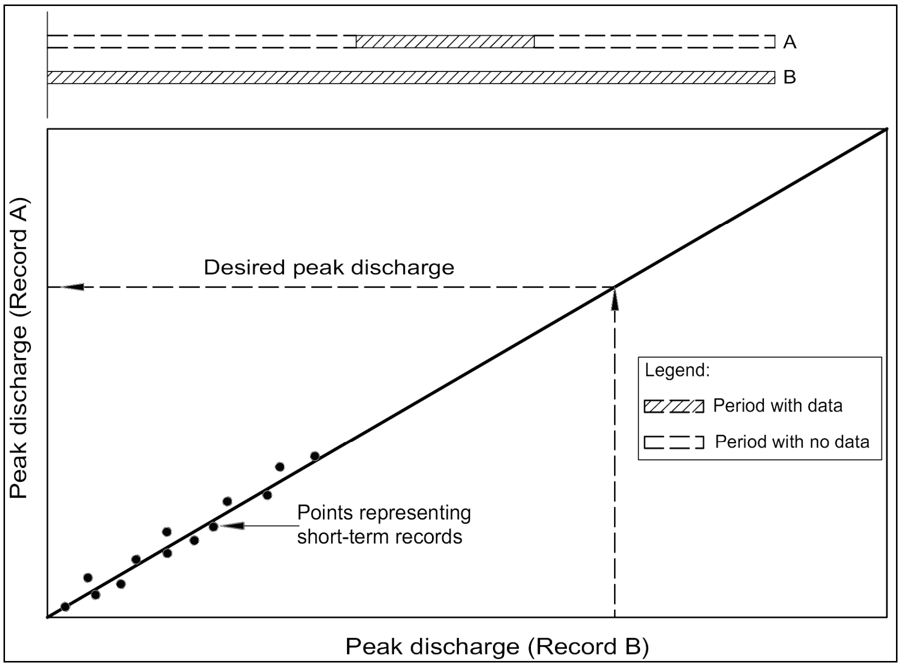


Figure 3.5: Example of extrapolation of record

3.5 DETERMINISTIC METHODS

3.5.1 Rational method

3.5.1.1 Background and principles

This method was first proposed in 1851 by the Irish engineer, Mulvaney. Since then it has become one of the best-known, and apparently the most widely used methods for determining peak flows from small catchments. The basis of the relationship is the law of the conservation of mass and the hypothesis that the flow rate is directly proportional to the size of the contributing area and the rainfall intensity, with the latter a function of the return period. The peak flow is obtained from the following relationship:

$$Q = \frac{CIA}{3{,}6} \qquad \text{… (3.8)}$$

where:
- Q = peak flow (m³/s)
- C = run-off coefficient (dimensionless)
- I = average rainfall intensity over catchment (mm/hour)
- A = effective area of catchment (km²)
- 3,6 = conversion factor

The rational formula represents outflow in the hydraulic continuity equation, and its application is based on the following assumptions:

- The rainfall has a uniform area distribution across the total contributing catchment.
- The rainfall has a uniform time distribution for at least a duration equal to the time of concentration (T_C).
- The peak discharge occurs when the total catchment contributes to the flow, occurring at the end of the critical storm duration, or time of concentration (T_C).
- The run-off coefficient, C remains constant throughout the duration of the storm.
- The return period of the peak flow, T, is the same as that of the rainfall intensity.

Despite this method's shortcomings and widespread criticism, it provides realistic results if it is used circumspectly, and it has generally provide good results in studies when compared with other methods. Although it is generally recommended that the method should only be applied to catchments smaller than 15 km², it can in some cases be used by experienced engineers for larger catchments [3.25].

Many of the assumptions listed above are to a greater or lesser degree also applicable to other methods of flood calculation.

3.5.1.2 Run-off coefficient (C)

3.5.1.2.1 Recommended C values

The run-off coefficient in the rational method is an integrated value representing the most significant factors influencing the rainfall-run-off relationship. It reflects the part of the storm rainfall contributing to the peak flood run-off at the outlet of the catchment. There is no objective theoretical method for determining C, and as a result the subjective elements of experience and engineering judgement play a very important part in the successful application of this method.

Table 3.7 [3.5] provides a description of recommended values of C.

Table 3.7: Recommended values of run-off factor C for use in the rational method

<table>
<tr><th colspan="5">Rural (C_1)</th><th colspan="2">Urban (C_2)</th></tr>
<tr><th rowspan="2">Component</th><th rowspan="2">Classification</th><th colspan="3">Mean annual rainfall (mm)</th><th rowspan="2">Use</th><th rowspan="2">Factor</th></tr>
<tr><th>< 600</th><th>600 - 900</th><th>> 900</th></tr>
<tr><td>Surface slope (C_s)</td><td>Vleis and pans (<3%)
Flat areas (3 to 10%)
Hilly (10 to 30%)
Steep areas (>30%)</td><td>0,01
0,06
0,12
0,22</td><td>0,03
0,08
0,16
0,26</td><td>0,05
0,11
0,20
0,30</td><td rowspan="3">Lawns
- Sandy, flat (<2%)
- Sandy, steep (>7%)
- Heavy soil, flat (<2%)
- Heavy soil, steep (>7%)

Residential areas
- Houses
- Flats

Industry
- Light industry
- Heavy industry

Business
- City centre
- Suburban
- Streets
- Maximum flood</td><td rowspan="3">
0,05 - 0,10
0,15 – 0,20
0,13 – 0,17
0,25 – 0,35

0,30 – 0,50
0,50 – 0,70

0,50 – 0,80
0,60 – 0,90

0,70 – 0,95
0,50 – 0,70
0,70 – 0,95
1,00</td></tr>
<tr><td>Permeability (C_p)</td><td>Very permeable
Permeable
Semi-permeable
Impermeable</td><td>0,03
0,06
0,12
0,21</td><td>0,04
0,08
0,16
0,26</td><td>0,05
0,10
0,20
0,30</td></tr>
<tr><td>Vegetation (C_v)</td><td>Thick bush and plantation
Light bush and farm lands
Grasslands
No vegetation</td><td>0,03
0,07
0,17
0,26</td><td>0,04
0,11
0,21
0,28</td><td>0,05
0,15
0,25
0,30</td></tr>
</table>

3.5.1.2.2 Rural areas

In rural areas there are five main factors affecting the value of C, namely catchment slope, permeability of the soil, vegetation, mean annual rainfall and return period.

Accurate and time-consuming calculation of the slope is not necessary. It may be determined with sufficient accuracy by selecting a representative part of the catchment on a contour map and determining a slope according to these contours. Steeper slopes give rise to higher run-off percentages.

The following classification could be used as a qualitative guide to the permeability of the soil:

- Very permeable - gravel, coarse sand
- Permeable - sandy, sandy loam
- Semi-permeable - silt, loam, clayey sand
- Impermeable - clay, peat rock

The classification could be made from a visual inspection of the terrain and/or by using the soil maps, available from the Government Printers. Where dolomite occurs, the following reduction factors are recommended for the dolomitic parts of a catchment to be applied to C_1 in **Table 3.7** on a pro rata basis (based on catchment slopes):

- Steep areas (slopes > 30%) - 0,50
- Hilly (10 to 30%) - 0,35
- Flat areas (3 to 10%) - 0,20
- Vleis and pans (slopes < 3%) - 0,10

Vegetation could be classified as follows:

- Forestry plantations
- Dense bush or bushveld
- Light bush and cultivated lands
- Grasslands
- No vegetation

As described in Section 3.3, the run-off increases as the density of the vegetation decreases. The vegetation should be determined by inspections *in loco*, although the publication by Acocks [(3.6)], "*Veld types of South Africa*" may also be useful.

Where the periodic felling of trees in forestry plantations could have a considerable influence on the run-off from a specific catchment, the C value should be increased by taking into account the proportionate part that would be left without effective plant cover. Where this proportionate part is greater than about 30 per cent of the catchment area covered with trees, the return period should be re-determined.

The mean annual rainfall also affects the run-off, as discussed previously. Recommended values of the catchment's response influenced by the slope C_s, permeability C_p and vegetation C_v are given in **Table 3.7** for different classes of mean annual rainfall.

The return period has an important effect on the run-off percentage. The relationship between rainfall and run-off is not linear and a catchment is often more saturated at the start of a storm with a long return period than is the case with storms of shorter return periods. It is thus recommended that the C value $C_1 = C_s + C_p + C_v$ be multiplied by the appropriate factor (F_t) from **Table 3.8**.

The influence of initial saturation is, however, also dependent on the catchment characteristics. The influence of the return period will thus be smaller for steep and impermeable catchments than for flat permeable catchments. For these cases the factors are given in **Table 3.8**.

Table 3.8: Adjustment factors for value of C_1

Return period (years)	**2**	**5**	**10**	**20**	**50**	**100**
Factor (Ft) for steep and impermeable catchments	0,75	0,80	0,85	0,90	0,95	1,00
Factor (Ft) for flat and permeable catchments	0,50	0,55	0,60	0,67	0,83	1,00

For the probable maximum flood (PMF), $C_1 = C_s + C_{pmax} + C_{vmax}$; $C_2 = 1$; and $F_t = 1$. Where C_{pmax} and C_{vmax} are in this case the maximum values from **Table 3.7**.

3.5.1.2.3 Urban areas
Recommended values of C for urban areas are given in **Table 3.7**. Because of the fairly large percentages of impermeable surface area in urban areas, it is normally not necessary to adjust the value of C according to the return period. Adjustment is, however, possible in accordance with **Table 3.8**.

3.5.1.3 Rainfall intensity (I)

The intensity of a design storm increases as the return period becomes longer and as the duration of the storm decreases. To obtain the largest possible peak discharge for a given return period using the rational method, the storm rainfall should have a duration equal to the time required for the whole catchment to contribute to run-off, defined as the time of concentration, T_C. If the storm has a shorter duration, it will not be possible for all the parts of the catchment to contribute simultaneously to run-off at the point of measurement. Consequently, the effective catchment area would be smaller than the actual area of the catchment.

Apart from the duration and return period, the intensity of rainfall is also related to the mean annual rainfall and to the rainfall region. The "depth-duration-return period" relationship depicted in **Figure 3.6** [(3.7)] may be used to determine point rainfall, which is then converted to intensity by dividing the point rainfall by the time of concentration ($P_{iT} = P_T / T_C$). In road drainage the volume of water that runs off as a result of a storm of less than 15 minutes duration is usually not large, and much of this run-off is absorbed in filling of the watercourses. Times of concentration of less than 15 minutes are thus generally not significant, and the maximum intensity is assumed to occur at approximately this time. It is difficult to calculate the rainfall intensity for storms less than 15 minutes (i.e. time of concentration of less than 15 minutes) and thus the intensity is based on the assumption that the storm duration is 15 minutes. The mean annual rainfall could be obtained from the simplified **Figure 3.7** or from the Weather Service [(3.8)] as well as other alternative sources [(3.9)] such as the Hydraulic Research Unit reports [(3.25)].

3.5.1.3.1 Time of concentration (T_C)
The time of concentration, T_C, is defined as the required time for a storm of uniform area and temporal distribution to contribute to the run-off from the catchment. In calculating the time of concentration, distinction is made between overland flow (sheet flow) and flow in defined watercourses.

(i) Calculation of the time of concentration for overland flow

This type of flow usually occurs in small, flat catchments or in upper reaches of catchments, where there is no clearly defined watercourse. Run-off, then, is in the form of thin layers of water flowing slowly over the fairly uneven ground surface. The Kerby formula is recommended for the calculation of T_C in this case. It is only applicable to parts where the slope is fairly even.

$$T_C = 0{,}604 \left(\frac{rL}{S^{0,5}} \right)^{0,467} \qquad \ldots (3.9)$$

where:

T_C = time of concentration (hours)

r = roughness coefficient obtained from **Table 3.9**

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)

S = slope of the catchment $S = \frac{H}{1000\ L}$ (m/m) (see **Figure 3.8)**

H = height of most remote point above outlet of catchment (m)

Table 3.9: Recommended values of r

Surface description	Recommended value of r
Paved areas	0,02
Clean compacted soil, no stones	0,1
Sparse grass over fairly rough surface	0,3
Medium grass cover	0,4
Thick grass cover	0,8

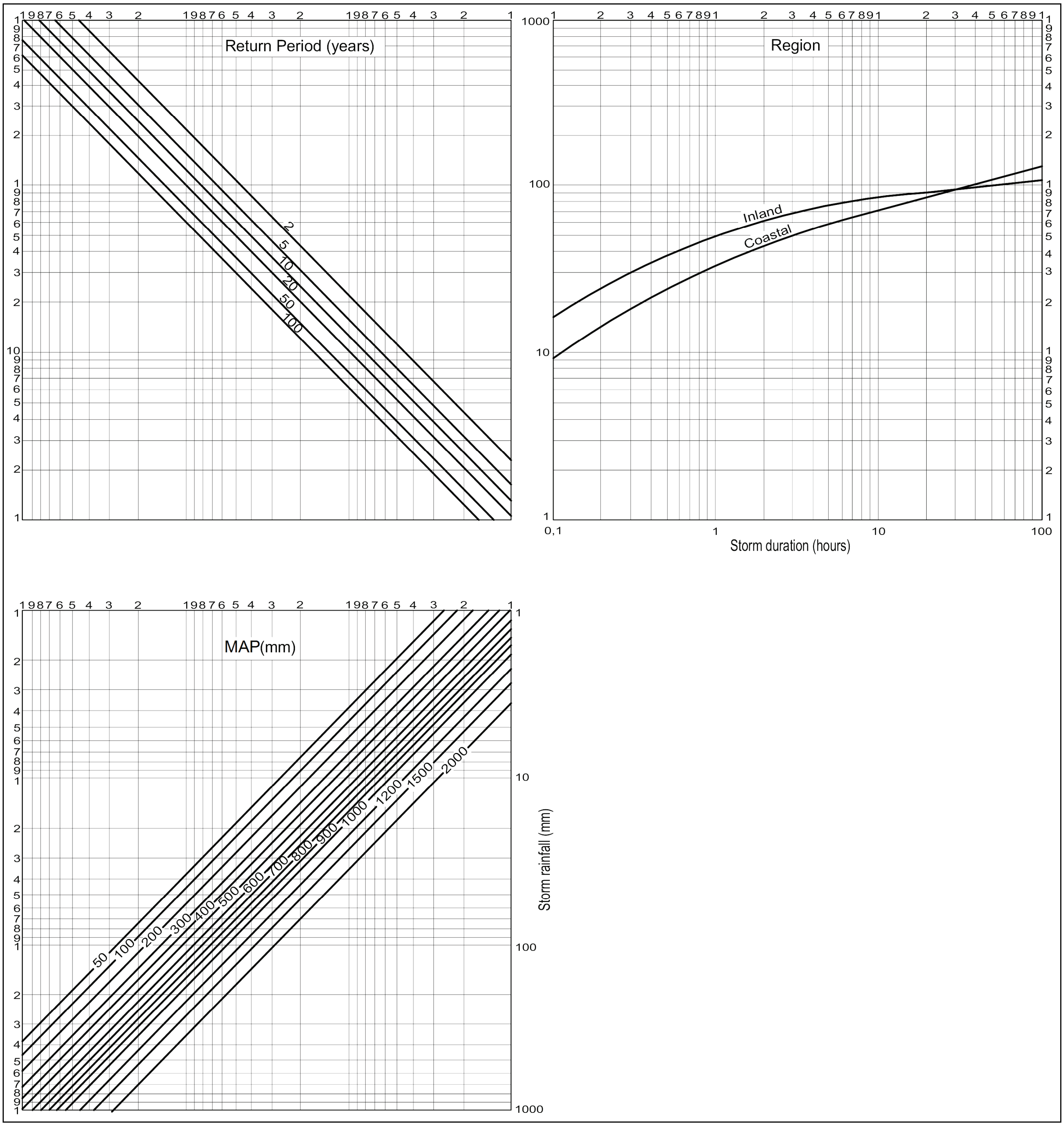


Figure 3.6: Depth-Duration-Frequency diagram for point rainfall

Figure 3.6 was obtained from HRU Report 2/78, *A Depth-Duration-Frequency Diagram for Point Rainfall in Southern Africa* (1978), which is an update from a similar diagram published in HRU Report 1/72, *Design Flood Determination in South Africa* in 1972 [3.7 & 3.25].

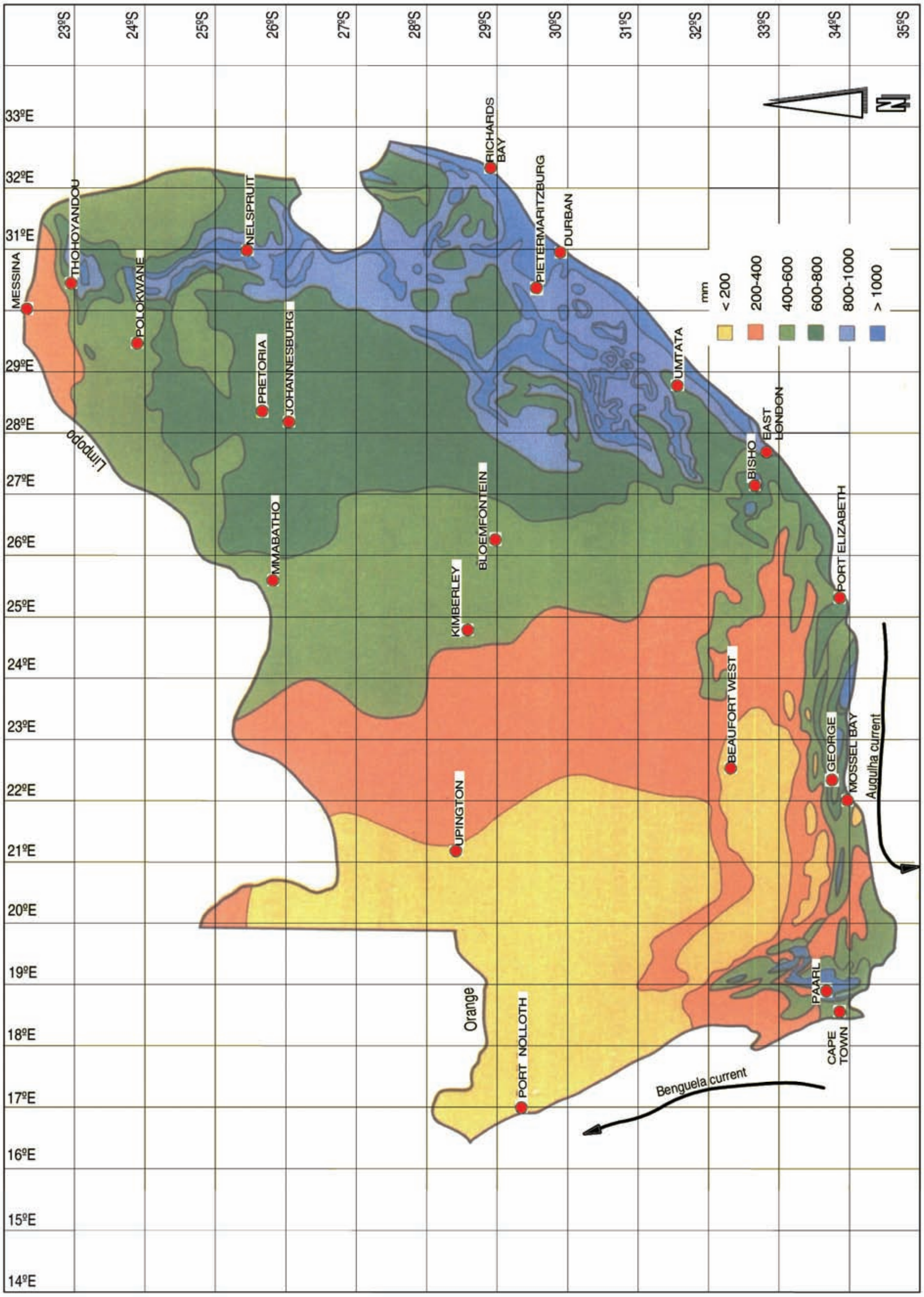


Figure 3.7: Average annual rainfall in southern Africa

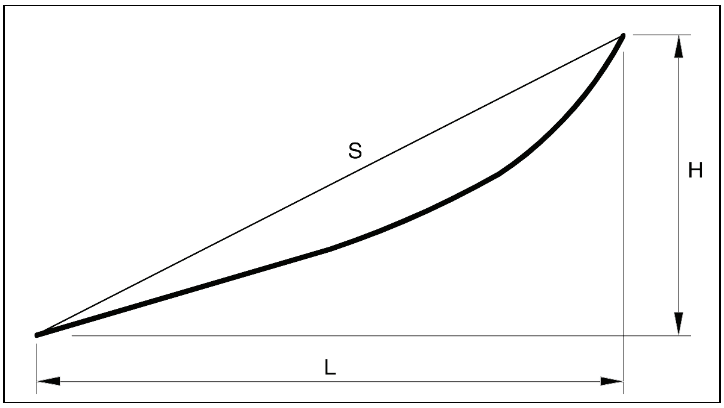


Figure 3.8: Slope definition for overland flow

(ii) Calculation of time of concentration for a defined watercourse

In a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels was developed by the US Soil Conservation Service.

$$T_C = \left(\frac{0,87L^2}{1\,000\,S_{av}} \right)^{0,385} \qquad \ldots (3.10)$$

where: T_C = time of concentration (hours)

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)

S_{av} = average slope (m/m)

The average slope may be determined graphically in two ways. The first procedure is based on the balance of areas obtained by balancing the areas above and below the line of average slope, as shown in **Figure 3.9**. Alternatively the formula developed by the US Geological Survey, and referred to as the 1085-slope method could be used (**Figure 3.10)**.

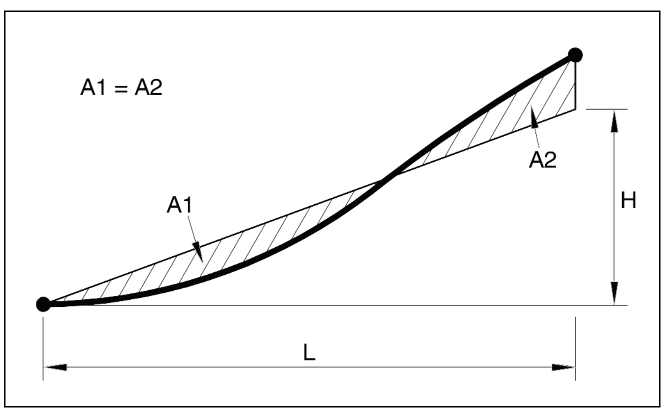


Figure 3.9: Slope according to weighted area method

In most cases the longest water path includes both overland and channel flow. In large catchments the channel flow is usually dominant, but in small catchments it may be necessary to determine T_C as the sum of the flow times, for overland and channel flow. To obtain a broad indication, it may usually be accepted that a defined watercourse exists when the average slope of the catchment is greater than 5 per cent, and the catchment itself is larger than 5 km².

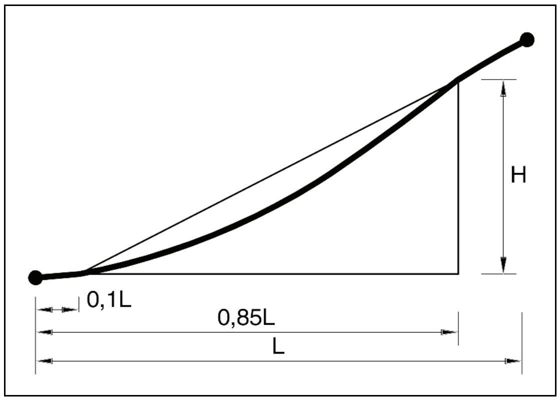


Figure 3.10: 1085-Slope according to "US Geological survey"

The formula for determining the slope according to the 1085 slope method reads:

$$S_{av} = \frac{H_{0,85L} - H_{0,10L}}{(1\,000)(0{,}75L)} \qquad \text{or} \qquad S_{av} = \frac{H}{(1\,000)(0{,}75L)} \qquad \ldots (3.11)$$

where:

S_{av}	=	average slope (m/m)
$H_{0,10L}$	=	elevation height at 10% of the length of the watercourse (m)
$H_{0,85L}$	=	elevation height at 85% of the length of the watercourse (m)
L	=	length of watercourse (km)
H	=	$H_{0,85L} - H_{0,10L}$ (m)

The height of waterfalls and high rapids are subtracted from the gross H value.

(iii) Calculation of the time of concentration for urban areas

In urban areas the time of concentration should be determined, where applicable, by means of the flow velocities according to the Chezy or Manning equation for uniform flow through representative cross-sections with representative slopes.

In road drainage the volume of water that runs off as a result of a storm of less than 15 minutes' duration is usually not large, and much of this run-off is absorbed in filling of the watercourses. Times of concentration of less than 15 minutes are thus generally not significant.

It is sound practice to calculate the average flow velocity $(v = L/T_C)$ after determining T_C in order to ensure that it falls within realistic limits. Typical values of the flow velocity range from 0,1 to 4 m/s, depending on the natural conditions.

3.5.1.4 Effective catchment area (A)

The effective area is that part of the total catchment which would contribute to the peak flow. Pans or areas that are artificially cut off should consequently be excluded.

3.5.1.5 Simplified hydrograph for the Rational method

Although the rational method is not strictly suitable for determining hydrographs, a simple triangular hydrograph could be used for low-risk application, such as flood routing through a culvert or the determination of the run-off volume. A typical triangular hydrograph is shown in **Figure 3.11**.

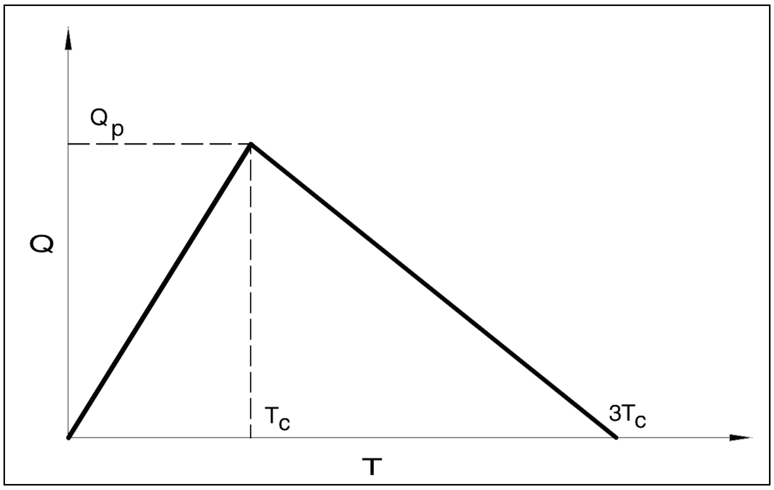


Figure 3.11: Simplified (triangular) hydrograph for the rational method

This is a highly idealised hydrograph with a shape, which is only approached in a long catchment of uniform slope and width. It should also be noted that the run-off volume is greater than the proportionate part of the storm rainfall that runs off during the time of concentration. As a result, real storms are usually of lower intensity both before and after their theoretical durations.

3.5.2 Alternative Rational method

3.5.2.1 Background and principles

This method is an adaptation of the traditional rational method. The main difference is in the calculation of the point precipitation. The rational method uses the depth-duration-return period diagram (**Figure 3.6**) to determine the point precipitation, whilst the alternative method uses the modified recalibrated Hershfield equation as proposed by Alexander[3.1] for storm durations of up to 6 hours, and the Department of Water Affairs' technical report TR102[3.15] for duration of 1 to 7 days.

The alternative procedure will thus be the same as the traditional procedure in determining the catchment area and run-off coefficients. The method of calculating the average rainfall intensity over the catchment is the main difference, and this is discussed in the following paragraphs.

3.5.2.2 Rainfall intensity (I)

As described for the rational method, the intensity of a design storm increases as the return period becomes longer and as the duration of the storm decreases. The time of concentration, based on the average catchment slope and distance that the water particles have to travel, is still calculated in the same manner.

The intensity of rainfall is related to the mean annual rainfall and to the rainfall region. The modified recalibrated Hershfield relationship [3.1] could be used to determine point rainfall, which is then converted to intensity by dividing the point rainfall by the time of concentration for storm durations of up to 6 hours.

$$P_{t,T} = 1{,}13(0{,}41 + 0{,}64\ln T)(-0{,}11 + 0{,}27\ln t)\left(0{,}79M^{0{,}69}R^{0{,}20}\right) \qquad \text{… (3.12)}$$

where:

$P_{t,T}$	=	precipitation depth for a duration of *t* minutes and a return period of T years
t	=	duration in minutes
T	=	return period
M	=	2-year return period daily rainfall from TR102 [3.15]
R	=	average number of days per year on which thunder was heard (**Figure 3.12**)[3.28]

For storm durations between 6 and 24 hours, linear interpolation is used between the calculated point rainfall from the modified Hershfield equation and the 1-day point rainfall from TR102[3.15]. For storm durations longer than 24 hours, linear interpolation is used between n-day rainfall values from TR102. Because two different methods are used to derive the point rainfall values, the calculated 6-hour rainfall value may be higher than the 24-hour value (from TR102). If the time of concentration is less than 24 hours and the Hershfield value is higher than the 24-hour value (from TR102), then it is reduced to equal the 24-hour value. This assumption is realistic as the storm precipitation mechanisms are such that short duration rainfalls exceeding 4 hours are often close to the 24-hour value.

A typical weather service station's particulars, together with data as provided in TR102, are shown in **Table 3.10** below with the rainfall station number reference grid shown in **Figure 3.13**.

Table 3.10: Typical format of the TR102 rainfall data

Weather Service station		Pretoria (Waterkloof)					
Weather Service station no		513437					
Mean annual precipitation		761 mm					
Coordinates		25° 47' & 28° 15'					
Duration (days)	**Return period**						
	2	**5**	**10**	**20**	**50**	**100**	**200**
1 day	69	98	121	146	183	215	250
2 days	86	121	149	178	222	259	301
3 days	96	137	169	204	255	299	347
7 days	120	170	208	247	305	353	405

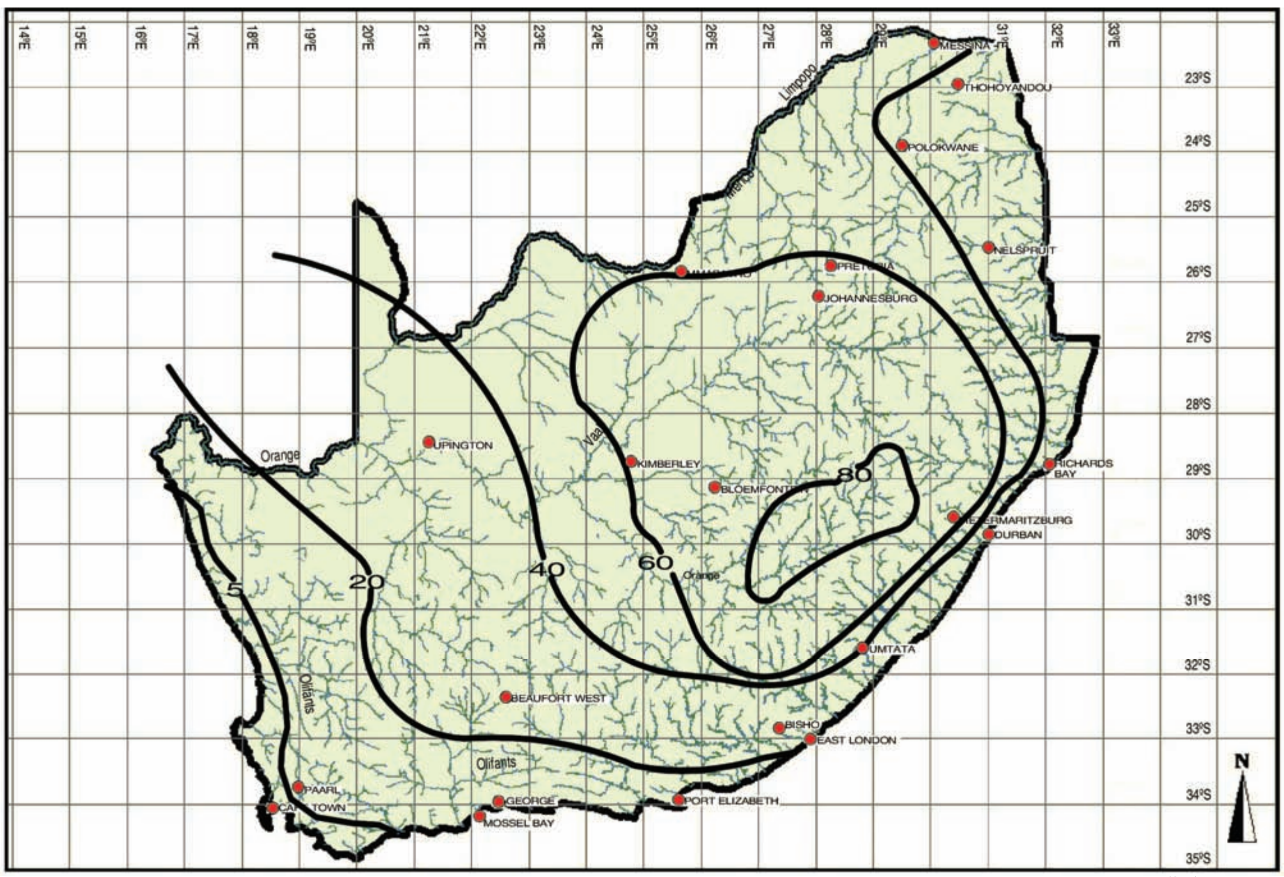


Figure 3.12: Average number of days per year on which thunder was heard (Alexander [3.1])

Detailed information on the number of days on which thunder was heard can be obtained from *Climate tables of South Africa (WB42)*[3.28].

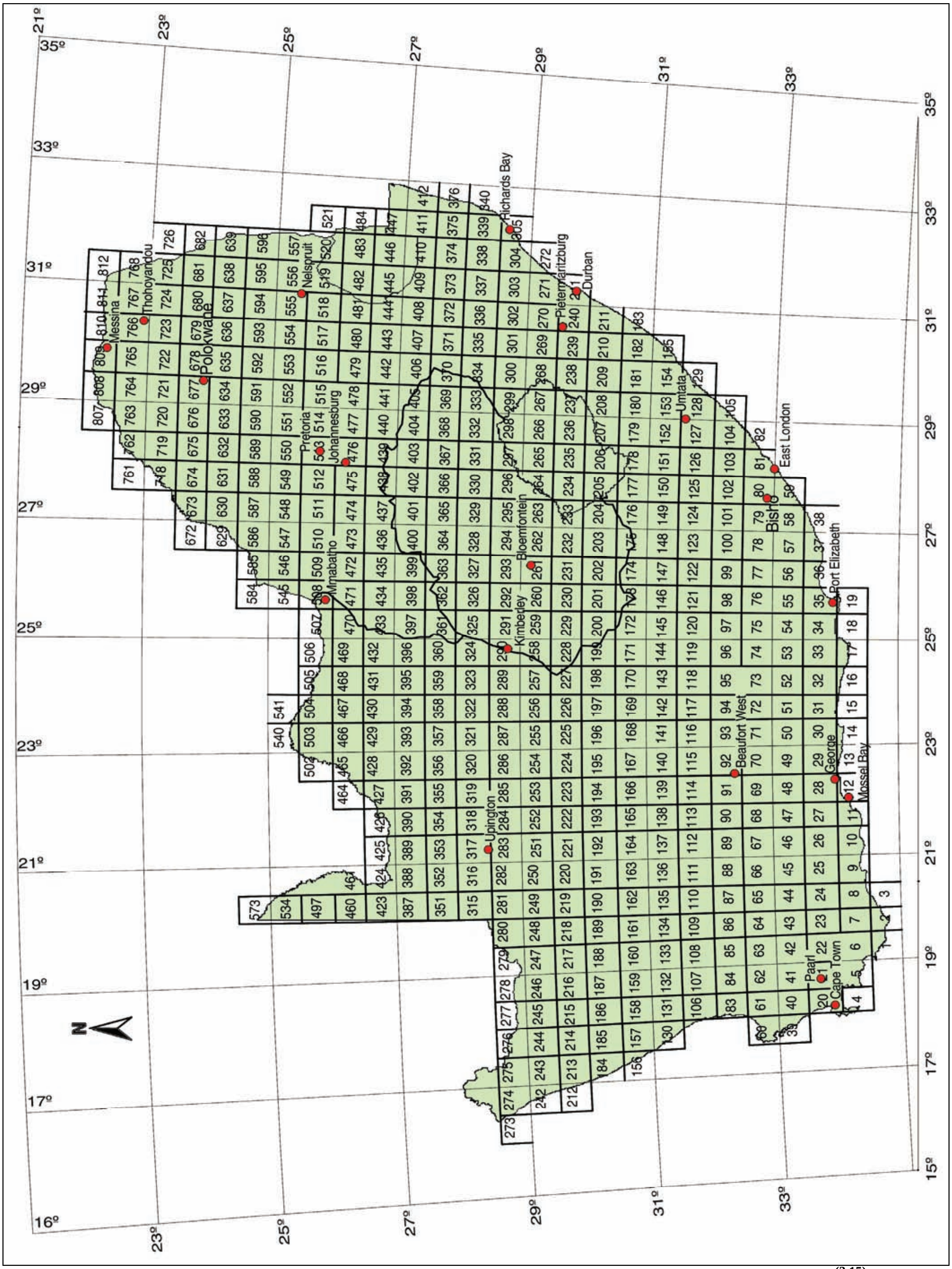


Figure 3.13: South African Weather Service rainfall station numbers (reference grid)[3.15]

Note: The reference grid refers to the first couple of digits of the rainfall station number

The area reduction factor may be determined using a graphical relationship (**Figure 3.14**) proposed by Alexander [(3.1)], or calculated utilizing Equation 3.13, which is based on the UK Flood Studies Report[(in 3.1)].

$$ARF = \left(90000 - 12800\ln A + 9830\ln\left(60T_C\right)\right)^{0,4} \qquad \text{… (3.13)}$$

where:

ARF	=	area reduction factor as a percentage (should be less than 100%)
A	=	catchment area (km²)
T_C	=	time of concentration (hours)

The area reduction factor may also be calculated based on the adjustment curves for point rainfall shown in **Figures 3.20** and **3.21**.

The peak flow rate is obtained using the Rational method's equation, Equation 3.8.

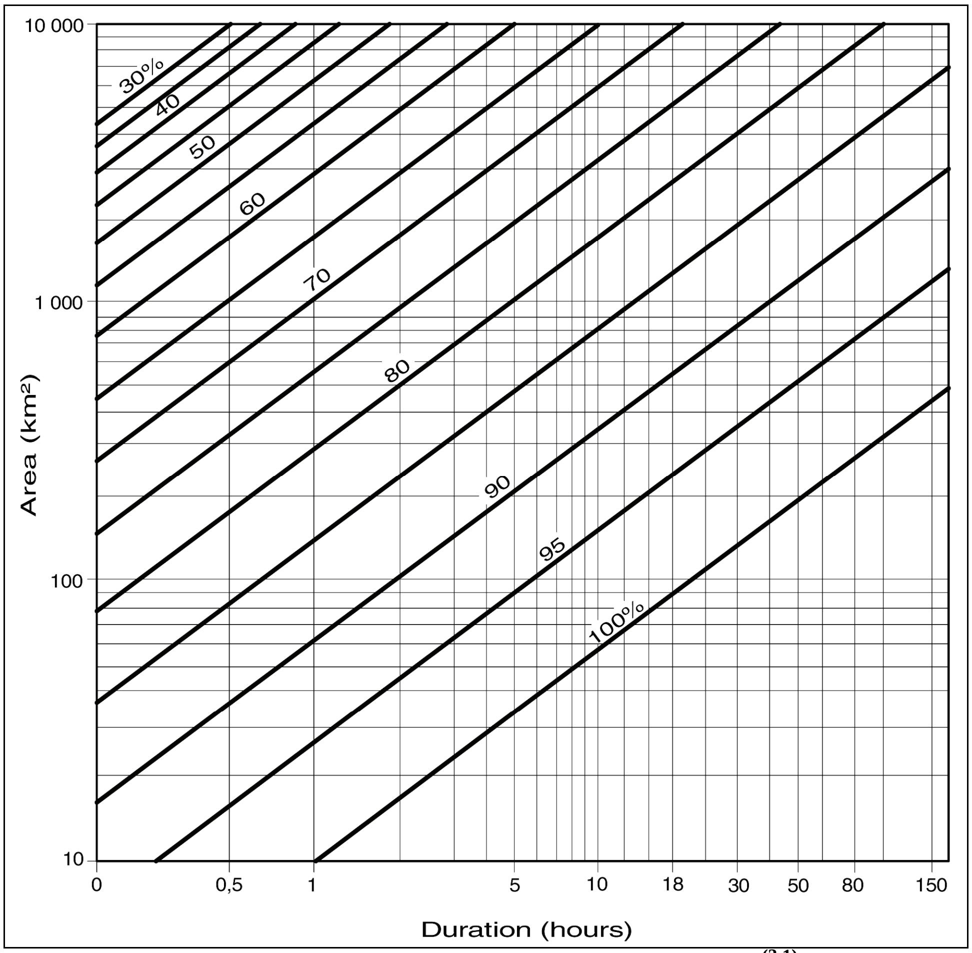


Figure 3.14: Area reduction factors (Alexander [(3.1)])

3.5.3 Unit Hydrograph method

3.5.3.1 Background

Unit hydrographs are applicable to catchments of between 15 and 5000 km^2. The specific recommended method is described in detail in Report 1/72 of the Hydrological Research Unit, University of the Witwatersrand [(3.7)]. Although the concept could also be extended to catchments larger than 5 000 km^2, this should be done only by engineers with the necessary skill and experience.

In large catchments the physical characteristics become increasingly complex and difficult to describe empirically. As discussed in Section 3.3, the significance of the characteristics of a catchment change in importance as the size of the catchment increases.

By using the concept of a unit hydrograph, the constant unique physical parameters of a catchment are established in the typical form of a hydrograph, and the size and duration may be further determined by considering the intensity and duration of rainfall. A unit hydrograph is a characteristic of a specific catchment, and is defined in metric terms as the hydrograph of one millimetre of run-off following rainfall of unit duration with uniform spatial and time distribution over the catchment. The duration of the hydrograph is thus proportional to the duration of the rainfall and the volume of the hydrograph is proportional to the intensity of the rainfall.

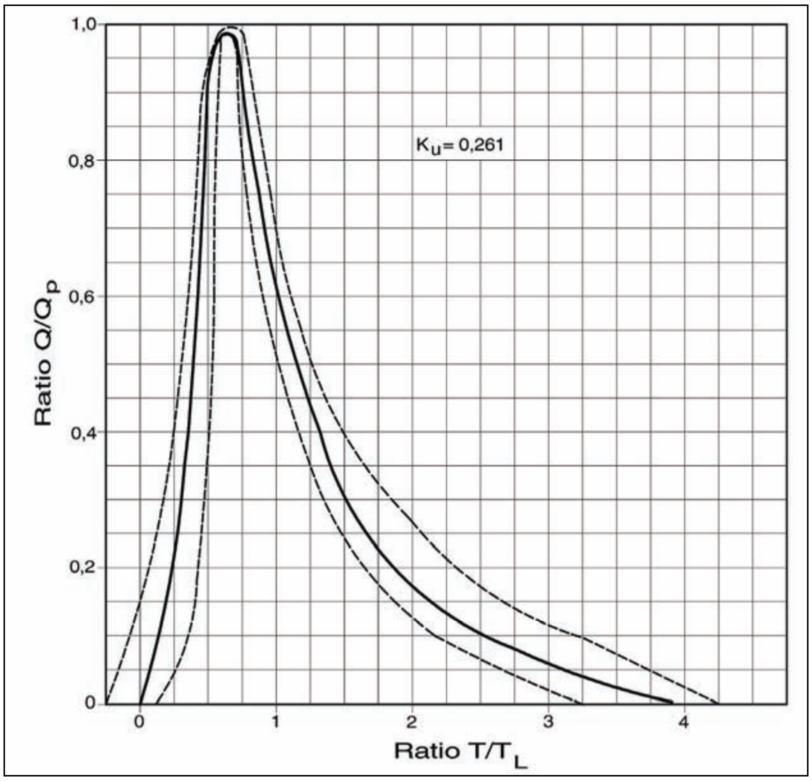


Figure 3.15: Unit Hydrograph with confidence bands

The Hydrological Research Unit has used historical data to derive unit hydrographs for 96 river measuring stations in South Africa. From these 96 unit hydrographs, nine synthetic hydrographs have been derived for nine regions in South Africa with similar catchment characteristics such as topography, soil type, vegetation and rainfall. A typical regional unit hydrograph with confidence bands is shown in **Figure 3.15** and the regional division in **Figure 3.16** [3.7].

3.5.3.2 Lag time and catchment index

Because of the general complexity of medium to large catchments, the lag time is not calculated arithmetically, but according to empirical curves. The ratios of lag time to catchment area indexes for the nine different regions are given in **Figure 3.17** based on veld type[3.7]. The catchment index is calculated by means of the following formula:

$$\text{Index} = \frac{L\,L_C}{\sqrt{S}} \qquad \ldots (3.14)$$

where:

- L = hydraulic length of catchment (km)
- L_C = distance between outlet and centroid of the catchment area (km)
- S = average slope of stream as for the rational method (m/m)

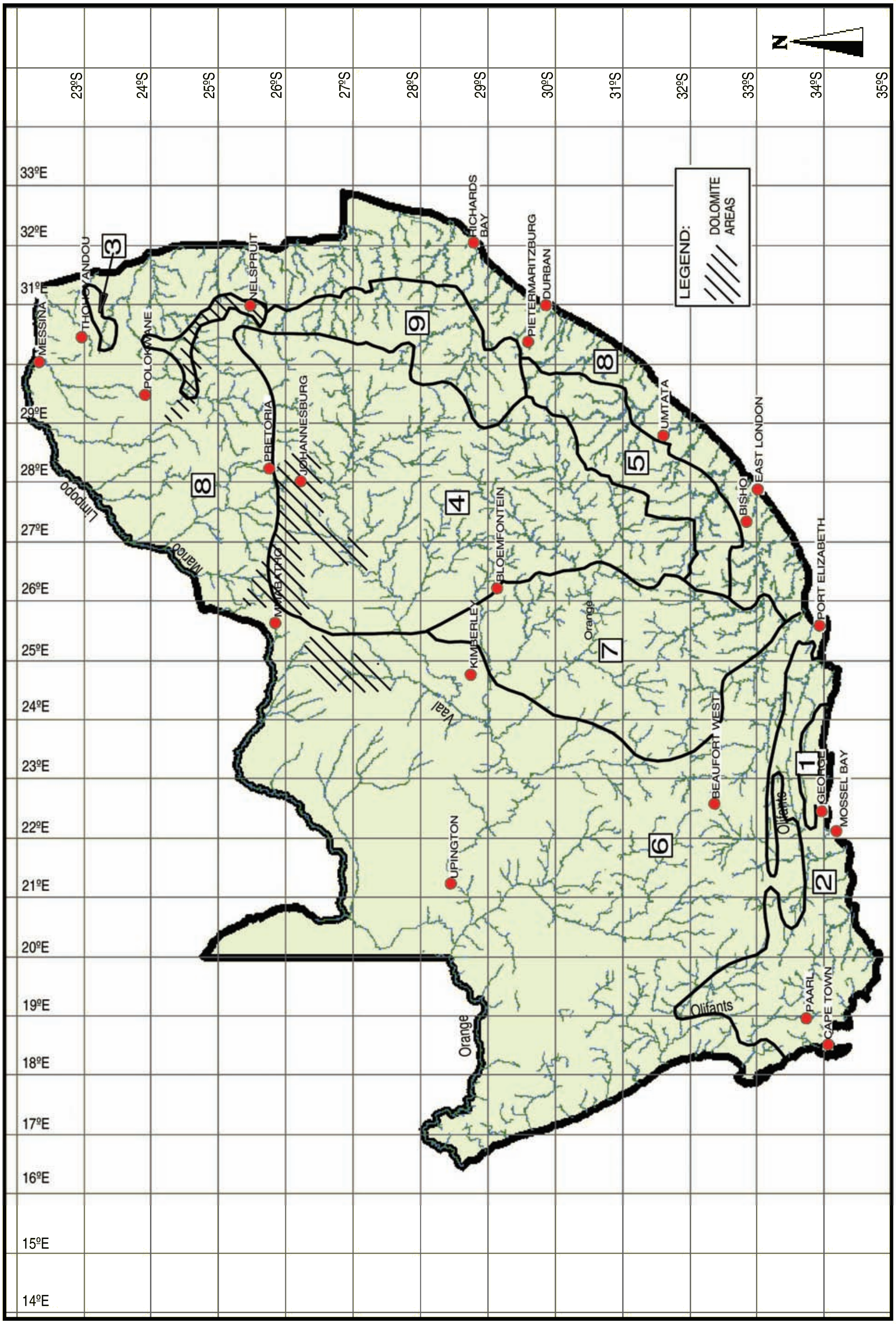


Figure 3.16: Regions with generalised veld types in South Africa

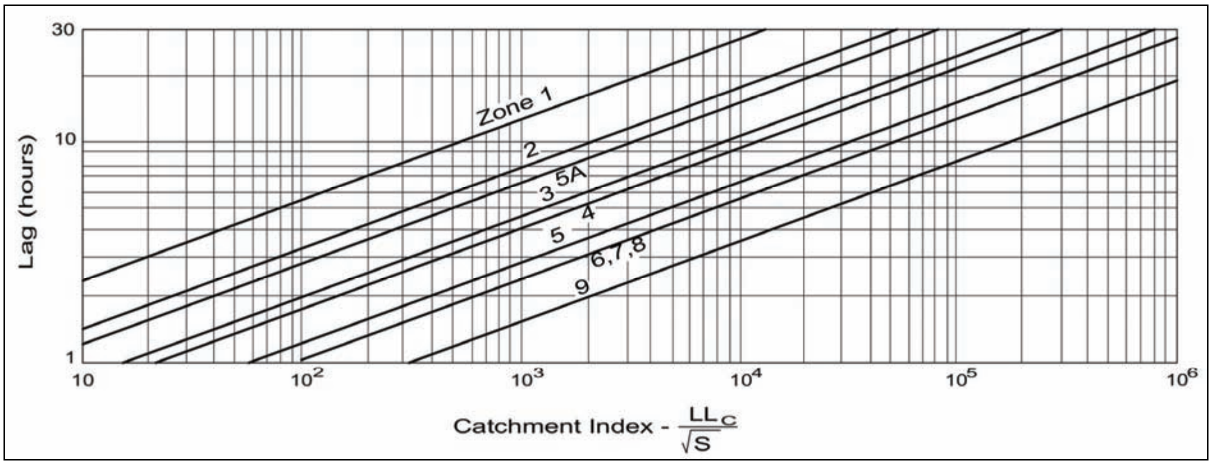


Figure 3.17: Ratio of lag time to catchment index

3.5.3.3 Rainfall input

The annual average rainfall for the area should be obtained from the South African Weather Service or from the simplified southern Africa rainfall map (**Figure 3.7**). The duration of storms that cause the maximum peak flows are obtained by trial and error. Point rainfall for various durations, normally shorter than or equal to the lag time, are obtained from **Figure 3.6**.

To calculate the probable maximum flood, the point rainfall could be obtained from **Figures 3.18a** and **3.18b**, and the maximum effective run-off from **Figure 3.19** (3.7). However, where the probable maximum flood is of greater importance, it is best to use the more accurate methods as described in paragraph 4.4(iii) of Report 1/72 (3.7).

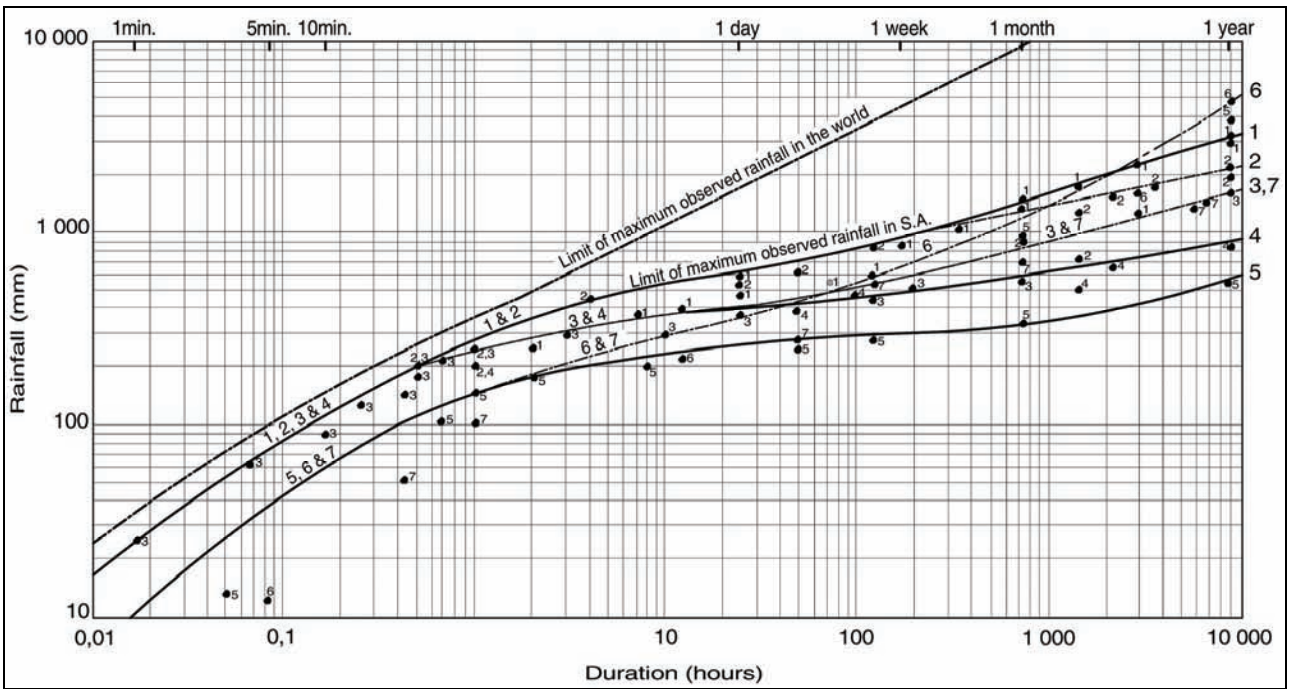


Figure 3.18a: Maximum observed point rainfall in South Africa

Note: Regions in Figure 3.18a are according to Figure 3.18b (3.7)

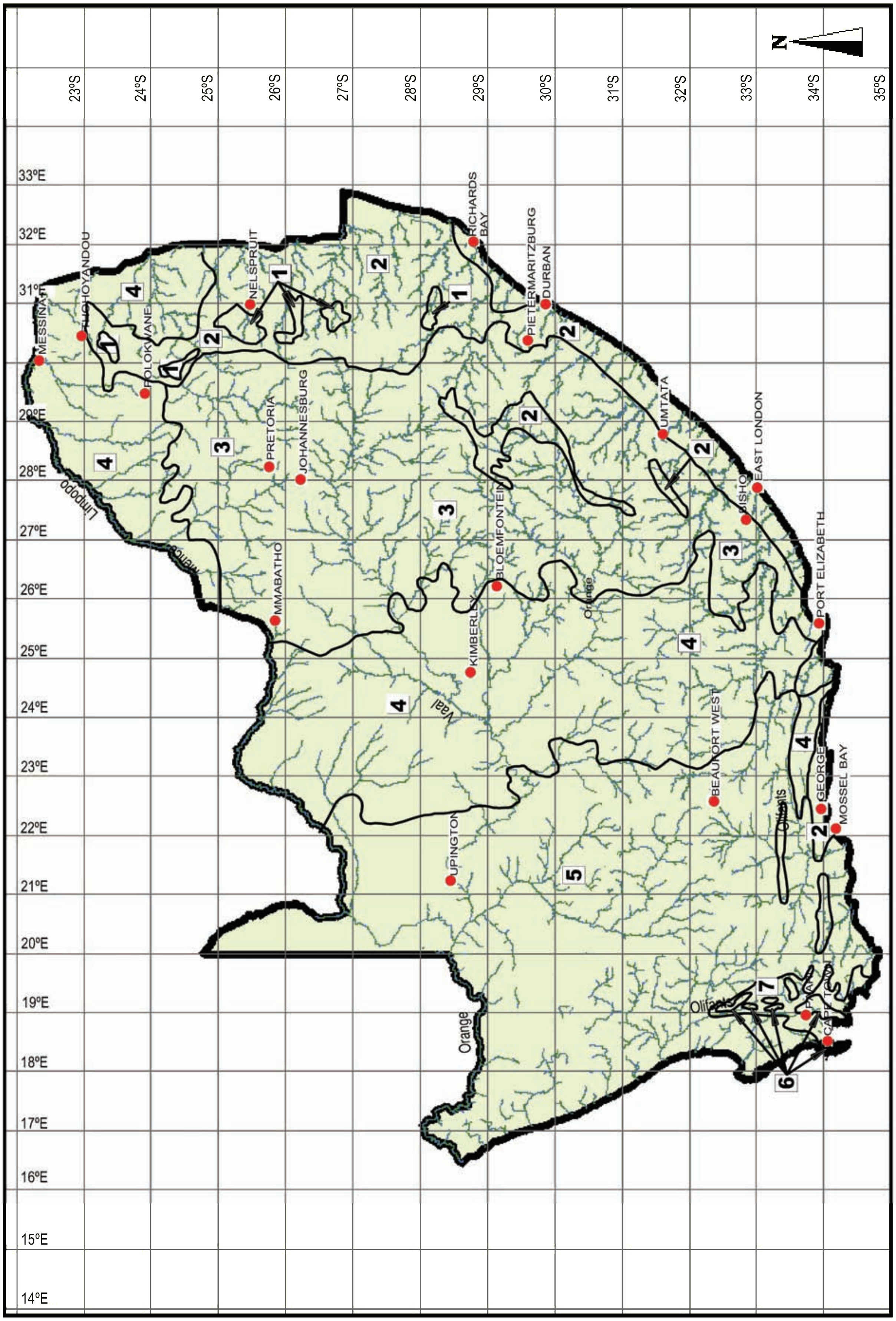


Figure 3.18b: Areas experiencing similar extreme point rainfalls

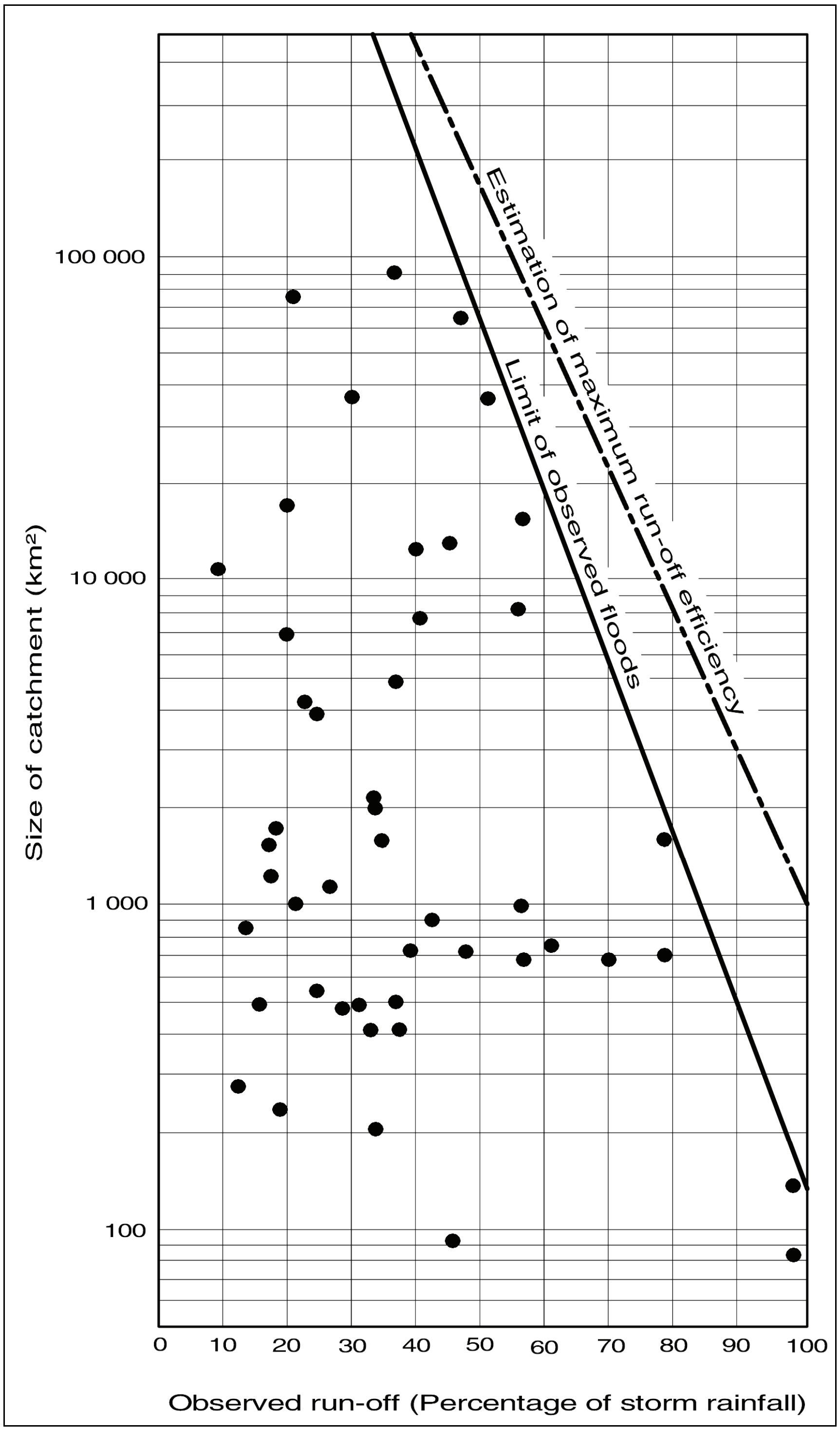


Figure 3.19: Expected maximum run-off percentage of storm rainfall

Due to the large sizes of the catchments for which the unit hydrographs are applicable, and also because of the area distribution characteristics of rainfall, the point rainfall should be adjusted to be representative for a catchment. A further distinction could be drawn between storms in small areas, smaller than 800 km², and storms in medium-sized areas, between 800 and 5 000 km². Adjustment curves for point rainfall, or so-called area reduction factors, are given in **Figure 3.20** for small and in **Figure 3.21** [3.7] for medium and large areas.

The percentage run-off from rainfall is not calculated empirically, but is determined from historical data for each region. The ratio between storm rainfall and percentage run-off for the different regions is given in **Figure 3.22** [3.7].

It should be noted that run-off as determined by means of the Unit Hydrograph method applies mostly to undisturbed rural areas. The percentage run-off as determined with the aid of **Figure 3.23**, and which is analogous to the rational C value, would thus have to be proportionately adjusted where considerable changes take place in the hydrological characteristics of a catchment.

3.5.3.4 Calculation of the flood peak with the Unit Hydrograph procedure

The K_U value is obtained from **Table 3.11** [(3.7)] for calculating the peak flow of the unit hydrograph using the formula:

$$Q_p = K_u \frac{A}{T_L} \qquad \text{… (3.15)}$$

where:

Q_P = unit hydrograph peak discharge (m^3/s)
A = size of catchment (km^2)
T_L = lag time (hours)

Table 3.11: Values of K_U for various veld types

Regional number (Figure 3.16)	**Factor K_u**
1	0,261
2	0,306
3	0,277
4	0,386
5	0,351
5a	0,488
6	0,265
7	0,315
8	0,367
9	0,321

The appropriate unit hydrograph is obtained from the regional classification in **Table 3.12** [(3.7)]. The unit hydrograph may then be dimensionalised by using the values of Q_P and T_L.

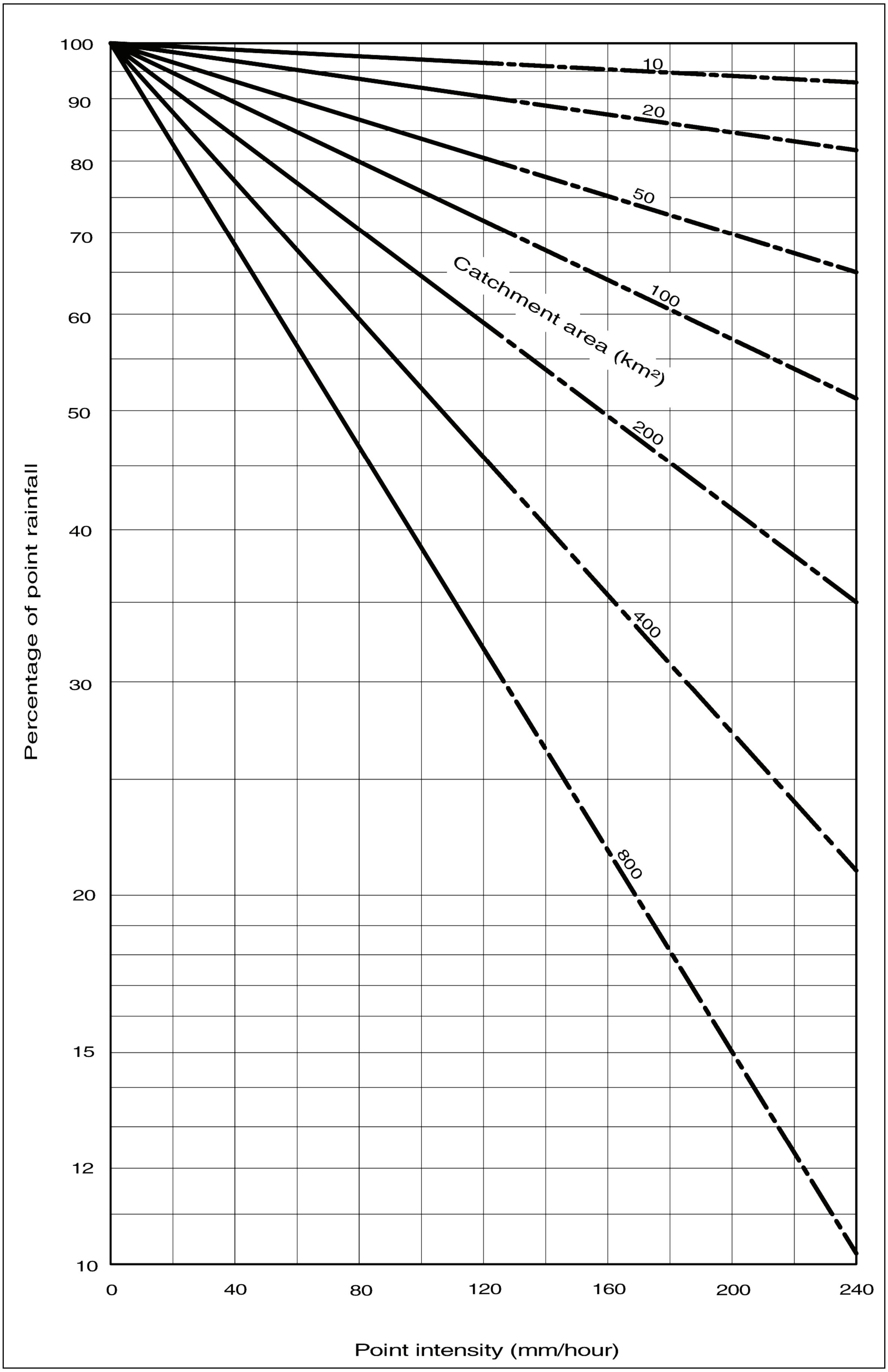


Figure 3.20: Expected percentage run-off as a function of point intensity (small areas), ARF

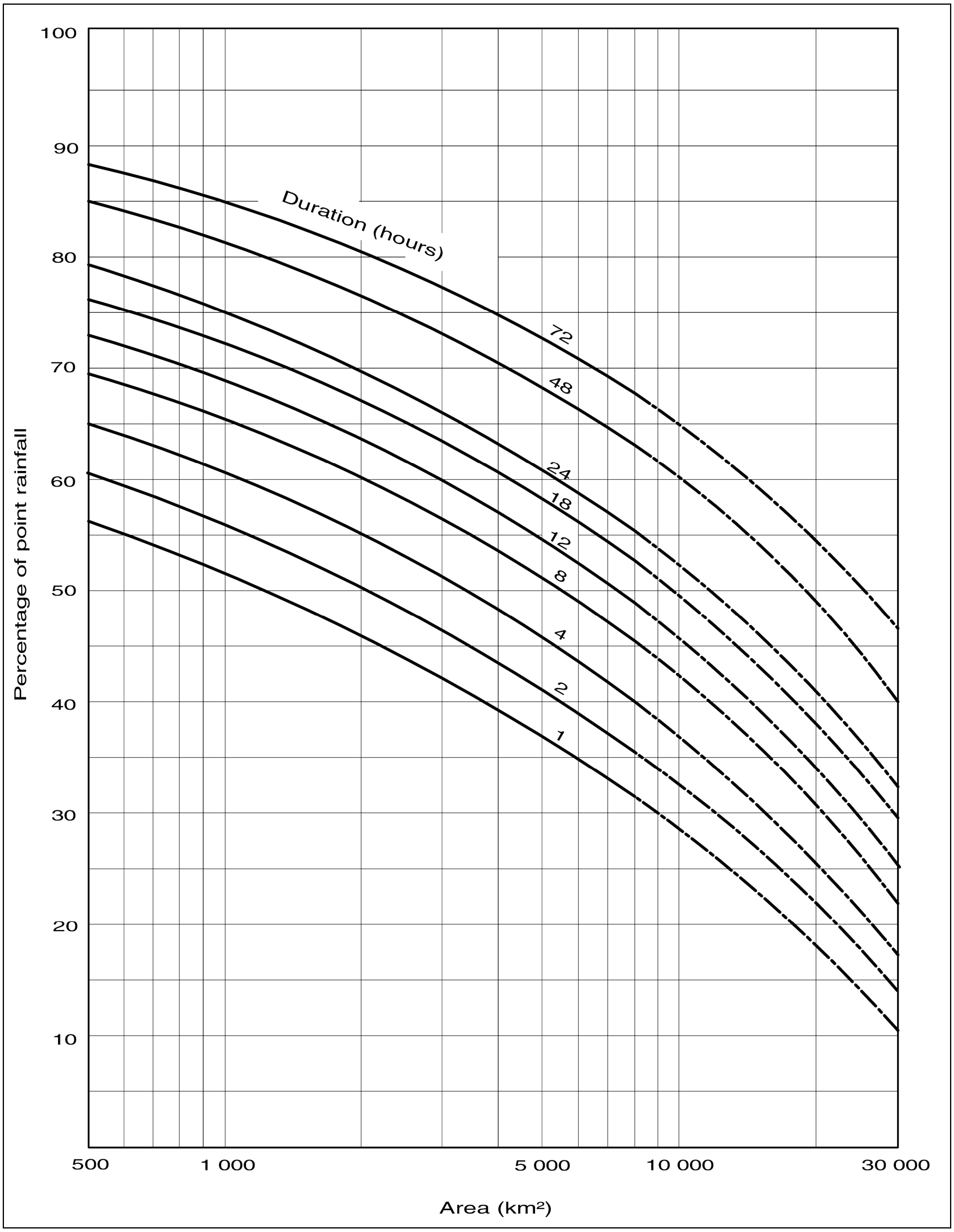


Figure 3.21: Expected percentage run-off as a function of storm duration (medium to large areas), ARF

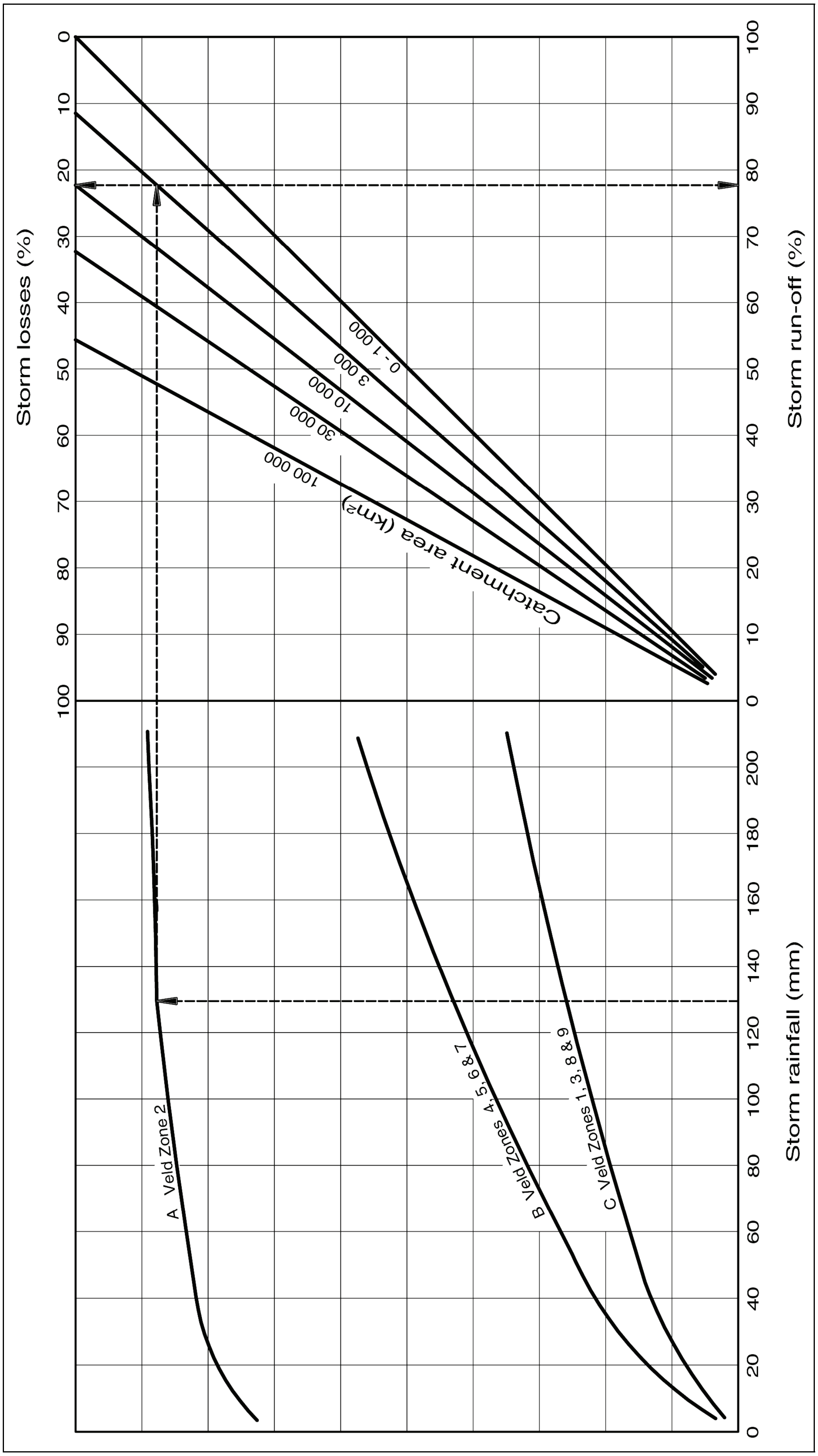


Figure 3.22: Average storm losses

Table 3.12: Dimensionless one hour unit hydrographs for various veld types

Time as T/T_L	Run-off as Q/Q_P for veld-type regions									
	1	2	3	4	5	5a	6	7	8	9
0	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,05	0,035	0,012	0,010	0,011	0,018	0,004	0,024	0,006	0,006	0,011
0,10	0,070	0,024	0,023	0,024	0,038	0,011	0,052	0,014	0,014	0,027
0,15	0,112	0,036	0,039	0,038	0,063	0,019	0,087	0,024	0,025	0,043
0,20	0,163	0,052	0,057	0,041	0,095	0,027	0,140	0,032	0,035	0,065
0,25	0,228	0,072	0,074	0,070	0,142	0,037	0,260	0,044	0,050	0,093
0,30	0,306	0,091	0,106	0,089	0,220	0,05	0,700	0,058	0,069	0,142
0,35	0,414	0,121	0,139	0,111	0,315	0,064	0,983	0,074	0,100	0,225
0,40	0,524	0,152	0,184	0,138	0,500	0,083	1,000	0,095	0,150	0,350
0,45	0,709	0,198	0,261	0,175	0,685	0,107	0,970	0,121	0,245	0,570
0,50	0,921	0,258	0,376	0,220	0,810	0,140	0,915	0,160	0,655	0,772
0,55	0,983	0,342	0,518	0,350	0,936	0,210	0,848	0,275	0,905	0,930
0,60	0,996	0,472	0,670	0,700	0,985	0,425	0,795	0,480	0,980	0,982
0,65	0,998	0,676	0,809	0,980	1,000	0,885	0,754	0,700	0,994	1,000
0,70	0,964	0,940	0,970	1,000	0,960	0,958	0,714	0,950	0,991	0,985
0,75	0,893	0,991	1,000	0,987	0,800	0,993	0,678	0,975	0,966	0,945
0,80	0,826	0,995	0,990	0,885	0,675	0,991	0,641	0,993	0,860	0,900
0,85	0,758	0,973	0,935	0,760	0,588	0,955	0,605	1,000	0,755	0,814
0,90	0,700	0,888	0,840	0,670	0,524	0,740	0,572	0,995	0,655	0,750
0,95	0,652	0,807	0,755	0,580	0,473	0,535	0,540	0,980	0,565	0,670
1,00	0,605	0,741	0,675	0,530	0,432	0,440	0,514	0,900	0,500	0,600
1,05	0,563	0,678	0,612	0,470	0,397	0,385	0,488	0,805	0,440	0,530
1,10	0,525	0,622	0,546	0,430	0,365	0,340	0,465	0,730	0,392	0,472
1,15	0,491	0,567	0,500	0,393	0,340	0,300	0,443	0,655	0,355	0,413
1,20	0,463	0,513	0,460	0,364	0,315	0,265	0,422	0,590	0,322	0,364
1,25	0,437	0,467	0,424	0,336	0,295	0,235	0,402	0,530	0,294	0,316
1,30	0,411	0,425	0,395	0,310	0,276	0,209	0,382	0,477	0,270	0,280
1,35	0,387	0,394	0,368	0,288	0,260	0,187	0,365	0,432	0,250	0,260
1,40	0,362	0,364	0,347	0,271	0,242	0,169	0,347	0,388	0,231	0,241
1,45	0,341	0,338	0,325	0,252	0,228	0,152	0,330	0,350	0,215	0,225
1,50	0,321	0,313	0,305	0,235	0,214	0,140	0,315	0,308	0,200	0,210
1,55	0,302	0,291	0,290	0,218	0,200	0,128	0,300	0,280	0,186	0,198
1,60	0,283	0,272	0,276	0,201	0,187	0,116	0,287	0,255	0,174	0,188
1,65	0,265	0,253	0,264	0,187	0,174	0,105	0,274	0,232	0,164	0,176
1,70	0,252	0,236	0,252	0,172	0,163	0,097	0,260	0,211	0,155	0,168
1,75	0,238	0,220	0,238	0,159	0,152	0,088	0,249	0,194	0,146	0,158
1,80	0,226	0,206	0,228	0,147	0,143	0,081	0,237	0,177	0,137	0,151
1,85	0,215	0,192	0,216	0,136	0,134	0,074	0,225	0,164	0,130	0,144
1,90	0,204	0,181	0,208	0,125	0,126	0,067	0,214	0,152	0,122	0,137
1,95	0,194	0,171	0,200	0,115	0,120	0,061	0,203	0,140	0,115	0,131
2,00	0,183	0,160	0,194	0,108	0,112	0,055	0,193	0,130	0,110	0,124
2,05	0,174	0,152	0,186	0,098	0,106	0,050	0,183	0,120	0,103	0,119
2,10	0,165	0,143	0,178	0,089	0,100	0,046	0,173	0,111	0,098	0,113
2,15	0,157	0,136	0,171	0,081	0,094	0,041	0,164	0,102	0,091	0,108
2,20	0,149	0,130	0,165	0,074	0,088	0,038	0,155	0,094	0,086	0,103
2,25	0,142	0,123	0,158	0,068	0,084	0,034	0,147	0,087	0,081	0,097
2,30	0,135	0,118	0,152	0,062	0,079	0,031	0,138	0,081	0,075	0,093
2,35	0,128	0,114	0,147	0,056	0,074	0,028	0,130	0,075	0,070	0,087
2,40	0,121	0,108	0,142	0,052	0,070	0,025	0,122	0,069	0,066	0,085
2,45	0,116	0,104	0,139	0,047	0,066	0,023	0,115	0,063	0,062	0,079
2,50	0,110	0,100	0,132	0,043	0,062	0,021	0,109	0,058	0,058	0,075
2,55	0,105	0,096	0,128	0,039	0,058	0,019	0,102	0,053	0,054	0,071
2,60	0,100	0,093	0,124	0,035	0,055	0,017	0,097	0,049	0,050	0,070
2,65	0,096	0,089	0,120	0,032	0,051	0,015	0,090	0,045	0,047	0,063
2,70	0,091	0,085	0,114	0,029	0,048	0,013	0,085	0,041	0,044	0,061
2,75	0,087	0,081	0,111	0,026	0,045	0,012	0,080	0,039	0,041	0,055
2,80	0,082	0,078	0,107	0,023	0,042	0,011	0,075	0,036	0,038	0,053
2,85	0,078	0,074	0,103	0,021	0,039	0,010	0,069	0,033	0,035	0,049
2,90	0,074	0,070	0,099	0,019	0,036	0,009	0,064	0,030	0,032	0,045
2,95	0,070	0,066	0,095	0,017	0,033	0,008	0,059	0,029	0,029	0,041
3,00	0,066	0,063	0,091	0,016	0,030	0,006	0,054	0,026	0,026	0,038
3,05	0,062	0,060	0,087	0,012	0,027	0,004	0,049	0,023	0,024	0,035
3,10	0,057	0,056	0,084	0,011	0,025	0,003	0,044	0,021	0,022	0,030
3,15	0,054	0,053	0,081	0,009	0,022	0,002	0,040	0,019	0,020	0,027
3,20	0,050	0,050	0,078	0,008	0,020	0,001	0,036	0,017	0,019	0,022
3,25	0,047	0,047	0,075	0,006	0,018	0,000	0,031	0,015	0,017	0,018
3,30	0,043	0,044	0,071	0,004	0,016	0,000	0,027	0,013	0,015	0,014
3,35	0,039	0,040	0,068	0,003	0,013	0,000	0,022	0,011	0,013	0,010
3,40	0,036	0,037	0,064	0,002	0,011	0,000	0,018	0,010	0,011	0,007
3,45	0,032	0,034	0,062	0,001	0,010	0,000	0,013	0,008	0,009	0,004
3,50	0,029	0,031	0,059	0,000	0,008	0,000	0,010	0,006	0,007	0,002
3,55	0,025	0,027	0,056	0,000	0,006	0,000	0,005	0,005	0,005	0,000
3,60	0,022	0,024	0,051	0,000	0,004	0,000	0,000	0,004	0,004	0,000
3,65	0,019	0,021	0,048	0,000	0,002	0,000	0,000	0,001	0,002	0,000
3,70	0,016	0,018	0,046	0,000	0,001	0,000	0,000	0,000	0,001	0,000
3,75	0,012	0,015	0,043	0,000	0,000	0,000	0,000	0,000	0,000	0,000
3,80	0,009	0,011	0,040	0,000	0,000	0,000	0,000	0,000	0,000	0,000
3,85	0,005	0,008	0,037	0,000	0,000	0,000	0,000	0,000	0,000	0,000
3,90	0,003	0,005	0,035	0,000	0,000	0,000	0,000	0,000	0,000	0,000
3,95	0,000	0,002	0,032	0,000	0,000	0,000	0,000	0,000	0,000	0,000
4,00	0,000	0,000	0,029	0,000	0,000	0,000	0,000	0,000	0,000	0,000
4,05	0,000	0,000	0,027	0,000	0,000	0,000	0,000	0,000	0,000	0,000
4,10	0,000	0,000	0,024	0,000	0,000	0,000	0,000	0,000	0,000	0,000
4,15	0,000	0,000	0,021	0,000	0,000	0,000	0,000	0,000	0,000	0,000
4,20	0,000	0,000	0,011	0,000	0,000	0,000	0,000	0,000	0,000	0,000
4,25	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000

3.5.3.5 Changes in hydrograph duration

Since the standard duration of a unit hydrograph is one hour (from the one-hour rainfall), the duration should be increased or decreased to make provision for other rainfall durations. (Hydrographs of half an hour or other durations can also be used as units).

To increase the duration of a unit hydrograph, a one-hour hydrograph may be added to a second (similar) hydrograph, with the second hydrograph's origin starting one hour later on the time scale. The resulting added values should then be divided by two to obtain a two-hour unit hydrograph. The principle is illustrated in **Figure 3.23** and may also be applied to other multiples of whole numbers.

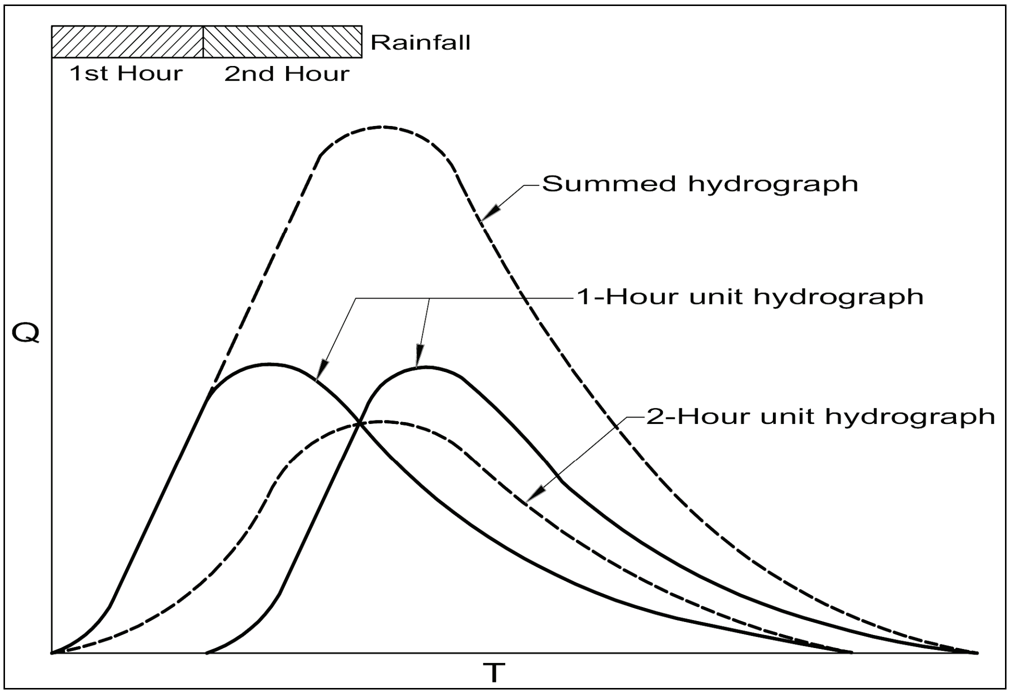


Figure 3.23: Increasing unit hydrograph duration

However, when the duration has to be decreased, or increased in fractions of the original unit hydrograph duration, the S-curve technique should be used. An S-curve is obtained by staggering/lagging a number of unit hydrographs by the unit duration and then summing them as shown in **Figure 3.24**.

In practice there are often oscillations of the summed values of the unit hydrograph compiling the S-curve. These fluctuations result from $(\Delta Q_1 - \Delta Q_2)_{\Delta t} < 0$ (demonstrated in **Figure 3.37**). This can normally be ascribed to time intervals being too large for accurate calculation, or to the duration of the unit hydrograph being too long in relation to the lag in the catchment.

Once the S-curve is known, a unit hydrograph of any duration could be calculated by staggering/lagging a second S-curve by the duration of the first one and then subtracting the one from the other. The resulting values then only have to be multiplied by a proportionate factor to obtain another unit run-off. This unit hydrograph may then again be dimensionalised using the values of Q_P and T_L.

Usually the method is applied arithmetically rather than graphically. It is consequently advisable to calculate the run-off values for the original unit hydrograph at such time intervals that the duration of the required hydrographs would be divisible by these time intervals.

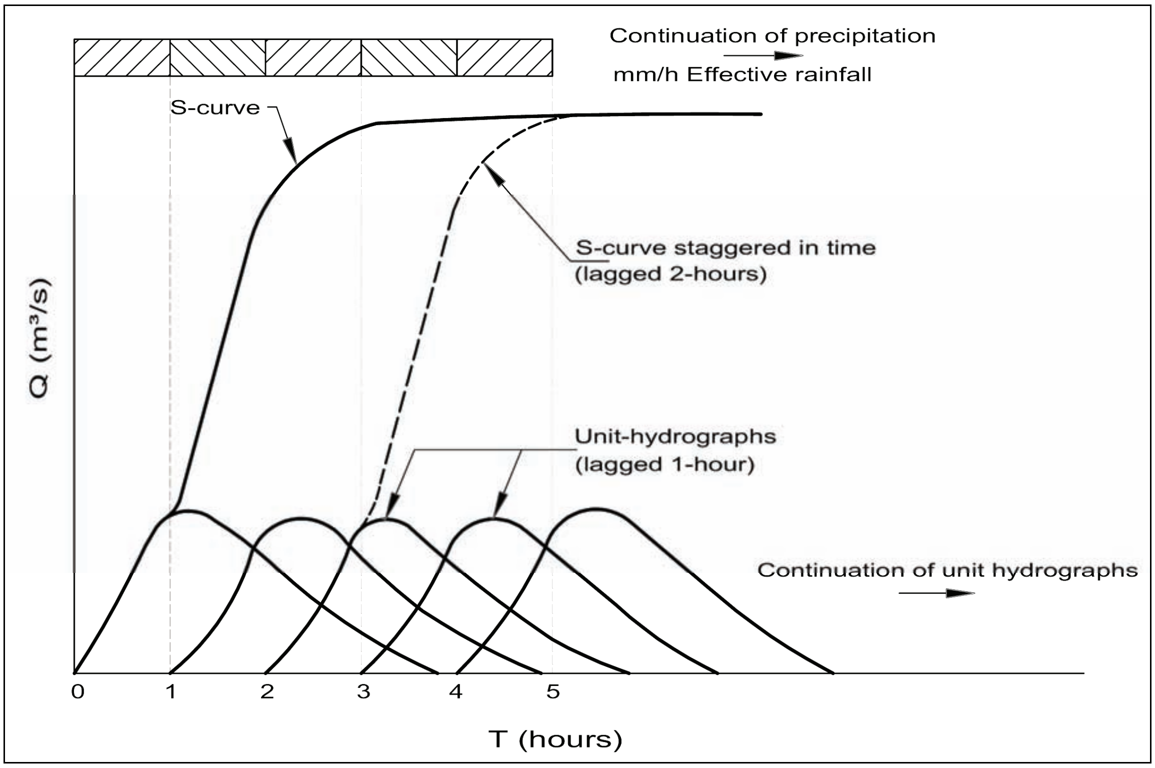


Figure 3.24: Illustration of S-curve

3.5.3.6 Determination of design hydrographs

Once the hydrographs of different durations have been dimensionalised, the peak discharge can be determined for each duration. A curve of peak discharges vs durations should then be plotted, and the turning point determined, which also represents the peak discharge. If a turning point is not reached, hydrographs with other durations should also be considered. By setting the storm duration equal to the lag time, an approximation of the peak discharge can usually be obtained. Storm durations shorter and longer than the lag time should, however, be evaluated. Where only the peak discharge is important, it is sufficient to dimensionalise the unit hydrographs only in the vicinity of the peak of the unit hydrograph.

With flood attenuation, both peak discharge and flood volume are important. To obtain the highest resultant flow rate, consequently not only the hydrograph with the highest peak discharge, but also those of shorter and longer duration should be used in the routing calculations.

A standard form for calculations involved in the application of the unit hydrograph method is given in **Appendix 3C** (3.5).

3.5.4 Standard Design Flood Method

3.5.4.1 Introduction

The unacceptably frequent severe damage to civil engineering structures by floods in South Africa is largely due to the wide band of uncertainty around all estimates of the flood magnitude-frequency relationship, combined with steep increases in flood magnitudes with an increase in return periods. These uncertainties are not always accommodated in current design flood estimation procedures, and hence the Standard Design Flood Method (SDF), which is a simple and robust method, has been developed (3.14). The SDF Method relieves one from having to evaluate the relative applicability of alternative methods for determining the design flood. It encourages one to use engineering factors of safety to accommodate uncertainties in the hydrological analyses, rather than investigate, evaluate and apply alternative hydrological procedures (3.14).

The method is based on historical data that sufficiently define the flood frequency relationships. The river flow and rainfall records used for the development and calibration of the method are sufficiently long and extensive to provide stable values for the parameters of C (discharge coefficient) and rainfall intensity.

3.5.4.2 Theoretical base of the SDF method

3.5.4.2.1 General

The different flood calculation methods were reviewed, and it was found that the rational method, which was developed in 1850 and which is the most widely used method in South Africa and abroad, should be the basis for the SDF.

The conventional rational, unit hydrograph and other rainfall run-off methods rest on the assumption that catchment characteristics play a dominant role in the flood magnitude-frequency relationship. It is apparent that destructive floods in South Africa are caused by rainfall events that have durations well in excess of the catchment response times for all but the largest catchments. Heavy rainfall during the first part of the storm brings the catchment close to saturation, and streams and rivers commence flowing strongly as the heavy rainfall continues. It has also been observed that the severe rainfall events are often preceded by above normal seasonal rainfall. In these circumstances catchment characteristics have only a minor influence on flood magnitude.

The rational formula, which has been discussed in Section 3.5.1, reads:

$$Q = \frac{CIA}{3,6} \qquad \dots (3.16)$$

In the conventional rational method the run-off coefficient C is determined by giving numerical values to the catchment characteristics that influence run-off. In the SDF method the run-off coefficient is a calibrated value, based on the statistical analysis of recorded data within the region. This makes the run-off coefficient a regional parameter and not a site-specific value.

The SDF is a numerically regionally calibrated version of the rational method. The only information required for its application is the area of the catchment, the length and slope of the main stream, and the drainage basin in which it is located according to Figure 3.25.

3.5.4.2.2 Regional definition of the run-off coefficient

A major component in the development of the SDF was the identification of regions with homogeneous flood-producing characteristics. The geographical requirement was that the boundaries had to follow catchment watersheds.

This ensured that they could be readily identified on topographical maps of any scale, and could be related to the Department of Water Affairs and Forestry's (DWAF) drainage region numbering system.

The regions were called 'basins' to avoid confusion with the DWAF's drainage regions, and the South African Weather Service's rainfall districts. The second requirement was that, where possible, each basin should contain at least one gauging station from the DWAF publication "Catalogue of hydrological catchment parameters"[3.26]. This publication contains a list of selected representative flow-gauging stations.

The delineation of the drainage basins was based on the need for homogeneous flood-producing characteristics. A number of experience-based criteria were used for this identification. **One representative rainfall station from TR102 was then selected for each basin** [3.15]. Average values may superficially appear to be preferable, but there are hydrological reasons for choosing representativeness rather than averages. There was no need to update the TR102 data for the same reason. The 29 drainage basins are shown in **Figure 3.25**.

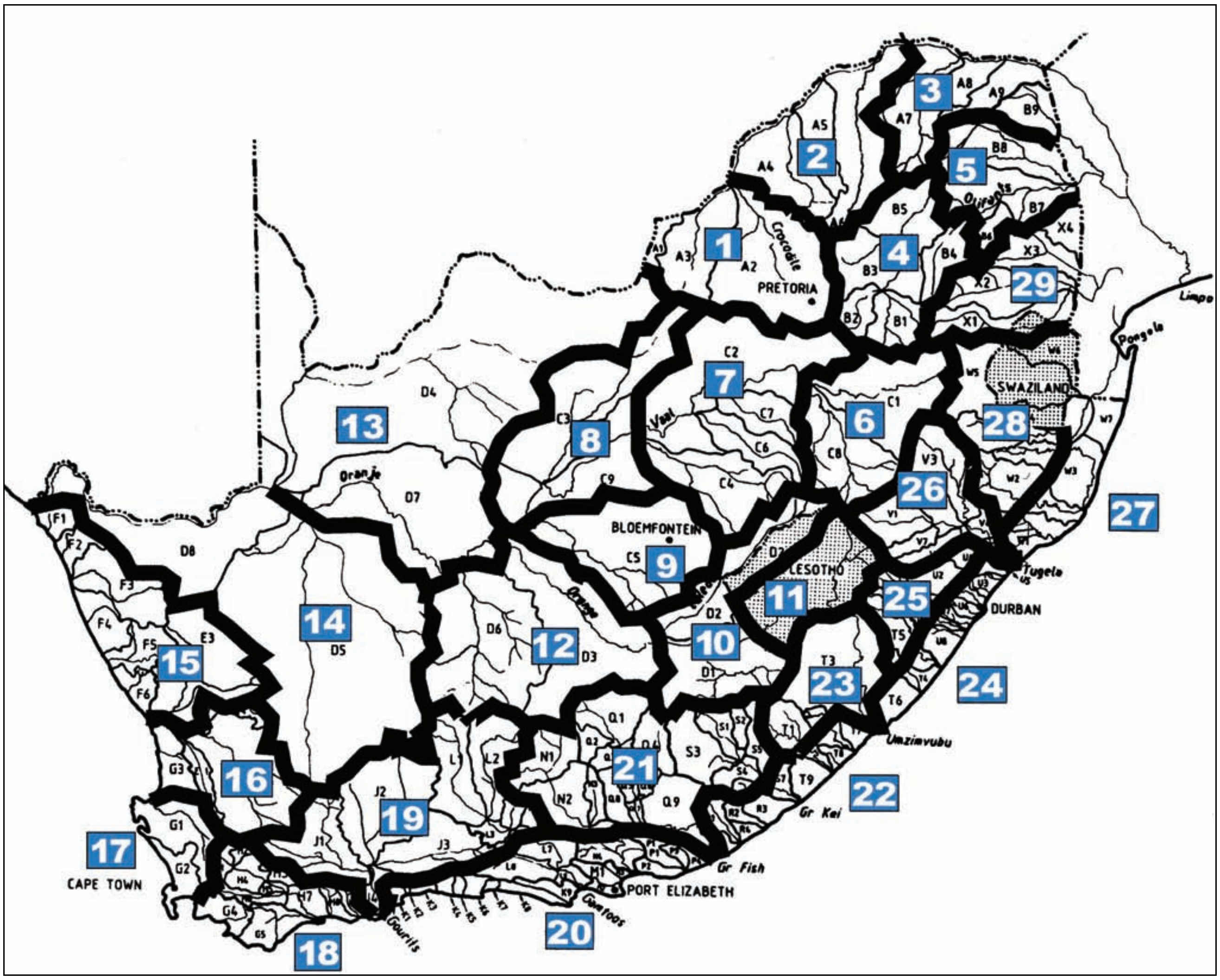


Figure 3.25: Standard Design Flood drainage basins

3.5.4.3 The development of the SDF method

Substantial studies were undertaken with the planning and design objectives in mind. All relevant hydrological and meteorological records were assembled in a standard computer-readable format. These included annual flood maxima at 152 representative flow-gauging stations with a combined record of 6 728 years. They had an average record length of more than 40 years and several records approached 100 years in length. There were a number of stations with intermittently observed maxima dating back to the 1840s.

The 29 daily rainfall stations in **Table 3B.1** (in **Appendix 3B**) were selected from the 2 400 sites in the Department of Environment Affairs (now Department of Water Affairs and Forestry) technical report TR102, *Southern African storm rainfall*. These are the representative rainfall stations for the 29 basins.

Many of the flow records have single, and occasionally more, exceptionally high values that appear as high outliers on probability plots. Other records at sites within regions vulnerable to these severe events have not yet experienced these. The calculated frequency-magnitude relationships at the sites with and without these high outliers are very different. This creates conceptual problems with both statistical analysis methods as well as with rainfall run-off models.

The remaining components that are unique to each drainage basin are listed in **Tables 3B.1** and **3B.2**. These are the statistical properties of the daily rainfall at the representative rainfall station for each drainage basin derived from TR102, and the representative percentage run-off coefficients C_2 and C_{100} for return periods of 2 and 100 years respectively. The run-off coefficients for other return periods are derived by interpolation between these two values.

Large values of C_2 and C_{100} pairs indicate that a large proportion of the representative rainfall contributes to the flood peak. Large proportional differences between C_2 and C_{100} indicate the presence of factors - principally antecedent soil moisture status - that introduce additional variability (coefficient of variation) into the rainfall run-off process.

There are many factors that influence the rainfall run-off relationship. Factors that have a large influence on the SDF and need the designer's consideration, are catchments in dolomitic areas where the flood run-off may be less than half of the SDF values, and catchments in the south-western Cape such as the Breë River, that have very flat growth curves; i.e. low coefficients of variation. In these rivers the SDF may appreciably over-estimate the flood magnitude for long return periods. Otherwise, the SDF produced satisfactorily conservative results in the representative catchments used in the development of the method. The method is amenable to hand calculation.

3.5.4.4 Design hydrograph form

Similar to the rational method, it is assumed that the SDF has a fixed triangular hydrograph shape with the duration of the rising limb equal to the time of concentration, T_C, the peak value equal to the estimated flood peak; and the duration of the falling limb equal to twice the time of concentration.

3.5.4.5 SDF calculation procedure

Refer to **Tables 3B.1** and **3B.2** (in **Appendix 3B**) for the information required for the calculation of flood magnitudes with the SDF method.

The second and third columns in **Table 3B.1** are the SAWS station identification numbers from TR102. This publication provides the information required for determining the point rainfall for the specified return period and the calculated time of concentration in step 6 below. M is the average of the annual daily maximum rainfalls, and R is the average number of days per year on which thunder was heard. These two values are used in the modified Hershfield equation (step 6). C_2 and C_{100} are the run-off coefficients as used in the equation in Step 8. MAP is the mean annual precipitation and MAE is the mean annual Symons Pan evaporation. These two values are supplied for information only, and are not used in the analysis. They indicate the substantial role played by antecedent evaporation in the flood rainfall run-off process, and how annual evaporation varies inversely with annual rainfall.

The calculation sequence is as follows:

Step 1 - Identify the drainage basin in which the site is located from **Figure 3.25**.

Step 2 - Identify the site on a topographical map, preferably 1:50 000 scale, and demarcate the catchment boundary. Copy the boundary onto tracing paper and place over 5mm or similar squared paper. Count the number of squares within the catchment including squares more than halfway into the catchment. Apply a factor to convert the number of squares to the catchment area A (km^2). GIS software packages or a planimeter can also be used to determine the catchment area.

Step 3 - Identify the main channel on the map from the site to the catchment boundary, and measure its length using dividers set at 0,2 km, (1,0 km on a 1:250 000 map) or use a wheel. In the latter case multiply the length by a scale factor of 1,2 to compensate for the loss of resolution. Derive the length of the main channel L (km).

Step 4 - Determine the elevation of the main channel in metres at two points located at 10% and 85% of the main channel length upstream of the site. Divide the difference in elevation between these two sites by 75% of the main channel length. This is the 1085-slope S (m/km).

Step 5 - Apply the US Soil Conservation Service formula to determine the time of concentration T_C (hours) as suggested in HRU1/72 [(3.23)].

$$Tc = \left[\frac{0{,}87L^2}{1000S_{av}}\right]^{0{,}385} \qquad \text{… (3.17)}$$

where:

T_C = time of concentration (hours)
L = watercourse length (km)
S_{av} = average slope (m/m)

Step 6 - Convert T_C (hours) to t (minutes). Determine the point precipitation depth $P_{t,T}$ (mm) for the time of concentration t (min) and the return period T (years). If the time of concentration is more than 24 hours use linear interpolation of the values for the reference rainfall station from TR102 listed in **Table 3B.2**. Otherwise use the modified Hershfield equation.

$$P_{t,T} = 1{,}13(0{,}41 + 0{,}64\ln T)(-0{,}11 + 0{,}27\ln t)\left(0{,}79M^{0{,}69}R^{0{,}20}\right) \qquad \text{… (3.18)}$$

where:

M = mean of the annual daily maxima from **Table 3B.1**.
R = average number of days per year on which thunder was heard from **Table 3B.1**.

Step 7 - Multiply the point precipitation depth $P_{t,T}$ (mm) by the area reduction factor ARF (%) to determine the average rainfall over the catchment for the required return period T (years). The corresponding rainfall intensity I_T (mm/h) is obtained by dividing this value by the time of concentration.

$$ARF = (90000 - 12800\ln A + 9830\ln t)^{0{,}4} \qquad \text{… (3.19)}$$

Step 8 - The above steps constitute the standard procedure used in the conventional rational method. The SDF uses calibrated run-off coefficients C_2 (2-year return period) and C_{100} (100-year return period) from **Table 3B.1** instead of determining them from catchment characteristics. The run-off coefficients for the range of return periods T (years) are derived by applying the return period factors Y_T in **Table 3.13**, using the relationship in the equation below:

Table 3.13: Return period factors

T =	2	5	10	20	50	100	200
Y_T =	0	0,84	1,28	1,64	2,05	2,33	2,58

$$C_T = \frac{C_2}{100} + \left(\frac{Y_T}{2{,}33}\right)\left(\frac{C_{100}}{100} - \frac{C_2}{100}\right) \qquad \text{… (3.20)}$$

Step 9 - Finally, the flood peak Q_T (m^3/s) for the required return period T is calculated from:

$$Q_T = \frac{C_T I_T A}{3{,}6} \qquad \text{… (3.21)}$$

which is the standard format used in the rational method.

Step 10 - The SDF hydrograph is triangular in shape with the duration of the rising limb equal to the time of concentration T_C (hours), and that of the falling limb equal to twice the time of concentration. Use linear interpolation between these values.

Note: It is essential that the above procedure and equations be used to determine the SDF. Under no circumstances should alternative sources of information or equations be used, as these would invalidate the calibration and verification procedures on which they are based.

3.5.4.6 Additional SDF comparison

Given the limited experience with the SDF method at the time of publication, the authors requested an independent review of records that had not been used by the developer of the SDF method. Five additional stations in each of the 29 basins were selected and significant differences in some of the basins between the fitted distribution function (LP3) values, based on the recorded data and those calculated by means of the SDF method were found. The differences did not follow clear patterns. It is believed that these results can serve as valuable additional information in the application of the SDF method and the report has been included on the supporting CD.

3.6 EMPIRICAL METHODS

3.6.1 General

Empirical methods are mostly based on simple correlations between peak flow rates and other catchment characteristics derived in order to establish general regional parameters. The peak discharges determined according to these methods are thus likely to be less accurate than those obtained by using statistical or deterministic methods.

The reliability of empirical methods depends largely on the realistic demarcation of comparable flood-producing areas. Parameters for every region should preferably be based on historical data for the same region. Since the characteristics of a given catchment often differ markedly from the general conditions in the region, the results should be adjusted subjectively according to the indications provided in Section 3.3.

These methods are based mainly on flow measurements at measuring stations covering catchments that are seldom smaller than 10 km^2 and usually larger than 100 km^2. Consequently these methods are only applicable to medium and large catchments. As more data become available, the methods may be improved or replaced by new methods.

3.6.2 Empirical peak flow calculations for rural areas

From a study of the frequency distribution of annual maximum flood peaks at 83 measuring stations in South Africa, Midgley and Pitman [(3.10)] compiled regional curves of flood peaks, with the size of the catchment and the return period as variables. **This method is suitable for rural catchments larger than 100 km^2**. The following relationship was determined:

$$Q_T = K_{RP} \, A^{0,5} \qquad \text{… (3.22)}$$

where:

Q_T = peak flow for T-year return period (m³/s)

K_{RP}	=	constant for T-year return period (Original text referred to K_T but resulted in confusion with K_T used in equation 3.23). The value of K_{RP} can be back calculated from **Figure 3.27** by obtaining the Q_T value and substituting in equation 3.22
A	=	size of catchment (km²)

To establish the temporal flow rate variation it was required to include other characteristics of the catchment. Considering the topography, rainfall characteristics, soils, drainage patterns and plant cover, the country was divided into seven homogeneous flood regions with similar K values. The regional distribution according to homogeneous flood characteristics is shown in **Figure 3.26**, and the flood peak probability curves in **Figure 3.27** [3.7].

Although the 83 measuring stations represent only a small sample, and although the frequency distributions also contain inaccuracies, this method generally yields good estimates of the peak discharges in undeveloped areas. It is particularly **suitable for obtaining an advance indication of the order of magnitude of peak discharges, or to serve as a rough check on the results of non-statistical methods**.

3.6.3 Empirical deterministic peak discharge formula for return periods of up to 100 years

Peak discharges for **return periods less than or equal to 100 years** may be determined by means of an empirical deterministic method developed by Midgley and Pitman [3.10, 3.11]. The formula reads:

$$Q_T = 0{,}0377 K_T P A^{0,6} C^{0,2} \qquad \dots (3.23)$$

where:

Q_T	=	peak flow for T-year return period (m³/s)
K_T	=	constant for T-year return period (obtained from **Table 3.14**)
A	=	size of catchment (km²)
P	=	mean annual rainfall over catchment (mm/a) (see **Figure 3.8**)

and

$$C = \frac{A\sqrt{S}}{L\,L_C} \quad \text{(catchment parameter with regard to reaction time)} \qquad \dots (3.24)$$

and where:

S	=	average slope of stream (m/m)
L	=	hydraulic length of catchment (km)
L_C	=	distance between outlet and the centroid of the catchment (km)

Values of K_T have been compiled for the different veld-type regions of South Africa, shown in **Figure 3.16** and in **Table 3.14** [3.5].

Table 3.14: Constant values of K_T for different veld types

Return period T in years	Veld type (Figure 3.16)										
	1	2		3	4 & 5A	5	6		7	8	9
		Winter	All year				Winter	All year			
10	0,17	0,42	0,83	0,29	0,59	0,59	0,33	0,67	0,67	0,42	0,50
20	0,23	0,52	1,04	0,40	0,68	0,80	0,45	0,91	0,91	0,57	0,68
50	0,32	0,68	1,36	0,55	0,95	1,11	0,63	1,26	1,26	0,79	0,95
100	0,40	0,80	1,60	0,70	1,20	1,40	0,80	1,60	1,60	1,00	1,20

The formula usually yields results that are comparable to those of the synthetic hydrograph method. It should **preferably be applied to catchments larger than 100 km²**, but could be applied with caution to catchments larger than 10 km².

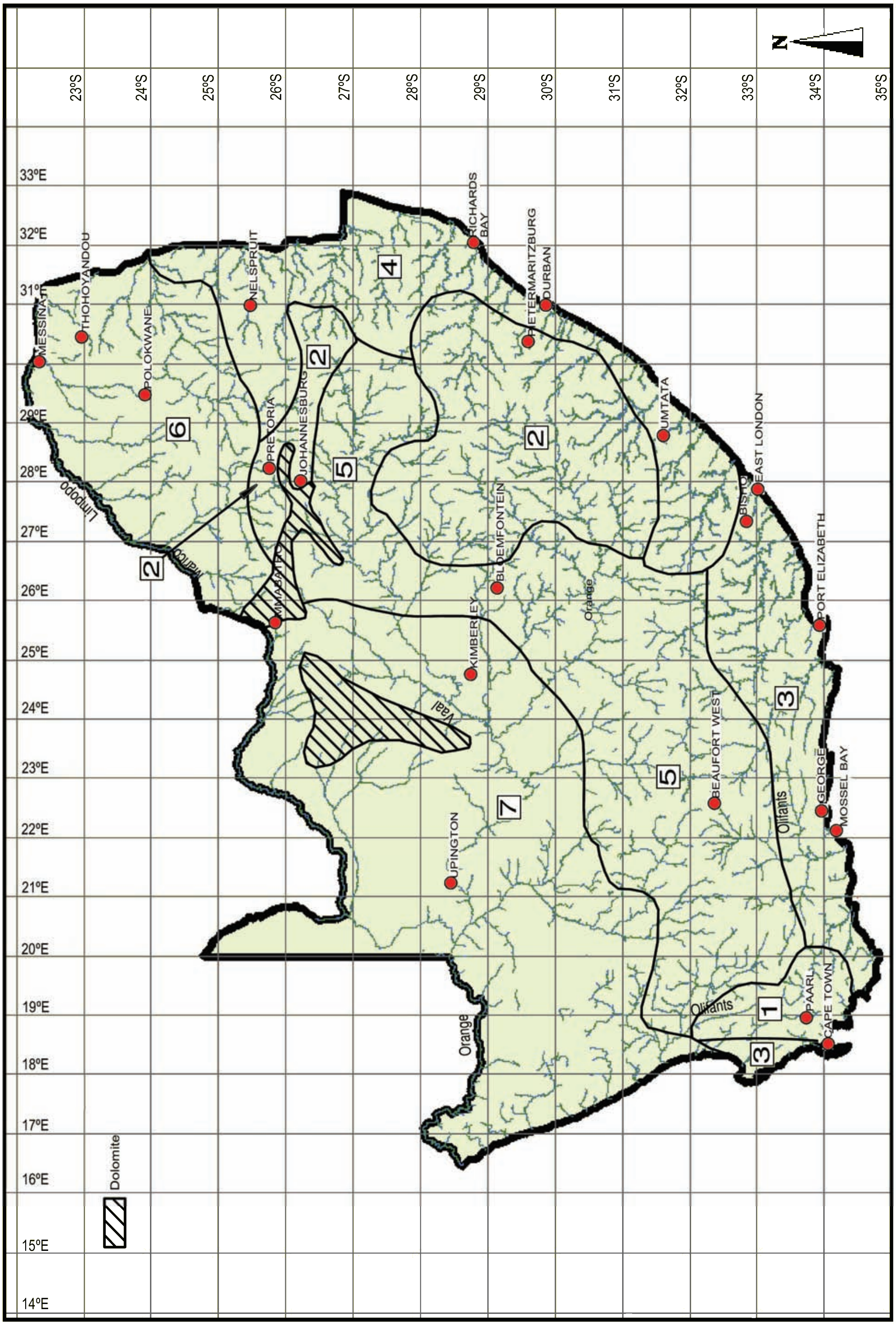


Figure 3.26: Homogeneous flood regions in South Africa

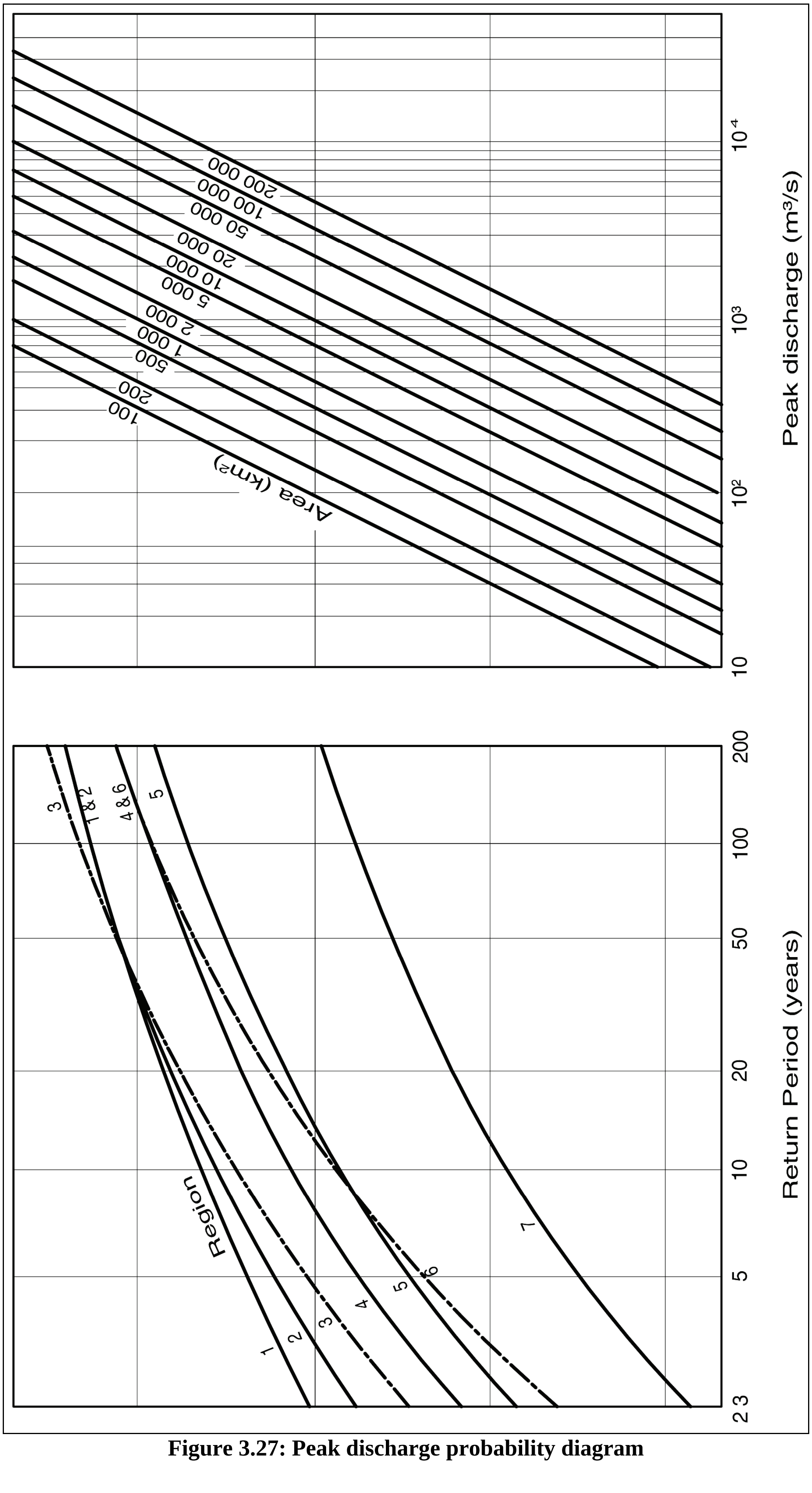


Figure 3.27: Peak discharge probability diagram

3.6.4 Experience curves of maximum flood peaks for the calculation of flood peaks for return periods of more than 100 years

In road drainage, floods with return periods of more than 100 years are rarely required. However, it is sometimes desirable or necessary to obtain realistic values for extreme peak floods and the accompanying water levels; particularly where human lives may be endangered and/or valuable property may be damaged.

Most of the existing methods fail to predict flood peaks for return periods in excess of 100 years. With statistical methods poor fits of statistical distributions and insufficient historical recorded data lengths are problematic. With deterministic methods, such as the unit hydrograph method, the problem arises that there is very little information available on area reduction factors for extreme rainfall, and the relative importance of the factors influencing run-off also changes. This lack of information consequently affects the reliability of the methods.

In an investigation for the Directorate of Water Affairs, Kovács [(3.11)] studied the approximately 300 highest flood peaks observed in South Africa between 1894 and 1979. The information was processed using the Francou-Rodier relationship [(3.12)], and five regional curves with confidence bands were compiled. The Francou-Rodier relationship, which is used to determine the regional maximum flood (RMF), reads:

$$Q_{RMF} = 10^6 \left(\frac{A}{10^8} \right)^{1-0,1K} \qquad \dots (3.25)$$

where:

Q_{RMF}	=	regional maximum flood peak flow rate (m³/s)
K	=	regional constant (Obtainable from the regional classification detailed in **Figure 3.28** and simplified in **Table 3.15**)
10^6	=	total world MAR (m³/s)
10^8	=	total world catchment area (km²)

The values of K vary from 0 to 6 with the latter value being the K value that envelopes world recorded floods, which have occurred in the high-rainfall cyclone-ravaged areas. The formula was also tested on more than one thousand reliably measured floods from all over the world.

The regional classification for maximum peak discharges in South Africa are named Kovács regions **K1 – K8 in this publication,** and are depicted in **Figure 3.28** [(3.13)]. This regional classification of Kovács regions supersedes the earlier work done by Kovács where only 5 regions were identified with linked K-values.

Table 3.15: RMF region classification in southern Africa

Kovács region	K *	Number of floods #	Transition zone		Flood zone	
			Area range (km²)	Q_{RMF} (m³/s)	Area range (km²)	Q_{RMF} (m³/s)
K1	2,8	6	1 – 500	$30A^{0,262}$	500 – 500 000	$1,74A^{0,72}$
K2	3,4	12	1 – 300	$50A^{0,265}$	300 – 500 000	$5,25A^{0,66}$
K3	4,0	26	1 – 300	$70A^{0,34}$	300 – 300 000	$15,9A^{0,60}$
K4	4,6	55	1 – 100	$100A^{0,38}$	100 – 100 000	$47,9A^{0,54}$
K5	5,0	155	1 – 100	$100A^{0,50}$	100 – 100 000	$100A^{0,50}$
K6	5,2	61	1 – 100	$100A^{0,56}$	100 – 30 000	$145A^{0,48}$
K7	5,4	34	1 – 100	$100A^{0,62}$	100 – 20 000	$209A^{0,46}$
K8	5,6	25	1 – 100	$100A^{0,68}$	100 – 10 000	$302A^{0,44}$

Notes: * *RMF K value as used in Equation 3.25*
\# *Recorded flood data are reflected in the DWAF report TR105 – Maximum flood peak discharges in South Africa: An empirical approach*

There are two regions where the K value used in Equation 3.25, is smaller than K1 and K2. For these regions Equation 3.25 should be used with a K value less than 2,8 for the region designated as "<K1" (Central Botswana) and a K-value less than 3,4 for the region "< K2" (West Coast of Namibia), see **Figure 3.28**.

In using the values obtained from **Table 3.15**, it should be borne in mind that they represent the upper realistic limits for every region. For a specific catchment with much lower run-off characteristics than the average for the region, the K-value may be adjusted downwards. The following conditions may, for example, be present in a catchment:

(i) An unusually flat catchment with wide flood plains and pans.

(ii) A very permeable surface or dolomite.

(iii) Dense bush or plantations, different from the rest of the area.

(iv) A clear decrease in the mean annual rainfall in the downstream direction of catchments larger than 1 000 km^2.

Such conditions occur in all the regions, except for region K1. As a rule of thumb it may be stated that the K-value should not be adjusted any lower than the value for the following region. Where catchments fall within two regions, the peak discharge should be adjusted proportionately, or the highest value may be accepted. For urban areas and catchments smaller than 10 km^2, it is better to determine the probable maximum flood by means of other methods as discussed in paragraph 3.5.

Knowledge of 50 to 200 year peak discharges is required in practice for the design of bridges and dams. According to Kovács [3.13] a simple unorthodox analysis of the K-value and the representative return period of entirely independent flood peaks has provided coefficients which represent the 50 to 200 year peaks as fractions of RMF. These Q_T/Q_{RMF} ratios are provided in **Appendix 3D** (**Tables 3D.1** and **3D.2**) and are dependent on the region as well as the effective catchment area.

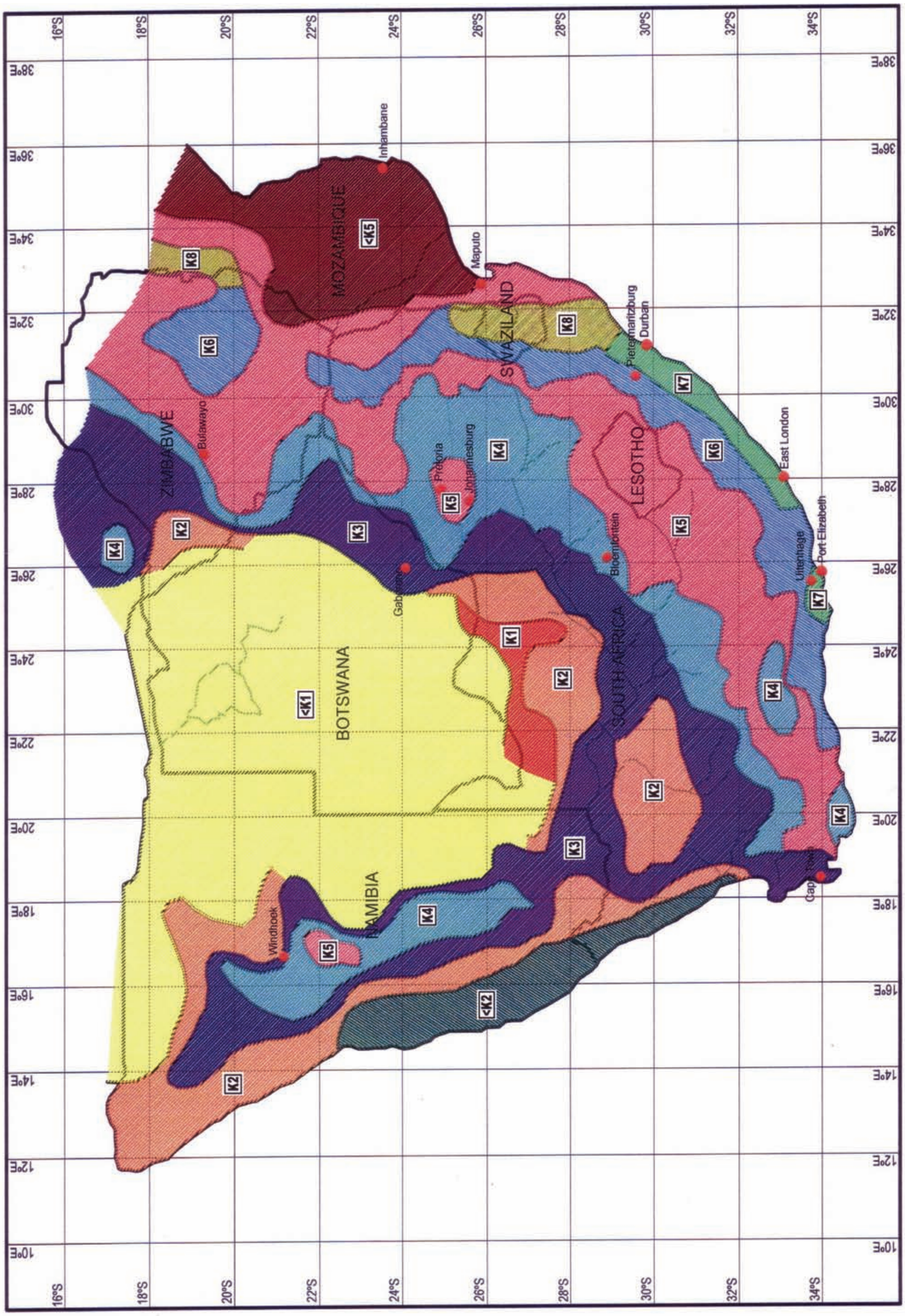


Figure 3.28: Maximum flood peak regions in southern Africa from Kovács (1988)[3.13]

3.7 METHODS THAT ARE NOT INCLUDED IN THIS MANUAL

3.7.1 SCS method

The SCS method (United States Soil Conservation Service Hydrograph Generating Technique)(3.16) is particularly suitable for computing flood peaks and run-off volumes for catchments smaller than 10 km² and with slopes of less than 30 per cent. It is mainly applicable to rural catchments but may also be used for urban areas (3.17). The basic method requires a considerable amount of calculation, but this may be greatly reduced by using nomograms.

The SCS method takes into account most of the factors that affect run-off, such as quantity, time distribution and duration of rainfall, land use, soil type, prevailing soil moisture conditions, and size and characteristics of the catchment. An advantage of the SCS method is that it enables empirical hydrographs to be fully calculated.

A detailed description of the SCS method and its application in South Africa is given by Schulze and Arnold (3.17, 3.18).

In contrast to the rational and some other methods, the SCS method does not contain rainfall intensity as a basic variable, but 24-hour rainfall data for different return periods are used to calculate the storm volume for design purposes. However, in determining the peaks, typical time distributions of the 24-hour storm rainfall are taken into account.

The typical time distribution of 24-hour storms for two climatic regions is given in **Figure 3.29** (3.19). Type I distribution applies to coastal areas with winter rainfall or rainfall throughout the year, and Type II to areas where convection activity (summer thunderstorms) is the main cause of flood rainfall over small catchments. The distribution areas of Type I and II storms are shown in **Figure 3.30** (3.19). Below is a useful graph (**Figure 3.29**) to determine the rainfall intensity, especially when working with very small catchments, which have a short time of concentration.

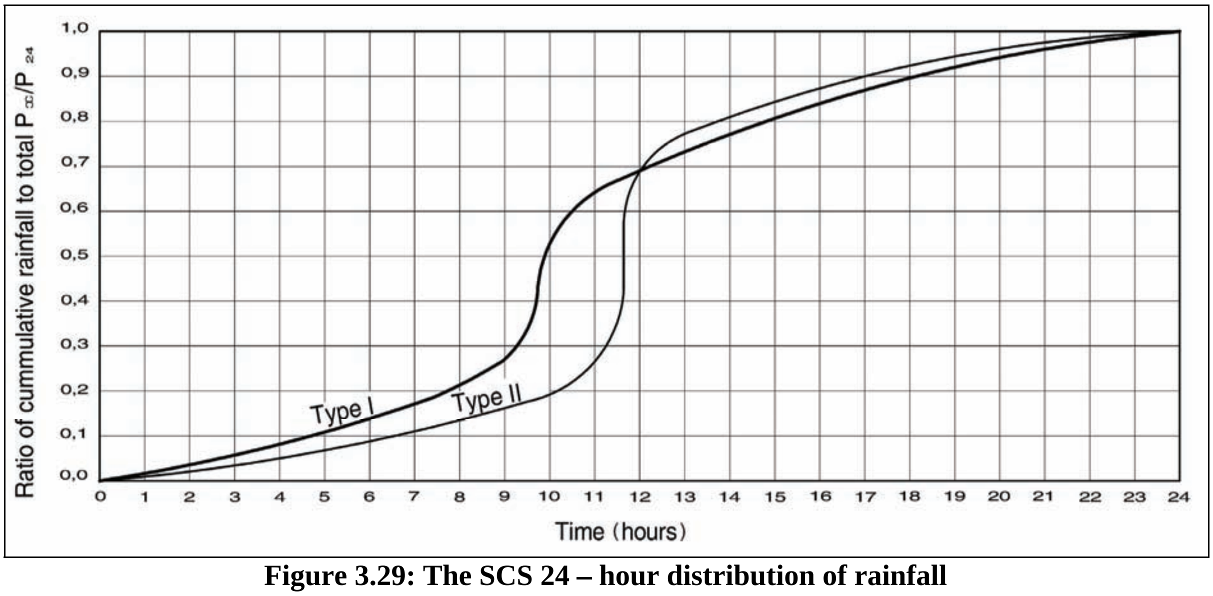


Figure 3.29: The SCS 24 – hour distribution of rainfall

Additional research on rain storms by Schmidt and Schulze(3.31, 3.32) produced four regionalised time distributions, which is incorporated in the SCS-SA software.

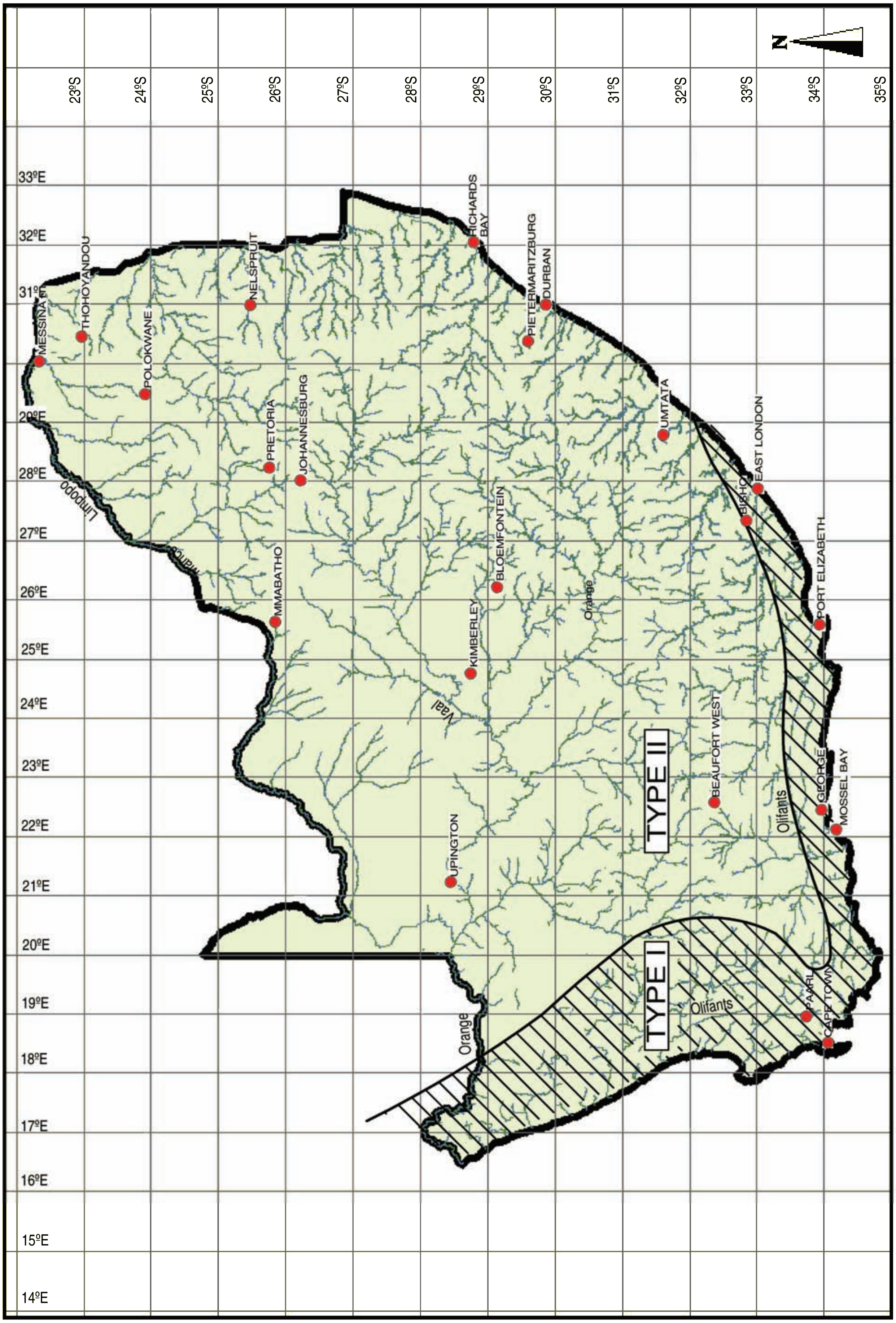


Figure 3.30: Area distribution of SCS storm types in SA

3.7.2 Run Hydrograph method

The Run Hydrograph method is based mainly on statistical regional analyses of historical data. A number of hydrographs with the same return period, but with different run-off volumes and peak discharges, may be calculated by means of this method. The method is not widely used and its application to road drainage is mainly as an alternative to or checks on the unit hydrograph method.

The run hydrograph theory was originally developed for determining average compound hydrographs with known return periods from records of river flow. For every given return period, however, there is a group of hydrographs with different run-off volumes and peak flows that fulfil this criterion. Hiemstra and Francis [(3.20)] give a detailed description of this method in a publication [(3.20)] obtainable from the Water Research Commission and a full description of the method is consequently not repeated here.

The results are similar to those obtained by means of the unit hydrograph method. However, the latter method tends to belong to the deterministic group of methods, whereas the run hydrograph method is statistically orientated. The run hydrograph method is generally more applicable to larger catchments than the synthetic hydrograph method.

Hydrographs with different peak discharge and run-off volumes, but with the same return period are required particularly for flood routing calculations for average and large catchments.

3.8 WORKED EXAMPLES

The main aim of this section is to provide the reader with a step-by-step explanation of the procedures used to calculate flood magnitudes for different return periods.

In the two paragraphs below, flood peaks will be calculated for a small as well as a large catchment using the various relevant deterministic, statistical and empirical methods as described in previous sections.

3.8.1 Worked example 3.1 - Small catchment

The first worked example reflects the flood calculation for a small bridge on the Moretele Spruit, which runs through the eastern part of Pretoria in a north-westerly direction (see **Figure 3.31**).

The small bridge, as shown in **Photograph 3.5**, is located in Pretoria East (location indicated on **Figure 3.31)**. The flooding of the bridge has to be analysed for the 1:20 year and 1:50 year recurrence interval flood peaks to determine the risk of flooding Hans Strijdom Drive, which has become an important artery in the eastern suburbs of Pretoria.

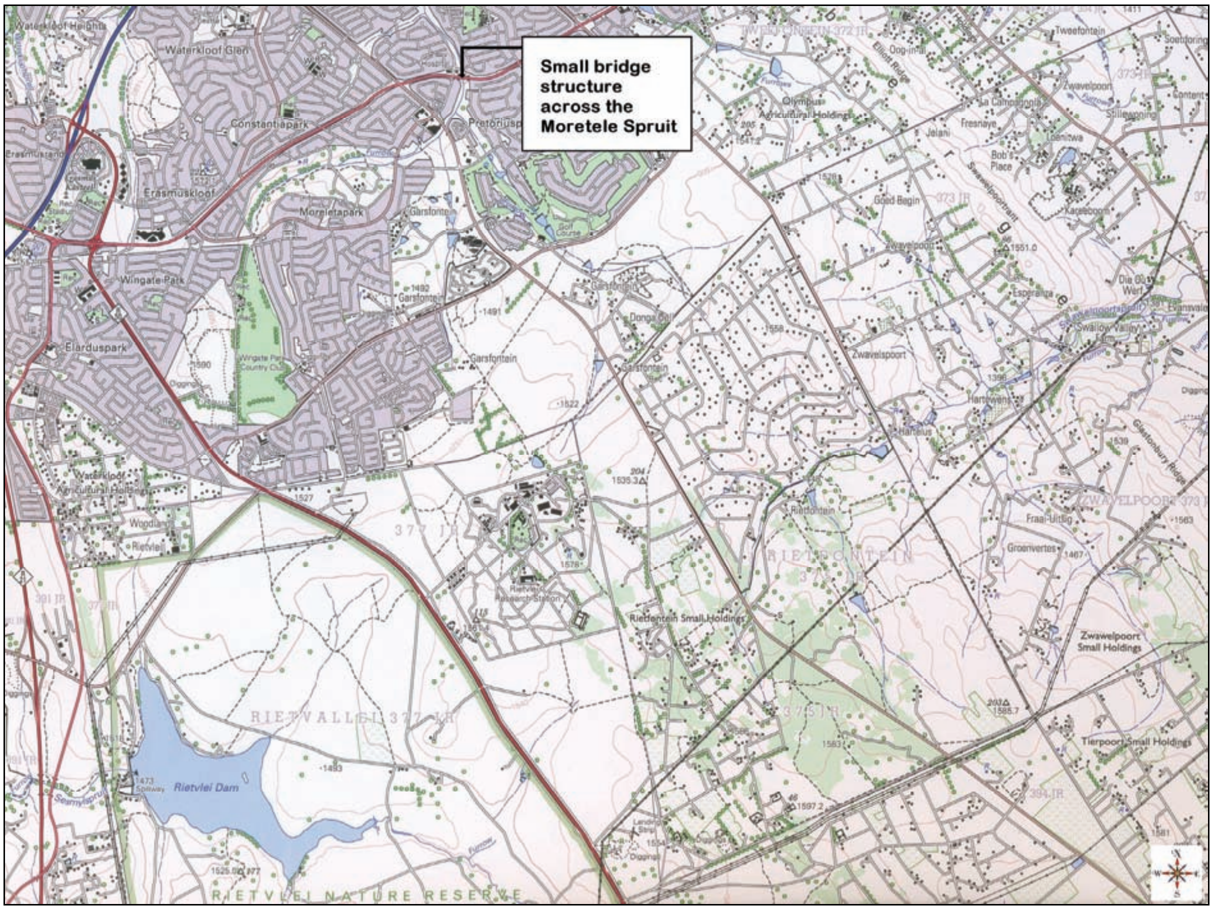


Figure 3.31: Moretele Spruit catchment area (shown on a 1:50 000 topographical map)

Photograph 3.5: Small bridge across the Moretele Spruit (Hans Strijdom Drive)

3.8.1.1 Rational method

Data requirements

The Rational method requires the following data:

- Area of catchment
- Length of longest watercourse and average slope to calculate time of concentration
- Catchment characteristics to calculate run-off coefficients
- Mean annual rainfall and rainfall region to determine average rainfall intensity

Calculation procedure

Step 1: Determine the catchment area (km²).
Topographical maps (1: 50 000) are normally used to determine the area of a catchment. However, the accuracy and contour intervals on these maps are not always as required and it is often useful to obtain 1: 10 000 maps, if available. Ortho-photographs should also be used, if available. Use graph paper or a planimeter to determine the total catchment area, which will contribute to the peak flow. Pans or areas that are artificially isolated should thus be excluded.

The catchment area of the Moretele Spruit up to the small bridge is *28,5 km²* as shown in **Figure 3.32**.

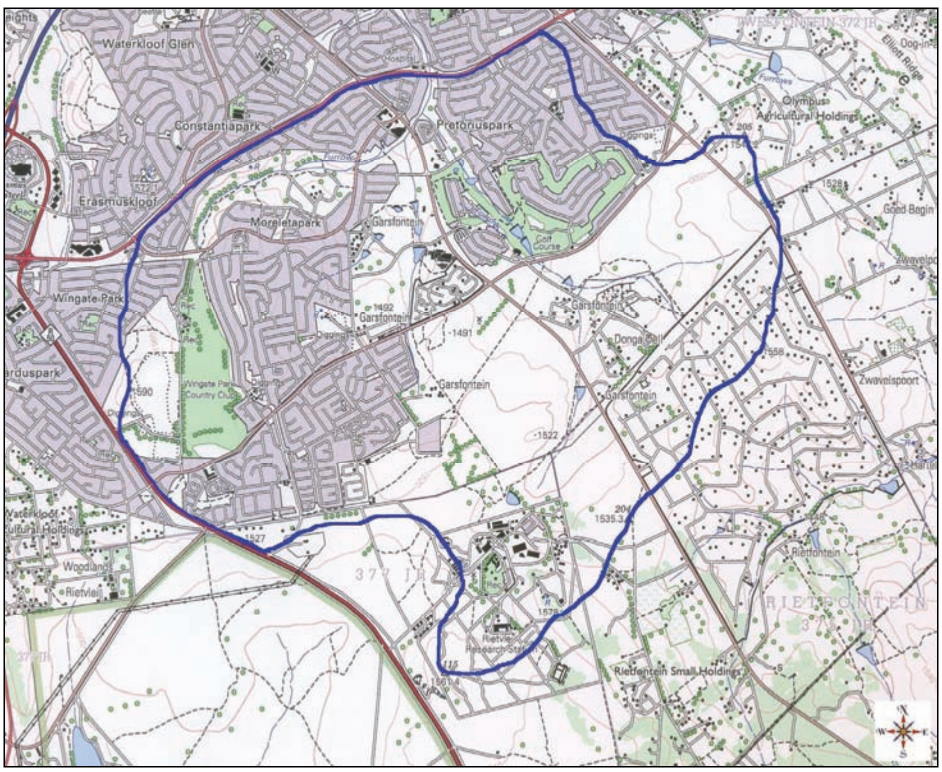


Figure 3.32: Determined catchment area

Step 2: Determine the length of the longest watercourse (km).
For the defined catchment area as required for Step 1, the longest watercourse and its length are determined. The length of the watercourse for this example is L = *7,25 k*m.

Step 3: Determine the average slope of the longest watercourse.
Utilising the 10-85 method (m/m) as developed by the US Geological Survey, and tested by the UK Institute of Hydrology, calculate the average slope. A longitudinal profile of the Moretele Spruit along the longest watercourse is shown in **Figure 3.33**.

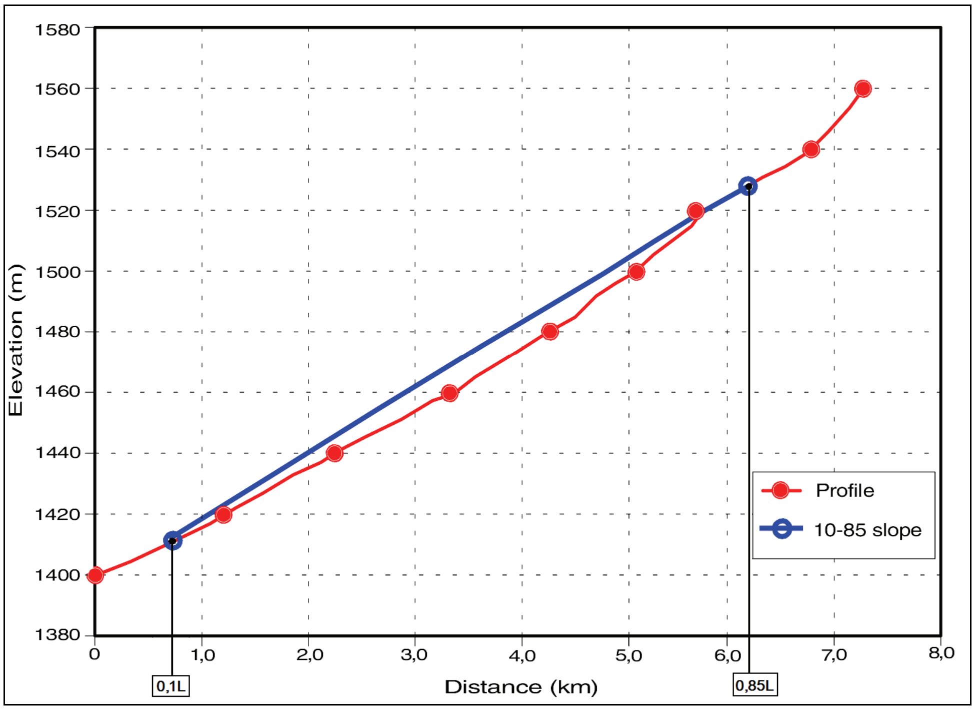


Figure 3.33: Longitudinal profile of the Moretele Spruit

$$S_{av} = \frac{H_{0,85L} - H_{0,10L}}{(1\,000)(0,75L)} \qquad \text{… (3.26)}$$

where:

S_{av} = average slope (m/m)
$H_{0,10L}$ = elevation height at 10% of the length of the watercourse (m)
$H_{0,85L}$ = elevation height at 85% of the length of the watercourse (m)
L = length of watercourse (km)

The elevation at 10% of the length of the longest watercourse is $H_{0,10L} = 1\,412,1\ m$ and at 85% of the length the elevation is $H_{0,85L} = 1\,528,8\ m$.

The calculated average slope for this example is *0,02146 m/m.*

Step 4: Calculate the time of concentration from catchment characteristics. The recommended empirical formula for calculating the time of concentration in natural channels has been developed by the US Soil Conservation Services.

$$T_C = \left(\frac{0,87L^2}{1\,000\,S_{av}}\right)^{0,385} \qquad \text{… (3.27)}$$

where:

T_C = time of concentration (hours)
L = length of watercourse (km)
S_{av} = average slope (m/m)

In most cases the longest waterpath includes both overland and channel flows. In large catchments the channel flow is usually dominant, but in small catchments it may be necessary to determine T_C as the sum of the flow times for both the overland and channel flow stretches. To obtain a broad indication, it may usually be accepted that a defined watercourse exists when the average slope of the catchment is greater than 5 per cent and the catchment itself is larger than 5 km².

The time of concentration of the Moretele Spruit up to the specific point is $T_C = 1,338$ *hours*.

Step 5: Obtain the mean annual rainfall (MAR), from the South African Weather Service or from the simplified **Figure 3.7** (less accurate).

When there are two or more rainfall stations in the catchment area the Thiessen method or weighted area method can be used to determine the representative rainfall for the catchment. The catchment area in this example contains one rainfall station within the catchment and two adjacent as shown in **Figure 3.34**.

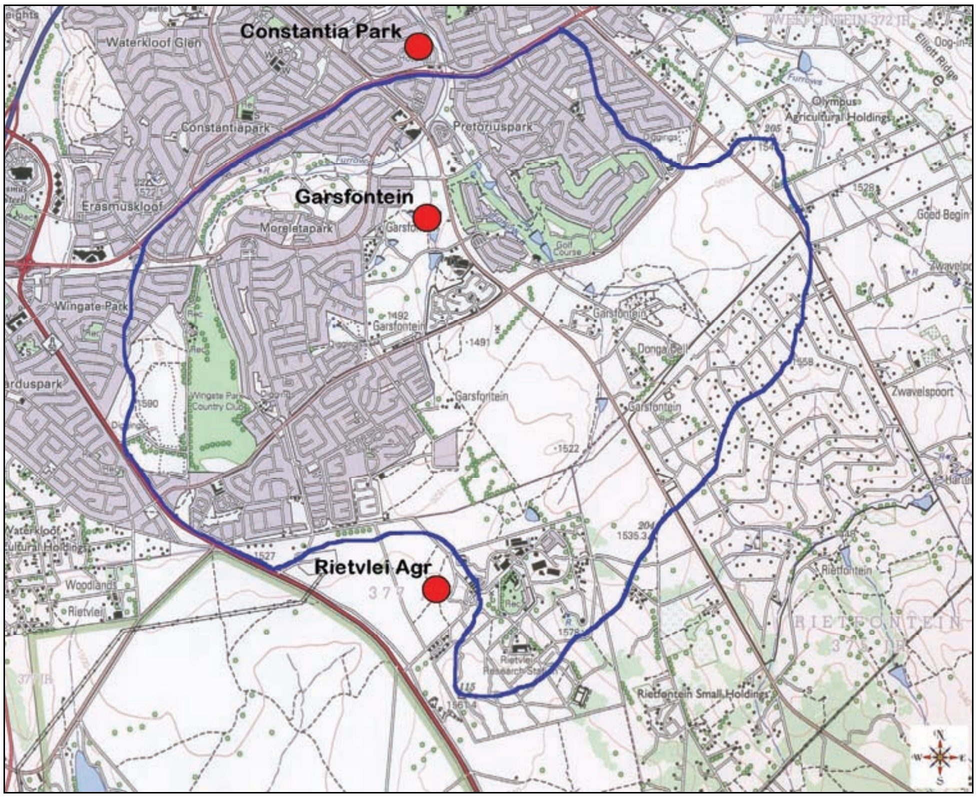


Figure 3.34: Rainfall stations used in determining the representative MAR

The weighted area method was used to determine the representative mean annual rainfall as shown in **Table 3.16** based on the applicable areas. All the stations in this example are located in a straight line and thus the Thiessen method could not be utilized in this example.

Table 3.16: Mean annual rainfall of catchment area [3.21]

Weather Service rainfall station	Latitude D M	Longitude D M	MAR (mm)	Area (km^2)
0513529 – Garsfontein	25° 49'	28° 18'	771,8	16,53
0513531 - Rietvlei Agr.	25° 51'	28° 18'	714,0	9,69
0513528 - Constantia Park	25° 48'	28° 18'	702,5	2,28
Total			746,6	28,5

Also determine the rainfall region in which the catchment falls.

The mean annual rainfall (MAR) for this catchment is *746,6 mm* (see **Table 3.16**) and the catchment is located in the inland region.

Step 6: Determine the point rainfall values (P_T) (mm) for the required return periods. Based on the mean annual rainfall (MAR), the rainfall region, the time of concentration (T_C) and the required return period **Figure 3.6** can be used to determine the point rainfall. As shown in **Figure 3.35** (which is a copy of **Figure 3.6**) the point rainfall for the 1:20 year and 1:50 year return periods are determined using the co-axial diagram.

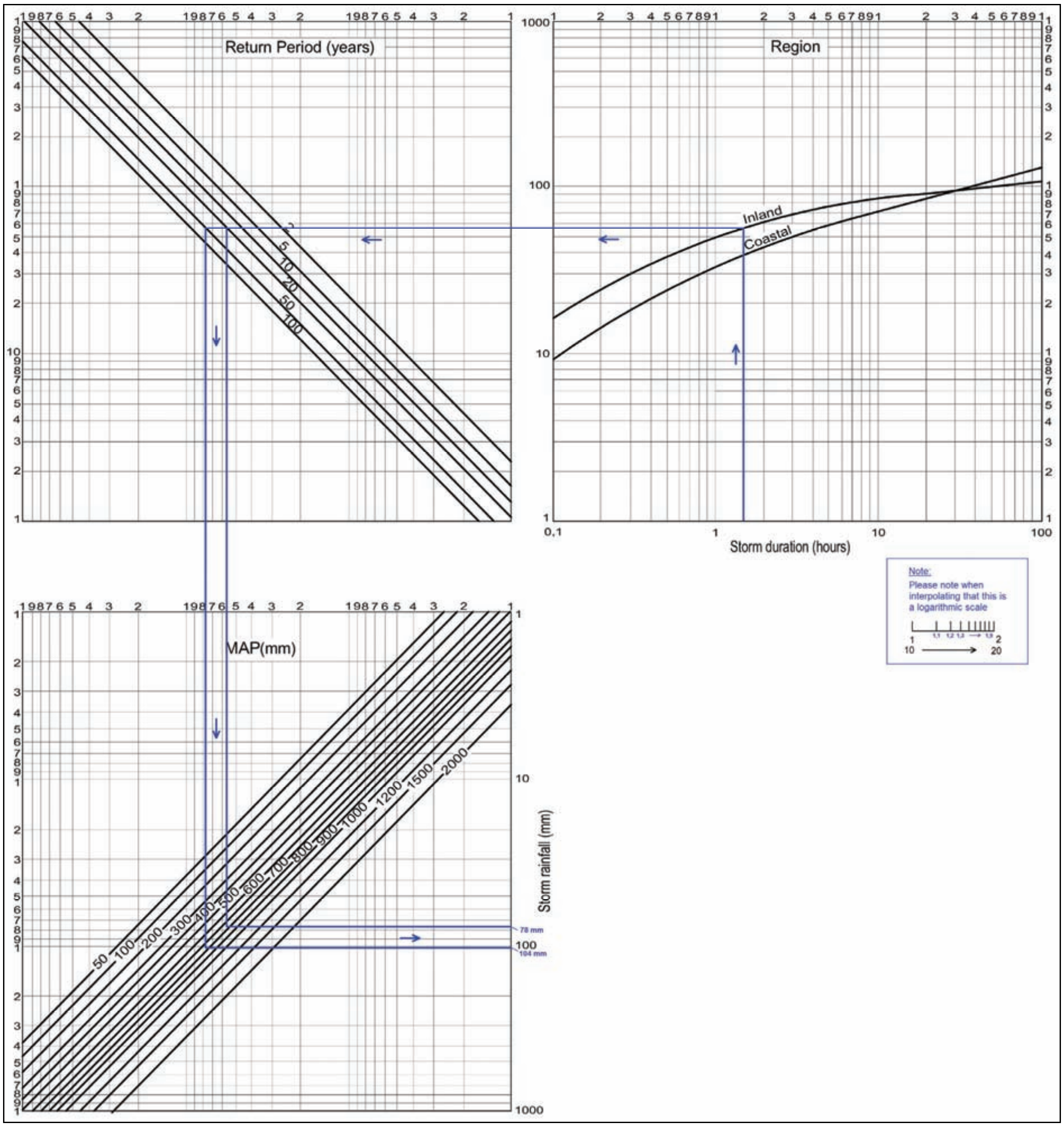


Figure 3.35: Determining the point rainfall

The point rainfall for the 1:20 and 1:50 year return periods is P_{20} = *78 mm* and P_{50} = *104 mm* respectively.

Step 7: Calculate the point intensity (mm/hour)

The point intensity (P_{iT}) is the point rainfall divided by the time of concentration (if T_C > 0,25 hours). If $T_C \leq$ 0,25 hours divide by 0,25 hours.

$$P_{iT} = \frac{P_T}{T_C} \qquad \dots (3.28)$$

where:

P_{iT}	=	point intensity for the different return periods (mm/h)
P_T	=	point rainfall (mm)
T_C	=	time of concentration (hours)

The point intensities for the 1:20 and 1:50 year return periods are P_{i20} = *58,3 mm/h* and P_{i50} = *77,7 mm/h* respectively.

Step 8: Determine the area reduction factors (ARF) for the different return periods either from **Figure 3.20** or **3.21**.

In this example the catchment area is small and thus **Figure 3.20** is used. The resulting ARFs are ARF_{20} = *94%* and ARF_{50} = *91%* for the 1:20 and 1:50 years return periods respectively.

Step 9: Determine the average rainfall intensity or effective catchment precipitation.

$$I_T = P_{iT}\left(\frac{ARF_T}{100}\right) \qquad \text{… (3.29)}$$

where:

I_T = rainfall intensity averaged over the catchment in millimetres/hour for the return period T.

ARF_T = area reduction factor as a percentage for return period T (should be smaller than 100%)

P_{iT} = point intensities for the different return periods (mm/h)

The average rainfall intensities are I_{20} = *54,80 mm/h* and I_{50} = *70,71 mm/h*.

Step 10: Identify the catchment characteristics to determine the run-off coefficient.
The run-off coefficient in the rational method is an integrated value representing the many factors influencing the rainfall run-off relationship. There is no objective theoretical method for determining C and as a result the subjective elements of experience and engineering judgement play a very important role in the successful application of this method.

Table 3.7 gives a chart for the calculation of the run-off coefficient, with recommended values of C.

The Moretele Spruit catchment is classified as *40%* rural and *60%* urban based on the latest information available (i.e. topographical maps and confirmed by a visit to the catchment (see **Figure 3.32**)) i.e. α = *0,4*; β = *0,6* and γ = *0,0*.

Based on the available data from the catchment, the following table was compiled to characterise the catchment (**Table 3.17** and **3.18**).

Table 3.17: Catchment characteristics (Rural)

Rural (C_1)		
Component	**Classification**	**%**
Surface slope (C_S)	Vleis and pans (<3%) Flat areas (3 to 10%) Hilly (10 to 30%) Steep areas (>30%)	20 70 10 0
Permeability (C_P)	Very permeable Permeable Semi-permeable Impermeable	0 50 50 0
Vegetation (C_V)	Thick bush and plantation Light bush and farm lands Grasslands No vegetation	0 45 50 5

Utilising **Table 3.7** the run-off coefficient for the **rural area** is calculated using the following formula:

$$C_1 = C_S + C_P + C_V \qquad \text{… (3.30)}$$

where:

C_1 = run-off coefficient with a value between zero and one
C_S = run-off coefficient according to average catchment slope
C_P = run-off coefficient according to average soil permeability
C_V = run-off coefficient according to average vegetal growth

The average rainfall falls between 600 and 900 mm, and thus:

$$\begin{aligned} C_1 &= (0{,}20 \text{ x } 0{,}03 + 0{,}70 \text{ x } 0{,}08 + 0{,}10 \text{ x } 0{,}16) \\ &\quad + (0{,}50 \text{ x } 0{,}08 + 0{,}50 \text{ x } 0{,}16) \\ &\quad + (0{,}45 \text{ x } 0{,}11 + 0{,}50 \text{ x } 0{,}21 + 0{,}05 \text{ x } 0{,}28) \\ C_1 &= 0{,}3665 \end{aligned}$$

If it is estimated that up to 10% ($D_{\%}$) of the area could be dolomitic, then the run-off factor should be reduced as described earlier in this chapter. Based on the defined slopes, the following factors (D_{factor}) are used to adjust the run-off coefficient.

- Vleis and pans (slopes <3%) - 0,10
- Flat areas (3 to 10%) - 0,20
- Hilly (10 to 30%) - 0,35
- Steep areas (slopes >30%) - 0,50

$$\begin{aligned} C_{1D} &= C_1(1 - D_{\%}) + C_1 D_{\%}\left(\sum (D_{factor} \text{ x } C_{S\%})\right) \\ C_{1D} &= 0{,}3665(1 - 0{,}1) + 0{,}3665(0{,}1)(0{,}10 \text{ x } 0{,}20 + 0{,}20 \text{ x } 0{,}70 + 0{,}35 \text{ x } 0{,}1) \\ C_{1D} &= 0{,}337 \end{aligned}$$

C_{1D} is the rural run-off coefficient that incorporates the effect of the dolomitic area.

The influence of initial saturation is incorporated by means of an adjustment factor as detailed in **Table 3.8**. Using these adjustment factors (F_T) for rural areas, the run-off coefficients (C_{1D}) for the 1:20 and 1:50 year return periods are adjusted as follows with F_{20} = *0,67* and F_{50} = *0,83*.

$$C_{1T} = C_{1D} \text{ x } F_T \qquad \text{… (3.31)}$$

Thus $C_{1(20)}$ = *0,2258* and $C_{1(50)}$ = *0,2797*

The run-off coefficient for the **urban area** (C_2) is calculated using the defined catchment characteristics (**Table 3.18**) and the most conservative coefficients of **Table 3.7** (for this example).

$$\begin{aligned} C_2 &= (0{,}20 \text{ x } 0{,}10 + 0{,}10 \text{ x } 0{,}20) \\ &\quad + (0{,}40 \text{ x } 0{,}50 + 0{,}05 \text{ x } 0{,}70) \\ &\quad + (0{,}50 \text{ x } 0{,}80) \\ &\quad + (0{,}10 \text{ x } 0{,}70 + 0{,}10 \text{ x } 0{,}95) \\ C_2 &= 0{,}48 \end{aligned}$$

Table 3.18: Catchment characteristics (Urban)

Urban (C_2)	
Use	**%**
Lawns	
- Sandy, flat (<2%)	20
- Sandy, steep (>7%)	10
- Heavy soil, flat (<2%)	0
- Heavy soil, steep (>7%)	0
Residential areas	
- Houses	40
- Flats	5
Industry	
- Light industry	5
- Heavy industry	0
Business	
- City centre	0
- Suburban	10
- Streets	10
- Maximum flood	0

The combined run-off coefficient is calculated as follows:

$$C_T = \alpha\, C_{1T} + \beta\, C_2 + \gamma\, C_3 \qquad \text{… (3.32)}$$

With α = 0,4; β = 0,6 and γ = 0,0.

$C_{20} = 0{,}3783$

$C_{50} = 0{,}3999$

Step 11: Determine the peak flow for each of the required return periods utilising the simple linear relationship:

$$Q_T = \frac{C_T I_T A}{3{,}6} \qquad \text{… (3.33)}$$

where:

Q_T	=	peak flow rate for T-year return period (m³/s)
C_T	=	combined run-off coefficient for T-year return period
I_T	=	average rainfall intensity over catchment for a specific return period (mm/hour)
A	=	effective area of catchment (km²)
3,6	=	conversion factor

The peak flow rates based on the rational method for the 1:20 year and 1:50 year return periods (calculated by means of equation 3.33) are:

Q_{20} = 164 m³/s

Q_{50} = 224 m³/s

3.8.1.2 Alternative Rational method

Data requirements

The Alternative Rational method requires the following data:

- Area of catchment
- Length of longest watercourse and average slope to calculate time of concentration
- Catchment characteristics to calculate run-off coefficients
- Mean annual rainfall to determine average rainfall intensity based on the modified Hershfield relationship or TR102 (Equation 3.12)

Calculation procedure

Step 1: The first four steps of the Alternative Rational method are exactly the same as that of the standard Rational method. From these steps the T_C = *1,338 hours* and A = *28,5 km* was determined. The combined run-off coefficient is also calculated (see **Step 10**, C_{20} = *0,3783* and C_{50} = *0,3999)*

Step 2: Determine the representative rainfall from the available TR102 [3.15] South African Weather Service stations in and around the catchment (see **Table 3.19**).

Table 3.19: Representative rainfall station from TR102

Weather Service station	Pretoria (The Willows)						
Weather Service station no	513524						
Mean annual precipitation	647 mm						
Coordinates	25° 44' 28° 18'						
Duration (days)	**Return period**						
	2	**5**	**10**	**20**	**50**	**100**	**200**
1 day	60	83	101	121	150	175	202
2 days	75	105	129	155	192	224	259
3 days	83	117	143	171	211	245	282
7 days	110	160	199	241	303	355	412

Step 3: Based on the calculated time of concentration and representative rainfall, determine the precipitation depth. In this example the time of concentration is 80 minutes, in other words less than 6 hours, and thus the modified Hershfield relationship will be used [3.1].

$$P_{t,T} = 1{,}13\left(0{,}41 + 0{,}64\ln T\right)\left(-0{,}11 + 0{,}27\ln t\right)\left(0{,}79M^{0{,}69}R^{0{,}20}\right) \qquad \ldots (3.34)$$

where:

$P_{t,T}$ = precipitation depth for a duration of t minutes and a return period of T years (mm)
t = duration (minutes)
T = return period
M = 2-year return period daily rainfall from TR102 [3.15]
R = average number of days per year on which thunder was heard (days/year) (**Figure 3.12**)

The average number of days on which thunder was heard (R) is equal to *61* and M is *60* (from **Table 3.19**). The calculated precipitation depths are:
P_{t20} = *85,62 mm* and P_{t50} = *107,20 mm*

Step 4: Calculate the point intensity (mm/hour)
The point intensity (P_{iT}) is the point rainfall divided by the time of concentration (if T_C > 0,25 hours). If $T_C \le$ 0,25 hours, divide by 0,25 hours.

$$P_{iT} = \frac{P_{tT}}{T_C} \qquad \ldots (3.35)$$

where:

P_{iT} = point intensity for the different return periods (mm/h)
P_{tT} = precipitation depth for a duration of t minutes and a return period of T years (mm)
T_C = time of concentration (hours)

The point intensity for the 1:20 and 1:50 year return periods is P_{i20} = *63,99 mm/h* and P_{i50} = *80,12 mm/h* respectively.

Step 5: Determine the area reduction factors (ARF) for the different return periods from **Figure 3.13** or equation 3.36 below.

$$ARF = \left(90000 - 12800\ln A + 9830\ln\left(60T_C\right)\right)^{0,4} \qquad \text{… (3.36)}$$

where:

ARF	=	area reduction factor as a percentage (should be less than 100%)
A	=	catchment area (km²)
T_C	=	time of concentration (hours)

The resulting ARF = *96 %* for the 1:20 and 1:50 years return periods.

Step 6: Determine the average rainfall intensity or effective catchment precipitation

$$I_T = P_{iT}\left(\frac{ARF_T}{100}\right) \qquad \text{… (3.37)}$$

where:

I_T	=	rainfall intensity averaged over the catchment (mm/h) for the return period T.
ARF_T	=	area reduction factor as a percentage for return period T (should be smaller than 100 %)
P_{iT}	=	point intensities for the different return periods (mm/h)

The average rainfall intensities are I_{20} = *61,4 mm/h* and I_{50} = *76,9 mm/h*.

Step 7: Determine the peak flow for each of the required return periods utilising the simple linear relationship:

$$Q_T = \frac{C_T I_T A}{3,6} \qquad \text{… (3.38)}$$

where:

Q_T	=	peak flow rate for T-year return period (m³/s)
C_T	=	combined run-off coefficient for T-year return period
I_T	=	average rainfall intensity over catchment for a specific return period (mm/hour)
A	=	effective area of catchment (km²)
3,6	=	conversion factor

The peak flow rates based on the Rational method for the 1:20 year and 1:50 year return periods (calculated by means of equation 3.38) are:
***Q_{20}* = 184 m³/s** and ***Q_{50}* = 243 m³/s**

3.8.1.3 Unit Hydrograph method

Step 1: The first three steps of the Rational method described above are also applicable to the Unit Hydrograph method, thus A = *28,5 km*, L = *7,25 km* and S = *0,02146 m/m*.

Step 2: Determine the veld-type zone in which the catchment is located from **Figure 3.16**. The catchment of the Moretele Spruit falls in *Zone 8*.

Step 3: Calculate the catchment index by means of the following formula:

$$\text{Index} = \frac{L\ L_C}{\sqrt{S}} \qquad \text{… (3.39)}$$

where:

L	=	hydraulic length of catchment (km)
L_C	=	distance between outlet and centroid of catchment (km)
S	=	average slope (as for Rational method in m/m)

The measured length from the catchment outlet along the watercourse and then perpendicular to the centroid is L_C = *4,65 km*. The calculated catchment index is *230,1*.

Step 4: Determine the lag time in hours from **Figure 3.17** based on the catchment index and veld-type zone. The Lag time (T_L) equals *1,35 hours*.

Step 5: From **Table 3.11** obtain the value of K_U for the specific veld-type. For this example Ku = *0,367*.

Step 6: The peak flow rate for the unit hydrograph according to the regional classification given in **Table 3.11**, in zone 8 is calculated using the following formula:

$$Q_p = K_u \frac{A}{T_L} \qquad \text{… (3.40)}$$

where:

Q_P	=	peak flow rate of unit hydrograph (m³/s)
A	=	size of catchment (km²)
T_L	=	Lag time (hours)

The unit hydrograph peak discharge is *7,75 m³/s.*

Step 7: Obtain the mean annual rainfall (MAR), as described for the Rational method. The determined MAR for this catchment is *746,6 mm/a*.

Step 8: This step has to be repeated for different storm durations as well as for different return periods. The main aim is to determine the effective rainfall (he_{iT}) for the different storm durations with which the dimensionalised unit hydrograph peak flow could then be multiplied.

Step 8.1: Determine the point rainfall for the required return periods (P_T) based on the mean annual rainfall (MAR), the rainfall region, and the storm duration (T_{SD}). Point rainfall for various durations, normally shorter than or equal to the lag time, is obtained. **Figure 3.6** may be used to determine the point rainfall. To calculate the probable maximum flood, the point rainfall could be obtained from **Figure 3.18a** and **b**, with the maximum effective run-off from **Figure 3.18** [3.7]. For this example the point rainfalls for the 0,5 hour, 1 hour and 2 hour storms have been determined for the different return periods 1:20 year and 1:50 year (see **Table 3.20**).

Step 8.2: Calculate the point rainfall intensity (mm/hour). The point intensity (P_{iT}) is the point rainfall divided by the storm duration (T_{SD}).

$$P_{iT} = \frac{P_T}{T_{SD}} \qquad \text{… (3.41)}$$

where:

P_{iT}	=	point intensities for the different return periods (mm/h)
P_T	=	point rainfall (mm)
T_{SD}	=	storm duration (hours). If duration < 0,25 hours use 0,25 hours.

See solution in **Table 3.20**.

Step 8.3: Determine the area reduction factors (ARF_{iT}) for the different return periods based on the catchment area and different storm durations from either **Figure 3.20** or **3.21**. In this example the catchment area is small, and thus **Figure 3.20** will be used. The determined ARF_{iT} values are shown in **Table 3.20**.

Step 8.4: Calculate the average rainfall (P_{AvgiT}) for the different return periods and storm durations. This is the area reduction factor (ARF_{iT}) multiplied by the point rainfall (P_T). The average rainfall values are shown in **Table 3.20**.

Step 8.5: Determine the flood run-off factor from **Figure 3.22**. This factor is based on the average rainfall, veld-type zone and catchment area. The flood run-off factors (f_{iT}) are given in **Table 3.20**.

Step 8.6: Calculate the effective rainfall (he_{iT}) for each return period and selected storm duration by multiplying the flood run-off factors (f_{iT}) with the average rainfall values (P_{AvgiT}).

Table 3.20: Calculation of effective rainfall values (he_{iT})

Return period		**1:20**				**1:50**			
Description	**Unit**								
Storm duration (T_{SD})	hours	0,25	0,5	1	2	0,25	0,5	1	2
Point rainfall (P_T)	mm	41	63	84	97	50	77	102	118
Point intensity (P_{iT})	mm/hour	164	126	84	48,5	200	154	102	59
Area reduction factor (ARF_{iT})	%	84	88	92	95	80	85	90	94
Average rainfall (P_{AvgiT})	mm	34,44	55,44	77,28	92,15	40,00	65,45	91,80	110,92
Flood run-off factor (f_{iT})	%	12	16	19	22	13	18	22	24
Effective rainfall (he_{iT})	mm	4,13	8,87	14,68	20,27	5,20	11,78	20,20	26,62

Step 9: The maximum flood peak is obtained by multiplying the effective rainfall for specific storm durations with the unit hydrograph peak flow. The duration of storms that cause the maximum peak discharge is obtained by trial and error. Since the standard duration of a unit hydrograph is one hour (from the one-hour rainfall), the duration should be increased or decreased to make provision for other rainfall durations.

An S-curve is obtained by staggering a number of unit hydrographs by the unit duration and then summing them as shown in **Figure 3.24**. It is recommended that an S-curve be constructed in all cases.

Once the S-curve has been drawn, lagging an identical second S-curve by the duration and then subtracting one from the other provides a unit hydrograph of the lagged duration. The resulting values only need to be multiplied by a proportionate factor to obtain a new unit run-off hydrograph. This unit hydrograph could again be dimensionalised using the values of Q_P and T_L. It is thus advisable to calculate the run-off values for the original unit hydrograph at such time intervals that the duration of the required hydrographs will be divisible by these time intervals.

The dimensionless one-hour unit hydrograph for veld-type zone 8 is shown in **Figure 3.36** and the constructed S-curve in **Figure 3.37**. The rising and falling limbs of the unit hydrograph used to construct the S-curve are not equal. If the ordinates of the staggered unit hydrographs are summed, the constructed S-curve is not constantly increasing until it reaches the maximum value thereof. As illustrated in this example, this leads to an uneven S-curve as shown in **Figure 3.37**, described in paragraph 3.5.3.5. It is suggested that this be rectified as shown by preventing the S-curve values for example $(Q/Q_p)_t$ being less than $(Q/Q_p)_{t-1}$. This approach is conservative, which could probably lead to an over-estimation in calculating the volume of discharge, but should provide a conceptually correct answer in terms of the flood peak value.

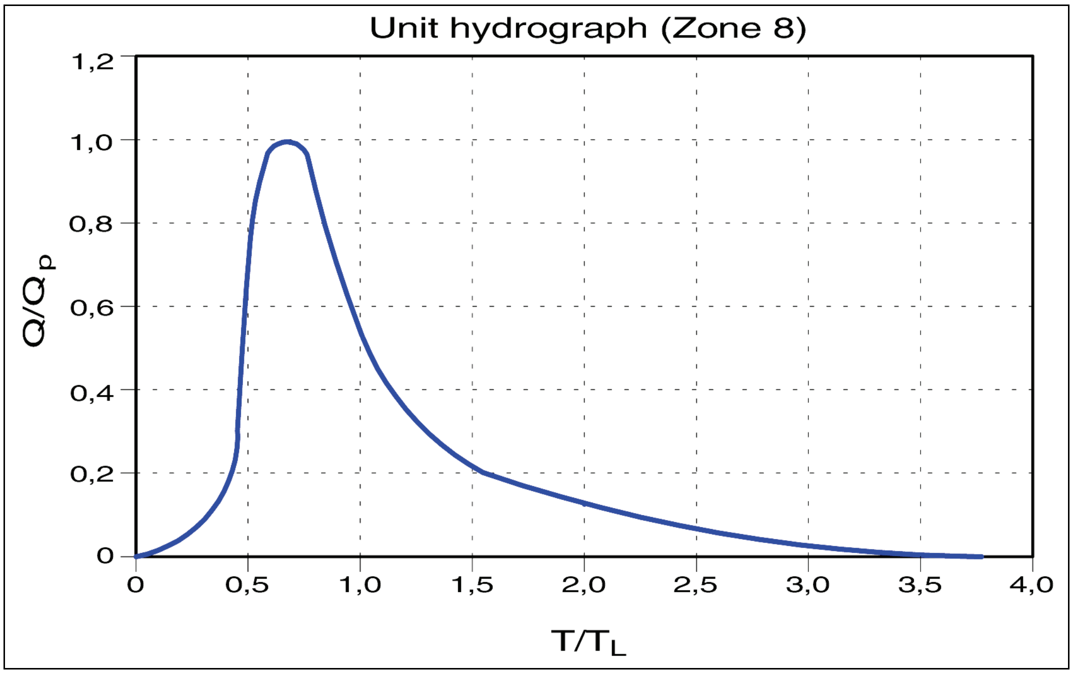


Figure 3.36: Dimensionless one-hour unit hydrograph for veld type Zone 8

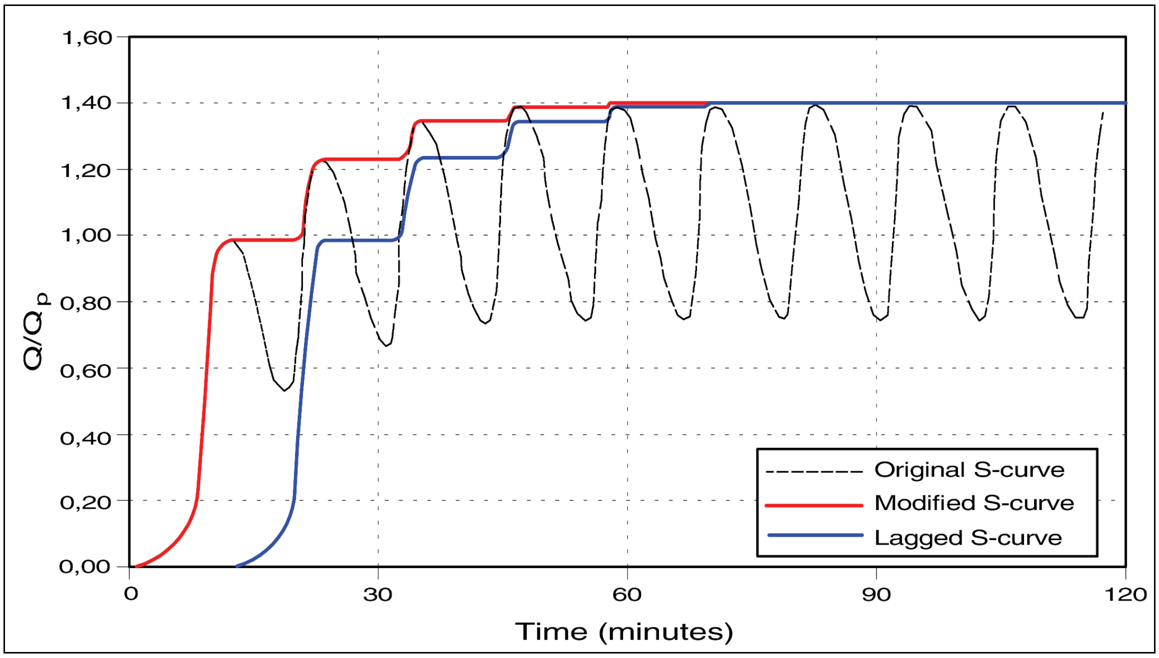


Figure 3.37: S-curve for veld type Zone 8 and lagged by 0,25 hour

Step 10: The 0,25, 0,5 and 2-hour unit hydrograph peaks are obtained by multiplying the maximum obtained values from the previous step with the peak flow rate of the unit hydrograph (Q_p). (See **Table 3.21**.)

Step 11: The last step is to calculate the peak flows for the different return periods and storm durations. The maximum peak flow for each return period is then used as the design peak flood for that specific return period as shown in **Table 3.21**. The peak values are adjusted as indicated for $Q_{PiT}/Q_P < 1$; in this case 0,9923.

Table 3.21: Peak flows utilising the discharge unit hydrograph method

Return period		**1:20**				**1:50**			
Variable	**Unit**								
Storage duration (T_{SD})	hours	0,25	0,5	1	2	0,25	0,5	1	2
Effective rainfall (he_{iT})	mm	4,13	8,87	14,68	20,27	5,20	11,78	20,20	26,62
Unit hydrograph peak (Q_{PiT})	m³/s	*24,52*	*14,48*	*7,68*	*4,77*	*24,52*	*14,48*	*7,68*	*4,77*
Peak flow (Q_{iT})	m³/s	*101,34*	*128,42*	*112,73*	*96,64*	*127,50*	*170,56*	*155,05*	*126,91*
Adjusted for $Q_{PiT}/Q_P < 1$	m³/s	*102,12*	***129,42***	*113,60*	*97,39*	*128,49*	***171,88***	*156,26*	*127,89*

The peak flow rates, based on the unit hydrograph method, for the 1:20 year and 1:50 year return periods are:
$\mathbf{Q_{20}}$ ***= 129 m³/s*** and $\mathbf{Q_{50}}$ ***= 172 m³/s***

3.8.1.4 SDF method

The calculation sequence to determine the flood peaks based on the SDF method is as follows:

Step 1: Identify the drainage basin in which the site is located from **Figure 3.25**. The catchment falls in drainage basin number *1*.

Step 2: Determine the area of the catchment. In this example it has already been calculated as A = *28,5 km²*.

Step 3: Determine the length of the main channel. In this example it has already been calculated as L = *7,25 km*.

Step 4: Determine the average slope of the catchment as described for the Rational method. The calculated average slope (S) for this example is *0,02146 m/m*.

Step 5: Apply the US Soil Conservation Service formula to determine the time of concentration T_C (hours) as suggested in HRU1/72 [(3.23)].

$$T_C = \left[\frac{0,87L^2}{1000S_{av}}\right]^{0,385} \quad \text{… (3.42)}$$

The time of concentration is as calculated earlier, i.e. T_C = *1,338 hours*.

Step 6: Convert T_C (hours) to t (minutes) and determine the point precipitation depth P_{tT} (mm) for the different the return period T (years). In this example the modified Hershfield equation will be used.

$$P_{t,T} = 1,13(0,41 + 0,64\ln T)(-0,11 + 0,27\ln t)\left(0,79M^{0,69}R^{0,20}\right) \quad \text{… (3.43)}$$

where:

M = mean of the annual daily maxima from **Table 3B.1** equals *56 mm*.

R = average number of days per year on which thunder was heard from **Table 3B.1** equals *30*.

Table 3.22: Calculating point precipitation

Return period	1:20	1:50
$\mathbf{P_{tT}}$ **(mm)**	*70,84*	*88,69*

Step 7: Multiply the point precipitation depth P_{tT} (mm) by the area reduction factor ARF (%) to determine the average rainfall over the catchment for the required return period (P_{AvgT}). The corresponding rainfall intensity I_T (mm/h) is obtained by dividing this value by the time of concentration.

$$ARF = (90000 - 12800 \ln A + 9830 \ln t)^{0,4} \qquad \text{… (3.44)}$$

Table 3.23: Calculating point intensity

Return period	1:20	1:50
ARF (%)	*96*	*96*
P_{AvgT} (mm)	*68,00*	*85,14*
I_T (mm/hour)	*50,83*	*63,63*

Step 8: The above steps constitute the standard procedures used in the conventional rational method. The SDF uses calibrated run-off coefficients C_2 (2-year return period) and C_{100} (100-year return period) from **Table 3B.1** instead of determining these from catchment characteristics. The run-off coefficients for the range of return periods are derived by applying the return period factors Y_T in **Table 3.13**, using the equation below:

$$C_T = \frac{C_2}{100} + \left(\frac{Y_T}{2,33}\right)\left(\frac{C_{100}}{100} - \frac{C_2}{100}\right) \qquad \text{… (3.45)}$$

With the calibrated coefficients being C_2 = 10 % and C_{100} = 40 %, and the return period factors Y_{20} = *1,64* and Y_{50} = *2,05* the run-off coefficients are calculated for the 1:20 and 1:50 year return periods:

C_{20} = *0,3112* and C_{50} = *0,3639*

Step 9: Finally, the flood peak Q_T (m³/s) for the required return period is calculated as follows:

$$Q_T = \frac{C_T I_T A}{3,6} \qquad \text{… (3.46)}$$

Table 3.24: Calculating flood peaks

Return period	1:20	1:50
C_T	0,3112	0,3639
I_T (mm/hour)	50,83	63,63
Q_T (m³/s)	***125***	***183***

Step 10: The SDF hydrograph is triangular in shape with the duration of the rising limb equal to the time of concentration T_C (hours), and that of the falling limb equal to twice the time of concentration. Use linear interpolation between these two values.

3.8.1.5 Empirical methods

Peak discharges for return periods less than or equal to 100 years can be determined by means of an empirical deterministic method developed by Midgley and Pitman [3.10, 3.11]. The formula reads:

$$Q_T = 0,0377 K_T P A^{0,6} C^{0,2} \qquad \text{… (3.47)}$$

where:

- Q_T = peak flow for T return period (m³/s)
- K_T = coefficient based on veld-type region (see **Figure 3.16** and **Table 3.14**).
- P = mean annual rainfall over catchment (mm/a) (see **Figure 3.7**)

and $C = \frac{A\sqrt{S}}{L\,L_C}$ (Catchment parameter with regard to reaction time) … (3.48)

where:

A	=	area of catchment (km²)
S	=	average slope of stream (m/m)
L	=	hydraulic length of catchment (km)
L_C	=	distance between outlet and centre of gravity of catchment (km)

With the catchment being in veld-type zone 8, the K_T values are *0,57* and *0,79* for the 1:20 and 1:50 year return periods respectively. The calculated catchment parameter C with regard to reaction time is *0,1238*.

The flood peaks as determined with the empirical method for the 1:20 and 1:50 year return periods are ***78,85 m³/s*** and ***109,28 m³/s*** respectively.

3.8.1.6 Comparison of solutions

Table 3.25 summarises the results for the five methods used in this example.

Table 3.25: Comparison of solutions

Method	Return period	
	1:20	1:50
Rational	164	224
Alternative Rational	184	243
Unit Hydrograph	129	172
SDF	125	183
Empirical	79	109

Table 3.26 illustrates the effects/sensitivity with regard to some of the catchment parameters implied or determined in this example, assuming the other parameters were calculated correctly.

Table 3.26: What if…

Nr	Question	Comment
1	The area of the catchment is actually 30 km² (1,5 km² larger). What will be the effect on the floods calculated with the Rational method be?	The area is directly proportional to the flood peak. An increase in area results in an increase in flood peak. Some of the other parameters such as time of concentration will, however, also change. New flood peaks will be: Q_{20} = 173 m³/s and Q_{50} = 238 m³/s
2	What will the effect of future urbanisation (increase of 15% of the total area) be?	The run-off coefficient will increase resulting in an increase in flood peak. This however depends on the topographical characteristics see paragraph 3.3.2. New flood peaks will be: Q_{20} = 181 m³/s and Q_{50} = 249 m³/s
3	How does the 1:50 year flood peak utilizing the Q_T/Q_{RMF} relationship in **Appendix 3D** compare with the other calculated flood peaks?	The Q_{RMF} = 534 m³/s using **Figure 3.28** and **Table 3.15**. From **Appendix 3D** the Q_T/Q_{RMF} = 0,416 resulting in the Q_{50} = 222 m³/s which is very similar in magnitude as the Rational and SDF methods.
4	What effect will an increase of 10% of the MAR have on the floods calculated with the Rational method?	It has an influence on the anticipated rainfall intensity for the specific return period storms. New flood peaks will be: Q_{20} = 177 m³/s and Q_{50} = 250 m³/s, i.e. approximately 8% larger in magnitude.
5	What effect does the dolomites have on the calculated flood peaks?	Where dolomite occurs, reduction factors are recommended for the dolomitic parts of a catchment to be applied to C_s. With no dolomites the flows will increase to Q_{20} = 168 m³/s and Q_{50} = 237 m³/s

3.8.2 Worked example 3.2 - Large catchment

The second worked example is a design flood calculation for a new low-level bridge across the Tsitsa River, which runs through the Eastern Cape in a south-easterly direction (see **Figure 3.38**).

The position of the proposed Tsitsa low-level river bridge is shown on **Figure 3.38**.

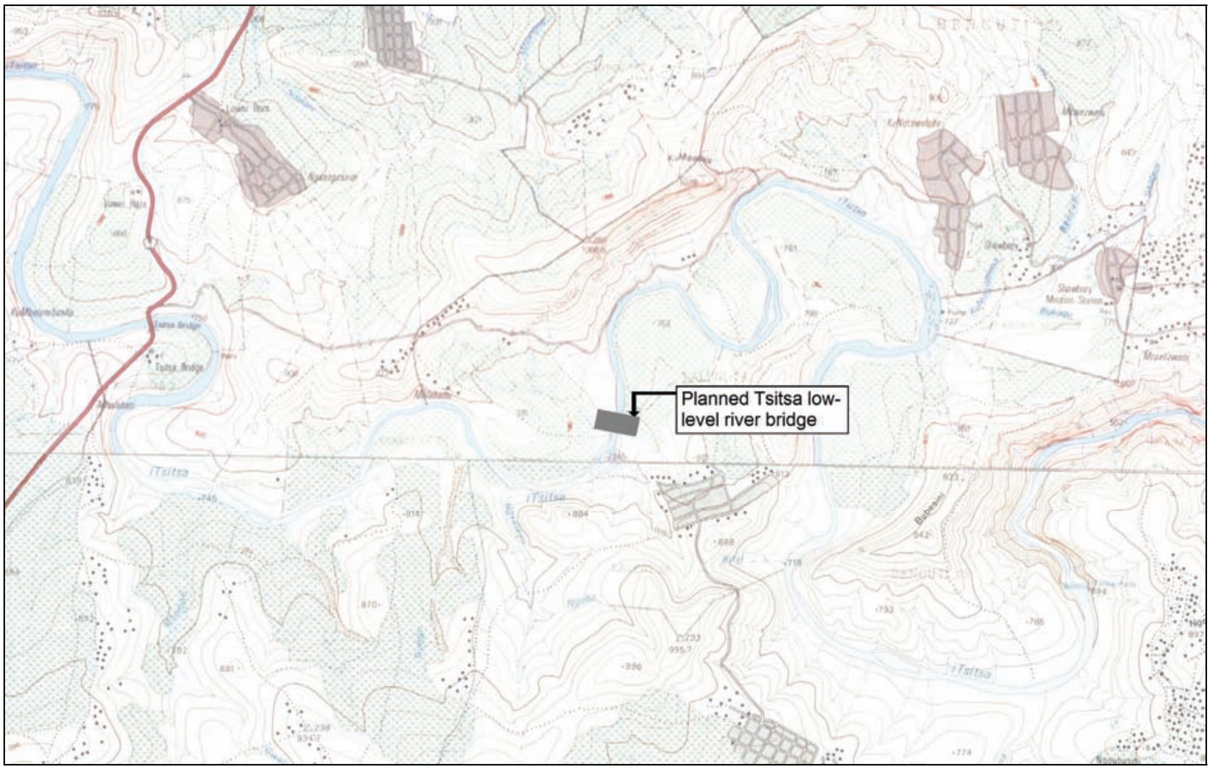


Figure 3.38: Proposed Tsitsa low-level river bridge position

The catchment characteristics were determined (see **Table 3.27**), and are used in the calculation of the flood peaks for various recurrence intervals.

Table 3.27: Catchment characteristics

Description of characteristic	Determined value	Comment
Catchment area (see **Figure 3.39**)	4318 km²	Catchment area may be clearly defined
Length of longest watercourse (see **Figure 3.39**)	179,5 km	Starts at Antelope Spruit, joins the Tsitsana River and further downstream joins the Ixnu River to form the Tsitsa River
Height difference (1085-method) (See **Figure 3.40**)	500 m	Total height difference equals 1 814 m, very steep slopes along the upper reaches of the water course
Average catchment slope	0,37%	See detailed description below
Distance to catchment centroid	85 km	
SDF Drainage basin number	23	
Average rainfall	860 mm	Based on calculated average from a number of weather stations in the T35 drainage basin [3.21]
RMF K-factor	5,0 – 5,2	Catchment area falls within regions K5 and K6 (assume highest value). (See **Figure 3.28**)

Table 3.27: Catchment characteristics (continued)

Description of characteristic	Determined value	Comment
Description of catchment run-off characteristics	Rural area only with a combination of flat and hilly zones, steep slopes along perimeter of catchment and pans with slopes <3%; permeability varying from permeable to semi-permeable, light bush and cultivated lands, as well as grasslands.	
Generalised veld type zone (**Figure 3.16**)	Zone 2	
Gauging station (see **Photograph 3.6**)	T3H016 Tsitsa River @ Xonkonxa Latitude: 31°14’13’’ Longitude: 28°51’15’’	This gauging station is close to the N2 river bridge approximately 5 km upstream of the proposed bridge site

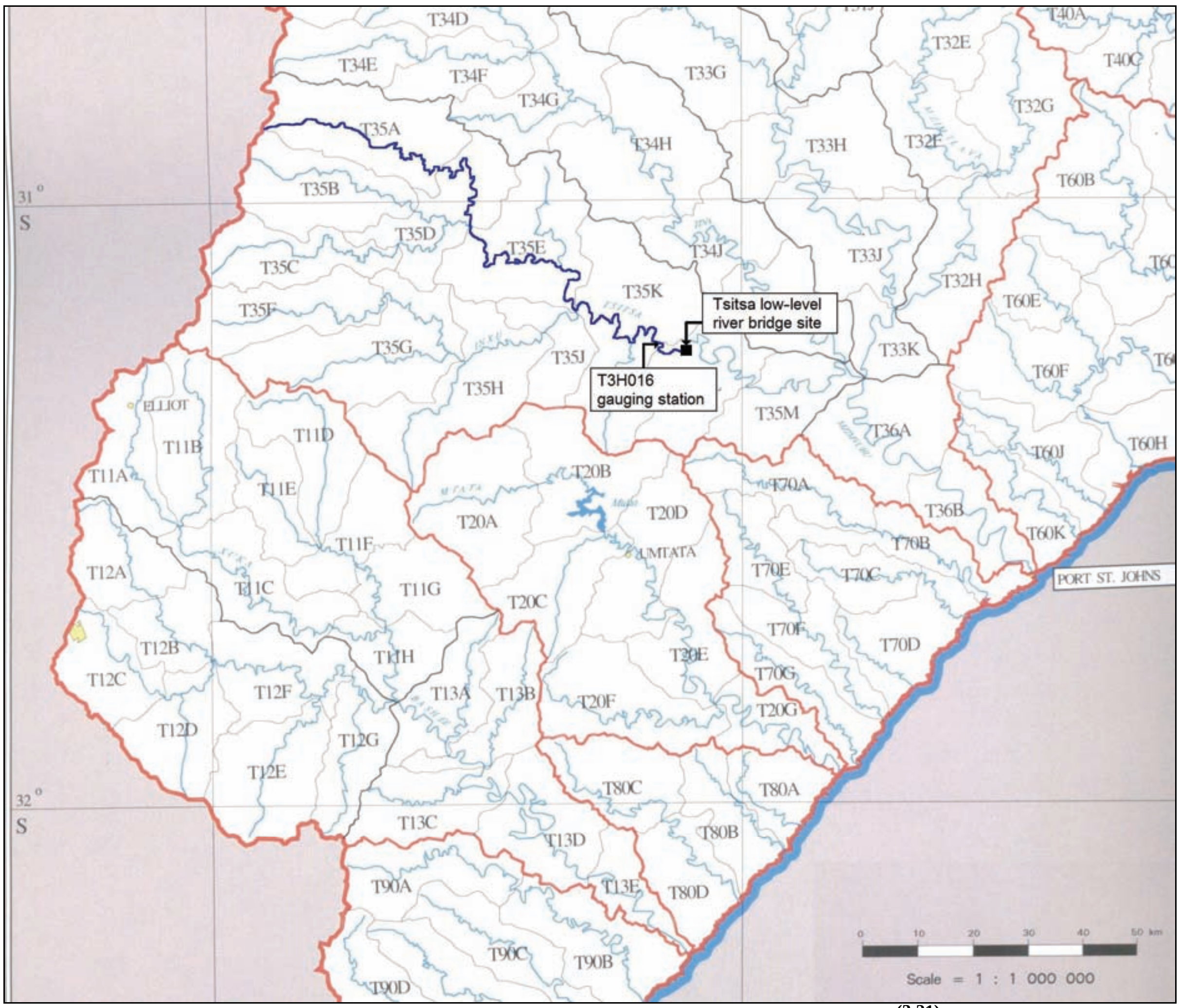


Figure 3.39: Catchment area and longest watercourse [(3.21)]

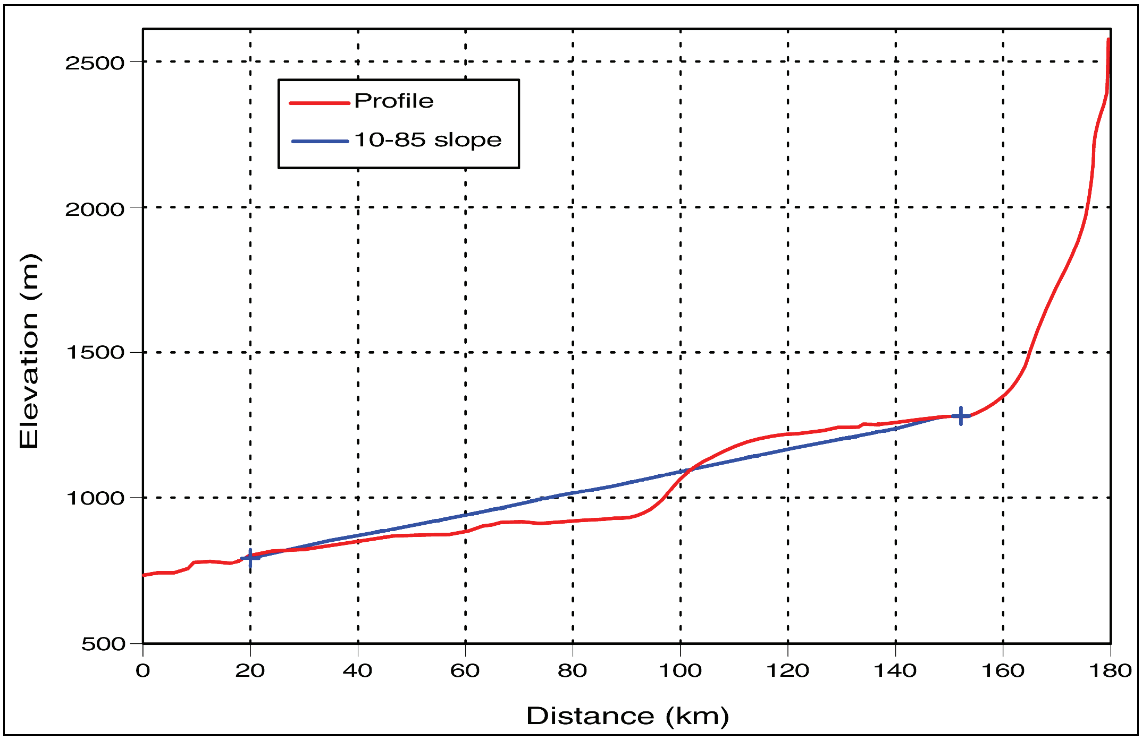
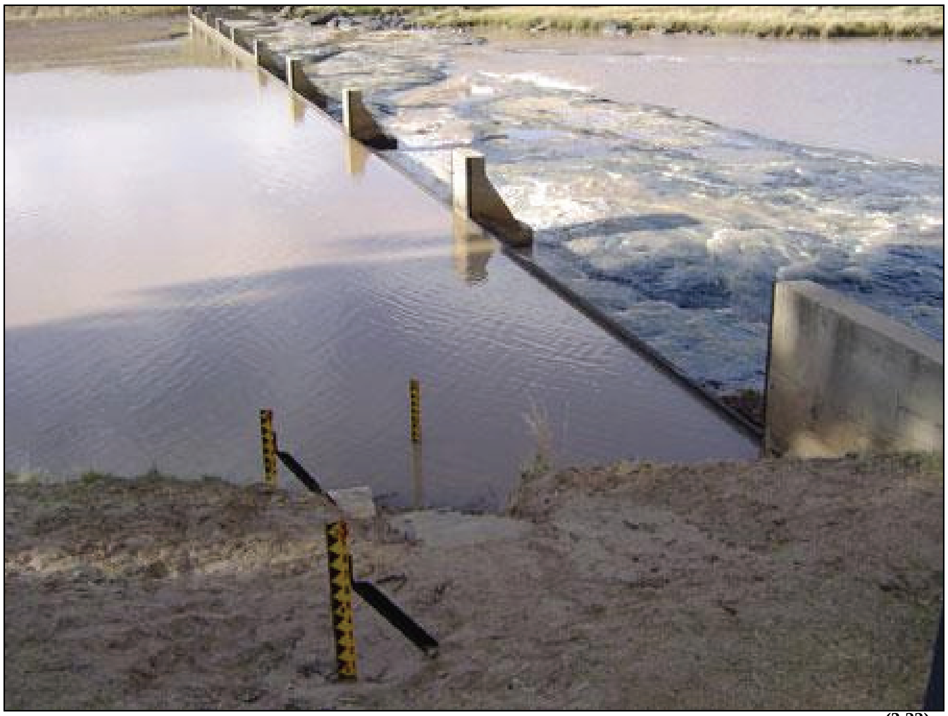


Figure 3.40: Watercourse profile and average slope



Photograph 3.6: Gauging station (T3H016 Tsitsa River @ Xonkonxa)[3.22]

3.8.2.1 Statistical method

There is a gauging station approximately 5 km upstream of the proposed bridge site, and flow data from 1951/52 until 1997/98 are available. Historical flood data for the gauging station were obtained from the Department of Water Affairs and Forestry [3.21]. The gauging station, T3H016, is situated at the N2-Bridge. The data unfortunately contain periods, full hydrological years and parts thereof, during which the flow was not measured. The hydraulic capacity of the structure, which is 1091 m³/s, was exceeded on at least three occasions. The data as used in the statistical analysis are shown in **Table 3.28**.

Table 3.28: Historical annual maximum flood peaks for T3H016

Year	Discharge (m^3/s)	Comment	Description	Adopted peak flow value (m^3/s)
1951/52	60			60
1952/53	112			112
1953/54	120	Incomplete year	Includes summer period	120
1954/55	197	Incomplete year	Too short year	-1
1955/56	126			126
1956/57	188			188
1957/58	188			188
1958/59	274			274
1959/60	185			185
1960/61	52	Incomplete year	Too short year	-1
1961/62	338			338
1962/63	788			788
1963/64	488			488
1964/65	322			322
1965/66	343			343
1966/67	492			492
1967/68	111			111
1968/69	380	Incomplete year	Includes summer period	380
1969/70	303	Incomplete year	Too short year	-1
1970/71	660			660
1971/72	1091	Rating limit exceeded	From TR105	1364
1972/73	229			229
1973/74	326			326
1974/75	297			297
1975/76	1091	Rating limit exceeded	From TR105	1699
1976/77	1091	Rating limit exceeded	From TR105	1347
1977/78	926			926
1978/79	82	Incomplete year	Too short year	-1
1979/80	97	Incomplete year	Includes summer period	97
1980/81	-1	Missing data		-1
1981/82	-1	Missing data		-1
1982/83	-1	Missing data		-1
1983/84	295			295
1984/85	851			851
1985/86	443			443
1986/87	332			332
1987/88	881			881
1988/89	904			904
1989/90	672			672
1990/91	90			90
1991/92	301			301
1992/93	110			110
1993/94	580			580
1994/95	273			273
1995/96	995	Incomplete data	Includes summer period	995
1996/97	486	Incomplete data	Includes summer period	486
1997/98	709	Incomplete data	Includes summer period	709
1998 - 2004	-1	Missing data		-1

The flood peak of March 1976 measured at the Tsitsa River gauging station was one of the flood peaks used in the derivation of the Francou-Rodier K-values for this specific region. The historical data indicated that the flood peak was higher than 1091 m³/s, which is the maximum capacity of the gauging station. In a technical report entitled, "Maximum flood peak discharges in South Africa: An empirical approach", Report No. TR105, by Kovács[3.11], a peak flow of 1 699 m³/s was used in deriving the Francou Rodier K-value. The maximum recorded water level was 3,48 m. This represented the highest peak flow recorded up to March 1976. The other high peak flow of February 1972 was less than this peak flow (i.e. between 1 091 and 1 699 m³/s), and it was estimated as 1 364 m³/s.

Another peak flow event, during which the measuring weir was overtopped, occurred in October 1976. It is estimated that this peak flow was 1347 m³/s.

Based on the historical flow records the results from statistical analyses are shown in **Table 3.29**. Equations and statistical tables are included in **Appendix 3A**.

Table 3.29: Statistical analyses for Tsitsa river gauging station T3H016

Return period	Extreme value Type 1	General extreme value	Log normal	Log Pearson Type 3	Log extreme value
2	415	410	348	356	304
5	749	749	702	706	635
10	970	971	1012	998	1033
20	118	1191	1366	1320	1649
50	1458	1481	1922	1796	3021
100	1664	1703	2428	2196	4754

The frequency distribution curve that fitted the data the best, was the Log Pearson Type 3 curve (LP3) (see **Figure 3.41**).

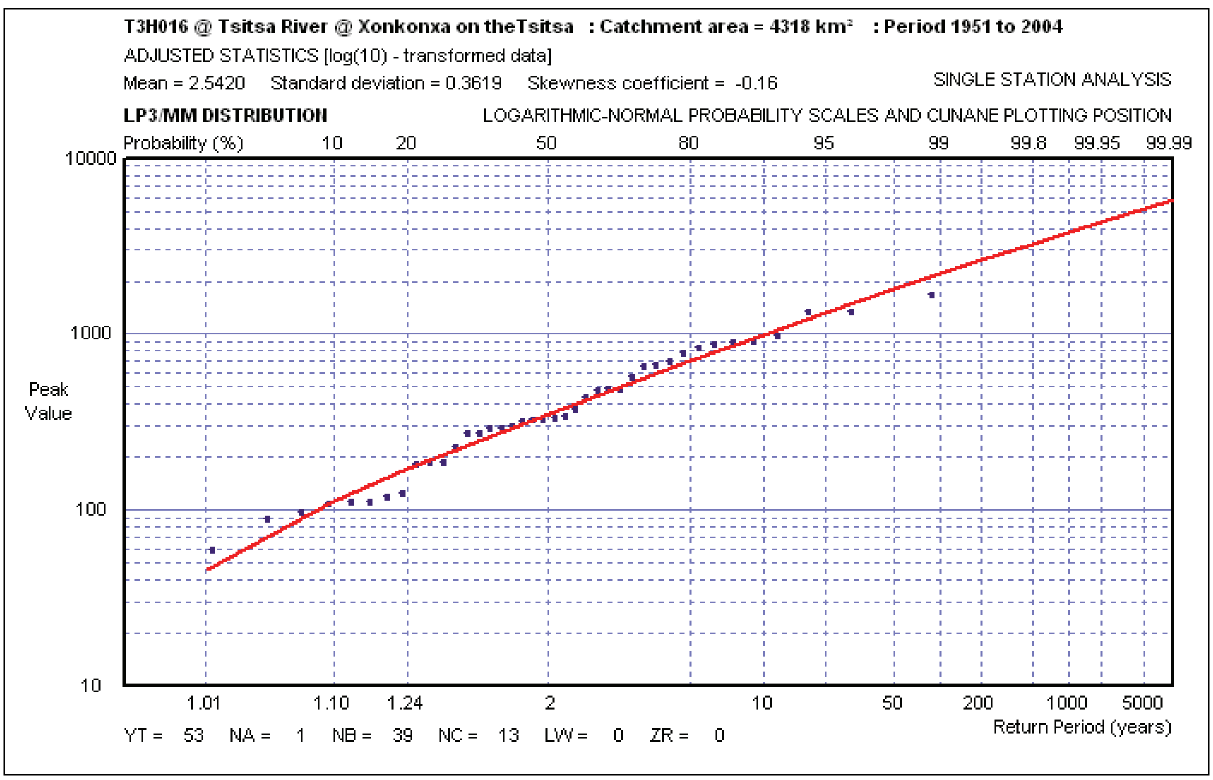


Figure 3.41: Log Pearson Type 3 fit through historical data points

The following gauge record information relates to **Figure 3.41**.
YT (record length in years) = 53
NA (peaks ≥ high threshold) = 1
NB (peaks between thresholds excluding missing data) = 39
LW (non-zero peaks below low threshold) = 0
ZR (zero flows) = 0
NC (missing data) = 13

3.8.2.2 SDF method

The calculation sequence to determine the flood peaks has been described in the first worked example, and will therefore not be repeated here. The main results are presented in **Table 3.30**. In this example the point precipitation is obtained from the weather service station selected for this basin from TR102; i.e. Station nr. 180439 @ INSIZWA.

Table 3.30: Results of SDF calculation

Description	Answer obtained
Area (km²)	4318
L (km)	179,5
S (m/m)	0,0037
T_C (hours)	31,16
M (mm)	60
R (days)	45
C_2 (%)	10
C_{100} (%)	80

Return period	1:10	1:20	1:50	1:100
P_{tT}	103	120	145	166
ARF (%)	80	80	80	80
P_{avgT} (mm)	82	95	116	132
I_T (mm/hour)	2,63	3,05	3,72	4,24
C_T	0,485	0,593	0,716	0,80
Q_T (m³/s)	1530	2169	3195	4069

3.8.2.3 Empirical methods

Peak discharges for return periods less than or equal to 100 years could be determined by means of an empirical deterministic method developed by Midgley and Pitman [3.10, 3.11]. The formula reads:

$$Q_T = 0{,}0377 K_T P A^{0,6} C^{0,2} \qquad \text{… (3.49)}$$

where: Q_T = peak flow for T-year return period (m³/s)
K_T = constant for T-year return period (Obtained from **Table 3.14**) and in this example the catchment falls within *Zone 2 (All year)* with K_T values of K_{10} = *0,83*, K_{20} = *1,04*, K_{50} = *1,36* and K_{100} = *1,60*
A = size of catchment (km²)
P = mean annual rainfall over catchment = *860 mm/a* (See **Table 3.27**)

and

$$C = \frac{A\sqrt{S}}{L L_c} \quad \text{(Catchment parameter with regard to reaction time)} \qquad \text{… (3.50)}$$

where:
A = area of catchment = *4318 km²*
S = average slope of stream = *0,0037 m/m*
L = hydraulic length of catchment = *179,5 km*
L_C = distance between outlet and centroid of catchment = *85 km*

The calculated catchment parameter C with regard to reaction time is *0,01721*.

The flood peaks as determined using the empirical method (equation 3.44) for the different return periods are shown in **Table 3.31**.

Table 3.31: Flood peaks based on empirical method

Return period	Peak flows (m^3/s)
10	1 812
20	2 270
50	2 969
100	3 493

The regional maximum flood may be calculated as follows:

Step 1: Determine the catchment area: *4318 km²*.

Step 2: Identify the region in which the site is located (**Figure 3.28**). In this example, as shown in **Table 3.27**, the region is K6 (higher K-value). Note that the regions on the map refer to the location of the site and not to the catchment. Only if the site is located near a boundary between regions would it be necessary to consider adjusting the K-factor.

Step 3: Utilise the equation provided in **Table 3.15** to calculate the RMF as *8059 m³/s*

3.8.2.4 Comparisons of solutions

Comparing the calculated flood peaks below (**Table 3.32**) provides an overview of the range of expected floods. Based on the flood calculations above, a structurally sound low-level bridge structure was designed to withstand the high floods and provide a safe crossing during the lower floods (**Photograph 3.7**). Further analysis was also performed on the historical flow data to ensure that the bridge will not be inundated for prolonged periods, cutting off communities from each other.

Table 3.32: Comparison of calculated peak flows (m^3/s)

Return period	Empirical	Standard design flood	Regional maximum flood (RMF)	Statistical (LP3)
10	1 812	1 530		999
20	2 270	2 169		1 323
50	2 969	3 195	3 951*	1 790
100	3 493	4 069	4 796*	2 202
			8 059	

*Using Q_T/Q_{RMF} ratios as detailed in **Appendix 3D**.

Photograph 3.7: Tsitsa crossing (also see Photograph 6.4 in Chapter 6)

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APPENDIX 3A
STATISTICAL ANALYSIS

Single station direct statistical analysis

The following frequency distributions are discussed for untransformed and $\log_{10}$-transformed data:

- **Untransformed data**
  Normal, Extreme Value Type 1 and General Extreme Value
- **$\mathbf{Log_{10}}$-transformed data**
  Log-Normal, Log-Gumbel and Log-Pearson Type III

Step 1: Determine the mean, standard deviation and skewness coefficient of the raw data and the $\log_{10}$ transformed data as follows:

Mean $$\bar{x} = \frac{\sum x}{N} \qquad \text{...(3A.1)}$$

Standard deviation $$s = \left[\frac{\sum \left(x - \bar{x}\right)^2}{N - 1}\right]^{0,5} \qquad \text{...(3A.2)}$$

Skewness coefficient $$g = \left(\frac{N}{(N-1)(N-2)}\right)\left(\frac{\sum \left(x - \bar{x}\right)^3}{s^3}\right) \qquad \text{...(3A.3)}$$

Coefficient of variation $$c_V = \frac{s}{\bar{x}} \qquad \text{...(3A.4)}$$

where:

x	=	observed value (or of the logarithm of the observed value for the log distributions)
$\bar{x}$	=	mean of observed values (or of the logarithm of the observed value for the log distributions)
N	=	the total number of observations
s	=	the standard deviation of the observed values (or of the logarithm of the observed values)
g	=	skewness coefficient
c_V	=	coefficient of variation

Step 2: The peak value for the desired return period and assumed distribution function can be derived for each of the frequency distributions as follows:

- **Normal distribution**

  The normal distribution is applicable where the observed values represent the effects of a large number of independent processes. The distribution is symmetrical about the mean and is therefore only suitable for data where the skewness coefficient (g) is equal to, or close to zero. The spread about the mean is a function of the coefficient of variation, Alexander (3.1). For high coefficient of variation values, the bottom tail may extend below zero and may result in negative flows being generated when the distribution is applied to untransformed data.

  The standardized normal distribution has a cumulative distribution function:

  $$G(y) = \int_{-\infty}^{y} \frac{1}{\sqrt{2\pi}} e^{-0,5\left[y^2\right]dy} \qquad \text{...(3A.5)}$$

  where y is the standardized variable and is related to x by:

  $$y = \frac{\left(x - \bar{x}\right)}{s} \qquad \text{...(3A.6)}$$

The value of y for a given value of G(y) cannot be solved directly from Equation 3A.4, and hence published tables have to be used. Based on the return period, read from **Table 3A.1b** the value of G(y) and obtain y from **Table 3A.1a**.

$$Q_T = \bar{x} + sy \qquad \text{…(3A.7)}$$

- **Extreme Value Type 1 (EV1/MM) distribution**

  From **Table 3A.4** (g = 1,14) read the value of W_T for the required return period. Calculate Q_T directly using:

$$Q_T = \bar{x} + s(0{,}780W_T - 0{,}450) \qquad \text{…(3A.8)}$$

- **General Extreme Value (GEV/MM) distribution**

  For the known value of the skewness coefficient (g) read off the value W_T from **Table 3A.4** and the values of k, E(y) and var(y) from **Table 3A.2** by using linear interpolation.

  For EV2 distribution:

$$Q_T = \bar{x} + \sqrt{\frac{s^2}{\mathrm{var(y)}}}\left(1 - E(y) - kW_T\right) \qquad \text{…(3A.9)}$$

  For EV3 distribution:

$$Q_T = \bar{x} + \sqrt{\frac{s^2}{\mathrm{var(y)}}}\left(1 - E(y) + kW_T\right) \qquad \text{…(3A.10)}$$

- **Log-normal (LN/MM) distribution**

  For this distribution, the logarithms of the data are assumed to be normally distributed. Based on the skewness coefficient (g), obtain the value W_T for the required return period from **Table 3A.3**.

$$Q_T = \mathrm{antilog}\left[\overline{\log(x)} + s_{\log}\ W_T\right] \qquad \text{…(3A.11)}$$

where:

s_{log} = the standard deviation of the logarithms of the observed values as shown in equation 3A.11

$$s_{\log} = \left[\frac{\sum\left(\log(x) - \overline{\log(x)}\right)^2}{N-1}\right]^{0,5} \qquad \text{…(3A.12)}$$

and

$\overline{\log(x)}$ = the logarithm of the mean of the observed values

Confidence bands

The confidence, with which the values of the magnitude-return period relationships are estimated, depends on the number of observations contained in the data set. The greater the number of observations, the greater the degree of assurance, and subsequently the narrower the confidence band [3.1]. The displacement of the two-sided 95% confidence band about the estimated value can be read from **Table 3A.3** where N is the number of observations. The 95% confidence limits are:

$$Q_{T(95\%)} = \text{antilog}\left[\overline{\log(x)} + s_{\log}\,(W_T \pm W_\alpha)\right] \qquad \ldots(3A.13)$$

where:

W_α = displacement of the confidence band (column 5 in **Table 3A.3**)

- **Log-Gumbel (Log-Extreme Value Type 1) (LEV1/MM) distribution**

  From **Table 3A.4** (g = 1,14) read the value of W_T for the required return period. Calculate Q_T directly using:

$$Q_T = \text{antilog}\left[\overline{\log(x)} + s_{\log}\,(0{,}780W_T - 0{,}450)\right] \qquad \ldots(3A.14)$$

- **Log-Pearson Type III (LP3/MM) distribution**

  From **Table 3A.3** determine the value of W_T for the known skewness coefficient (g) of the log-transformed data by linear interpolation.

$$Q_T = \text{antilog}\left[\overline{\log(x)} + s_{\log}\,W_T\right] \qquad \ldots(3A.15)$$

Based on an example from *Flood Risk Reduction Measures* by WJR Alexander[(3.1)] the incorporation of the influence of historical information, missing data and outliers is required to determine the confidence of the results.

It is thus required to calculate the historically weighted mean ($\bar{x}_h$), standard deviation (s_h) and skewness coefficient (g_h).

$$\bar{x}_h = \frac{\left((WT)\sum x_b + \sum x_a\right)}{\left(YT - (WT)(LW)\right)} \qquad \ldots(3A.16)$$

$$s_h = \left[\frac{\left((WT)\sum d_b^2 + \sum d_a^2\right)}{\left(YT - (WT)(LW) - 1\right)}\right]^{0,5} \qquad \ldots(3A.17)$$

$$g_h = \left[\frac{\dfrac{\left(YT - (WT)(LW)\right)\left((LW)\sum d_b^3 + \sum d_a^3\right)}{s^3}}{\left((YT)(WT) - 1\right)\left(YT - (WT)(LW) - 2\right)}\right] \qquad \ldots(3A.18)$$

where:

YT	=	total time span (= NA + NB + NC)
WT	=	weight applied to data = (YT – NA) / NB
NA	=	floods equal to or above the high threshold
NB	=	floods between high and low thresholds
NC	=	missing data
LW	=	low outliers including zero flows
ZR	=	zero flows

and where:

x_a	=	is the value of a peak equal to or above the high threshold
x_b	=	is the value of a peak below the high threshold
d_a and d_b	=	are deviations of $x_a + x_b$ from $\bar{x}_h$

All values being the logarithms of the data.

These historically weighted values of the mean, standard deviation and the skewness coefficient are then used in the equations for the LN/MM, LP3/MM, EV1/MM and GEV/MM distribution in the usual way [(3.1)].

For a detailed description of the adjustment required when:

- low outliers are removed from the data;
- gauged zero flows exist; or
- how to handle missing data;

see *Flood Risk Reduction Measures* by Alexander [(3.1)].

Using the data set as provided in **Chapter 3**, Section 3.8.2.1, Example 3.2 for the Tsitsa River, and utilizing Equations 3A.1 to 3A.15 (if the missing data is not included in the statistical analysis) will provide the following results:

Table 3A.5a: Summary of parameters - Example 3.2 (missing data excluded)

Variable	Untransformed data	Transformed data
$\bar{x}$	484,550	2,5463
YT	40	40
NC	0	0
s	390,677	0,366
g	1,344	-0,1462
c_V	0,8063	0,1437

Table 3A.5b: Summary of results – Example 3.2 (missing data excluded)

Return period	N/MM	EV1/MM	GEV/MM	LN/MM	LEV1/MM	LP3/MM
2	485	421	414	352	307	358
5	715	778	758	714	645	719
10	985	1019	991	1035	1057	1022
20	1127	1257	1221	1401	1696	1359
50	1287	1573	1527	1980	3126	1850
100	1394	1819	1764	2507	4953	2286

The next set of results is based on the historically weighted mean ($\bar{x}_h$), standard deviation (s_h) and skewness coefficient (g_h) (**Table 3A.6a** and **b**) which incorporates the missing data.

Table 3A.6a: Summary of parameters - Example 3.2 (missing data included)

Variable	Untransformed data	Transformed data
$\bar{x}_h$	476,912	2,542
YT	53	53
NC	13	13
s_h	378,305	0,362
g_h	1,304	-0,157
c_V	0,793	0,142

Table 3A.6b: Summary of results - Example 3.2 (missing data included)

Return period	N/MM	EV1/MM	GEV/MM	LN/MM	LEV1/MM	LP3/MM
2	477	416	418	348	304	355
5	700	749	726	702	635	706
10	962	971	934	1012	1034	999
20	1099	1183	1137	1367	1651	1323
50	1254	1457	1407	1924	3022	1790
100	1357	1664	1615	2429	4764	2202

Step 3: Graphical representation of historical data

Arrange the observed data in descending order of magnitude and assign to each value a rank number starting from one. Determine the plotting position (return period) for each value using the Weibull formula. The general equation is given below and the values for the constants *a* and *b* are provided in **Table 3A.7**.

$$T = \frac{n + a}{m - b} \qquad \ldots (3A.19)$$

where:

T = return period in years

n = length of record in years

m = number, in descending order, of the ranked annual peak floods

a = constant (see **Table 3A.7**)

b = constant (see **Table 3A.7**)

If the horizontal axis has a probability classification, the probability (P) is calculated as:

$$P = \frac{1}{T} \qquad \ldots (3A.20)$$

Some of the commonly used plotting positions recommended for use in hydrological analyses are given in **Table 3A.7**. If several distributions are plotted on a single graph, then the general purpose Cunane plotting position should be used.

Table 3A.7: Commonly used plotting positions (3.1)

Type	Plotting position	Distribution
Weibull (1939)	a = 1 & b = 0	Normal, Pearson 3
Blom (1958)	a = 0,25 & b = 0,375	Normal
Gringorten (1963)	a = 0,12 & b = 0,44	Exponential, EV1 & GEV
Cunane (1978) average of above two	a = 0,2 & b = 0,4	General purpose
Beard (1962)	a = 0,4 & b = 0,3	Pearson 3
Greenwood (1979)	a = 0 & b = 0,35	Wakeby, GEV

Plot the values against their estimated return periods on log-probability paper; draw the best fitting straight line through the plotted points and extrapolate to determine the estimated maximum value for the required return period.

Table 3A.1: Properties of the standardized normal distribution

Table 3A.1a

Standardized normal distribution				
y	G(y)%		y	G(y)%
0,00	50,00		-0,00	50,00
0,05	51,99		-0,05	48,01
0,10	53,98		-0,10	46,02
0,15	55,96		-0,15	44,04
0,20	57,93		-0,20	42,07
0,25	59,87		-0,25	40,13
0,30	61,79		-0,30	38,21
0,35	63,68		-0,35	36,32
0,40	65,54		-0,40	34,46
0,45	67,36		-0,45	62,64
0,50	69,14		-0,50	30,86
0,55	70,88		-0,55	29,12
0,60	82,57		-0,60	27,43
0,65	74,22		-0,65	25,78
0,70	75,81		-0,70	24,19
0,75	77,34		-0,75	22,66
0,80	78,81		-0,80	21,19
0,85	80,24		-0,85	19,76
0,90	81,59		-0,90	18,41
0,95	82,89		-0,95	17,11
1,00	84,13		-1,00	15,87
1,05	85,31		-1,05	14,69
1,10	86,43		-1,10	13,57
1,15	87,49		-1,15	12,51
1,20	88,49		-1,20	11,51
1,25	89,44		-1,25	10,56
1,30	90,32		-1,30	9,68
1,35	91,15		-1,35	8,85
1,40	91,26		-1,40	8,08
1,45	92,65		-1,45	7,35
1,50	93,32		-1,50	6,68
1,55	93,94		-1,55	6,06
1,60	94,52		-1,60	5,48
1,65	95,05		-1,65	4,95
1,70	95,54		-1,70	4,46
1,75	95,99		-1,75	4,01
1,80	96,41		-1,80	3,59
1,85	96,78		-1,85	3,22
1,90	97,13		-1,90	2,87
1,95	97,44		-1,95	2,56
2,00	97,72		-2,00	2,28
2,05	97,98		-2,05	2,02
2,10	98,21		-2,10	1,79
2,15	98,43		-2,15	1,57
2,20	98,61		-2,20	1,39
2,25	98,78		-2,25	1,22
2,30	98,93		-2,30	1,07
2,35	99,06		-2,35	0,94
2,40	99,18		-2,40	0,82
2,45	99,29		-2,45	0,71
2,50	99,38		-2,50	0,62
2,55	99,46		-2,55	0,54
2,60	99,53		-2,60	0,47
2,65	99,60		-2,65	0,40
2,70	99,65		-2,70	0,35
2,75	99,70		-2,75	0,30
2,80	99,74		-2,80	0,26
2,85	99,78		-2,85	0,22
2,90	99,81		-2,90	0,19
2,95	99,84		-2,95	0,16
3,00	99,86		-3,00	0,14
3,05	99,88		-3,05	0,16
3,10	99,90		-3,10	0,10
3,15	99,92		-3,15	0,08
3,20	99,93		-3,20	0,07
3,25	99,94		-3,25	0,06
3,30	99,95		-3,30	0,05
3,35	99,96		-3,35	0,04
3,40	99,97		-3,40	0,03
3,45	99,97		--3,45	0,03
3,50	99,98		--3,50	0,02
3,55	99,98		--3,55	0,02
3,60	99,98		--3,60	0,02
3,65	99,99		--3,65	0,01
3,70	99,99		--3,70	0,01
3,75	99,99		--3,75	0,01

Table 3A.1b

Standardized normal distribution		
T	G(y)%	W_T
1000	0,1	-3,09
500	0,2	-2,88
200	0,5	-2,58
100	1,0	-2,33
50	2,0	-2,05
20	5,0	-1,64
10	10,0	-1,28
5	20,0	-0,84
2	50,0	0,00
5	80,0	0,84
10	90,0	1,28
20	95,0	1,64
50	98,0	2,05
100	99,0	2,33
200	99,5	2,58
500	99,8	2,88
1000	99,9	3,09
5000	99,98	3,55
10000	99,99	3,72

Table 3A.2: Parameters of the standardized general extreme value distribution

Standardized general extreme value distribution			
g	**k**	**E(y)**	**var(y)**
-2,000	1,406	-1,247	3,204
-1,900	1,321	-1,182	2,505
-1,800	1,240	-1,127	1,984
-1,700	1,163	-1,080	1,590
-1,600	1,089	-1,041	1,287
-1,500	1,018	-1,008	1,052
-1,400	0,950	-0,980	0,868
-1,300	0,885	-0,957	0,721
-1,200	0,824	-0,938	0,602
-1,100	0,765	-0,922	0,507
-1,000	0,708	-0,910	0,428
-0,900	0,655	-0,901	0,362
-0,800	0,604	-0,894	0,307
-0,700	0,555	-0,889	0,261
-0,600	0,509	-0,887	0,222
-0,500	0,465	-0,886	0,188
-0,400	0,424	-0,886	0,159
-0,300	0,384	-0,888	0,134
-0,200	0,346	-0,892	0,112
-0,100	0,311	-0,896	0,094
0,000	0,277	-0,901	0,077
0,100	0,245	-0,907	0,063
0,200	0,215	-0,914	0,050
0,300	0,187	-0,922	0,039
0,400	0,160	-0,930	0,030
0,500	0,134	-0,938	0,022
0,600	0,110	-0,947	0,016
0,700	0,088	-0,956	0,010
0,800	0,067	-0,966	0,006
0,900	0,047	-0,975	0,003
1,000	0,028	-0,985	0,001
1,100	0,010	-0,994	0,000
1,200	-0,006	1,004	0,000
1,300	-0,022	1,013	0,001
1,400	-0,037	1,023	0,002
1,500	-0,050	1,032	0,005
1,600	-0,063	1,041	0,008
1,700	-0,075	1,049	0,011
1,800	-0,086	1,058	0,016
1,900	-0,097	1,066	0,021
2,000	-0,107	1,074	0,026
2,100	-0,116	1,082	0,032
2,200	-0,125	1,089	0,038
2,300	-0,133	1,097	0,044
2,400	-0,140	1,104	0,051
2,500	-0,148	1,110	0,058
2,600	-0,154	1,116	0,065
2,700	-0,160	1,123	0,072
2,800	-0,166	1,128	0,080
2,900	-0,172	1,134	0,087
3,000	-0,177	1,139	0,094
3,100	-0,182	1,145	0,102
3,200	-0,187	1,150	0,110
3,300	-0,191	1,154	0,117
3,400	-0,195	1,159	0,125
3,500	-0,199	1,163	0,132
3,600	-0,203	1,168	0,140
3,700	-0,207	1,172	0,148
3,800	-0,210	1,176	0,155
3,900	-0,213	1,180	0,163
4,000	-0,217	1,183	0,170
4,100	-0,220	1,187	0,178
4,200	-0,223	1,191	0,186
4,300	-0,225	1,194	0,193
4,400	-0,228	1,197	0,201
4,500	-0,231	1,201	0,208
4,600	-0,233	1,204	0,215
4,700	-0,236	1,207	0,223
4,800	-0,238	1,210	0,230
4,900	-0,240	1,213	0,237
5,000	-0,242	1,215	0,244

Table 3A.3: Values of the standardized variate W_T for the normal, exponential and Pearson Type III distributions

Return period (years)	Non-exceedance probability	Normal distribution			Exponential distribution	Pearson Type III distribution (Values of W_T)										
		W_T	Confidence limits W_α			g										
			75%	95%		-1,0	-0,8	-0,6	-0,4	-0,2	0,0	0,2	0,4	0,6	0,8	1,0
2	0,50	0,00	$1,63/\sqrt{2N}$	$2,77/\sqrt{2N}$	0,69	0,16	0,13	0,10	0,07	0,03	0,00	-0,03	-0,70	-0,10	-0,13	-0,16
5	0,80	0,84	$1,89/\sqrt{2N}$	$3,23/\sqrt{2N}$	1,61	0,85	0,87	0,86	0,86	0,85	0,84	0,83	0,82	0,80	0,78	0,76
10	0,90	1,28	$2,20/\sqrt{2N}$	$3,74/\sqrt{2N}$	2,30	1,13	1,17	1,20	1,23	1,26	1,28	1,30	1,32	1,33	1,34	1,34
20	0,95	1,64	$2,49/\sqrt{2N}$	$4,25/\sqrt{2N}$	3,00	1,32	1,39	1,46	1,52	1,59	1,64	1,70	1,75	1,80	1,84	1,88
50	0,98	2,05	$2,87/\sqrt{2N}$	$4,89/\sqrt{2N}$	3,91	1,49	1,61	1,72	1,83	1,94	2,05	2,16	2,26	2,36	2,45	2,54
100	0,99	2,33	$3,13/\sqrt{2N}$	$5,34/\sqrt{2N}$	4,61	1,59	1,73	1,88	2,03	2,18	2,33	2,47	2,62	2,76	2,89	3,02
200	0,995	2,58	$3,38/\sqrt{2N}$	$5,76/\sqrt{2N}$	5,30	1,66	1,84	2,02	2,20	2,39	2,58	2,76	2,95	3,13	3,31	3,49
500	0,998	2,88	$3,69/\sqrt{2N}$	$6,27/\sqrt{2N}$	6,21						2,88					
1000	0,999	3,09	$3,91/\sqrt{2N}$	$6,66/\sqrt{2N}$	6,91	1,79	2,02	2,27	2,53	2,81	3,09	3,38	3,67	3,96	4,24	4,53
10000	0,9999	3,72	$4,58/\sqrt{2N}$	$7,80/\sqrt{2N}$	9,21	1,88	2,18	2,53	2,90	3,30	3,72	4,15	4,60	5,05	5,50	5,96

Table 3A.4: Values of the standardized variate W_T for the general extreme value distribution

Return period (years)	General extreme value (Values of W_T)																									
	g																									
	-1,0	-0,8	-0,6	-0,4	-0,2	0,0	0,2	0,4	0,6	0,8	1,0	1,14	1,2	1,4	1,6	1,8	2,0	2,5	3,0	3,5	4,0	4,5	5,0	5,5	6,0
	EV3											EV1	EV2												
2		0,33	0,34	0,34	0,34	0,35	0,35	0,36	0,36	0,36	0,37	0,37	0,37	0,37	0,37	0,37	0,37	0,38	0,38	0,38	0,38	0,38	0,38	0,38	0,38
5		1,01	1,06	1,12	1,17	1,23	1,28	1,33	1,38	1,43	1,47	1,50	1,51	1,55	1,58	1,60	1,63	1,68	1,72	1,75	1,77	1,79	1,80	1,82	1,83
10		1,28	1,37	1,46	1,57	1,67	1,78	1,89	2,00	2,10	2,19	2,25	2,28	2,35	2,43	2,49	2,55	2,67	2,76	2,84	2,90	2,94	2,98	3,01	3,04
20		1,44	1,57	1,71	1,86	2,02	2,19	2,37	2,54	2,71	2,86	2,97	3,01	3,15	3,28	3,40	3,50	3,73	3,91	4,05	4,16	4,25	4,33	4,39	4,45
50		1,58	1,74	1,93	2,15	2,38	2,64	2,90	3,18	3,45	3,72	3,90	3,97	4,22	4,45	4,66	4,86	5,28	5,62	5,89	6,12	6,30	6,46	6,59	6,70
100		1,64	1,83	2,05	2,31	2,60	2,92	3,26	3,62	3,99	4,35	4,60	4,71	5,05	5,38	5,68	5,97	6,59	7,10	7,52	7,86	8,15	8,39	8,59	8,77
200		1,68	1,89	2,14	2,44	2,77	3,16	3,58	4,02	4,49	4,97	5,30	5,44	5,90	6,34	6,76	7,16	8,04	8,77	9,38	9,89	10,31	10,67	10,97	11,24
500		1,72	1,95	2,23	2,56	2,96	3,42	3,94	4,51	5,13	5,76	6,21	6,41	7,05	7,68	8,29	8,87	10,19	11,32	12,26	13,06	13,74	14,32	14,81	15,24
1000		1,74	1,98	2,27	2,64	3,07	3,59	4,19	4,86	5,58	6,35	6,91	7,15	7,95	8,75	9,53	10,29	12,02	13,53	14,82	15,92	16,86	17,66	18,36	18,96
10000		1,76	2,02	2,36	2,78	3,32	4,00	4,83	5,81	6,96	8,24	9,21	9,64	11,13	12,68	14,25	15,82	19,65	23,19	26,34	29,13	31,58	33,73	35,63	37,31

WEB2P

APPENDIX 3B
STANDARD DESIGN FLOOD METHOD

Table 3B.1: Information required for the calculation of the SDF

Basin	SAWS station number	SAWS site	M (mm)	R (days)	C_2 (%)	C_{100} (%)	MAP (mm)	MAE (mm)
1	546 204	Struan	56	30	10	40	550	1800
2	675 125	Autoriteit	62	44	5	30	450	1900
3	760 324	Siloam	64	28	5	40	470	1700
4	553 351	Waterval	58	20	10	50	630	1600
5	680 059	Leydsdorp	78	10	15	70	620	1700
6	369 030	Siloam	51	54	15	60	670	1500
7	328 726	Olivine	49	39	15	60	510	1700
8	322 071	Danielskuil	47	39	5	20	380	2100
9	258 452	Jacobsdal	43	47	15	60	380	1800
10	233 049	Wonderboom	54	55	10	50	560	1600
11	236 521	Mashai	39	66	40	80	430	1400
12	143 258	Scheurfontein	39	52	5	30	290	2100
13	284 361	Wilgenhoutsdrif	40	55	5	15	70	2600
14	110 385	Middelpos	25	13	10	30	140	2400
15	157 874	Garies	22	11	5	20	130	2100
16	160 807	Loeriesfontein	28	11	10	40	210	1900
17	84 558	Elandspoort	45	1	40	80	500	1500
18	22 113	La Motte	59	4	30	60	810	1400
19	69 483	Letjiesbos	34	16	10	35	160	2200
20	34 762	Uitenhage	53	12	15	60	480	1600
21	76 884	Albertvale	45	23	10	35	460	1700
22	80 569	Umzoniana	84	26	15	60	820	1200
23	180 439	Insizwa	60	45	10	80	890	1200
24	240 269	Newlands	76	15	15	80	910	1200
25	239 138	Whitson	55	9	10	80	830	1200
26	336 283	Nqutu	61	17	15	50	760	1500
27	339 415	Hill Farm	85	17	30	80	890	1400
28	483 193	Maliba Ranch	75	54	15	60	740	1400
29	556 088	Mayfern	66	11	15	50	740	1600

Table 3B.2: Daily rainfall from TR102

Basin number	Station number	Name	Latitude	Longitude	Years of record	Mean annual rainfall (mm)	Duration (days)	Minimum annual recorded	Maximum annual recorded	Maxima for return periods (years for the duration) (mm)						
										2	5	10	20	50	100	200
1	546204	STRUAN	25°24'	26°07'	48	549	1	23	111	56	80	99	119	150	177	206
							2	32	155	71	105	132	161	205	243	286
							3	42	216	80	117	146	177	224	263	308
							7	42	284	102	154	196	242	310	369	435
2	675125	AUTORITEIT	23°35'	28°05'	45	452	1	24	178	62	93	117	145	187	223	264
							2	32	216	74	111	140	173	222	265	313
							3	32	254	80	122	156	193	250	300	355
							7	37	254	94	144	183	225	289	344	405
3	766324	SILOAM	22°54'	30°11'	46	472	1	25	188	64	95	119	146	187	222	262
							2	32	268	76	112	142	174	221	263	309
							3	33	329	84	129	165	205	266	319	378
							7	36	381	103	165	215	271	356	432	517
4	553351	WATERVAL	25°21'	29°42'	51	627	1	36	100	58	76	89	102	122	138	155
							2	40	140	69	90	106	123	146	165	185
							3	45	140	76	99	115	132	156	175	195
							7	53	184	98	131	154	178	211	238	266
5	680059	LEYDSDORP	23°59'	30°22'	45	625	1	23	195	78	116	146	181	233	279	331
							2	23	330	99	156	203	257	341	416	503
							3	27	357	105	165	215	271	358	435	524
							7	27	377	135	225	301	389	528	653	798
6	369030	SYLVAN	28°00'	29°01'	44	668	1	37	92	51	65	74	84	97	108	120
							2	42	115	64	85	99	113	133	149	166
							3	42	134	74	98	116	134	160	181	204
							7	50	145	92	121	142	164	193	217	242
7	328726	OLIVINE	28°06'	26°55'	45	507	1	22	103	49	68	82	96	118	137	157
							2	25	110	62	87	107	128	158	184	213
							3	34	119	68	94	115	136	167	193	221
							7	39	150	84	118	144	172	211	243	279
8	322071	DANIELSKUIL	28°11'	23°33'	61	377	1	11	116	47	69	86	104	132	156	183
							2	21	156	60	91	116	144	187	224	267
							3	21	186	65	100	128	160	208	250	297
							7	22	245	79	126	164	207	272	329	393
9	258458	JACOBSDAL	29°08'	24°46'	86	376	1	16	99	43	61	75	91	114	133	155
							2	20	141	54	78	98	119	151	179	210
							3	20	181	59	87	109	134	171	203	238
							7	27	239	70	104	131	160	203	240	280

Table 3B.2: Daily rainfall from TR102 (continued)

Basin number	Station number	Name	Latitude	Longitude	Years of record	Mean annual rainfall (mm)	Duration (days)	Minimum annual recorded	Maximum annual recorded	Maxima for return periods (years for the duration) (mm)						
										2	5	10	20	50	100	200
10	233049	WONDERBOOM	29°49'	27°02'	66	560	1	23	127	54	73	88	103	124	143	162
							2	37	148	66	88	105	122	146	166	188
							3	37	169	75	102	123	144	175	200	227
							7	37	238	97	140	172	206	256	298	343
11	236521	MASHAI	29°41'	28°48'	45	429	1	9	160	39	53	64	75	92	106	122
							2	15	160	47	65	79	93	113	130	149
							3	15	160	53	73	88	104	127	147	167
							7	15	160	69	97	118	141	173	199	228
12	143258	SCHEURFONTEIN	31°18'	24°09'	64	288	1	16	84	39	54	66	79	97	112	129
							2	17	116	47	67	82	99	123	143	165
							3	17	130	51	75	93	113	143	168	195
							7	26	179	62	92	116	141	179	218	245
13	284361	WILGENHOUTSDRIFT	28°31'	21°43'	40	270	1	9	117	40	59	73	90	115	135	159
							2	10	117	49	75	97	120	157	188	224
							3	10	132	52	82	106	133	175	212	250
							7	11	218	62	99	129	163	213	257	301
14	110385	MIDDELPOS	31°55'	20°13'	65	143	1	9	71	25	38	50	62	82	99	118
							2	9	71	30	49	65	84	113	139	170
							3	9	71	31	51	68	88	119	147	179
							7	10	87	34	57	76	98	131	161	196
15	157874	GARIES	30°34'	18°00'	63	130	1	5	58	22	32	39	46	57	66	76
							2	5	61	26	37	46	55	69	80	93
							3	5	61	27	40	50	61	78	92	107
							7	9	69	30	45	57	70	88	104	122
16	160807	LOERIESFONTEIN	30°57'	19°27'	47	212	1	13	66	28	39	48	57	70	81	93
							2	16	93	35	48	58	69	84	97	110
							3	16	93	37	51	63	74	91	105	120
							7	16	106	43	60	73	85	104	118	134
17	84558	ELANDSFONTEIN	32°18'	18°49'	51	498	1	26	101	45	59	69	80	96	108	122
							2	29	137	60	83	101	119	146	169	193
							3	31	149	68	96	118	141	174	202	234
							7	44	179	86	126	157	190	240	281	328
18	22113	LA MOTTE	33°53'	19°04'	58	812	1	35	180	59	77	91	105	125	142	160
							2	48	230	82	111	134	158	193	223	254
							3	50	277	93	129	155	184	225	260	297
							7	64	418	126	183	227	275	345	405	471
19	69483	LETJIESBOS	32°33'	22°17'	62	165	1	5	183	34	55	72	92	124	152	185
							2	8	200	38	64	87	112	153	190	233
							3	8	203	40	68	93	121	166	206	254
							7	8	225	45	79	110	145	202	254	315

Table 3B.2: Daily rainfall from TR102 (continued)

Basin number	Station number	Name	Latitude	Longitude	Years of record	Mean annual rainfall (mm)	Duration (days)	Minimum annual recorded	Maximum annual recorded	Maxima for return periods (years for the duration) (mm)						
										2	5	10	20	50	100	200
20	34762	UITENHAGE	33°42'	25°26'	61	475	1	22	191	53	80	103	129	170	206	248
							2	26	204	65	102	132	167	221	269	325
							3	26	204	70	110	144	182	242	296	358
							7	26	228	82	131	171	217	287	350	422
21	76884	ALBERTVALE	32°44'	26°00'	73	457	1	22	163	45	64	80	97	123	145	170
							2	23	199	56	82	102	126	161	191	225
							3	27	228	60	86	107	130	165	194	227
							7	30	256	71	104	130	158	199	234	273
22	80569	UNZONIANA	32°59'	27°49'	57	821	1	24	386	84	134	178	229	312	389	480
							2	29	638	109	182	248	326	455	576	721
							3	36	733	121	205	280	371	521	661	830
							7	39	866	140	227	302	385	523	661	830
23	180439	INSIZWA	30°49'	29°15'	63	890	1	28	124	60	80	95	111	135	154	175
							2	39	173	76	102	121	140	169	193	218
							3	44	173	85	113	134	156	187	212	240
							7	60	205	108	141	165	189	222	249	277
24	240269	NEWLANDS	29°59'	30°39'	58	912	1	34	242	76	114	145	181	235	284	340
							2	45	288	95	142	181	224	290	348	415
							3	51	314	105	154	192	235	298	354	415
							7	64	315	126	179	219	262	325	378	436
25	239138	WHITSON	29°48'	30°05'	42	829	1	36	179	55	71	83	95	113	127	143
							2	45	230	71	94	111	129	155	176	199
							3	50	245	80	108	129	150	181	207	235
							7	60	257	104	138	162	187	221	250	279
26	336283	NQUTU	28°13'	30°40'	52	760	1	28	124	61	84	102	121	150	175	202
							2	35	133	76	105	128	152	187	217	250
							3	35	148	84	117	141	168	205	237	272
							7	44	198	108	151	182	215	263	302	345
27	339415	HILL FARM	28°25'	32°14'	56	893	1	26	376	85	130	167	210	278	339	410
							2	35	395	107	167	218	277	369	453	550
							3	36	397	119	188	246	314	420	517	628
							7	39	407	143	223	290	364	480	581	698
28	483193	MILIBA RANCH	26°13'	31°37'	40	740	1	34	237	75	104	127	151	187	218	252
							2	40	281	89	126	154	185	230	268	310
							3	40	281	99	142	175	212	265	311	361
							7	58	353	122	171	209	248	305	353	405
29	556088	MAYFERN	25°28'	31°03'	46	737	1	31	333	66	93	113	135	168	196	227
							2	39	352	78	108	130	154	189	218	250
							3	44	360	89	125	153	183	227	265	306
							7	45	416	113	159	194	232	286	331	380

WEB2P

APPENDIX 3C
STANDARD FLOOD CALCULATION FORMS

WEB2P

RATIONAL METHOD

Description of catchment			
River detail			
Calculated by		**Date**	

Physical characteristics					
Size of catchment (A)		km²	Rainfall region		
Longest watercourse (L)		km	**Area distribution factors**		
Average slope (S_{av})		m/m	Rural (α)	Urban (β)	Lakes (γ)
Dolomite area ($D_{\%}$)		%			
Mean annual rainfall (MAR)⓪#		mm			

Rural①				**Urban②**			
Surface slope	**%**	**Factor**	**C_s**	**Description**	**%**	**Factor**	**C_2**
Vleis and pans				**Lawns**			
Flat areas				Sandy, flat (<2%)			
Hilly				Sandy, steep (>7%)			
Steep areas				Heavy soil, flat (<2%)			
Total	100	-		Heavy soil, steep (>7%)			
Permeability	**%**	**Factor**	**C_p**	**Residential areas**			
Very permeable				Houses			
Permeable				Flats			
Semi-permeable				**Industry**			
Impermeable				Light industry			
Total	100	-		Heavy industry			
Vegetation	**%**	**Factor**	**C_v**	**Business**			
Thick bush and plantation				City centre			
Light bush and farm-lands				Suburban			
Grasslands				Streets			
No vegetation				Maximum flood			
Total	100	-		Total (C_2)	100	-	

Time of concentration (T_C)		Notes:
Overland flow③	Defined watercourse	
$T_C = 0{,}604\left(\frac{rL}{\sqrt{S_{av}}}\right)^{0{,}467}$	$T_C = \left(\frac{0{,}87L^2}{1000S_{av}}\right)^{0{,}385}$	
hours	hours	

Run-off coefficient							
Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**Max**
Run-off coefficient, C_1 ($C_1 = C_S + C_P + C_V$)							
Adjusted for dolomitic areas, C_{1D} ($= C_1(1 - D_{\%}) + C_1D_{\%}(\sum(D_{factor} \times C_{S\%}))$)④							
Adjustment factor for initial saturation, F_t⑤							
Adjusted run-off coefficient, C_{1T} ($= C_{1D} \times F_t$)							
Combined run-off coefficient C_T ($= \alpha C_{1T} + \beta C_2 + \gamma C_3$)							

Rainfall							
Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**Max**
Point rainfall (mm), P_T⑥							
Point intensity (mm/hour), P_{iT} ($= P_T/T_C$)							
Area reduction factor (%), ARF_T⑦							
Average intensity (mm/hour), I_T ($= P_{iT} \times ARF_T$)							
Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**Max**
Peak flow (m³/s), $Q_T = \frac{C_T I_T A}{3{,}6}$							

Note: # Reference to the appropriate figures and tables is made in the legend table of this method. **R - Page 1/2**

WEB2P

LEGEND TABLE Rational method	
ID	Reference
⓪	Figure 3.7
①	Table 3C.1
②	Table 3C.2
③	Table 3C.3
④	Table 3C.4
⑤	Table 3C.5
⑥	Figure 3.6
⑦	Figure 3.20 or 3.21

Table 3C.1

Rural (C_1)

Component	Classification	Mean annual rainfall (mm)		
		600	600 - 900	900
Surface slope (C_S)	Vleis and pans (<3%)	0,01	0,03	0,05
	Flat areas (3 to 10%)	0,06	0,08	0,11
	Hilly (10 to 30%)	0,12	0,16	0,20
	Steep areas (>30%)	0,22	0,26	0,30
Permeability (C_P)	Very permeable	0,03	0,04	0,05
	Permeable	0,06	0,08	0,10
	Semi-permeable	0,12	0,16	0,20
	Impermeable	0,21	0,26	0,30
Vegetation (C_V)	Thick bush and plantation	0,03	0,04	0,05
	Light bush and farm-lands	0,07	0,11	0,15
	Grasslands	0,17	0,21	0,25
	No vegetation	0,26	0,28	0,30

Table 3C.2

Urban (C_2)

Use	Factor
Lawns	
Sandy, flat (< 2%)	0,05 - 0,10
Sandy, steep (>7%)	0,15 - 0,20
Heavy soil, flat (< 2%)	0,13 - 0,17
Heavy soil, steep (>7%)	0,25 - 0,35
Residential areas	
Houses	0,30 - 0,50
Flats	0,50 - 0,70
Industry	
Light industry	0,50 - 0,80
Heavy industry	0,60 - 0,90
Business	
City centre	0,70 - 0,95
Suburban	0,50 - 0,70
Streets	0,70 - 0,95
Maximum flood	1,00

Table 3C.3

Surface description	Recommended value of r
Paved areas	0,02
Clean compacted soil, no stones	0,1
Sparse grass over fairly rough surface	0,3
Medium grass cover	0,4
Thick grass cover	0,8

Table 3C.4

Adjustment factor to C_s

Surface slope classification	D_{factor}
Steep areas (slopes >30%)	0,50
Hilly (10 to 30%)	0,35
Flat areas (3 to 10%)	0,20
Vleis and pans (slopes <3%)	0,10

Table 3C.5

Return period (years)	2	5	10	20	50	100
Adjustment factor (F_t) for steep and impermeable catchments	0,75	0,80	0,85	0,90	0,95	1,00
Adjustment factor (F_t) for flat and permeable catchments	0,50	0,55	0,60	0,67	0,83	1,00

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WEB2P

ALTERNATIVE RATIONAL METHOD

Description of catchment			
River detail			
Calculated by		**Date**	

Physical characteristics

Size of catchment (A)		km²	Days of thunder per year (R)②		days/year
Longest watercourse (L)		km	Weather Service station⑩		
Average slope (S_{av})		m/m	Weather Service number⑩		
Dolomite area ($D_{\%}$)		%	**Area distribution factors**		
Mean annual rainfall (MAR)⓪#		mm	Rural (α)	Urban (β)	Lakes (γ)
2-year return period rainfall (M) ①		mm			

Rural③				**Urban④**			
Surface slope	**%**	**Factor**	**C_s**	**Description**	**%**	**Factor**	**C_2**
Vleis and pans				**Lawns**			
Flat areas				Sandy, flat (<2%)			
Hilly				Sandy, steep (>7%)			
Steep areas				Heavy soil, flat (<2%)			
Total	100	-		Heavy soil, steep (>7%)			
Permeability	**%**	**Factor**	**C_p**	**Residential areas**			
Very permeable				Houses			
Permeable				Flats			
Semi-permeable				**Industry**			
Impermeable				Light industry			
Total	100	-		Heavy industry			
Vegetation	**%**	**Factor**	**C_v**	**Business**			
Thick bush and plantation				City centre			
Light bush and farm-lands				Suburban			
Grasslands				Streets			
No vegetation				Maximum flood			
Total	100	-		Total (C_2)	100	-	

Time of concentration (T_C)		Notes:
Overland flow⑤	Defined watercourse	
$T_C = 0{,}604\left(\dfrac{rL}{\sqrt{S_{av}}}\right)^{0{,}467}$	$T_C = \left(\dfrac{0{,}87L^2}{1000S_{av}}\right)^{0{,}385}$	
hours	hours	

Run-off coefficient

Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**Max**
Run-off coefficient, C_1 ($C_1 = C_S + C_P + C_V$)							
Adjusted for dolomitic areas, C_{1D} ($= C_1(1 - D_{\%})+C_1D_{\%}(\sum(D_{factor} \times C_{S\%}))$)⑥							
Adjustment factor for initial saturation, F_t⑦							
Adjusted run-off coefficient, C_{1T} ($= C_{1D} \times F_t$)							
Combined run-off coefficient C_T ($= \alpha C_{1T} + \beta C_2 + \gamma C_3$)							

Rainfall

Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**Max**
Point rainfall (mm), P_T⑧							
Point intensity (mm/hour), P_{iT} ($= P_T/T_C$)							
Area reduction factor (%), ARF_T⑨							
Average intensity (mm/hour), I_T ($= P_{iT} \times ARF_T$)							
Peak flow (m³/s) $Q_T = \dfrac{C_T I_T A}{3{,}6}$							

Note: # Reference to the appropriate figures and tables is made in the legend table of this method. **AR - Page 1/2**

ALTERNATIVE RATIONAL METHOD

n-day rainfall data							
Weather Service station							
Weather Service station number							
Mean annual precipitation (MAP)				mm			
Coordinates			&				
Duration (days)	**Return period (years), T**						
	2	**5**	**10**	**20**	**50**	**100**	**200**
1 day							
2 days							
3 days							
7 days							

Table 3C.6

Selection criteria	Calculation method
$T_C < 6$ hours	Modified Hershfield equation $P_{t,T} = 1{,}13(0{,}41 + 0{,}64\ln T)(-0{,}11 + 0{,}27\ln t)(0{,}79M^{0{,}69}R^{0{,}20})$
6 hours $\leq T_C <$ 24 hours	Linear interpolation between calculated modified Hershfield equation point rainfall and 1-day point rainfall from TR102
$T_C \geq$ 24 hours	Linear interpolation between n-day point rainfall values from TR102

LEGEND TABLE Alternative Rational method	
ID	**Reference**
⓪	Figure 3.7
①	TR102 (3.15) or other
②	Figure 3.12 (3.28)
③	Table 3C.1
④	Table 3C.2
⑤	Table 3C.3
⑥	Table 3C.4
⑦	Table 3C.5
⑧	Table 3C.6
⑨	Figure 3.14
⑩	Figure 3.13,TR102(3.15) or other

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UNIT HYDROGRAPH METHOD

Description of catchment				
River detail				
Calculated by			**Date**	

Physical characteristics						
Size of catchment (A)		km²	Veld type①			
Longest watercourse (L)		km	Lag (T_L)②			
Average slope (S_{av})		m/m	Coefficient (K_U) ③			
Length to catchment centroid (L_C)		km	Peak discharge of unit hydrograph (Q_P)	$Q_p = \frac{K_U A}{T_L}$		m³/s
Mean annual rainfall (MAR)⓪#		mm				
Catchment index $I_C = \frac{L L_C}{\sqrt{S_{av}}}$						

Return period (years), T				**T =**			
Storm duration (hours), T_{SD}							
Point rainfall (mm), P_T④							
Point intensity (mm/hour), P_{iT} (= P_T/T_{SD})							
Area reduction factor, ARF_{iT}⑤							
Average rainfall (mm), P_{AvgiT} (= P_T x ARF_{iT})							
Flood run-off factor (%), f_{iT}⑥							
Effective rain (mm), he_{iT} (= f_{iT} x P_{AvgiT})							

Time (hours), t	$\frac{t}{T_L}$ ⑦	**T_{SD} = 1 hour**		**T_{SD} =**		**hours**	**T_{SD} =**		**hours**
		$\frac{Q}{Q_p}$ ⑦	S-curve, $S_1$⑧	Lagged S-curve, S_{2T}	$S_1 - S_{2T}$	$\frac{S_1 - S_{2T}}{T_{SD}}$	Lagged S-curve, S_{2T}	$S_1 - S_{2T}$	$\frac{S_1 - S_{2T}}{T_{SD}}$

Return period (years)				**T =**			
Storm duration (hours), T_{SD}							
Unit Hydrograph peak (m³/s), Q_{PiT}⑨							
Peak discharge (m³/s), Q_{iT} (= Q_{PiT} x he_{iT})							
Adjusted peak for $Q_{PiT}/Q_p < 1$ (m³/s)							

LEGEND TABLE
Unit Hydrograph method

ID	Reference	ID	Reference
⓪	Figure 3.7	⑤	Figure 3.20 or 3.21
①	Figure 3.16	⑥	Figure 3.22
②	Figure 3.17	⑦	Table 3.12
③	Table 3.11	⑧	Paragraph 3.5.3.5
④	Figure 3.6	⑨	$Q_{PiT} = Q_p$ x $[(S_1 - S_{2T})/T_{SD}]_{max}$

Note: # Reference to the appropriate figures and tables is made in the legend table of this method.

STANDARD DESIGN FLOOD METHOD

Description of catchment			
River detail			
Calculated by		**Date**	

Physical characteristics						
Size of catchment (A)		km^2	Time of concentration (T_C)	$T_C = \left(\frac{0{,}87L^2}{1000S_{av}}\right)^{0{,}385}$		hours
Longest watercourse (L)		km				
Average slope (S_{av})		m/m				
SDF basin⓪#			Time of concentration, t (= $60T_C$)			minutes
2-year return period rainfall (M)①		mm	Days of thunder per year (R)①			days/year

TR102 n-day rainfall data				
Weather Service station		Mean annual precipitation (MAP)		mm
Weather Service station no.		Coordinates	&	

Duration (days)	**Return period (years)**						
	2	**5**	**10**	**20**	**50**	**100**	**200**
1 day							
2 days							
3 days							
7 days							

Rainfall							
Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Point precipitation depth (mm), $P_{t,T}$②							
Area reduction factor (%), ARF $\left(= (90000 - 12800 \ln A + 9830 \ln t)^{0{,}4}\right)$							
Average intensity (mm/hour), I_T (= $P_{t,T}$ x ARF / T_C)							

Run-off coefficients				
Calibration factors①	C_2 (2-year return period) (%)		C_{100} (100-year return period) (%)	

Return period (years)	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Return period factors (Y_T)	0	0,84	1,28	1,64	2,05	2,33	2,58
Run-off coefficient (C_T), $C_T = \frac{C_2}{100} + \left(\frac{Y_T}{2{,}33}\right)\left(\frac{C_{100}}{100} - \frac{C_2}{100}\right)$							
Peak flow (m^3/s), $Q_T = \frac{C_T I_T A}{3{,}6}$							

LEGEND TABLE Standard Design Flood method			
ID	**Reference**	**ID**	**Reference**
⓪	Figure 3.25	②	Table 3C.7
①	Table 3B.1		

Table 3C.7

Criteria	**Calculation method**
T_C < 6 hours	Modified Hershfield equation $P_{t,T} = 1{,}13(0{,}41 + 0{,}64\ln T)(-0{,}11 + 0{,}27\ln t)(0{,}79M^{0{,}69}R^{0{,}20})$
6 hours ≤ T_C < 24 hours	Linear interpolation between calculated modified Hershfield equation point rainfall and 1-day point rainfall from TR102
T_C ≥ 24 hours	Linear interpolation between n-day point rainfall values from TR102

Note: # Reference to the appropriate figures and tables is made in the legend table of this method. **S - Page 1/1**

EMPIRICAL METHODS

Description of catchment					
River detail					
Calculated by			**Date**		
Physical characteristics					
Size of catchment (A)		km²	Veld type①		
Longest watercourse (L)		km	Catchment parameter (C) with regard to reaction time	$C = \frac{A\sqrt{S}}{L L_C}$	
Length to catchment centroid (L_C)		km			
Average slope (S_{av})		m/m	Kovács region②		
Mean annual rainfall (P)⓪#		mm			

Return period (years), T	**10**	**20**	**50**	**100**
Constant value for K_T ③				
Peak flow (m³/s), Q_T based on Midgley & Pitman $Q_T = 0{,}0377 K_T P A^{0,6} C^{0,2}$				
Peak flow (m³/s), Q_{RMF} based on Kovács④				
Return period (years), T	**50**	**100**	**200**	
Q_T/Q_{RMF} ratios⑤				
Peak flow (m³/s), based on Q_T/Q_{RMF} ratios				

LEGEND TABLE
Empirical methods

ID	Reference	ID	Reference	ID	Reference
⓪	Figure 3.7	②	Figure 3.28	④	Table 3C.9
①	Figure 3.16	③	Table 3C.8	⑤	Table 3D.1 or 3D.2

Table 3C.8
Constant values of K_T

Return period T in years	Veld type (Figure 3.16)										
	1	2		3	4 & 5A	5	6		7	8	9
		Winter	All year				Winter	All year			
10	0,17	0,42	0,83	0,29	0,59	0,59	0,33	0,67	0,67	0,42	0,50
20	0,23	0,52	1,04	0,40	0,68	0,80	0,45	0,91	0,91	0,57	0,68
50	0,32	0,68	1,36	0,55	0,95	1,11	0,63	1,26	1,26	0,79	0,95
100	0,40	0,80	1,60	0,70	1,20	1,40	0,80	1,60	1,60	1,00	1,20

Table 3C.9
RMF region classification in southern Africa

Kovács region	K *	Number of floods #	Transition zone		Flood zone	
			Area range (km²)	Q_{RMF} (m³/s)	Area range (km²)	Q_{RMF} (m³/s)
K1	2,8	6	1 – 500	$30A^{0,262}$	500 – 500 000	$1{,}74A^{0,72}$
K2	3,4	12	1 – 300	$50A^{0,265}$	300 – 500 000	$5{,}25A^{0,66}$
K3	4,0	26	1 – 300	$70A^{0,34}$	300 – 300 000	$15{,}9A^{0,60}$
K4	4,6	55	1 – 100	$100A^{0,38}$	100 – 100 000	$47{,}9A^{0,54}$
K5	5,0	155	1 – 100	$100A^{0,50}$	100 – 100 000	$100A^{0,50}$
K6	5,2	61	1 – 100	$100A^{0,56}$	100 – 30 000	$145A^{0,48}$
K7	5,4	34	1 – 100	$100A^{0,62}$	100 – 20 000	$209A^{0,46}$
K8	5,6	25	1 – 100	$100A^{0,68}$	100 – 10 000	$302A^{0,44}$

Notes:
* *RMF K value as used in Equation 3.25*
\# *Recorded flood data are reflected in the DWAF report TR105 – Maximum flood peak discharges in South Africa: An empirical approach*

Note: # Reference to the appropriate figures and tables is made in the legend table of this method. **E -Page 1/1**

APPENDIX 3D
Q_T/Q_{RMF} RATIOS FOR DIFFERENT CATCHMENT AREAS

Region	Return period (years)	K_T	Effective catchment area - A_e (km^2)									
			≤ 10*	30*	100	300	1 000	3 000	10 000	30 000	100 000	300 000
K8 (5,6)	50	5,06	0,537	0,508	0,474	0,503	0,537	0,570	0,607			
	100	5,25	0,668	0,645	0,617	0,640	0,668	0,695	0,724			
	200	5,41	0,803	0,788	0,769	0,784	0,803	0,821	0,838			
K7 (5,4)	50	4,70	0,447	0,416	0,380	0,411	0,447	0,482	0,523			
	100	4,89	0,556	0,525	0,492	0,523	0,556	0,588	0,623			
	200	5,04	0,661	0,635	0,607	0,633	0,661	0,687	0,716			
K6 (5,2)	50	4,50	0,447	0,416	0,380	0,411	0,447	0,482	0,526	0,566		
	100	4,69	0,556	0,528	0,494	0,524	0,556	0,588	0,626	0,660		
	200	4,86	0,676	0,650	0,624	0,650	0,676	0,701	0,733	0,758		
K5 (5 - except in SW Cape)	50	4,30	0,447	0,416	0,380	0,411	0,447	0,482	0,525	0,567	0,617	
	100	4,48	0,550	0,521	0,488	0,517	0,550	0,582	0,619	0,657	0,699	
	200	4,64	0,661	0,636	0,608	0,633	0,661	0,687	0,718	0,748	0,780	
K5 (5 - G, H in SW Cape)	50	4,45	0,531	0,502	0,468	0,497	0,531	0,564				
	100	4,63	0,654	0,629	0,600	0,625	0,654	0,680				
	200	4,78	0,777	0,758	0,738	0,757	0,777	0,795				
K4 (4,6)	50	3,84	0,416	0,385	0,350	0,381	0,416	0,453	0,496	0,541	0,591	
	100	4,04	0,524	0,495	0,462	0,491	0,524	0,558	0,597	0,636	0,679	
	200	4,20	0,629	0,603	0,576	0,602	0,629	0,660	0,692	0,724	0,758	
K3 (4)	50	3,26	0,426	0,426	0,426	0,390	0,426	0,463	0,506	0,548	0,602	0,651
	100	3,50	0,562	0,562	0,562	0,529	0,562	0,595	0,631	0,666	0,710	0,749
	200	3,68	0,692	0,692	0,692	0,665	0,692	0,718	0,745	0,771	0,804	0,831
K2 (3,4)**	50	2,40	0,317	0,317	0,317	0,281	0,317	0,353	0,398	0,444	0,500	0,560
	100	2,66	0,428	0,428	0,428	0,391	0,428	0,463	0,506	0,549	0,598	0,651
	200	2,91	0,570	0,570	0,570	0,536	0,570	0,600	0,638	0,672	0,710	0,753

Table 3D.1: Q_T/Q_{RMF} ratios for different catchment areas in South Africa, Lesotho and Swaziland [3.13]

Note: ** Estimated ratios*

*** Ratios of this region may also be used in region* ***K1*** *(2,8)*

Region	Return period (years)	K_T	Effective catchment area - A_e (km^2)									
			≤ 10*	30*	100	300	1 000	3 000	10 000	30 000	100 000	300 000
Namibia												
K5 (5)	50	4,50	0,562	0,534	0,501	0,529	0,562	0,594	0,631			
	100	4,70	0,708	0,686	0,661	0,683	0,708	0,732	0,759			
	200	4,85	0,841	0,828	0,813	0,826	0,841	0,855	0,871			
K4 (4,6)	50	4,14	0,589	0,561	0,530	0,558	0,589	0,620	0,654	0,690	0,727	
	100	4,34	0,741	0,721	0,699	0,719	0,741	0,763	0,787	0,811	0,835	
	200	4,48	0,871	0,860	0,848	0,860	0,871	0,883	0,895	0,909	0,920	
K3 (4)	50	3,50	0,562	0,562	0,562	0,529	0,562	0,595	0,631	0,666	0,710	
	100	3,66	0,676	0,676	0,676	0,648	0,676	0,703	0,731	0,759	0,793	
	200	3,77	0,767	0,767	0,767	0,746	0,767	0,788	0,809	0,829	0,856	
K2 (3,4)	50	2,88	0,550	0,550	0,550	0,517	0,550	0,585	0,619	0,656	0,696	
	100	3,01	0,639	0,639	0,639	0,610	0,639	0,669	0,698	0,729	0,762	
	200	3,13	0,733	0,733	0,733	0,711	0,733	0,758	0,779	0,803	0,828	
Zimbabwe												
K6 (5,2)**	50	4,65	0,531	0,502	0,468	0,497	0,531	0,564	0,603	0,640		
	100	4,86	0,676	0,652	0,625	0,649	0,676	0,702	0,731	0,759		
	200	5,03	0,822	0,807	0,791	0,806	0,822	0,838	0,855	0,871		

Table 3D.2: Q_T/Q_{RMF} ratios for different catchment areas in Namibia and Zimbabwe [(3.13)]

Note: ** Estimated ratios*

*** In region **K5** use the same ratios as those applicable to South Africa*

CHAPTER 4 - HYDRAULIC CALCULATIONS

SJ van Vuuren and A Rooseboom

4.1 GENERAL

Hydraulic calculations are performed in order to determine the values of variables that describe flow conditions, e.g. flow depths, flow velocities and pressures. Only three fundamental laws or principles are generally applied in hydraulic calculations:

- Conservation of mass (continuity principle);
- Conservation of energy; and
- Conservation of momentum.

Depending on what information is available and what answer is required, every hydraulic calculation involves the application of one or more of these fundamental laws or principles.

This chapter contains the basic equations together with empirical information, as well as guidance on the application of the equations in drainage analyses.

A distinction is made between open-channel flow and pipe flow. These aspects are respectively covered in paragraphs 4.2 and 4.3. In open-channel flow the pressure at the water surface remains atmospheric, whereas a conduit under pipe flow conditions basically flows "full". The pressure varies, and could be higher or lower than atmospheric pressure.

The laws of conservation of energy and momentum are both derived from Newton's second law. They can be transposed and simplified in numerous ways, and care should be taken to ensure that the format being used is indeed valid.

Virtually all calculations involve the law of conservation of mass. The law of conservation of energy is used where energy losses can be calculated, or are small enough to be disregarded. Similarly the law of conservation of momentum may be used to calculate forces that act upon bodies of fluid or to analyse flow conditions where all the forces that act upon a body of fluid may be quantified.

Table 4.1 provides the Road Map for this Chapter, and reflects the different problems and procedures that are covered in this chapter.

Table 4.1: Road Map 4 – Hydraulic calculations

<table>
<tr><th colspan="5">ROAD MAP 4</th></tr>
<tr><th colspan="2">Typical problems</th><th rowspan="2">Input information</th><th rowspan="2">Supporting software</th><th>Hand calculation</th></tr>
<tr><th>Topic</th><th>Paragraph</th><th>Example</th></tr>
<tr><td>Conservation of mass</td><td>4.2.3</td><td rowspan="3">Discharge and conduit configuration</td><td rowspan="7">The supporting utility software program is designed to address these problems</td><td rowspan="2">4.1 to 4.6</td></tr>
<tr><td>Conservation of energy</td><td>4.2.4</td></tr>
<tr><td>Controls</td><td>4.2.5</td><td>4.4 to 4.6</td></tr>
<tr><td>Friction losses</td><td>4.2.6</td><td>Conduit roughness</td><td rowspan="3">4.1 to 4.6</td></tr>
<tr><td>Transmission losses</td><td>4.2.7</td><td>Discharge and transition geometry</td></tr>
<tr><td>Conservation of momentum</td><td>4.2.9</td><td rowspan="2">Available energy or discharge and conduit configuration</td></tr>
<tr><td>Pipe flow</td><td>4.3</td><td>4.7</td></tr>
</table>

4.2 OPEN CHANNEL FLOW

4.2.1 Introduction

The general design problems in open channel flow are as follows:

- For a given discharge (from hydrological or other calculations), calculate the flow depths and/or velocities at given sections.
- For a given discharge, calculate the required conduit size to convey the flow.

In the reversed form of the problem, the flow depth is known and the discharge needs to be calculated (known as the Flow Measurement Problem).

Analyses of flow conditions should always begin at a section where the depth of flow and the velocity for the given discharge can be uniquely determined; i.e. at a control.

The following may be used as controls (see paragraph 4.2.5):

- A section where the flow characteristics have been calculated, based on a remote control section;
- A section where the flow changes from subcritical flow upstream ($Fr < 1$) to supercritical flow downstream ($Fr > 1$);

- A section where the flow is uniform, i.e. the average velocity does not change with distance;

- A position where the specific energy, E, for a given discharge is a minimum i.e. Fr = 1.

In all other cases where the depth of flow is forced away from the normal (uniform) depth, e.g. with damming upstream of a bridge, the depth of flow cannot be determined directly and needs to be determined from the flow depth at an appropriate control section. At a control there is usually some uncertainty about the appropriate calculation of the depth of flow. Under these conditions it is assumed that:

- the most conservative value (maximum or minimum flow depth) that could realistically be expected may be accepted, or, preferably,

- the maximum and minimum possible values of the depth at the control section may be calculated and then tested to determine whether the effect of the difference has been sufficiently attenuated (reduced) up to the area where the results are important.

Starting at a control section, calculations are performed upstream in the case of subcritical flow and downstream in the case of supercritical flow. Current state-of-the-art software for flow profile calculations is:

- able to identify the directions in which control is exercised;

- select the correct surface profiles; and

- perform the appropriate calculations for gradually varied flow.

If flow is supercritical upstream and subcritical downstream, calculations should proceed from both sides to find the position where the transition occurs (hydraulic jump). The hydraulic jump could be analysed by means of the momentum equation.

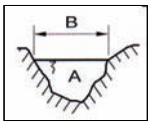
4.2.2 General definitions associated with open channel flow

The following definitions are important, and are listed here for case of reference.

Control: Section at which depth of flow for a given flow (discharge) can be uniquely determined.

Convergent flow: Downstream cross-sectional area < upstream cross-sectional area.

Critical flow: Open channel flow corresponding with the lowest possible energy level. Characterised by $\frac{Q^2B}{gA^3} = 1$ with Q = discharge, B = sectional width at the water surface and A = sectional area as depicted in the schematic.



Divergent flow: Downstream sectional area > upstream sectional area.

(Froude number)² = $Fr^2 = \frac{Q^2B}{gA^3}$ applicable to any shape of cross-section.

$Fr^2 = \frac{\bar{v}^2}{gy}$ for rectangular channel sections with $\bar{v}$ = average velocity and y = depth of flow.

Flow line: Line tangential to direction of flow at any point. No sustained flow is possible across a flow line.

Friction losses: Represent the application of energy to maintain flow without taking transition losses into account.

Hydrostatic pressure distribution: Pressure intensity increases linearly downwards from zero at a free surface, so that the pressure intensity (p) at a point at depth h equals γh; with $\gamma = \rho g$,
where:

ρ = density of the fluid (kg/m^3) and
g = gravitational acceleration (m/s^2)

This applies where flow lines are parallel or where flow velocities are low.

Laminar flow: Occurs at very low velocities. Rarely encountered in practice. Characterised by $\frac{vR}{\nu} < 500$; v = velocity, R = hydraulic radius; ν = kinematic viscosity with the value for water ≈ 1,14 x 10^{-6} m²/s (at 20 °C) for general design purposes.

Non-uniform flow: Flow conditions change with distance.

Open channel flow: Flow surface open to the atmosphere.

Pipe flow: Full flow conditions. (Pressures normally ≥ atmospheric pressure, but could also be < atmospheric pressure.)

Sheet flow: Flow in a broad stream of shallow depth.

Steady conditions: Flow conditions do not change with time.

Subcritical flow: Downstream control prevails and analyses should be performed in the upstream direction; Fr < 1.

Supercritical flow: Upstream control prevails and analyses should be performed in the downstream direction; Fr >1. Cannot be dammed without first changing to subcritical flow.

Transition losses: Energy losses associated with a change of velocity in magnitude or direction.

Turbulent flow: Most common type of flow; $\frac{\overline{v}R}{\nu} > 5000$.

Uniform flow: Conditions do not change with distance.

Unsteady flow: Conditions change with time.

In the following paragraphs the continuity of mass, energy and momentum will be discussed.

4.2.3 Principle of continuity of mass (Conservation of mass)

The application of the principle of continuity is usually the least complicated and the most accurate method of calculation and, therefore, forms the cornerstone of most hydraulic calculations.

According to the continuity equation the difference between the sum of the inflows and the sum of the outflows that enter and leave a defined space (control volume) should be equal to the rate of change in the volume of fluid contained within the space.

The complete continuity equation for cases where the flow may be regarded as incompressible (virtually all cases of road drainage) reads:

$$\sum \text{ Inflows} - \sum \text{ Outflows} = \text{storage accumulation per unit time} = \frac{dV}{dt} \qquad \text{… (4.1)}$$

where:

dV = change of volume (m^3) over a time step, dt (s).

In road drainage the complete equation 4.1 is usually only applied where flood attenuation, e.g. due to temporary storage upstream of culverts, is taken into consideration. In most other cases where no temporal storage occurs the following relationship is valid:

$$\sum \text{Inflows} = \sum \text{Outflows} \qquad \text{… (4.2)}$$

or

$$\sum Q_{In} = \sum Q_{out} \qquad \text{… (4.3)}$$

$$Q = \bar{v}A \qquad \text{… (4.4)}$$

where:

Q = flow rate (m^3/s)
$\bar{v}$ = average flow velocity (m/s)
A = sectional area (m^2)
V = volume stored (m^3)

The following rules are important in applying the principle of continuity:

Choose a control volume. This is the three-dimensional space for which the mass balance is being calculated. The space should be shaped so that:

- flow sections are taken across flows where discharges are known or have to be calculated;
- flow sections are taken where the flow lines are parallel or almost parallel;
- flow sections are perpendicular to the flow lines. This means that a section is not necessarily straight. In the case of subcritical sheet flow, for instance, flow lines could be drawn perpendicular to the contour lines and then the principle of continuity should be satisfied between successive flow lines as shown in **Figure 4.1**.

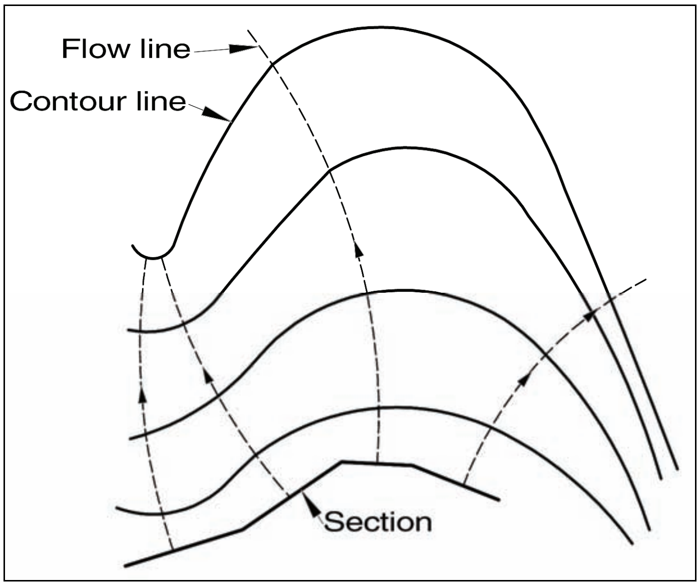


Figure 4.1: Choice of sections, where the direction of the flow changes across a section (Plan view with the flow direction as indicated by the flow lines)

4.2.4 Principle of conservation of energy (Energy principle)

The energy principle should always be satisfied, and it is often applied together with the principle of continuity (mass) to:

- determine relationships between velocities and depths at different sections when energy losses are calculable or are small enough to be neglected;

- calculate energy losses between sections where the values of the energy components are known.

In its "complete" form, expressed in terms of energy per unit weight, the Bernoulli equation for open channel flow applies along a flow line. It reads as follows:

$$\frac{\alpha_1 \bar{v}_1^2}{2g} + y_1 \mathrm{Cos}\,\theta_1 + z_1 = \frac{\alpha_2 \bar{v}_2^2}{2g} + y_2 \mathrm{Cos}\,\theta_2 + z_2 + \Sigma h_{f_{1-2}} + \Sigma h_{l_{1-2}} \qquad \ldots(4.5)$$

where:

α_i	=	coefficient compensating for variations in velocity across a section
$\bar{v}_i$	=	average velocity across a section (m/s)
g	=	gravitational acceleration (a value of 9,81 m/s² is generally used for design purposes) (m/s²)
y_i	=	depth of flow measured perpendicular to the streambed (m)
θ_i	=	longitudinal bed slope angle (°)
z_i	=	bed level at point where depth of flow = y_i (m)
$\Sigma h_{f_{1-2}}$	=	friction losses between sections 1 and 2 (m)
$\Sigma h_{l_{1-2}}$	=	sum of transition losses between 1 and 2 (m)

Subscript 1 refers to the upstream section and subscript 2 to the downstream section.

Each term represents either energy content or energy loss per unit weight, with the resulting dimension being length as illustrated in **Figure 4.2**. It is recommended that in order to obtain a clear picture, the different terms be represented graphically as in **Figure 4.2**.

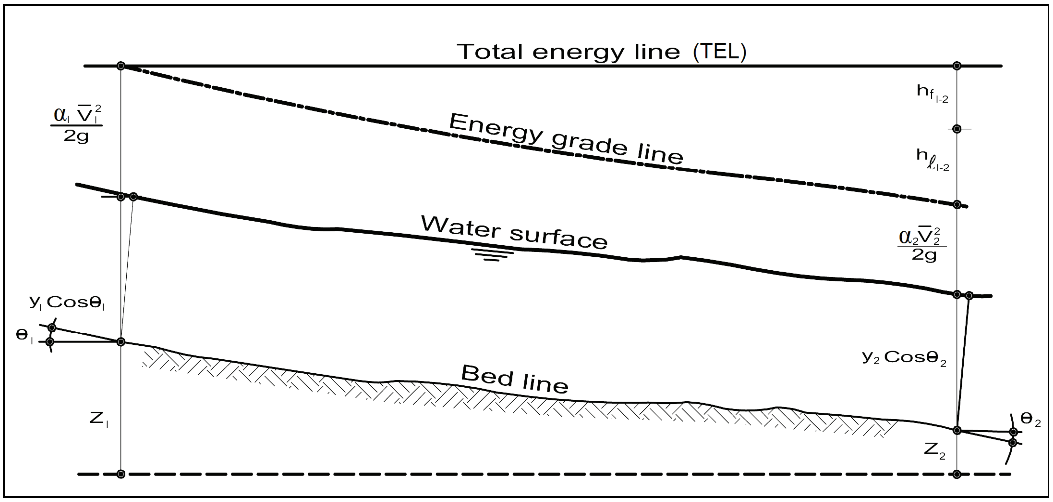


Figure 4.2: Longitudinal section along the flow path which reflects the different energy components

Depending on the circumstances, the energy equation may take on different forms as shown in **Table 4.2**.

Table 4.2: Different simplifications in the application of the energy equation

Simplified energy equations

(i) The first simplification that arises where slopes are not very steep ($\theta < 10^\circ$ or < 1: 5) is that:

$$y\cos\theta \approx y$$

(ii) In the case of **complex cross-sections**, α may be assumed to be equal to 1,05 **with** $\bar{v}_m$ = **average velocity in the main (deepest) channel** :

$$\alpha \approx 1{,}05$$

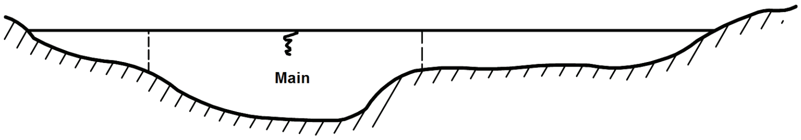


Figure 4.3: Complex cross-section

Thus

$$1{,}05\frac{\bar{v}_{m1}^2}{2g} + y_1 + z_1 = 1{,}05\frac{\bar{v}_{m2}^2}{2g} + y_2 + z_2 + \sum h_{f\,1-2} + \sum h_{l\,1-2} \qquad \ldots(4.6)$$

$\frac{\alpha\bar{v}_m^2}{2g}$ represents the total kinetic energy with $\frac{\bar{v}_m^2}{2g}$ the translational energy component and $(\alpha - 1)\frac{\bar{v}_m^2}{2g}$ the rotational energy component(4.1). The most accurate estimate of the kinetic energy component is obtained where the highest flow velocity occurs, and the rotational energy component has its lowest value. It is not yet practicable to allow for variations in rotational energy, as well as for the associated curved shape of the water surface across a section. The surface line is assumed to be horizontal across the section.

(iii) In the case of **single channels** it is generally assumed that α = 1.

$$\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2 + \sum h_{f\,1-2} + \sum h_{l\,1-2} \qquad \ldots(4.7)$$

Table 4.2: Different simplifications in the application of the energy equation (continued)

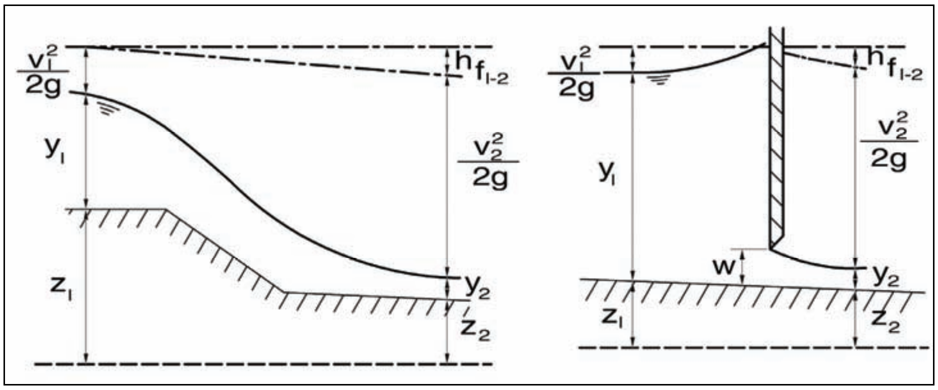
Simplified energy equations
(iv) With **near uniform flow, where the sectional area does not increase by more than 40%** downstream within a distance of 20 times the average hydraulic radius, the transition losses can be neglected, and it follows that: $$\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2 + \sum h_{f\,1-2} \qquad \ldots(4.8)$$ (Refer to Section 4.2.7 for applicable transition energy loss coefficients.)
(v) In the case of **converging flow over a short distance**, the downstream velocity is mainly determined by conversion of potential energy into kinetic energy. Energy losses may be ignored without causing much of an error in the calculation of v_2 and y_2.
 **Figure 4.4: Strongly converging flow**
In these cases $$\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2 \qquad \ldots(4.9)$$ or $$q = C_c w \sqrt{2gy_1 \frac{y_1}{y_1 + y_2}} \qquad \ldots(4.10)$$ the **orifice formula** with C_c = contraction coefficient (≈ 0,6) w = vertical sluice opening (m)
(vi) When $v_1 = v_2$ (**uniform flow**) $$z_1 - z_2 = \sum h_{f\,1-2} \qquad \ldots(4.11)$$ $$hf_{1-2} = \frac{\bar{v}^2 L}{C^2 R} \text{ (Chézy equation)} \qquad \ldots(4.12)$$ $$hf_{1-2} = \frac{\lambda \bar{v}^2 L}{2gR} \text{ (Darcy-Weisbach equation)} \qquad \ldots(4.13)$$ $$hf_{1-2} = \frac{n^2 \bar{v}^2 L}{R^{4/3}} \text{ (Manning equation)} \qquad \ldots(4.14)$$

Table 4.2: Different simplifications in the application of the energy equation (continued)

Simplified energy equations
(vii) In the case of uniform man-made water courses, it is often convenient to define the energy level relative to the bed, and the energy thus defined is the **specific energy, E**: $E = y + \alpha \frac{\overline{v}^2}{2g}$ …(4.15) Critical conditions are associated with $\frac{Q^2 B}{gA^3} = 1$
(viii) In open channel flow a given discharge cannot move past a section with less specific energy than the critical specific energy. This condition is referred to as "critical" and is characterised by a Froude number = 1. (The condition occurs where the channel's slope increases from less than the critical slope to larger than the critical slope, and the energy level is drawn down to the lowest possible value.)

4.2.5 Defining controls in open channel flow

All flow calculations should begin at controls; i.e. sections at which the energy level for a given discharge is known. Three types of control are identified as:

- sections where the energy level has been determined by previous calculation, starting at a more remote control; as for example when an analysis has been done for a river upstream of a dam and the depth of flow in a tributary, which joins within the backwater of the dam, has to be calculated;
- obstructions (man-made or natural) that induce flow to change from subcritical flow (upstream) to supercritical flow (downstream);
- sections where there is uniform open-channel flow, and where the depth of flow can be determined by using the Chézy or Manning equation.

One control could invalidate another in that the local depths of flow may be forced higher or lower. For example, a barrier across a river serves only as a control as long as the downstream water levels are low enough not to increase the water levels higher up in the river by drowning out the control.

The various controls are discussed below and the most basic formulae are given. In important cases, more sophisticated formulae should be used.

4.2.5.1 Points of release

Here the normal (uniform) depth of flow decreases from greater than critical to less than critical; for example at rapids or at the edges of road shoulders. To identify a point of release, determine whether $y_{n_1} > y_c > y_{n_2}$ (The normal depth, y_n, is determined by means of the Chézy or Manning equation.)

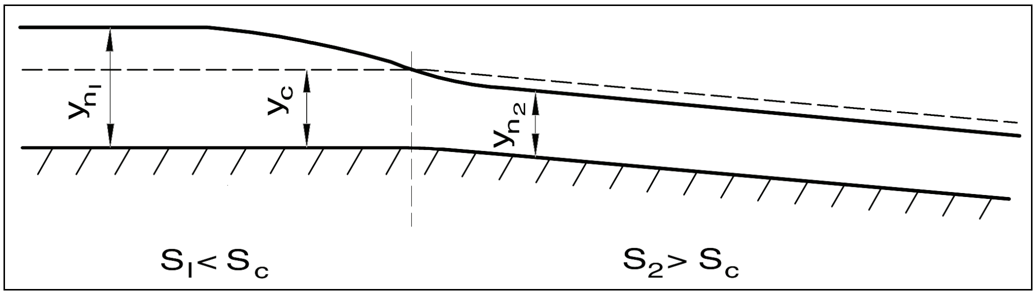


Figure 4.5: Point of release as a control

Just upstream of the point where the slope increases from mild ($y_n > y_c$) to steep ($y_n < y_c$), critical conditions occur, and here:

$$\frac{Q^2 B}{g A^3} = 1 \qquad \ldots(4.16)$$

4.2.5.2 Broad-crested conditions

Broad-crested conditions (L > 3H with $H = h + \frac{\bar{v}^2}{2g}$) occur for example at fully silted-up weirs in streams or where water flows across an embankment. This condition is equivalent to a release point with a very steep downstream slope.

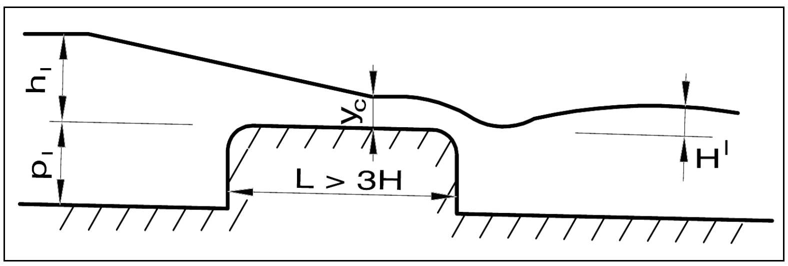


Figure 4.6: Broad-crested weirs

In this case critical conditions occur near the downstream end, and hence $\frac{Q^2 B}{g A^3} = 1$ at this section. This control is valid [4.2] as long as $H^1 < 0{,}8\ h_1$ (not drowned).

With $H_1 = h_1 + \frac{\bar{v}_1^2}{2g}$ and $\bar{v} = \sqrt{g y_c}$, since Fr = 1 it follows that:

$$Q = C_D \sqrt{g y_c}\, A = C_D \sqrt{g}\ b \left(\frac{2}{3} H_1\right)^{1,5} \qquad \ldots(4.17)$$

4.2.5.3 Sharp-crested weir (b << H)

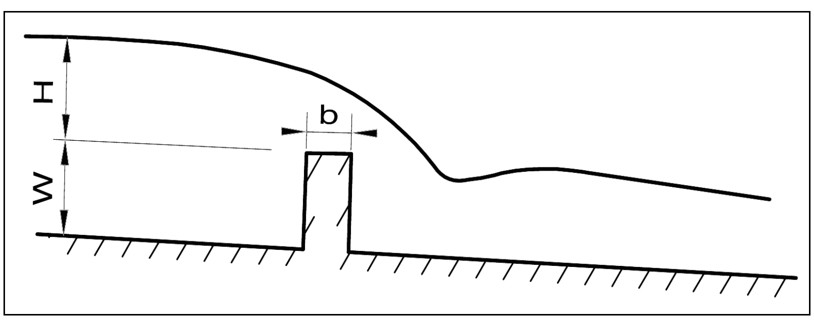


Figure 4.7: Sharp-crested weirs

This condition occurs where water flows over a thin-walled weir. The discharge for long weirs (wide streams) is given by [4.2]:

$$q = \frac{2}{3}\left(0{,}611 + 0{,}08\,\frac{H}{W}\right)\sqrt{2g}H^{\frac{3}{2}} \qquad \ldots(4.18)$$

where:

q = discharge per unit width (m^3/s/m)

With "short" weirs a value of 0,2H should be subtracted from the total length of the overflow section (B) to obtain the effective overflow width (B - 0,2H). This is a valid control as long as the downstream water surface is below the crest of the weir.

4.2.5.4 Cases between broad-crested and sharp-crested conditions

In such cases one may interpolate between the results of the equations for broad-crested and sharp-crested weirs.

4.2.5.5 Culvert inlets

Culverts with inlet control are discussed in **Chapter 7**. With high damming levels, orifice flow occurs.

4.2.5.6 General contractions

Where a contraction of section occurs (either in width or as a result of a local rising of the bed or through both width reduction and bed rising), a control may be formed. A control comes into being when the upstream energy level is lower than the minimum energy level (critical condition) needed at the contraction to pass the full discharge. Such conditions may be found, for example, at bridges with small openings or at rocky contractions across rivers.

4.2.5.7 Under-outlets (Orifices)

Orifice flow conditions may occur in road drainage at grid inlets (**Chapter 5**) and at culvert inlets (**Chapter 7**).

4.2.5.8 Uniform flow conditions

True uniform conditions only occur in long straight sections of a channel with a uniform cross-section. When a uniform control section needs to be found, one should ensure that uniform flow conditions do in fact prevail. This means that the depth of flow depends only on the local slope, roughness and sectional shape and is not affected by controls or obstructions upstream or downstream.

Either the Chézy or Manning equation is then used to determine the normal depth of flow. With Q (discharge) known, the slope (S), roughness coefficient (k_s or n) and section dimensions should be determined. For this purpose the longest possible "straight" section of the watercourse should be selected so that the slope can be determined with sufficient accuracy.

The cross-section should also be fairly uniform so that the transition losses can be limited between two consecutive cross-sections. In estimating the overall roughness coefficient, greater weight should be given to the roughness along the deeper parts than to the roughness along shallower parts of a cross-section.

Moreover, the minimum and maximum possible values for the variables should be determined so that minimum and maximum possible "uniform" depths of flow may be obtained. Once these depths are known, one may determine the depths upstream ($Fr < 1$) and downstream ($Fr > 1$) at the specific cross-sections of interest. Fortunately, if an incorrect depth is used at the control, the error will diminish with progressive analyses away from the control section. It is thus possible to test whether the maximum and minimum possible control depths eventually give the same results, or whether one should start calculations with the most conservative value.

When the Froude number at the control section is for instance between 0,8 and 1,2, the flow could easily change from subcritical to supercritical or vice versa, and it **cannot simply be accepted that the flow will be purely supercritical or subcritical**. In such a case a conservative value (maximum or minimum) should be accepted for the depth of flow.

4.2.5.9 Combination conditions

An example of a combined control condition that occurs in road drainage is where water flows simultaneously over a road and through a culvert underneath the road.

In this case the flow over the road is analysed according to release conditions, taking energy losses into account, and the culvert flow is analysed separately according to culvert flow formulae. The upstream flow depth is varied until the total flow equals the design discharge.

4.2.6 Friction losses

Strictly speaking, the various expressions for friction losses are only applicable to uniform flow conditions, but they are used to determine approximate energy losses in gradually changing non-uniform flow.

When analysing friction losses, a distinction must necessarily be made between three different flow conditions. The different conditions are roughly identified according to the Reynolds number, which is defined by:

$$R_e = \frac{\overline{v}R}{\nu}$$

with:

$\overline{v}$ = average flow velocity (m/s)
R = hydraulic radius (m)
ν = kinematic viscosity, with a general design value of $1{,}14 \times 10^{-6}$ m²/s for water

The value of R_e is then used to define the flow type as follows:

- Laminar flow ($R_e \leq 500$)
- Transition flow ($500 < R_e < 5000$)
- Turbulent flow ($R_e \geq 5000$)

In the following paragraphs relationships are provided to calculate the friction loss for the different flow types.

4.2.6.1 Friction loss calculation for laminar flow ($R_e \leq 500$)

Laminar open channel flow of water occurs only at extremely low velocities, and is only of practical interest where very shallow flow conditions occur; for example with shallow run-off over paved surfaces.

In such cases energy losses have been expressed in terms of the Darcy-Weisbach formula [4.3], even though this formula is strictly speaking only applicable to turbulent flow.

$$h_f = \frac{f\,L\,\bar{v}^2}{8\,g\,R} \qquad \text{…(4.19)}$$

where:

h_f	=	energy loss over distance L (m)
f	=	roughness coefficient
L	=	distance (m)
$\bar{v}$	=	average velocity (m/s)
g	=	gravitational acceleration (m/s²)
R	=	hydraulic radius i.e. area divided by wetted perimeter (m)

In laminar flow the roughness coefficient, f, is expressed in terms of the R_e number and a resistance coefficient K_0

$$f = \frac{K_o}{R_e} \qquad \text{…(4.20)}$$

Useful values of K_0 for application to drainage are as follows [4.3]:

Table 4.3: Typical values of K_0

Surface	K_0
Smooth	24
Concrete and bitumen	24 - 108
Pure sand	30 - 120
Gravel surface	90 - 400
Pure clay or loam	100 - 500
Eroded ground with little vegetation	1 000 - 4 000
Short prairie grass	3 000 - 10 000
Blue-grass	2 000 - 40 000

In interpreting these values it should be borne in mind that an increased value of K_0 mainly represents a decrease in effective flow area.

4.2.6.2 Friction loss calculation for transition conditions ($500 < R_e < 5000$)

It is seldom possible to calculate friction losses for flow accurately in this range. Maximum possible values for losses are obtained by applying the relationships for turbulent flow, and minimum values according to the relationships for laminar flow.

4.2.6.3 Friction loss calculation for turbulent flow ($R_e \geq 5000$)

Complete theoretical analysis [4.1], as well as experimentation has shown that the following equation is fundamentally correct and, as reflected here, may be generally applied for design purposes:

$$\bar{v} = 5{,}75\sqrt{gRS}\,\log\frac{12R}{k_s + \dfrac{3{,}3\upsilon}{\sqrt{gRS}}} \qquad \text{…(4.21)}$$

where:

$\bar{v}$	=	average velocity (m/s)
R	=	hydraulic radius (m)
S	=	energy slope, which is equal to bed slope only when flow is uniform (m/m)
k_s	=	roughness coefficient, representing the size of irregularities on bed and sides (m)
υ	=	kinematic viscosity ($\approx$ 1,14 x 10^{-6} m²/s for water)

As given here the equation is generally applicable to turbulent flow with "rough" or "smooth-wall" conditions.

Since "smooth-wall" conditions rarely occur in open channel flow (only with plastic pipes, etc), the equation can often be applied in the following simplified form:

$$\bar{v} = 5{,}75\sqrt{gRS}\,\log\left(\frac{12R}{k_s}\right) \qquad \text{…(4.22)}$$

which is equivalent to the Chézy equation

$$\bar{v} = C\sqrt{RS} \qquad \text{…(4.23)}$$

with:

$$C = 5{,}75\sqrt{g}\,\log\left(\frac{12R}{k_s}\right) \qquad \text{…(4.24)}$$

or

$$C = 18\log\left(\frac{12R}{k_s}\right) \qquad \text{…(4.25)}$$

with the units of C being $m^{0,5}/s$, if SI units are used.

Another formula that is not fundamentally correct, but which is generally used, is the Manning equation:

$$\bar{v} = \frac{R^{\frac{2}{3}}S^{\frac{1}{2}}}{n} \qquad \text{…(4.26)}$$

where:

n = roughness coefficient with units of $s/m^{1/3}$

Note that the roughness coefficient n is not a constant, but varies in roughness with the change in hydraulic radius.

Information available from various sources on k_s-values and n-values is given in **Figures 4.8a**, **4.8b** and **4.8c** for practical use. The descriptions of different conditions are given opposite the applicable average k_s-values. When estimating a k_s-value for practical use, it is useful to bear in mind that the k_s-value represents the effective average size of eddies formed to fit in with the irregularities on the bed. In this way a realistic estimate of extreme k_s-values may be made for any condition. As the velocity equation for "rough-wall" conditions (Equation 4.22) is rather insensitive to changes in the k_s-value, a satisfactory and realistic answer can usually be obtained when calculating the velocity. Where the k_s-value is very low, for example in the case of plastic pipes, one should test to see whether rough-turbulent conditions are indeed present:

$$\frac{\sqrt{gRS}\,k_s}{\upsilon} > 30 \qquad \text{…(4.27)}$$

If the value obtained in the above equation is less than 30, the complete Equation 4.21 should be used to calculate the flow velocity.

The Manning equation as such does not compensate sufficiently for changes in the n-value with hydraulic radius. Applicable n-values may be read off **Figure 4.8c**.

In cases of extreme roughness, for example where willow trees grow in a river, the Manning and Chézy equations are not strictly applicable, since large-scale transition losses dominate rather than friction losses. In such cases roughness coefficients should be handled carefully.

In sand bedded rivers, roughness could vary greatly as different bed-forms develop. Roughness coefficients in such rivers need to be determined through an iterative process [4.5].

Photographs 4.1 to **4.5** reflect some typical roughness values in South African rivers.

Photograph 4.1: River section in the Komati River. Estimated roughness = 0,4 m

Photograph 4.2: River section in the Klip River. Estimated roughness = 0,10 m

Photograph 4.3: River section in the Limpopo River. Estimated roughness = 0,50 m

Photograph 4.4: River section in the Berg River. Estimated roughness = 0,3 m

Photograph 4.5: Concrete side drain. Estimated roughness = 0,002 m (Manning n-value = 0,016 s/m$^{1/3}$)

CHANNELS AND RIVERS	FLOOD PLAINS AND OVERLAND FLOW	STONE, ROCK AND BRICK	k_s (m)
	Dense Willows (n~0,2)		10
Very weedy reaches	Heavy stand of timber, stage reaching branches		8
Deep pools in minor streams			
Sluggish reaches, weedy deep pools	Heavy stand of timber, stage below branches		6
			4
Dense weeds, high as flow depth in channels	Medium to dense bush		
Mountain streams, cobbles with large boulders	Light brush and trees		2
			1
Clean, winding streams, some pools and chutes	Cleared land with tree stumps no sprouts	Jagged and irregular rock cuts	
			0,8
Dense weeds or aquatic	Mature field crops	Smooth and uniform rock cuts	0,6
Plates in deep channels			
Mountain streams, cobbles and few boulders	Pasture, no brush, long grass mature row crops	Rubble or riprap placed above water	0,4
Natural straight streams, no rifts or deep pools	Cultivated area, no crop pasture, no brush, short grass	Hand placed pitching	
			0,2
Excavated channel with short grass			
			0,1
Clean uniform gravel section			
		Rubble masonry, cemented	0,08
Excavated or dredged earth channel weathered			0,06
			0,04
	Bare clay loam (Eroded)	Random stone in mortar	
			0,02
	Graveled surface	Paved invert sewer smooth bottom	
Clean, newly excavated earth channel			
			0,01
		Dressed stone in mortar	
			0,008
			0,006
	Rough asphalt or concrete	Vitrified subdrain with open joint	
		Dressed ashlar brickwork lined with cement mortar	0,004
		Vitrified sewer with manholes, inlets, etc. Sanitary sewers coated with sewage slimes including bends, etc.	
			0,002
	Smooth concrete or asphalt, bare sand	Vitrified sewer glazed brickwork	
		Common drainage tile	
			0,001

Figure 4.8a: Average roughness coefficients for rough turbulent flow (Refer to Figure 4.8c)

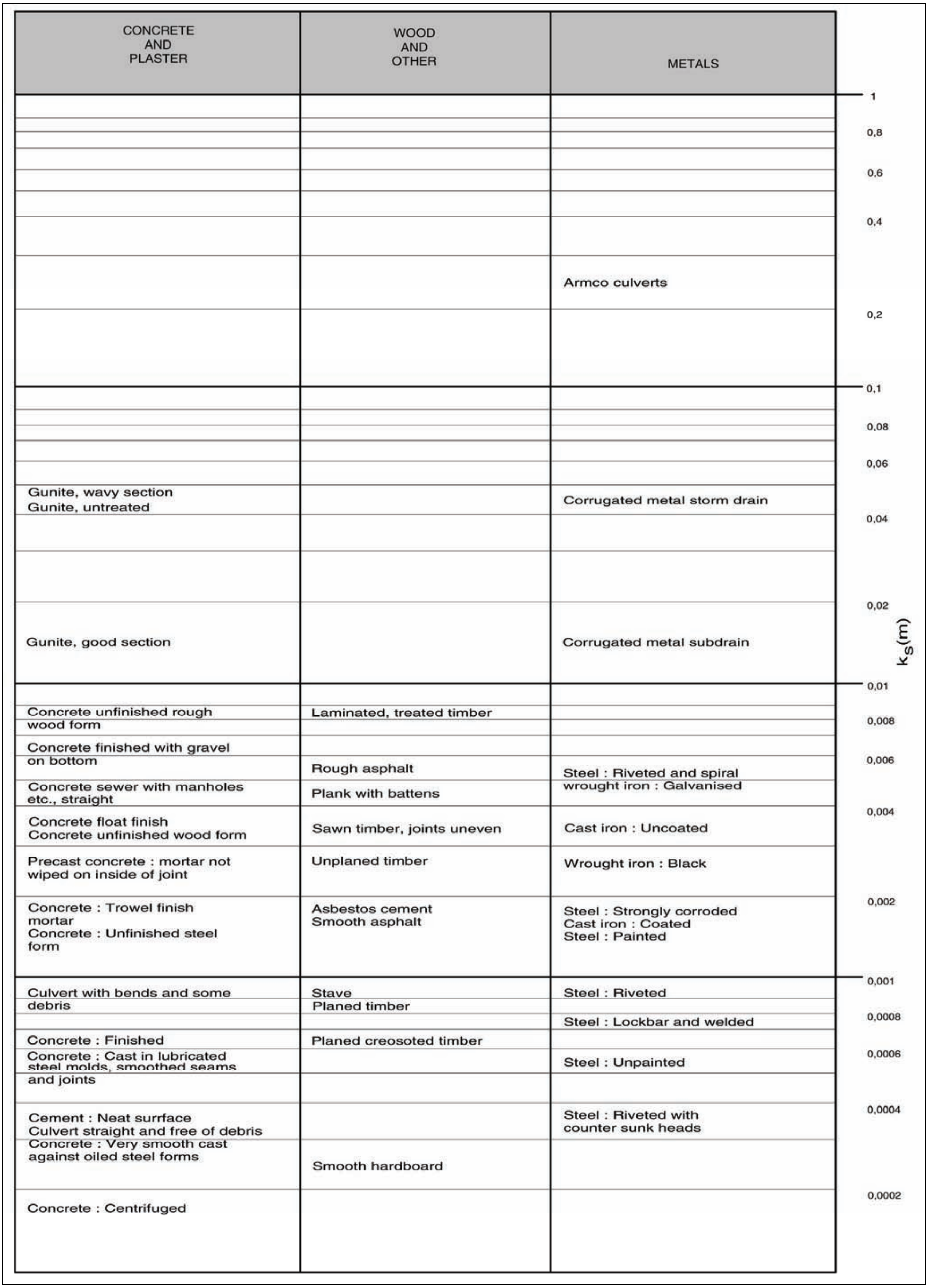


Figure 4.8b: Average roughness coefficients for rough turbulent flow (Refer to Figure 4.8c)

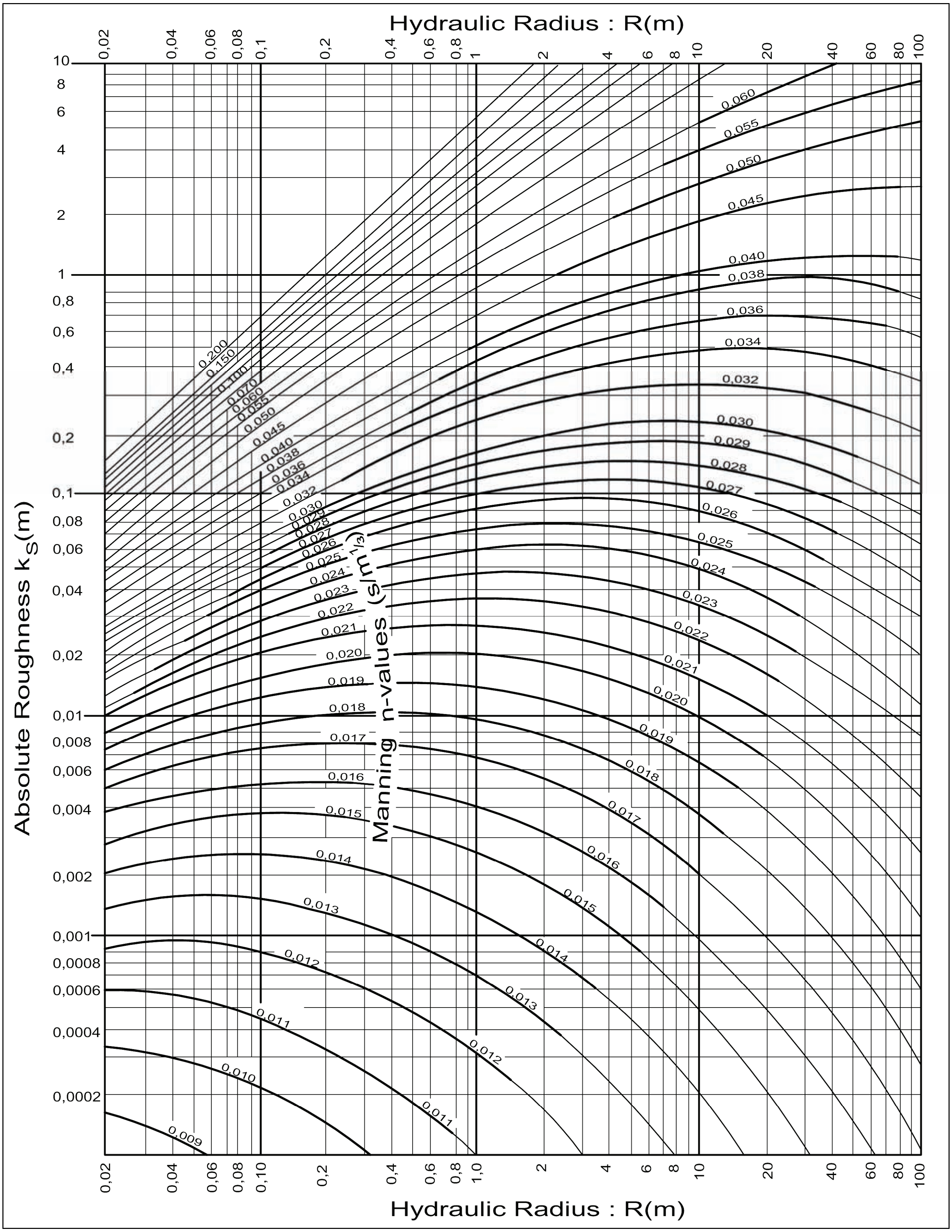


Figure 4.8c: Average roughness coefficient for rough turbulent flow
(A copy of this diagram has been included on the supporting CD.)

4.2.7 Transition losses

Transition losses occur where there are changes in the magnitude and/or direction of flow velocities. Unlike friction losses, which represent overall application of energy to maintain flow, transition losses represent energy applied to maintain local eddies in separation zones. These eddies develop locally where the stream breaks away from the solid boundary. There is a particular tendency to break away where the cross-section becomes larger downstream, or where other major changes in flow direction take place, for example at bends. Convergence losses are usually much smaller than divergence losses.

Losses may be diminished by shaping a conduit so as to decrease the size of the separation eddies. Uniform open channel flow along a channel is only possible if the channel bed drops across each transition by a height equal to the local transition loss.

It is much more difficult to quantify transition losses in open channel flow than in pipe flow, due to a range of possible values of the hydraulic radius for given dimensions of the conduit.

The following formulae are recommended for general use, since these are fundamentally correct and usually provide conservative but realistic answers.

Divergence losses (downstream section A_2 larger than upstream section A_1)

The recommended equation is applicable to subcritical flow Fr < 1) and reads as follows:

$$h_l = C_L \frac{\overline{v}_1^2}{2g}\left(1 - \frac{A_1}{A_2}\right)^2 \qquad \ldots(4.28)$$

where:

h_l	=	transition loss (m)
C_L	=	loss coefficient with maximum value = 1 for sudden transitions and a minimum value of 0,3 for gradual transitions (taper of 1:4)
$\overline{v}_1$	=	upstream average velocity (m/s)
A_1	=	upstream sectional area (m²)
A_2	=	downstream sectional area (m²)

If C_L =1, this implies that there is no recovery of kinetic energy whilst the velocity decreases from v_1 to v_2. If the value of C_L lies between 0,3 and 1,0, this implies some recovery of energy. A value of 0,3 is only applicable if the taper is at least equal to the optimum practical value of 1:4, as shown in **Figure 4.9**.

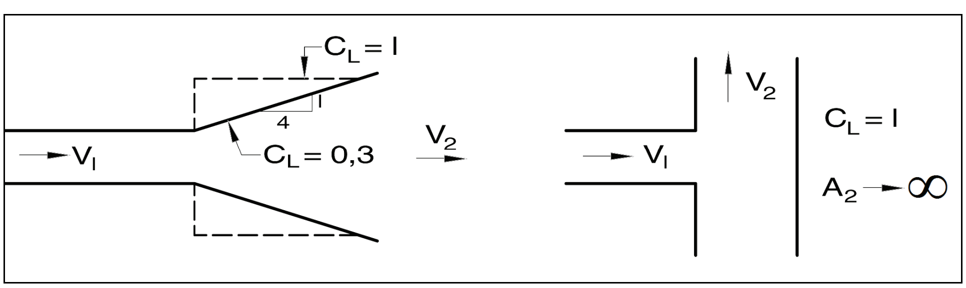


Figure 4.9: Divergence loss coefficient

Since supercritical flow is subject to upstream control, it is not able to spread over a short distance or to rapidly change direction without changing to subcritical flow (hydraulic jump). In the latter case the energy losses are very high, and an analysis should be performed by means of the momentum principle rather than the energy principle.

Convergence losses (subcritical flow)

The recommended formula reads:

$$h_l = C_L \frac{\overline{v}_2^2}{2g} \qquad \ldots(4.29)$$

where:

h_l	=	transition loss (m)
$\overline{v}_2$	=	downstream average velocity (m/s)
C_L	=	0,35 for sudden contractions (0,18 for rounded contractions)

Bend losses (subcritical flow)

The recommended formula reads [(4.4)] :

$$h_l = \frac{2B}{r_c}\frac{\overline{v}^2}{2g} \qquad \ldots(4.30)$$

where:

h_l	=	transition loss (m)
B	=	channel width (m)
r_c	=	centre line radius (m)
$\overline{v}$	=	uniform channel average velocity (m/s)

This equation has been found to be valid for direction changes between 90° and 180° mainly because the losses at bends are concentrated where the stream initially breaks away from the solid boundary. The losses at bends vary very little for directional changes between 90° and 180°. It was also established that the losses at clear direction changes of less than 90° can also be described by this relationship.

4.2.8 Rules for the application of the energy equation

Practical application of the energy equation (together with the continuity principle) typically consists of working in steps along the flow route, starting at a point where flow conditions are known (control) and ending where flow conditions need to be known. Cross-sections are selected so that energy losses can be calculated between successive sections, either as transition losses or as friction losses. It is practicable in the selection of sections to start off by selecting sections just upstream and downstream of significant transition losses, and thereafter to sub-divide the rest of the channel into reaches in between cross-sections, which may be regarded as near uniform.

The following steps may be followed:

- Identify every possible control that may affect the depth of flow in the area of interest.

- In the case of subcritical flow, analyses **may** only be performed upstream and in the case of supercritical flow, only downstream.

- Choose consecutive sections from the control(s) to the end of the area in which the depths of flow are required. Sections are chosen perpendicular to flow lines, and only where the flow lines do not curve sharply. Sections are chosen at controls and otherwise upstream and downstream of areas with significant transition losses. Sections outside regions of transition losses are chosen such that the **sectional areas and hydraulic radii do not**

vary by more than 40 per cent from one section to the next. This ensures that friction losses are calculated with sufficient accuracy.

- If the sectional areas do not increase by more than 40 per cent in the stream direction within a distance of 20 times the mean hydraulic radius, there is **no need** to allow for transition losses.

- Start with the most likely control and determine the energy level. (Later ensure that the controls being used are indeed valid.)

With $(y_1 + z_1)$ or $(y_2 + z_2)$ known at the control, estimate the value of:

$$\left(y_2 + z_2\right), \text{ if } \left(\frac{Q^2B}{gA^3} > 1\right) \text{ or } \left(y_1 + z_1\right), \text{ if } \left(\frac{Q^2B}{gA^3} < 1\right)$$

and calculate the corresponding energy level at the next section, as well as the energy losses between the two sections. Thereafter test whether the estimated and calculated energy levels agree within say 10 mm. If not, choose an improved value of (y + z) and repeat.

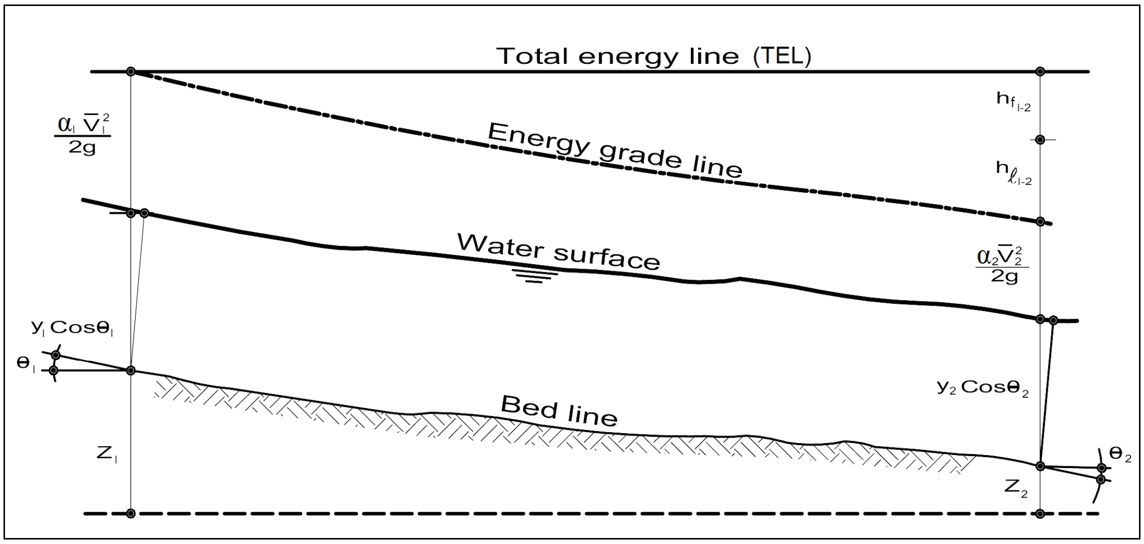


Figure 4.10: Energy components

Choose subsections such that the depth and roughness measure k_s do not vary by more than a factor of 2 across every subsection. Roughness coefficients are obtained from **Figure 4.8**. Every subsection should be more or less perpendicular to the direction of flow. **Table 4.4** reflects some of the calculation steps required for the application of the conservation of energy law.

Table 4.4: Calculation steps in applying the law of conservation of energy

Subcritical flow (Fr < 1)	Supercritical flow (Fr > 1)
Determine $z_2 + y_2$ (at control) Estimate $z_1 + y_1$ (upstream) $\left(\approx z_2 + y_2 + \Delta x S_{f_2}\right)$ Calculate $\alpha_1 \frac{\bar{v}_1^2}{2g}$	Determine $z_1 + y_1$ (at control) Estimate $z_2 + y_2$ (downstream) $\left(\approx z_1 + y_1 - \Delta x S_{f_1}\right)$ Calculate $\alpha_2 \frac{\bar{v}_2^2}{2g}$

Test to determine whether Equation 4.5 is satisfied:

$$\frac{\alpha_1 \bar{v}_1^2}{2g} + \left(z_1 + y_1\right) = \frac{\alpha_2 \bar{v}_2^2}{2g} + \left(z_2 + y_2\right) + \Sigma h_{f_{1-2}} + \Sigma h_{l_{1-2}} \qquad \ldots(4.31)$$

Single streams

$\alpha = 1$

$$\bar{v} = \frac{Q}{A} \qquad \text{...(4.32)}$$

$$R = \frac{A}{P} \qquad \text{...(4.33)}$$

$$h_f = \overline{S_f}\Delta x = \left(\frac{\bar{v}_1^2}{C_1^2 R_1} + \frac{\bar{v}_2^2}{C_2^2 R_2}\right)\frac{\Delta x}{2} \text{ (Chézy)} \qquad \text{...(4.34)}$$

where:

$$C = 18 \log\left(\frac{12R}{k_s}\right) \qquad \text{...(4.35)}$$

or

$$h_f = \left(\frac{\bar{v}_1^2 n_1^2}{R_1^{\frac{4}{3}}} + \frac{\bar{v}_2^2 n_2^2}{R_2^{\frac{4}{3}}}\right)\frac{\Delta x}{2} \text{ (Manning)} \qquad \text{...(4.36)}$$

For k_s and n-values consult **Figure 4.8**. ***Transition*** losses, h_l approach the value of 0, if $A_2 < 1{,}4A_1$, and $\Delta x > 20\bar{R}$. Otherwise see Section 4.2.7 for the determination of ***transition*** losses, h_l.

Complex sections

$\alpha = 1{,}05$

where:

$\bar{v}$ = highest average velocity in main (deepest) channel (e.g. $\bar{v}_c$ in the schematic layout, **Figure 4.11**)

$$\bar{v}_c = C_c \sqrt{R_c S_f} \qquad \text{...(4.37)}$$

From the above calculations the conveyance, K, can be determined if the above relationship is reorganized/rewritten as shown below.

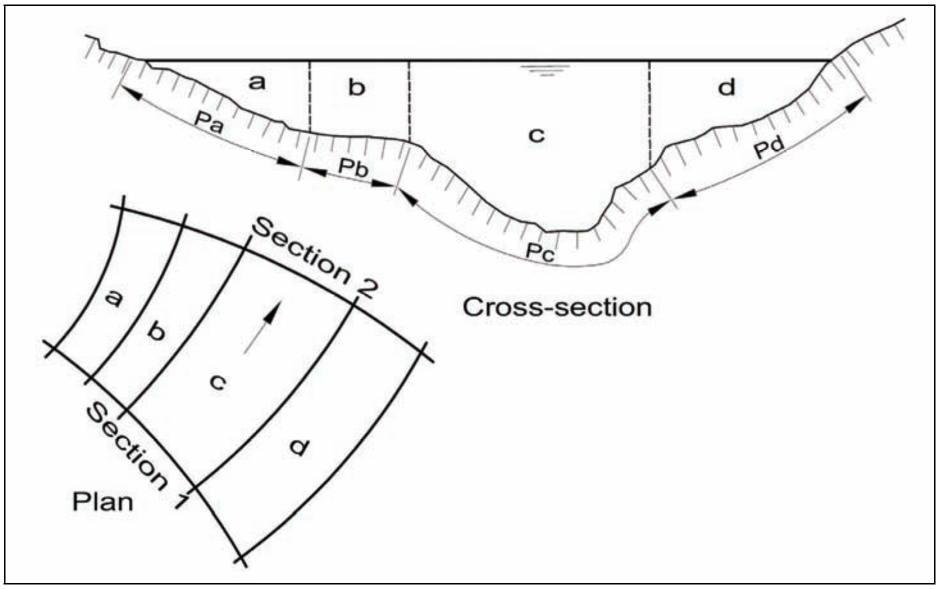


Figure 4.11: Complex section

$$S_f = \left(\frac{\Sigma Q}{\Sigma K}\right)^2 \qquad \text{...(4.38)}$$

$$R_c = \frac{A_c}{P_c} \qquad \text{...(4.39)}$$

$$K_c = C_c A_c \sqrt{R_c} \text{ (Chezy)} \qquad \text{...(4.40)}$$

$$C_c = \left(18 \log \frac{12R_c}{k_{sc}}\right) \quad (m^{1/2}/s) \qquad \text{...(4.41)}$$

or

$$K_c = \frac{A_c R_c^{\frac{2}{3}}}{n} \text{ (Manning)} \qquad \text{...(4.42)}$$

$$h_f = \frac{(\Sigma Q)^2}{(\Sigma K)^2} \Delta x \qquad \text{...(4.43)}$$

or

ΣQ = Total discharge

$$\Sigma Q = (K_a + K_b + K_c + ...)S_f^{1/2} \qquad \text{...(4.44)}$$

Branched flows

When a stream branches into two or more streams for some distance, as illustrated in **Figure 4.12**, the following procedure should be applied:

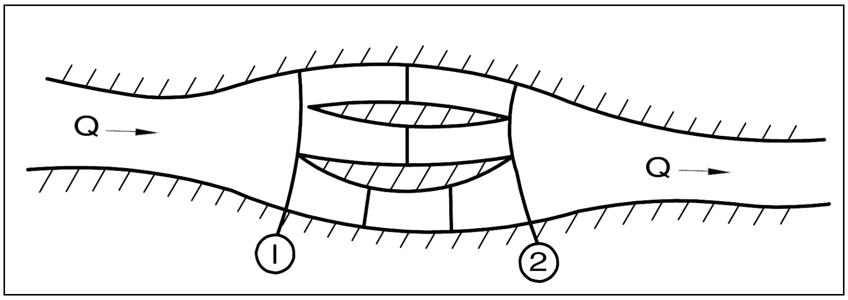


Figure 4.12: Branched Flows

Perform the analysis as before up to just downstream (Fr < 1) or upstream (Fr > 1) of the division. Then estimate what fraction of the total flow passes through each branch. Calculate the energy losses for flow through each branch and adjust the flows until the same energy losses are obtained through each branch between (1) and (2).

4.2.9 Conservation of momentum (Momentum principle)

The momentum equation can be used to analyse flow conditions where:

- It is necessary to calculate the forces exerted on obstructions, or
- The relationships between flow conditions have to be determined in areas where large unknown energy losses occur, e.g. with hydraulic jumps.

In the general form for steady conditions, the momentum equation for a chosen x direction could be defined as follows:

> Sum of <u>all</u> the external force components acting on a flow system (control volume) in the x direction = (Sum of the momentum components of the outflows in the x direction) – (Sum of the momentum components of the inflows in the x direction).

The momentum equation is always applied across a control volume as shown in **Figure 4.13**.

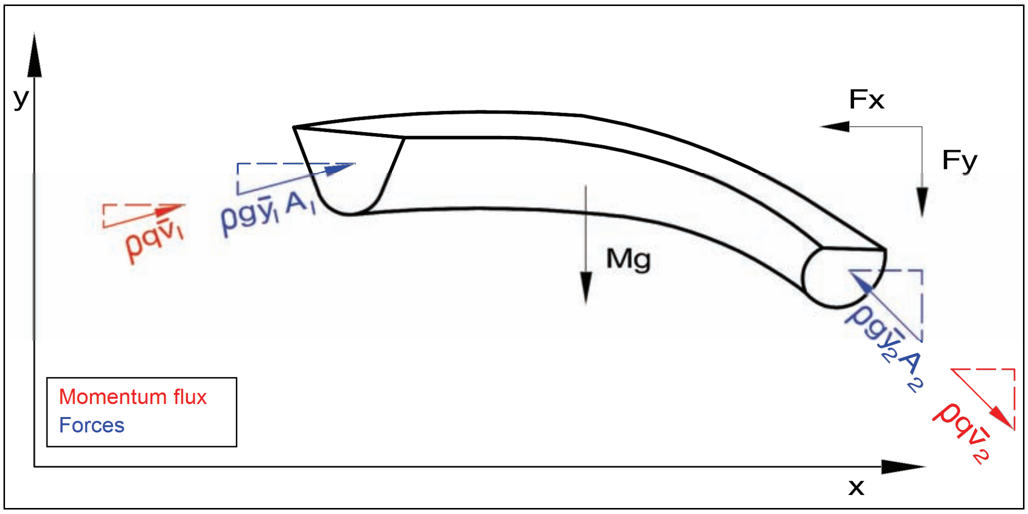


Figure 4.13: Momentum flux $(\rho q \bar{v})$ and the forces $(\rho g \bar{y} A)$ acting on the control volume

In the typical case shown above:

x – direction: $$(\gamma\bar{y}_1 A_1)_x - (\gamma\bar{y}_2 A_2)_x - F_x = \rho Q(\bar{v}_{2x} - \bar{v}_{1x}) \quad \ldots(4.45)$$

y – direction: $$(\gamma\bar{y}_1 A_1)_y - (\gamma\bar{y}_2 A_2)_y - F_y - Mg = \rho Q(\bar{v}_{2y} - \bar{v}_{1y}) \quad \ldots(4.46)$$

where:

$\gamma\bar{y}A$	=	external hydrostatic force width
γ	=	specific weight for water (9,81 x 10^3 N/m³)
$\bar{y}$	=	distance between water surface and centre of gravity of section (m)
A	=	sectional area (m²)
$(\gamma\bar{y}A)_x$	=	force component in x direction
$(\gamma\bar{y}A)_y$	=	force component in y direction
$\bar{v}_x$	=	average velocity component in x direction (m/s)
$\bar{v}_y$	=	average velocity component in y direction (m/s)
F_x	=	size of force component exerted on flow by the solid boundary and obstructions in x direction (N)
F_y	=	size of force component exerted on flow by the solid boundary and obstructions in y direction (N)
Mg	=	weight of the enclosed fluid mass (N)

4.2.10 Rules for the application of the momentum equation

- Select axes. It is convenient to work with horizontal and vertical axes.

- Choose sections perpendicular to the direction of flow at sections where the flow lines are parallel. The sections are chosen such that either all the external forces operating on the enclosed fluid are known, or so that the body of the fluid is included on which the unknown operating forces have to be determined.

- Identify all external forces, namely:
  - o weight of enclosed fluid (control volume);
  - o hydrostatic forces exerted on the cross sections (boundaries of the control volume) as a result of the pressure at the sections;
  - o forces exerted on fluid by the boundary and obstructions.

In open channel flow it is usually realistic to accept that the variation in pressure is hydrostatic where the flow lines are parallel and/or where the fluid moves very slowly (e.g. break-away eddies). Where supercritical flow penetrates partially right up to an obstruction, the pressure may be greater than hydrostatic, as illustrated [4.2] in **Figure 4.14**.

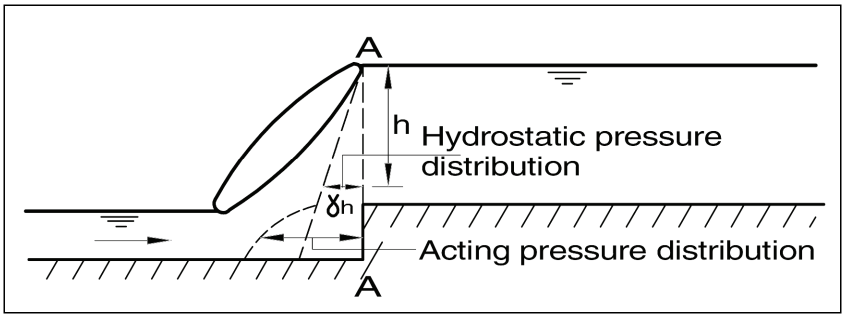
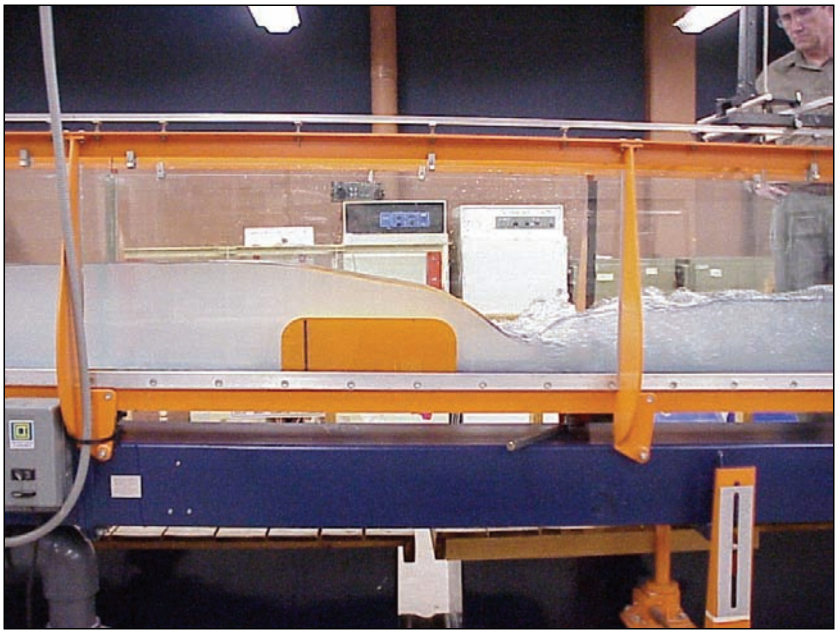


Figure 4.14: Pressure distribution at a step

Once the equations have been compiled one can solve the unknowns. (The momentum equation is generally used with the continuity equation.) It should be remembered that the force components in the momentum equation are those that are exerted on the fluid at the selected cross sections. The force components exerted by the fluid on obstructions are equal and opposite to the calculated values.

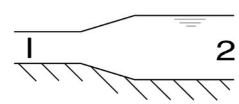
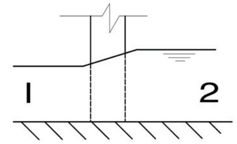
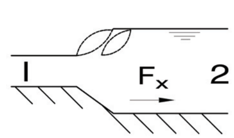
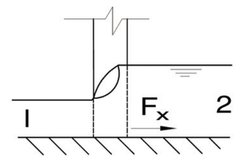
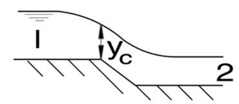
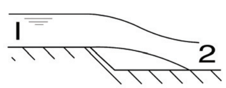
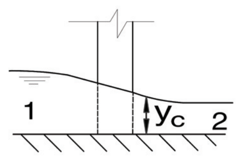


Photograph 4.6: Hydraulic jump

4.2.11 Analysis of transition conditions

Table 4.5 (a) to **(c)** is a summary of the most common transition relationships.

Table 4.5 (a): Summary of the analysis of transition conditions (Slope increase, bed drops or the channel widens)

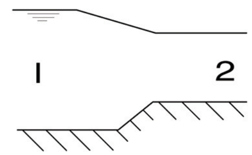
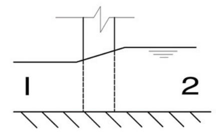
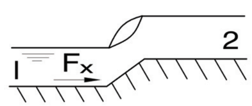
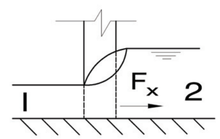
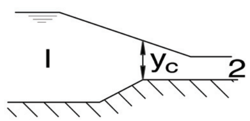
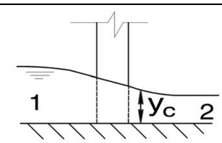
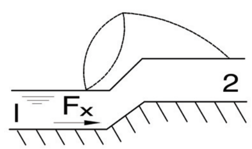
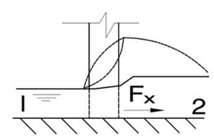
Froude	Longitudinal section		Control	Appropriate equations	Energy loss h_{1-2}	#
	Slope changes	Section widens				
$Fr_1 < Fr_2 < 1$			Y_2 determined by downstream conditions	$\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2 + h_{1-2}$ or equations 4.45 and 4.46 can be used	$h_{1-2} = C_L \frac{\bar{v}^2}{2g}(1 - \frac{A_1}{A_2})^2$ $0{,}3 > C_L < 1{,}0$	1
$Fr_1 > 1$ $Fr_2 < 1$			Y_2 determined by downstream conditions Y_1 determined by upstream conditions	Small slopes $(\gamma \bar{y}_1 A_1)_X - (\gamma \bar{y}_2 A_2)_X + F_X = \rho Q(\bar{v}_2 - \bar{v}_1)_X$	Large energy loss	2
$Fr_1 < 1$ $Fr_2 > 1$		Not possible	Critical conditions at point of release	$\frac{Q^2 B}{g A^3} = 1$ Critical conditions at the release point $\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2$	Small energy loss	3
$Fr_1 > 1$ $Fr_2 > 1$			Y_1 determined by upstream conditions	$\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2$	Small energy loss	4

Note: #

1 If the downstream channel is long, y_2 will approach y_n.
2 Due to high-energy losses, the momentum equation should be used. At a stable hydraulic jump the forces are balanced.
3 Under conditions where the floor drops and the section widens, the flow might not follow the channel sides.
4 The stream will not always follow the sides or the bottom everywhere.

Possibility of back circulation near the sides might be experienced when the cross-sectional area increases rapidly.

Table 4.5 (b): Summary of the analysis of transition conditions (Slope decreases, bed steps upwards or the channel converges)

Froude	Longitudinal section: Floor rises	Longitudinal section: Section converges	Control	Appropriate equation	Energy loss h_{1-2}	#
$Fr_1 < 1$ $Fr_2 < 1$			Y_2 determined by downstream conditions	$\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2 + h_{1-2}$	$h_{1-2} = C_L \frac{\bar{v}^2}{2g}$ $0{,}18 > C_L < 0{,}35$	**5**
$Fr_1 > 1$ $Fr_2 < 1$			Y_2 determined by downstream conditions Y_1 determined by upstream conditions	$(\gamma \overline{y_1} A_1)_X - (\gamma \overline{y_2} A_2)_X + F_X = \rho Q(\bar{v}_2 - \bar{v}_1)_X$ applicable to small slopes. Difficult to determine F_x	Large energy loss	**6**
$Fr_1 < 1$ $Fr_2 > 1$			Critical conditions at point of release	$\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2 + h_{1-2}$	$h_{1-2} = C_L \frac{\bar{v}^2}{2g}$ $0{,}18 > C_L < 0{,}35$	**7**
$Fr_1 > Fr_2 > 1$			Y_1 determined by upstream conditions. Damming can occur when the criteria in Note 8 is met	$\frac{\bar{v}_1^2}{2g} + y_1 + z_1 = \frac{\bar{v}_2^2}{2g} + y_2 + z_2 + h_{1-2}$, for limited damming	Energy loss is small unless dammed – then large energy loss will occur across the hydraulic jump	**8**

Note: #

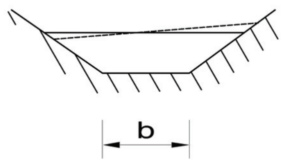
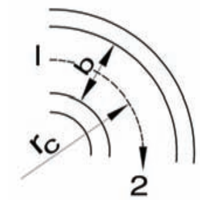
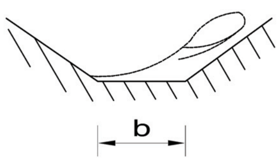
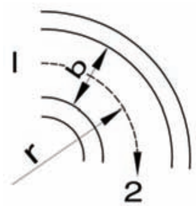
5 If the downstream channel is long, y_2 will approach y_n.

6 If F_x > than the hydrostatic force, refer to paragraph 4.2.9

7 Analysis may be conducted upstream and downstream from the critical section. Critical conditions will occur at the section of maximum contraction.

8 Damming occurs when $\frac{\bar{v}_1^2}{2g} + y_1 + z_1 < \frac{\bar{v}_2^2}{2g} + y_2 + z_2 + h_{1-2}$ or if the force F_x is high.

Table 4.5 (c): Summary of the analysis of transition conditions (Bends in channel sections)

Froude	Longitudinal section		Control	Appropriate equation	Energy loss h_{1-2}	#
$Fr_1 < 1$ $Fr_2 < 1$			Y_2 determined by downstream conditions	$\frac{\overline{v}_1^2}{2g} + y_1 + z_1 = \frac{\overline{v}_2^2}{2g} + y_2 + z_2 + h_{1-2}$	$h_{1-2} = \frac{2b}{r_c}\frac{\overline{v}_1^2}{2g}$	**9**
$Fr_1 > 1$ $Fr_2 > 1$			Y_1 determined by upstream conditions	$\frac{\overline{v}_1^2}{2g} + y_1 + z_1 = \frac{\overline{v}_2^2}{2g} + y_2 + z_2 + h_{1-2}$	Losses high when damming occurs	**10**

Note: #

9 Refer to Section 5.4 for the required raising of the channel wall on the outside of the bend.

10 Stream might overtop the channel or a hydraulic jump can be formed. Changing the flow direction under supercritical conditions should be avoided at all times. Experienced persons can conduct the analyses.

4.3 PIPE FLOW

4.3.1 Introduction

The general design problem reads: given a discharge, what difference in energy level is needed to pass the fluid through a pipe or culvert; or: what size of conduit is needed to convey the fluid with a predetermined difference in energy levels?

Just as with open channel flow, only the continuity, energy and momentum equations are used. Virtually all analyses of flow conditions are done by means of the combination of continuity and energy equations, whereas the momentum equation is used mainly in drainage pipes to determine forces at bends and other transitions.

Flow analyses should again begin at controls; i.e. at sections where the energy levels for given discharges are known and calculations essentially proceed upstream. Since the hydraulic radius can only have one value under full flow conditions, pipe flow analyses are simpler than those for open channel flow.

The most important applications of pipe flow theory in road drainage pertain to culverts and storm water pipes under full-flow conditions.

4.3.2 Principle of continuity

Except where flood attenuation is considered, only the continuity equation for steady incompressible flow is typically of use in drainage.

It reads:

$$\Sigma \text{ Inflows} = \Sigma \text{ Outflows} \qquad \ldots(4.47)$$

or

$$\Sigma Q_{in} = \Sigma Q_{out} \qquad \ldots(4.48)$$

Again, sections are taken perpendicular to the direction of flow and only where the flow lines are straight and parallel.

4.3.3 Energy principle

The energy principle is represented by the Bernoulli equation for pipe flow.

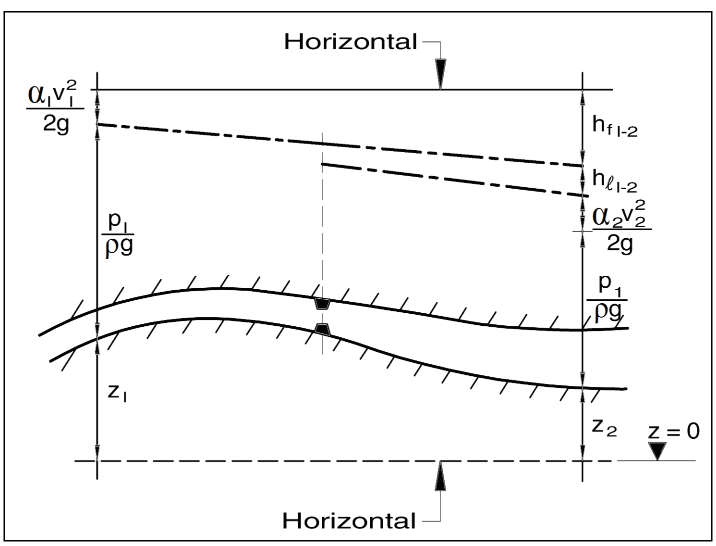


Figure 4.15: Energy components for pressurized flow in a pipe

$$\frac{\alpha_1 \overline{v}_1^2}{2g} + \frac{p_1}{\gamma} + z_1 = \frac{\alpha_2 \overline{v}_2^2}{2g} + \frac{p_2}{\gamma} + z_2 + h_{f_{1-2}} + h_{t_{1-2}} \qquad \ldots(4.49)$$

where:

α_i	=	velocity coefficient with a value of 1 in most practical applications
$\overline{v}_i$	=	average velocity (m/s)
Q	=	discharge (m³/s)
A	=	cross-sectional area ($\frac{\pi D^2}{4}$ for round pipes with diameter D) (m²)
p	=	intensity of pressure at centre-line (Pa)
γ	=	specific weight (value for water 9,81 x 10^3 N/m^3)
z	=	centre line elevation (m)
h_f	=	friction losses, which may be expressed in terms of the Chézy or Manning equations (m)
h_t	=	transition losses occurring where the flow velocity changes in magnitude or direction (m)

4.3.4 Application of the energy equation

Start at a control. The following could be used as controls:

- Energy levels determined by drowned open channel flow conditions; or
- Free outflow where p/γ = 0 and the energy level may be calculated.

Calculations also begin with a known or estimated conduit size. Friction losses (in the normal case of turbulent flow) could be calculated by means of the Chézy or Manning equations (See **Figure 4.8** for roughness coefficients.)

Chézy:

$$h_{f_{1-2}} = \frac{\overline{v}^2 L}{C^2 R} \qquad \ldots(4.50)$$

where:

$$\overline{v} = 5{,}75\sqrt{gRS}\,\log \frac{12R}{k_s + \frac{3{,}3\upsilon}{\sqrt{gRS}}} \qquad \ldots(4.51)$$

and:

$\overline{v}$	=	average velocity (discharge divided by area) (m/s)
L	=	pipe length (m)
R	=	hydraulic radius (m)

$$R = \frac{A}{P} = \frac{D}{4} \text{ for circular pipes (m)} \qquad \ldots(4.52)$$

k_s	=	measure of absolute roughness (from **Figure 4.8**)
υ	=	kinematic viscosity with a design value of 1,14 x 10^{-6} m²/s for water
S	=	energy gradient = $\frac{h_f}{L}$

If "rough" pipes are used, i.e. pipes not made of plastic or similar smooth material, the value of C may be calculated as follows:

$$C = 5{,}75\sqrt{g}\,\log\left(\frac{12R}{k_s}\right) \qquad \ldots(4.53)$$

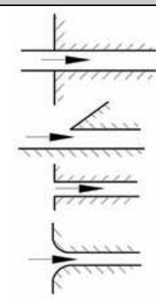
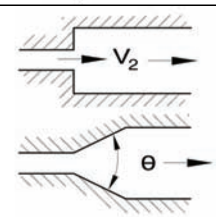
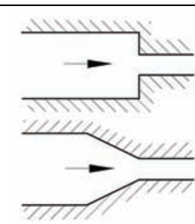
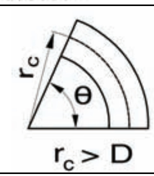
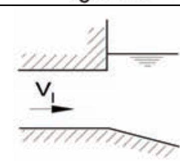
Manning:

$$h_{f_{1\text{-}2}} = \frac{\bar{v}^2 n^2}{R^{\frac{4}{3}}} \qquad \ldots(4.54)$$

Should be applied only to "rough" conduits.

Refer to **Figure 4.8c** for n-values.

Table 4.6: Transition losses in pipe flows

Description		Sketch	k-value
Inlets $h_1 = \frac{k\bar{v}^2}{2g}$ ($\bar{v}$ = average velocity in conduit)	Protruding Oblique Blunt Well-rounded		0,9 0,7 0,5 0,2
Diverging sections $h_1 = \frac{k(\bar{v}_1 - \bar{v}_2)^2}{2g}$	Sudden Cone 45° < θ < 180° θ = 30° θ = 15°		1,0 1,0 0,7 0,2
Converging sections $h_1 = \frac{k\bar{v}^2}{2g}$	Sudden Cone		0,5 0,25
Bends $h_1 = \frac{k\bar{v}_2^2}{2g}$	θ = 90° θ = 45°		0,4 0,3
Outlets $h_1 = \frac{k\bar{v}_1^2}{2g}\left(1 - \frac{A_1}{A_2}\right)^2$	Sudden		1,0

4.3.5 Principle of momentum conservation

Since the principle of momentum conservation has limited application here, only the case in which forces at bends need to be determined is analysed, as shown in **Figure 4.16**.

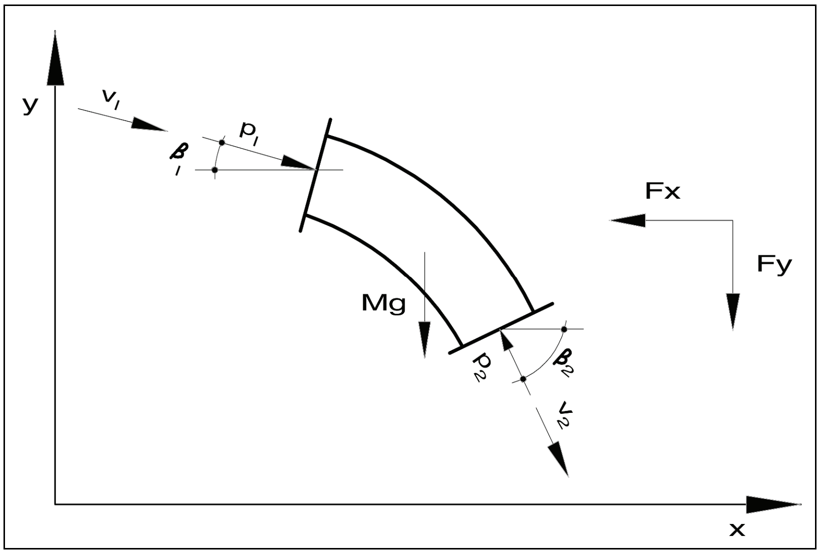


Figure 4.16: Forces acting on an isolated bend

Analytical procedure:

Calculate p_1 and p_2, the values of the intensities of pressure, by means of the Bernoulli equation. Take F_x and F_y as the components of the force exercised on the fluid by the bend (opposite of the force exercised on the bend by the fluid).

Compile equations:

x – direction: $$-F_x + p_1A_1\cos\beta_1 - p_2A_2\cos\beta_2 = \rho Q(\bar{v}_2\cos\beta_2 - \bar{v}_1\cos\beta_1) \quad \ldots(4.55)$$

y – direction: $$-p_1A_1\sin\beta_1 + p_2A_2\sin\beta_2 - Mg - F_y = \rho Q(-\bar{v}_2\sin\beta_2 + \bar{v}_1\sin\beta_1) \quad \ldots(4.56)$$

Where:

p_1 and p_2	=	intensities of pressure on either side of the bend (N/m^2)
A_1 and A_2	=	sectional areas on the upstream and downstream sides (m^2)
F_x and F_y	=	force components exerted by the solid boundary on the water (opposite and equal to the force exercised by the water on the solid boundary) (N)
β_1 and β_2	=	angles of direction (°)
ρ	=	mass density = 1 000 kg/m³ for water
Q	=	discharge (m³/s)
$\bar{v}_1$ and $\bar{v}_2$	=	upstream and downstream average velocities (m/s)
Mg	=	weight of enclosed fluid (N)

Solve F_x and F_y and invert to obtain the force exerted on the bend. (If horizontal forces are considered, the Mg term will, of course, fall away).

Unless the pipe itself and its joints are strong enough, anchors should take up the resulting force components. Concrete anchor blocks are often used for this purpose.

- F_y should be $\leq$ W = weight of the anchor block + weight of the pipe section
- F_x should be $\geq$ W$\tan\delta$ + passive resistance force, where W = weight of the anchor block + weight of the enclosed water volume + weight of the pipe section and δ = friction angle (degrees).

Where the surrounding soil could become saturated, W should be reduced by the weight of an equal volume of water.

4.4 WORKED EXAMPLES

4.4.1 Example 4.1 - Flow characterisation, energy gradient and normal depth

Problem description Example 4.1

The total discharge through a channel section is 477 m³/s. The dimensions and absolute roughness values for the channel are shown below (**Figure 4.17**).

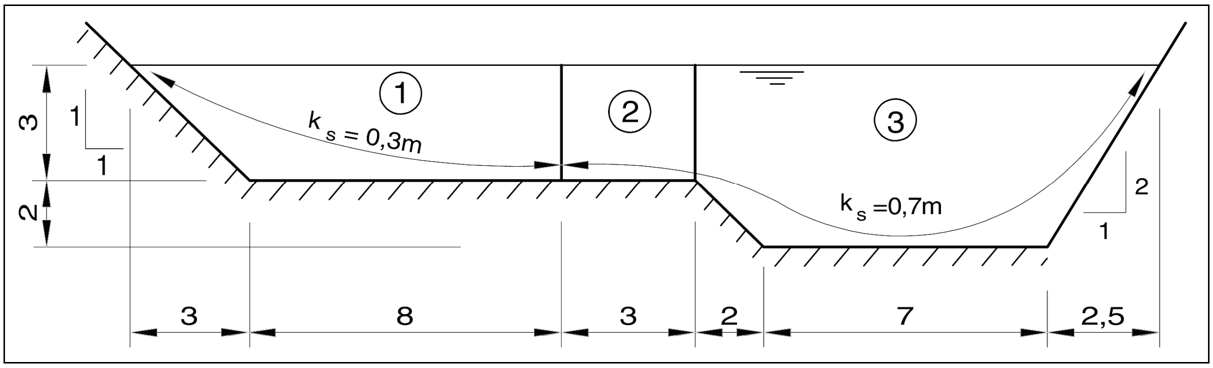


Figure 4.17: Cross-section of channel

Determine:

(i) The energy gradient (S_f).
(ii) Whether the flow is sub- or supercritical.
(iii) The average velocity through section 3.
(iv) Whether the flow is laminar or turbulent.
(v) The normal flow depth.

Solution Example 4.1

Divide the channel as shown above and derive the following details:

Parameter	Section		
	1	2	3
Area (A)	28,5 m²	9,0 m²	49,25 m²
Wetted perimeter (P)	12,24 m	3,0 m	15,42 m
Hydraulic radius (R = A/P)	2,33 m	3,0 m	3,19 m
Absolute roughness (k_s)	0,3 m	0,7 m	0,7 m
Chézy $C = 18 \log\left(\frac{12R}{k_s}\right)$	35,4 $m^{½}/s$	30,8 $m^{½}/s$	31,3 $m^{½}/s$

- **The energy gradient (S_f)**

By assuming uniform flow conditions the local slope of the channel, S_0, may be set equal to the energy slope, S_f.

Continuity of mass and energy (Chézy equations) combined provide the following relationships:

$$Q_{total} = \sum Q$$
$$Q_{total} = A_1 C_1 \sqrt{R_1 S_f} + A_2 C_2 \sqrt{R_2 S_f} + A_3 C_3 \sqrt{R_3 S_f}$$

$$\therefore Q_{total} = \left((28{,}5)(35{,}4)\sqrt{2{,}33} + (9{,}0)(30{,}8)\sqrt{3{,}0} + (49{,}25)(31{,}3)\sqrt{3{,}19}\right)\sqrt{S_f}$$

$$\therefore Q_{total} = (1540 + 480 + 2753)\sqrt{S_f}$$

$$\therefore S_f = \frac{Q_{total}^2}{4773^2} = 0{,}01 \text{ m/m (Energy gradient)}$$

- **Determine the flow regime**

$$(\text{Froude})^2 = \frac{Q^2 B}{gA^3} = \frac{(477)^2\,(25{,}5)}{(9{,}81)(86{,}75)^3} = Fr^2 = 0{,}91 < 1{,}0 \text{ thus subcritical}$$

- **Determine the average velocity through section 3**

$$\bar{v}_3 = \frac{Q_3}{A_3} = \frac{2753}{4773} \times \frac{477}{49{,}25} = 5{,}59 \text{ m/s}$$

- **Identify the flow type**

Calculate the Reynolds Number

$$R_e = \frac{vR}{\upsilon}$$

$$\bar{v} = \frac{\sum Q}{\sum A} = \frac{477}{28{,}5 + 9 + 49{,}25} = 5{,}50 \text{ m/s}$$

$$\bar{R} = \frac{\sum A}{\sum P} = \frac{86{,}75}{12{,}24 + 3{,}0 + 15{,}42} = 2{,}83 \text{ m}$$

υ = 1,14 x 10^{-6} m²/s (kinematic viscosity of water)

$$R_e = \frac{\bar{v}\bar{R}}{\upsilon} = \frac{(5{,}50)(2{,}83)}{1{,}14 \text{ x } 10^{-6}} = 13{,}65 \times 10^6$$

$R_e >> 2000 \;\therefore$ Highly turbulent

- **Calculation of normal flow depth**

The normal (uniform) flow depth for a given discharge is calculated by the same procedure, except that the flow depth is the unknown quantity and the energy gradient, S_f, is equal to the average (near constant) bed slope, S_0.

$$Q_{total} = 477{,}0 = A_1 C_1 \sqrt{R_1 S_f} + A_2 C_2 \sqrt{R_2 S_f} + A_3 C_3 \sqrt{R_3 S_f}$$

As shown above, the area, wetted perimeter and hydraulic radius can be written in terms of the unknown depth, Y. If Y is the depth in section 3, then the variables may be written as detailed on the next page.

Parameter	Section		
	1	2	3
Area (A) (m^2)	$\frac{1}{2}Y^2 + 6Y - 14$	$3(Y - 2)$	$\left(\frac{Y^2}{4} + 9Y - 2\right)$
Wetted perimeter (P) (m)	$\left(8 + \sqrt{2}(Y - 2)\right)$	3,0	$\left(\sqrt{8} + 7 + \sqrt{\frac{5}{4}}Y\right)$
Hydraulic radius (R = A/P) (m)	(A/P) *	$(Y - 2)$	(A/P) *
Absolute roughness (k_s) (m)	0,3	0,7	0,7
Chézy $C = 18\log\left(\frac{12R}{k_s}\right)$ ($m^{½}/s$)	*	*	*

Note:

* *The relationship was not documented, but could be obtained from the combination of the given relationships.*

With a known slope, S_0, and flow rate, Q, Y can be solved. In this case Y = 5 m.

4.4.2 Example 4.2 - Gradually varying river flow (backwater calculation – simple sectional details)

Problem description Example 4.2

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 below.

Q_{50} = 43,3 m³/s

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 43,3 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 the next table 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!

Results Example 4.2

			Calculation based on uniform flow assumption						
Section ID	**Position ID**	**Invert (m)**	**Slope (local) (m/m)**	**Y_n (m)**	**A (m^2)**	**P (m)**	**R (m)**	**Cal Q (m^3/s)**	**Fr**
a *	**b**	**c**	**d**	**e**	**f**	**g**	**h**	**i**	**j**
1	Downstream	3,02	0,003	2,258	23,750	23,649	1,004	43,3	0,463
2		3,24	0,006	2,109	19,017	19,554	0,973	43,3	0,606
3	Site	3,75	0,004	2,379	26,305	25,626	1,027	43,3	0,408
4		3,99	0,007	2,020	19,874	20,748	0,958	43,3	0,581
5	Upstream	4,42							

* Refer to the description of the table on the next page

From the above table it could be concluded (Fr << 1) that the flow will be subcritical and hence that the control will be downstream.

Now start with the assumption of a flow depth at section 1 (downstream) and work your way upwards by applying the continuity of energy. 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.

Section ID	**ΔX (m)**	**H total energy (m)**	**Fr**	**Area (m^2)**	**P (m)**	**Velocity (m/s)**	**E_1 (m)**	**Sf (m/m)**	**$(So\text{-}Sf)_{avg}$ (m/m)**	**$E_{2'}$ (m)**	**$\Delta(E_{2'}\text{-}E_1)$ (m)**	**Water level (m)**
a	**k**	**l**	**m**	**n**	**o**	**p**	**q**	**r**	**s**	**t**	**u**	**v**
1		2,427	0,463	23,75	16,10	1,82	5,447	0,002				5,278
	65								-0,0007	5,496	0,0000	
2		2,160	0,740	16,28	13,28	2,66	5,496	0,006				5,355
	82								-0,0030	5,740	-0,0000	
3		1,633	0,936	13,88	12,98	3,12	5,740	0,012				5,755
	67								-0,0057	6,125	-0,0000	
4		2,044	0,715	16,95	13,86	2,55	6,125	0,006				6,032
	66								-0,0061	6,526	-0,0003	
5		1,360	1,207	11,32	11,68	3,83	6,526	0,019				6,210

Notes: A brief description of the columns content for the above tables

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

4.4.3 Example 4.3 - Gradually varying river flow (backwater calculation)

Problem description Example 4.3 (Adapted, Example 5-4[4.2])

The discharge in a river with a single watercourse is 2 549 m³/s and the value of the Manning roughness coefficient is 0,033 s/m$^{1/3}$ for the whole area under consideration. The flow is subcritical (Fr < 1). Determine the depth of flow upstream and downstream of a bridge. The bridge is located between chainages 2,462 and 2,495 km and it is estimated that the secondary loss coefficient at the bridge, K_L = 0,18. Details of the cross-sectional characteristics (area and wetted perimeter for a given flow depth) were obtained from the surveyed data and are **not repeated** here.

Solution Example 4.3

In this example the solution will be based on the conservation of energy.

Firstly, a control should be found. By predetermination from the acting control (downstream for subcritical flow), it is found that the level of the water surface at chainage 1 819 m is 19,875 m. Since one is dealing with a single watercourse α = 1, and the calculations steps is detailed on the next page. Where flow spills over the banks and the velocity changes considerably over the section, the methods of calculation for multiple sections should be applied.

Sections are chosen just upstream and downstream of areas with large transition losses (e.g. bridges), and otherwise such that consecutive sectional areas do not differ by more than 40 per cent. The head loss of 0,116 m at the bridge may be calculated by means of the formulae in **Chapter 8**.

Column ID													
0	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
Cross-section number	**Chainage (m)**	**z (m)**	**A (m^2)**	$\frac{v^2}{2g}$ **(m)**	**H (m)**	**P (m)**	**R (m)**	S_f **(x 10^{-3})**	$\overline{S_f}$ **(x 10^{-3})**	**Δx (m)**	h_f **(m)**	h_ℓ **(m)**	**H (m)**
1	1 819	19,875	1 110	0,269	20,144	189,9	5,845	0,545	-	-	-		-
2	2 060	19,963	994	0,336	20,299	156,7	6,343	0,610	0,578	241	0,139		20,283
	2 060	19,946	991	0,338	20,284	156,7	6,324	0,616	0,581	241	0,140		20,284
3	2 462	20,239	715	0,648	20,887	152,4	4,692	1,762	1,189	402	0,478		20,762
	2 462	20,114	692	0,692	20,806	151,5	4,568	1,950	1,283	402	0,516		20,800
Bridge 4	2 495	20,230	715	0,648	20,878	152,4	4,692	1,762				0,116	-
5	2 704	20,730	827	0,485	21,215	167,9	4,926	1,234	1,490	209	0,313		21,191
	2 704	20,706	822	0,491	21,197	167,6	4,905	1,257	1,509	209	0,315		21,193

Column ID	Description of the column contents
0	Cross-section number
1	Chainage
2	Water surface elevation (estimated)
3	Cross-sectional flow area
4	Kinetic energy head
5	Total energy head = 2 + 4 (estimated
6	Wetted perimeter
7	Hydraulic radius
8	Local energy gradient
9	Average energy gradient between sections
10	Distance increment
11	Friction head loss
12	Transition head loss
13	Calculated energy head = previous head + losses

4.4.4 Example 4.4 – Negligible energy losses (converging flow over short distance)

Problem description Example 4.4

A concrete chute with a stream width of 0,6 m conveys water down the side of an embankment 3,0 m high with a slope of 1,5 vertical to 1,0 horizontal. The discharge is 0,1 m³/s and the water flows away from a trough in the road profile.

Calculate the flow velocity, depth of flow and Froude number at the toe.

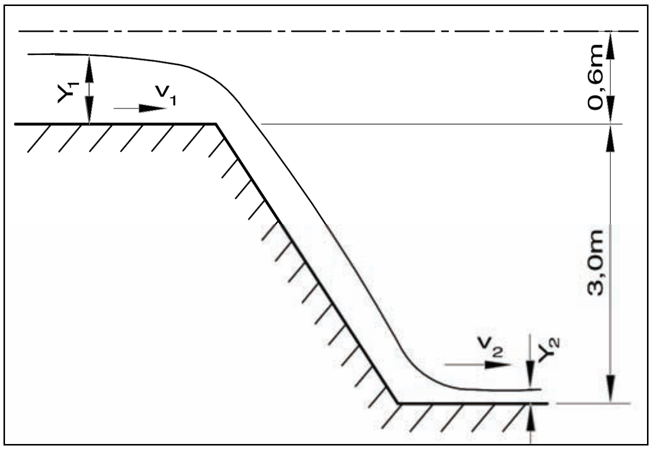


Figure 4.18: Concrete chute down embankment

Solution Example 4.4

Since the water has to dam in order to run off, a control is created at the upper end (Fr = 1)

$$\therefore \frac{Q^2 B}{g A_c^3} = 1$$

With:

Q	=	0,1 m³/s
B	=	0,6 m
A_c	=	$B_c y_c = 0{,}6 y_c$
g	=	9,81 m/s²
y_c	=	0,14 m
$\bar{v}_c$	=	1,190 m/s

and $E_c = y_c + \frac{\bar{v}_c^2}{2g} = 0{,}14 + \frac{(1{,}190)^2}{2(9{,}81)} = 0{,}212 \text{ m}$

Since the channel is very steep, the energy losses will be small in relation to the change in level.

Consequently $H = 3{,}0 + 0{,}212 = y_2 + \frac{\bar{v}_2^2}{2g}$ can be assumed.

$q = \bar{v}_2 y_2 = 0{,}167 \text{ m}^3\text{/sm}$

$\bar{v}_2$ = 7,9 m/s; y_2 = 0,021 m and $Fr_2 = \frac{\bar{v}_2}{\sqrt{g y_2}} = 17$

The actual velocity will be slightly lower. (If H>> y_2, then $\bar{v}_2 \approx \sqrt{2gH}$)

4.4.5 Example 4.5 – Transition losses

Problem description Example 4.5

The normal (uniform) flow depth in a long 2 m wide, rectangular canal is 2 m and the normal flow velocity 2 m/s. The Manning n-value is 0,02 s/m$^{1/3}$. There is a 90° bend with a centre-line radius of 7 m. Calculate the Froude number for uniform flow conditions.

Solution Example 4.5

Calculate the flow depths just upstream and just downstream of the 90° bend.

$$Fr_n = \frac{\bar{v}_n}{\sqrt{gy_n}} = \frac{2{,}0}{\sqrt{(9{,}81)(2{,}0)}} = 0{,}452$$ (based on the normal flow depth)

$\therefore$ Downstream control and hence the depth just downstream of the bend will be 2 m.

Energy head loss through bend: $h_\ell = \frac{2B}{r_c}.\frac{\bar{v}^2}{2g} = \frac{(2{,}0)(2{,}0)(2{,}0)^2}{(7{,}0)(2)(9{,}81)} = 0{,}1165 \text{ m}$

Energy equation:

Upstream energy head: $H_1 = H_2 + h_1 = \frac{\bar{v}^2}{2g} + y_2 + h_1$

$$H_1 = \frac{(2{,}0)^2}{(2)(9{,}81)} + 2{,}0 + 0{,}1165 = 2{,}321 \text{ m}$$

$$\therefore \quad y_1 + \frac{\bar{v}_1^2}{2g} = 2{,}321 \text{ m}$$

Write the velocity in terms of the upstream flow depth (y_1) using continuity:

$$\bar{v}_1 = \frac{Q}{A_1} = \frac{(2{,}0)(2{,}0+2{,}0)}{(y_1)(2{,}0)} = \frac{4{,}0}{y_1}$$

$\therefore \quad y_1 + \frac{\left(\frac{4{,}0}{y_1}\right)^2}{(2)(9{,}81)} = 2{,}321 \text{ m}$. Solving the only unknown term - y_1

y_1 = 2,143 m (depth upstream of bend)

4.4.6 Example 4.6 – Identification of acting controls

Problem description Example 4.6

Water flows across a 16 m wide road. The road has cross-falls of 2%.

Calculate the discharge per unit width that would flow across the road when the adjacent level rises 0,5 m above the shoulder, see **Figure 4.19** on the next page.

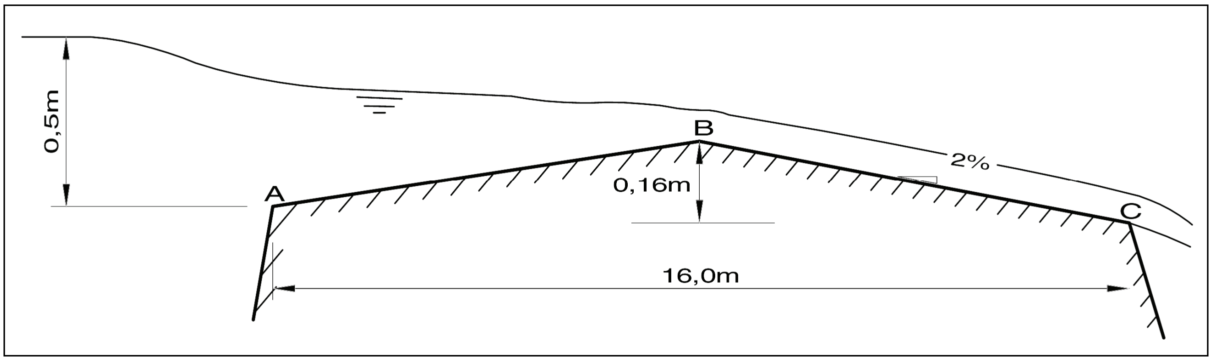


Figure 4.19: Flow across the road

Solution Example 4.6

Assume that the control (point of release) occurs at the crown (B). If there are no energy losses between A and B, the specific energy at the top of the crown will be 0,5 – 0,16 = 0,34 m and the critical depth y_c = 0,227 m (two thirds of the specific energy).

The corresponding discharge per unit width:

$$q = \overline{v}_c y_c = \sqrt{g} y_c^{\frac{3}{2}} = 0{,}338 \text{ m}^3/\text{s.m}$$

Assume that the actual discharge is 0,3 m³/s.m

$$y_c = \sqrt[3]{\frac{q^2}{g}} = 0{,}209 \text{ m} \qquad \text{and} \qquad E_c = \frac{3}{2} y_c = 0{,}314 \text{ m}$$

With this discharge, the normal flow depth (Manning) will be given by:

$$q = \frac{y^{\frac{5}{3}} S^{\frac{1}{2}}}{n}$$

With a Manning n-value = 0,013 s/m$^{1/3}$

$$0{,}3 = \frac{y^{\frac{5}{3}} (0{,}02)^{\frac{1}{2}}}{0{,}013}$$

$$y_n = 0{,}116 \text{ m} < y_c = 0{,}209 \text{ m}$$

The slope of 2% is thus hydraulically steep and the control is indeed at B. If y_n was found to be greater than y_c this would mean that the control was at C, and the depth there would be y_c, from which point calculations would then progress upstream.

Because the depths of flow are small, one should test to see whether the flow is indeed turbulent.

$$Re = \frac{\overline{v}y}{\upsilon} = \frac{0{,}3}{1{,}14 \times 10^{-6}} = 2{,}6 \times 10^5 >> 2000$$, indicating the flow is turbulent.

Now determine the depth y_A at A.

$$0{,}5 = y_A + \frac{\overline{v}_A^2}{2g} + 0{,}35 \frac{\overline{v}_A^2}{2g}$$ (Energy equation with provision for transition losses)

and $y_A \overline{v}_A = 0{,}3$ (continuity)

Thus y_A = 0,472 m

The depth varies from 0,209 to 0,472 m and since the cross-sectional areas differ by more than 40%, more than one increment should be used. Use three increments; i.e. depths of: 0,209; 0,297; 0,384 and 0,472 m (see **Figure 4.20**).

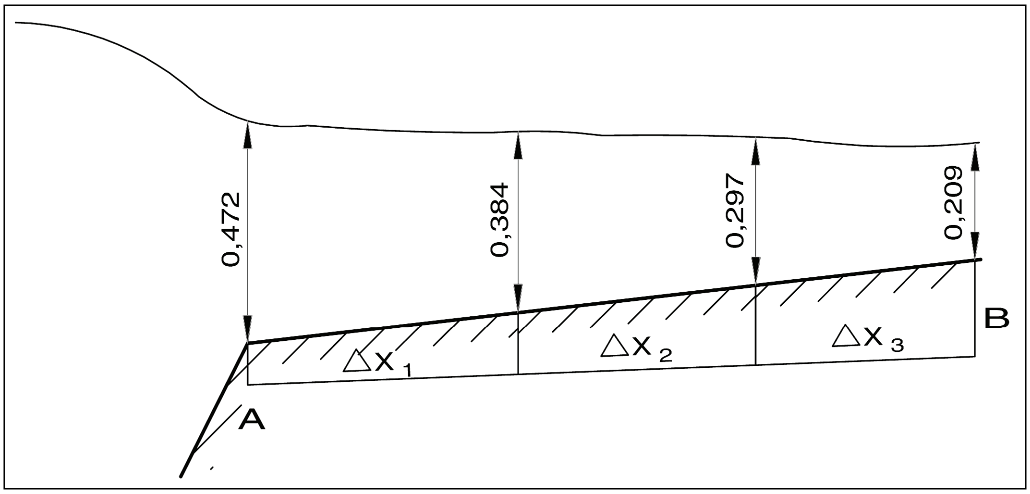


Figure 4.20: Increments of flow depth across the road

$$\frac{dE}{\Delta x} = S_o - \bar{S}_f \quad \text{(Energy equation for prismatic channels)}$$

$$\Delta x = \frac{dE}{S_o - \bar{S}_f}$$

$$dE = \left(y_1 + \frac{\bar{v}_1^2}{2g} \right) - \left(y_2 + \frac{\bar{v}_2^2}{2g} \right)$$

S_o = -0,02 m/m (uphill)

$$S_f = \frac{\bar{v}^2 n^2}{y^{\frac{4}{3}}}$$

y (m)	E (m)	R (m)	S_f (m/m)	$S_{f(average)}$ (m/m)	Δx (m)
0,209	0,314		0,002810		
		0,035		0,001840	1,60
0,297	0,349		0,000870		
		0,066		0,000620	3,20
0,384	0,415		0,000370		
		0,078		0,000278	3,85
0,472	0,493		0,000186		

ΣΔx = 8,65 m

ΣΔx = 8,65 > 8,0 thus the discharge per unit width should be less (i.e. q should be smaller).
Choose smaller q and repeat until ΣΔx ≈ 8,0 m for a more accurate answer.

4.4.7 Example 4.7 – Pipe flow

Problem description Example 4.7

Water flows from a submerged catch pit at a kerb inlet through a 20 m concrete drain pipe where it releases into a river stream. The inside diameter of the pipe is 0,3 m. The pipe has a slope of 0,5 % and the depth of the water in the catch pit above the top of the pipe is 0,2 m (i.e. available head).

Calculate the discharge rate for this set-up.

Solution Example 4.7

To solve this problem the conservation of energy principle will be used i.e. equation 4.49 (repeated below). The entrance (point 1) and exit (point 2), are open to atmosphere and thus $p_1/\gamma = p_2/\gamma = 0$ if the stream line is selected along the top of the water surface.

$$\frac{\overline{v}_1^2}{2g} + \frac{p_1}{\gamma} + z_1 = \frac{\overline{v}_2^2}{2g} + \frac{p_2}{\gamma} + z_2 + h_{f_{1-2}} + h_{\iota_{1-2}}$$

The velocity in the catch pit ($\overline{v}_1$) is negligible (large catch pit area in comparison with pipe area). If the datum line is selected through the invert of the pipe outlet (point 2) z_2 will be zero and z_1 can be calculated as follows:

$$z_1 = (0,2) + (20)(0,005) = 0,3$$

The first term is the height above the pipe inlet and the second term the length of the pipe multiplied by the slope of the pipe to obtain the vertical height difference between inlet and outlet.

The last two terms in the energy equation are the loss terms. The friction loss can be calculated using the Chezy equation (Equation 4.50) with the C-value determined by Equation 4.53. The absolute roughness value of the pipe can be obtained from **Table 4.10b,** k_s = 0,0004 m.

$$C = 5,75\sqrt{g}\,\log\left(\frac{12R}{k_s}\right) \quad \text{or} \quad C = 5,75\sqrt{g}\,\log\left(\frac{12\left(\frac{D}{4}\right)}{k_s}\right) \text{ for circular pipes.}$$

$$C = 5,75\sqrt{9,81}\,\log\left(\frac{12\left(\frac{(0,3)}{4}\right)}{0,0004}\right) = 60,37$$

$$h_{f_{1\text{-}2}} = \frac{\overline{v}^2 L}{C^2 R}$$

$$h_{f_{1\text{-}2}} = \frac{\overline{v}_2^2\,(20)}{(60,37)^2\left(\frac{0,3}{4}\right)} = 0,07317\overline{v}_2^2$$

A secondary inlet loss will occur at the entrance. From **Table 4.6,** an inlet coefficient of k = 0,5 will be used for a blunt entrance. The diameter does not change and the velocity where the water enters the pipe can be assumed to be equal to $\overline{v}_2$.

$$h_1 = \frac{k\overline{v}_2^2}{2g} = \frac{(0{,}5)\,\overline{v}_2^2}{2(9{,}81)} = 0{,}02548\overline{v}_2^2$$

The energy equation is simplified as follows:

$$\frac{(0)^2}{2(9{,}81)} + 0 + (0{,}3) = \frac{\overline{v}_2^2}{2(9{,}81)} + 0 + 0 + \left(0{,}07317\overline{v}_2^2\right) + \left(0{,}02548\overline{v}_2^2\right)$$

$\overline{v}_2 = 1{,}416$ m/s

$$Q = \overline{v}_2 A_2 = (1{,}416)\left(\pi\left(\frac{0{,}3}{2}\right)^2\right) = 0{,}1\ m^3/s$$

This pipe can discharge 100 l/s if it is allowed to dam at the entrance up to 500 mm above the pipe inlet level.

4.5 SUPPORTING SOFTWARE (HYDRAULIC CALCULATIONS)

The theoretical overview presented in this chapter forms the basis for a number of typical problems that can be solved with the use of the supporting software. Typical problems, which have been included in the software, include:

- Determination of the flow regime for a given cross-section and discharge
- Transfer of forces onto a partially obstructed flow path
- Calculation of the normal flow depth for a simple or complex cross-section
- Calculation of friction and secondary losses
- Analysis of diverging and converging flows
- Estimation of the friction parameter.

4.6 REFERENCES

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4.3 Mockmore, C.E. (1944). *Flow Round Bends in Stable Channels.* Trans Am Soc Civ Engrs Vol 109.

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CHAPTER 5 - SURFACE DRAINAGE

A Rooseboom

5.1 INTRODUCTION

Surface road drainage systems serve to transfer storm water flows that collect on and along road structures to limit the risk to:

- road users;
- road structures; and
- the environment.

This chapter deals with the design of components of surface drainage systems, with the exception of culverts, low-level crossings and bridges, which are dealt with in separate chapters. **Table 5.1**, **Road Map 5**, reflects the aspects that are covered in this chapter. Sub-surface drainage is essentially different from surface drainage and is therefore also covered in a separate chapter.

Surface road drainage design commences with the consideration of local precipitation that runs off across a road surface in a thin stream. It ends where each accumulated stream, impacted upon by roadworks, may be released into an existing natural or man-made waterway without any increased risk of erosion damage or flooding further downstream.

Acceptable risk in surface road drainage is often expressed in terms of water surface levels not to be exceeded during flood events with given return periods.

5.2 RUN-OFF ON ROADWAYS AND SHOULDERS

5.2.1 General

The accumulation of run-off on roadways and shoulders constitutes the most common hydraulic road surface risk. Dangerous conditions arise when the pressure within the water contained between a vehicle's wheel and the road surface is equal to the pressure exerted by the wheel. As the water pressure increases, the wheel increasingly loses contact with the road surface, and control over the vehicle diminishes. This condition is known as hydroplaning [5.1].

In the design of roads, the flow depth of surface run-off across road surfaces is not a primary design variable, because the normal geometric standards would generally ensure that this depth remains within acceptable limits. It is nevertheless important for the designer to be conversant with the variables involved in the assessment of the hydroplaning risk. The most important factors which need to be considered are:

- vehicle speed
- depth of flow
- type of road surface
- wheel load
- type of tyre and
- tyre pressure.

Each of these aspects is briefly discussed hereafter.

Table 5.1: Road Map for surface drainage

ROAD MAP 5				
Typical theme		**Input information**	**Hand calculation**	
Topic	**Par.**		**Problem**	**Example**
Run-off from roads and shoulders	5.2	Road surface, depth of flow and vehicle speed	Flow depth on road surface	5.1
Kerbs, berms and outlets	5.3	Cross-sectional parameters, hydraulic description and applicable formulae, flow regime	Flow depth in side channel	5.2
Discharge channels	5.4	Freeboard, permissible slopes for erosion protection, slope, roughness and flow rate	Drop grid capacity for subcritical flow	5.3
			Erosion potential	5.5
Discharge chutes	5.5	Flow rate, profile detail and orientation	Dissipation of energy on steep slopes	Design procedure discussed in the text
Erosion protection on steep slopes	5.7	Flow rate, erosion potential of material and slope detail	Select appropriate protection options	Guide indicates the use of plant bundles

5.2.2 Vehicle speed

There is an appropriate design speed for each type of road. This design speed represents the expected vehicle speed at which the risk will be acceptable. Under wet conditions the safe vehicle speed will reduce due to possible hydroplaning. Hydroplaning rarely occurs if the vehicle speed is lower than 80 km/h.

5.2.3 Depth of flow

Depth of flow is the primary variable in the analysis of run-off on roadways and shoulders. The following norms are recommended for surface flow on travelled lanes:

- The flow depth during a 1:5-year storm should not exceed 6 mm.
- The minimum slope along the flow path should be 2%. In the case of wide road surfaces, the normal cross-fall may be increased to 2,5%.

It is not possible to apply these guidelines strictly in every case, e.g. where there is a change in super-elevation.

The flow depths on the road surface for different slopes and rainfall intensities can be obtained from **Figure 5.1**.

5.2.4 Road surface

Road surfaces are given a rough finish to improve road holding and skid resistance without causing excessive tyre wear and road noise.

Figure 5.1 is based on a road surface of average roughness.

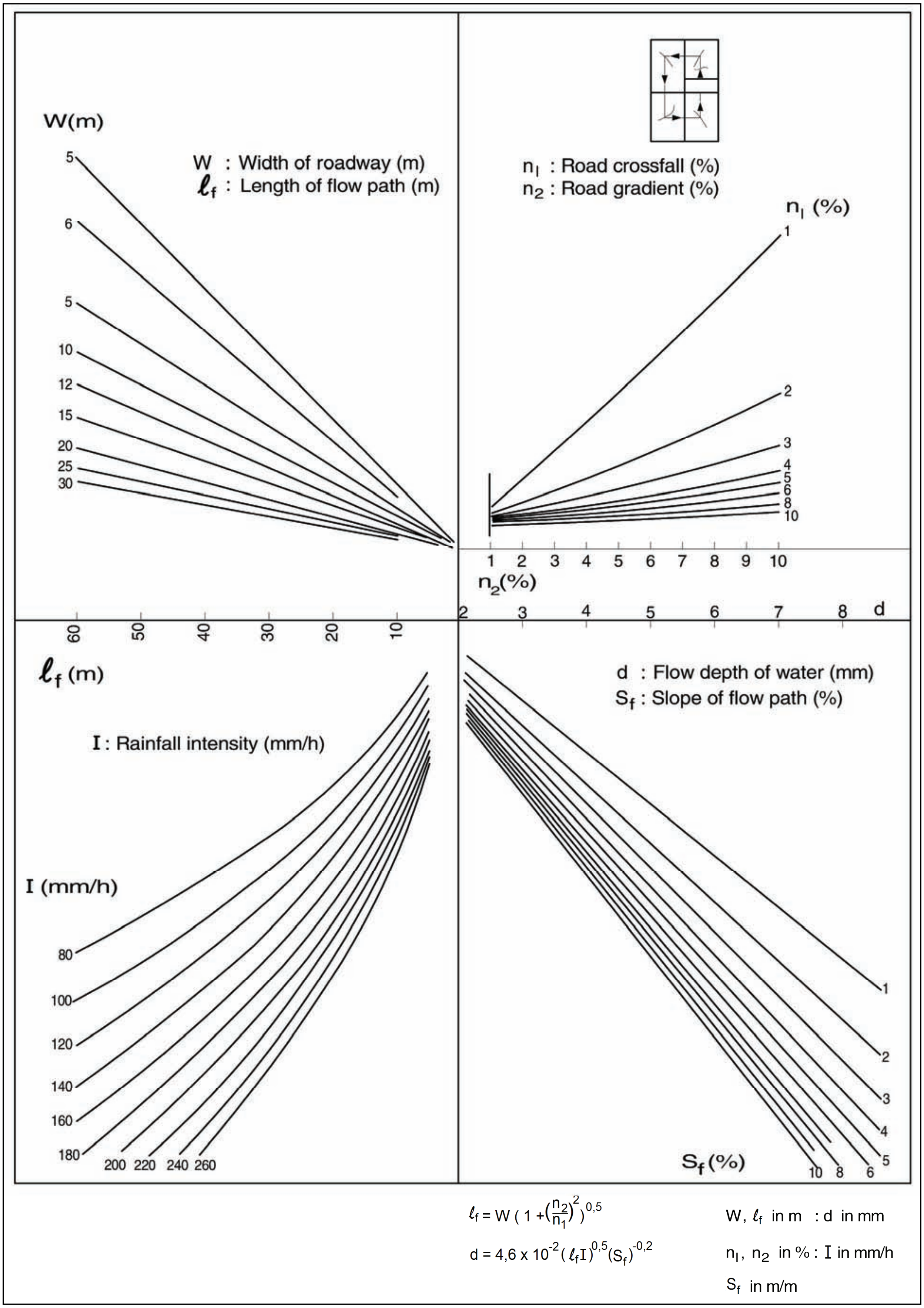


Figure 5.1: Depth of sheet flow on road surface (Laminar flow conditions assumed)

5.2.5 Wheel load, tyre type and tyre pressure

Wheel load, the type of tyre and the type of pressure also affect hydroplaning. However due to the wide variations possible and it not being practical to relate different speed limits to different types of tyre, these aspects are ignored, and only the geometric road elements are considered using a typical wheel load, tyre type and tyre pressure.

5.2.6 Further requirements regarding surface drainage

In the design of any drainage element it is essential to consider connected elements and to determine whether such elements will influence each other. Road surface run-off is allowed to discharge freely over the side of an embankment only where the erosion potential is low.

Where the erosion potential is significant, run-off is allowed to accumulate alongside berms or kerbs along the side of a road to be released at regular enough intervals so that the maximum permissible water levels are not exceeded.

In the case of flow over roadways and shoulders, maximum water levels linked to return periods should be determined for flow along berms and side channels. The normal requirements for freeways are shown schematically in **Figure 5.2**.

5.3 KERBS, BERMS AND OUTLETS

5.3.1 General

The purposes of kerbs, berms and outlets are to collect surface run-off and to discharge flows at specific points in a controlled manner to limit traffic risk, as well as to protect erodible areas such as embankment slopes.

5.3.2 Use of kerbs, berms and outlets

Kerbs are rarely found along freeways because they are expensive and may be dangerous to traffic where there are no guardrails. At cross-roads and junctions, as well as on bridges kerbing is used to channel traffic, to prevent vehicles from crossing sidewalks and to form storm water channels with capacities greater than those created by berms.

A berm is a small ridge placed at the top of an embankment to prevent erosion by run-off down the side of the embankment. Berms may be temporary (earth) or permanent (asphalt or concrete) and are normally used under the following conditions:

- If surface water could flow down an embankment and cause erosion;
- If the road gradient > 0,5%; and
- If fills are more than 3 m high.

To ensure traffic safety, permanent berms are normally only used together with guardrails. This combination should be aligned so that the wheel of a vehicle cannot be caught between the rail and the berm.

Outlets in berms and kerbs should be spaced to ensure that:

- the requirements with regard to the permissible flow depths in **Figure 5.2** are satisfied;
- adequate freeboard is allowed along each kerb or berm;
- deep, rapid flow along the road shoulder is prevented;
- intermediate outlets intercept at least 80% of oncoming flows, and lowest outlets may accommodate 100%;

- the total cost of the combination of outlets plus discharge chutes (lengths are important) is kept to a minimum;
- unnecessary concentration of water is prevented. As a rule of thumb the following spacings should not be exceeded between outlets:
  - side channels and median drains - 200 m
  - channels above cuttings - 300 m

It is important that the drainage system should always be considered as a whole; for example the placing of an outlet above a culvert means lower expenditure on the discharge chute and exit structure.

It is also important that the shape and layout of kerbs, berms and outlets are hydraulically correct to ensure satisfactory operation of drainage systems.

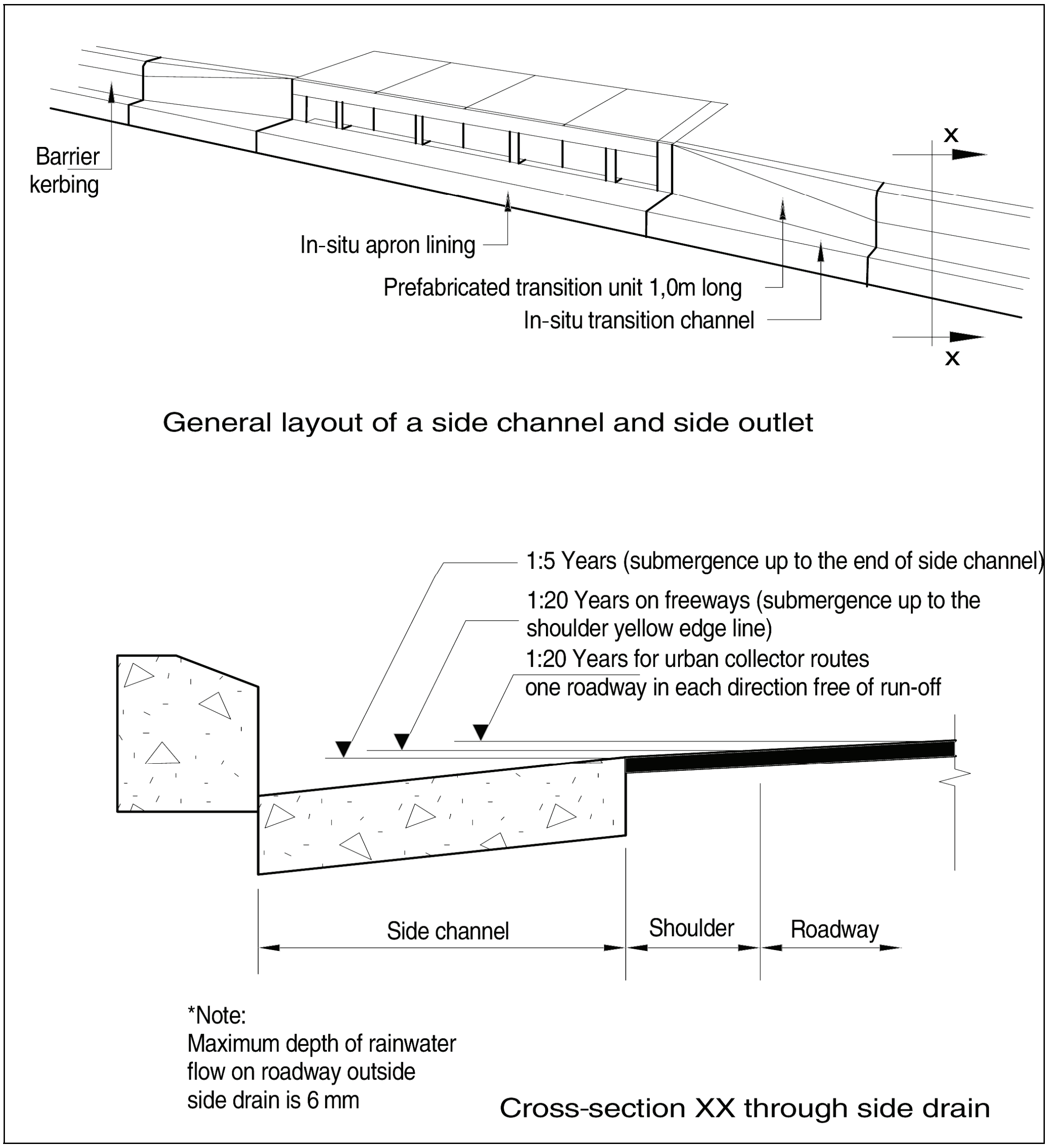


Figure 5.2: Maximum permissible depths of flow alongside freeways

5.3.3 Hydraulics of kerb and berm flows

Kerb and berm flows are open channel type flows as described in **Chapter 4**. The limits in terms of maximum depths of flow are shown in **Figure 5.2**.

The known variables are usually the cross-section of the kerb or berm, the road crossfall and the longitudinal slope. The unknown variables are the maximum capacity of the kerb or berm and the catchment that would yield this maximum discharge.

Cross-sectional details of some of the typical kerbs are provided in **Figure 5.3**. For these typical cross-sections, the nomogram in **Figure 5.4** could be used for determining the hydraulic discharge capacities through triangular channel sections. The catchments that would yield these discharges could then be determined through hydrological calculations, e.g. Rational method or other procedures as discussed in **Chapter 3**.

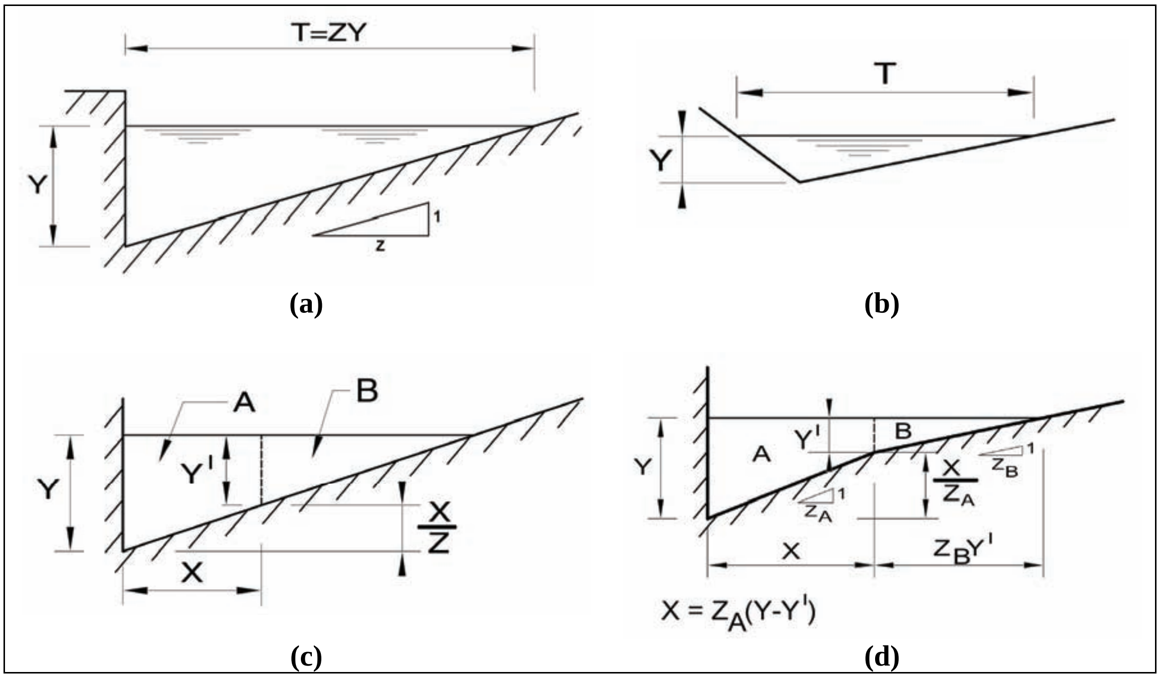


Figure 5.3: Typical triangular cross-sectional details of kerbs

The nomogram in **Figure 5.4** is used as follows:

- Connect the values of the Z/n Ratio (*n* represents Manning's roughness parameter – **Figure 4.8**) and the slope with a straight line.
- Use the intersection of this line with the turning line to determine the discharge, Q for a given flow depth y, or alternatively determine the flow depth y for a given discharge Q.

By adding and subtracting the discharges for triangular sub-areas, the discharges for composite sections can also be calculated (**Worked example 5.2**).

In the case of a triangular channel section (**Figure 5.3(b)**) the value of Z = T/Y.

5.3.4 Kerb and berm outlets

Outlets along kerbs and berms are divided into two main groups: drop outlets and side outlets. Two types of side outlets are identified, giving a total of three types of outlets that differ in terms of hydraulic design (**Table 5.2**). The hydraulics of outlets is covered in the sections that follow.

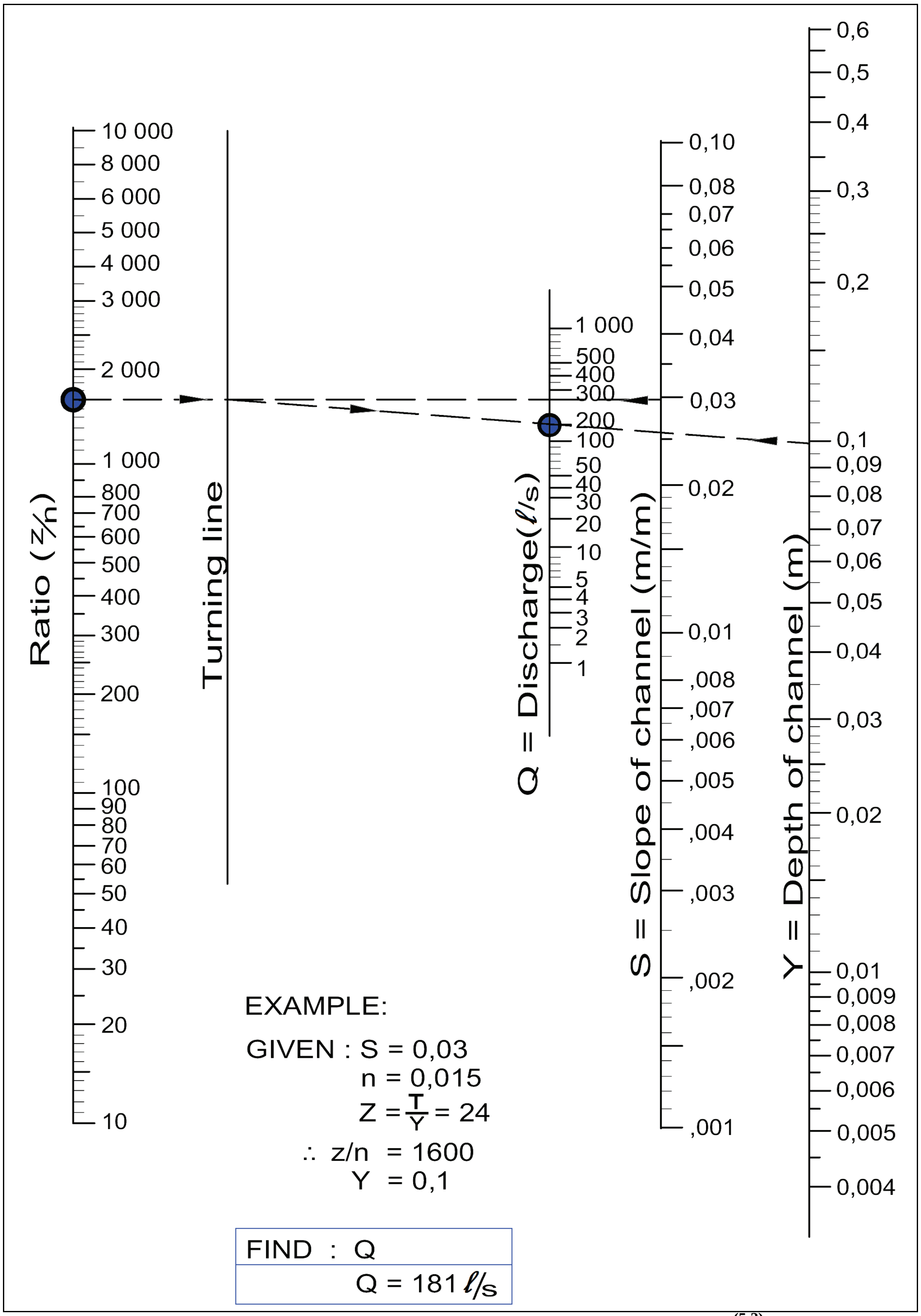
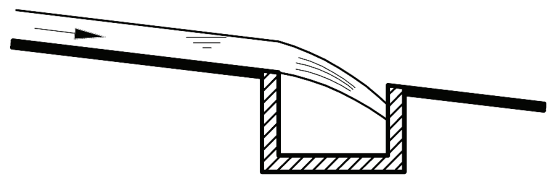
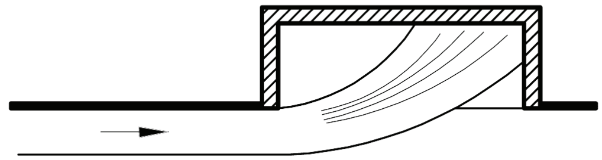
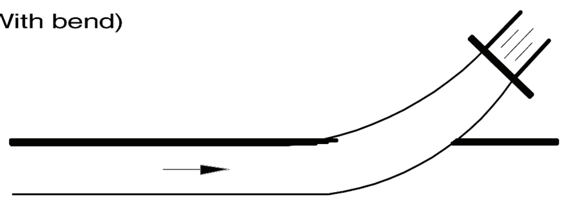


Figure 5.4: Nomogram for flow in triangular channels (5.2)

Table 5.2: Some physical details of the outlet types

Type	Remarks
I Drop outlet  Long Section	Special provision to be made for high oncoming velocities
II Side outlet (Overspill section)  Plan	Not practical to discharge full oncoming flows at high velocities
III Side outlet (With bend)  Plan	Radius of bend dependent on change in direction, cross fall and Froude-number of oncoming flow

Drop outlets (Type I) feed oncoming flows downwards into underground drainage systems.

Side outlets with overspill sections (Type II) discharge oncoming flows sideways (past berms or kerbs) into collector channels and/or chutes.

Side outlets with bends (Type III) steer oncoming flows past kerbs or berms to be released some distance away where erosion can be limited.

Photographs 5.1 and **5.2** show typical kerb inlets.

Photograph 5.1: Detail of a typical kerb inlet with a transition (side outlet)

Photograph 5.2: Kerb inlet with deposited debris (side outlet)

5.3.5 Hydraulic design of drop outlets (grid outlets)

5.3.5.1 General

Drop outlets (Type I outlets) discharge storm water downwards into an underground drainage system and the outlets are made safe for pedestrians and traffic by placing grids over the openings. Such outlets are thus also known as grid outlets (**Photograph 5.3**).

Drop outlets function under the following flow conditions:

Subcritical approach flows	- free or submerged outflows
Supercritical approach flows	- free outflows.

To determine whether the oncoming flow is subcritical or supercritical, the Froude number (Fr) needs to be calculated, as was indicated in **Section 4.2.2**.

The discharge Q is calculated by means of hydrological methods (**Chapter 3**) and the flow depth for a given uniform flow section could be calculated by using the Manning or Chézy equation.

5.3.5.2 Subcritical approach flows

Outflows may be either free-flow or drowned.

Free outflow conditions: These conditions are reflected in **Figure 5.5** and may be analysed by applying the broad-crested weir formula as described below.

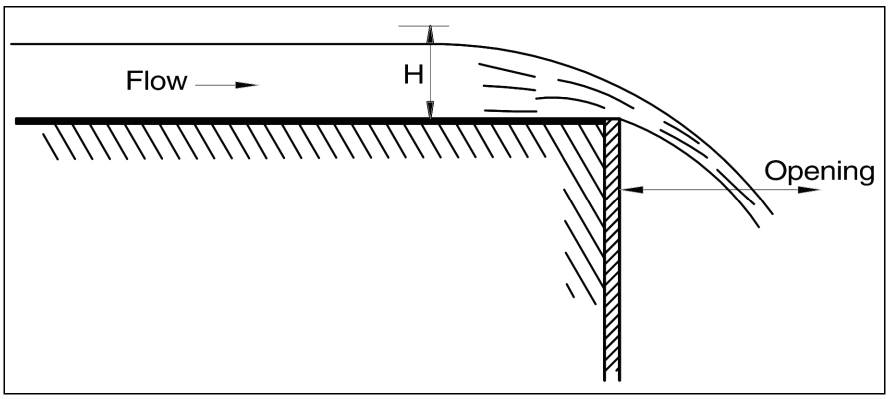


Figure 5.5: Section through drop outlet with a free outflow and subcritical approach flow conditions

Broad-crested weir formula:

$$Q = C_D bH\sqrt{gH} \qquad \ldots(5.1)$$

$$Q = 1{,}7bH^{3/2} \qquad \ldots(5.2)$$

This is a theoretical relationship for critical conditions along the crest and a contraction coefficient, C_D of 0,6, where:

Q	=	flow rate (m³/s)
C_D	=	discharge coefficient
B	=	total flow width (m)
H	=	energy head ≈ flow depth for upstream conditions (m)

With allowance for sideways contraction of oncoming flows the formula becomes:

$$Q = 1{,}45bH^{\frac{3}{2}} \qquad \ldots(5.3)$$

Subcritical submerged flow conditions: Such conditions are analysed by applying the orifice formula (also refer to **Figure 5.8**).

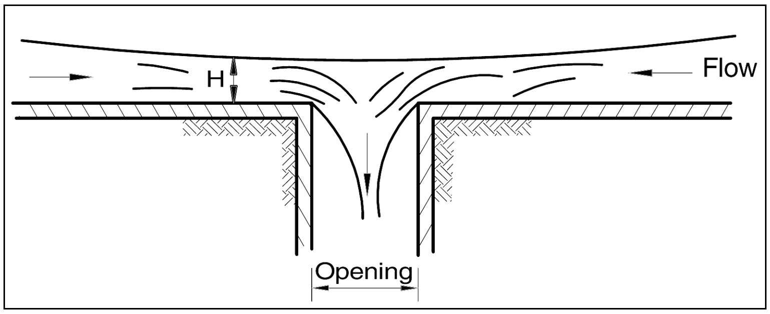


Figure 5.6: Section through outlet: Drowned conditions

$$Q=CFA\sqrt{2gH} \qquad \ldots(5.4)$$

where:

C = inlet coefficient (0,6 for sharp edges or 0,8 for rounded edges)
F = blockage factor (say 0,5)
A = effective cross-sectional plan area of the opening (m^2)
H = total energy head above grid (m)

Supercritical approach (also see **Figure 5.9):** These conditions are dealt with basically by providing an opening in the direction of flow of at least the same area as the sectional area of the oncoming stream.

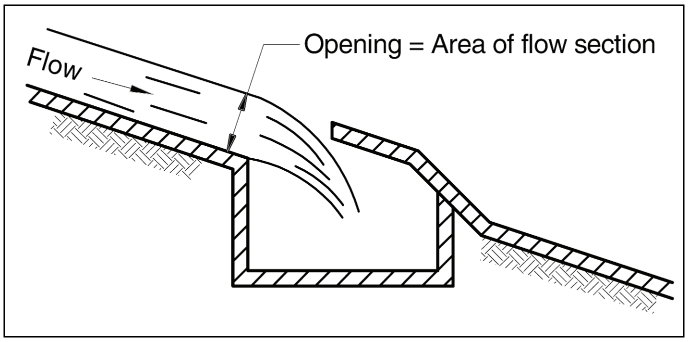


Figure 5.7: Drop outlet for supercritical approach flow

In practice, dealing with supercritical approach flows is not always simple, especially if a grid has to be placed across the flow or a change in direction needs to be accommodated. Furthermore, the grid is not always as wide as the channel. The higher the Froude number of the oncoming flow, and the greater the change in direction that this flow has to undergo, the greater the deviation from the simplified approach shown in **Figure 5.7**. For example **Figure 5.9** shows that for Froude numbers > 5 the effective opening should be larger than the section of the approaching stream.

Figure 5.8 (subcritical) and **Figure 5.9** (supercritical) were prepared for the design of grid inlets with sub- and supercritical approach flows respectively. The following should be borne in mind:

Under conditions of subcritical flow towards a drop outlet:

- **Figure 5.8** applies to subcritical oncoming flows and horizontal grids.
- The direction of the bars relative to the flow direction of the oncoming water is not very important if flow velocities are low. However, with high velocities the bars should be placed in the flow direction.
- In the case of the quoted broad-crested weir formula no blockage factor is included.

When supercritical flow conditions prevail, **Figure 5.9** (based on physical modelling) should be used to determine the discharge capacity and the following aspects need to be kept in mind:

- The graph to the left of **Figure 5.9** was developed for supercritical oncoming flows as a dimensionless Fr : $\frac{100d}{D}$ graph. (The discharge Q for a d-value is determined via the Manning or Chezy equation.)

- The graph with discharges given in *l/s* only applies to standard 900 mm x 900 mm grids.

- The layout is such that the bed slope of the channel increases to 1:10 just upstream of the grid, to increase the specific energy of the flow without changing the overall slope of the channel.

- The experimental grid was square.

- The experimental graph was converted into a Q:s graph for a specific grid and channel shape. **When changes of direction occur, or if the flow section is changed significantly, the graph in Figure 5.9 should not be used**.

- No blockage factor was taken into account.

- In the case of supercritical flow it is very important for **grid bars to be placed in the direction of flow**.

NOTE: *In all cases it should be emphasised that a problem should be viewed as a whole; for example, in the case of supercritical flow through an outlet that is, say, 50% blocked, and where flow is contained by a berm, damming could occur. This damming could then change the conditions from supercritical to subcritical.*

5.3.6 Hydraulic design of side outlets

Side outlets (Types II and III) discharge stormwater into drainage systems above or below ground level. Typical side outlets are berm outlets, kerb outlets and outlets at the lowest points of side channels.

Hydraulically, side outlets are not as effective as drop outlets, especially for the discharge of supercritical flow. These outlets are in general use, however, and have practical advantages such as low blockage risks. In addition, in some cases they are cheaper than drop outlets.

Subcritical flow (Type II)

The calculations to be performed for different flow conditions are indicated below:

- Subcritical flow conditions can be analysed approximately by applying the broad-crested weir formula, provided that oncoming velocities are low.
- However, the use of **Figures 5.11** and **5.12** is recommended, since these are based on experimental results and were re-calibrated [5.3] after full-scale tests.

Supercritical flow (Type II or Type III)

A side outlet for supercritical flow may be one of two types, i.e. (see **Table 5.2**):

Type II - Energy is dissipated in a collecting trough and changes in flow direction are brought about where subcritical conditions occur (**Figure 5.10**).

Type III - The direction of flow is changed by means of a gradual horizontal curve so that water does not spill over the side of the channel.

The left half of **Figure 5.11** provides depths of flow alongside berms for a 2% cross-slope. The depths of flow thus obtained on the vertical axis can be transposed to the right hand figure to obtain the minimum length of side opening required to pass either 80% or 100% of oncoming flow over the side.

It is generally economical to allow some 20% of the oncoming flow to pass if there is a lower outlet further along the kerb or berm. A lowest outlet should accommodate all the flow, and no water flowing alongside the kerb or berm may flow over the road, ramp or crossroad, nor flow freely over the embankment.

The radii recommended for Type III outlets are shown in **Figure 5.14**. This type of outlet is typically found at the end of a cutting. A drop outlet or stilling basin may be used to finish off this type of outlet to limit erosion downstream.

The most **important differences between Type II and Type III outlets are** as follows:

- Type II is often hydraulically less effective than Type III.
- The design of a Type II outlet is less critical because direction change occurs under subcritical conditions.
- In the case of embankments where space is limited, it is easier to accommodate Type II outlets.

Type II outlets should be used on high embankments, and either Type II or III may be used in other positions such as at the end of a cutting or on a low embankment. The design of discharge chutes and pipes is discussed in Sections 5.5 and 5.6.3. Since outlet water velocities tend to be high in both cases, the necessary erosion protection measures should be considered downstream.

5.3.7 Kerb inlets with transitions

Figure 5.12 may be used to design any type of side outlet where a minimum length of overspill is required as in the case of box inlets. Box inlets that are strong enough to withstand traffic wheel loads are expensive and hence less costly transitions were developed. These transitions are to be used in conjunction with shorter box sections to provide the long overspill lengths which are required for discharging water on steep slopes (**Figure 5.13**). Grobler [(5.3)] has expanded on earlier work [(5.4,5.5,5.6)] and has improved on the accuracy of earlier results (**Figure 5.11**) by performing full-scale tests. A single diagram (**Figure 5.12**), which can be used to design outlets with transitions, was developed [(5.3)]. It was established that the capacities of Type II outlets are not sensitive to variations in cross-slope, and **Figures 5.11** and **5.12** may be used for a wide range of conditions.

Figure 5.13 contains detailed drawings of the Pretoria-type outlets, which have been widely used. **Photographs 5.3** and **5.4** show some grid inlets.

Photograph 5.3: Concrete side drain with grid inlet

Photograph 5.4: View of a combined grid inlet and kerb inlet

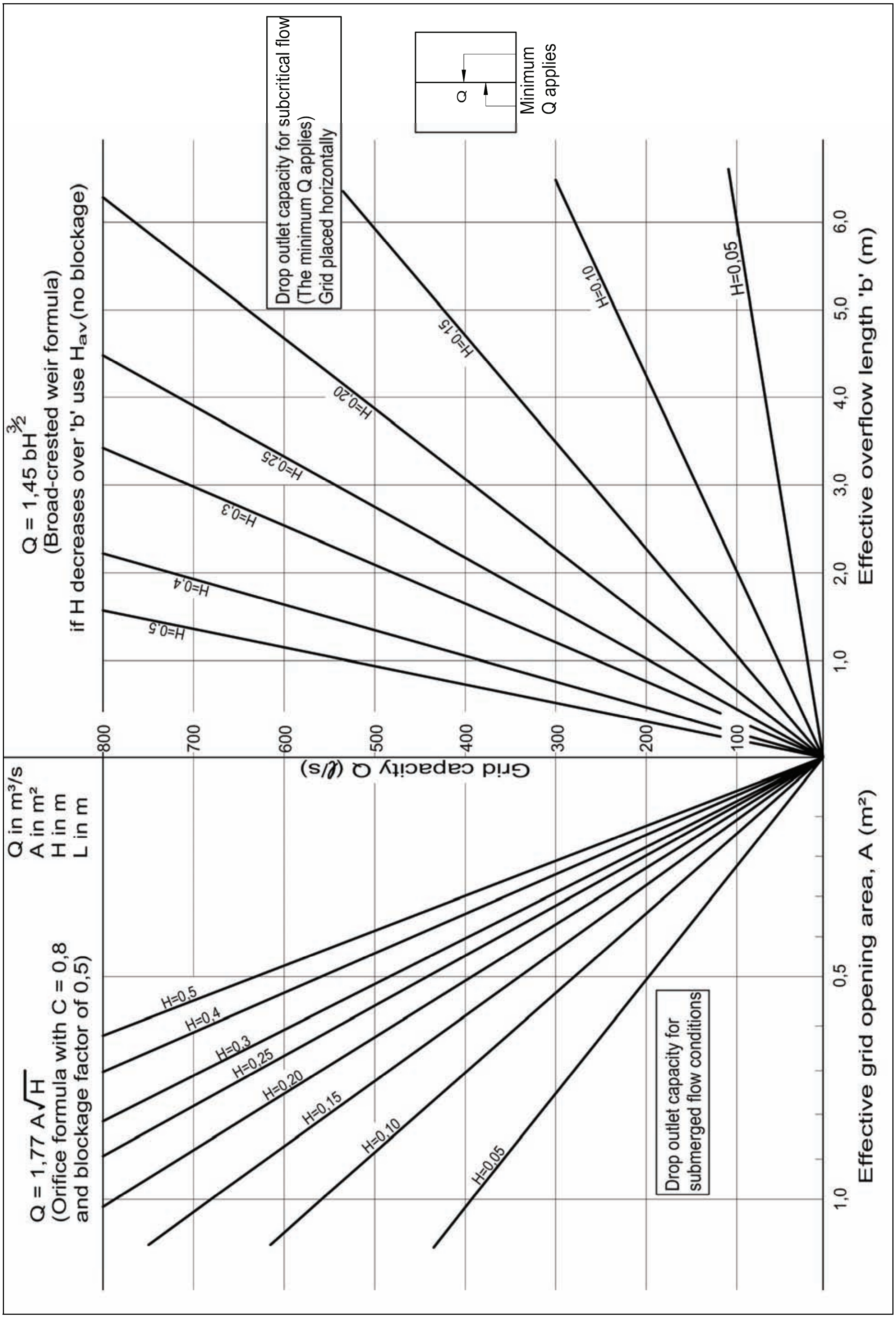


Figure 5.8: Drop outlet: Capacity for subcritical approach flow

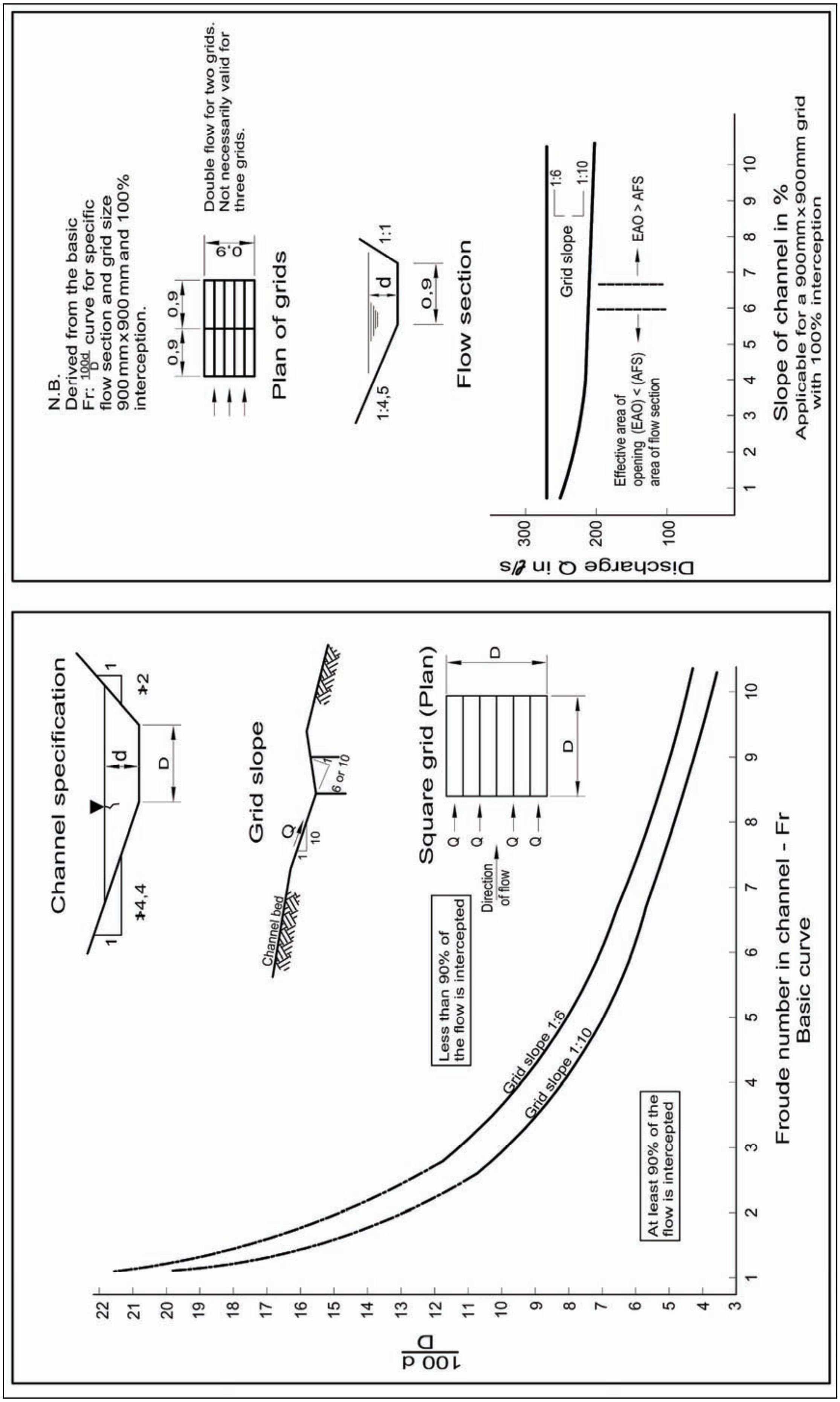


Figure 5.9: Grid capacities for supercritical flow

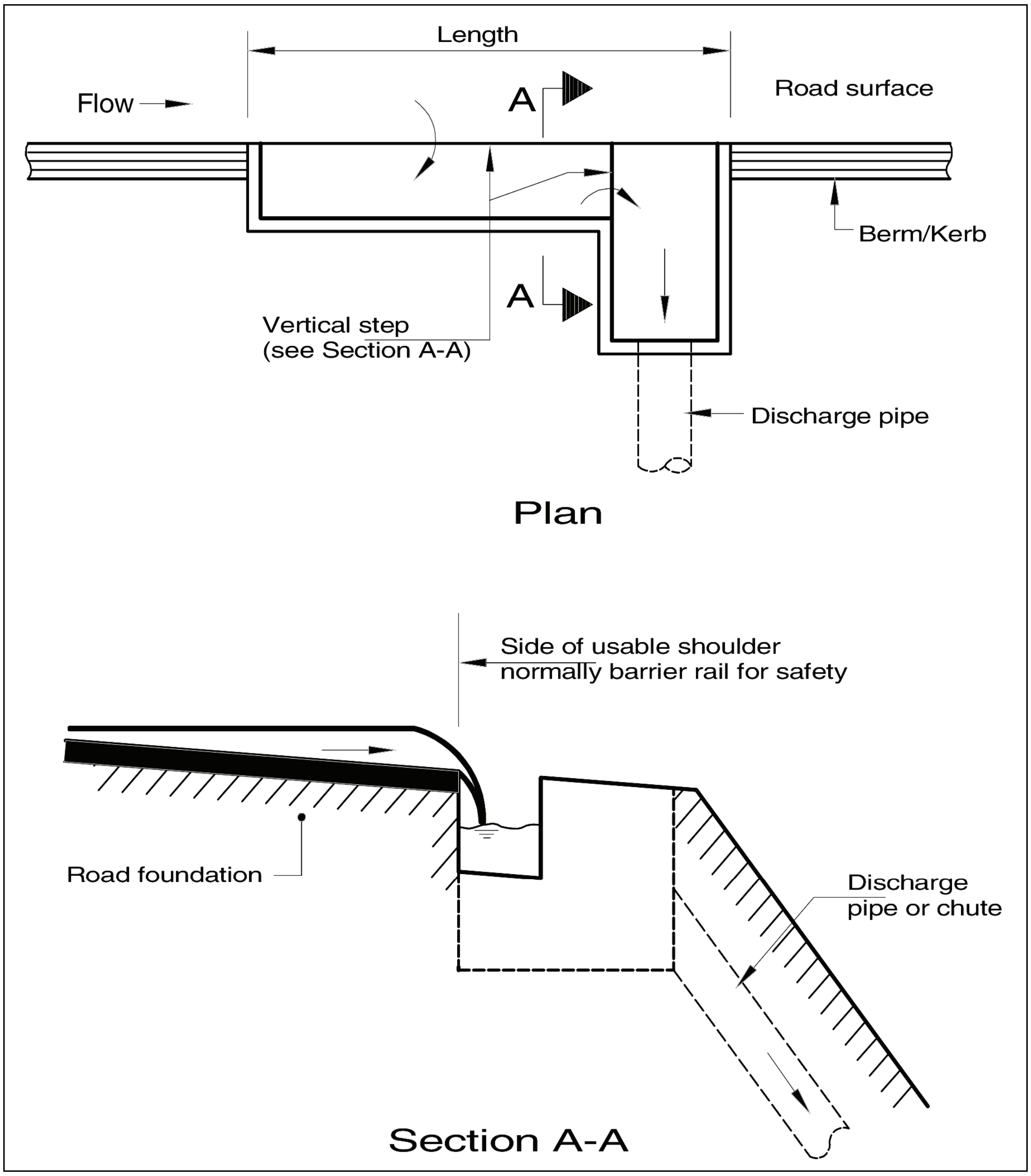


Figure 5.10: Details of Type II side outlets

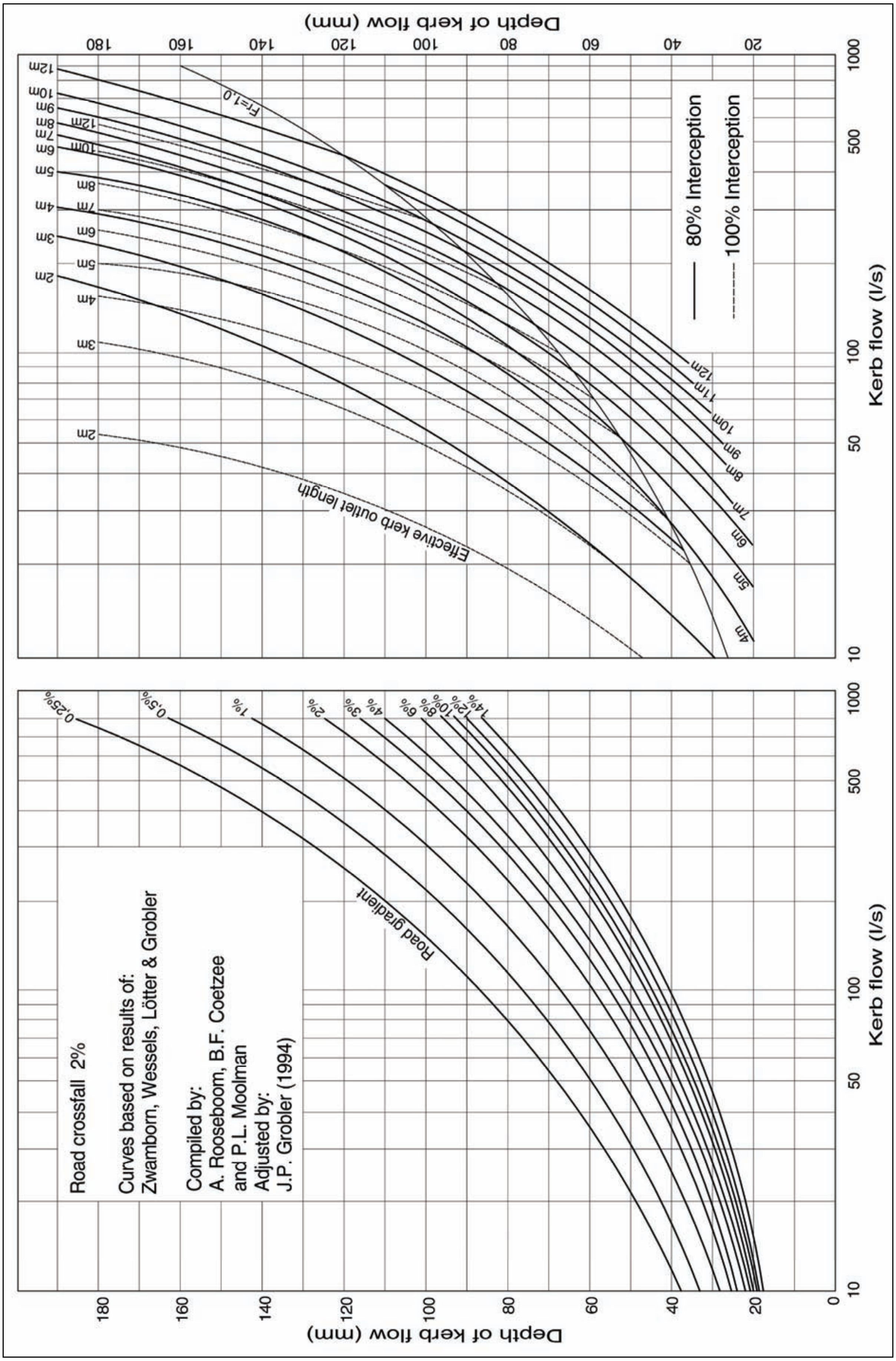
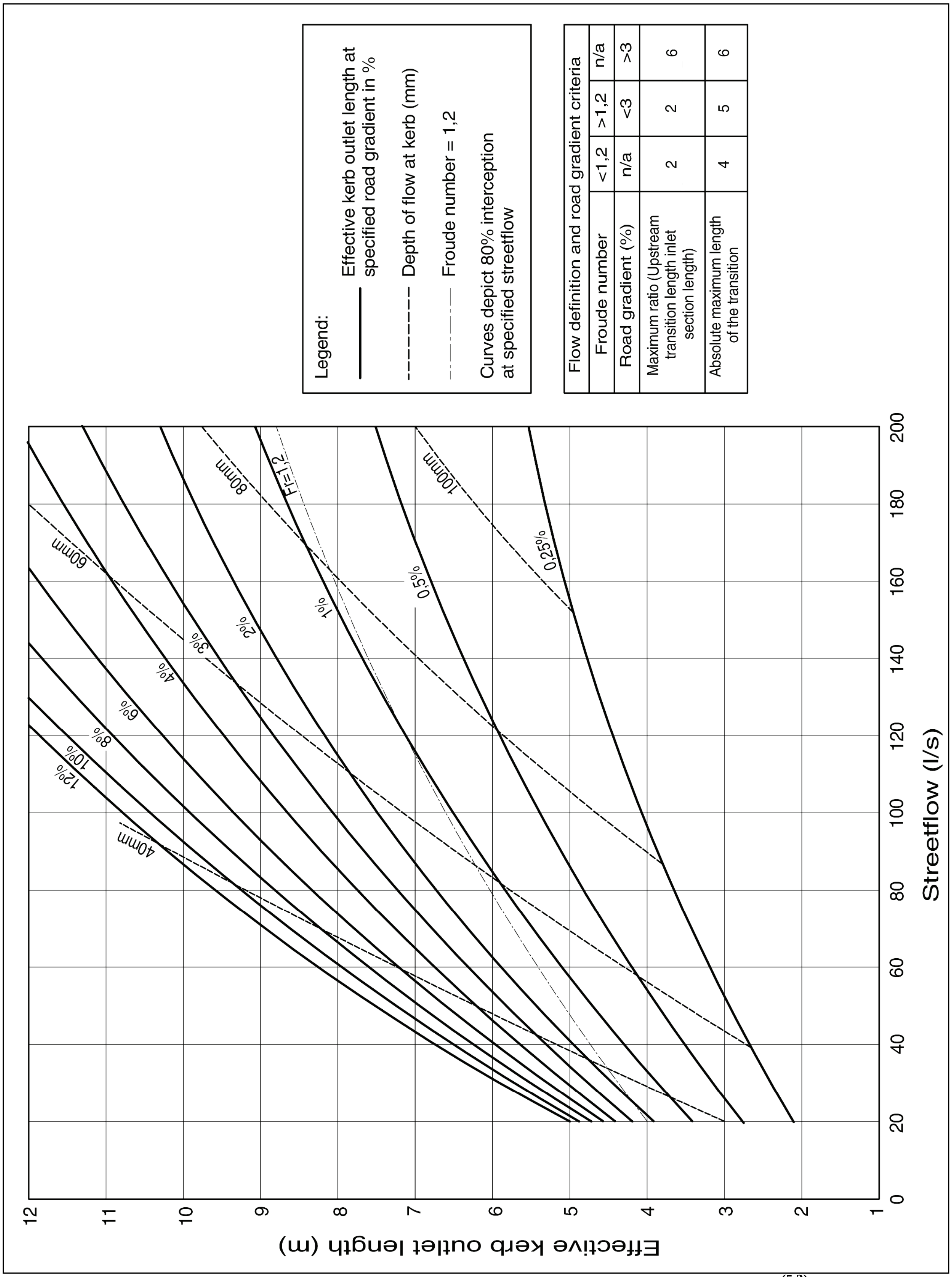


Figure 5.11: Required total side outlet lengths for Type II kerb outlet



Flow definition and road gradient criteria			
Froude number	<1,2	>1,2	n/a
Road gradient (%)	n/a	<3	>3
Maximum ratio (Upstream transition length inlet section length)	2	2	6
Absolute maximum length of the transition	4	5	6

Figure 5.12: Design curves for Type II kerb outlets with transitions (5.3)

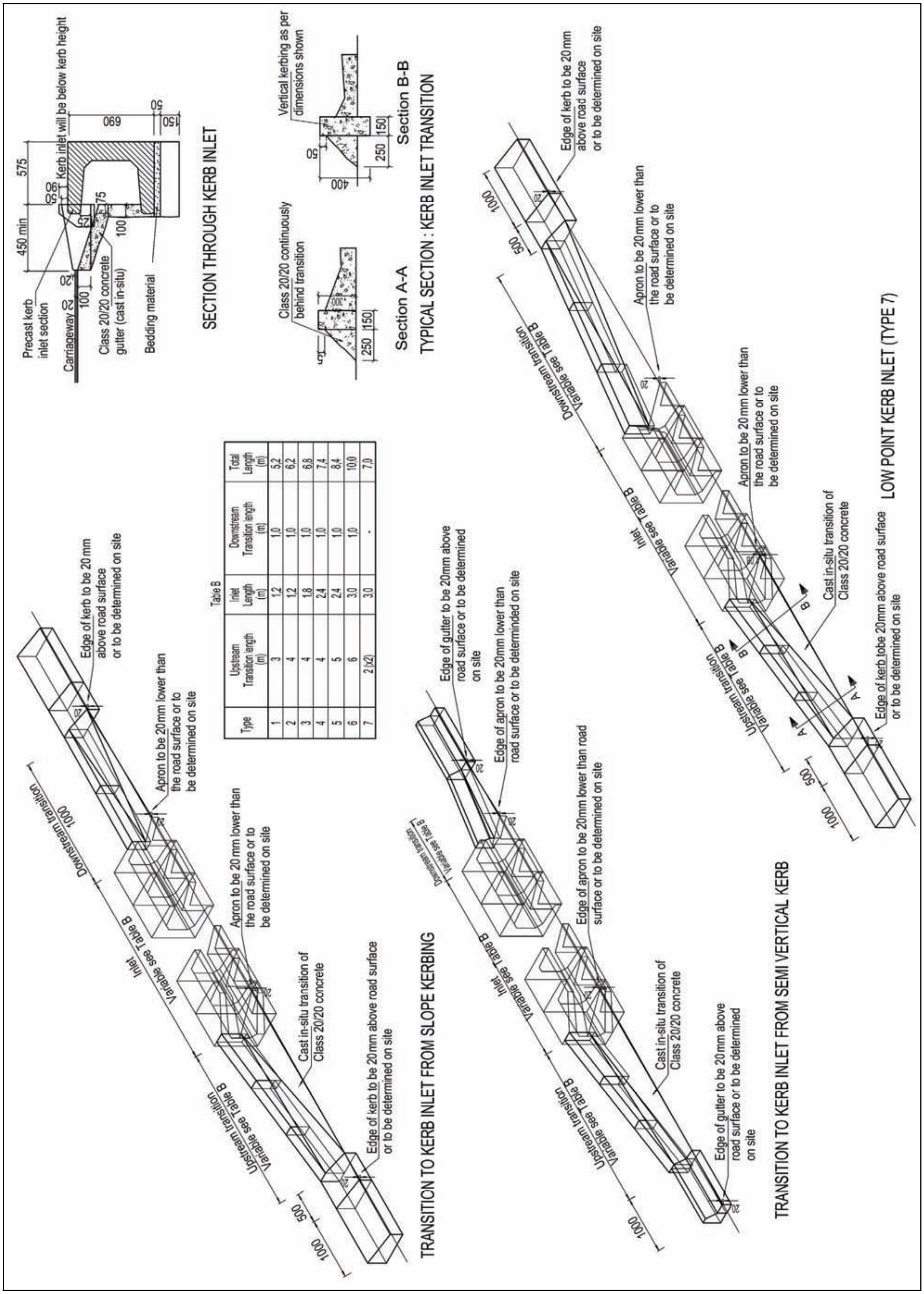


Figure 5.13: Pretoria Type II outlets

(This standard drawing is available in electronic format on the supporting CD)

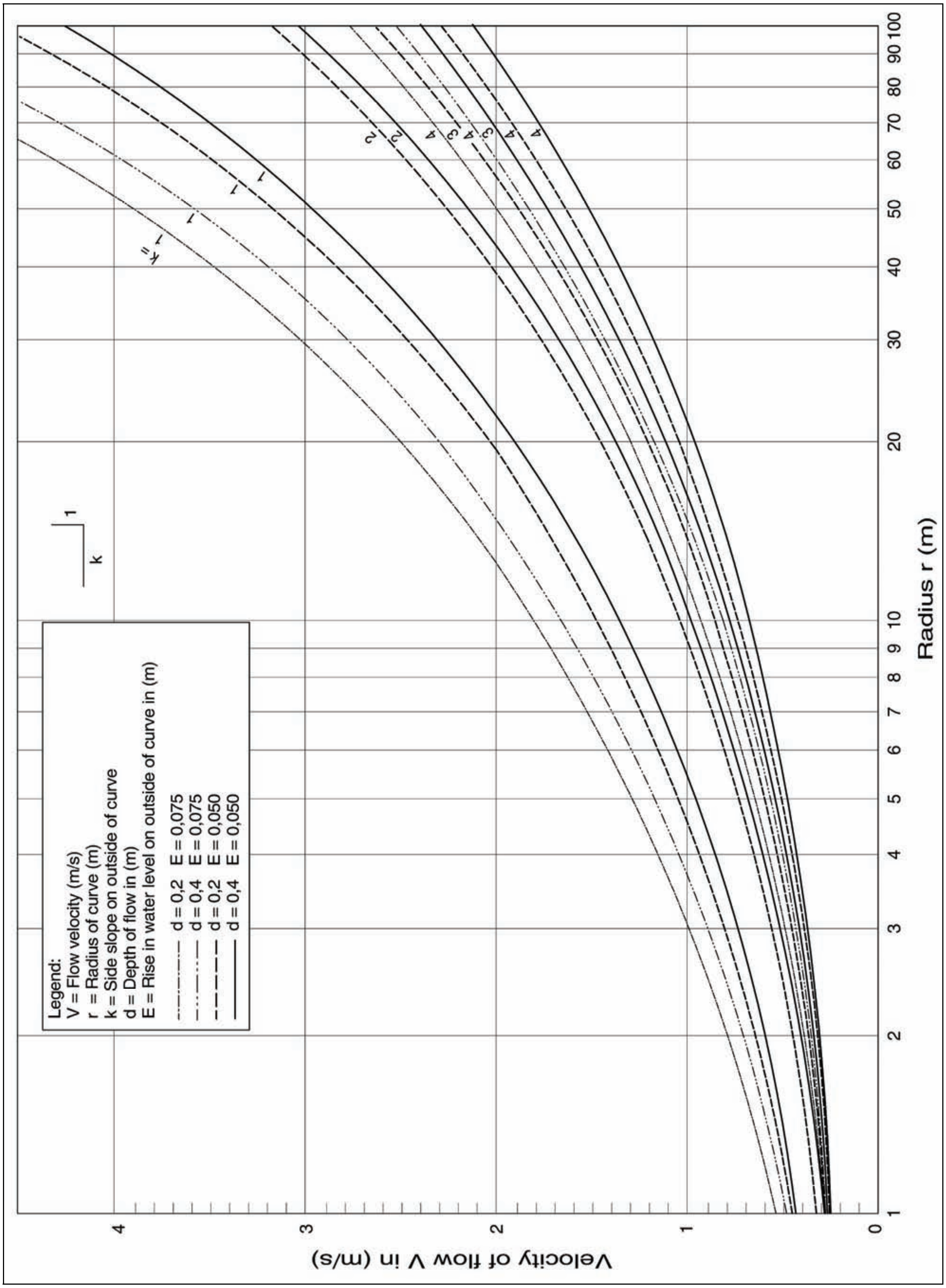


Figure 5.14: Maximum radii for Type III kerb outlets

5.4 DISCHARGE CHANNELS

5.4.1 Introduction

Discharge channels are open waterways, typically with longitudinal slopes of less than 10%. Such channels are excavated canals or natural gullies, and are often protected against scour by grassing, concrete linings, stone pitching, and so on.

It is sometimes necessary to use linings even if scour is not a problem, e.g. in the case of culvert outlets where lush growth of vegetation may obstruct the outflow. When it is important for channels to retain the same position and depth after cleaning, and where sedimentation is a problem, lining should be considered.

Although, from theoretical hydraulics and considerations of erosion, the ideal channel section would have an approximately parabolic form, other considerations generally result in the use of trapezoidal sections, except where the width has to be restricted, in which case rectangular sections are often used.

To calculate the capacity of a channel, the freeboard as well as permissible slopes and velocities should be determined.

5.4.2 Freeboard and provision for wave action

Where there are no other restrictions, the following minimum values for freeboard (5.7) are recommended for road drainage:

Table 5.3: Freeboard: Straight portions of channels

Channel Section	Fr < 1	Fr > 1
Rectangular	0,15 E	0,25 y
Trapezoidal	0,20 E	0,30 y

where: Fr = Froude number = $\sqrt{\dfrac{Q^2B}{gA^3}}$ …(5.5)

E = specific energy (m)

$$E = y + \frac{\overline{v}^2}{2g} \quad \text{(m)} \qquad \text{…(5.6)}$$

y = depth of flow at deepest point (m)

$\overline{v}$ = average velocity (m/s)

If the Froude number lies between about 0,8 and l,2, flow conditions are unstable and both the normal depth of flow and the freeboard height should be calculated conservatively.

In addition to the freeboard indicated above, the following additional freeboard heights should be provided at curves to allow for surging and wave action:

Table 5.4: Additional freeboard around bends

Channel section	Fr < 1	Fr > 1
Rectangular	$\frac{3\bar{v}^2 b}{4\,gr}$	$\frac{1{,}2\bar{v}^2 b}{gr}$
Trapezoidal	$\frac{\bar{v}^2(b+2Ky)}{2(gr-2K\bar{v}^2)}$	$\frac{\bar{v}^2(b+2Ky)}{gr-2K\bar{v}^2}$

Where:

$\bar{v}$ = average velocity in straight portion of channel (m/s)
b = width of channel bed (m)
r = centreline radius of channel (r should not be smaller than three times the width at the water surface) (m)
K = co-tangent of side slope (K = 0 for rectangular channel sections)
y = depth of flow in straight portion of channel (m)

In the case of subcritical flow (Fr < 1), the additional freeboard would be required at the outside of the curve only, but for supercritical flow (Fr > 1) it is necessary on both the inside and the outside.

Where stormwater canals flow at full capacity only rarely, it has become acceptable practice in South Africa to design these canals without freeboard, provided that barriers are provided alongside the canals. These barriers are placed perpendicular to the edges of the canal, and they prevent erosion gulleys from forming alongside the edges of the canal when intermittent spillage occurs over the edges because of wave action.

5.4.3 Permissible slopes of drains

The slopes of unpaved median drains may not be less than 0,5%, and those of paved drains not less than 0,25% (5.8). On very small longitudinal slopes the local slope of the median drain may be made greater than that of the road by the use of a longitudinal saw-tooth pattern and the draining away of water at the low points.

Acceptable maximum side slopes for unlined channels are reflected in **Table 5.5**.

Table 5.5: Acceptable maximum side slopes for unlined channels

Material type	Maximum slide slope (Vertical : Horizontal)
Rock	Almost vertical
Stiff clay, soil with concrete lining	1 : 1 up to 1: 2
Soil with stone pitching, large earth channels	1 : 1
Firm clay or small earth channels	1 : 1,5
Loose, sandy soil	1 : 2,5
Sandy clay, porous clay	1 : 3
Grassed channels*	1 : 3 to 1 : 4
Lined channels	**

Notes:
* *For maintenance purposes the side slopes of grassed channels should not be steeper than 1 (vert.) : 4 (hor.), and they should never be steeper than 1:3.*
** *Side slopes for lined channels should not be much steeper than the above values.*

5.4.4 Calculation of hydraulic capacity of channels

The maximum discharge capacity of a channel under **uniform flow conditions** could be calculated by means of formulae such as those of Manning or Chézy (Section 4.2.6).

Uniform flow conditions apply only if:

- the depth of flow is not forced to deviate from the normal depth by a secondary control;
- the channel drops at each transition by the energy head loss for the transition (Section 4.2.7).

5.4.5 Permissible velocities: soils and grass covers

At velocities that are too low, sediment may be deposited, and at velocities that are too high erosion and structural damage to channels may occur.

Figure 5.15 indicates the minimum recommended average velocities at different depths to prevent the deposition of fine sandy material. The figure also contains graphs of the maximum permissible velocities for different types of soil. These graphs were developed based on data from the *Highway design – Manual of instructions* (5.8). The figures should be conservatively applied where gullies may form.

Grass covers could provide extra protection against soil erosion, and recommended design values are given in **Table 5.6**. The effectiveness of grass covers depends on the type of grass, the type of soil, as well as the annual rainfall. First of all, one should determine how well the grass would establish itself, and then a design velocity may be decided upon by the combined use of **Figure 5.15** and **Table 5.6**. The permissible velocity for soil with grass cover must be limited to a maximum of 1,3 times the allowable velocity for unprotected soil. (5.20).

Table 5.6: Permissible maximum velocities (m/s) for grass covers (5.9)

Mean annual rainfall (mm)	**< 600**			**600 – 700**			**> 700**		
Type of grass	**% Clay content in the soil**								
	> 15	6 - 15	< 6	> 15	6 - 15	< 6	> 15	6 - 15	< 6
Kikuyu	–	–	–	1,8	1,5	0,8	2,5	2,0	1,2
NK 37				2,0	1,5	0,8	2,0	1,5	1,0
K11	–	–	–	1,5	0,8	0,6	2,0	1,5	1,0
Rhodes	–	–	–	1,2	0,8	0,6	.1,5	1,0	0,8
*E Curvula	1,0	0,8	0,8	1,2	0,8	0,6	1,5	1,0	0,8
Blue Buffalo	1,0	0,8	0,8	1,2	0,8	0,6	–	–	–
Paspalum didatum	–	–	–	1,2	0,8	0,6	2,0	1,5	1,0

Note:

*These values are valid for a depth of flow of 0,3 m and on slopes of less than 3%. For other depths the permissible velocities may be adjusted proportionally to the curves in **Figure 5.15**.*

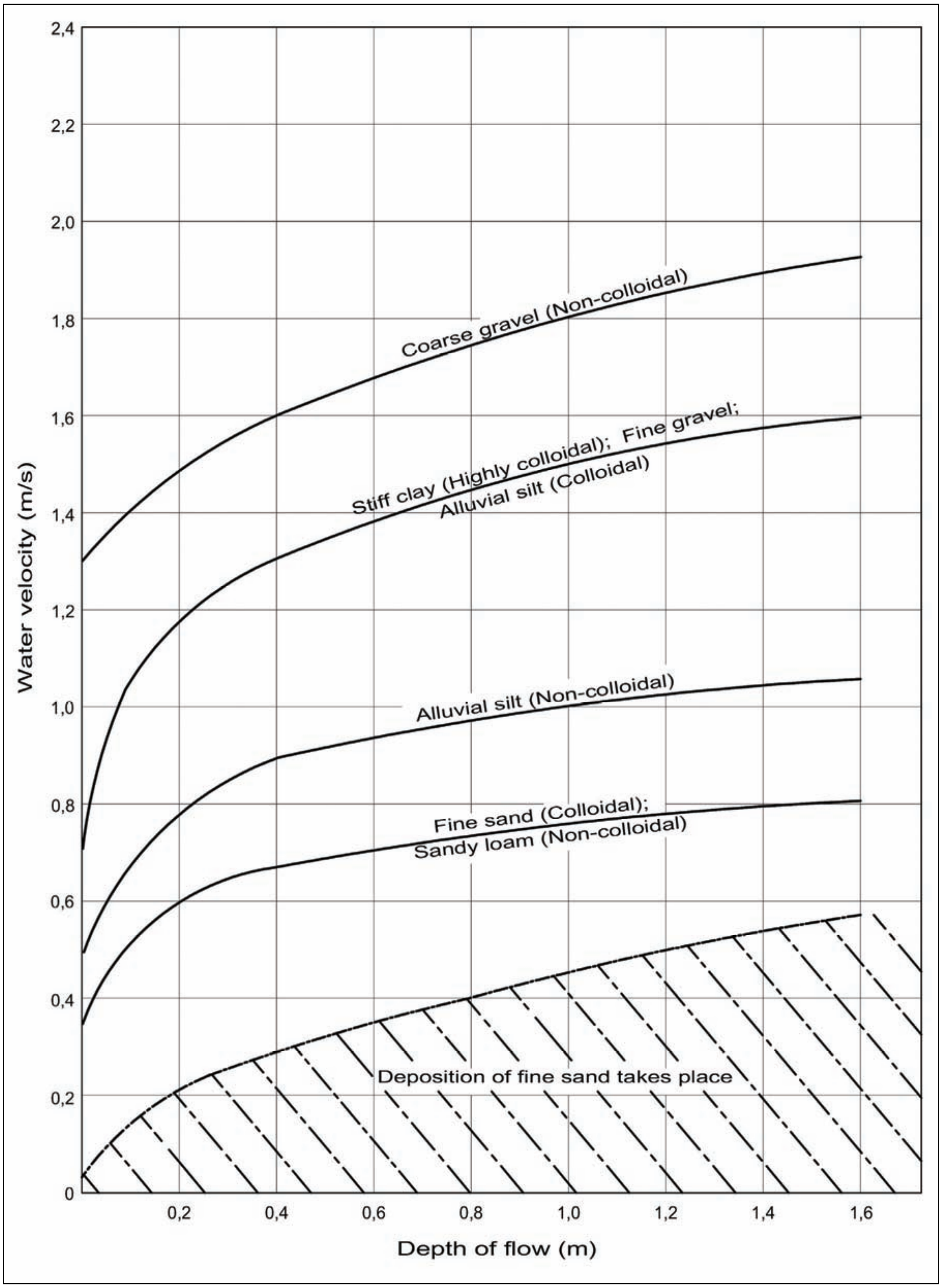


Figure 5.15: Permissible velocities, erosion and deposition

5.4.6 Channel linings: Solid materials

A channel may be directly or indirectly protected against erosion. For direct protection, the bottom and sides of the channel are covered with a lining that is less erodible than the *in-situ* material. For indirect protection, introducing obstructions that cause damming reduces the flow velocity or erosive capacity.

Construction materials generally used for channel linings are:

- concrete;
- rip-rap and stone pitching;
- prefabricated paving blocks; and
- gabions.

5.4.6.1 Concrete linings

The minimum channel lining thicknesses given in **Table 5.7** are based on past experience. Water velocities over linings with joints or cracks should not be higher than about 2,5 m/s[(5.10)] for a thickness of 60 mm, because pulsating pressure changes at joints may cause pieces to break away. Where such failure cannot occur, for example with heavily reinforced sections, velocities of up to 8 m/s and even higher are acceptable, if they occur only occasionally. In this case overlapping joints should be used. The safe velocity over shale is 3 m/s; over dolomite, sandstone or limestone 3 to 7 m/s; and over hard rock such as granite, up to 15 m/s or even higher [5.11)].

Table 5.7: Thicknesses of channel linings [(5.10)]

Total channel depth	Thickness of concrete lining
0 - 0,5 m	60 mm
0,5 - 1,5 m	75 mm
> 1,5 m	100 mm

Unbalanced hydraulic forces acting on concrete linings could cause serious damage in the form of cracking and displacement, which is very difficult and costly to repair. The following conditions may be **particularly harmful**:

- When a canal **empties rapidly**, whilst the water table below the concrete lining drops at a lower rate, the lining may pop up and suffer serious damage, especially during re-filling of the canal. To prevent this from occurring the pressure difference could be relieved by providing drainage below the lining or by installing valves in the lining. Measures to prevent concrete canals from emptying rapidly (long-weirs) should also be considered during the design.

- Where **active soils** occur below concrete linings and they shrink during dry periods, gaps tend to form between the linings and the soils as the soils shrink. When surface run-off from alongside the canal fills these gaps whilst the flow depth within the canal is low, serious cracking may occur [(5.11)]. For this reason, seals should be provided to prevent the entry of water along the canal lining edge.

- Serious cracking could also occur when canals are **filled rapidly** and gaps are present between the lining and the supporting soil. Cracking problems could be particularly severe where the *in-situ* soils are active as well as dispersive.

To overcome the problem with active soils, it is often prescribed that canals should be over-excavated by 600 mm and backfilled with selected compacted material. Flexible linings such as pre-cast concrete panels on top of plastic linings are, however, preferable where active soils are encountered.

5.4.6.2 Riprap and stone pitching

Protective stone layers are often used as an economical means of combating erosion. A major advantage of stone layers is their ability to deform and remain effective with subsidence and limited undermining.

The Shields parameters [5.12] are the best-known criteria for determining whether non-cohesive particles would be lifted off the bed by a given flow. When ordinary stones (relative density 2,65) with diameters > 6 mm are used, the **required diameters of stone d_1 that would just resist lifting from the bed and d_2, the required diameter to resist lifting from the sides**, may be calculated via the following equations:

$$d_1 > 11\,Ds \qquad \ldots(5.7)$$

(50% of the mass of the stone used for protection should be $\geq d_1$. The general form of equation 5.7 is:

$d_1 > 11\,Rs$)

where:

D	=	depth of flow in channel (m)
s	=	slope in longitudinal direction of channel (m/m)
R	=	the hydraulic radius (m)

By using a relationship deduced by Lane [5.12] the **diameter of stone required to protect the sides of trapezoidal channels can be calculated**:

$$d_2 \geq \frac{8{,}3Ds}{\cos\theta\left[1-\dfrac{\tan^2\theta}{\tan^2 Ø}\right]^{\frac{1}{2}}} \qquad \ldots(5.8)$$

where:

θ	=	angle of slope of sides of the channel (°)
Ø	=	natural angle of slope of stone material (°)

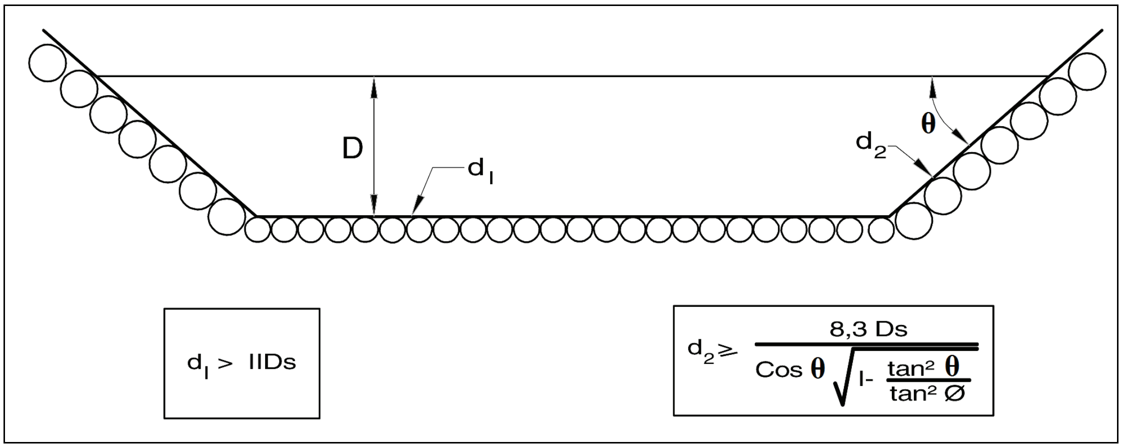


Figure 5.16: Required sizes of the stone for erosion protection of loose bed channels
(The side slope, θ, should always be smaller than the angle of repose, Ø, to ensure stability.)

Values of Ø (the natural slope) for different stone sizes are given in **Figure 5.17** [5.12]. The lines were extended according to Shen [5.13].

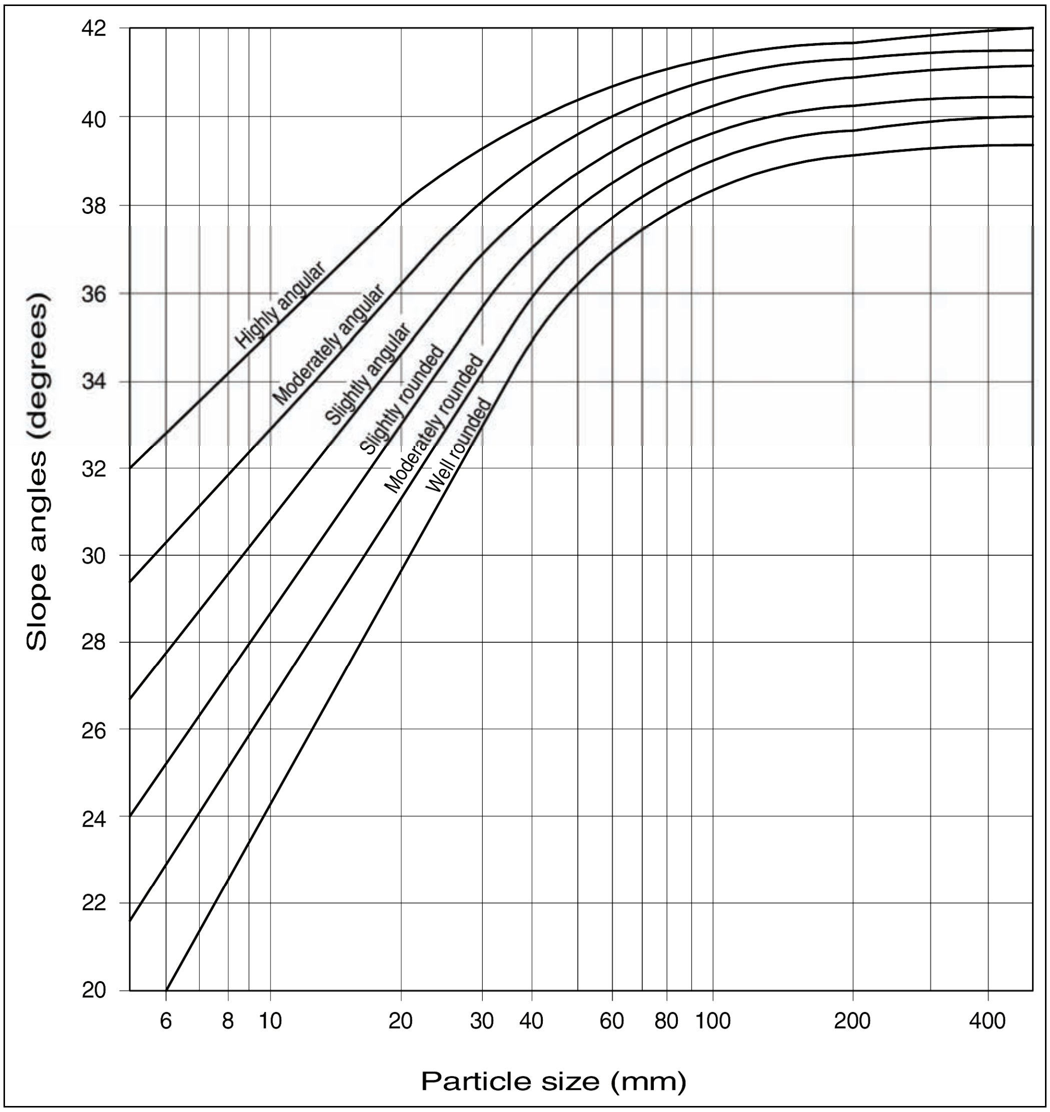


Figure 5.17: Natural slope angles (Ø) for different sizes of stone

Because flow around bends has increased erosive capability, the value for stone sizes calculated according to the above formulae should be increased where the protective layers are used around curves (5.14), as reflected in **Table 5.8**.

Table 5.8: Multiplication factor to increase the required particle size to prevent scour in curved channels

Channel curvature	Factor
Gentle	1,3
Sharp	1,6

Because flowing water tends to lift fine material out from under coarse material, care should be taken that the protective layer is not undermined. This can be achieved by providing filters.

Where stone alone is used, successive layers of adequate thickness and appropriate material sizes based on the following criteria may be used (5.14):

- The thickness of the protection layers should be two to three times the diameter of the stones.
- If the stones are hand-packed, the layer should at least be 1½ times the stone diameter, but always at least 200 mm thick.

- The ratio of stone sizes (diameters) in successive layers should be between 1:5 and 1:10, depending on the size-distribution curves of the material. An adequate number of layers should be placed between the natural material and the final protective layer to ensure stable protection.

Synthetic meshes (Geofabrics) are commonly used to form filters.

5.4.6.3 Prefabricated paving blocks

Where any loose units, regardless of shape, are used for bed protection, the settling velocity, V_s, in water of individual units should be greater than [5.15]:

$$V_s \geq \frac{\sqrt{gDs}}{0,12} \qquad \ldots(5.9)$$

where:

V_s = settling velocity (m/s)
D = depth of flow (m)
s = energy gradient (m/m)

If the effectiveness of the units depends on interaction between different blocks or anchoring by grass, precautions should be taken against the large-scale failure of the protective layer resulting from a local failure, or before the grass becomes established.

Filters should be provided, where necessary, so that underlying material cannot be washed out from underneath the protective units.

5.4.6.4 Gabions

The use of gabions for direct protection may be advantageous, especially where available or manageable stone sizes are not adequate for the stones to function as separate units. It is very important to use the correct stone sizes in relation to the mesh sizes, so that it will not be possible for stones to be washed out. The wire boxes should be tightly packed with stones according to the supplied guidelines.

Design consists of calculating the required size for single stones, described elsewhere, and then determining the equivalent size of gabions. The mass of gabions per unit area covered should be at least 1,5 to 2 times that of the stone calculated in accordance with equations 5.7 and 5.8.

Cut-off walls, suitable filter material (as per the manufacturer's prescription) or linings should be provided where necessary. The life of the wire cages should be guaranteed and plastic coverings should not be used where there is a danger of veld fires.

The corrosion potential should be taken into consideration, especially in industrial and mining areas. **Photographs 5.5** and **5.6** provide details of mattresses whilst **Photographs 5.7** and **5.8** show gabions that can be used for erosion protection.

Photograph 5.5: Flexible concrete mattress for erosion protection in channels

Photograph 5.6: Flexible concrete mattress failure due to weak supporting soils

Photograph 5.7: Gabions used for bank stabilisation (Courtesy: Land Rehabilitation Systems)

Photograph 5.8: Gabions used for erosion protection (Courtesy: Land Rehabilitation Systems)

5.4.7 Indirect protection and stepped energy dissipation

Where considerations of safety permit, it is often more economical to protect a channel indirectly, rather than directly. This is done by the concentrated dissipation of energy at successive structures; the erosive capacity of the flow between the structures is thus decreased, so that continuous direct protection is not required here. Due to problems experienced with the calculations for indirect protection, methods of calculation and design data are treated in a fair amount of detail here.

Three of the variations for indirect protection of channels that could be used are shown schematically in **Figure 5.18**. These variations of stepped energy dissipators will be referred to **Layouts I, II** and **III**.

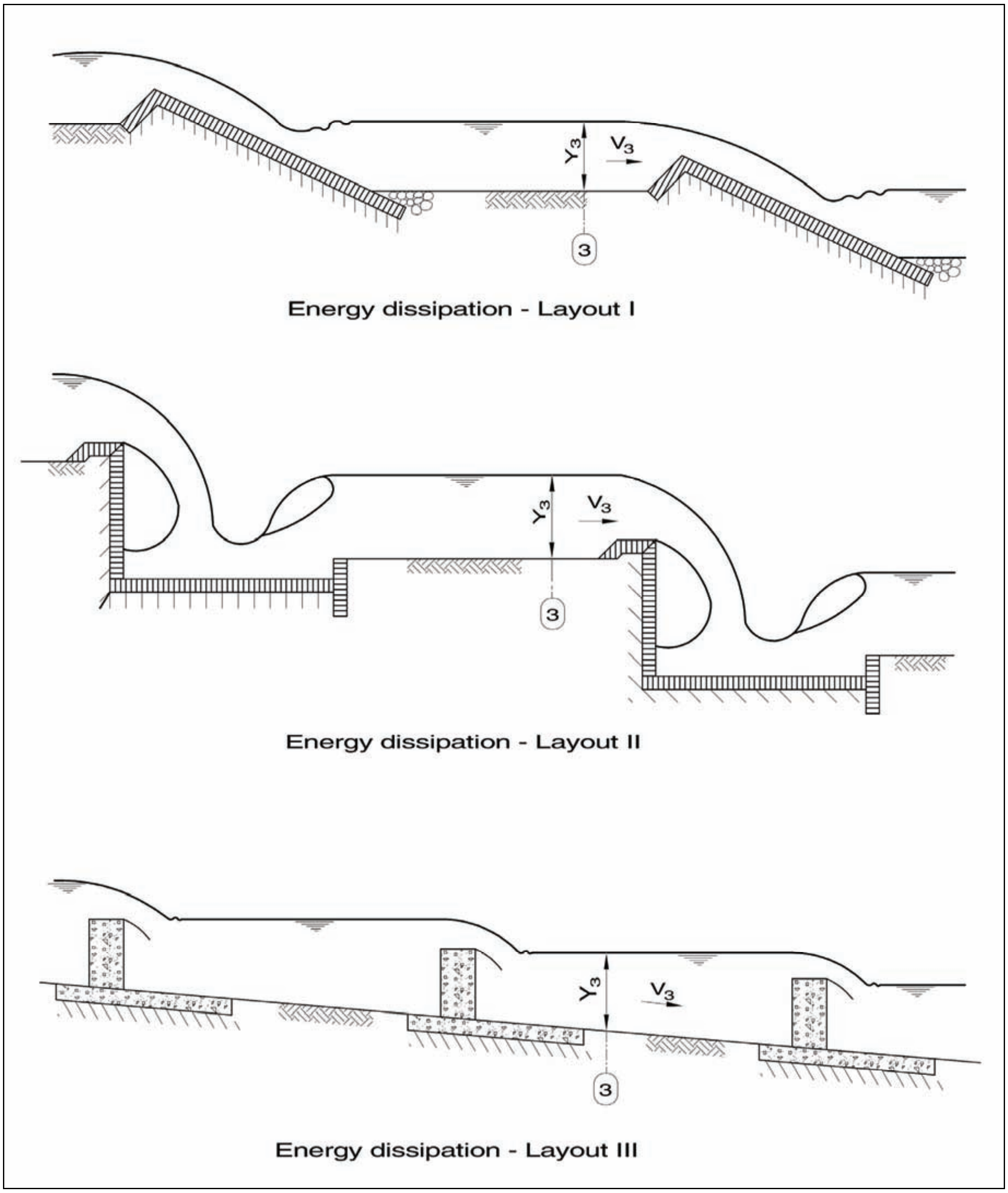


Figure 5.18: Layouts I, II and III for stepped energy dissipators

Photographs 5.9 and **5.10** depict examples of stepped energy dissipators.

Photograph 5.9: Stepped energy dissipator (Hout Bay River, Cape Town)

Photograph 5.10: Stepped energy dissipator along the N3

Layout I sometimes presents an economical solution, especially where the watercourse is not deep and a large area has to be protected against erosion. Construction is quite simple, because no formwork and little reinforcement are required. **Table 5.7** gives the required concrete thickness. **Layout II** offers an effective but expensive solution, and is rarely more economical than **Layout I**. **Layout II** could, however, be used to good effect, for example between the culverts under the carriageways of a freeway.

Especially where deep existing watercourses are to be protected from scour, **Layout III** often provides the best solution. The dams may be masonry, gabions or concrete.

When energy has to be dissipated at the bottom of a long slope, a conventional stilling basin may be used (**Chapter 7**). However, it is often more economical to use stepped energy dissipation for the layouts shown in **Figure 5.18**, especially on steep slopes for which the design calculations are done as described below.

Design procedures for energy dissipation as an indirect way to protect channels

Layout I – Hydraulic analysis

In each case the starting point is to determine the design discharge, Q (obtained from hydrological calculations). Then calculate a flow depth (y_3) and a flow velocity (v_3) at which unprotected soil would not be eroded under the design discharge. (**Figure 5.15 and Table 5.6**) A detailed analysis is given below.

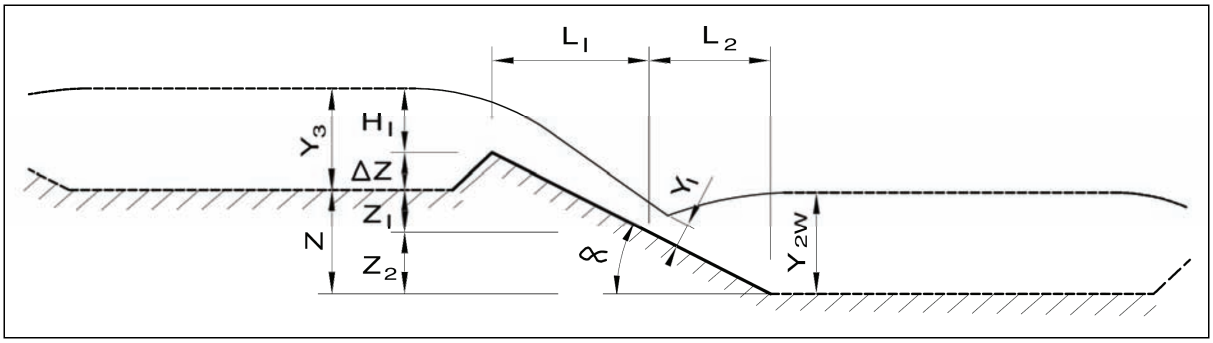


Figure 5.19: Longitudinal section through Layout I dissipator

Calculate both the required step height Δz and damming height, H_1, taking into account whether broad-crested or sharp-crested conditions apply (Section 4.2.5). The depth $y_3 = \Delta z + H_1$. Then calculate y_{2w}, the depth just downstream of the step, in accordance with the y_3 value at the following step (Section 4.2.4), (If the floor slope as well as the distance is small, $y_{2w} \approx y_3$ – based on the conservation of energy principle).

Now assume: $y_2 = \dfrac{y_{2w}}{1{,}3}$

where:

y_2 = equivalent sequent jump depth with a horizontal bed [(5.16)]

Then calculate y_1. For a "wide" stream, in accordance with the momentum principle:

$$\frac{y_1}{y_2} = ½\left[\sqrt{1+\frac{8v_2^2}{gy_2}}-1\right] \qquad \ldots(5.10)$$

Select the slope of the step, α, on the basis of considerations of soil mechanics, e.g. 20°.

L_2 and z_2 could now be calculated [(5.16)] from the following relationships:

$$L_2 = 0{,}82y_2 (\tan\alpha)^{-0,78} \qquad \text{…(5.11)}$$

$$z_2 = L_2 \tan\alpha \qquad \text{…(5.12)}$$

(If the value of y_{2w} is increased to more than the minimum value of 1,3 y_2, the front of the jump would advance up the slope by the same vertical distance. It is not normally practical, however, to make the value of y_{2w} greater than 1,3 y_2.)

The value of z_1 is now calculated (Section 4.2.4):

$$z_1 + y_3 + \frac{v_3^2}{2g} \approx \frac{v_1^2}{2g} + y_1 \cos\alpha \qquad \text{(energy principle)} \qquad \text{…(5.13)}$$

and $v_3\, y_3 = v_1\, y_1$ (continuity)

When the values of z_1 and z_2 are known, the effective step height may be calculated ($z = z_1 + z_2$), and the spacing of the steps is fitted in with the topography. Since the flow between each jump and crest is subcritical, the flow may be turned gradually in plan in these areas, and the crests of the steps need not be parallel.

Layout II – Hydraulic analysis

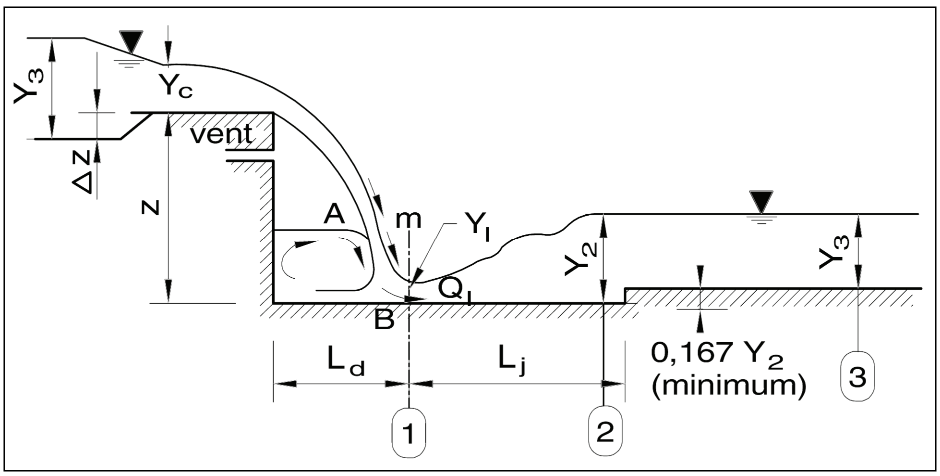


Figure 5.20: Layout II – Waterfall structure

Start by calculating $y_c = \sqrt[3]{\frac{q^2}{g}}$ and then determine the required step height Δz to ensure an acceptable value for y_3.

$$y_3 + \frac{v_3^2}{2g} \approx \Delta z + \frac{3y_c}{2} \qquad \text{(energy principle, Section 4.2.4)}$$

$$y_3\, v_3 = y_c v_c = y_c \sqrt{g y_c} \qquad \text{(from continuity and critical relationship)}$$

Next y_2 should be calculated from y_3 by means of backwater calculation (Section 4.2.8).

The following relationships should all be satisfied [(5.16)]:

$$\frac{y_1}{z} = 0{,}54\left(\frac{y_c}{z}\right)^{1,275} \qquad \text{…(5.14)}$$

$$\frac{y_1}{y_c} = 0{,}54\left(\frac{y_c}{z}\right)^{0{,}275} \qquad \text{…(5.15)}$$

$$\frac{y_2}{z} = 1{,}66\left(\frac{y_c}{z}\right)^{0{,}81} \qquad \text{…(5.16)}$$

$$\frac{L_d}{z} = 4{,}30\left(\frac{y_c}{z}\right)^{0{,}81} \qquad \text{…(5.17)}$$

$$L_j = 6{,}9(y_2 - y_1) \qquad \text{…(5.18)}$$

Layout III – Hydraulic analysis

Calculate the damming level H (Section 4.2.5), as well as the dam height z to find the required depth y_3 at the most critical section. In cases where flow through the dam (gabions) occurs, allowance can be made for the through-flow.

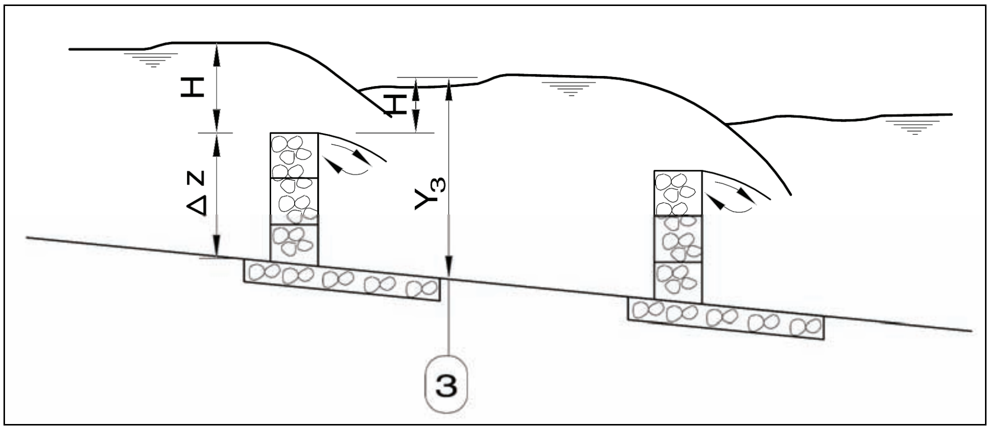


Figure 5.21: Section through Layout III energy dissipator

In the spacing of the weirs, the water downstream of each weir should be as deep as possible. Under sharp-crest conditions, however, the surface downstream may not be higher than the crest of the dam; i.e. no submergence is allowed. Under broad-crested conditions σ, the submergence ratio may not be greater than 0,8 (the modular limit), otherwise additional damming could take place.

The length of channel protection that is required downstream may be estimated from the design procedure for Layout II structures (**Figure 5.20**).

5.5 DISCHARGE CHUTES

5.5.1 Introduction

Discharge chutes (**Figure 5.22**) are often used to discharge water down very steep slopes. Metal chutes cannot be used on slopes steeper than **4 Vertical : 1 Horizontal** [(5.8)], and separate concrete chute sections that fit into one another should not be used on slopes steeper than **1 Vert. : 1½ Hor**.

The most important consideration in chute design is that the flow cannot easily change direction downstream of the point of release; i.e. where supercritical flow occurs. Consequently the flow upstream of the point of release should be subcritical. Where the flow across the shoulder is supercritical, subcritical flow can be ensured by the use of a collector channel. The channel should be long enough to intercept the oncoming flow. Its depth should be more than 1½ y_c so that it will not become submerged. In calculating the approximate velocity at the bottom of the chute, energy losses may be ignored.

Then: $$\frac{3}{2}y_c + z = y_2 + \frac{v_2^2}{2g}$$

and if $y_2 << z$ it follows $$v_2 = \sqrt{\left(\frac{3}{2}y_c + z\right)2g}$$

$$v_2 = \left[\frac{3}{2}\left(\frac{q^2}{g}\right)^{1/3} + z\right]^{\frac{1}{2}}\sqrt{2g}$$

The same basic erosion protection measures as for culverts may be used at the outlet of a discharge chute.

Photograph 5.11 depicts an example of a discharge chute.

Photograph 5.11: Discharge chute, prefabricated blocks

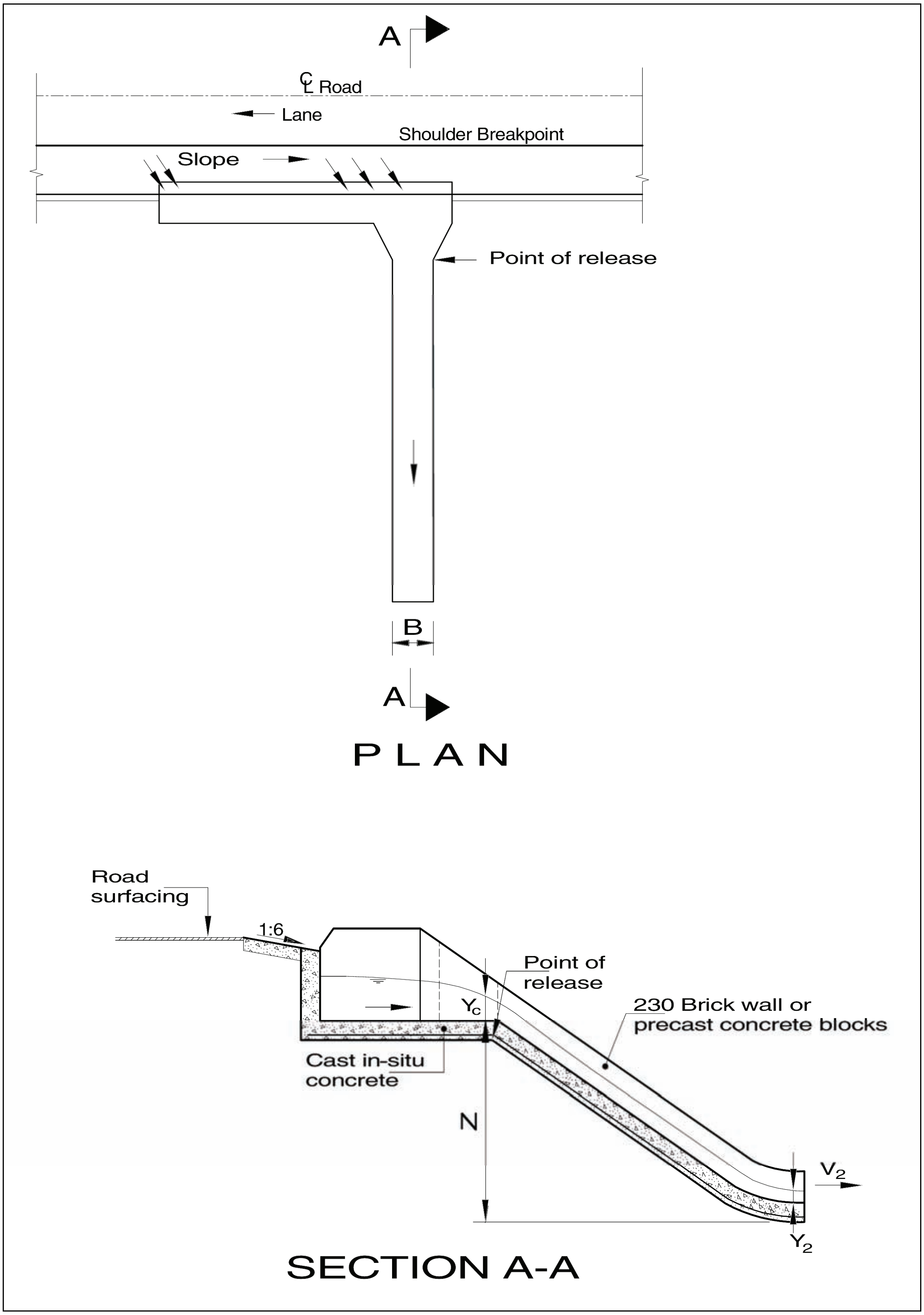


Figure 5.22: Collector channel and discharge chute

5.5.2 Stepped energy dissipation in very steep channels of chutes

The energy in a very steep channel or chute can also be dissipated stepwise as shown in **Figure 5.23**. For slopes of up to about 40° and a rectangular section (width B), the system is designed as described below [5.17].

This system was earlier described as a solution for very steep culverts, but has not found wide application mainly because of debris being trapped. The principle has been applied in small chutes down the sides of embankments.

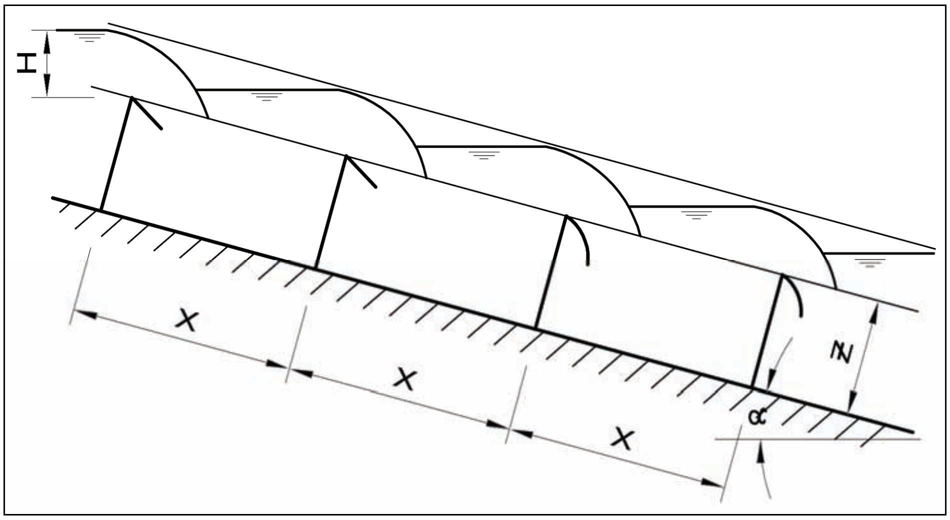


Figure 5.23: Section through stepped energy dissipator

For a given discharge, Q, the following equation applies:

$$Q = K_L \sqrt{2g}\, BH^{\frac{3}{2}} \qquad \text{…(5.19)}$$

where:

K_L	=	discharge coefficient
H	=	head (m)

In addition,

x	=	distance between obstructions (m)
z	=	height of obstructions (m)

In design, an estimated value for $^H/_B$ is used and the appropriate $^Z/_B$ value is read off from **Figure 5.24** for the slope angle α. For these values of $^H/_B$ and d, a value for K_L may be read off from **Figure 5.25**. Test whether the above equation for discharge is satisfied. If not, select a new value for H/B and repeat the procedure. If the slope becomes steeper than 40°, simple obstructions are no longer effective [5.17], and obstructions with openings placed alternately left and right in the channel are to be used.

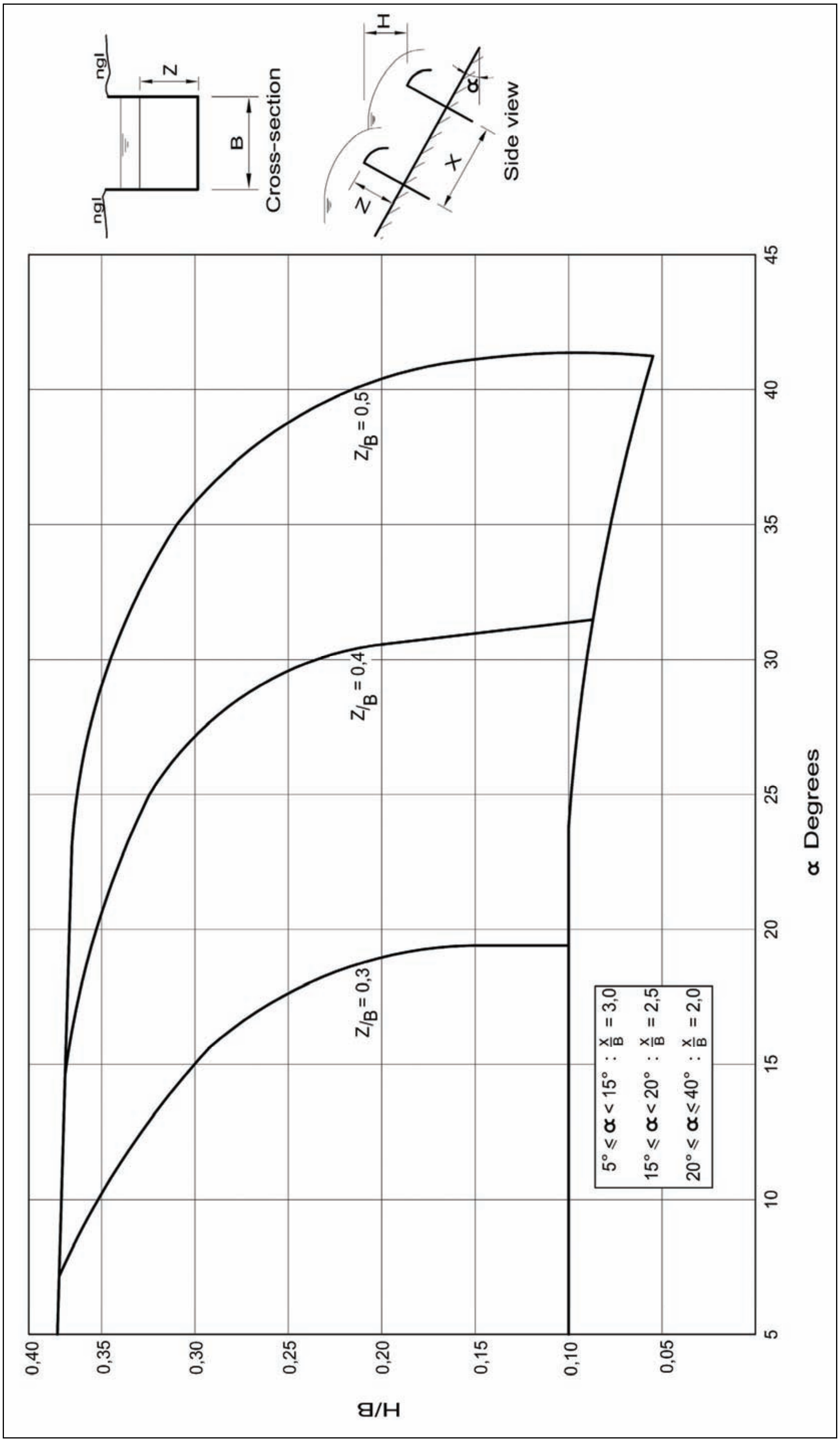


Figure 5.24: Energy dissipators in very steep channels: Optimum dimensions

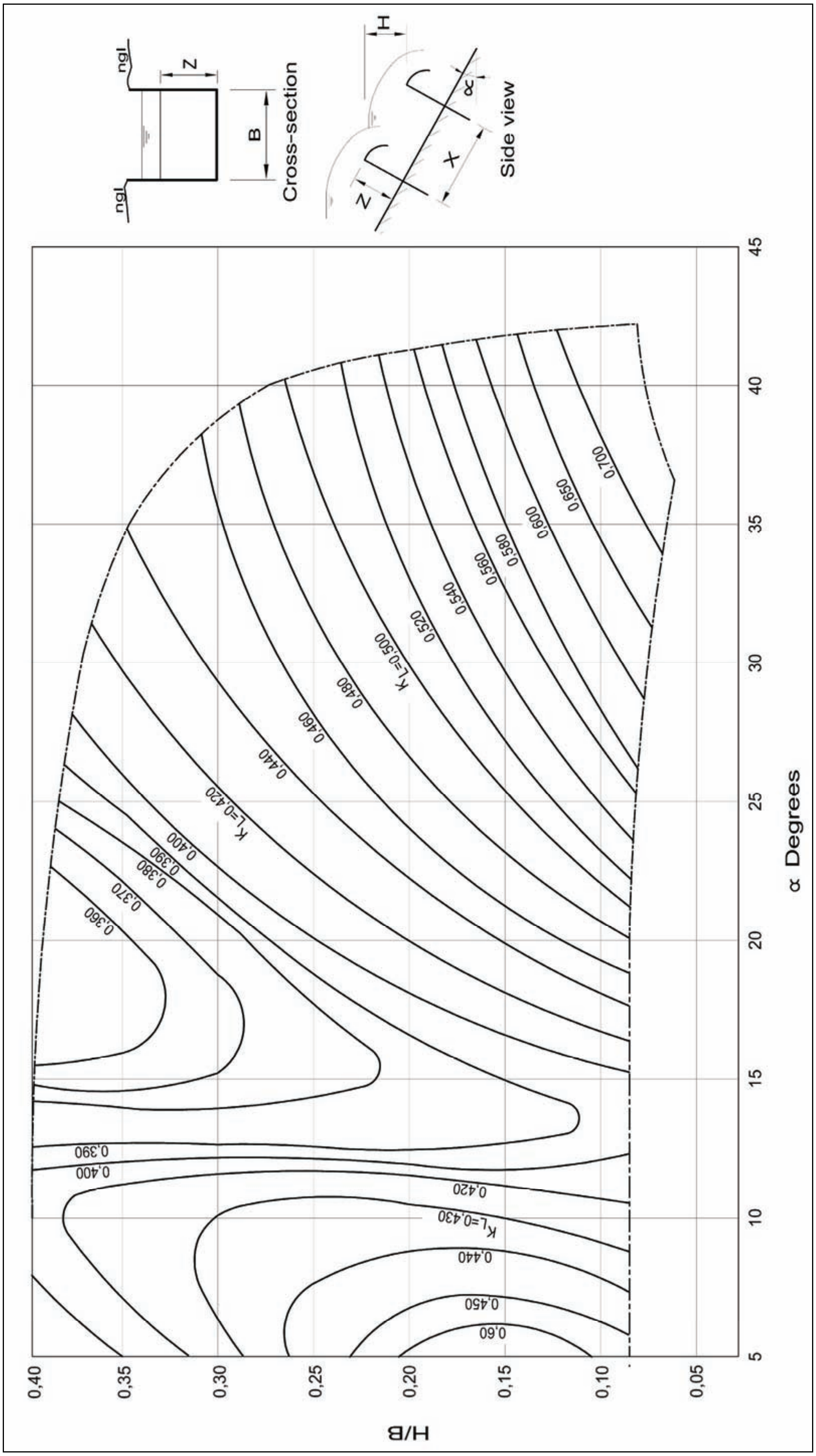


Figure 5.25: Discharge coefficients for energy dissipators in very steep channels

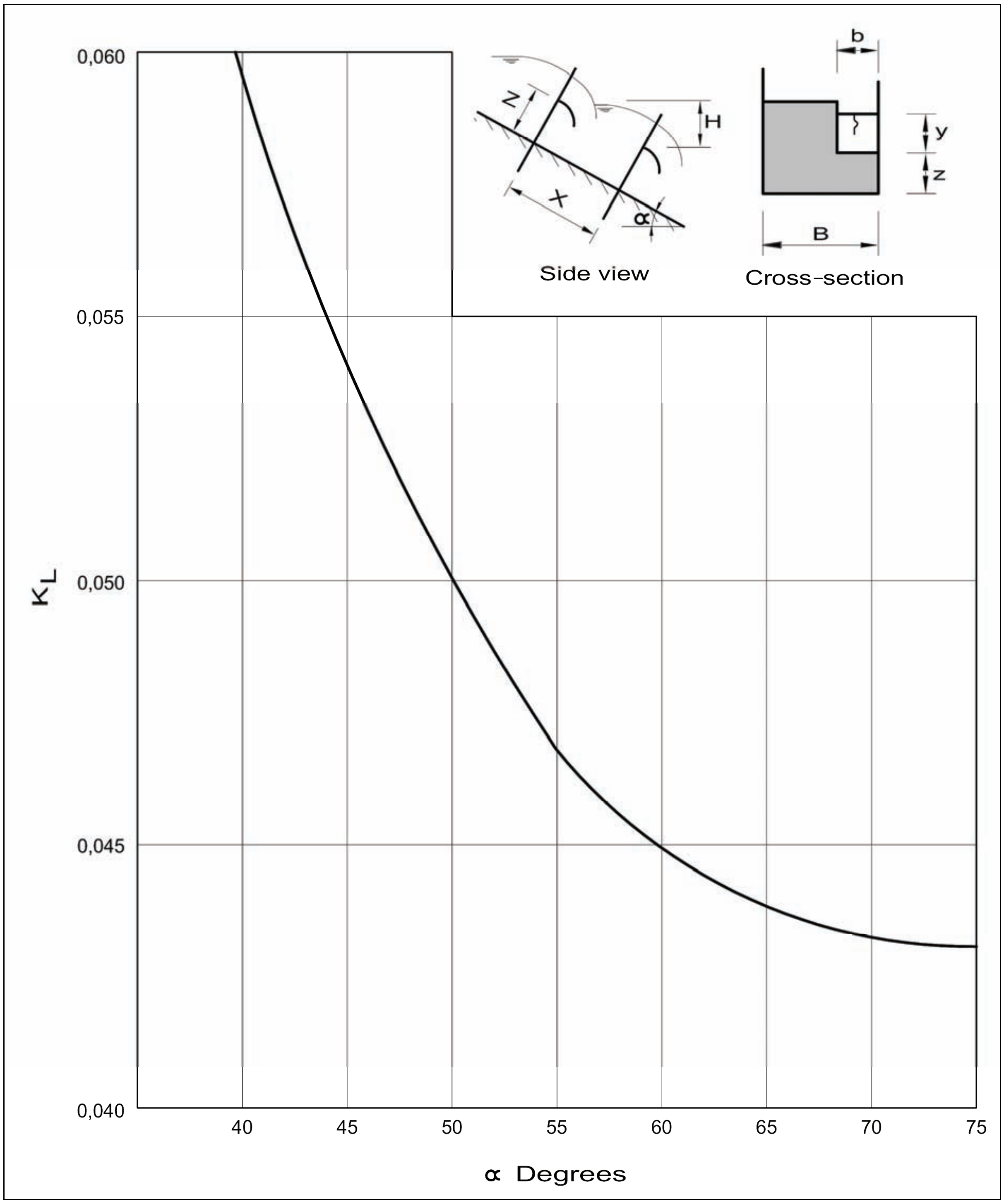


Figure 5.26: Discharge coefficients for energy dissipators in very steep channels

In this case the discharge equation reads as follows:

$$Q = K_L \sqrt{g} B^{\frac{5}{2}} \qquad \ldots(5.20)$$

and the ratios below were standardised at approximate optimum values:

$\frac{X}{B} = 0{,}5$ $\qquad \frac{z}{B} = 0{,}3$

$\frac{b}{B} = 0{,}4$ $\qquad \beta = 45°$

For a given value of α, the value of K_L could be read off from **Figure 5.26**, and the required value of B and those of other dimensions may be calculated directly.

A similar system was developed for trapezoidal channels with slopes of up to 45°.

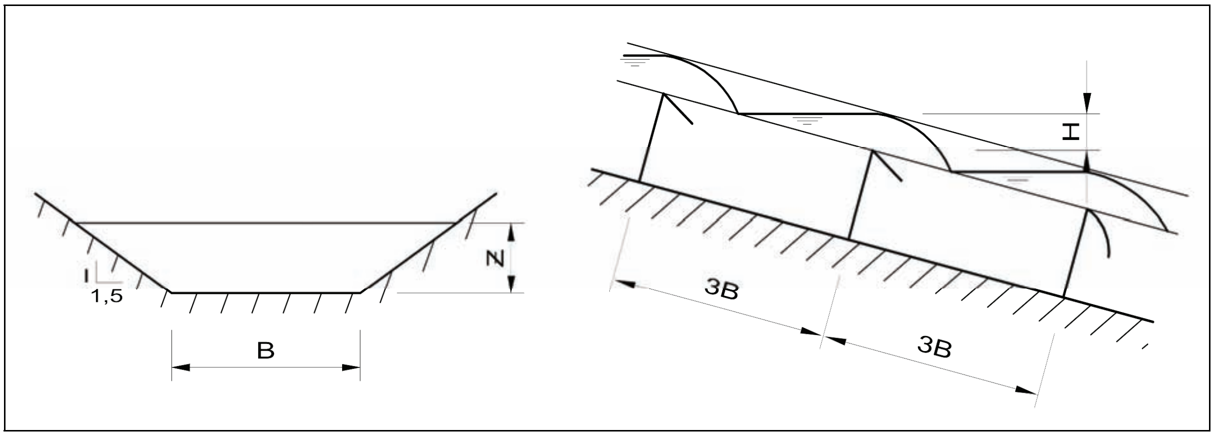


Figure 5.27: Definition sketch – Stepped energy dissipation in trapezoidal channels

The discharge equation in this case (with side slopes standardised at 1:1.5 and x at 3B is:

$$Q = K_L \sqrt{2g}(B + 3z)H^{\frac{5}{2}} \qquad \text{…(5.21)}$$

With Q known, values are estimated for B and H/B.

From **Figure 5.28** values for K_L and z/B can now be read off. The value of z/B is 0,5 below the dashed line and 0,75 above it.

Now test whether the values for K_L, B, z and H satisfy the equation above and select improved values for H and B until the equation is satisfied.

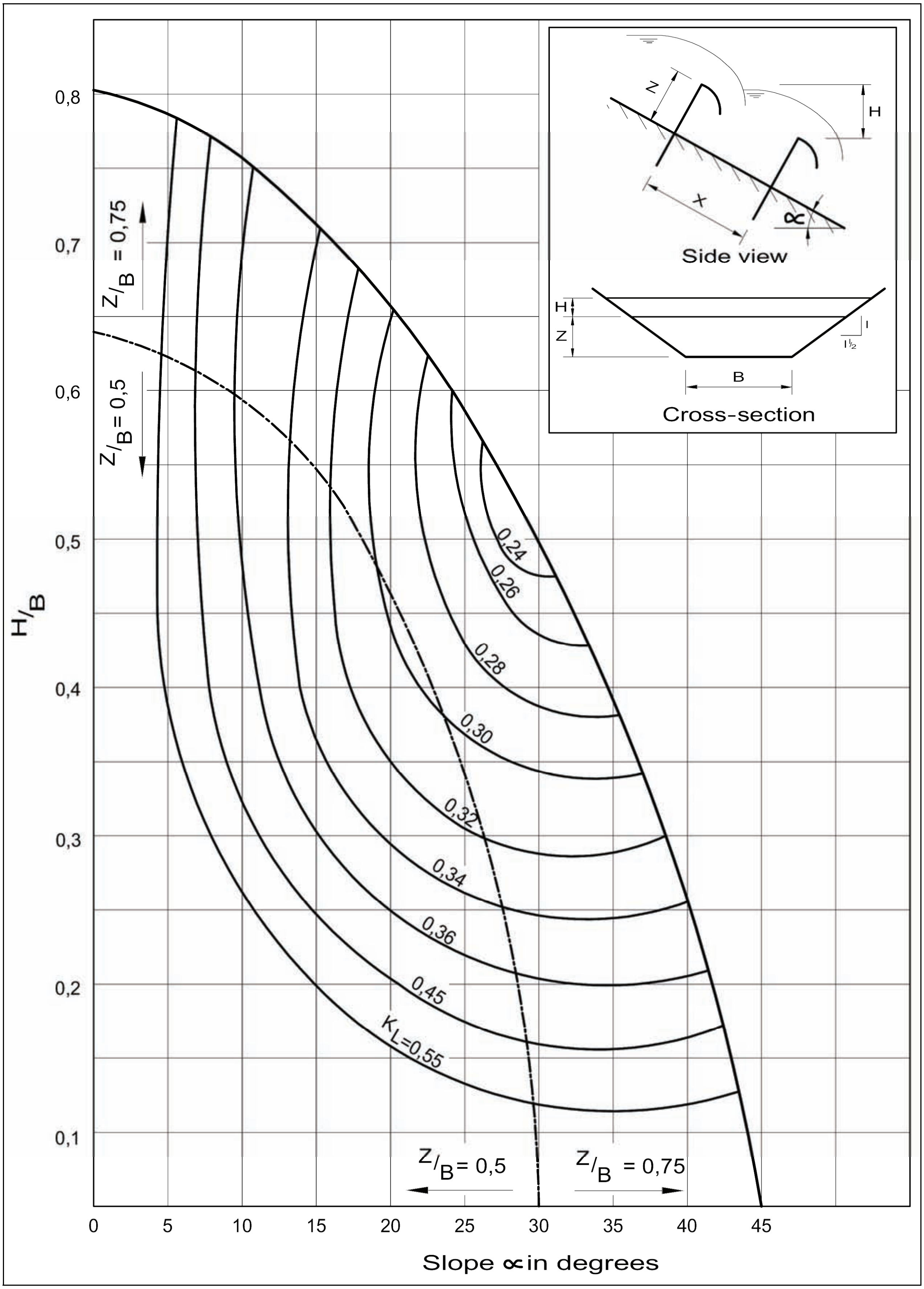


Figure 5.28: Discharge coefficients for obstructions in trapezoidal channels

5.6 DISCHARGE PIPES

5.6.1 Introduction

Discharge pipes are normally laid underground and serve:

- as connector pipes from draw-off structures, such as grid inlets to storm water mains, culverts, etc.;
- to discharge water down erodible slopes.

5.6.2 Underground connector pipes

The following criteria should be applied when underground connector pipes are used:

- Pipes running in the same direction as the road should not be placed under the travelled way and the minimum allowable diameter for such pipes is 450 mm.
- Manholes should be provided at least every 200 - 350 m for diameters ≥ 1 200 mm and at least every 100 - 200 m for diameters < 1 200 mm.
- Manholes should also be provided at:
  - all changes in direction > 10°;
  - changes in size of the pipes;
  - at all junctions; as well as
  - all slope reductions. (Slope reductions should however be avoided if at all possible.)
- Where it is impractical or dangerous to deal with supercritical flow in median and side drains, the water may be drained off or discharged at regular intervals into underground storm water mains.

5.6.3 Discharge pipes down side slopes

As in the case of discharge chutes (**Section 5.5**), the greatest problem here is to transfer supercritical flows into the down-pipes. This may be achieved by using a collector channel in which the flow depth is greater than the critical flow depth (subcritical flow). Fitting a cone inlet, which would improve the inlet conditions, could increase the capacity of a steep discharge pipe. The sectional area at the release point will control the capacity. Test the available specific energy value at different sections to determine where the minimum value occurs; i.e. where the control is formed.

The following criteria should be incorporated in the design of down pipes:

- Diameters < 200 mm should not be used, and where necessary a grid should be provided (on top of the collector channel) to prevent blockage. When used on steep slopes, lengths greater than 20 m should be well anchored.

- Where the danger of erosion is not great, a T-piece could be fitted at the lower end of a steep downpipe to dissipate energy. In other cases the same measures as for culverts may be adopted.

5.7 COMBATING EROSION ON STEEP SLOPES

5.7.1 Introduction

The erosion potential on embankments increases rapidly down slopes, due to the following factors:

- Rates of flow increase as rainwater accumulates;
- Flow velocities tend to increase.

With large-scale work, it is possible to determine beforehand, using the different methods [5.18] on what combinations of slopes and heights of embankments and cuts, serious erosion would take place. With high embankments the water could be diverted by a stepped system of berm channels if necessary, and the areas in between protected from erosion as described below.

5.7.2 Stepped energy dissipation on steep slopes

For erosion protection on steep slopes of typical cuts and embankments, vegetative protection along the contours is generally the best solution, especially if the flow velocity of the water could be repeatedly reduced by the vegetation. Where streams of water become too large, the methods given in 5.4.7 may be employed, or the water may be discharged through berm channels.

Hedges are established by a combination of vegetative and structural materials. They may sometimes be used on slopes steeper than 1 Vert. : 1½ Hor., but are more effective on flatter slopes such as 1 Vert. : 2 Hor. Their effectiveness is based on the fact that the speed of the water is repeatedly reduced, attenuated and absorbed to some extent. A good vegetative cover enhanced by the use of the hedges could be aesthetically pleasing.

The most important aspects [5.19] of stepped spillways are summarised here:

- The slope should be given a stable finish; the maximum slope is often 1:1,5 but a slope of 1:2 is preferable. Water should be diverted at the top, as far as possible.
- Underground drainage should be provided at potential wet spots.
- Toe protection should be constructed, as reflected below:
  - If there is little space (**Figure 29a**); or
  - where more space is available (**Figure 5.29b**); and
  - form erosion hedges along contours **Figure 5.29c**.

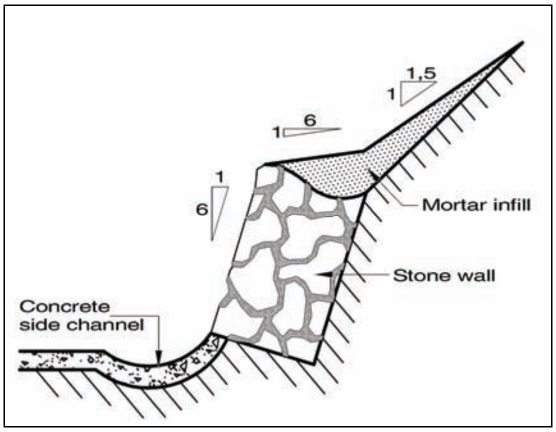


Figure 5.29a: Toe protection options

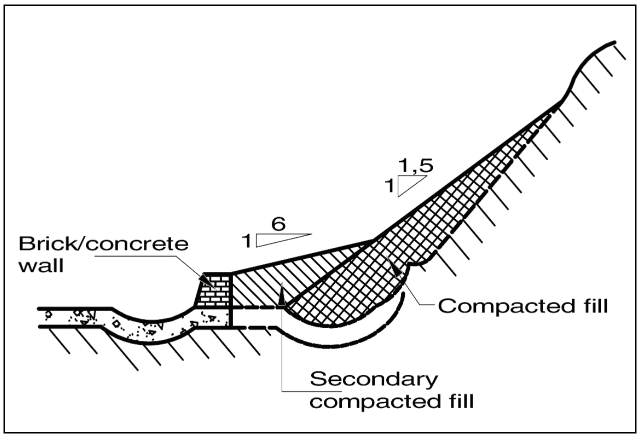


Figure 5.29b: Toe protection options

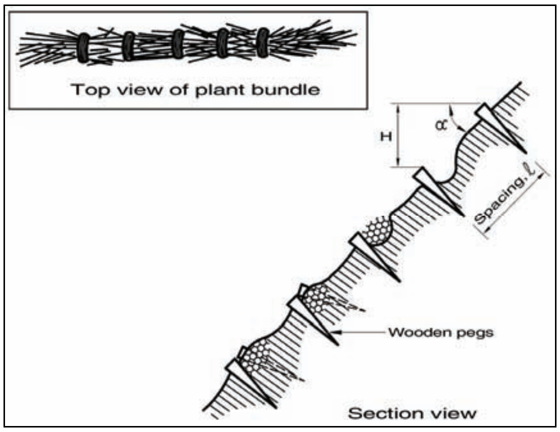


Figure 5.29c: Slope protection option and spacing of hedges

If the water is dammed effectively at each hedge, the spacing may be determined approximately as follows: Determine the permissible flow velocity (V_c) for the given soil from **Figure 5.20** (depth of flow ≈ 0).

Equate: $$v_c = \sqrt{2gH} \text{ or} \qquad \ldots(5.22)$$

$$H = \frac{v_c^2}{2g}$$

The spacing may now be determined in terms of the slope.

$$\ell = \frac{H}{\sin\alpha} \qquad \ldots(5.23)$$

It is often possible to determine the value of H by observing the vertical distance down existing nearby slopes where serious erosion has developed.

If plant bundles are used to restrict erosion on steep slopes, the procedure for installation is as follows:

Step 1

Begin at the bottom of the slope and knock in wooden pegs along the contour line.

Step 2

Dig a furrow above the pegs, with a sectional area of half that of the bundles to be used (diameter of bundles e.g. 200 mm). The bundles should preferably contain shoots that may take root, reeds or grass and should preferably be indigenous.

Step 3

Place the bundles in the furrows.

Step 4

Knock in more pegs through and next to the bundles. If necessary, cover the bundles with soil tamped down so as to stimulate the growth of the shoots.

Planks may be used instead of bundles. (Tree trunks have also been successfully used in a staggered pattern.)

Step 5

Suitable vegetation has now been established. To keep subsequent maintenance to a minimum, vegetation from the surrounding environment should preferably be used and should be chosen, established and cared for in consultation with botanists.

Step 6

Maintenance is carried out where necessary, especially during the early stages.

5.7.3 Direct protection

Direct protection is used mainly where vegetation protection will not be successful; for example, where too little sunlight penetrates (such as under bridge decks) or in very dry areas. Continuous paving, such as paving blocks or grouted stone, often provide the best solution for these scenarios. Erosion caused by people, where the public has access, should also be taken into account or be prevented by providing proper access in the form of well proportioned steps.

5.8 WORKED EXAMPLES

A number of typical problems are explained below.

5.8.1 Worked Example 5.1

Problem description Example 5.1

Determine the depth of flow if the rainfall intensity is 100 mm/h on a roadway with a width of 10 m and a cross-fall of 2%. The road gradient is 6%.

The aim should always be to limit the flow depth on the road surface to a maximum of 6 mm to prevent hydroplaning.

Solution Example 5.1

Figure 5.1 provides the relationship between the road gradient and road cross-fall, width of the roadway, the energy slope and the flow depth.

Figure 5.1 could be used to determine the flow depth by starting with the road gradient (n_2) and moving anti-clockwise on the nomograph.

S_f, the energy slope is dependant on n_1 and n_2 and may be calculated as follows:

$$S_f = \sqrt{n_1^2 + n_2^2}$$

$$S_f = \sqrt{2^2 + 6^2}$$

$$S_f = 6{,}32\%$$

Calculate the flow path length, L_f:

$$L_f = W\left(1 + \frac{n_2^2}{n_1^2}\right)^{\frac{1}{2}} = (10)\left(1 + \frac{(6)^2}{(2)^2}\right)^{\frac{1}{2}}$$

$$L_f = 31{,}62$$

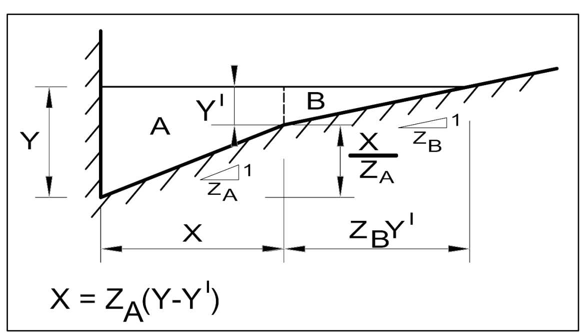
$$d = \left(4{,}6x10^{-2}\right)(L_f I)^{0,5}\left(S_f^{-0.2}\right)$$

$$d = \left(4{,}6x10^{-2}\right)((31{,}62)(100))^{0,5}(0{,}0632)^{-0.2} = 4{,}49 \text{ mm}$$

5.8.2 Worked Example 5.2

Problem description Example 5.2

Determine the flow capacity in a side channel with the dimensions reflected below:



Manning roughness, n = 0,015 s/m$^{1/3}$
Flow depth, Y = 100 mm
Y^1 = 40 mm
$1/Z_A$ = 1/20
$1/Z_B$ = 1/40
Road gradient = 5 %

Solution Example 5.2

The following relationships could be obtained for geometry (units in mm):

$X = Z_A(Y - Y^1)$
$X = (20)(100 - 40)$
$X = 1200$
$Z_B Y^1 = (40)(40)$
$Z_B Y^1 = 1600$
Top width, T = 1200 + 1600 = 2800 mm
$$Z = \frac{T}{Y}$$
From the graph (for a simple triangular channel), **Figure 5.4**: Q = ± 0,28 m³/s

$Q = Q_A + Q_B$
S = 0,05 m/m
Manning n = 0,015 s/m$^{1/3}$

Parameter	Section A	Section B
Cross-sectional area (m²)	$A_A = (0,5)(0,1 + 0,04)(1,2)$ $A_A = 0,084$	$A_B = (0,5)(0,040)(1,6)$ $A_B = 0,032$
Wetted perimeter (m)	$P_A = 0,1 + \left[(0,06)^2 + (1,2)^2\right]^{0,5}$ $P_A = 1,302$	$P_A = \left[(0,04)^2 + (1,6)^2\right]^{0,5}$ $P_A = 1,6005$
Hydraulic radius (m)	$R_A = 0,06454$	$R_B = 0,06454$
Flow rate from Manning $Q = \frac{R^{0,667} S^{0,5}}{n} A$	$Q_A = 0,2015$	$Q_B = 0,03514$

The total capacity of the channel is:

$Q_{total} = 0,2366$ m³/s

5.8.3 Worked Example 5.3

Problem description Example 5.3

Determine the flow capacity of a drop grid inlet, dimensions of 0,9 by 0,6 m and a submergence of 0,2 m if the approaching flow is subcritical. For comparison with **Figure 5.6** assume that the discharge coefficient = 0,8 and the blockage factor, F = 0,5.

Solution Example 5.3

Figure 5.6 provides the relationship of flow rate for an orifice control or a broad-crested weir.

$$Q = 1,77A\sqrt{H}$$

$$A = (0,9)(0,6) = 0,54 \text{ m}^2$$

$$Q = 1,77(0,54)\sqrt{0,2}$$

$Q = 0,428$ m³/s

The calculation of the flow rate was based on the orifice equation.

5.8.4 Worked Example 5.4

Problem description Example 5.4

Determine the kerb flow rate if the flow depth is 100 mm and the road gradient is 4%. The road cross-fall is 2% and the Manning roughness is 0,015 s/m$^{1/3}$.

Solution Example 5.4

The Manning equation may be used.

$$Q = \frac{1}{n}\frac{A^{\frac{5}{3}}}{P^{\frac{2}{3}}}\sqrt{S}$$

$$A = \frac{1}{2}YT = (0,5)(0,1)(5,0) = 0,25 \text{ m}^2$$

$$P = Y + \sqrt{Y^2 + T^2} = 0,1 + \sqrt{(0,1)^2 + (5,0)^2} = 5,101 \text{ m}$$

$$S = \frac{4}{100} = 0,04 \text{ m/m}$$

$$Q = \frac{1}{(0,015)}\frac{(0,25)^{\frac{5}{3}}}{(5,101)^{\frac{2}{3}}}\sqrt{0,04} = 0,446 \text{ m}^3\text{/s}.$$

Strictly speaking, the very wide section with variable velocities should be subdivided into narrower sections.

5.8.5 Worked Example 5.5

Problem description Example 5.5

Determine the maximum flow depth and velocity in a wide channel with a slope, S is 2%. The channel is lined with stones (relative density 2,65) and representative size (more than 50% by mass) of 250 mm.

Solution Example 5.5

For a wide channel it is known that:

R = D = y

where:

R	=	hydraulic radius (m)
D	=	y = flow depth (m)

From Chezy:

$$V = 18\log\left(\frac{12R}{k_s}\right)\sqrt{RS}$$

where:

S	=	slope (m/m)

From Shields (Equation 5.7) it follows that for a stable channel:

d_1 > 11 DS

$$D = y = \frac{(0{,}25)}{(11)(0{,}02)} = 1{,}14 \text{ m}$$

Now the velocity can be calculated:

$$v = 18\log\left(\frac{12(1{,}14)}{(0{,}25)}\right)\sqrt{(1{,}14)(0{,}02)}$$

v = 4,72 m/s

5.8.6 Worked Example 5.6

Problem description Example 5.6

Determine the required diameter of stones to protect the sides and bottom of a trapezoidal channel with side angles of 25° and a flow depth of 1,8 m. The stones are slightly angular and have an angle of repose of 30°. The channel slope is 0,1%.

Solution Example 5.6

For a stable bed the particle size (d_1) should at least be:

$$d_1 = 11DS = (11)(1{,}8)(0{,}001) = 0{,}0198 \text{ m}$$

For stable side slopes the particle size (d_2) should at least be:

$$d_2 = \frac{8{,}3Ds}{\cos\theta\sqrt{1 - \frac{\tan^2\theta}{\tan^2\varphi}}}$$

$$d_2 = \frac{(8{,}3)(1{,}8)(0{,}001)}{\cos(25)\sqrt{1 - \frac{\tan^2(25)}{\tan^2(30)}}} = 0{,}028 \text{ m}$$

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CHAPTER 6 - LOW-LEVEL RIVER CROSSINGS

PA Pienaar

6.1 INTRODUCTION

6.1.1 Use of these guidelines

The objective of this chapter is to provide guidance for the design of low-level river crossings (LLRC). The applicability of recommendations in this chapter should be evaluated in terms of the specific conditions and circumstances being designed for. **Table 6.1**, **Road Map 6**, reflects aspects that are covered in this chapter.

6.1.2 Definition of low-level river crossings

A low-level river crossing (LLRC) is a submersible road structure, designed in such a 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. The different aspects that need to be considered and references to sections in this chapter are reflected in **Table 6.1** (**Road Map 6**).

6.1.3 Terminology

A number of terms are used to describe these structures, such as:

- low-level river crossing;
- low-level bridge;
- low-level structure;
- low-water structure;
- submersible structure;
- low-water stream crossing;
- causeway;
- vented causeway; and
- small bridge.

In this chapter, the term *low-level river crossing*, abbreviated as LLRC, is generically used.

6.1.4 Classification

LLRCs are classified as follows:

- **Drift -** A drift is defined as a specially 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 or commercial products such as Armorflex (concrete blocks held together longitudinally with polyester, galvanised steel or stainless steel cables) and Hyson Cells. These are mats comprising square, hollow geocells - fabricated from thin plastic film – and filled *in-situ* with grout to form a layer of interlocking concrete blocks. Drifts are also referred to in the literature as *fords*.

Figure 6.1 depicts examples of drifts.

- **Causeway -** A vented causeway (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 could be formed by means of pre-cast pipes or portal culverts, corrugated iron void formers, short span decks (less than 2 m), etc. Vented causeways are also referred to in the literature as *vented fords*.

Figure 6.2 (a) to (c) depicts examples of causeways.

Table 6.1: Road Map 6 – Low-level Crossings

ROAD MAP 6			
Typical topics			
Description	**Par**	**Input information**	**Example problem**
Low-level river crossings	6.1.2	Foundation conditions and traffic loads.	Implementation of a LLRC for a river section on a tertiary road linking rural settlements on both banks of a river. The route is inaccessible for vehicles, which use an alternative route via the main road with a length of 45 km. Current traffic volume is 50 vehicles per day and it is expected to increase to 300 vehicles per day, should a proper river crossing structure be provided. The expected traffic growth rate for the next 20 years is 2 percent per year.
Application of LLRCs	6.1.5	Cost benefit analysis including the cost of capital, discount rate, economic life and yearly costs.	
Selection of the structure type	6.4	Road usage and user cost relationships.	
Hydraulic considerations	6.5	Flood peaks and recurrence interval.	
Structural design considerations	6.6	Road usage, flows, foundation conditions, available shapes and erosion protection.	

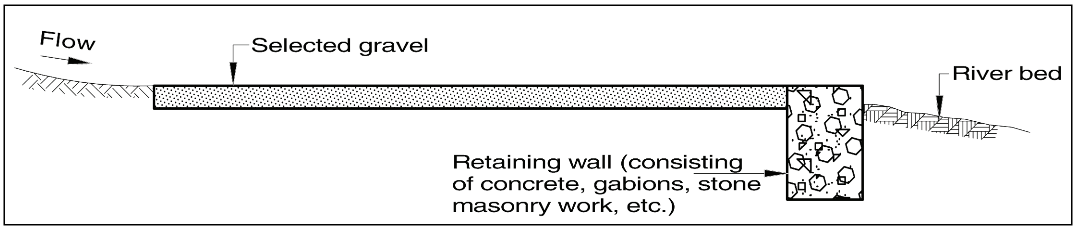


Figure 6.1 (a): Examples of drifts (cross-sections through drifts)

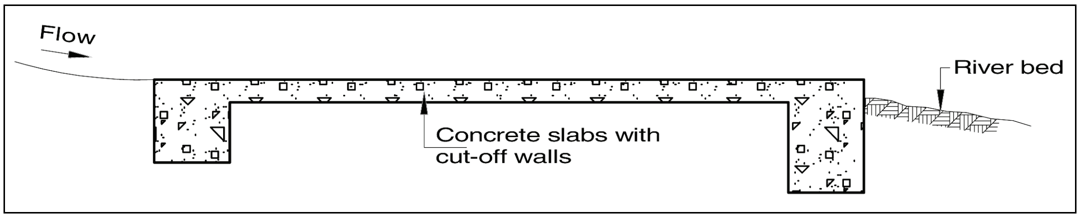


Figure 6.1 (b): Examples of drifts (cross-sections through drifts)

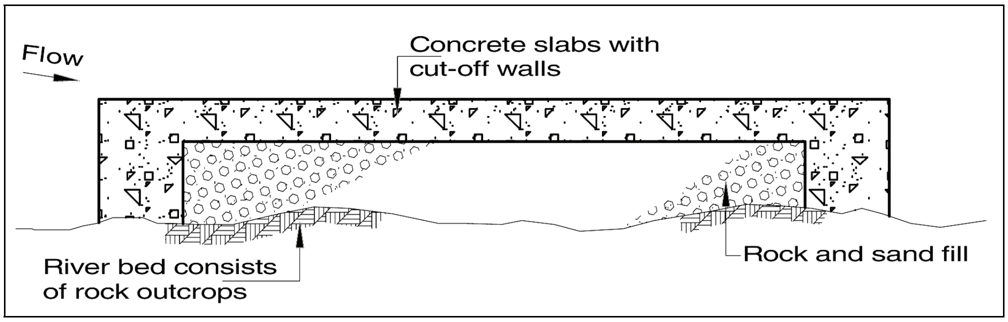


Figure 6.1 (c): Examples of drifts (cross-sections through drifts)

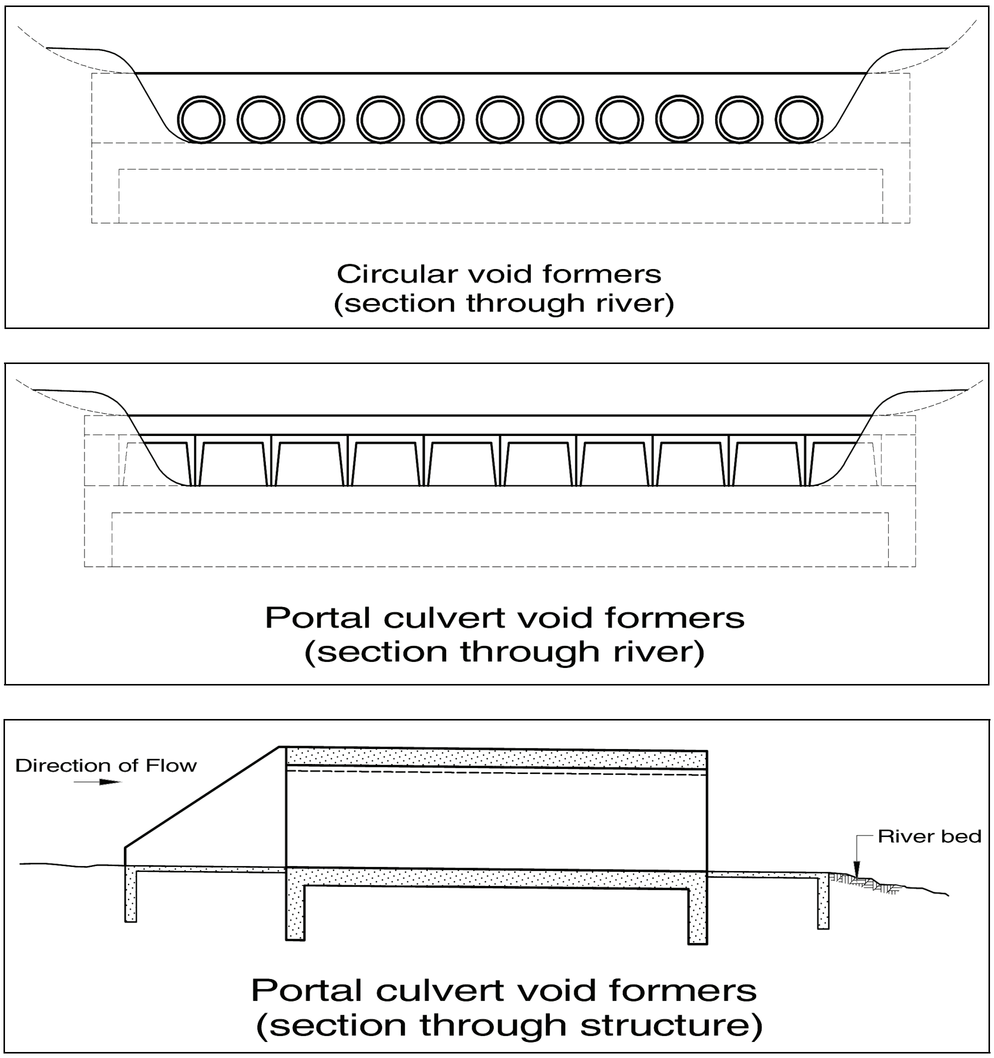


Figure 6.2: Examples of causeways

- **Low-level bridge** - A low-level bridge is defined as a structure consisting of a short-span deck (typically between 4 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 typically less than 2 m.

Figure 6.3 depicts a typical low-level bridge.

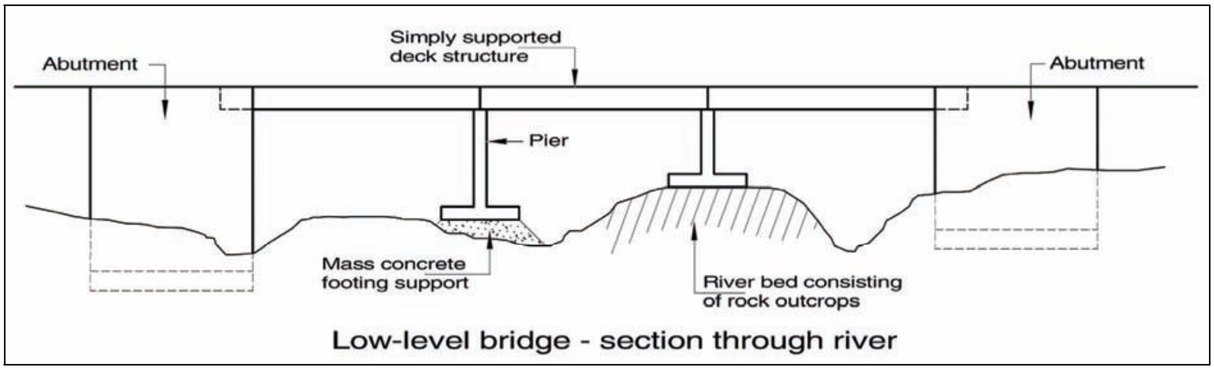
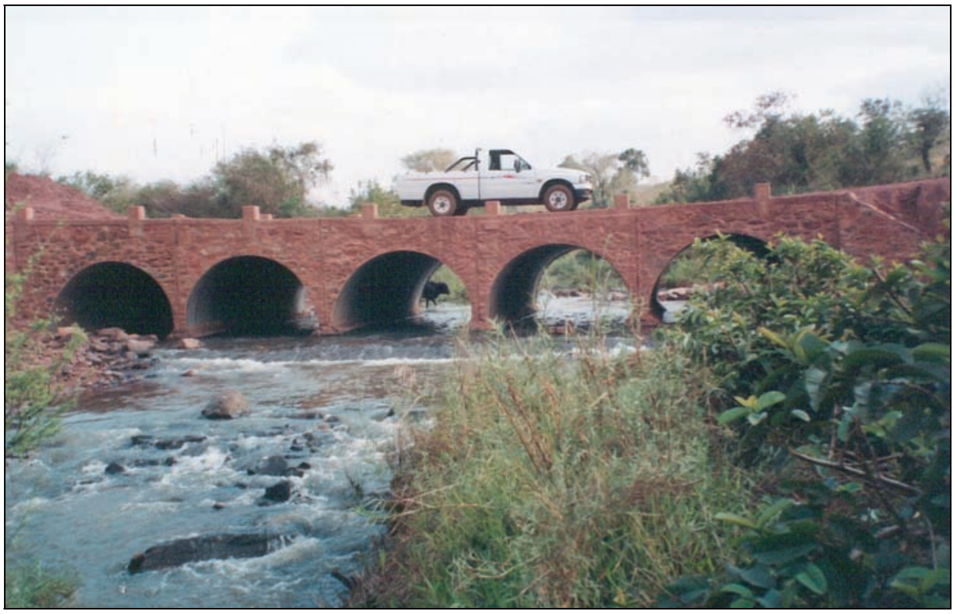


Figure 6.3: Example of a low-level bridge

Photographs 6.1 and **6.2** show examples of low-level structures.

Photograph 6.1: A low-level river crossing provides safe access across the wide river



Photograph 6.2: A low-level crossing constructed with stone

6.1.5 Application of LLRCs

6.1.5.1 Basic characteristics and limitations

LLRCs are designed to be inundated from time to time. It is a basic characteristic of these structures that, when inundated, they are not available for use. It is necessary for the users of these structures to be aware of this limitation, and to accept and respect it. The second basic characteristic is that the cost of construction is generally considerably lower than that of a conventional bridge.

LLRCs are appropriate under the following circumstances:

- when the inundation of the structure, and the associated disruption in traffic flow, is acceptable for short periods of time;
- where alternative routes that can be used during flooding are available;
- where traffic volumes are low, typically on the tertiary road network;
- where high-level bridges are not economically justified; and
- where funding available for construction is limited.

Limitations of LLRCs, therefore, are:

- the fact that they are not available for use from time to time;
- that they have a risk associated with them in that road users may try to use them during periods of inundation, and be washed away; and that
- they may require maintenance after floods, e.g. to remove debris.

6.1.5.2 Road network aspects

The road network aspects to be considered are:

- If a particular community or land use has one access road only 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 cost and the economic justification of the options. Under low traffic volume conditions, say less that 500 vehicles per day, the return on investment of a LLRC is generally better than that of a high-level bridge. The reason is the considerably lower construction cost, while disruption in traffic is generally of a limited nature.

- If an alternative access route to a community or land use is available during periods of flooding, the impact of using the alternative route in terms of travel cost and travel time should be assessed in relation to the possible savings in construction cost associated with a LLRC.

With larger 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 periods. Rather than designing all river-crossing structures for the same return period, variation in the return periods used for design could be considered. In this way the number of accessible structures during flooding will be reduced, whilst alternatives will remain available. In contrast with the first option, a situation may occur where all the structures under consideration are overtopped at the same time. Generally, one would prefer structures on the primary road network not to be inundated during floods, while inundation of structures on the tertiary network is acceptable.

6.1.6 Key focus areas to be considered in the design of LLRCs

In the design and construction of any river-crossing structure a number of aspects need to be considered. These include:

- site selection;
- interaction with the river: hydrology and hydraulics;
- interaction with the road passing over the bridge: geometric alignment and width;
- interaction with road users: vehicles and pedestrians;
- interaction with the sub-soil: foundation conditions;
- elements of the structure itself, i.e.;
    - sub-structure (abutments, piers and foundations)
    - super-structure (deck , guide blocks, etc.)
    - approach fills and
    - erosion protection measures.

This manual is, in essence, a drainage guide. Although reference will be made to all of the above aspects, the emphasis will be on those that relate to drainage: i.e. location, hydrology and hydraulics, as well as erosion protection. For detailed guidelines on foundation investigations, structural design of bridges, etc, the reader is referred to documentation specifically focussed on those aspects.

6.2 SITE SELECTION

6.2.1 Straight river section

A LLRC should be located within a straight section of a river where the river flow is as uniform as possible. Riverbanks on the outsides of bends tend to erode which might lead to the floodwater by-passing the structure during flooding. Refer to **Figure 6.4** in this regard.

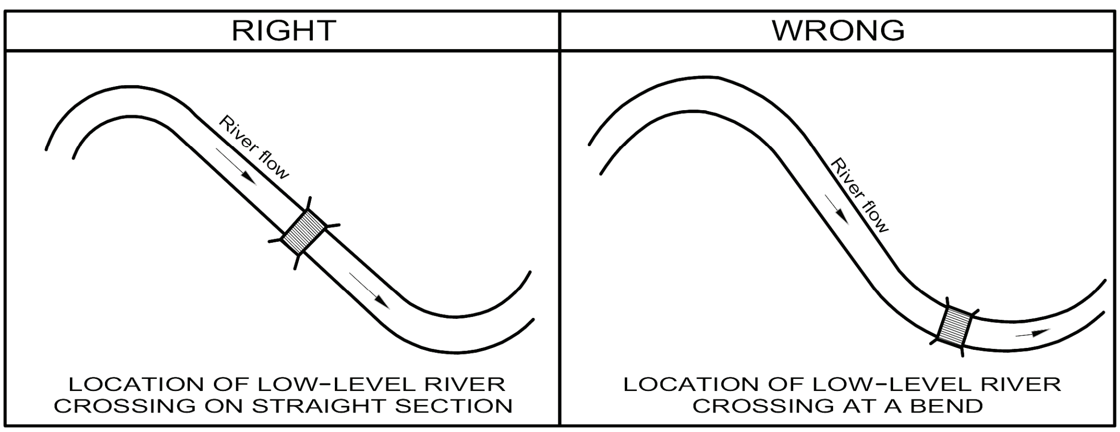


Figure 6.4: Location of LLRC in river

6.2.2 River channel cross-section

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 cost. Benefits also cover the possibility that the narrower section is associated with less weathered *in-situ* material, which may offer better founding 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. Refer to **Figure 6.5** in this regard.

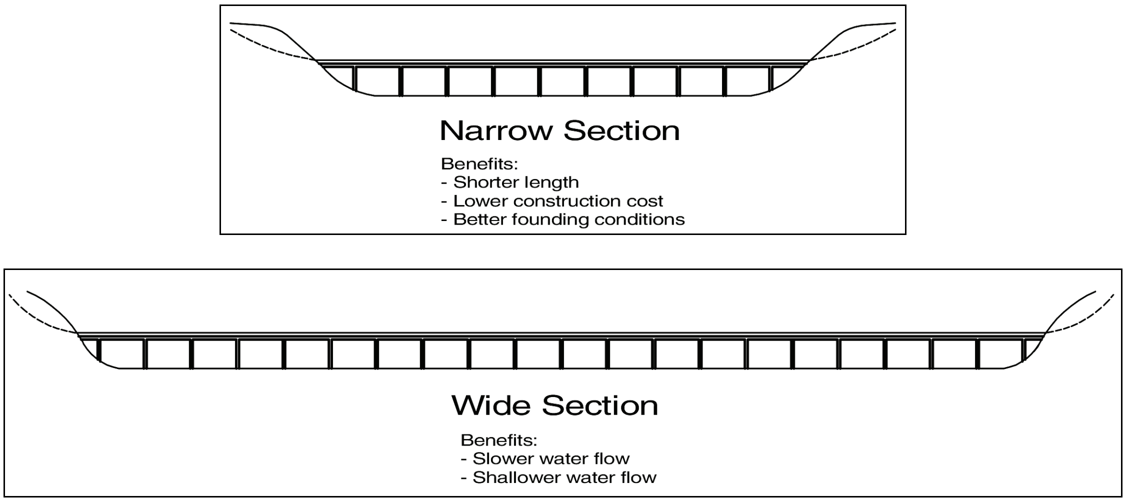


Figure 6.5: Wide versus narrow section in river

Generally speaking, a LLRC should be as low as possible to minimise the impact on water flow by the obstruction placed in the river channel. The depth of the natural river channel, compared to the width, should be considered. With narrow, deep sections the depth-to-width ratio may not allow the structure to be placed at a low-level due to the geometric alignment limitations of the road that crosses the structure. LLRCs are generally more suitable for river cross-sections with low depth-to-width ratios. Refer to **Figure 6.6** in this regard.

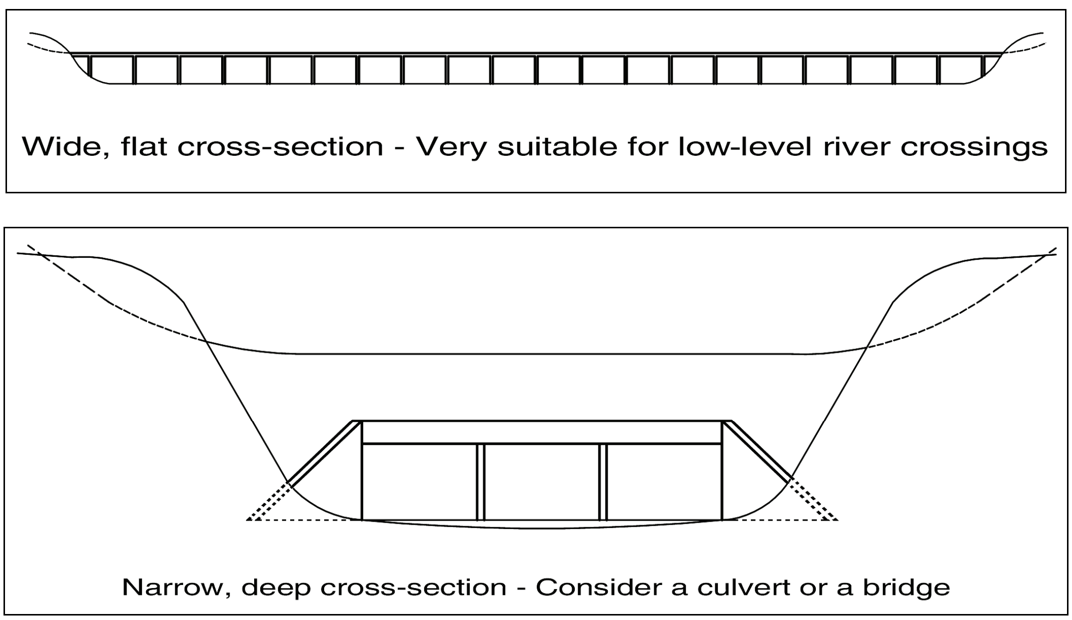


Figure 6.6: Effect of cross-section shape

Where the river cross-section consists of a channel located within a wider flood plain, it may be appropriate to place the structure at that height where overtopping of the structure occurs at the same time when river flow exceeds the channel capacity and extends onto the flood plain. The benefit of this approach is that the flow velocity over the structure is relatively lower. In such a case the access road should be designed as close to ground level as possible to minimise the obstruction to water flow. Attention should also be paid to erosion protection of the road. Refer to **Figure 6.7** in this regard.

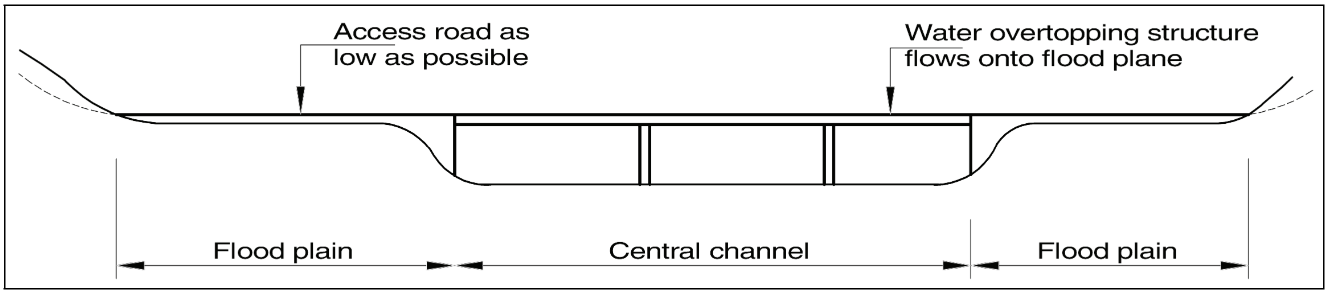


Figure 6.7: Effect of flood plain

6.2.3 Angle of crossing

Crossing of a river at an angle should be avoided. A skew approach, coupled with the possible blocking of openings with debris, tends to direct the full force of the river towards one of the riverbanks, which increases the possibility of the approach being washed away.

The structure should also be straight. A horizontally curved structure will be subject to similar problems of undesirable concentration of flow. Generally it is also extremely dangerous for road users to cross such a structure when inundated. Refer to **Figure 6.8**.

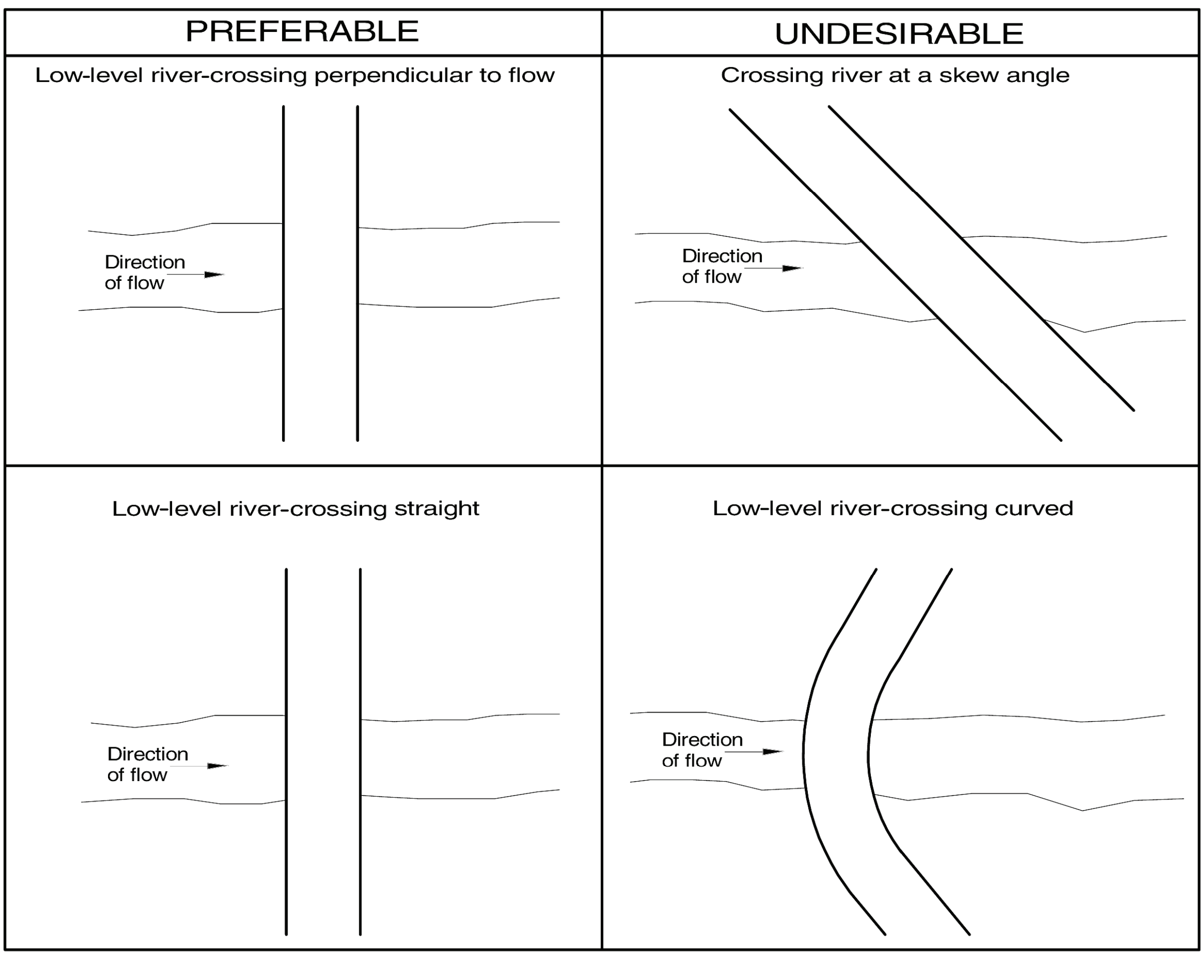


Figure 6.8: Avoid skew or curved LLRCs

6.3 HYDROLOGY

6.3.1 Methods to calculate run-off

Chapter 3 deals with a number of hydrological calculation methods that may be applied. With LLRCs a high level of accuracy in the determination of the design flood is not necessary, as the structure is designed to be over-topped.

Very often, therefore, only the rational method will be used, which is one of the most commonly used methods in South-Africa, especially if the catchment area does not exceed 500 km². If there is a need to determine the expected period of inundation, more sophisticated methods should be used, for example statistical analysis of nearby gauging station data.

6.3.2 Selection of appropriate return period

As discussed in **Chapter 3** of this manual, the return period is the average period over a large number of years during which an event (flood) repeats or exceeds itself. A flood is considered to be a mass wave that progresses along a watercourse and changes the water level, discharge, flow velocity and water surface slope over time at any specific location along the watercourse.

The decision on the return period to be used in the design of a LLRC is influenced by:

- the flow rate-time relationship that is characterized by the catchment's parameters and the storm event;
- the seasonal variation of the river flow (perennial rivers are less suitable for low-level structures);
- the classification of the associated road;
- the availability of alternative routes;
- the cost of the structure; and
- anticipated maintenance and repair costs after flooding.

6.3.3 Determination of design flow rate

The basic principle of a LLRC is that it should be provided at as low a level as is possible, in order to limit the impact of water flow on the structure under flooded conditions. However, there is also a limit as to how low the structure can be placed, depending on:

- geometric design parameters of the road crossing the LLRC; and
- the discharge that needs to be accommodated underneath the structure, or within acceptable depth over the structure, to provide a certain level of service and access.

The design approach for LLRCs is based on the definition of design levels, which provide an indication of the level of service to be provided by the structure. Three design levels, with the associated frequency and duration, are defined as indicated in **Table 6.2.** If, for example, design level 1 is selected the design flow could be expected to be exceeded 1,3 times per year on average, with the average flood duration (period during which the design flow is exceeded) being 9 hours. The range of these values as observed in practice is also shown (minimum and maximum values). **Table 6.2** is based on an analysis of river flow data over 20 years at 41 hydrological gauging stations located in the northern provinces of South Africa [6.5].

Table 6.2: Levels of design for low-level structures
(based on observed data from the Northern Province)

Design level	Dimensionless factor, f_i	Average no of times flow can be expected to be exceeded per year			Average length of period flow is exceeded (hours)		
		Min value	Max value	Average value	Min value	Max value	Average value
1	0,25	0,0	4,2	1,3	0,0	30	9,0
2	0,50	0,0	2,4	0,8	0,0	13	5,5
3	1,00	0,0	1,4	0,5	0,0	6	3,4

The **suggested approach** for the determination of the design level is as follows:

- Design level 1 is taken as the initial choice
- The design level is increased to design level 2 if:
  - traffic volume passing over the structure exceeds 250 vehicles per day;
  - the additional travel distance along an alternative route exceeds 20 km.
- The design level is increased to design level 3 if:
  - traffic volume passing over the structure exceeds 500 vehicles per day;
  - the additional travel distance along an alternative route exceeds 50 km;
  - there is no alternative route available.

The designer must use his judgement in applying this approach, and should clarify the selected design level with the owner of the structure.

Once the design level has been determined, the design discharge is calculated as follows:

$$Q_{design} = f_i \; Q_2 \qquad \ldots(6.1)$$

where:

Q_{design} = design discharge (m^3/s)
f_i = a dimensionless factor related to the design level chosen as shown in **Table 6.2**
Q_2 = discharge with a 1:2 year return period (m^3/s)

The structure should be designed in such a way that:

$$Q_{over} + Q_{under} \geq Q_{design} \qquad \ldots(6.2)$$

where:

Q_{over} = discharge that can be accommodated over the structure within the acceptable flow depth as defined below (m^3/s)
Q_{under} = discharge capacity of the openings through the structure (m^3/s), if any

With regard to flow depth, it is accepted that a vehicle should not pass over a LLRC if the depth of flow over the structure exceeds the under-body ground clearance height of the vehicle. The flow velocity, however, also needs to be taken into account. Design depth values are as follows:

- Supercritical flow: Maximum depth 100 mm
- Subcritical flow: Maximum depth 150 mm

6.4 SELECTION OF STRUCTURE TYPE

6.4.1 LLRCs and culverts

A decision needs to be made whether a culvert or a LLRC is more appropriate. The following guidelines are provided in this regard:

- The cost of a culvert should be compared to the cost of a LLRC. For small catchment areas (typically smaller than 10 km²) culverts tend to cost less. In the case of larger catchment areas, LLRCs generally offer the lowest cost solutions. Culverts or high level bridges will, however, be used in larger catchment areas where it is not possible to construct LLRCs, for example due to geometric road alignment limitations, the unacceptability of inundation during flooding or very poor founding conditions.

- As far at topography is concerned, culverts tend to be more appropriate in mountainous terrain where fills are required at low points in the road alignment. In rolling and flat terrain LLRCs are generally more appropriate.

- Culverts are designed for higher return periods than LLRCs. They will, therefore, be preferred on roads where occasional disruption of traffic flow due to flooding is not acceptable.

6.4.2 LLRCs and high-level bridges

The cost of bridge structures may form a significant portion of the cost of a road construction project. The construction costs of LLRCs are generally considerably lower than those of high-level bridges. In view of the rainfall characteristics and the tendency of short, high intensity rainfalls encountered in smaller catchments in South Africa, there is considerable room for cost savings by using LLRCs. The following guidelines are provided with regard to the choice between a high-level bridge and a LLRC:

- On high order roads (Road Classes 1 to 3)* high-level bridges are generally required because of the unacceptability of disruptions due to flooding.

- On lower order roads (Road Classes 4 to 6)* the cost of the LLRC option should be compared to the cost of the high-level bridge option. If there is a significant difference, a LLRC could be opted for, except in cases where it is not practical to construct a LLRC, for example due to geometric alignment constraints, or where inundation is unacceptable. Especially in the case of Class 4* roads, the comparative advantage needs to be discussed and agreed upon with the authority concerned.

- Where the additional cost of providing a high-level bridge is marginal, for example where the cost of the sub-structure is relatively high due to poor founding conditions, a high-level bridge may be warranted.

** For a description of the various classes of roads see **Chapter 8**.*

6.4.3 Road type

Any type of LLRC may be used with either paved or unpaved roads. In both cases it is important to ensure that the approaching road user is informed about the presence of such a structure, either by means of road signage, road layout (curved alignment) and/or good visibility of the structure.

6.5 HYDRAULIC CONSIDERATIONS

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, as discussed in Section 6.3.3.

6.5.1 Flow over the structure

The approach suggested is as follows:

- 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:

$$Q_{over} = \frac{A_{over}^{5/3} S_0^{1/2}}{n P_{over}^{2/3}} \qquad \ldots(6.3)$$

where:

	Q_{over}	=	the discharge that could be accommodated over the structure within the selected flow depth (m^3/s)
	A_{over}	=	area of flow over structure at the flow depth selected (m^2)
	S_0	=	slope in direction of flow, for example 0,02 or 0,03 m/m
	n	=	Manning n-value. For a concrete deck $n_{concrete}$ can be taken as 0,016 $s/m^{1/3}$
	P_{over}	=	wetted perimeter at the flow depth selected (m)

A_{over} and P_{over} are calculated as follows (Refer to **Figure 6.9a**):

$$A_{over} = A_1 + A_2 + A_3, \quad \text{or} \quad A_{over} = \frac{1}{3} d\sqrt{800 K_1 d} + dL_2 + \frac{1}{3} d\sqrt{800 K_3 d} \qquad \ldots(6.4)$$

and

$$P_{over} = P_1 + P_2 + P_3, \quad \text{or} \quad P_{over} = \frac{1}{2}\sqrt{800 K_1 d} + L_2 + \frac{1}{2}\sqrt{800 K_3 d} \qquad \ldots(6.5)$$

where:

A_1, A_2, A_3	=	the areas defined in **Figure 6.9a** (m^2)
d	=	depth of flow over the structure (m)
K1	=	the K value for vertical curve 1
K2	=	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.

The vertical road alignment, K (K_1 and K_3) should not be confused with K_{in1} and K_{out} on the following page. The symbol K is used because it is the symbol used in vertical road design methodology. Vertical design aspects are also addressed in Section 6.6.2.1 with values suggested for K.

Figure 6.9a also defines the symbols P_1, P_3 and L_2.

Establish whether the flow is indeed supercritical by calculating the Froude number, Fr:

$$Fr = \sqrt{\frac{Q_{over}^2\,B}{g\,A_{over}^3}} \qquad \ldots(6.6)$$

where:

B = $L_1 + L_2 + L_3$ (m), the width of the channel (or the length of the structure)
g = 9,81 m/s², the gravity constant

$$L_1 = \frac{1}{2}\sqrt{800K_1 d} \text{ and } L_3 = \frac{1}{2}\sqrt{800K_3 d}$$

Refer to the definition sketch (Figure 6.9a) for L_1, L_2 and L_3.

If Fr ≥ 1,0 it indicates that flow over the structure is supercritical at the selected flow depth, and that the methodology described here is applicable.

If Fr < 1,0 it indicates that flow over the structure is subcritical, and the analysis must be done as described elsewhere in this manual.

Note that in the calculation of the flow over the structure the effect of guide-blocks are for simplicity reasons ignored.

6.5.2 Flow passing through the structure

Assume outlet control (Refer to **Figure 6.9b** for a definition of the symbols):

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

$$Q_{under} = \overline{v}_{under} A_{eff} \qquad \ldots(6.7)$$

and where:

$$\overline{v}_{under} = \sqrt{\frac{H_1 - H_2}{\frac{C}{2g} + \frac{n_{eff}^2\,L_B}{R^{4/3}}}} \qquad \ldots(6.8)$$

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)
$\overline{v}_{under}$ = the velocity of flow through the structure (m/s)
C = factor that reflects the transition losses (Equation 6.13)

Determine the total energy height (H_1) 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 $\overline{v}_1 \approx 0$ m/s*

$$H_1 = h + x + D \qquad \ldots(6.9)$$

where:

x = the thickness of the deck (depending on the structural design outcome) (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}:

$$h = \frac{\bar{v}_2^2}{2g} + d \qquad \text{…(6.10)}$$

with:

$$\bar{v}_2 = \frac{Q_{over}}{A_{over}} \qquad \text{…(6.11)}$$

$$H_2 = D - L_B S_0 \qquad \text{…(6.12)}$$

where:

L_B	=	the total width of the deck of the structure (m)
S_0	=	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:

$$C = \sum (K_{inl} + K_{out})_{\text{each cell}} \qquad \text{…(6.13)}$$

K_{inl} and K_{out} are determined as follows for rectangular sections:

K_{inl} at outlet control:	Sudden transition:	K_{inl} =	0,5
	Gradual transition:	K_{inl} =	0,25
K_{out} at outlet control:	Sudden transition:	K_{out} =	1,0
	Gradual transition:	K_{out} =	1,0 for 45º < θ < 80º
			0,7 for θ = 30º
			0,2 for θ = 15º

where:

θ is the diversion angle.

Also refer to Section 4.3.4 where the energy principle is discussed.

n_{eff} is the effective Manning n-value for flow through the structure:

$$n_{eff} = \frac{\sum (n_{cell}\ P_{cell})}{P_{eff}} \qquad \text{…(6.14)}$$

and

$$n_{cell} = \frac{P_{concrete}\ n_{concrete}}{P_{cell}} + \frac{P_{river}\ n_{river}}{P_{cell}} \qquad \text{…(6.15)}$$

where:

$P_{concrete}$	=	the part of the wetted perimeter that has a 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}	=	Σ P_{cell} (effective wetted perimeter for the flow passing through the structure) (m)
$n_{concrete}$	=	the Manning roughness coefficient of concrete (s/m$^{1/3}$) (typically 0,016 s/m$^{1/3}$ - refer to Section 4.2.6 for more detail)
n_{river}	=	the Manning roughness coefficient of the river bed (s/m$^{1/3}$) (typically 0,03 s/m$^{1/3}$ - refer to Section 4.2.6 for more detail)
R	=	A_{eff} / P_{eff} (hydraulic radius (m))

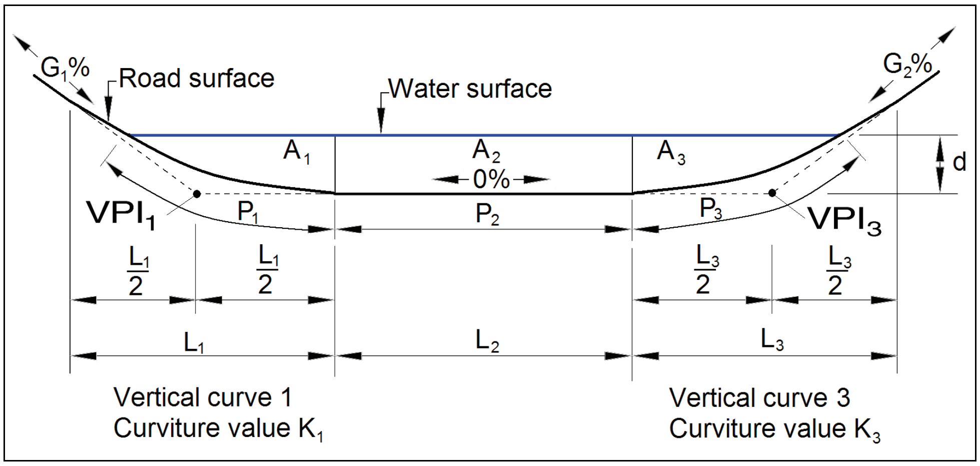


Figure 6.9a: Definition of symbols for the flow over the structure

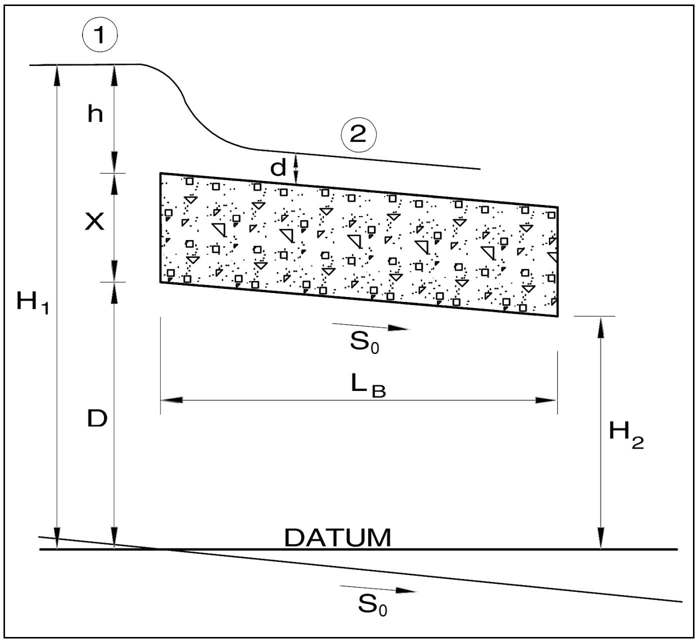


Figure 6.9b: Definition of symbols

6.5.3 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 and/or length of the structure would have to be increased, and the flow capacity be checked again.

6.6 STRUCTURAL DESIGN CONSIDERATIONS

6.6.1 Structural dimensions

6.6.1.1 Number of lanes

LLRCs are particularly appropriate on tertiary roads, characterised by low traffic volumes. As tertiary roads under most conditions have two lanes (one per direction) LLRCs will generally also be provided with two lanes. In certain instances, however, the provision of a single-lane structure may be justified. A single-lane structure may be considered where:

- The approach gradients are moderate and there is no significant curvature on the immediate approach roads.

- The LLRC is long and considerable savings associated with a single-lane structure are likely.

- There is good visibility on approaches to the structure, and LLRCs are infrequent on an otherwise good section of road.

- Traffic volume is not expected to exceed 500 vehicles per day during the life of the structure.

- Pedestrian volume is low, less than 100 pedestrians in the peak hour. For longer bridges provision can be made for a dedicated walkway with or without handrails. Details are provided in Section 6.6.1.6.

6.6.1.2 Width of structure

The following widths are recommended (**Figure 6.10**):

- Two-lane structure: 7,5 m between the guide-blocks
- Single-lane structure: 4,0 m between guide-blocks.

Widths of between 4,0 and 7,0 m should be avoided, as these widths create the impression that it is a two-lane structure, although vehicles cannot pass each other safely within this space.

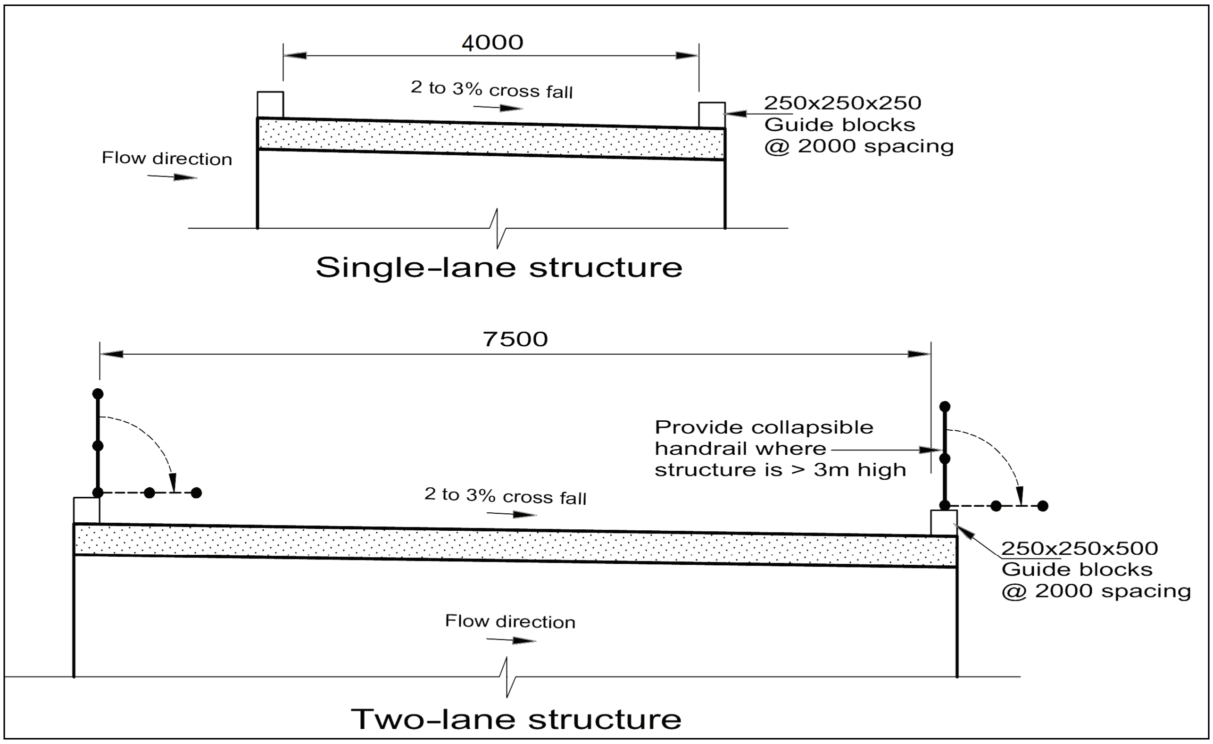


Figure 6.10: Width of LLRC

6.6.1.3 Cross-sectional and longitudinal slope of surface

A crossfall of 2 to 3% in the direction of flow is recommended to prevent sediment being deposited on the driving surface (Refer to **Figure 6.11**).

In the travel direction it is imperative that the structure is level (with the exception of short vertical curves at both ends of the structure). From the road user's perspective a variation in water depth when the structure is overtopped, is undesirable. From a hydraulic point of view concentration of water flowing over a LLRC is undesirable, especially if one end of the structure is lower than the other end (Refer to **Figure 6.11**).

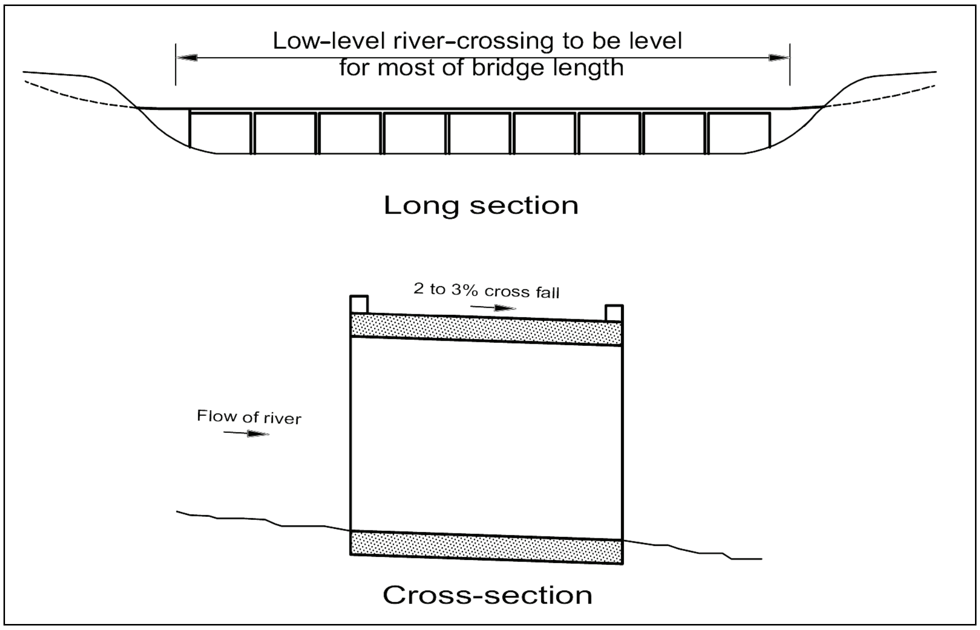


Figure 6.11: Cross-sectional and longitudinal slopes

6.6.1.4 Length of structure

It is imperative that the length of the structure equals the total width of the river. If the structure protrudes significantly above the river bed, the total length of the structure should be provided with openings underneath, so as not to create a dam wall across the river (Refer to **Figure 6.12**).

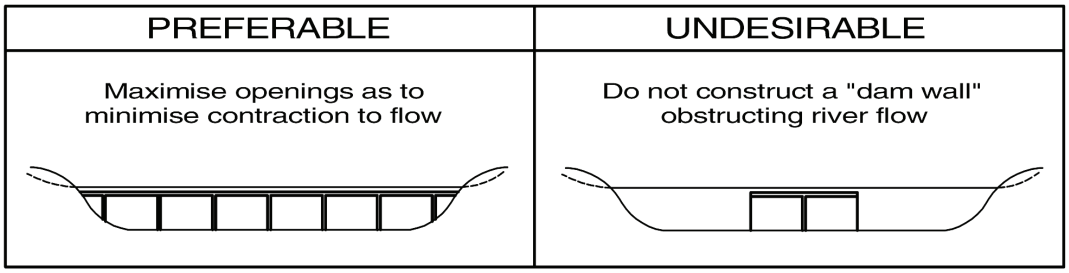


Figure 6.12: Cross-sectional area

6.6.1.5 Size of openings

The size of the openings through the structure depends on the design discharge. However, in the design of openings it is desirable to provide as much waterway area as possible within the river channel, so that the structure minimises obstruction of the river flow (Refer to **Figure 6.12**).

As a general guideline openings should be as large as possible to allow as much debris as possible to pass through the structure.

6.6.1.6 Guide-blocks and provision for pedestrians

Guide-blocks are to be provided on all LLRCs. Guide-blocks are placed along the edges of the deck with the objective of guiding the road user, and of assisting him in gauging the depth of flow over the structure. Guide-blocks should be designed to withstand debris blockages. Although a wide range of guide-block sizes and spacing are encountered in practice, the following are recommended: 250 mm blocks (cubical) at 2,0 m spacing. **Photograph 6.3** reflects the use of guide blocks.

Photograph 6.3: A low-level crossing near Steytlerville, showing the guide blocks

Where significant numbers of pedestrians are expected, especially on longer single-lane bridges, then separate provision should be made for the pedestrians by means of a dedicated walkway on the upstream side of the bridge deck. In addition, handrails that can collapse under larger floods should be provided where structures are higher than 3 m measured from top of deck level to river invert (also refer to Section 8.3 of **Chapter 8**). Such a bridge showing guide blocks, pedestrian walkway and collapsible handrail is reflected in **Photograph 6.4**.

Photograph 6.4: A low-level crossing on Tsitsa River with provision for pedestrians

6.6.1.7 Apron slabs and wing-walls

The use of apron slabs on both the upstream and downstream sides of a LLRC is recommended to absorb the energy of water flowing over the structure, and to protect the foundations of the structure. These slabs also contribute towards increased stability of water flows in the immediate vicinity of the structure. Apron slabs should have toe walls (cut-off walls) to prevent scouring and undermining of the slabs.

Apron slabs should preferably be built high enough to accommodate the 1:5 to 1:10 year flow. In remote areas where regular maintenance is not possible, consideration could be given to increase the return period that is used in designing the apron slab.

Aprons would normally be constructed using concrete. Reno mattresses could be considered under certain circumstances.

Wing-walls assist in directing water flow, and in protecting the abutments of the structure.

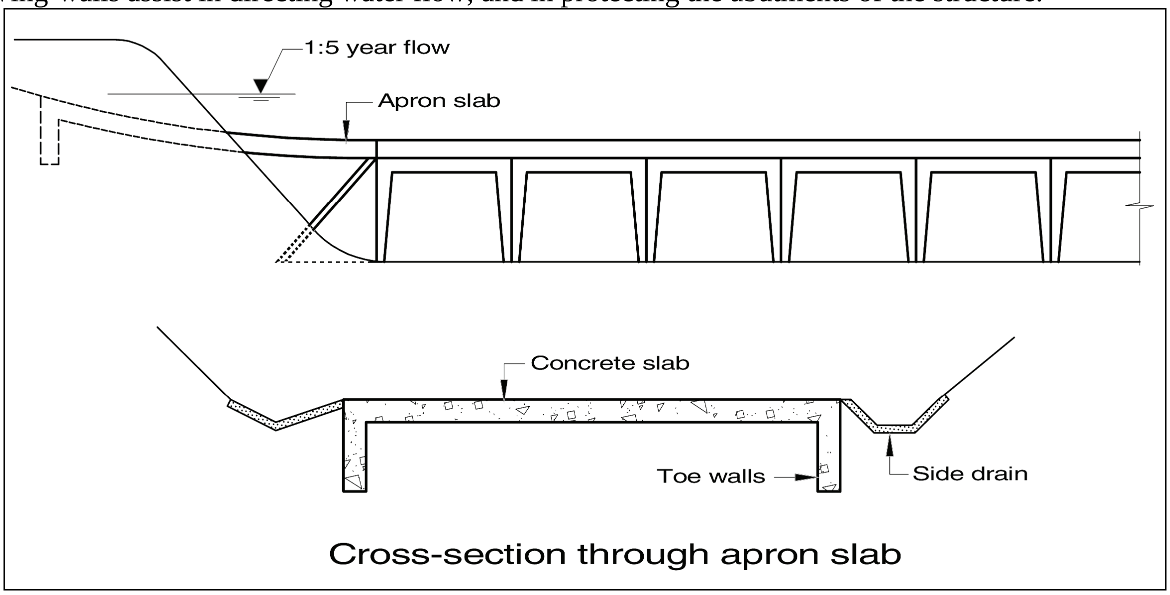


Figure 6.13: Apron slab on approach roads

6.6.1.8 Inclined buttresses on upstream side

It is considered good practice to provide LLRCs with inclined buttresses on the upstream side of each pier, to assist in lifting floating debris over the structure – not that this is always effective. It also provides additional stability in the case of debris loads. **Figure 6.14** shows an example of inclined buttresses.

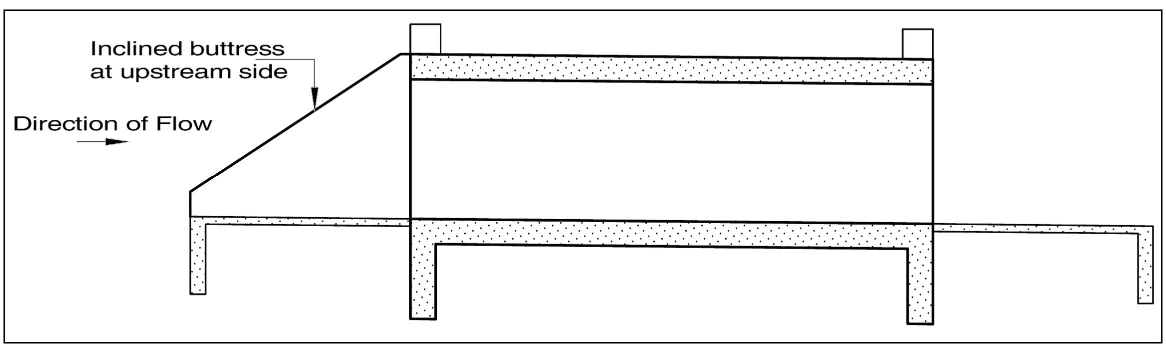


Figure 6.14: Inclined buttresses on upstream side

6.6.2 Road geometry

6.6.2.1 Vertical alignment

Generally, it is acceptable to use reduced geometric road design standards on the approaches to, and over LLRCs, provided that road users are warned; e.g. by means of speed reduction road signage, and provided that the environment changes sufficiently for them to realise that they are approaching a river. The maximum gradients suggested are shown in **Table 6.3a**.

Table 6.3a: Suggested maximum gradients for LLRC approach roads

Road type	Desirable maximum grade (%)	Absolute maximum grade (%)
Paved roads	10	12
Unpaved roads	8	10

Vertical curves in the alignment of the approach road are parabolic, as is normal for vertical road design. The **K-value** is defined as the horizontal length of road required for a 1% change in gradient. Minimum K-values are functions of design speed. Where practical, the design speed over a LLRC should be the same as for the associated road section. This is, however, often not possible. Where not possible, consideration may be given to lower the design speed over the LLRC, preferably by not more than 20 km/h, and in extreme cases up to an absolute maximum of 40 km/h. The necessary traffic signage must be provided. Typical values for sag vertical curves (as opposed to crest vertical curves which are not applicable to low level structures) are shown in **Table 6.3b**.

Table 6.3b: Suggested minimum K values for sag vertical curves

Design speed (km/hr)	K value
40	7
60	17
80	32
100	50
120	73

6.6.2.2 Horizontal alignment

The horizontal alignment of the approach roads should be designed according to the same standards as the road passing over the structure. The designer should specifically pay attention to the sight distance towards a LLRC, to ensure that the road user observes the structure in good time. Sharp horizontal curves in close proximity of a LLRC should be avoided for this reason.

6.6.3 Foundations

Ideally LLRCs should be founded on solid rock. Stiff clay may also offer suitable founding conditions, subject to the structure being designed to accommodate relative movement. Founding of LLRCs on unstable or sandy material is undesirable because of the tendency for riverbed material to become liquid under conditions of flooding. As a rule of thumb, one may assume that the depth of this instability may be as much as the depth of water flowing in the river channel at that point. The general scour equations, given in **Chapter 8** (Section 8.4.3), may also be applied.

Where rock is not available, raft foundations are recommended. The raft should consist of a slab (the floor slab of the LLRC) and toe walls on all sides of the slab to form a closed cell, as shown in **Figure 6.15**. The horizontal dimensions of the raft should generally not exceed 6 to 8 m. Toe walls should extend to non-erodable material, or to a depth equal to the depth of water flow with the 1:50 flood. Between the toe walls the raft should be filled with properly graded and compacted material which can withstand seepage water.

With longer structures multiple rafts are to be provided, which should then be secured to one another by means of dowels, for example. Refer to **Figure 6.15** in this regard.

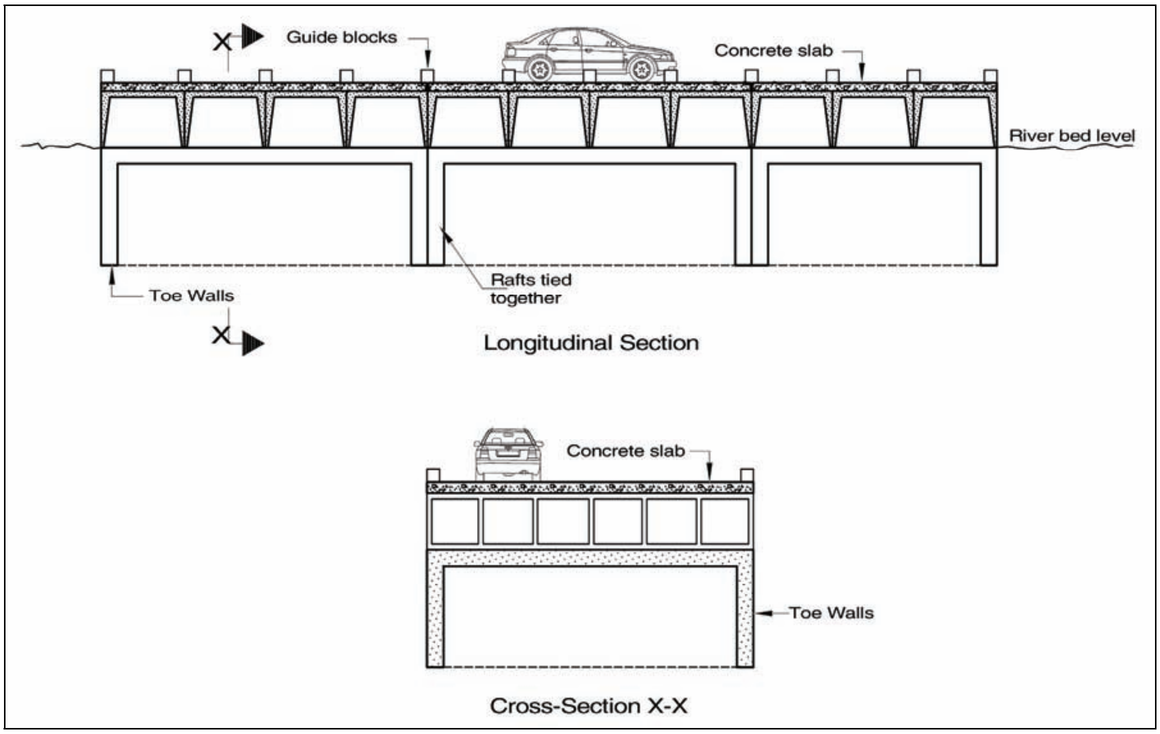


Figure 6.15: Raft foundation

Table 6.4 provides guidelines with regard to the effect of founding conditions on the selection of the LLRC structure type. Where more expensive foundations are required, for example piling, longer spans may be justified (8 to 15 m, versus the 6 m typically used in the case of good founding conditions).

Table 6.4: The selection of LLRC type according to founding conditions

Foundation condition	LLRC type		
	Drift	Causeway	Low-level bridge
Uneven rock	Unsuitable	Unsuitable	Very suitable
Even rock	Very suitable	Very suitable	Very suitable
Stiff clay	Suitable – use raft foundation	Suitable – use raft foundation	Spread footings, or piling/caissons may be required. Consider longer spans.
Sand	Consider a raft foundation	Consider a raft foundation	Piling/caissons required. Consider longer spans.

6.6.4 Structural design

6.6.4.1 Structural design approach

In the case of drifts and causeways it is advisable to limit the dimensions of units of construction to between 6 and 8 metres, to allow for thermal and other movement. These units should then be properly linked together by using dowels or keying them into each other.

In the case of low-level bridges (pier and deck construction) either a continuous beam approach or a simple supported slab approach may be used. It is important, however, to properly tie down the deck slab(s) to the piers by using dowels.

The use of I-beam slabs or slabs with voids that can trap air and induce buoyancy forces should be avoided.

Figure 6.16 shows various shapes, which could be considered for causeway openings. The use of arch shaped openings could be considered for the labour intensive construction option, as it generally only requires relatively simple steel reinforcement (refer **Figure 6.16**). The benefit of the arch type shown, compared to circular or semi-circular openings, is that it allows a higher percentage of openings underneath the deck, which reduces the extent of the obstruction to water flow.

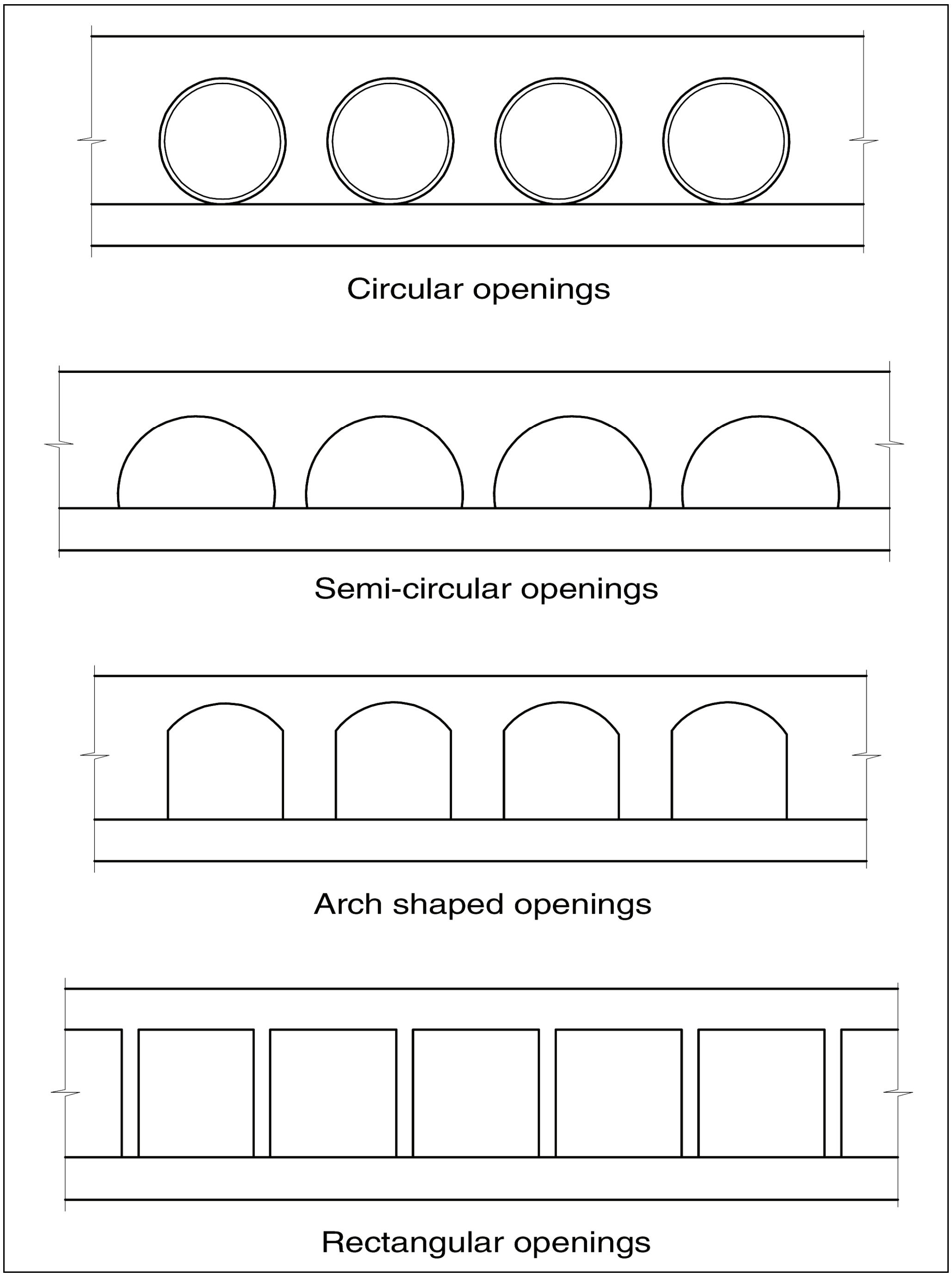


Figure 6.16: Various shapes of openings for causeways

With regard to the impact of the structure on water flow, it is recommended that the impact be kept as low as possible. Specifically, the ends of piers should be properly finished off to minimise obstruction to river flow.

6.6.4.2 Construction materials

The most commonly used material for LLRCs are concrete and stone masonry. The advantage of concrete is its durability and ability to withstand abrasion forces. The advantage of stone masonry construction is its suitability for labour-intensive construction.

In causeways pre-fabricated pipe and box-culverts are often used as void-formers. These elements have structural strength in themselves, but are relatively expensive. Where void-formers are totally encased in concrete or stone masonry work, consideration should be given to lower-cost options, such as corrugated iron void-formers. Armco-type elements should preferably not be used, based on cost considerations.

6.6.4.3 Structural loading

TMH 7 Parts 1 and 2: Code of practice for the design of highway bridges and culverts in South Africa (1981 as revised 1988) distinguishes between the following types of traffic loading on road bridges:

- normal loads (NA)
- abnormal loads (NB)
- super loads (NC).

The Code stipulates that while NC loading may be omitted on certain routes, all road bridges should be designed for both the NA and at least the NB24 load. These values are recommended for LLRCs, provided that the design engineer ascertains the applicability in each case. Obtaining the prior approval of the owner or responsible authority is considered essential.

6.6.5 Approach fills

The abutments of LLRCs should be keyed into the river banks where possible. In the case of flat river cross-sections the approach road should be constructed as close to natural ground level as possible, to avoid embankments being breached when overtopped by floods. Where the overtopping of embankments is likely, provision should be made for erosion protection by suitably cladding the upstream and downstream faces and the road formation. Flow depth and velocity for the 1:5 or 1:10 year flood should be determined, and provision should be made to accommodate their impact on the approach roads and structures.

In certain circumstances it may be necessary to protect banks downstream of the structure to a level calculated on the basis of a chosen design flood, preferably the 1:5 year flood.

6.6.6 Erosion protection

Because they are designed to be overtopped, erosion protection is particularly important with LLRCs. Attention should be given to:

- The downstream apron slab area;
- Approach roads to the structure; and
- The banks downstream of the LLRC.

Chapters 7 and **8** of this manual deal with this topic.

6.6.7 Safety

Driver safety and convenience should be addressed in the design of LLRCs. Human life may not be endangered. On the structure itself this can be accomplished by providing an adequate warning system in the form of guide-blocks.

Guide-blocks indicate both the limits of the structure and the depth of water flow over the structure. Guide-blocks are also spaced in such a way as to prevent a motor vehicle from sliding down the structure.

On the approach roads the necessary road signage should be erected, specifically some or all of these reflected in paragraphs 6.6.7.1 and 6.6.7.2 [(6.6)].

6.6.7.1 Warning signs

The following warning signs should be considered for LLC's:

- W350: Drift
- W326: Narrow bridge
- W327: One vehicle width structure
- W328: Road narrows both sides
- W202 and W203: Gentle curve
- W204 and W205: Sharp curve
- W348: Jetty edge or river bank
- W401 and W402: Danger plate

6.6.7.2 Regulatory Signs

The following regulatory signs could be applicable near LLC's:

- R1: Stop
- R6: Yield to oncoming traffic
- R201: Speed limit

6.7 OTHER CONSIDERATIONS

6.7.1 Community considerations

The perceptions of rural communities regarding LLRCs are sometimes not positive; the reasons being both the safety risks in the event of flooding and the lack of all-weather access. These concerns can only be addressed through proper communication and information being made available. Road users should be made aware of the safety risks and the need to use alternative routes during periods of flooding.

LLRCs offer more job creation potential during construction than most other elements of a roads project. This will benefit the local community in cases where workers are employed locally.

6.7.2 Labour-intensive construction

The suitability of LLRCs for labour-intensive construction methods is well known and is referred to in the text.

The use of local materials, specifically sand and stone for stone masonry construction, is encouraged, subject to it being environmentally and functionally acceptable.

6.7.3 Environmental considerations

Aspects to be addressed include:

- The environmental impacts of a LLRC, such as accelerated flow velocity, possible impact on the river banks and visual appearance.

- The need for regular maintenance, especially after periods of flooding, to remove debris, sand, and to repair minor damage.

6.7.4 Legal aspects

The legal aspects pertaining to drainage in general are covered in **Chapter 2**. With regard to LLRCs designs should be weighted against appropriate guidelines; such as those contained in this document. The acceptance of lower design standards than those indicated in the guidelines needs to be justified in terms of the risk involved and the financial and legal position of the "owner" of the structure.

6.7.5 Economic justification

Generally the level of economic justification of LLRCs is high when compared to high-level structures. The main reason for this is the relatively low construction cost of these structures, while the negative impact on road users is restricted to a maximum of a few days per year.

Costs to be considered in the economic evaluation of alternatives are:

- cost of construction;

- the useful life and cost of replacement or extension;

- the cost and inconvenience to road users because of delays in the case of the crossing not being accessible (with no alternative route available);

- the cost to road users associated with the use of alternative routes in the case of the crossing not being accessible (where available); and

- maintenance cost, especially maintenance and/or repairs after flooding.

The delays (length of period during which the design flow is exceeded) to be expected may be determined from **Table 6.2**. By considering the time cost of road users, the cost of delays can be quantified. The cost of using alternative routes may include direct travel cost, as well as time cost of road users. Maintenance costs would need to be estimated from experience, while road construction costs may be estimated quite accurately.

6.8 WORKED EXAMPLES

6.8.1 Problem description Example 6.1

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. Although the current traffic volume is 50 vehicles per day, it is expected to increase to 300 vehicles per day, should a proper river crossing structure be provided. The expected traffic growth rate for the next 20 years is 2% per year.

At the point of the crossing the river has a catchment area of 360 km². The 1:2 year flood has been determined as 120 m³/s.

Test pits were excavated in the sandy riverbed. Solid rock was encountered at depths varying between 1,2 m and 2,0 m. Rock is also day lighting in places.

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_2) has a length of 20 m, and K_1 and K_3 are both 4 m (refer to **Figure 6.9a**). 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 1 400 mm above the riverbed.

6.8.2 Solution Example 6.1

- **Design flow rate**

The design level is determined as per Section 6.3.3. Design level 1 is taken as the initial choice. Because of the expected traffic volume of 300 vehicles per day exceeding the suggested 250 vehicles per day, the design level is increased to level 2. This is supported by the availability of an alternative route of length less than the suggested 50 km. As the criteria suggested for design level 3 are not met, design level 2 is selected.

From **Table 6.2** follows that f_2 is 0,50. The design flow rate is determined from equation 6.1:

$$Q_{design} = 0{,}5 \times 120 \text{ m}^3/\text{s} = 60 \text{ m}^3/\text{s}$$

- **Cross-section**

With a design period of 20 years and 2% growth in traffic per year, the anticipated 300 vehicles per day is expected to increase to 446 vehicles per day after 20 years. Because of this being less than the suggested 500 vehicles per day and visibility being good, a single-lane structure is opted for (Section 6.6.1.1). 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.

- **Hydraulic calculations**

The capacity of the structure is determined as the sum of the flow that can be accommodated over the structure (Section 6.5.1) and through the structure (Section 6.5.2).

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 6.3. S_0 is 0,02 (2% as above) and Manning n for concrete is 0,016 s/m$^{1/3}$. The cross-section area of flow is determined as follows (equation 6.4):

$A_{over} = \frac{1}{3} d\sqrt{800 K_1 d} + dL_2 + \frac{1}{3} d\sqrt{800 K_3 d}$ where K_1 and K_3 are both 4 and L_2 is 20 m (equation 6.4)

A_{over} = 3,19 m^2

$P_{over} = \frac{1}{2}\sqrt{800 K_1 d} + L_2 + \frac{1}{2}\sqrt{800 K_3 d}$ = 37,89 m (equation 6.5)

From equation 6.3:

$$Q_{over} = \frac{A_{over}^{5/3} S_0^{1/2}}{n\, P_{over}^{2/3}} = \frac{(3,19)^{5/3}(0,02)^{1/2}}{(0,016)(37,89)^{2/3}}$$

Q_{over} = 5,42 m³/s

Establish whether flow is indeed supercritical by calculating the Froude number (equation 6.6):

B = $L_1 + L_2 + L_3$, $L_1 = L_3$ = 8,94 m and L_2 is 20 m, giving B equal to 37,89 m

Fr = 1,87, which is > 1,0 m, confirming supercritical flow over the deck of the structure.

Flow passing through the structure

Assume outlet control, then $Q_{under} = \bar{v}_{under} A_{eff}$

$\bar{v}_{under}$ is determined from equation 6.8, for which the following is required:

Equation 6.10: $h = \frac{\bar{v}_2^2}{2g} + d$, where $\bar{v}_2$ = Q_{over}/A_{over} = 1,70 m/s

h = 0,247m

H_1 = h + x + D, where h is as above, x = 0,5 m and D = 1,4 m
H_1 = 2,147 m

$H_2 = D - L_B S_0$, 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_{out} = 1,0 (both sudden transitions), then
C = 6 x (0,5 + 1,0), see equation 6.13.
C = 9

P_{cell} is the total wetted perimeter of each cell (m).
P_{cell} = (5,7)(2) + (1,4)(2) = 14,2 m

$n_{cell} = \frac{P_{concrete}\, n_{concrete}}{P_{cell}} + \frac{P_{river}\, n_{river}}{P_{cell}}$ and assume n_{river} to be 0,03 s/m$^{1/3}$

$$n_{cell} = \frac{(5{,}7 + (2)(1{,}4))(0{,}016)}{14{,}2} + \frac{(5{,}7)(0{,}03)}{14{,}2}$$

$n_{cell} = 0{,}022\ s/m^{1/3}$

$P_{eff} = \Sigma P_{cell} = (6)(14{,}2) = 85{,}2\ m$

$$n_{eff} = \frac{\sum(n_{cell}\ P_{cell})}{P_{eff}}$$

$$n_{eff} = \frac{(6)((0{,}022)(14{,}2))}{85{,}2}$$

$n_{eff} = 0{,}022\ s/m^{1/3}$

$R = A_{eff}/P_{eff}$ where
$A_{eff} = (6\ )(5{,}7)(1{,}4) = 47{,}88\ m^2$

$R = 0{,}562\ m$

$\bar{v}_{under}$ from equation 6.8 is:

$$\bar{v}_{under} = \sqrt{\frac{H_1 - H_2}{\frac{C}{2g} + \frac{n_{eff}^2\ L_B}{R^{4/3}}}} = \sqrt{\frac{2{,}147 - 1{,}31}{\frac{9}{2(9{,}81)} + \frac{(0{,}022)^2\ (4{,}5)}{(0{,}562)^{4/3}}}}$$

$\bar{v}_{under} = 1{,}34\ m/s$

Also from equation 6.7 $Q_{under} = \bar{v}_{under}\ A_{eff} = 64{,}16\ m^3/s$

Design discharge

The capacity of the structure at the design level $Q_{over} + Q_{under} = 69{,}6\ m^3/s$

As $Q_{over} + Q_{under}$ is larger than Q_{design} (60 m³/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.

6.9 REFERENCES

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CHAPTER 7 - LESSER CULVERTS (AND STORMWATER CONDUITS)

A Rooseboom and SJ van Vuuren

7.1 OVERVIEW

This chapter covers the practical aspects of culvert design, as well as the tools which are required for the hydraulic design of lesser culverts. The tools may also be used for the hydraulic design of storm water pipes and networks, as the same principles and formulae are applicable.

Low-level crossings, lesser culverts and storm water pipes, as well as bridges and major culverts have a great deal in common in terms of hydrological and hydraulical design calculations. The different hydrological design procedures being applied depend more on the catchment size than on structure type. In hydraulic design calculations for all these structures a distinction is drawn between upstream (inlet) and downstream (outlet) control. There are also, however, major and distinct differences between these types of structures and they are, therefore, dealt with in separate chapters.

The term “lesser culverts” refers to culverts that are small enough to be designed by means of simplified hydraulic and hydrological analyses. More sophisticated procedures used in the hydraulic design of bridges are also applied to major culverts. For a definition of bridges and major culverts refer to Section 8.1 of **Chapter 8**.

Lesser culverts generally have opening widths of less than 2,1 m or a combined opening less than 5 m^2. Judgement is required, however, as a very steep, long culvert of lesser width, for instance, may require more sophisticated hydraulic and hydrological calculations for optimal design.

Whereas earlier guidelines on freeboard for bridges and culverts gave staggered values, this manual prescribes freeboard in terms of design peak discharge, without drawing distinctions between lesser culverts, major culverts and bridges (refer Section 8.2).

Table 7.1 (Road Map 7) reflects typical problems that are associated with lesser culverts and which are addressed in this chapter.

Table 7.1: Road Map of lesser culverts (and storm water conduits)

ROAD MAP 7						
Typical theme		**Input information**	**Worked examples**		**Supporting software**	**Other topics**
Topic	**Par.**		**Problem**	**Example**		
Practical considerations	7.2	n/a				
Determination of the required culvert size	7.4	Design flow rate, maximum water level, control characteristics and available culvert sizes	Determine the required culvert size for a given flow rate	7.1	Utility Programs for Drainage & HEC-RAS	Flood calculations - **Chapter 3**
Flood attenuation at culverts	7.5	Inflow hydrograph, discharge characteristics and the storage height relationship	Level pool flood routing	7.2		
Erosion protection downstream of culverts	7.7	Flow velocities at the exit, geometric data and downstream normal flow depth	Selection of erosion protection measures downstream of a culvert	7.3		

7.2 PRACTICAL CONSIDERATIONS

7.2.1 Introduction

Most of the serious cases of water-related damage to culverts and endangerment of traffic in the past could be attributed to one or more of the following conditions:

- Scour on the downstream side of a culvert because of inadequate erosion protection and inadequate energy dissipation, which could ultimately lead to failure.
- Overtopping and scour on the downstream side resulting from supercritical flow approaching the culvert entrance at an angle, forming a hydraulic jump at the entrance and then overtopping the structure.
- Overtopping and scour of the embankment and surface layers, because of the blockage of culvert inlets by debris.
- Scour around inlets, often due to water from a wide flood plain flowing strongly alongside the road towards the culvert inlet.
- Piping where a flow path develops between the culvert structure and fill, especially where the fill material is dispersive (see **Photograph 7.1**, flow path through a culvert joint).
- Popping up of a light-weight (metal) culvert inlet section as a result of a high water table in the fill around the culvert.

Photograph 7.1: Piping (flow path through culvert joint eroding dispersive material)
(Courtesy of: MVD Consulting Engineers)

A number of practical measures may be affected to limit risk and damage.

7.2.2 Practical measures

A culvert serves to convey water from the upstream to the downstream side of a road. In flat areas where embankments are constructed mainly to provide vertical space for culverts, the optimum balance between fill costs and drainage costs should be sought.

Every natural watercourse reflects the prevailing pattern of equilibrium between flow and erosion processes. In good design this balance is disturbed as little as possible.

With regard to culvert design, this means that:

- Flow is concentrated as little as possible. Culverts should preferably not be further apart than 100 m.

- The direction of flow should be changed as little as possible. Oncoming flow approaching at an angle to the inlet reduces the capacity of a culvert. On the downstream side the flow should be released in the original direction of flow to prevent alteration of the erosion pattern.

- Water velocities should be altered as little as possible. Retardation of flow may cause the deposition of sediment, and acceleration may cause scour. Deposition should be prevented where it may lead to a reduction in the capacity of a culvert, especially inside the culvert. Flow velocities through culverts should, therefore, not be lower than 1 m/s, and the slope of a culvert should accordingly not be less than about 1%. Where the stream returns to a natural watercourse, its erosive capacity should not be significantly higher than under the original conditions. If necessary, the velocity of flow should be reduced by means of energy dissipaters.

It should always be borne in mind that supercritical oncoming flow must be allowed to pass virtually undisturbed, otherwise considerable damming and erosion may take place. Supercritical flow can only change direction very gradually unless it switches to subcritical flow.

If the oncoming flow is supercritical and it is uncertain whether or not damming may occur, the case should be treated as an inlet-controlled case or should be analysed by an expert. A culvert should be placed at the lowest point of each embankment. Since long-lasting damming may lead to the saturation of fill and foundation material, all low points need to be properly drained.

For a given head and inlet control, an H/D ratio of 1,2 approximately yields the optimum hydraulic section (maximum Q for a given head). Because this is also a good practical value from the point of view of preventing inlet erosion and determining the minimum height of an embankment over a culvert, it is the value generally aimed at in culvert design. However it must be noted that for a flood with double the return period the road, for Road Classes 1 to 3, should not be overtopped.

The minimum acceptable recommended practical size for a culvert up to 30 m long is 600 mm in diameter or a rectangular section of 750 mm (wide) x 450 mm (high), and for a culvert longer than 30 m a minimum diameter of 900 mm or a rectangular section of 900 mm (wide) x 450 mm (high) should be considered with the aim of being able to conduct maintenance.

Where a great deal of debris will be transported, the culvert should be large enough to allow it to pass, or otherwise a debris grid needs to be provided upstream of the culvert.

The available flow area through the grid should be at least four times that of the culvert. The loss of energy or extra head resulting from the presence of the grid may be put at $\frac{\bar{v}^2}{2g}$, with $\bar{v}$ = average water velocity through the grid with allowance being made for blockage.

Provision should be made to prevent culvert misalignment when differential settlement can occur, by way of upward cambers and or steeper culvert slopes. Differential settlement can be minimized when the culvert is installed after sufficient fill has been placed.

The large hydrostatic buoyant forces that may arise at culvert inlets should be taken into account in structural design. These arise when a culvert empties rapidly, but the surrounding soil remains saturated with water.

Where piping in soils may pose a problem, all culvert joints should be watertight, and collars, enclosed in impervious material, should be provided. In this case the H/D ratio for metal pipes should not be greater than 1.

In culvert design the water level at the design discharge may not rise higher than the shoulder, especially where the culvert inlet is on the outside of a curve with super elevation. The possibility of soil mechanical failure should be taken into account in the determination of maximum heads, especially in the case where non-cohesive materials have been used for fill. A flood of two times the design return period should be accommodated below the shoulder break point. This is a freeboard requirement as described in Section 8.3 of **Chapter 8**.

Photograph 7.2: Typical culvert with wingwalls

7.3 CULVERT HYDRAULICS

A distinction should be made between UPSTREAM (INLET) CONTROL and DOWNSTREAM (OUTLET) CONTROL. The former occurs most often and is preferred, due to the following:

- It yields the smallest culvert cross-section for a given upstream head; and
- The higher flow velocities through the culvert prevent the deposition of sediment inside.

The theory of culvert flow is relatively simple, but its strict application is difficult. Therefore, aids have been developed to simplify design calculations.

The basis of culvert design is that the energy equation is satisfied, together with the continuity equation.

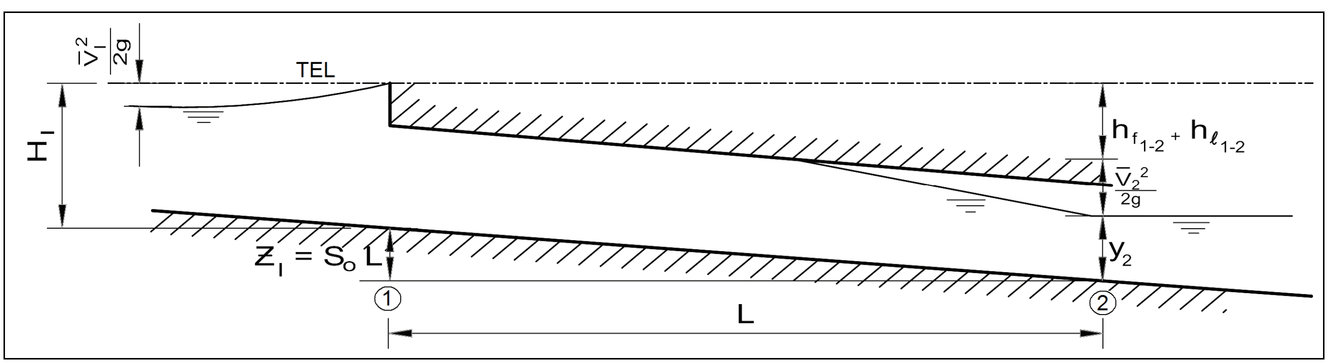


Figure 7.1: Energy components of flow through a culvert

Regardless of flow conditions through the culvert, the following equation must be satisfied:

$$z_1 + y_1 + \frac{\bar{v}_1^2}{2g} = z_2 + y_2 + \frac{\bar{v}_2^2}{2g} + h_{f\,1-2} + \sum h_{1\,1-2} \qquad \ldots(7.1)$$

where:

$h_{f\,1-2}$ = friction losses between cross-sections 1 and 2 (m)
z_1 and z_2 are the upstream and downstream bed elevations (m)
y_1 and y_2 are the upstream and downstream water depths (m), and
$\bar{v}_1$ and $\bar{v}_2$ are the upstream and downstream average flow velocities (m/s)

and

$\sum h_{1\,1-2}$ = transition losses between cross-section 1 and 2 (m)

It is convenient to set $z_1 = 0$, and then:

$$H_1 = H_2 + h_{f\ 1-2} + h_{1\ 1-2} \qquad \ldots(7.2)$$

Where:

H_1 and H_2 are the upstream and downstream energy levels, measured relative to the inlet invert level.

Because flow past a section is not possible with less specific energy than the critical value in open channel flow, this restriction often applies at the inlet.

In simplified culvert capacity determinations, for a given discharge Q:

- H_1 is calculated for full-flow conditions through the culvert (Outlet Control/Downstream Control).
- The minimum H_1 required at the inlet is calculated (Inlet Control/Upstream Control).
- The greater of these values of H_1 is accepted as representing the controlling flow level for the flow rate, Q.

Conversely, for a fixed value of H_1, the lower value of Q obtained by the two methods would apply.

Downstream (Outlet) control

In practice, the culvert should be full-flowing for at least part of its length for outlet or downstream control to apply.

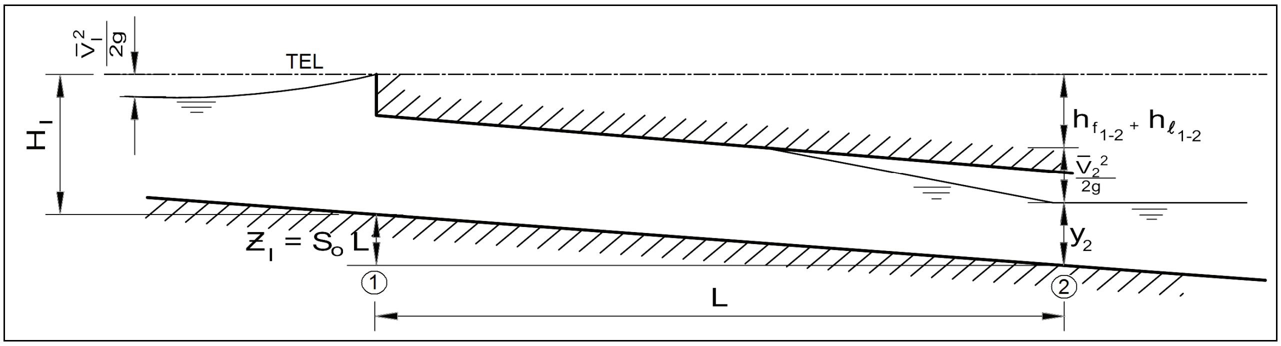


Figure 7.2: Section of a culvert partially flowing full

As before:

$$H_1 \text{ must be } = H_2 + h_{f\,1-2} + \sum h_{1\,1-2} - S_0 L \qquad \ldots(7.3)$$

H_2 must be determined beforehand according to downstream conditions.

By substitution in equation 7.3 for friction losses and transition losses, the following equations are obtained:

$$H_1 - H_2 = \frac{K_{in}\overline{v}_1^2}{2g} + \frac{K_{out}\overline{v}_2^2}{2g} + \frac{\overline{v}^2 n^2 L}{R^{\frac{4}{3}}} \quad \text{(Manning)} \qquad \ldots(7.4)$$

or

$$H_1 - H_2 = \frac{K_{in}\overline{v}_1^2}{2g} + \frac{K_{out}\overline{v}_2^2}{2g} + \frac{\overline{v}^2 L}{C^2 R} \quad \text{(Chezy)} \qquad \ldots(7.5)$$

Where K_{in} and K_{out} = inlet and outlet secondary loss coefficients.

The required roughness coefficients may be obtained from **Figure 4.8**.

If H_2 is known, as well as the dimensions and roughness of a culvert, the value of H_1 for a given discharge (Q) can be determined through step-by-step calculation.

These calculations have been performed with a value of n = 0,016 s/m$^{1/3}$, which is a realistic design value for concrete culverts, and the results are given in **Figure 7.4** (It has conservatively been assumed that the culvert is flowing full over its entire length.)

Upstream (Inlet) control

With upstream (inlet) control, flow passes from sub- to supercritical at the inlet and theoretically the Froude number is equal to 1, as depicted below.

$$\frac{Q^2 B}{g A^3} = 1 \qquad \ldots(7.6)$$

In order to determine H_1, allowance should be made for contraction effects and energy losses at the entry as shown in **Figure 7.3**. **Table 7.2** provides a summary of the equations for inlet control in round and rectangular culverts [(7.2)].

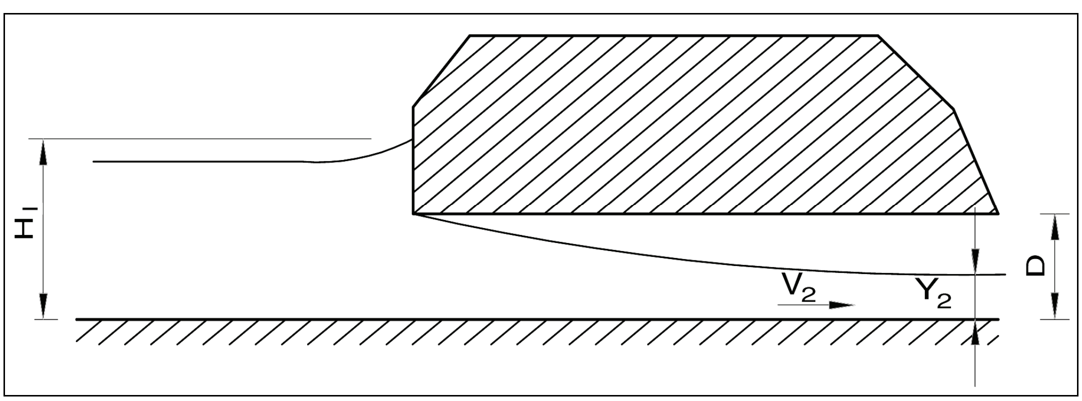


Figure 7.3: Flow through a culvert with upstream control

Table 7.2: Relationships for the flow rate under inlet control [7.2]

ROUND CULVERTS	RECTANGULAR CULVERTS
D = inside diameter (m)	D = height (inside) (m) B = width (inside) (m)
For : **$0 < H_1/D < 0{,}8$** $\frac{Q}{D^2\sqrt{gD}} = 0{,}48\left[\frac{S_0}{0{,}4}\right]^{0{,}05}\left[\frac{H_1}{D}\right]^{1{,}9}$	For: **$0 < H_1/D \le 1{,}2$** $Q = \frac{2}{3} C_B B H_1 \sqrt{\frac{2}{3} g H_1}$ Where: C_B = 1,0 for rounded inlets (r > 0,1B) C_B = 0,9 for square inlets
And for: **$0{,}8 < H_1/D \le 1{,}2$ *** $\frac{Q}{D^2\sqrt{gD}} = 0{,}44\left[\frac{S_0}{0{,}4}\right]^{0{,}05}\left[\frac{H_1}{D}\right]^{1{,}5}$ (S_0 = slope of culvert bed with slight effect on capacity) ***Note:*** ** For $H_1/D > 1{,}2$, the orifice formulae applies $Q = C_D A \sqrt{2g\left(H_1 - \frac{D}{2}\right)}$ with $C_D \approx 0{,}6$*	And for: **$H_1/D > 1{,}2$** $Q = C_h BD \sqrt{2g(H_1 - C_h D)}$ Where: C_h = 0,8 for rounded inlets C_h = 0,6 for square inlets

For a H/D ratio of 1,2 and with inlet control, the equations in **Table 7.2** are represented graphically in **Figure 7.4** (Inlet control procedure).

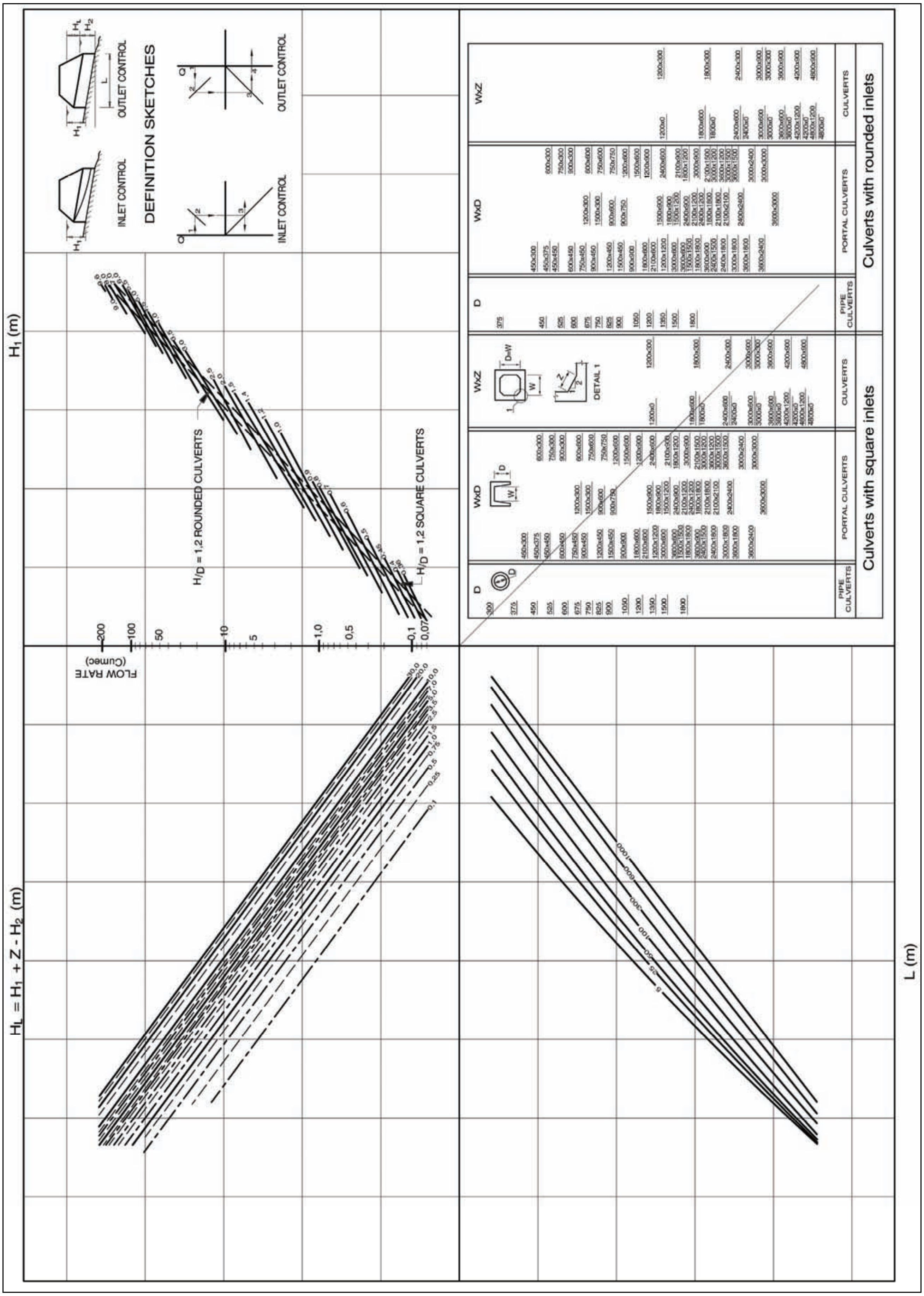


Figure 7.4: Diagram for the determination of sizes of culverts and storm water pipes

(A copy of this diagram has been included on the supporting CD)

7.4 DETERMINATION OF CULVERT SIZE

In the typical design problem, Q is known from hydrological calculations (**Chapter 3**), the maximum value for H_1 is known on the basis of the vertical road alignment and considerations of soil mechanics, and a suitable culvert has to be provided. The design return period is determined from the Q_{20} flow and from **Figure 8.2** in **Chapter 8**.

First of all, the maximum probable value of the downstream energy level, H_2, should be determined in accordance with downstream conditions (**Chapter 4 – Hydraulic Calculations**).

The sizes of culverts with adequate capacity may then be determined directly from **Figure 7.4**. **The size required for inlet control is determined first, and then the required size for outlet control. The larger calculated H_1 value from the two culvert sizes will apply, since it reflects the energy needed to get the flow into the culvert (inlet control) or the energy needed to maintain the flow through the culvert (outlet control).**

Especially in the case of very long culverts where inlet control applies, it is often possible to affect considerable savings by using a special inlet together with a smaller culvert section. In the following sections the notation is as follows:

y_n = normal flow depth (m)
y_c = critical flow depth (m)
S_0 = natural slope (m/m) and
S_c = critical slope (m/m), where Fr = 1

Inlets

Rounding-off of inlet corners normally only gives a small (5 to 10%) increase in culvert capacity. However, by adapting the inlet section the capacity of long culverts with inlet control can often be increased.

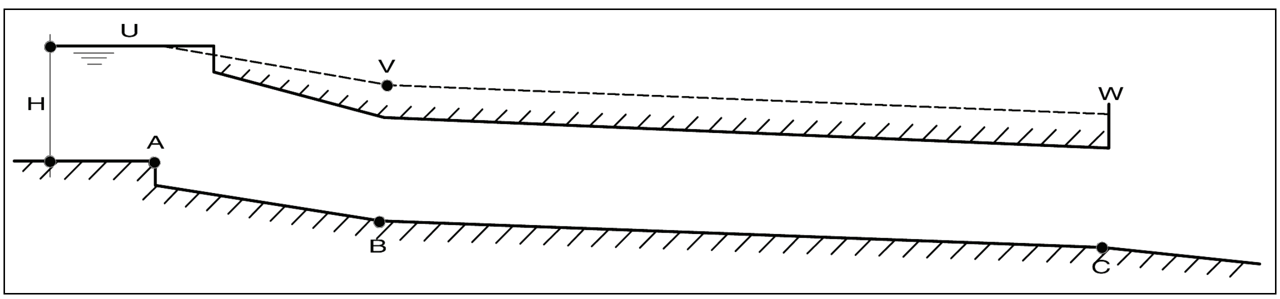


Figure 7.5: Steep culvert with dimensional and slope variations to increase the capacity (Changes at the entrance)

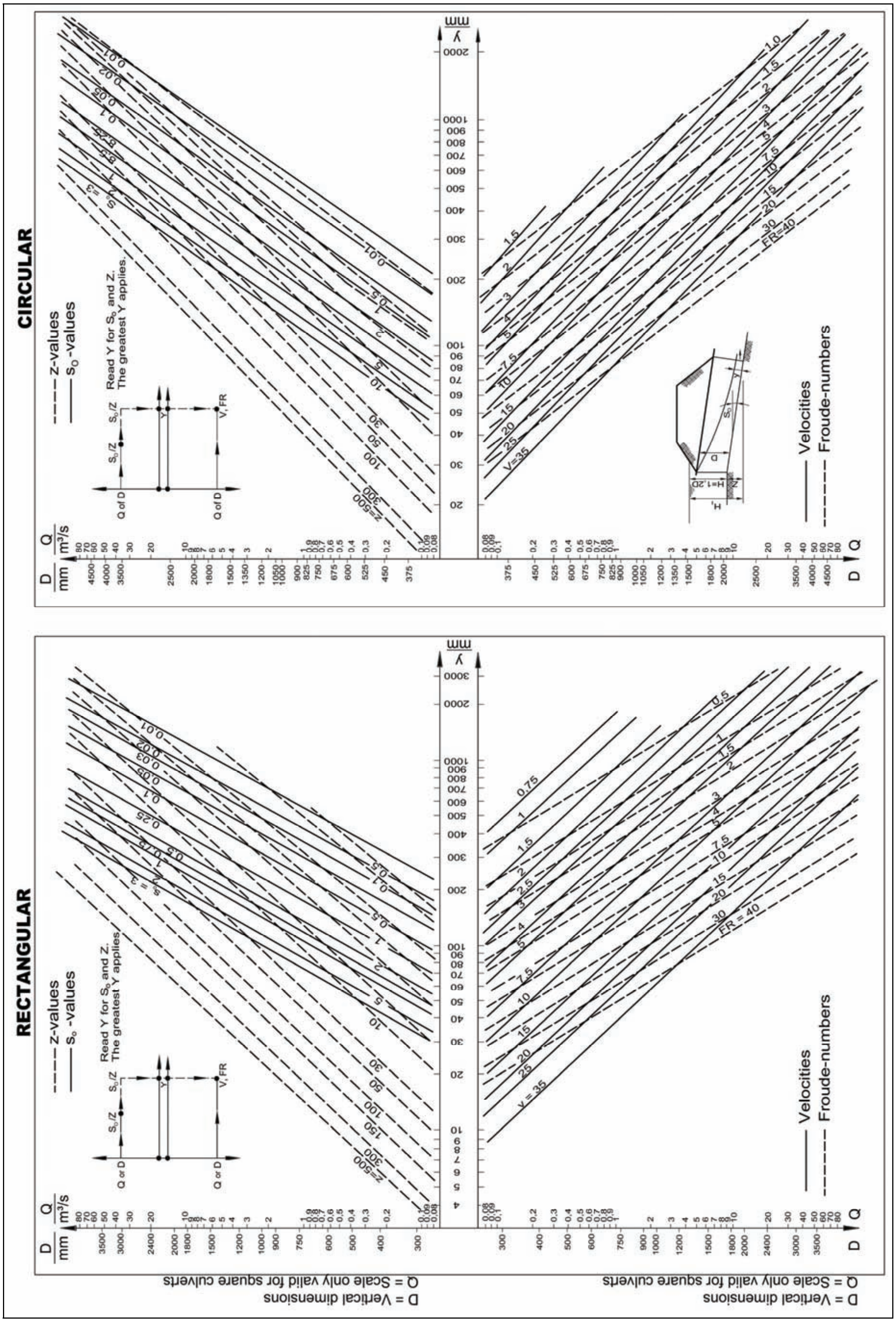


Figure 7.6: Diagram for the determination of outlet velocities for steep culverts and storm water pipes (Only $H_1/D = 1{,}2$, $S_o > S_c$)

(A copy of this diagram has been included on the supporting CD)

The slope downstream of B (control at the inlet), has to be hydraulically steep ($y_n < y_c$ or $S_0 > S_c$) and it should be ensured that the **specific energy values at all sections upstream of B are higher than those for critical conditions**. To achieve this it is necessary to increase the sectional area along the inlet portion.

The procedure for the determination of the layout for steep culverts is as follows:

As indicated in **Figure 7.5**, *A* and *C* are the natural levels at the culvert inlet and outlet.

Step 1:

For the given design discharge, Q and available head H_1, the required culvert section for inlet control is determined (**Table 7.1** or **Figure 7.4**). Select a smaller culvert section and determine the minimum slope required for the flow to be just sufficiently supercritical ($\frac{Q^2B}{gA^3} \approx 1{,}2$) <u>and</u> for the culvert to be almost full-flowing according to the Manning or Chézy equations.

Step 2:

Apply this section between *B* and *C* at the required slope.

Step 3:

Now draw in the energy line UVW (V and W are at a distance $y_n + \frac{\bar{v}_n^2}{2g}$ above the culvert bed, and *U* corresponds with the energy level at the inlet), see **Figure 7.5**. Gradually increase the culvert section upstream of *B* (make it wider and/or deeper) so that the distance (vertical) between the culvert invert and the energy line everywhere is:

$$\geq y_c + \frac{\bar{v}_c^2}{2g} \qquad \text{and} \qquad \frac{Q^2B_c}{gA_c^3} = 1$$

Step 4:

Ensure that the specific energy at the entrance where contraction occurs, is sufficient to pass the discharge through – see **Figure 7.4** or **Table 7.2**. If not, the inlet dimensions should be further increased.

If the invert level at the entrance is lowered beyond the natural ground level at the inlet, care must be taken that the overflow edge at *A* will be long enough for the specific energy above *A* to be enough ($H_1 \geq y_c + \frac{\bar{v}_c^2}{2g}$) to pass the discharge.

7.5 FLOOD ATTENUATION AT CULVERTS

When the stage storage relationship upstream from the reservoir is favourable for temporary storage, a smaller culvert section could be considered. The attenuation of the flow through the smaller culvert will reduce the peak discharge through the culvert. The effect of the temporary storage on the inflow hydrograph is not limited to the reduction of the peak flow rate (attenuation), but would also cause the peak flow to occur at a later point in time.

The relationship that forms the basis of flood attenuation calculations is the continuity equation. This is applied for a given time step, Δt over which the average inflow, $\bar{I}$, minus the average outflow, $\bar{O}$, should be equal to the change in the storage volume, ΔS. This can be written as follows:

$$\bar{I}\Delta t - \bar{O}\Delta t = \Delta S \qquad \ldots(7.7)$$

where:

$\bar{I}$	=	average inflow (m³/s)
$\bar{O}$	=	average outflow (m³/s)
ΔS	=	change in storage volume (m³) and
Δt	=	time step that is used (seconds)

This relationship may be written as:

$$\frac{I_1 + I_2}{2}\Delta t - \frac{O_1 + O_2}{2}\Delta t = S_2 - S_1 \qquad \ldots(7.8)$$

where:

$\frac{I_1 + I_2}{2}\Delta t$	=	average volumetric inflow (m³)
$\frac{O_1 + O_2}{2}\Delta t$	=	average volumetric outflow (m³)
$S_2 - S_1$	=	change in storage volume (m³)

This relationship may also be written as follows:

$$S_2 - S_1 = \frac{I_1 + I_2}{2}dt - \frac{O_1 + O_2}{2}dt \qquad \ldots(7.9)$$

If the known terms are grouped together it follows that:

$$\left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) = \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) + \frac{I_1 + I_2}{2} - O_1 \qquad \ldots(7.10)$$

It is helpful to develop a graphical relationship between the function $\left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right)$ and the outflow O_1 for a selected time step Δt. This is referred to as the auxiliary function, N vs O, which is used to simplify the solution of the outflow time relationship, as shown in **Figure 7.7**.

With $\left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) = N_2$ and $\left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) = N_1$, the solution could then be written as follows:

$$N_2 = N_1 + \frac{I_1 + I_2}{2} - O_1 \qquad \ldots(7.11)$$

Where:

N	=	auxiliary function (m³/s)

The inflow hydrograph is normally given or could be obtained from the methods given in **Chapter 3**. Furthermore, for a given stage at the culvert entrance the outflow rate, O, may be determined from the methods described in Sections 7.3 and 7.4. **Figure 7.7** reflects a typical example of an auxiliary curve.

Beginning at time T = 1, with a known outflow, O_1 and known inflow relationship, $I_{1,}$ I_2, I_3. ...., I_n and storage value, S_1 and the relationship for N_i determined from equation 7.10, N_2 can be solved and from the auxiliary function (Equation 7.11 or **Figure 7.7**), O_2 can be determined.

In summary the level pool routing analysis is performed as follows:

- Using hydrological calculations (**Chapter 3**) obtain an inflow hydrograph (discharge - time curve) for run-off from the culvert catchment.

- Determine the storage volume (S) for different water levels at the site, and assume that the storage volume = 0 at the invert level of the culvert. (Assume that any storage volume below this invert will not result in any attenuation.)

- Select a culvert with suitable dimensions smaller than those required for the peak discharge according to **Figure 7.4** (because temporal storage will result in a smaller outflow rate).

- Determine the outflows for different heads in accordance with **Figure 7.4**.

- Draw a graph showing the relationship between outflow (O) and the auxiliary function

  $$N = \frac{S}{\Delta t} + \frac{O}{2} \qquad \ldots(7.12)$$

  where:

S	=	temporal storage or ponding volume (m³)
Δt	=	selected time increments (say, 0,1 of time from the start of the inflow to the peak inflow (time of concentration or the lag time)), seconds
O	=	outflow through culvert (m³/s)

- For each time increment Δt, calculate

  $$\frac{I_1 + I_2}{2} - O_1 = \Delta N \qquad \ldots(7.13)$$

  where:

  I_1 and I_2 = the inflows into the ponded area at times t_1 and t_2 respectively with ($\Delta t = t_2 - t_1$) and O_1 = the outflow at t_1 (beginning of time step Δt).

- Calculate successive values of N by putting $N_2 = N_1 + \Delta N$. Corresponding values of the outflow, O, for known N-values, can be read off from the graph.

- When the maximum discharge has been determined, the maximum head (stage at the culvert inlet) may be calculated by means of the appropriate equations in Section 7.3 or interpolated from **Figure 7.4**.

If necessary, new culvert dimensions are selected and the process is repeated until a satisfactory head is obtained.

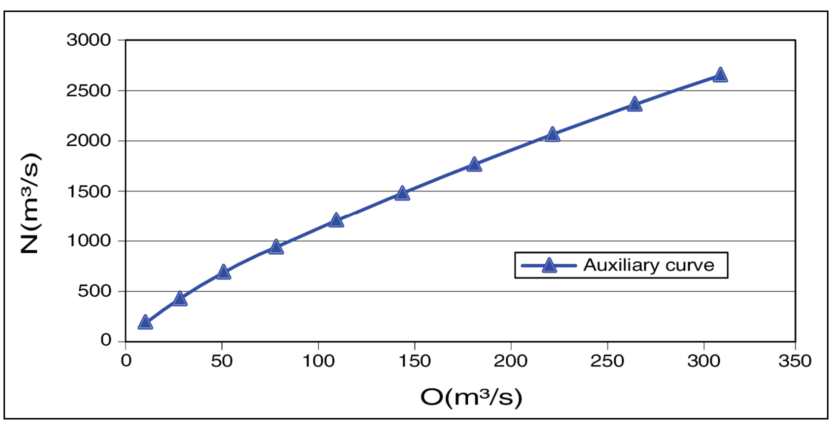


Figure 7.7: A typical example of the auxiliary function

7.6 EROSION PREVENTION UPSTREAM OF CULVERTS

Erosion is generally found where:

- water velocities are high, and
- the direction of flow changes rapidly.

When conventional inlets with wing-walls (standard design) are used, scour upstream of culverts is rarely a problem. The provision of pitching over a distance of twice the vertical dimension upstream of the culvert inlet (or upstream of the concrete slab between the wing walls), with stones 200 mm in size is usually sufficient [7.1].

Where problems are expected, additional protective measures may be required (**Chapter 8**).

7.7 EROSION PREVENTION DOWNSTREAM OF CULVERTS

Before erosion protection at a culvert outlet can be designed, the water velocity at the outlet needs to be determined. (The depth of flow may then be calculated from the continuity equation.)

Outlet velocities

In the case of **outlet control**, the downstream flow depth determines the sectional area at the outlet and the corresponding velocity can be calculated for a given value of Q with $(V = \frac{Q}{A})$.

More important is the case in which inlet control applies and the flow is supercritical at the outlet (upstream control).

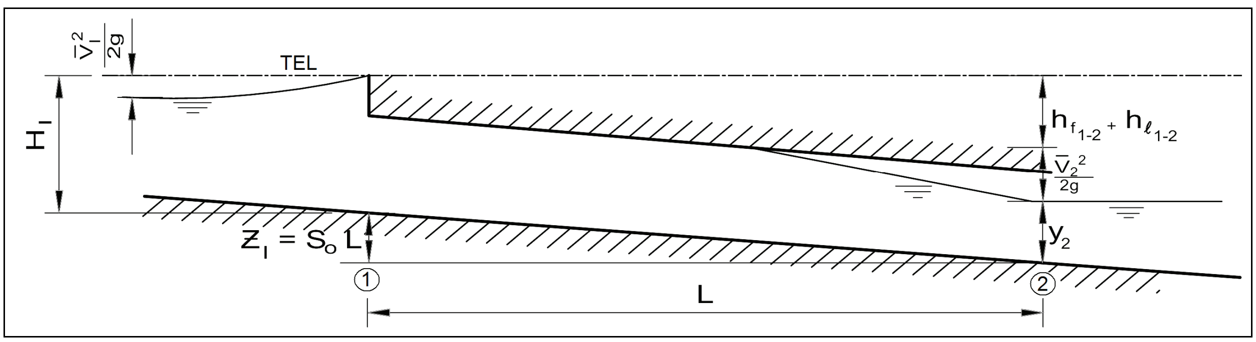


Figure 7.8: Definition sketch of the energy components through a culvert

For section 2 (just inside or outside the culvert) the following equation applies in the case of non-submerged conditions (free outflow):

$$H_1 + z_1 = y_2 + \frac{\bar{v}_2^2}{2g} + h_{f_{1-2}} + h_{1_{1-2}} \qquad \text{...(7.14)}$$

Once again, h_f can be calculated according to the Manning or Chézy roughness coefficients, and h_1 equals the entry loss at section 1.

Outlet velocities just inside or outside the culvert outlet may be read off from **Figure 7.6** when inlet control applies. **Figure 7.6** is based on a conservatively low value for n of 0,014 s/m$^{1/3}$ to allow for the most unfavourable conditions (high velocities). The outlet velocity cannot be higher than that given by the Manning equation for uniform flow through the culvert, and in the case of steep slopes is limited by the difference in height between the inlet and the outlet, while the equilibrium uniform velocity has not yet been reached).

Protective measures

The primary aim of downstream erosion protection is to release water into the natural channel at a velocity no greater than the original velocity and in the same direction.

Energy dissipation normally takes place downstream of the culvert, but may also take place in steps inside the culvert.

For effective energy dissipation and erosion protection downstream of the culvert, the following alternatives are available for consideration:

- Grass covers (Refer to Section 5.4.5).
- Types II, III, IV of stone pitching as developed by the US Waterways Experiment Station[(7.3)] (**Figure 7.10**).
- Type V stilling basin with baffle developed for culverts at the University of Pretoria from the US Bureau of Reclamation's stilling basin for dam outlets[(7.4)] (**Figure 7.11**).

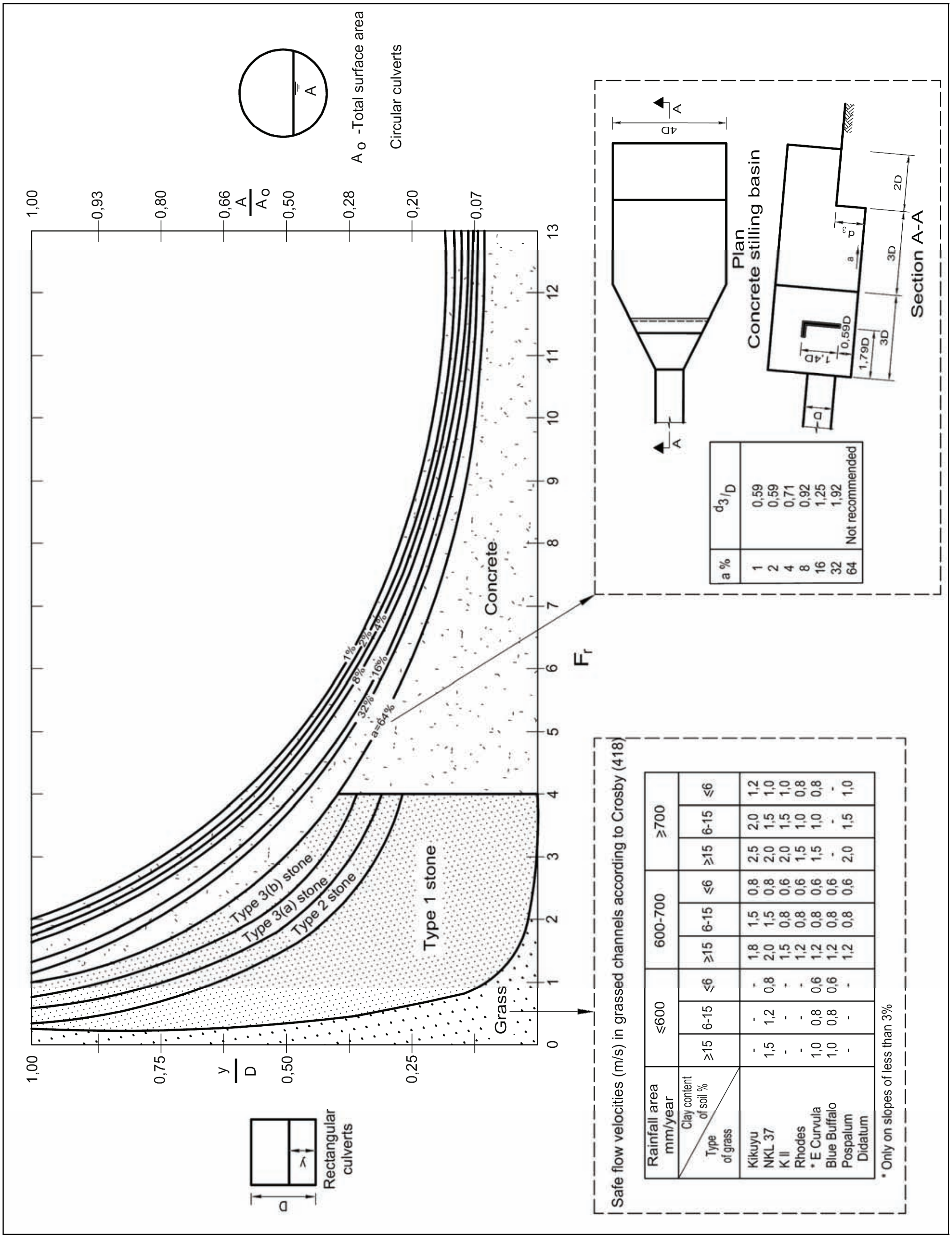
Taking into account different aspects, such as maximum permissible water velocities, maximum stone sizes that can be handled by labourers, and relative costs, **Figure 7.9** approximately indicates the optimum solutions for different flow conditions.

The following procedure is used when applying **Figure 7.9**:

- Determine the outlet velocity at the entrance to the stilling basin in accordance with **Figure 7.6**.
- Calculate the depth of flow from the continuity equation.
- Calculate the Froude number, *Fr*, as well as $\frac{y}{D}$ where *D* is the height or diameter of the culvert and *y* is the outlet flow depth.
- Read off which type is the most suitable, and obtain the required dimensions from the detailed data.

If another type of protection is preferred, it has to be checked to see whether it may be used under the prevailing conditions.

Where a grass cover is used, the outlet should be above ground level and if necessary, the area around the outlet should be paved to prevent grass and/or deposits of sediment from blocking the outlet.



Rainfall area mm/year	≤600			600-700			≥700		
Clay content of soil % / Type of grass	≥15	6-15	≤6	≥15	6-15	≤6	≥15	6-15	≤6
Kikuyu	-	-	-	1,8	1,5	0,8	2,5	2,0	1,2
NKL 37	1,5	1,2	0,8	2,0	1,5	0,8	2,0	1,5	1,0
K II	-	-	-	1,5	0,8	0,6	2,0	1,5	1,0
Rhodes	-	-	-	1,2	0,8	0,6	1,5	1,0	0,8
* E Curvula	1,0	0,8	0,6	1,2	0,8	0,6	1,5	1,0	0,8
Blue Buffalo	1,0	0,8	0,6	1,2	0,8	0,6	-	-	-
Pospalum Didatum	-	-	-	1,2	0,8	0,6	2,0	1,5	1,0

a %	d_3/D
1	0,59
2	0,59
4	0,71
8	0,92
16	1,25
32	1,92
64	Not recommended

Figure 7.9: Limiting values for different methods of erosion protection at culvert outlets
(A copy of this diagram has been included on the supporting CD)

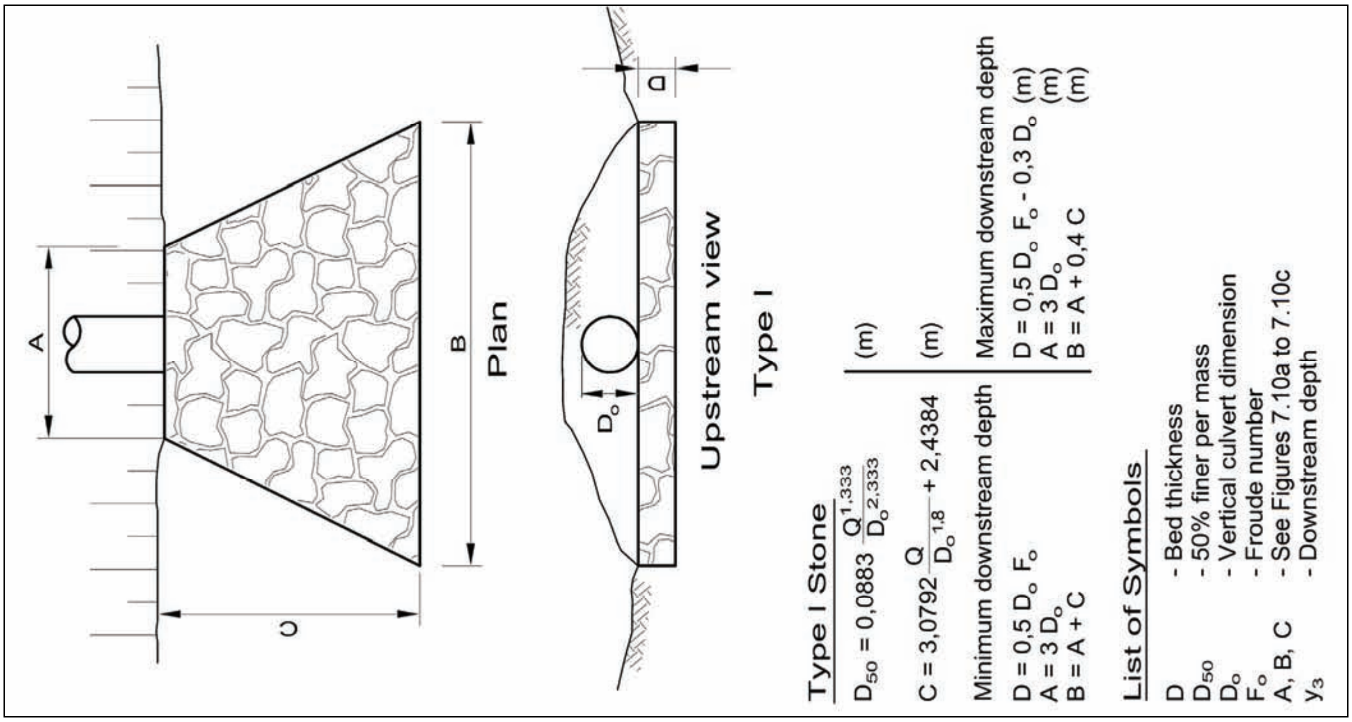


Figure 7.10a: US Army waterways stone stilling basins – Type 1

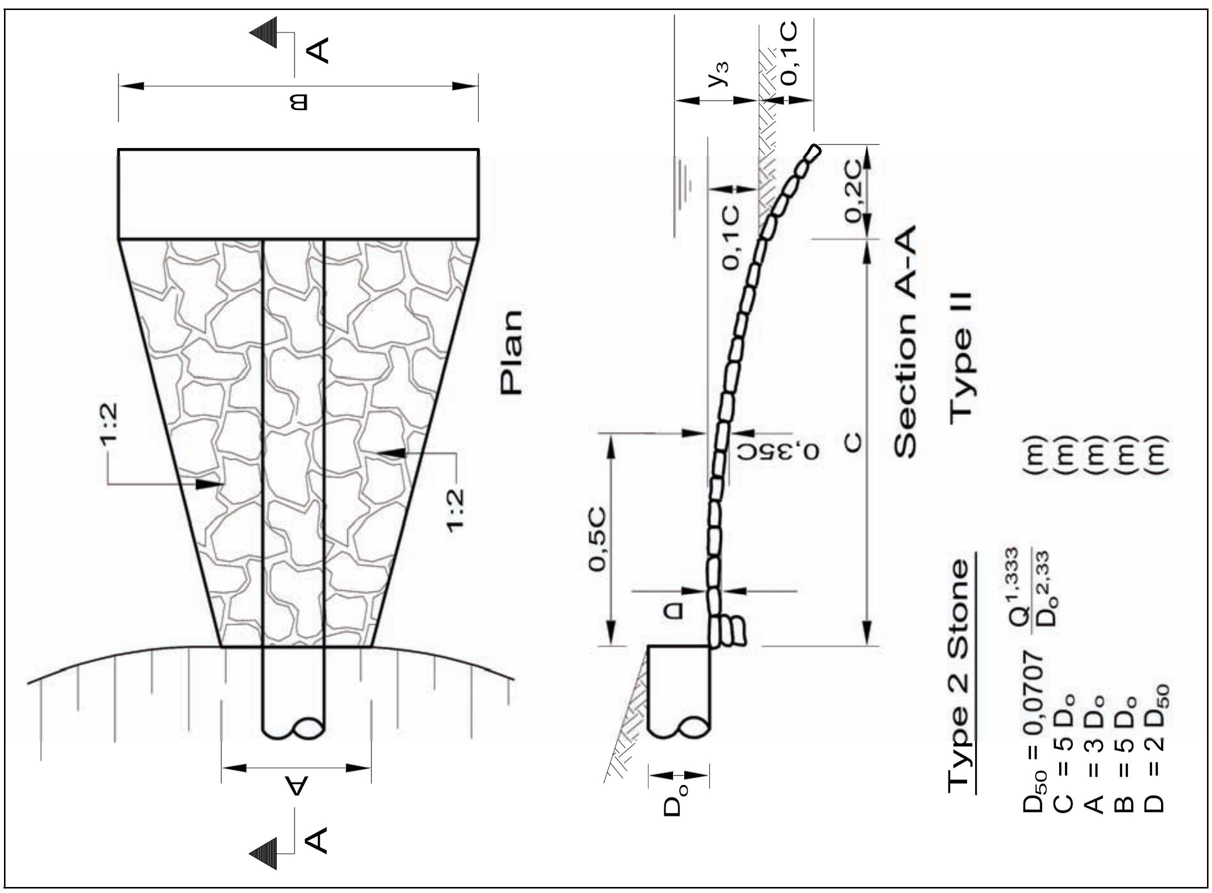


Figure 7.10b: US Army waterways stone stilling basins – Type II

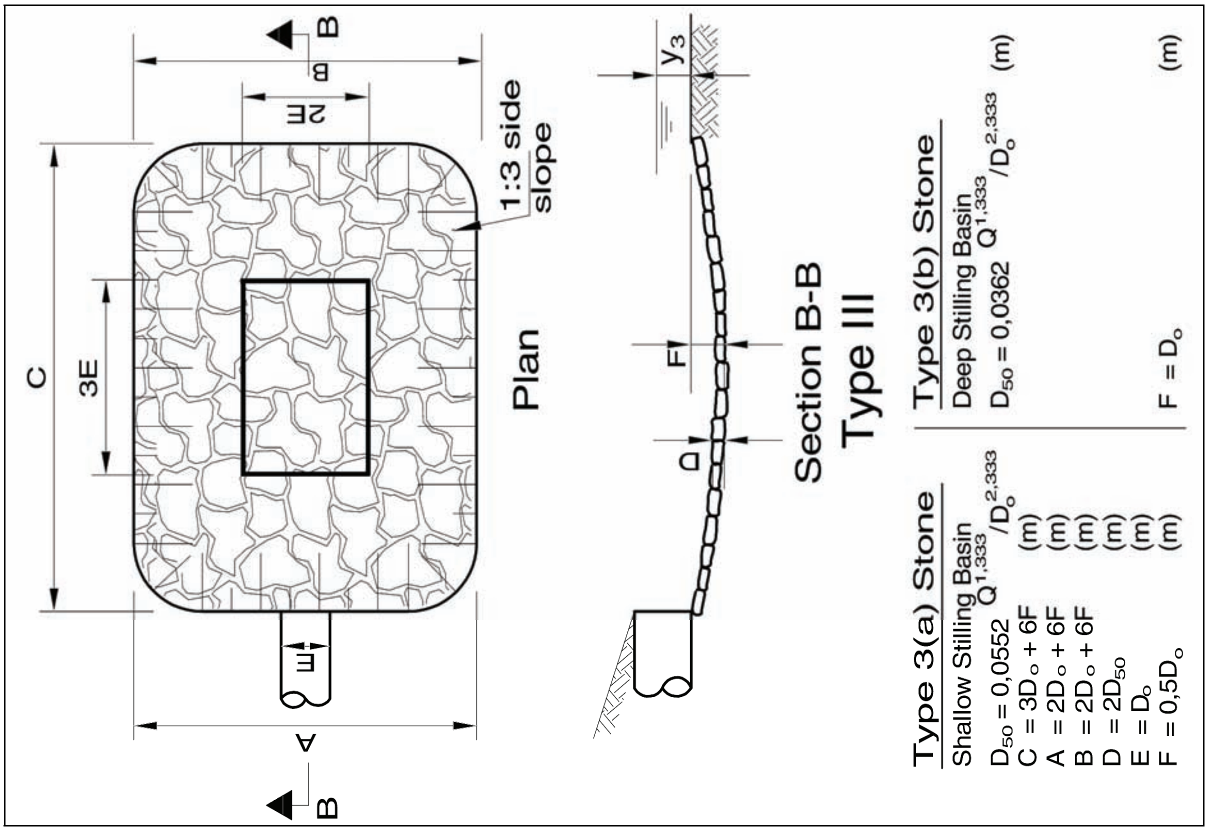


Figure 7.10c: US Army waterways stone stilling basins – Type III

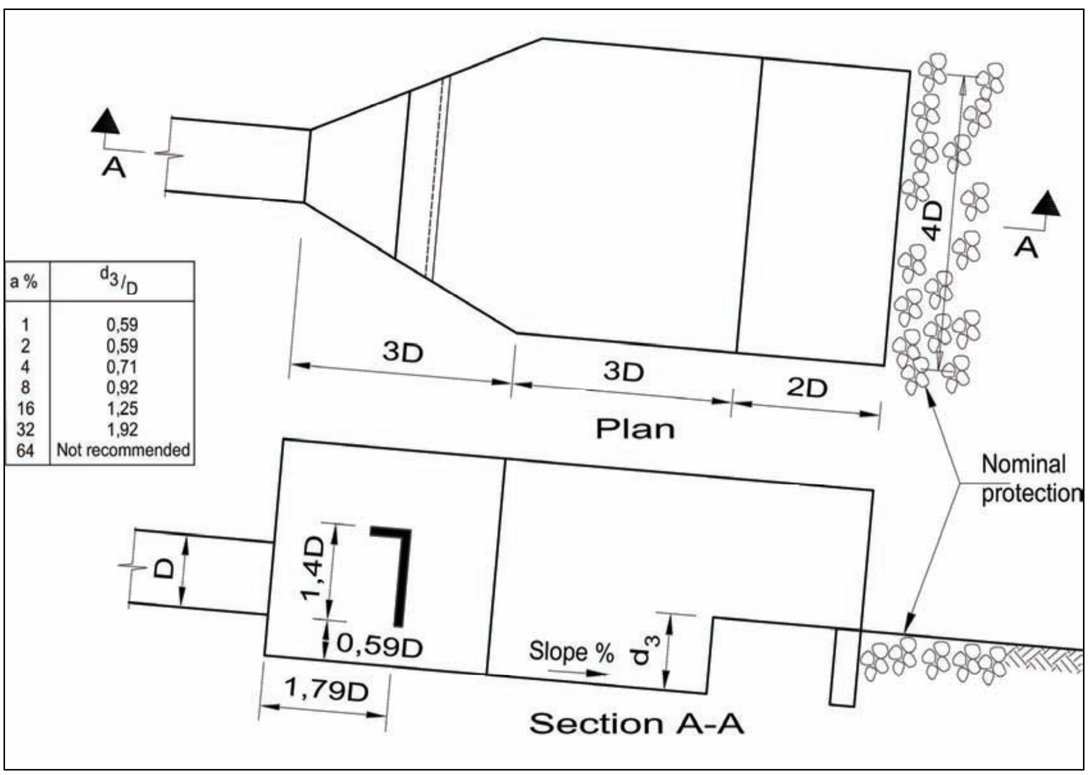


Figure 7.11: Type V Stilling Basin

7.8 WORKED EXAMPLES

7.8.1 Example 7.1 - Determination of the required culvert size

Problem description Example 7.1

A culvert size needs to be determined which would handle the design flood (Q_D). The calculated 1:20 year flood (Q_{20}) is 98 m^3/s and the road can be assumed to be a Class 4 road. From **Figure 8.2** the design flood frequency is determined as T = 8 years and the calculated 1:8 year flood (Q_8) is 44,5 m^3/s. No significant debris is anticipated since the catchment area consists mainly of grassland. The client favours the use of circular (pipe) culverts. The final level of the roadworks across the river will be at a level of 2,5 m above the riverbed.

The absolute roughness, k_s, for the trapezoidal river channel is 0,1 m. The cross-sectional details are provided below in **Figure 7.12**. The natural slope of the river, S_0, is 0,0015 m/m upstream from the culvert and it is 0,004 m/m downstream from the culvert. The culvert will be placed at the same slope as the upstream river section. The submergence of the culvert should be limited to a H/D ratio of about 1,2.

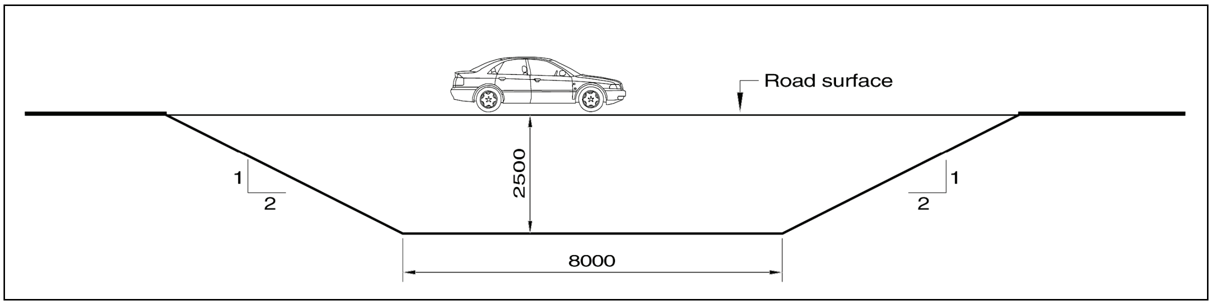


Figure 7.12: Cross-sectional details of the natural channel

Solution Example 7.1

First calculate the normal flow depth, Y_n and flow conditions in the channel upstream and downstream of the planned culvert.

- **Determine the upstream normal flow depth**

For a uniform channel the relationship of Chèzy can be used.

$$Q = C\sqrt{RS}\,A$$

where:

Q	=	flow rate (m^3/s)
C	=	Chèzy constant
R	=	hydraulic radius (m)
A	=	area (m^2)
$Y_{n\ upstream}$	=	Y = upstream normal flow depth (notation used here) (m)

$$44{,}5 = 18\log\left(\frac{12R}{k_s}\right)\sqrt{RS}\,A$$

$$44{,}5 = 18\log\left(\frac{12A}{k_s P}\right)\sqrt{\frac{AS}{P}}\,A$$

Substituting the values for A, P, S and k_s in the above equation provides:

$$44{,}5 = 18\ \log\left(\frac{(12)\left(8Y + (2)(0{,}5)(Y^2)(2)\right)}{(0{,}1)(8 + (2)\sqrt{5}\,Y)}\right)\sqrt{\frac{\left(8Y + (2)(0{,}5)(Y^2)(2)\right)(0{,}0015)}{(8 + (2)\sqrt{5}\,Y)}}\left(8Y + (2)(0{,}5)(Y^2)(2)\right)$$

Solve the upstream normal flow depth Y in the above equation.

Y = 2,0 m and A = 24 m^2, hence V = 1,851 m/s.

The flow type can be determined by calculating the Froude number,

$Fr^2 = \dfrac{Q^2 B}{gA^3} = 0{,}2327$ and Fr = 0,482

The flow is thus subcritical (Fr < 1) and therefore the cross-sectional area of the river may be reduced, resulting in a deceleration of the flow and some damming upstream from the intended structure *(Note: In the case of supercritical flow it is not allowed to decelerate the flow, because it might lead to the creation of a hydraulic jump that might breach the downstream structure).*

- **Determine the downstream normal flow depth**

In a similar way as above, the downstream normal flow depth can be determined. In this case the downstream normal flow depth, Y_{ds} = 1,541 m, Fr_{ds} = 0,758 and the flow is subcritical.

Total energy head upstream of the culvert, $H_1 = \dfrac{\overline{v}^2}{2g} + Y = 2{,}1745\ \text{m}$

- **Determine the size of the culverts to manage the flow**

The height difference between the river bed and the final road level is 2,5 m. If the optimum H/D ratio of 1,2 is used the maximum vertical dimension of the culvert (D) is 2,5/1,2 = 2,08. Based on **Figure 7.15** multiples of 1,8 m diameter pipe culverts will be used.

For a culvert with a diameter of 1,8 m and the downstream flow depth of 1,541 m, the flow will probably be inlet controlled (to be verified) and the flow can be evaluated based on the relationship for inlet control (**Table 7.2**).

In this example, the maximum $\dfrac{H_1}{D} = \dfrac{2{,}5}{1{,}8} = 1{,}39$ and the flow rate through a culvert can be determined.

Table 7.2 reflects that for a circular pipe culvert, under submerged conditions with H/D ≥ 1,2, the flow rate can be determined as follows:

$$Q = C_D\sqrt{2gH_1}\ A$$

$$Q = (0{,}6)\sqrt{(2)(9{,}81)\left(2{,}5 - \frac{1{,}8}{2}\right)}\left(\pi\frac{(1{,}8)^2}{4}\right) = 8{,}55\ \text{m}^3/\text{s}$$

The number of pipes required = 44,5/8,55 = 5,20.

Determine if it is practical to install 6 pipe culverts in the cross-section of the river.

Assume the distance between the pipes is 100 mm and the wall thickness of the pipes is about 78 mm, then the total width of six culverts will be = (6)[1,8+(2)(0,078)] +(5)(0,1) = 12,236 m. With some groundwork it is possible to place the culverts as is shown in **Figure 7.13**.

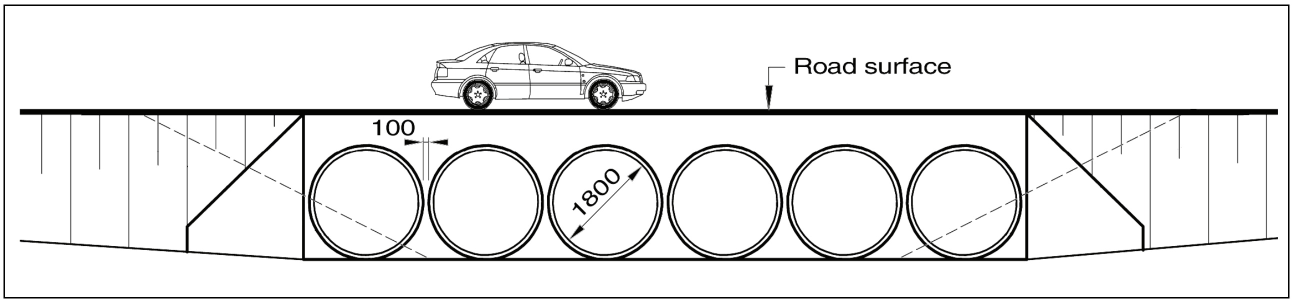


Figure 7.13: Positioning of the 5 pipe culverts

There are, however, also box culverts that could have been used here.

Figure 7.4 reflects the required culvert size for a given (design) flow rate and a H/D ratio of 1,2. Reference to pipe and portal (box) culverts are reflected here.

By means of **Figure 7.4** and by assuming that a portal (box) culvert could be used as an alternative to the calculation above, the required culvert size for Inlet Control conditions could be obtained.

Assume that 5 culverts will be used, the flow per culvert = 44,5/5 = 8,9 m³/s.

Using **Figure 7.4** for a square culvert and following the lines for **Inlet Control** (clockwise), the value for H_1 = 2,4 m for the flow of 8,9 m³/s, a 1800 x 1800 mm portal culvert could be selected (as shown in **Figure 7.14**). The H/D ratio will however be 1,33 and the capacity of the 5 culverts needs to be verified.

This result can be checked with the following formula (**Table 7.2**):

$$Q = \frac{2}{3} C_B B H_1 \sqrt{\frac{2}{3} g H_1}$$

Where C_B = 1 for rounded inlets (r > 0,1B) and C_B = 0,9 for square inlets.

$Q = \frac{2}{3}(0,9)(1,8)(2,4)\sqrt{\frac{2}{3}(9,81)(2,4)}$ = 10,27 m³/s, and hence 5 culverts will be sufficient.

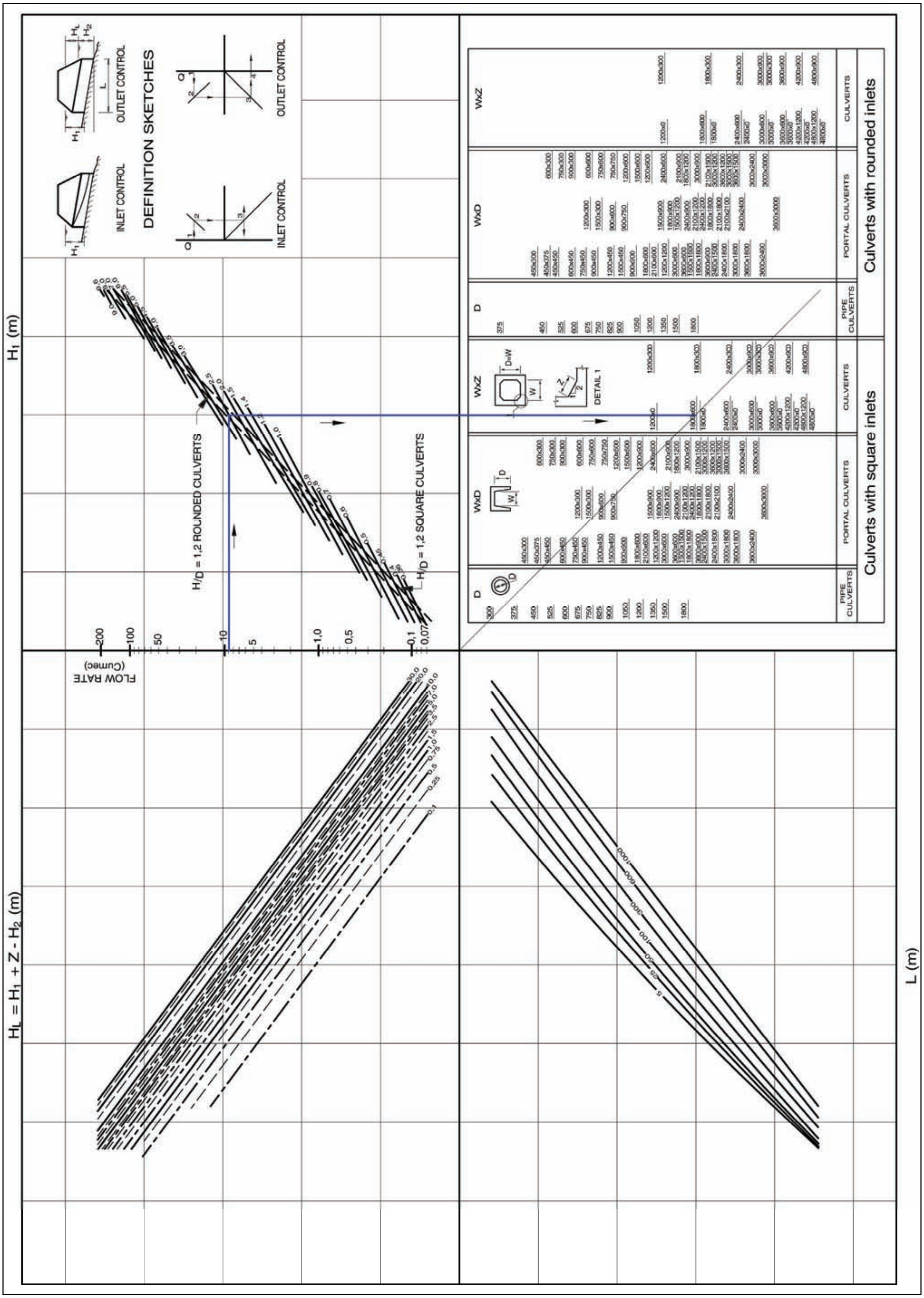


Figure 7.14: Determining culvert size (inlet control)
(This is a reproduction of **Figure 7.4**)

- **Evaluation of the same problem with the upstream slope equal to the downstream slope**

It follows from the new slope details ($S_{0\ upstream}$ and $S_{0\ downstream}$ is 0,0015 m/m) that the upstream and downstream normal flow depths will be 2,0 m, as was determined before. If the upstream water level is limited to a maximum of 2,5 m to prevent the inundation of the road, the culvert flow rate can be determined as follows.

For inlet control conditions the length, roughness, slope and hydraulic radius of the culvert have no influence on the discharge rate. For outlet control these variables do influence the flow rate and have to be considered.

Assume that the following information is still valid:

Slope of the culvert, S_0 = 0,0015 m/m
Roughness of the culvert, k_s = 0,002 m
Diameter of the culvert, D = 1 800 mm
Length of the culvert, L = 25 m
By assuming that 6 culverts will be used the flow rate per culvert = 44,5/6 = 7,417 m³/s.

It was reflected above that the upstream flow is subcritical, i.e. Fr = 0,482, hence downstream control will be experienced in the channel prior to the placing of the culvert.

By applying the energy equation between the upstream/inlet (Position subscript 1) and the downstream/outlet (Position subscript 2) (as represented in **Figure 7.1**) the required upstream energy, H_1, can be determined by using the energy principle as follows:

$H_1 + S_0 L = H_2 + h_{l_{1-2}} + h_{f_{1-2}}$ (between Position 1 (upstream) and Position 2 (downstream)):

For the flow of 7,417 m³/s the flow velocity in the pipe culvert can be determined as follows:

$$\bar{v} = \frac{7,417}{\pi(0,9)^2} = 2,915 \text{ m/s}$$

The secondary losses, h_l, can be determined as follows:

$$h_{l_{1-2}} = h_{l\ inlet} + h_{l\ outlet}$$

$$h_{l_{1-2}} = (K_{inlet} + K_{outlet})\frac{\bar{v}^2}{2g} = (0,5+1)\frac{(2,915)^2}{2(9,81)} = 0,649 \text{ m}$$

The friction losses, h_f, can also be determined, assuming that full bore flow conditions in the culvert will prevail:

$$h_f = \frac{\lambda L \bar{v}^2}{2gD}$$

For rough turbulent flow conditions,

$$\frac{1}{\sqrt{\lambda}} = 2\log(\frac{3,7D}{k_s})$$

$$\lambda = 0,02015$$

$$h_{f_{1-2}} = 0,121 \text{ m}$$

For outlet control conditions, the upstream conditions can now be determined:

$$H_1 = H_2 - Z_1 + h_{l_{1-2}} + h_{f_{1-2}}$$

H_1 = 2,0 - 0,0015(25) + 0,649 + 0,121

H_1 = 2,733 m

The value of H_1 for outlet control is greater than the maximum allowable damming height of 2,5 m, hence outlet control will be maintained through the culvert. The only way to reduce the upstream flow depth is to consider more culverts of similar dimension in parallel or larger culverts.

Assume that seven culverts will be used. With the flow of 44,5/7 = 6,357 m³/s ($\overline{v}$ = 2,498 m/s) and the value of the losses, $H_L = h_{l_{1-2}} + h_{f_{1-2}}$ = 0,477 + 0,089 = 0,566 m, the upstream conditions can now be determined.

$H_1 = H_2 - Z_1 + H_L$

H_1 = 2,0 - 0,0015(25) + 0,566

H_1 = 2,529 m (which is still greater than the allowable upstream damming height)

To illustrated the use of **Figure 7.4**, start at Q = 6,357 m³/s (flow rate in each of the 7 culverts) and H_L = 0,566 m, then it can be seen that the 1 800 mm pipe culvert is marginally insufficient to transport the flow (**Figure 7.15**). **Figure 7.4** could thus be used to consider other culvert sizes.

Placing the seven culverts in parallel will also however result in a section width of about (7)[1,8+(2)(0,078)] +(6)(0,1) = 14,3 m, which is much wider than the river base of 8,0 m.

The number of 1,8 m pipe culverts required to prevent overtopping of the road will be eight. Alternatively portal culverts with a larger vertical dimension (2,1 m) could be used following the same procedure as above.

An economic/technical assessment of the alternatives i.e. re-alignment of the road against provision of a practical culvert design have to be conducted to select the solution for implementation without changing the risk of failure.

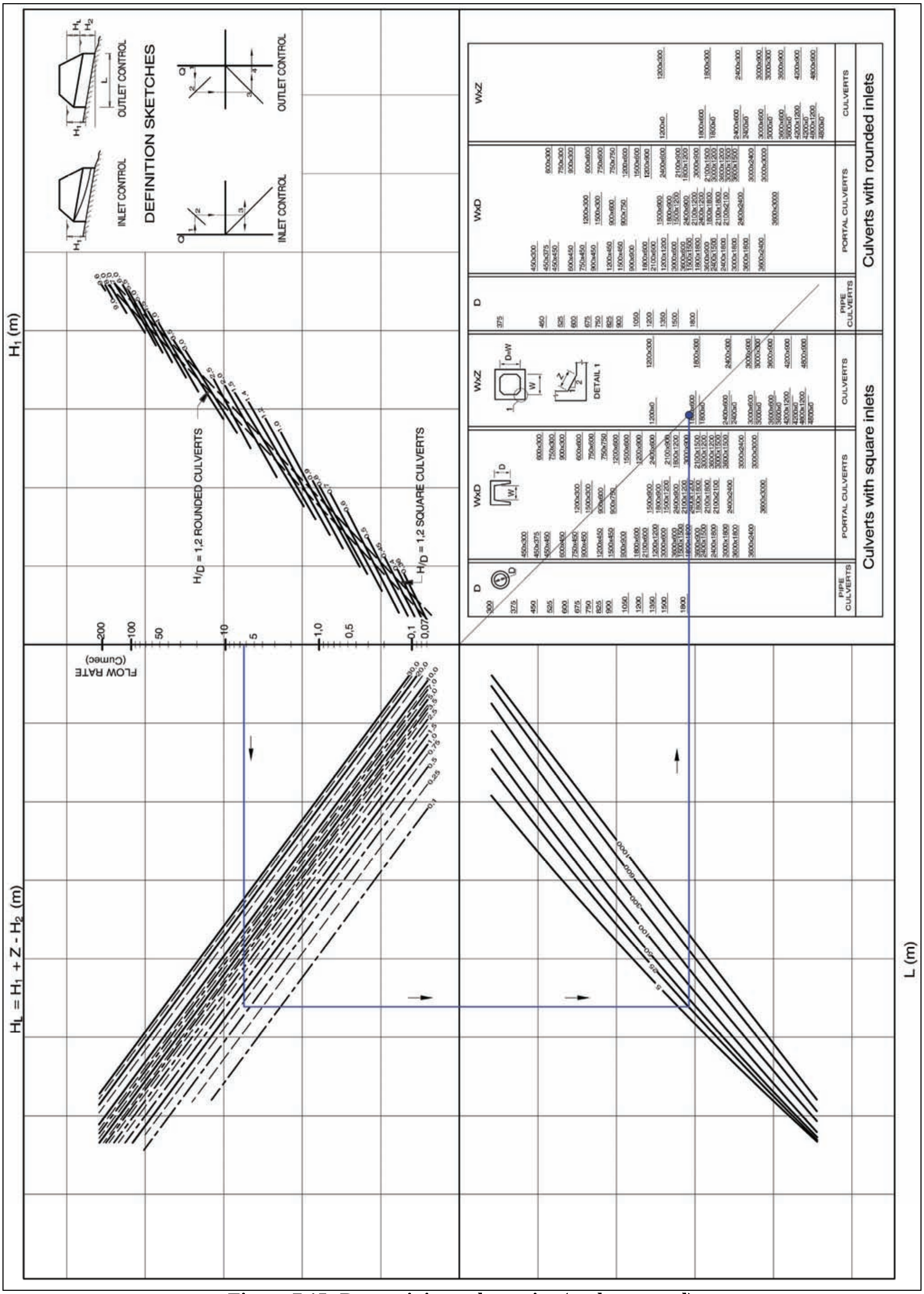


Figure 7.15: Determining culvert size (outlet control)

7.8.2 Example 7.2 – Level pool routing

Problem description Example 7.2

You have to determine the attenuation and translation that results from the routing of a given inflow hydrograph through a dam (The methodology is the same as for a culvert, using an outflow equation from **Table 7.2** or the continuity of energy relationship). The following is known:

Outflow stage relationship of the spillway of the dam is given by:

$$Q = C_d L H^{1,5}$$

where:

Q	=	discharge (m^3/s)
C_d	=	discharge coefficient
L	=	length of the spillway (m)
H	=	total energy head (measured above the spillway level) (m)

In this case the outflow can be determined by the following relationship: $Q = 110H^{1,5}$

Area-volume relationship of the storage volume is given as indicated below:

Surface area at the spill level = 7,5 km^2
Surface area at a level above spill level = 7,5 + 1,5H km^2
H = reflects the difference between the free surface level and the spill level, i.e. total energy (m)

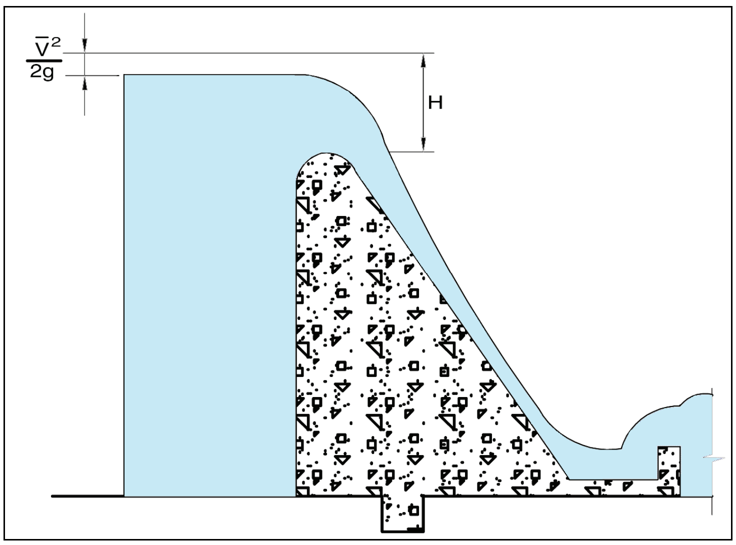


Figure 7.16: Section through the spillway of the dam

The inflow hydrograph is given in **Figure 7.17**.

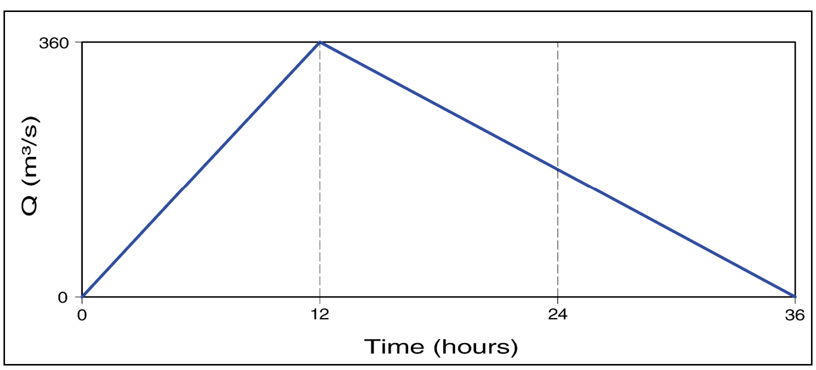


Figure 7.17: Inflow hydrograph

Solution Example 7.2

It is known that the storage relationship is:

$$S = \int_{O}^{H} A\,dH = 10^6 \int_{O}^{H} (7,5 + 1,5H)\,dH$$

$$S = 10^6\ (7,5\ H + 0,75\ H^2 + k)$$

It is known that S = 0, when H = 0 and hence the integration constant, k = 0.

Assume that the time step, Δt = 2 hours = 7200 seconds; then it follows that in the auxiliary function:

$$N = \frac{S}{\Delta t} + \frac{O}{2}:$$

$N_2 = N_1 + \frac{I_1 + I_2}{2} - O_1$ and by substituting the known values, it follows:

A distance away from the spill section where the velocity approaches zero in the dam the difference between the water level and the spillway level reflects the total energy, i.e. h = H

$$N = \frac{10^6}{7\,200}(7,5\ H + 0,75\ H^2) + 55\ H^{1,5}$$

$$N = 1041,7H + 104,17H^2 + 55\ H^{1,5}$$

$$N = 104,17H(10 + H + 0,53\sqrt{H})$$

The relationship for N and H to be used in the auxiliary function is shown in **Table 7.3** (and graphically in **Figure 7.18**).

Table 7.3: Relationship of N versus O, for different H-values

H	O	N
0,2	9,8	217,4
0,4	27,8	447,3
0,6	51,1	688,1
0,8	78,7	939,4
1,0	110,0	1200,9
1,2	144,6	1472,3
1,4	182,2	1753,7
1,6	222,6	2044,7
1,8	265,6	2345,4
2,0	311,1	2655,6

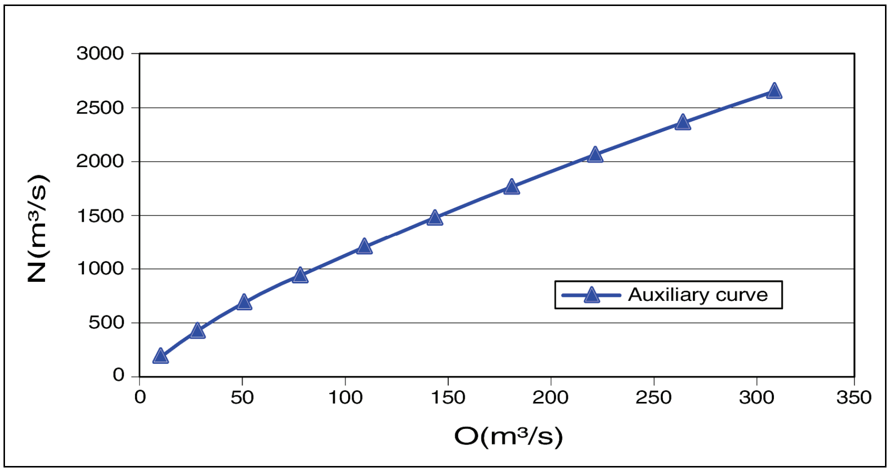


Figure 7.18: Graphical presentation of the auxiliary function

If the inflow and outflow hydrographs are plotted (**Figure 7.19**) it will be observed that:

- the intersect of the hydrographs coincides with the maximum storage; and
- the maximum outflow rate will be associated with the time of the intersect.

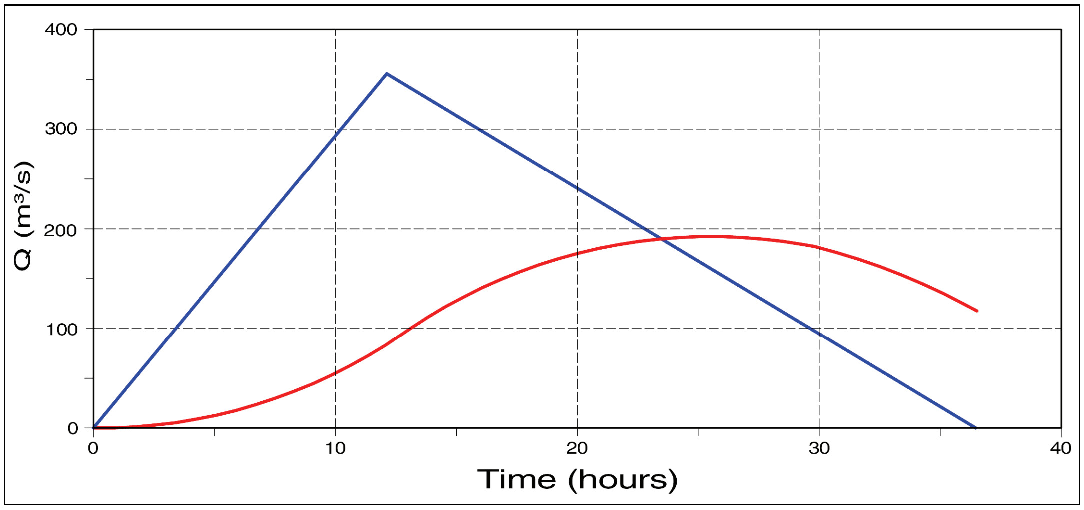


Figure 7.19: The inflow and calculated outflow hydrographs

Summary of the results:

Attenuation = 360-180 = **180 m^3/s**
Translation = 24-12 = **12 h**

7.8.3 Example 7.3 - Erosion protection downstream from a culvert

Problem description Example 7.3

In **Example 7.1** it was indicated that for inlet control it is possible to convey 8,9 m^3/s (for H/D = 1,2) through a 1,8 m diameter circular culvert. You are now requested to design the protection works for a single 1,8 m diameter culvert, functioning under inlet control conditions with H/D = 1,2. The concrete culvert is 28 m long with an estimated absolute roughness of 0,003 m. The culvert will be installed at a slope of 0,01 m/m.

The flow releases into a natural trapezoidal river section with a base width of 2,0 m and side slopes of 1V:2H. The natural slope of the river is 0,004 m/m and the roughness is 0,05 m. Details of the cross-section are given in **Figure 7.20**.

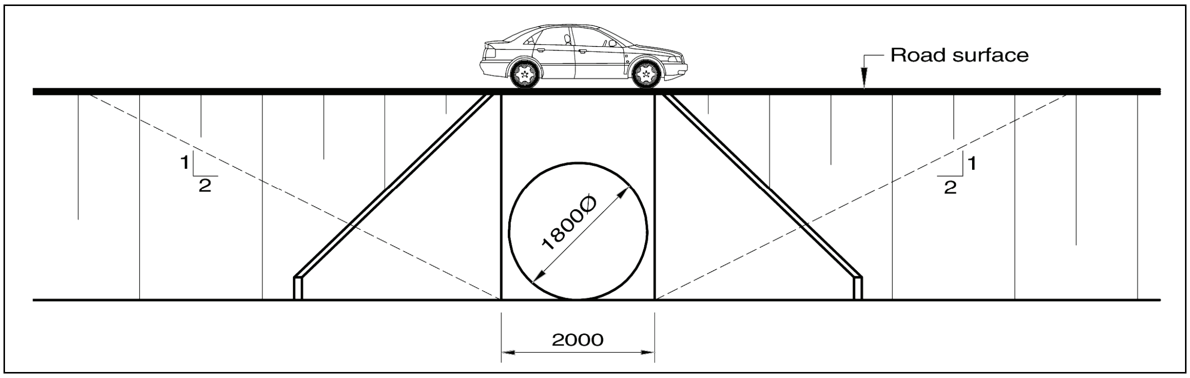


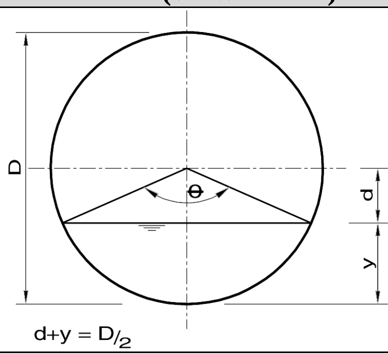
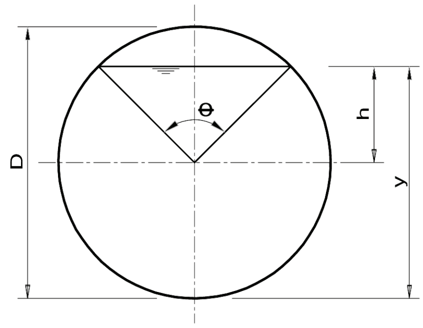
Figure 7.20: Upstream view of the culvert

Solution Example 7.3

The uniform flow equations of Manning and Chezy can only be used if uniform flow occurs. Uniform flow will occur if the cross-sectional parameters, roughness and slope remain constant. With the culvert length of only 28 m it is unlikely that the normal flow depth will be reached within the culvert. By assuming that the normal flow depth will be reached, the analysis is conservative, resulting in a flow depth that is less (for the slope steeper that the critical slope, $S_0 > S_c$) or the flow depth will be greater for a subcritical slope ($S_0 < S_c$).

Table 7.4 reflects the cross-sectional parameters for a circular pipe

Table 7.4: Sectional parameters for a circular cross-section

Variable	Y < D/2 (θ < π radials)	Y > D/2 (θ > π radials)
Cross-sectional view		
Area, A (m^2)	$R^2\theta - 2(0,5)(R-y)\mathrm{Sin}\left(\frac{\theta}{2}\right)R$	$\pi R^2 - (R^2\theta) - 2(0,5)(R-y)\mathrm{Sin}\left(\frac{\theta}{2}\right)R$
Wetter perimeter, P (m)	$R(\theta)$	$R(2\pi - \theta)$
Hydraulic radius, R (m)	$\frac{A}{P}$	

Firstly the normal flow depth, y_n, downstream from the culvert in the river is determined by using the Chezy formula:

$$\bar{v} = C\sqrt{RS}$$

$$Q = vA = 18\left(\log\frac{12R}{k_s}\right)\sqrt{RS}A$$

where:

Q	=	flow rate (m^3/s)
$\bar{v}$	=	average velocity (m/s)
R	=	hydraulic radius (m)

k_s = absolute roughness (m)
S = slope of the river section (m/m)

By substituting the known values:

$$8{,}9 = 18\left(\log\frac{12R}{0{,}05}\right)\sqrt{R(0{,}004)}A$$

with:

$$A = (2)(y_n) + 2(0{,}5)(y_n)(2y_n)$$
$$P = 2 + (2)\sqrt{5}(y_n)$$
$$R = \frac{A}{P}$$

where:

y_n = the unknown normal flow depth (m)

Solving for y_n:

y_n = 1,068 m

This indicates that the normal flow depth in the river section downstream from the culvert will have no backwater influence on the culvert flow ($y_n < D$).

Now the flow depth at the outlet of the culvert is determined. (Refer to the reasoning above where the influence of assuming uniform flow in the culvert was explained.)

The critical slope in the culvert can be determined.

Critical conditions will occur when Fr = 1, and the critical slope S_c can be determined for full flow conditions as follows:

$$S_c = \frac{\bar{v}^2}{C^2R} = \frac{Q^2}{C^2A^2R}$$

S_c = 0,00792 m/m

This indicates that the flow depth in the culvert will reduce downstream from the position where critical flow occurs near the inlet of the culvert because $S_0 > S_c$.

If it is assumed that the flow depth in the culvert Y is more than D/2, solve for a potential flow depth, Y.

It is found that:

y = 1,329 m; A = 2,015 m² ; B = 0,997 m (top width of flow); $\bar{v}$ = 4,417 m/s and Fr = 0,984 (subcritical).

y/D = 1,329/1,8 = 0,738

Figure 7.9 can now be used to select the appropriate erosion protection.

Figure 7.9 reflects that the appropriate protection is at the border of Type II, hence Type III is selected. **Figure 7.10c** can now be used to obtain dimensions for outlet erosion protection as shown in **Figure 7.21** with calculated dimensions given below.

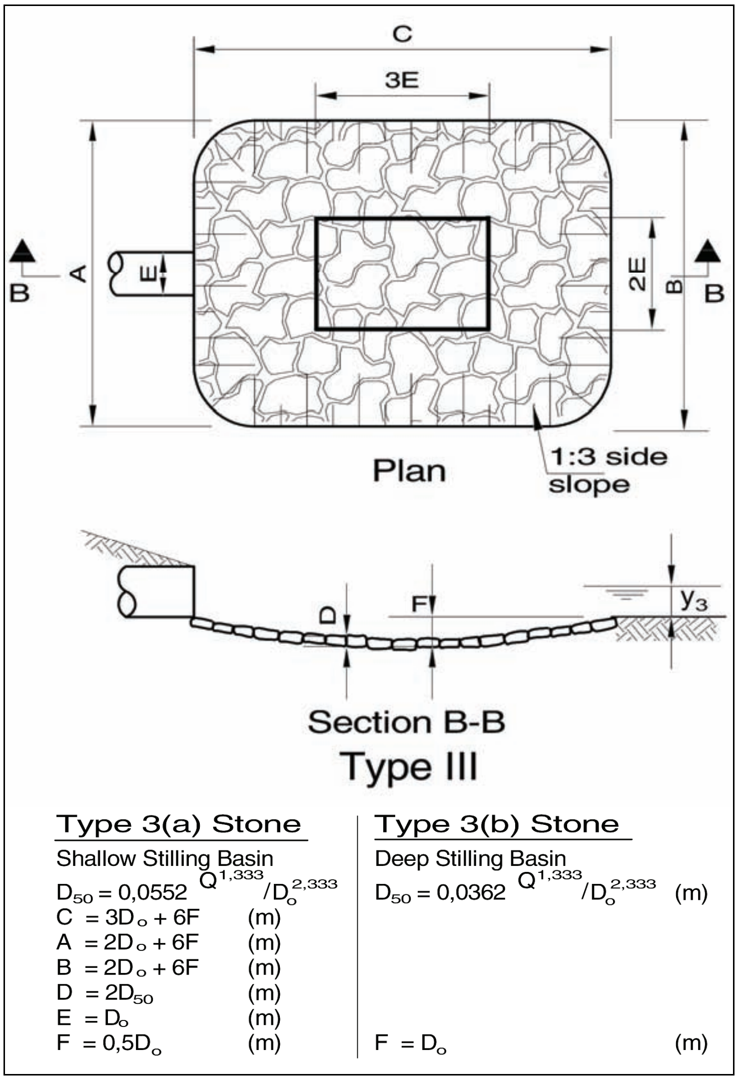


Figure 7.21: Dimensions for the outlet erosion protection

Calculated dimensions:

D_{50} = 0,258 m
C = 10,8 m
A = 9,0 m
B = 9,0 m
D = 0,517 m
E = 1,8 m
F = 0,9 m

Using **Figure 7.6** with a flow of 8,9 m³/s in a circular culvert the values of Froude and the flow depth can also be obtained which will correspond with the calculated values.

7.9 REFERENCES

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CHAPTER 8 - BRIDGES AND MAJOR CULVERTS

AM Jansen van Vuuren, A Rooseboom and EJ Kruger

8.1 INTRODUCTION

The hydraulic design of bridges and major culverts, as well as the practical aspects of bridge site location and of scour protection are covered in this chapter.

Definitions applicable in this chapter are as follows:

- **Bridge**
  A structure shall be classified as a bridge if **any one** or more of the following apply:

  - o The clear span (as measured horizontally at the soffit along the road or rail centre line between the faces of its supports) exceeds 6 m.
  - o The individual clear spans (as measured horizontally at the soffit along the road or rail centre line between the faces of its supports) exceed 1,5 m **and** the overall length measured between abutment faces exceeds 20 m.
  - o The opening height, which is the maximum vertical distance from the streambed or structure floor at the inlet, to the soffit of the superstructure, exceeds 6 m.
  - o Where the total cross-sectional opening is equal or larger than 36 m².

- **Major culvert**
  A cellular structure with dimensions less than those defining a bridge but with clear span (as measured horizontally at the soffit perpendicular to the faces of its supports) length equal to or larger than 2,1 m, or diameter equal to or larger than 2,1 m, or a culvert with a total cross-sectional opening equal to or larger than 5 m².

- **Lesser culvert**
  All culverts smaller than that defined as a major culvert (See **Chapter 6**)

- **Backwater (or damming; afflux)**
  The maximum rise in water level upstream of the waterway constriction, as shown in **Figure 8.1**.

- **Freeboard (F_D and F_{SBP})**
  Freeboard is the height difference between the design high flood level (that includes the backwater) and the lowest level along the soffit of the deck or shoulder breakpoint as defined in **Figure 8.1**. Note that the freeboard is positive if the design high flood level is below the deck soffit, and negative if the design high flood level is above the deck soffit or the shoulder breakpoint as applicable.

- **Shoulder breakpoint** – Defined in **Figure 8.1**.

Important Note: It must be noted by the reader that the definitions for bridges and major culverts are purely for categorisation purposes for structures management from a risk viewpoint and do not refer to the form of the structure. Thus a bridge may take the structural form of a culvert and a culvert may have the structural form of a bridge. It will be noted that in this chapter no distinction is made between bridges and culverts for freeboard or other requirements. This is the major departure from previous editions of this manual. Lesser culverts should also follow the freeboard requirement as outlined in this chapter.

Table 8.1: Road Map for bridges and major culverts

<table>
<tr><th colspan="7">ROAD MAP 8</th></tr>
<tr><th colspan="2">Typical problems</th><th rowspan="2">Input information</th><th rowspan="2">Worked examples</th><th rowspan="2">Supporting software</th><th colspan="2">Other topics</th></tr>
<tr><th>Topic</th><th>Par.</th><th>Topic</th><th>Par.</th></tr>
<tr><td>Determination of the required freeboard at bridges and major culverts</td><td>8.2</td><td>Road Class, Q_{20} and Q_T (design peak discharge), associated social, environmental and structural impact associated with overtopping</td><td>-</td><td>-</td><td>Risk of overtopping and failure (LCC) Flood calculations</td><td>2.5, 2.6 & Chapter 3</td></tr>
<tr><td>Calculation of the backwater</td><td>8.3</td><td>Flow rate, cross-sectional parameters, representative roughness, eccentricity and orientation of the opening, flows in the left, right and centre channels (where applicable)</td><td>8.1 in 8.7.1</td><td rowspan="7">HEC-RAS</td><td>Sub- or supercritical flow conditions in the bridge</td><td>8.3</td></tr>
<tr><td rowspan="5">Scour estimation at bridges</td><td rowspan="5">8.4</td><td>Flow rate, cross-sectional parameters, representative particle characteristics, longitudinal channel characteristics and sediment loads. Physical properties of clay</td><td rowspan="5">8.2 in 8.7.2</td><td>Long and short term general scour</td><td rowspan="4">8.4.3</td></tr>
<tr><td>Pier dimensions and shape, orientation, form of the pier nose</td><td>Local scour</td></tr>
<tr><td>Dimensions of narrowed section.</td><td>Contraction scour</td></tr>
<tr><td>Influenced by all the above factors</td><td>Total scour</td></tr>
<tr><td>Summary of all the steps required</td><td>Procedures for estimation of scour</td><td>8.4.7</td></tr>
<tr><td>Scour counter-measures at bridges</td><td>8.5</td><td>Erodibility, topography and alignment of the road</td><td>-</td><td>Counter-measures at bridges</td><td>Table 8.11</td></tr>
</table>

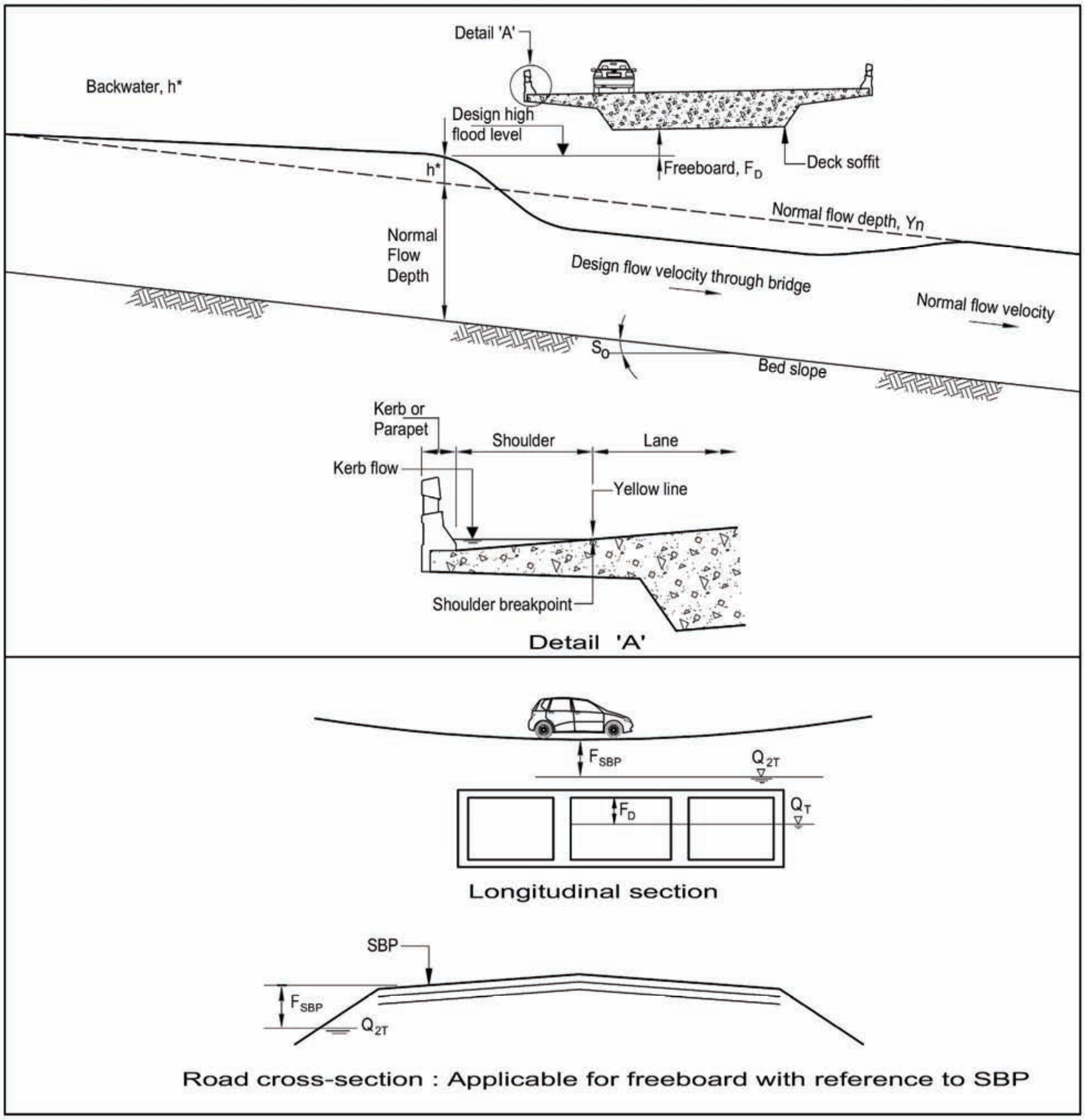

Figure 8.1: Illustration of the hydraulic definitions

Freeboard requirements are depicted in Section 8.2 and the selected design flood frequency depends on the classification of the road.

8.2 ROAD CLASIFICATION

Roads are classified into six classes. These definitions correspond to the "Road Infrastructure Strategic Framework for South Africa" and are as follows [(8.33)]:

- **Class 1 (Primary Distributors)**
  Strategic function: High mobility roads with limited access for rapid movement of large volumes of traffic.
  Typical description: These are roads through regions of national importance such as between provincial capitals and key cities. These include routes between South Africa and adjoining countries that have significant national economic or social road traffic. These are also roads that access major freight and passenger terminals including major ports and airports.

- **Class 2 (Regional Distributor)**
  Strategic function: Relative high mobility roads with lower levels of access for movement of large volume traffic.
  Typical descriptions: These are roads through regions of provincial importance such as between provincial capitals, large towns, municipal administration centres and transport hubs of regional importance. These include routes between Class 1 roads and key centres that have significant economic, social, tourism or recreational roles. This includes roads that carry limited national economic or social road traffic between South Africa and adjoining countries.

- **Class 3 (District Distributor)**
  Strategic function: Moderate mobility roads with controlled higher levels of access for movement of traffic in rural and urban areas of regional importance.
  Typical descriptions: These are roads between town centres, rural residential areas and villages; between towns and industrial/farming areas; between residential and local industrial/farming areas; and linkages between Class 2 and/or Class 1 routes.

- **Class 4 (District Collector)**
  Strategic function: Lower mobility roads with high levels of access for lower traffic volumes in urban and rural areas of local importance.
  Typical descriptions: These are roads between villages, farming areas and scattered rural settlements and communities, which primarily serve local services and access to markets; and roads linking Class 3 roads.

- **Class 5 (Access Roads)**
  Strategic function: Very low mobility roads with high levels of access for low traffic volumes in urban and rural areas.
  Typical descriptions: These are roads within residential communities; roads linking Class 3 or 4 roads to residential communities; direct access to individual industries and businesses; and access to specific destinations to heritage sites, national parks, mines, forests etc.

- **Class 6 (Non-motorised Access Ways)**
  Strategic function: Public rights of way for non-motorised transport.
  Typical descriptions: These are roads to provide safe access and mobility for pedestrians, cyclists and animal drawn transport.

Important Note: It is required that, **before** any detail design work is undertaken, the designer shall obtain in writing the classification of the road for flood return periods from the applicable road authority. This is particularly relevant for upgrading projects where an existing road changes to a higher class. Normally in such cases all existing structures are checked for hydraulic capacity. As a guide, structures that have hydraulic capacities that satisfy the requirements of a class of road one below the class of the upgraded road could be considered acceptable. Those structures that are totally hydraulically inadequate should be replaced with structures that can accommodate the correct floods for the new upgraded class of road.

8.3 DESIGN FLOOD FREQUENCY AND FREEBOARD REQUIREMENTS

The Design Flood: The design flood, Q_T, is the flow rate (m^3/s) with a return period of T years for which the hydraulic structure will be designed for freeboard requirements to the deck soffit. The 20-year flood is used as the "indicator" flood and it in essence reflects the risk classification of the road river-crossing at the bridge site. The selection of the return period for design is normally determined using the classification of the road class, but may also be influenced in certain cases by the cost of the structure and if the risk of failure can be managed and other factors. The applicable design floods are determined according to the methodology described in **Chapter 3** and the determination of the natural or "normal" flow depths is set out in **Chapter 4**. The design flood frequency, T, can be obtained from **Figure 8.2**.

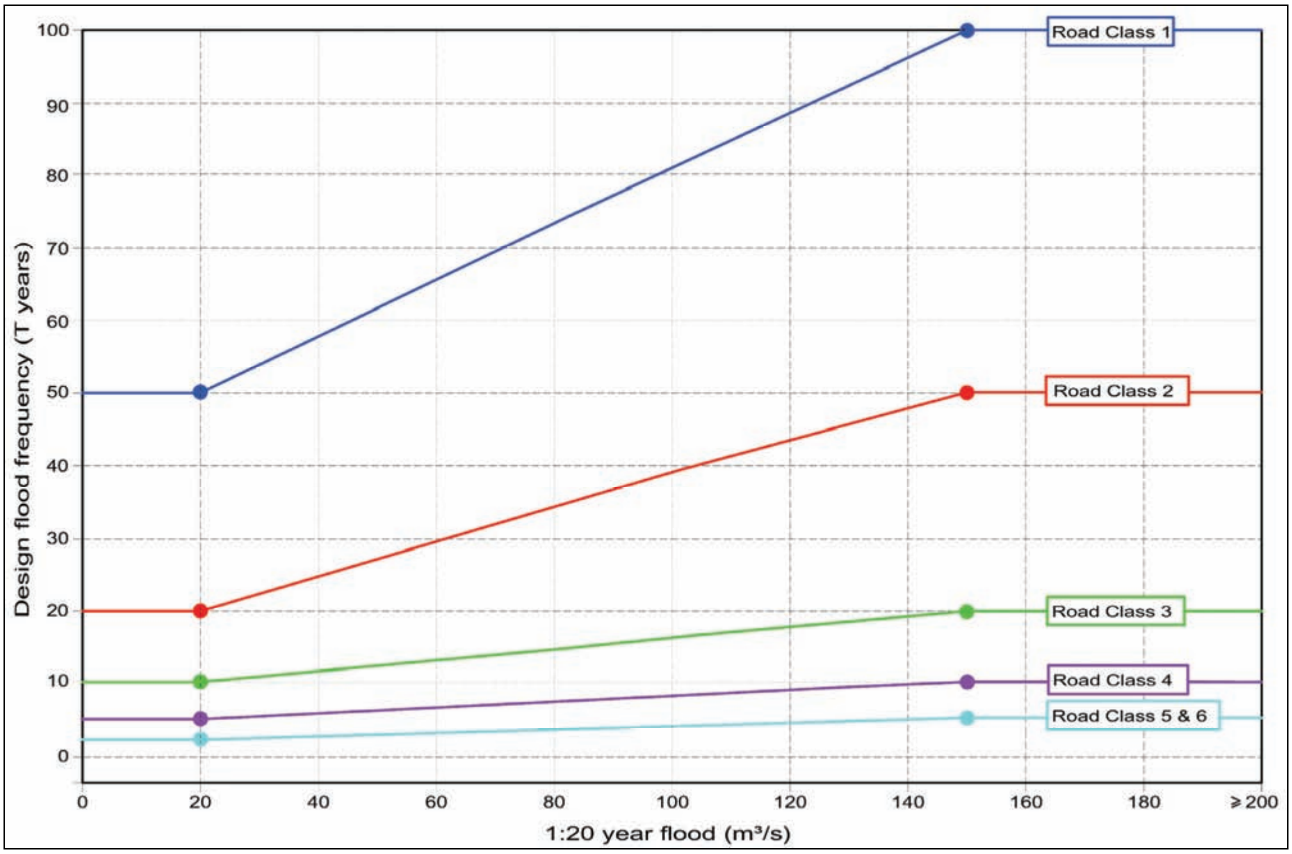

Figure 8.2: Design flood frequency estimate

Freeboard: There are two freeboard requirements that need to be met. The first freeboard requirement, $\mathbf{F_D}$, is the distance of the design flood, $\mathbf{Q_T}$, below a deck soffit (underside of deck) and the second freeboard requirement, $\mathbf{F_{SBP}}$, is the distance of a flood, $\mathbf{Q_{2T}}$, below the lowest shoulder break point of the road. Freeboard below deck soffit **($\mathbf{F_D}$)** as shown in **Figure 8.3** is dependent on the magnitude of the design flood, $\mathbf{Q_T}$, whose return period is determined from **Figure 8.2** for the applicable road class. Freeboard to shoulder breakpoint **($\mathbf{F_{SBP}}$)** shall be zero or positive for a flood having **twice** the recurrence interval **(2T)** of the design flood (see also **Figure 8.1**). Please note this **is not** $2Q_T$ but a flood with magnitude of $\mathbf{Q_{2T}}$. The freeboard requirement to shoulder breakpoint, F_{SBP}, shall generally only apply to Road Classes 1, 2 & 3.

The required freeboard below deck soffit, $\mathbf{F_D}$, is shown in **Figure 8.3**. It should be noted that the maximum discharge capacity of the largest lesser pipe culvert size (2,1 m diameter) operating as inlet control, with H/D = 1,2, is in the order of 10 m³/s. This discharge coincides with the transition between lesser and major culverts. From **Figure 8.3** it is evident that major structures with design flood magnitudes of between 10 and ± 90 m³/s may have negative freeboards. This implies that such structures act hydraulically as culverts and not as bridges.

The design flood, Q_T, and freeboard requirements above should be regarded as minimum standards. The engineer should investigate the impacts of debris, standing wave action, and super-elevation of the flow on the freeboard to be provided. Additional freeboard shall be provided where sediment build-up is likely, e.g. if the structure is positioned upstream of an existing or a planned dam. Such build-up often extends above the full supply level of the dam.

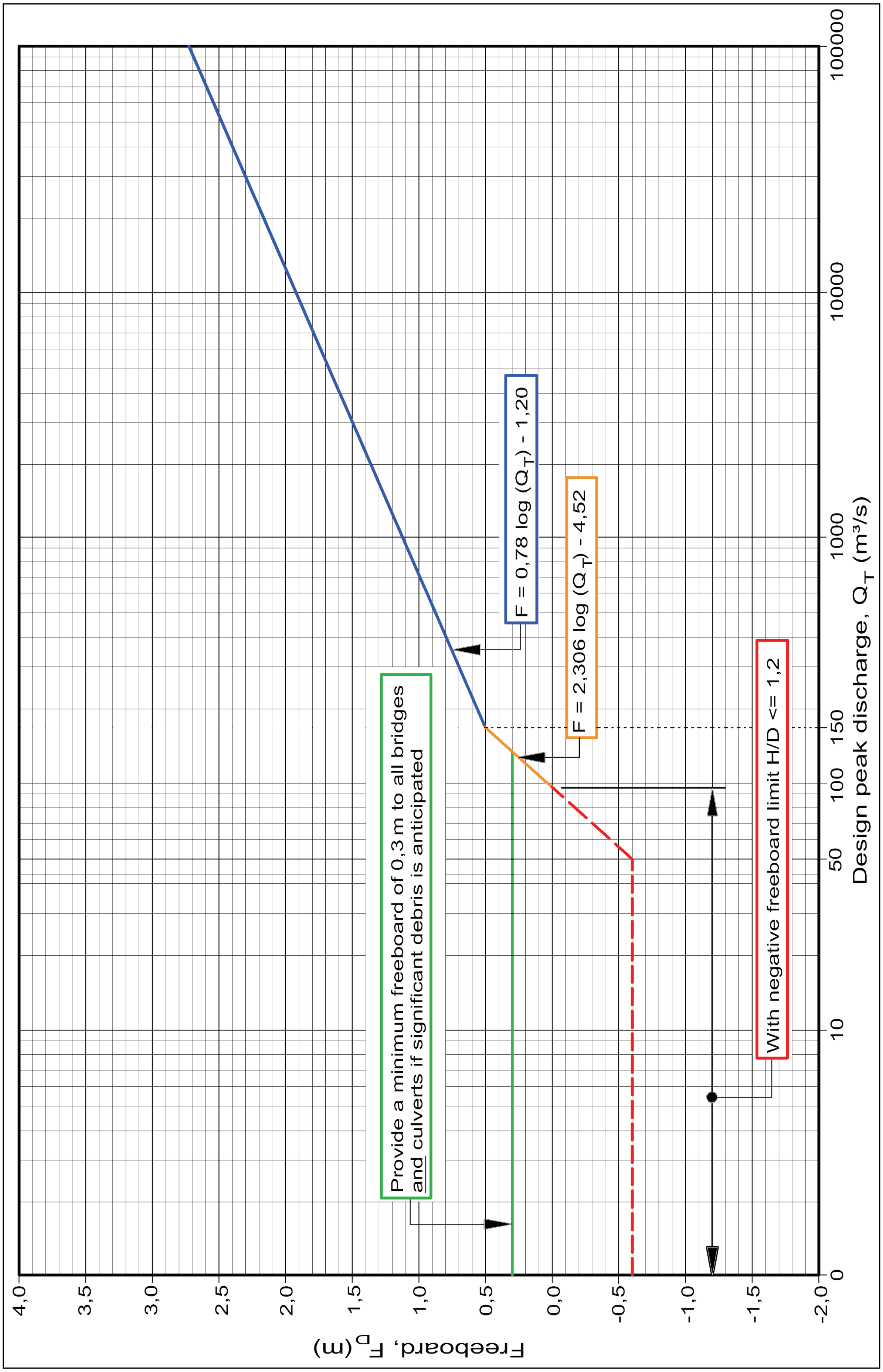

Figure 8.3: Require freeboard from the calculated backwater to the deck's soffit of bridges and culverts

Debris build-up: This aspect needs careful evaluation considering the risks involved and the economy of the structure. The backwater effect due to accumulation of debris should be taken into account where appropriate without being overtly conservative. Where large debris is likely, the recommended minimum span lengths should be at least 7,5 m for all roads but preferably 15 m for Class 1 to Class 3 roads where the Indicator 20 year flood is 150 m^3/s or greater. A typical failure partly due to the build-up of debris is shown in **Photograph 8.1** below. A minimum freeboard of 0,3 m for **all** bridges and culverts should be provided where significant debris is anticipated. Local conditions should be studied when freeboard is determined and local maintenance teams should be consulted.

Photograph 8.1: Failure partly due to debris build-up

Wave action: Standing waves are normally associated with high flow velocities, large variation in roughness and abrupt changes in the cross-sectional flow area. Bridge crossings over large impoundments or in coastal areas, wind generated wave heights and run-up as well as the tidal influence need to be considered. Section 7 of Volume II of TRH 25 (CSRA, 1994) [(2-2)] contains detailed design guidelines on this topic.

Super elevation of flow: Directional changes at bends will result in a centrifugal acceleration that will reflect in the conveyance channel as an increase in the water depth on the outside of the bend. **Table 5.4** reflects the requirements in channels. Although it is not ideal to locate a bridge at a bend in the river, this aspect will have to be considered. The three-dimensional nature of the problem might require a physical modelling of the layout.

Overtopping of the structure: There is always a probability that the design flood might be exceeded, resulting in the overtopping of the structure and the aprons. The impact of overtopping and the potential alleviating options should be considered.

For Class 4, Class 5 and Class 6 roads and all structures built below a flood with a 50-year return period flood line should be designed to be stable when overtopped. Due cognisance should be taken of embankment scour and careful consideration must be given as to the type of pedestrian and traffic barriers that are used. Collapsible pedestrian barriers or guide blocks shall be used where appropriate. The designer is referred to **Chapter 6** dealing with submersible structures. However guide blocks can only be considered for bridges that are less than 3 m high, from top of deck level. For higher bridges a combination of guide blocks and collapsible railings could be used. Basic economic considerations could however, for return periods less than 20 years, require the bridge to be designed for a higher return period to prevent the maintenance problems associated with the hand rails if overtopping occurs frequently. This aspect must be discussed with the authority concerned.

The volume of pedestrians using a bridge is a critical consideration. Solid traffic barriers should, if possible, be avoided on bridges that will be overtopped.

It is professionally irresponsible to place bridges or culverts below the 50-year flood level and to ignore the fact that these structures will be overtopped at some stage during their structural design life. The design flood return period may have to be increased to that required of the next higher Class of road. Refer such cases to the roads authority for a decision.

8.4 BACKWATER DETERMINATION

A bridge often reduces the available cross-sectional flow area of a stream. This leads to additional energy losses and causes upstream damming of subcritical flow. The flow patterns through a normal (subcritical) crossing are shown in **Figure 8.4** (8.4). Where supercritical flow is maintained, the depth of flow increases where the stream section narrows. Because urban or agricultural development often limits permissible backwater levels, land use plays a part in the determination of bridge openings. The sizing of a bridge opening is also affected by the scour or sediment build-up that may take place. Scour at bridges is dealt with in Section 8.4.

Once the backwater, h* (or afflux) has been determined, the freeboard of the proposed bridge or culvert openings should conform to the minimum requirements in Section 8.2, adjusted for factors such as a high debris load.

The following design limit criteria should be implemented (8.26):

- Backwater generally not more than 0,6 m.
- The design flow velocity through the constriction should generally be less than 4 m/s.
- In addition to the above, the ratio of the design flow velocity through the structure to the natural flow velocity should not exceed 1,67 due to scour considerations.

Detailed backwater calculations are described by Bradley in the publication *Hydraulics of Bridge Waterways* (8.4). Full details of the methodologies to be applied to four types of flow through a constriction are contained in Volume I of TRH25 (8.7). In this manual, only the more common cases of slight backwater (Type I flow) and more severe backwater (Type II flow) are dealt with to illustrate the concepts and to assist in hand calculations. Software to determine the bridge backwater is available. HEC-RAS, for example, incorporates this methodology (referred to as the WSPRO method), as well as solves the standard step energy equation, or the momentum balance, or the empirical Yarnell equations, and is widely used. The software is included on the supporting CD.

In the analysis of backwater the following procedures should be followed:

Determine the Froude number: $$Fr = \left(\frac{Q^2 B_n}{g A_n^3}\right)^{\frac{1}{2}} \qquad \ldots(8.1)$$

for normal flow in the river, where:

Fr	=	Froude number
Q	=	design discharge (m^3/s)
B_n	=	total flow width for the normal stage (m)
A_n	=	total flow area for normal stage (m^2)
g	=	gravitational acceleration (m/s^2)

If the normal flow is subcritical, Type I or Type II flow may occur at the structure (**Figures 8.5** and **8.8** illustrate the flow patterns through the structure). Determine the backwater for both cases, and adopt the greater value.

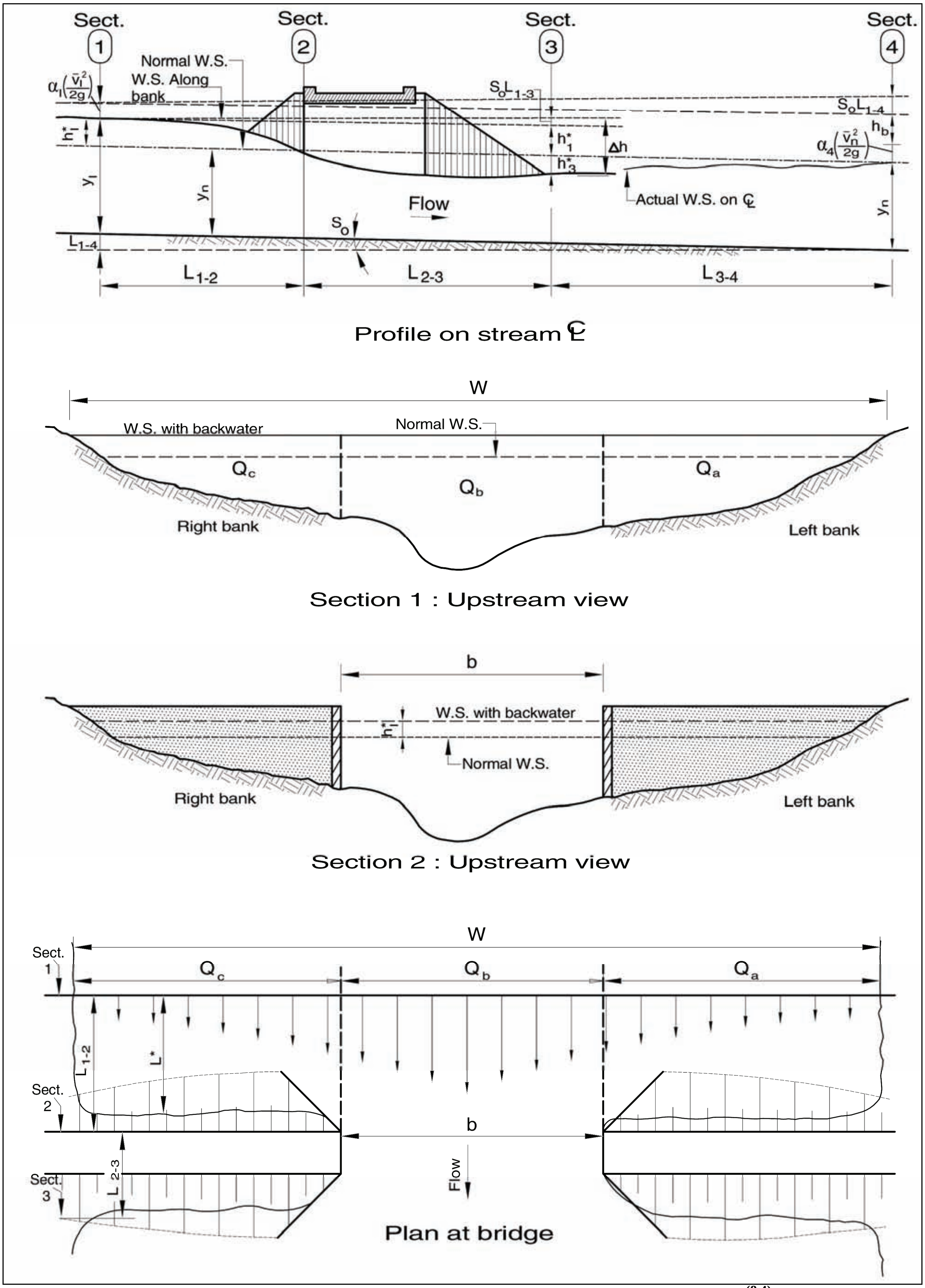

Figure 8.4: Water levels and flow distribution at normal crossings (8.4)

Backwater height, h_1^* for Type I flow through bridge

Figure 8.5 serves as a definition sketch. The backwater height is given by:

$$h_1^* = K^* \alpha_2 \frac{\bar{v}_{n2}^2}{2g} + \alpha_1 \left\{ \left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right\} \frac{\bar{v}_{n2}^2}{2g} \qquad \ldots(8.2)$$

where:

K^* = secondary energy loss coefficient

α_1, α_2 = velocity coefficients (see **Figure 8.6** and the description below)

$\bar{v}_{n2}$ = $\frac{Q}{A_{n2}}$ (m/s) where Q = design discharge (m³/s)

A_{n2} = projected flow area at constricted section 2 below normal water level of the river section (m²)

A_1 = flow area at section 1, including the influence of the backwater on the flow depth (m²)

A_4 = flow area at section 4 (m²)

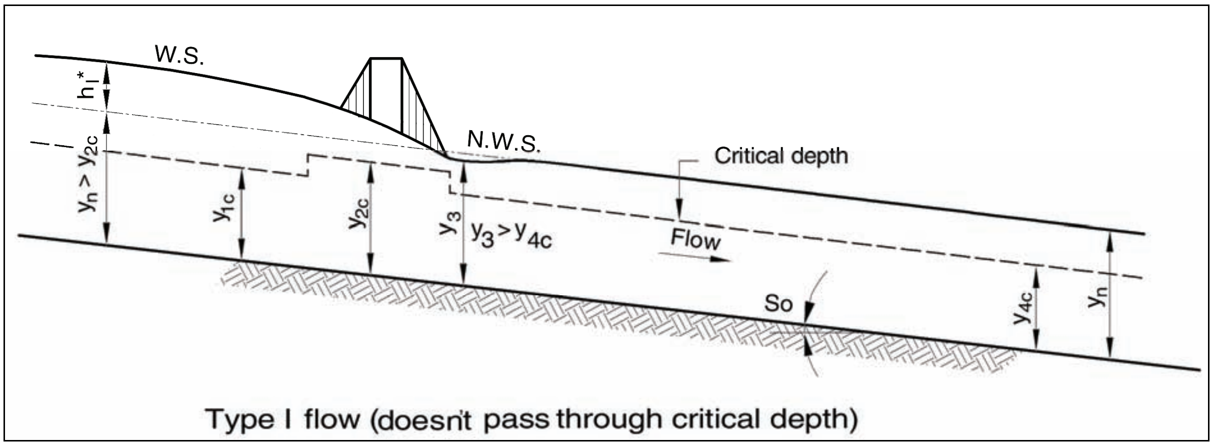

Figure 8.5: Type 1 flow through bridge

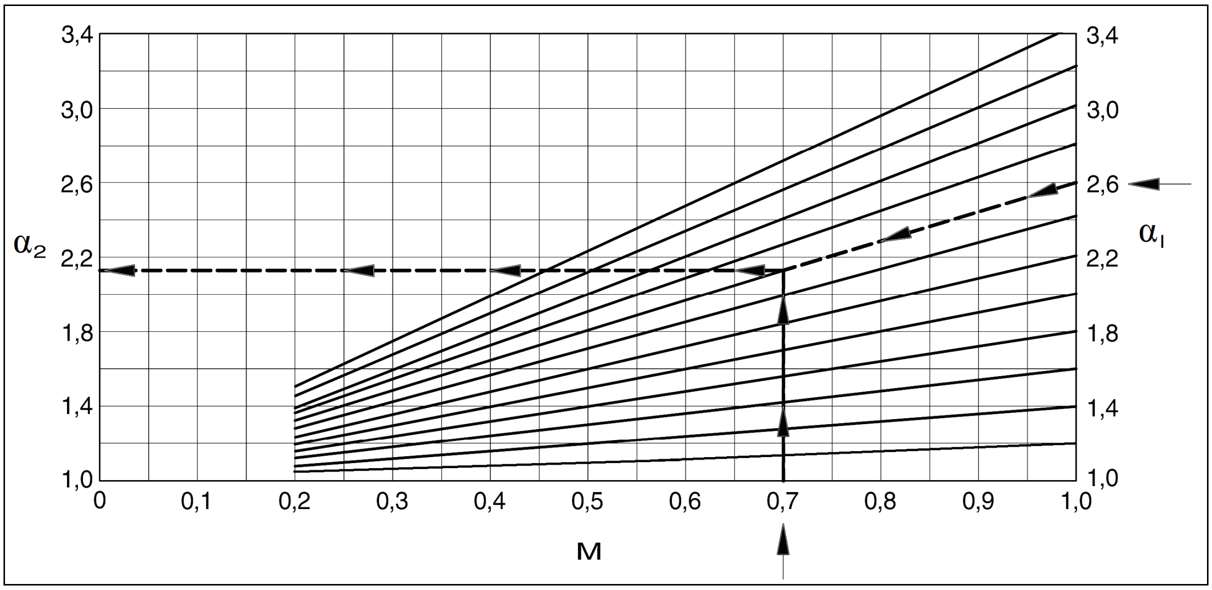

Figure 8.6: Estimation of the velocity coefficient, α_2

The bridge opening ratio $M = \frac{Q_b}{Q}$ (Refer to **Figures 8.6, 8.7** and **8.9**).

Calculation of velocity coefficients:

$$\alpha_1 = \frac{\sum (q\bar{v}^2)}{Q\bar{v}_1^2} \quad \text{with:}$$

$\bar{v}$ = average velocity through sub-channel (a, b or c) (m/s)
q = discharge through the sub-channel (m³/s)
Q = total discharge (m³/s)
$\bar{v}_1$ = average velocity through section 1 = $\frac{Q_1}{A_1}$ (m/s)

and α_2 is read from **Figure 8.6** with $M = \frac{Q_b}{Q}$ (from **Figure 8.4**).

The loss coefficient K^* can be determined from the chart in **Figure 8.7** (8.22), which summarises the graphs in *Hydraulics of Bridge Waterways* (8.4). The designer may consult the reference for more detailed charts. The graphs were compiled from the results of laboratory tests where a straight channel with uniform section and a constant bed slope was used. The slope of the bed was adjusted until uniform flow was obtained. Although bridges are usually built in rivers with non-uniform flow depths and velocities, Equation 8.2 and **Figure 8.7** can still be used as a good estimate for these conditions.

In complex cases, such as double bridges, consult TRH 25 (8.7). Note, however, that there is often more uncertainty about the undisturbed (normal) flow depth than about the increase in the flow depth that would be caused by a bridge.

The calculation procedure for Type II flow (**Figure 8.8**) is illustrated below:

Backwater height, h_1* for Type II bridge flow

Figure 8.8 serves as a definition sketch. The backwater height is given by:

$$h_1^* = \alpha_2 \frac{\bar{v}_{2c}^2}{2g}(C_b + 1) - \alpha_1 \frac{\bar{v}_1^2}{2g} + y_{2c} - \bar{y} \qquad \text{…(8.3)}$$

where:

$\bar{y}$ = projected normal flow depth in the constriction = $\frac{A_{n2}}{b}$ (m)

y_{2c} = critical depth in constriction = $\frac{A_{2c}}{b}$ (m)

$\bar{v}_1$ = average velocity through section 1 = $\frac{Q_1}{A_1}$ (m/s)

$\bar{v}_{2c}$ = critical velocity in constriction = $\frac{Q}{A_{2c}}$ (m/s)

α_2 = velocity head coefficient for the constriction (**Figure 8.6**)

The backwater coefficient C_b, can be obtained from **Figure 8.9**, whilst the other terms are as previously defined for Equation 8.2

If the normal flow is supercritical, the flow section should preferably not be constricted. Also ensure that the freeboard is adequate, so that the superstructure will not affect the flow and cause a hydraulic jump to occur. If the section is constricted, treat as Type II flow or obtain expert opinion.

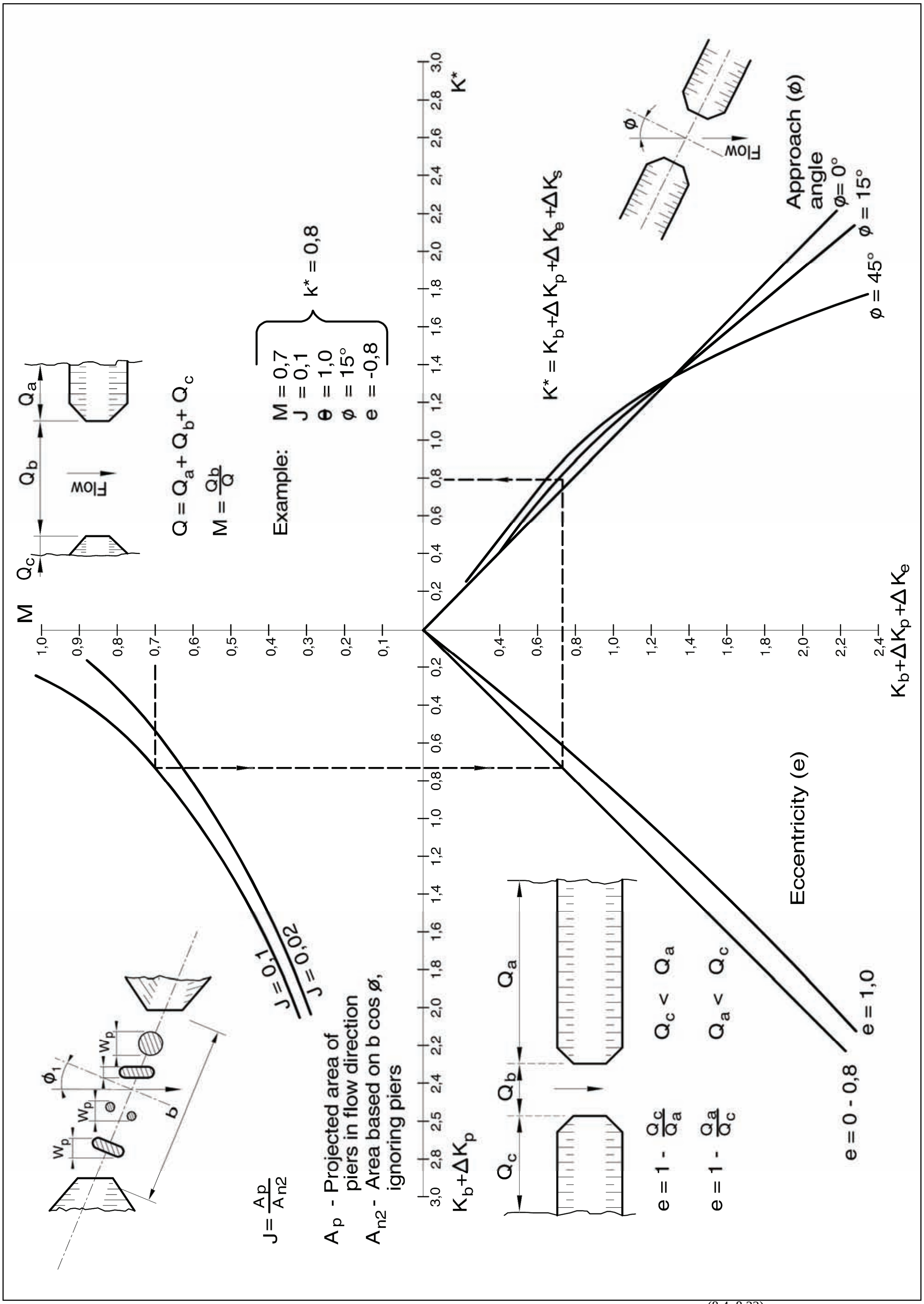

Figure 8.7: Chart to determine the backwater coefficient, K* (8.4; 8.22)

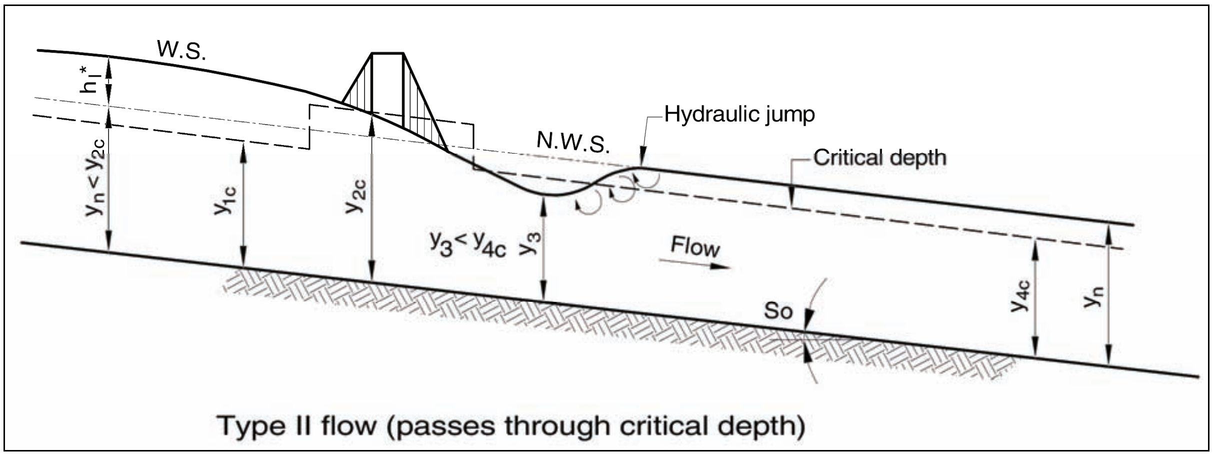

Figure 8.8: Type II flow with substantial damming and critical flow through the bridge

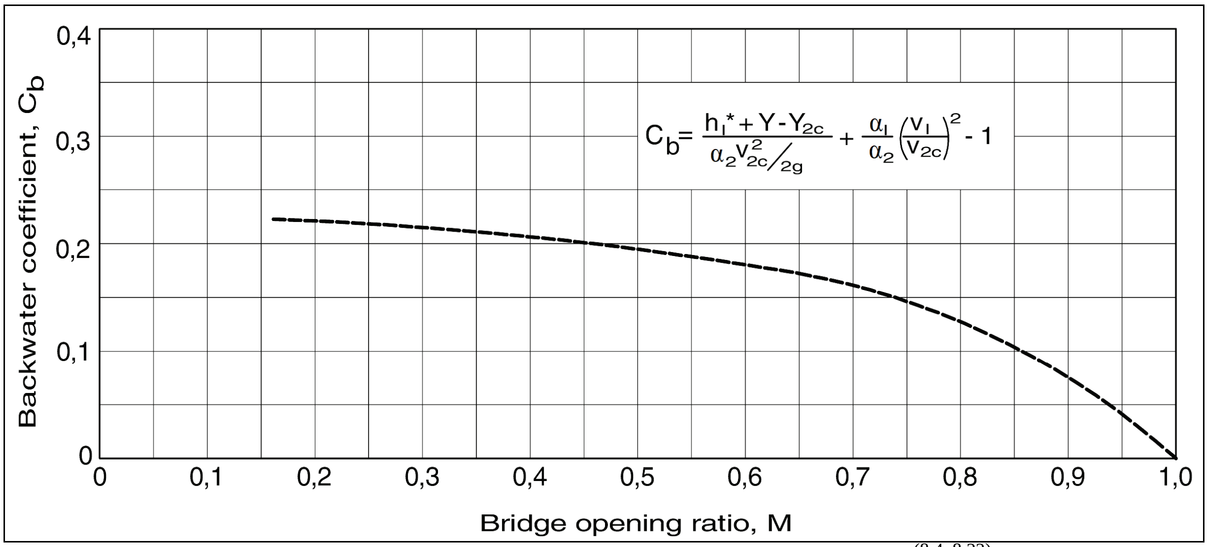

Figure 8.9: Estimation of the Backwater Coefficient, C_b (8.4, 8.22)

Note that where abnormal stage conditions are caused by downstream hydraulic controls, the unconstricted flood water level at the bridge should be determined by computing the flow profile as described in Section 4.2. The backwater is then calculated using a contracted version of the original Equation 8.2:

$$h_{1A}^* = K^* \alpha_2 \frac{v_{2A}^2}{2g} \qquad \text{…(8.2a)}$$

where the subscript A refers to abnormal stage conditions.

In order to compute the flow profile, sufficient information in the form of river cross-sections of the downstream river reach (for subcritical flow conditions) should be available.

Other points also to be borne in mind in the analysis of backwater are as follows:

- To determine the undisturbed (normal) flow depths, the prevailing controls should be properly identified.

- The presence of sand-spits at river mouths has to be considered carefully, because spits often block river mouths. A spit may wash away only after severe damming has taken place. This impact tends to become more serious as more water is extracted from a river and the spit is broken through less frequently.

- River channels are often in a state of change. Changes in the position of the main channel and flood plains may have an effect on backwater, especially if the angle of approaching flow is increased. The morphological changes of a river channel may be traced by studying aerial photographs taken over a period of time, if available.

- Man-made influences affecting backwater at a bridge site may include additional damming resulting from the sediment delta at the upper end of a dam basin well above the full supply level. On the other hand, when a dam far upstream attenuates flood peaks through storage, the river channel far downstream tends to become shallower and narrower, which could lead to increased flood levels.

- Excavation of material under a bridge to reduce damming is not recommended because sediment build-up and bed forms rapidly reduce the extra capacity. The effect of scour whereby a greater opening is created, is to be ignored when determining the required waterway opening, because the most reliable damming formulae have been calibrated in the field and they automatically allow for scour under flood conditions.

The methodology (8.4) described above is based on a number of assumptions, such as normal flow in relatively straight reaches of streams with subcritical flow conditions prevailing. In view of the limitations and assumptions inherent in the method, a list of references to other methods is given in CSRA (8.7). It was found (8.1) that the Bradley equations provided realistic answers in cases where the bridge opening ratio (M) was less than 0,85. However, the equations lead to underestimation of backwater where M > 0,85. It was recommended that the d'Aubuisson equation should be used in these cases. This equation is described in the literature (8.6, 8.30).

8.5 SCOUR ESTIMATION

8.5.1 General remarks

Scour is a major mechanism responsible for the failure or partial failure of bridges during flood events. This section provides practising engineers with simple process descriptions and selected equations that could be applied to assess the potential effect of scour on bridge structures.

Scour is a complex process. Some of the factors contributing to the complexity are:

- non-homogenous mixtures of water and sediment;
- three-dimensional flow patterns at bridges during floods;
- difficulties in establishing the actual geometrical properties of rivers under extreme flood conditions;
- difficulties in observing actual scour depths and processes in rivers during floods; and
- highly variable properties of *in-situ* bed materials around bridge foundations.

Various researchers have attempted to address the complexity by assuming dominant variables and then deducing simplified relationships to describe scour. The result has been a multitude of divergent approaches for determining scour. Generally, more weight should be attached to relationships that are fundamentally sound, i.e. that are based on a sound understanding of the underlying mechanisms involved in scour and which have been calibrated against actual prototype data for rivers, rather than laboratory data.

Total scour is often described in terms of the components of long-term and short-term general scour, contraction scour and local scour. This approach is used in this section.

Local scour is dealt with for both piers and abutments. As is common practice, distinction is drawn between scour in alluvial materials and scour in cohesive materials. Whereas the scour resistance in alluvial materials is linked to grain size, particle shape and armouring effects, the scour resistance in cohesive materials such as clay, is linked to the physiochemical properties of the material. Scour in cohesive materials consequently only takes place when the physiochemical bonds are broken (8.20).

Many equations for scour estimation in the literature have been derived as envelope curves from experimental data. Practical experience has been that due to scale effects laboratory conditions rarely provide proper simulation of sediment transport and scour processes in actual rivers under flood conditions. This implies that such equations have to be treated with circumspection. The focus in this section is on equations that have been either deduced from studies on actual rivers or have been calibrated against available prototype data.

8.5.2 Concepts and definitions

Incipient motion

Various relationships exist that define the boundary conditions under which a stream will begin to erode material along its bed and banks. The best known of these is probably the Shields relationship, which is currently still in use and is based on a representative particle size for a specific material density.

However, it has been argued (8.22) that particle size is neither a representative, nor a unique measure of transportability of sediments. Rooseboom (8.22) instead recommends the **use of settling velocity of particles in alluvial streams and critical tractive strength of clay in cohesive materials as representative of the transportability of sediments**.

Analysis of incipient movement in terms of stream power considerations has led to the representation of incipient motion of cohesionless materials as shown in **Figure 8.10** (Modified Liu Diagram), which expresses the boundary between sediment movement and no sediment movement in terms of a plot of a ‘shear Reynolds number’ against the ratio between shear velocity and settling velocity. For turbulent boundary conditions, the ratio between shear velocity and settling velocity (which represents the ratio of applied power over power required to suspend particles) is constant.

$$\frac{V_{*C}}{V_{SS}} = 0{,}12 \qquad \ldots(8.4)$$

with

$$V_* = \sqrt{gDS} \qquad \ldots(8.5)$$

where:

V_*	=	‘shear velocity’ (m/s)
V_{*C}	=	‘critical shear velocity’ (m/s)
g	=	gravitational acceleration (9,81 m/s^2)
D	=	flow depth (m)
S	=	energy slope (m/m)
V_{SS}	=	particle settling velocity (m/s) (**Figure 8.11**)

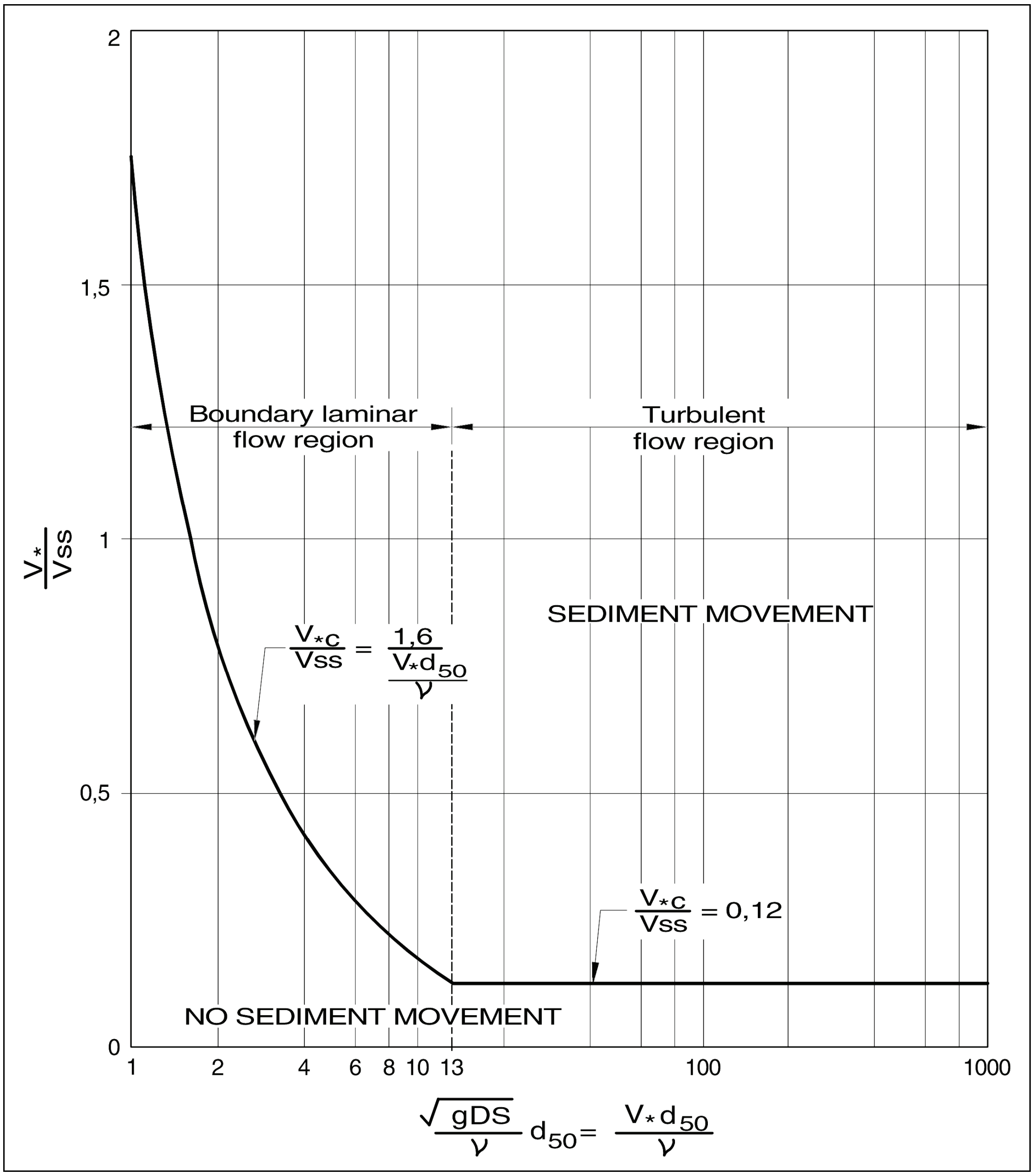

Figure 8.10: Modified Lui Diagram showing the relationships for incipient sediment movement

For laminar boundary conditions, the critical condition between sediment movement and no movement is given by:

$$\frac{V_{*C}}{V_{SS}} = \frac{1{,}6}{\frac{V_* d_{50}}{\nu}} \qquad \ldots(8.6)$$

where:

d_{50} = average particle diameter (m)

υ = kinematic fluid viscosity (m^2/s)

The boundary between turbulent and laminar conditions is defined by:

$$\frac{V_* d_{50}}{\nu} = 13 \qquad \ldots(8.7)$$

For values less than 13, the boundary flow condition is laminar and for values more than 13, boundary flow is turbulent. This boundary coincides with a particle size in the order of 5 mm.

The relationship between settling velocity and particle diameter is shown in **Figure 8.11**.

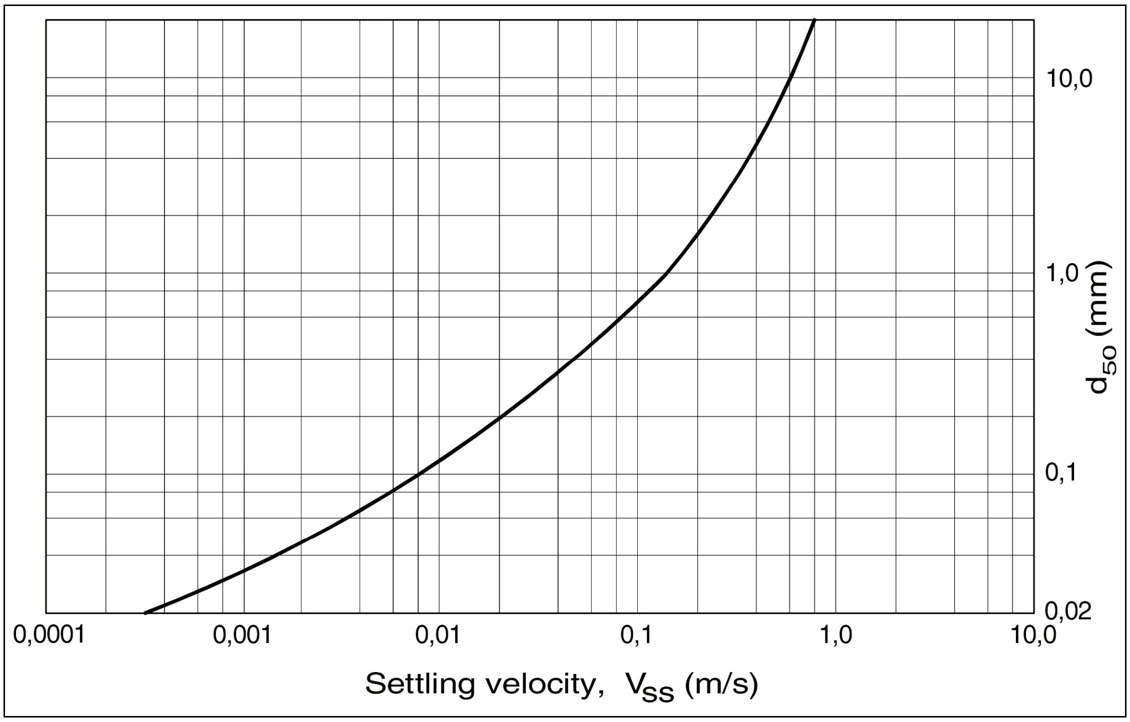

Figure 8.11: Settling velocity as a function of the sediment size
(Shape factor not taken into consideration) [8.13]

Sediment loads
South African rivers generally carry a significant amount of fine sediments in suspension. There is no singular relationship between discharge and suspended sediment load. Instead, the limiting factor tends to be the availability of transportable materials in upstream catchments. The amount of transportable material may be related to various factors such as rainfall, soil erodibility, slope and land-use [8.25]. These factors vary both in space and time. This means that the same discharge may be associated with sediment concentrations that vary by a factor of a thousand or more [8.7].

Total scour
Most approaches to scour determination, consider total scour as the sum of separate components that have been referred to before. These are long-term general scour short-term general scour, contraction scour and local scour [8.7].

Long-term general scour results in permanent deformation of the riverbed. The mechanisms that trigger long-term general scour may be natural (geomorphological changes due to events such as floods) or man-made (directly downstream of major dams). Determination of these long-term effects needs to be done by specialists as part of the feasibility studies for dams and other major projects.

Short-term general scour often makes the most significant contribution to the total scour at a bridge [8.7]. An estimation of the effects of short-term general scour could be made by using relationships for determining equilibrium flow depths and widths. Some of these relationships based on regime considerations and applied stream power considerations are provided in further sections of this manual.

Contraction scour takes place because of a decrease in channel width at a bridge site, which increases the unit width discharge (see **Photograph 8.4**). The effect of contraction scour is automatically discounted in the short-term general scour relationships, provided that the width of flow at the bridge is carefully determined [8.7].

Local scour results from the obstruction caused by piers, piles and abutments on the stream flow (see **Photograph 8.3**). This has proven to be a fertile research area for many years, with numerous relationships in existence that describe the local scour depth at piers of different configurations. The resulting scour depths calculated by means of the different relationships tend to vary greatly. There are many reasons for this, the main being that calibrations tend to be based on laboratory tests, which are subject to serious scale effects. An additional complicating factor is that actual river flow patterns differ significantly from laboratory conditions, with the result that some directives on scour strongly discourage the use of local scour formulae that are solely based on laboratory results [8.10].

In terms of Equation 8.7, physical models for studying bed scour should not contain particles smaller than about 5 mm. For hydraulic similarity, the plotted values of the functions on the vertical and horizontal axes in **Figure 8.10** for a prototype and a model should coincide.

Some areas of local scour are under-researched. These include scour in cohesive materials and scour at abutments. More prototype calibrations of scour formulae are needed in general.

Photograph 8.2: General scour

Photograph 8.3: Local scour at pier

Photograph 8.4: Constriction scour (Courtesy of: Free State Department of Transport)

8.5.3 Scour estimation

General remarks

The equations presented for the purpose of scour estimation have been selected to represent prototype conditions as far as possible. For short-term general scour, the regime equations (8.2,8.9) are recommended. A method based on applied stream power and calibrated on a limited number of South African prototype data is presented as a check method for total scour in alluvial rivers. For scour at piers and abutments, the regime equations of Blench and the HEC-18 equations (Colorado State University, or 'CSU' equations) are recommended. Although the CSU equation was derived on the basis of model studies, it has also been calibrated with limited prototype data.

Scour at abutments is problematic, with limited information being available. Regime equations and some rough factors presented by Faraday and Charlton (1983) (8.9) are presented for an initial estimate of the scour depth at abutments.

It is recommended that, where possible, scour depths should be calculated by more than one method and the answers compared. Judicious selection of the most representative scour depths should then be made, using other available data such as the results of Penetration Tests and engineering judgement.

Long-term general scour

Long-term general scour should be considered as part of the domain of specialists. Specialists will typically use procedures developed in the field of large dam engineering to determine the influence of major structures and make use of fairly sophisticated modelling tools in order to quantify long-term river aggradation or degradation effects. In addition, a multi-disciplinary team should do some long-term scour predictions, as knowledge of hydraulics, geomorphology, geology, etc. is required.

Short-term general scour

Short-term general scour in alluvial channels:

Faraday and Charlton (8.9) recommend the following relationships for the equilibrium dimensions for a channel, based on the work of Blench (8.2):

$$y = 0{,}38q^{0{,}67}D_{50}^{-0{,}17} \qquad \ldots(8.8)$$

and

$$B = 14Q^{0{,}5}D_{50}^{0{,}25}F_s^{-0{,}5} \qquad \ldots(8.9)$$

where:

- B = mean channel width (m)
- y = mean depth of flow (m)
- Q = equivalent steady discharge which would generate the channel geometry (m^3/s)
- q = discharge per unit width (Q/B) ($m^3/s.m$) (Note: To estimate channel geometry conditions under flood conditions the design flood flow may be used.) (8.9)
- D_{50} = median size of bed material (m)
- F_s = side factor to describe bank resistance to scour (**Table 8.2**)

The following side factors may be applied in the channel width equation (Equation 8.9):

Table 8.2: Side factors

Bank type	Value of F_s
Sandy loam	0,1
Silty clay loam	0,2
Cohesive banks	0,3

It is necessary to calculate the equilibrium width before the equilibrium depth can be calculated. The maximum channel depth, y_{max} can be determined by multiplying the calculated equilibrium depth with the factor from **Table 8.2**. Now the short term general scour depth, d_s, can be determined as the difference between y_{max} and the normal flow depth (y_n).

Short-term general scour in cohesive bed channels:

For cohesive bed channels, the equilibrium depth equation becomes:

$$y = 51{,}4n^{0{,}86} q^{0{,}86} \tau_c^{-0{,}43} \qquad \ldots(8.10)$$

where:

y	=	mean depth of flow (m)
n	=	Manning's coefficient of roughness ($s/m^{1/3}$)
q	=	discharge per unit width ($m^3/s.m$)
τ_c	=	critical tractive stress for scour to occur (N/m^2) –Refer to **Table 8.3**

Table 8.3: A guide to assessing the physical properties of clay [(8.9)]

Voids ratio	2,0-1,2	1,2-0,6	0,6-0,3	0,3-0,2
Dry bulk density (kg/m^3)	880-1200	1200-1650	1650-2030	2030-2210
Saturated bulk density (kg/m^3)	1550-1740	1740-2030	2030-2270	2270-2370
Type of soil	**τ_c - Critical tractive stress (N/m^2)**			
Sandy clay	1,9	7,5	15,7	30,2
Heavy Clay	1,5	6,7	14,6	27,0
Clay	1,2	5,9	13,5	25,4
Lean clay	1,0	4,6	10,2	16,8

The bulk densities in this table assume a specific particle density = 2,64 and the relationship with the voids ratio reads as follows:

$$\rho_d = \frac{\rho s}{e+1} \qquad \ldots(8.11)$$

and

$$\rho_s = \frac{\rho(s+e)}{e+1} \qquad \ldots(8.12)$$

where:

ρ	=	density of water (kg/m^3)
ρ_d	=	dry bulk density (kg/m^3)
ρ_s	=	saturated bulk density (kg/m^3)
s	=	specific gravity of soil particles
e	=	voids ratio of soil mass

Detailed descriptions of the type of soil (e.g. clay, lean clay, etc.) have not been provided [(8.9)]. It is proposed that the Casagrande classification be followed where 'lean clay' is clayey silts (CL), 'clay' is clay of medium plasticity (CI), 'heavy clay' is taken as clays of high plasticity (CH) and 'sandy clay' is well graded sands with small clay content (SC).

General:

The mean flow depth (y) calculated by means of the Equations 8.8 and 8.10, needs to be adjusted in order to calculate maximum flow depths that might result from short term general scour. The recommended factors are provided in **Table 8.4**.

Table 8.4: Factors to convert mean flow depth (y) to maximum channel depth

Description	Multiplying factor
Straight reach of channel	1,25[(*)]
Moderate bend	1,50
Severe bend	1,75
Right-angled abrupt turn	2,00

*Note: * Neill [(8.20)] recommends that this factor be increased to 1,50 in cases where dune movement takes place on the riverbed.*

The basic assumptions for which the Blench equations are valid include:

- steady flow;
- negligible bed transport;
- sediment transport through turbulent suspension that is sufficiently limited not to influence the calculations;
- channel sections and slopes that are uniform;
- viscosity that does not vary significantly;
- conditions under which the equations are applied that are similar to conditions for which the equations had been deduced. These conditions represent the ideal situation, which is not often found in rivers under extreme flood conditions [(8.2)]. Nevertheless, the equations are approximately valid as long as discharge and bed transport of sediment do not vary too quickly; and
- flow is in the rough turbulent phase.

Contraction scour

Rooseboom in TRH 25 [(8.7)] indicated that the formulae for short-term general scour may take contraction scour into account. The flow width that is used is set to the contraction width and by applying Equations 8.8 or 8.10 the contraction scour based on regime theory is calculated.

Where an existing bridge is evaluated which has a width less than the equilibrium flow width of the channel, or where fixed banks occur, the contraction scour depth has to be determined. In the following relationships a **distinction is made between sediment-laden and clear water flow**.

To test whether sediment-laden flow occurs, determine the average particle size, d_{50}, of the sediment in the river upstream of the bridge area. Use **Figure 8.11** [(8.13)] to determine its settling velocity V_{ss}. Calculate the value of Equation 8.7 to determine if the flow is in the laminar or turbulent region. To calculate the critical shear velocity, apply Equation 8.6 for laminar flow or Equation 8.4 for turbulent flow. The velocity at the boundary between sediment movement and no sediment movement (the 'critical' velocity), V_c, is determined from the logarithmic relationship:

$$V_c = 5{,}75 V_{*c} \log \frac{12R}{k_s} \qquad \ldots(8.13)$$

where:

R and k_s represent the hydraulic radius and the absolute roughness value just upstream of the bridge.

If the approach velocity V > V_c, sediment-laden flow takes place, else clear water flow occurs. The latter tends to occur at bridge openings on flood plains (relief bridges) where the velocity is lower and the resistance to scour greater. Velocity and sediment size become important in clear water scour estimates and therefore a different set of equations is used to estimate constriction scour for clear water flow conditions.

Sediment-laden flow:

For definition of terms see **Figure 8.12**. The equation below is applicable to a constriction of the river and was adapted from HEC 18 [(8.10)], based on the assumption that in southern African conditions the mode of bed material transport is mostly suspended bed material discharge:

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \left(\frac{B_1}{B_2}\right)^{2/3} \left(\frac{n_2}{n_1}\right)^{1/3} \qquad \ldots(8.14)$$

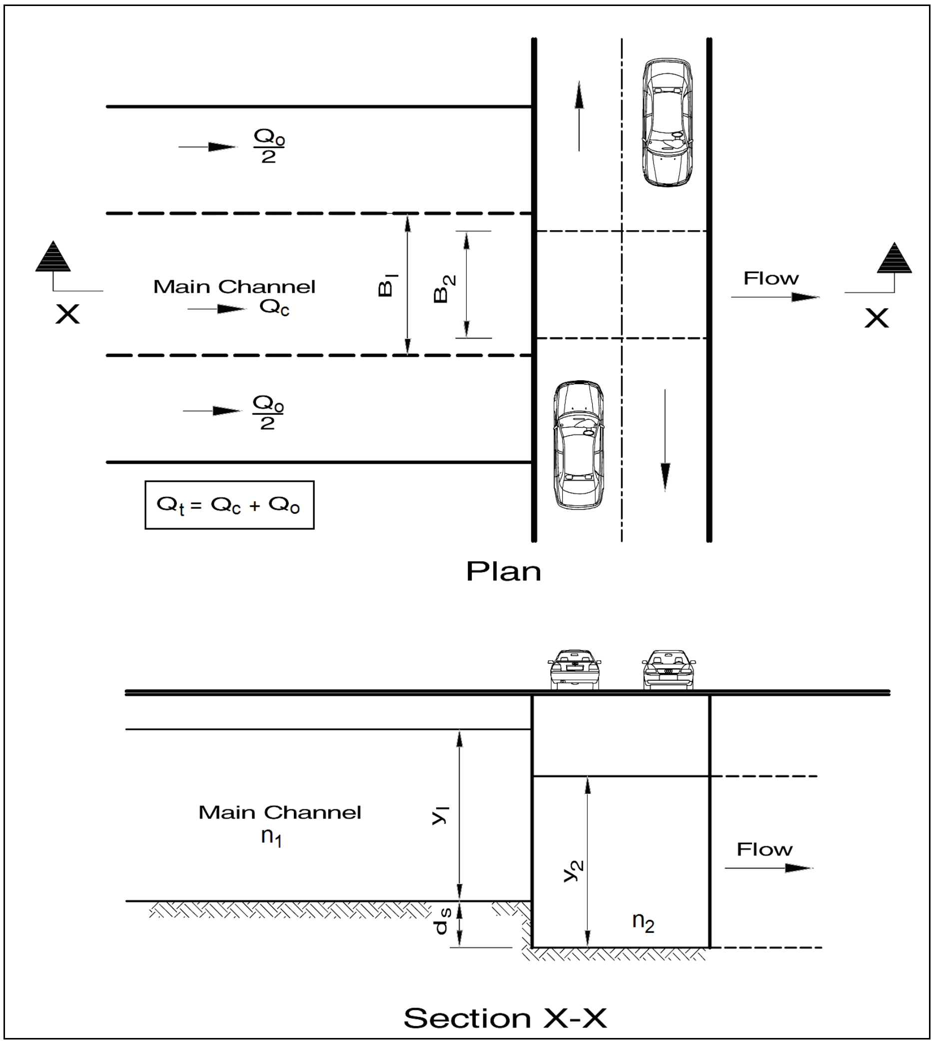

Figure 8.12: Long constriction in sediment-laden flow: definition of terms

The depth of scour is given by:

$$d_s = (y_2 - y_1) + (1 + K)\left(\frac{\bar{v}_2^2 - \bar{v}_1^2}{2g}\right) \qquad \ldots(8.15)$$

where K is the secondary energy loss coefficient for the constriction (refer to Section 4.2.7). If the degree of constriction is slight, it will be found that the last term in Equation 8.15 is negligible, and can be neglected.

The equations above apply to subcritical flow, with uniform flow upstream and downstream of the transition. The bed material is non-cohesive and is identical in both the wide and the constricted parts. The effect of varying sediment characteristics has been investigated [(8.32)] and found to play only a significant part in the case of severe constrictions.

Clear water flow:

For a definition of terms see **Figure 8.13.** The equation below was taken from HEC 18 [8.10].

$$y_2 = \left[\frac{Q^2}{40 D_m^{2/3} B_2^2} \right]^{3/7} \qquad ...(8.16)$$

and d_s is calculated from Equation 8.15.

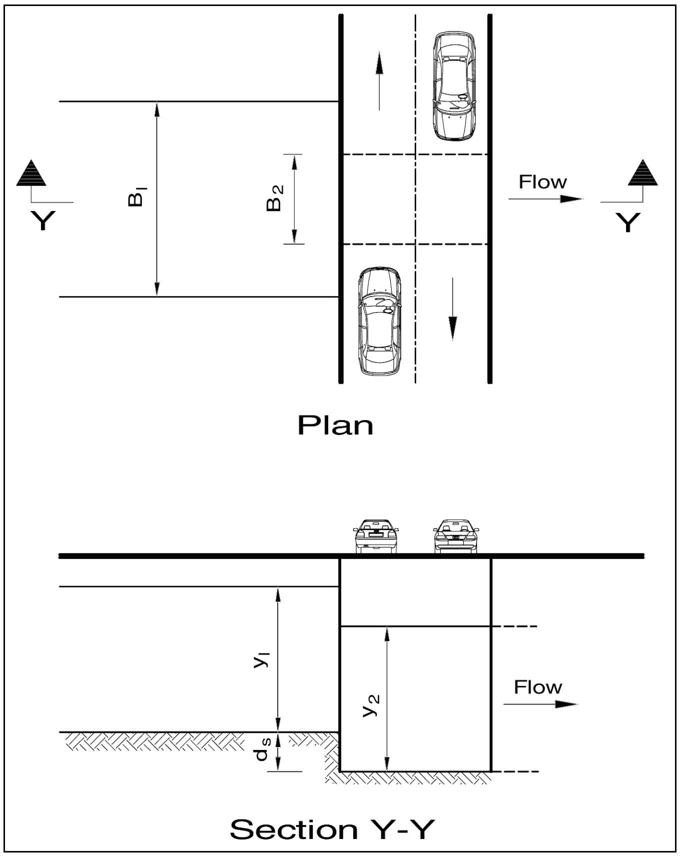

Figure 8.13: Long constriction in clear water flow: definition of terms

Note that Equation 8.16 is based on a Shields parameter of 0,039 and an assumption of homogeneous bed material. The term **D_m is the effective mean bed material size and is equal to 1,25D_{50}**.

The equations above for scour in constrictions should be regarded as the **first level of analysis** and experts using sediment transport models can perform a more detailed analysis.

Local scour

Local scour is difficult to quantify. However, its contribution to total scour in southern Africa is mostly not as critical as the contributions of short-term general and contraction scour.

The mechanism causing local scour is complex and is characterised by a downward flow at the upstream face of the pier (caused by stagnation pressure), a horse-shoe vortex at the base of the pier (which removes sediments) and oscillating slipstream vortices at the back of the pier that further remove sediments [8.18].

The Blench equation for local scour showed reasonable agreement when calculated values and a limited number of measured South African prototype values were compared [8.17].

The regime family of equations, of which this equation is a member, has the advantage of having been calibrated against actual observations of stream behaviour (mainly irrigation canals in India, of which the behaviour corresponds largely with that of prototype rivers). This equation is recommended for use under South African conditions.

The second set of equations are the CSU equations, which is the most frequently used formula in the USA (HEC 18) [(8.10)], and is provided for comparative purposes. The formula was derived from laboratory data and compared with limited prototype data.

A comparison was made between most of the existing formulae with published sets of prototype data by Johnson [(8.15)]. Unfortunately the regime formulae were not included in this study. In addition, the prototype data used contained no information on bed forms, sediment grading and flow direction and consequently a number of assumptions had to be made. The findings tentatively indicated that some of the formulae should not be used as they potentially under-predict local scour. These include the equations of Shen et al. (1969) [(8.27)] and Hancu, as quoted [(8.3)]. The equations by Melville and Sutherland [(8.19)] were found to over-predict scour significantly. The CSU equations [(8.10)] are conservative, although it predicted local scour reasonably well under most conditions.

Local scour at piers in alluvial channels (cohesionless material):

Blench [(8.3)] presented the following equation for calculating local scour in cohesionless material at bridge piers:

$$d_s = 1{,}8 y_0^{0{,}75} b^{0{,}25} - y_0 \qquad \dots(8.17)$$

where:

d_s	=	local scour depth at pier (m)
y_0	=	depth upstream of pier (m) (calculated by means of regime Equation 8.8)
b	=	pier width (m)

This depth is recommended for local scour at cylindrical piers. Corrections for other pier shapes should be done by multiplying the value obtained from Equation 8.17 with the correction factors in **Table 8.5**. To take the angle of attack into account, the correction factors in **Table 8.6** have to be used.

The conditions under which the Blench formula is valid were discussed under the section on general short-term scour.

The CSU equation [(8.10)] is recommended for comparative purposes:

$$\frac{y_s}{b} = 2{,}0 K_1 K_2 K_3 K_4 \left(\frac{y_1}{b}\right)^{0{,}35} Fr_1^{0{,}43} \qquad \dots(8.18)$$

where:

y_s	=	scour depth (m)
y_1	=	flow depth directly upstream of pier (m)
K_1	=	correction for pier nose shape
K_2	=	correction factor for angle of attack of flow
K_3	=	correction factor for bed condition
K_4	=	correction factor for armouring due to bed material size
b	=	pier width (m)
L	=	pier length (m)
Fr_1	=	Froude number directly upstream of the pier

$$Fr_1 = \frac{\bar{v}_1}{\sqrt{gy_1}} \qquad \ldots(8.19)$$

where:

$\bar{v}_1$ = mean velocity upstream of the pier (m/s)

g = gravitational acceleration (9,81 m/s^2)

The correction factors are given in **Tables 8.5** to **8.7**.

Table 8.5: Correction factor K_1, for pier nose shape

Shape of pier in plan #	Length/Width ratio (L/b)	K_1
Circular	1,0	1,0
Lenticular	2,0	0,91
	3,0	0,76
	4,0	0,67-0,73
	7,0	0,41
Parabolic nose		0,8
Triangular 60º		0,75
Triangular 90º		1,25
Elliptic	2,0	0,91
	3,0	0,83
Ogival	4,0	0,86-0,92
Rectangular	2,0	1,11
	4,0	1,11 (Hec 18) - 1,40 (F&C)
	6,0	1,11

Note: # Table 8.5 is based on the list by Faraday and Charlton (1983) [8.9]*, which is more complete than the list in HEC 18 documentation* [8.10]

Table 8.6: Correction factor K_2, for angle of attack of the flow

Angle (skew angle of flow)	L/b = 4	L/b = 8	L/b = 12
0	1,0	1,0	1,0
15	1,5	2,0	2,5
30	2,0	2,75	3,5
45	2,3	3,3	4,3
90	2,5	3,9	5,0

Note: In the case of L/b larger than 12, the ratio's for L/b = 12 should be used.

Table 8.7: Correction factor K_3, for bed conditions

Bed condition	Dune Height (m)	K_3
Clear-water scour	Not applicable	1,1
Plane bed and anti-dune flow	Not applicable	1,1
Small dunes	0,6 m – 3 m	1,1
Medium dunes	3 m – 9 m	1,1 – 1,2
Large dunes	≥ 9 m	1,3

The value of the correction factor for armouring, K_4, could be determined from the following sets of equations:

$$K_4 = [1 - 0{,}89(1 - v_R)^2]^{0{,}5} \qquad \ldots(8.20)$$

where:

$$v_R = [\frac{v_1 - v_i}{v_{c90} - v_i}] \qquad \ldots(8.21)$$

and

$$v_i = 0{,}645\left[\frac{D_{50}}{b}\right]^{0{,}053} v_{c50} \qquad \ldots(8.22)$$

with:

v_R = velocity ratio
v_1 = approach velocity (m/s)
v_i = approach velocity when particles at pier begin to move (m/s)
v_{c90} = critical velocity for D_{90} bed material size (m/s)
v_{c50} = critical velocity for D_{50} bed material size (m/s)
b = pier width (m)

and

$$v_c = 6{,}19 y^{1/6} D_c^{1/3} \qquad \ldots(8.23)$$

where:

D_c = critical particle size for the critical velocity v_c (m)

Local scour at piers in cohesive bed channels

The limited data were used to compile a rough guide on the expected scour depths at piers in cohesive material. **Table 8.8** reflects the findings (8.20).

Table 8.8: Local scour depths at piers in cohesive materials (8.20)

Pier shape (plan)	Inclination of pier faces	Depth of local scour (b=pier width)
Circular	Vertical	1,5b
Rectangular		2,0b
Lenticular		1,2b
Rectangle with semi-circular noses	Vertical	1,5b
	Inclined inwards towards top. (Angle more than 20˚ to vertical)	1,0b
	Inclined outwards towards top (Angle more than 20˚ to vertical)	2,0b

Local scour at abutments in alluvial channels (cohesionless material)

Local scour at abutments is difficult to quantify. Approximate estimates of scour may be made by applying an appropriate factor selected from the following table (**Table 8.9**) to the general (short term) average scour depth in order to obtain a maximum depth.

Table 8.9: Factors for estimating scour depth at abutments and training works [(8.9)]

Description	Factor
Nose of groynes or guidebanks	2,0 to 2,75
Flow impinging at right angles on bank	2,25
Flow parallel to bank	1,5 to 2,0

In cases where the abutments protrude into the river channel, a conservative approach is recommended in which the local scour level is taken as the lower value of the maximum scour at piers and the general scour level multiplied by a factor of 2,0 [(8.9)].

Local scour at abutments in cohesive bed channel:

In cases of cohesive scour at abutments, Faraday and Charlton [(8.9)] recommended the use of the appropriate Blench normal depth equation for cohesive beds, with the correction factors for maximum depth given in **Table 8.9**. Although these factors were derived for alluvial materials, they provide a first estimate of the scour in cohesive soils.

8.5.4 A check method for total scour based on applied stream power principles

The equation presented in this section was derived from applied stream power principles.

The assumptions for which the equation is valid are the following:

- Flow is one-dimensional. There is increasing evidence that general and contraction scour are dominant in sand-bedded rivers [(8.21, 8.23, 8.31)], which means that this assumption is approximately valid for extreme flood conditions.
- Flow is uniform and steady. Flow changes during floods, but under the assumption that these changes take place relatively slowly, flow may be considered uniform and steady at any specific moment in time.
- Flow is in the rough turbulent flow zone. Plotting the design flow data used in **Figure 8.10** and confirming whether the design flow conditions are rough turbulent may check this assumption.
- Equilibrium conditions prevail. This is not the case, but the use of the instantaneous flood peak value for calculations is thus conservative.

The total scour values calculated by means of this equation were compared with limited observed prototype data for five South African rivers. For this limited data set, the results were consistent. The equation is recommended as a check to be used in conjunction with the other equations for alluvial rivers provided in previous sections.

The form of the equation recommended for the calculation of scour in rivers is [(8.17)]:

$$\frac{C(Y_t)(v_{ss}k_s)^{1/3}}{q\sqrt{g}} = F \qquad \ldots(8.24)$$

where:

C = Chézy coefficient

$$Y_t = Y_0 + Y_s \qquad \ldots(8.25)$$

where:

Y_t = total maximum scour depth (m)
Y_0 = maximum general scour depth (m)
Y_s = local scour depth (m)

v_{ss} = particle settling velocity (m/s)
k_s = absolute roughness of river bed (m)
q = discharge per unit width (m³/s.m)
g = gravitational acceleration (9,81 m/s²)
F = constant obtained from measured data

The value of the constant, F, as calibrated on limited South African prototype data, is recommended at 0,8 as an envelope value. The depth of scour was assumed to be equal to the depth of piers, which failed, whilst their neighbours remained intact.

Estimation of the bed roughness in rivers under extreme flood conditions is obviously important in the case of this equation. In this regard, it is interesting to note the size of bed forms that were calculated [8.16, 8.29] based on observations of eyewitnesses, as well as evidence on video recordings of both the Domoina and 1987 KwaZulu-Natal floods. Typical bed form sizes varied between 1m and 3 m, while at a number of bridges (notably the John Ross bridge), waves of up to 5 m were observed. It was stated [8.20] that bed forms might be as high as half the depth of flow. The size of bed forms and associated absolute roughness could, however, be determined [8.24] and the results may be used to calculate the depths to which piers become exposed as bed forms move past. A number of piers, which went down 10 m in sand beds of rivers in South Africa, have failed due to scour. A depth of scour of 10 m is thus not unprecedented.

8.5.5 Special cases of scour

Scour at bridges in tidal areas

The mechanisms that cause scour at bridges in tidal zones are similar to those at bridges across non-tidal rivers. The implication is that the same relationships are valid to describe these scour mechanisms in tidal rivers [8.10]. The challenge lies in describing tidal flow conditions, which differ from non-tidal conditions. For a more detailed discussion of tidal effects at bridges, references that deal with tidal effects at bridges should be consulted [8.7, 8.10].

Where tidal processes are found to be complex, it is recommended that a specialist should quantify the flow conditions and recommend input variables to determine the effects of scour.

Pressure scour

One of the fundamental assumptions underlying most scour equations is the assumption of free-surface flow conditions. However, many bridges are designed to be overtopped at flows with recurrence intervals of between 20 and 50 years. It has been argued [8.14] that, while it **may be economically feasible for bridges to overtop at these intervals, a bridge foundation failure at these recurrence intervals would probably not often be found to be economically acceptable**.

At the stage when a bridge starts to be overtopped, the flow pattern at the bridge starts to resemble culvert flow, i.e. the flow becomes pressurised. In limited laboratory experiments conducted on pressure scour, it was found that scour increases relative to the depth under free-surface flow conditions. This could be ascribed mainly to the influence of the submerged bridge deck. The correction factors for this [8.14] are unfortunately based on a limited number of clear-water scour laboratory tests and are consequently not necessarily representative of the actual flood conditions in South African rivers. It is suggested that the assistance of a specialist should be enlisted in cases where pressure scour is considered to be problematic. The scour protection measures as described for lesser culverts can also be considered for smaller bridges.

8.5.6 Lateral movement of rivers

Lateral movement of the main channel of a river within a floodplain may have serious impacts on scour and bridge stability. These may include increased scour depths at piers and abutments, erosion of approaches to the bridge structure and changed angles of flow relative to piers and abutments. Lateral movement is influenced by factors such as the geomorphologic properties of the stream, bridge location, flow characteristics and characteristics of the bed and bank materials [8.10]. Inspection of the river is crucial in understanding the potential for lateral movement at the bridge site.

The common-sense understanding of these processes underlying the regime theory may be used as a framework to consider the potential impact of lateral stream movement at a bridge site.

In this regard, the following points may be useful [8.20]:

- The equilibrium geometry of rivers is established under high, but not extreme, flow conditions;
- A constriction in channel width would lead to increased scour of the river bed;
- Straightening of the river (e.g. in the case of river training) may result in large-scale local erosion if care is not taken; and
- Fixing the channel geometry at one point in an alluvial channel (e.g. at a bridge site) would probably lead to increased channel movement at another location.

The typical scour related problems that could be encountered as part of the dynamic processes in rivers were summarised and are shown in **Table 8.10**.

Table 8.10: Typical scour related problems that may be encountered in rivers [8.20]

River type	Stream type	Materials typically exposed	Dominant geomorphologic processes	Potential scour-related problems
Steep mountainous rivers	Boulder torrent	Bedrock/boulders	'Downcutting' and waterfalls	Erosion of river banks
	Braided gravel river	Sand, gravel, cobbles	Movement of coarse alluvium	Scour, choice of length of openings
	Alluvial fan	Sand, gravel, cobbles	Deposition of coarse alluvium; sudden channel shifts	Control of approach channel geometry; scour
Streams with moderate slopes	Entrenched river channel	Bedrock, shale	Thin layer of material is transported	Few
	Laterally meandering river	Sand, gravel, cobbles	Widening of river valley, sediment transport	Bank erosion and outflanking of bridge openings; scour; erosion of bridge approaches
Plains and streams with flat slopes	Meandering alluvial river	Sand and silt	Migration of meanders; erosion of river banks	Erosion of river banks and outflanking of bridge openings
		Clay, silt, cobbles	Degradation, erosion of river banks	Erosion of river banks
	Low velocity stream with multiple windings	Silt, sand	Relatively inactive	n.a.
	Lake crossings	Silt, clay, organics	n.a.	Soft foundations
Tidal areas	Deltas	Silt, sand	Deposition and frequent movement of meanders	Location of bridge openings, soft foundations
	Tidal estuaries	Silt, sand	Sediment deposition and multi-directional flow conditions	Soft foundations, scour
	Tidal basins	Sand, pebbles, clay, rock	Sediment transport, multi-directional flow	Abutment scour due to wave action

8.5.7 Procedures for initial estimation of scour at bridges

Seven steps that need to be undertaken to conduct an initial estimation of scour at bridges are discussed in the following paragraphs.

Step 1: Data acquisition phase

The following information may be relevant in estimating the effects of scour at bridges and should thus be gathered (8.10):

- Borehole logs define the geology of the bridge site (obtain advice from a geotechnical expert in defining the spacing and depth of drilling required). In addition to valuable information on the properties of the bed material and strata, the logs may potentially yield clues as to previous scour depths at the site (8.20).
- Bed material size, gradation and distribution in the vicinity of the bridge.
- Existing stream and floodplain cross-sectional survey information from upstream to downstream of the bridge, including all channel geometry details needed *inter alia* to perform backwater calculations for the site. The normal principles for defining geometries and calculating backwater profiles are valid and cross-sections should be selected with these in mind.
- Catchment characteristics. Remember to make a catchment inspection part of the site inspection.
- Any available scour data at structures in the vicinity of the site.
- Energy slope of the river for design flows. This is derived from backwater calculations for the bridge design.
- Historical flooding information. In the case of South Africa, valuable information may often be gleaned for the major rivers from reports compiled by the Department of Water Affairs and Forestry (DWAF) after major flood events.
- Location of the bridge site relative to other bridges, river features such as tributaries, confluences, etc. and human-made controls such as dams, training work, etc.
- Character of the river and its flows. Is it a stable perennial stream, or is it subject to flash floods, etc.?
- Geomorphology of the site.
- Erosion history of the stream where available.
- Development history of the stream, and the catchment.
- Other relevant factors.
- Aerial photographs of the area may yield valuable insights. Also useful may be a comparison of older photographs with more recent ones, where these are available.

From the above information an initial qualitative assessment of the potential scour impact on the bridge should be made.

Step 2: Analyse long-term change

Preferably consult an expert, especially if there are any major dams constructed or being planned in the vicinity of a bridge. The expert may use techniques, such as extrapolation of existing trends, worst-case scenarios or sophisticated computer modelling and good engineering judgement to make recommendations on potential long-term effects.

Step 3: Determine short-term general and contraction scour effects

The following parameters feed into the scour equations and need to be determined:

- Estimate the design flood for the site and use this value for further calculations (as described in **Chapter 3**).

- Estimate the contracted flow width at the bridge site for the purpose of determining short-term general and contraction scour effects. The unit width discharge could be calculated by dividing the design flood by the flow width.
- Make estimates of the bed roughness under flood conditions and representative sediment material sizes to be used in calculations.

For alluvial material:

- Use Equation 8.8 to determine mean flow depth and Equation 8.9 to determine mean channel width (if width is required), with side factor values from **Table 8.2**.
- Convert mean flow depth to maximum flow depth using the factors in **Table 8.4**.
- Calculate the scour depth as the difference between the maximum flow depth and the normal flow depth.

For cohesive material:

- Estimate cohesive material properties using **Table 8.3** and Equations 8.11 and 8.12.
- Use Equation 8.10 for mean flow depth.
- Convert to maximum flow depth using **Table 8.4**.
- Calculate the scour depth as the difference between the maximum flow depth and the normal flow depth.

Contraction scour is factored into Step 3 through the judicious choice of flow width at the bridge site. If an existing bridge or a channel with fixed banks is considered, apply the set of constriction scour Equations 8.13 to 8.16. The maximum scour depth from the different analyses should be used.

Step 4: Determine local scour effects

For piers in alluvial cohesionless materials:

- Use Equations 8.17 and 8.18 to compute local scour in two different ways. Obtain the factors needed for Equation 8.17 from **Tables 8.5** and **8.6**. Obtain the factors needed for Equation 8.18 from **Tables 8.5**, **8.6** and **8.7** and Equations 8.20, 8.21, 8.22 and 8.23. Compare answers obtained from Equations 8.17 and 8.18 and select a conservative answer using good engineering judgement.

For piers in cohesive materials:

- Use **Table 8.8** as a rough guide to estimate local scour at the piers.

For abutments in alluvial cohesionless materials:

- Apply factors in **Table 8.9** to the general short-term average scour depth obtained from Equation 8.8.

For abutments in cohesive materials:

- Apply the factors in **Table 8.9** to the short-term average scour depth obtained from Equation 8.9. This is a preliminary indication only.

Step 5: Determine total scour

Total scour is the sum of long-term general scour (where applicable), short-term general scour, (contraction scour) and local scour.

For total scour at piers in alluvial rivers, check the answer against values obtained by means of Equations 8.24 and 8.25. Select design values on the basis of good engineering judgement. Try and corroborate calculations with available published and on-site evidence.

Step 6: Plot design values

Plot the design scour depth values using the design water level (e.g. 1:50 year return period water level, as determined by the methods described in **Chapter 4** for a fixed-bed configuration).

Step 7: Assess the results obtained

Assess the results, taking into account all available qualitative and quantitative information available. In cases of significant complexity, or cases where significant financial and other risks exist, consider consulting an expert or doing a physical hydraulic model study of the bridge site.

8.5.8 Concluding remarks on the relationships for scour at bridges

The equations presented in this section provide bridge designers with simple methods in which an **initial estimate** of the potential effects of scour at bridges may be made. If serious problems are foreseen, expert advice should be acquired.

8.6 SCOUR COUNTERMEASURES AT BRIDGES

8.6.1 Introduction

Scour countermeasures at bridges are aimed at reducing the negative impact of shear stresses and turbulence and velocity variations near the boundary of the structure-water interface and of macroturbulent flow processes, such as eddies and helicoidal flows around bends [(8.20)].

This section does not attempt to provide a comprehensive coverage of all available scour countermeasures. Instead, it provides information on some of the frequently considered options that are available and potentially useful under local conditions.

The scour countermeasures are described below. These countermeasures broadly follow the categories into which these countermeasures have been divided previously [(8.12)]:

Hydraulic countermeasures (mainly river training structures and revetments) are designed to either modify erosive flow characteristics or to provide resistance against hydraulic and turbulence effects**.** **River training structures** may have to be put into place to counter the effects of bank erosion, migrating meanders and other dynamic river processes at bridge sites [(8.9)]. They are divided into transverse, longitudinal and areal types, depending on their orientation relative to the flow direction. The structures that are broadly discussed in following sections are **spurs** (transverse) and **dykes** and **berms** (or **guide banks)** (longitudinal). **Revetments and bed armouring** protect channel beds and banks against the erosive effects of river flow through the provision of a protective layer, covering a specified area of the channel. Appropriate revetment types may include riprap, gabions, precast concrete blocks, *in-situ* concrete and steel sheet piling. Considerations that need to be taken into account when making a selection include the extent of protection needed, cost, ease of construction and maintenance and environmental issues[(8.9)]. A fairly comprehensive subsection on **riprap protection** has been included. Some of the issues related to the use of **gabions** are also discussed. **Local scour armouring** uses similar protective layers to revetments, but has to take into account local flow patterns around structural elements, such as **piers** and **abutments**. Riprap protection of local elements is dealt with fairly comprehensively in the following sections.

Structural countermeasures involve the design of structural elements and foundations of bridges to minimise scour effects. These are broadly discussed.

Countermeasures during the maintenance phase include monitoring and implementation of further scour countermeasures when scour problems are identified.

Table 8.11, which was adapted from available information [8.12] provides a broad overview of the applicability of various scour countermeasures and is shown on separate pages.

8.6.2 General considerations in respect of scour countermeasures

Scour countermeasures at bridges should be seen as an integrated approach aiming to reduce the risk of scour damages to vulnerable structures. The approach should be a considered one in which design and construction of scour countermeasures are complemented with actions, such as continuous monitoring and where necessary, ongoing maintenance and expansion of scour protection measures [8.12].

The effective design and implementation of scour countermeasures should be an interdisciplinary effort, with inputs from hydraulic, geotechnical and structural engineers [8.12].

Selection of scour protection measures should be undertaken with environmental impact, construction and maintenance implications kept in mind [8.12].

Most importantly, the benefits of scour protection should be measured against the cost of provision of these countermeasures [8.12, 8.20]. For new bridges, the **most economical options tend to be the lowering of foundation depths to levels well below estimated scour depths and ensuring that abutments are placed back from eroding river banks [8.12].**

It is important to keep in mind that countermeasures themselves are often damaged and that erosion processes may take place in locations where this may not have been envisaged. This means that a large component of scour protection measures may often be implemented during the maintenance phase, as an economical option [8.12].

The FHWA[8.10, 8.11, 8.12] recommends the following principles in the design of bridge scour countermeasures:

- Comparison of costs against benefits is of prime importance except in certain cases where routes are of strategic importance.
- Designs should be based on ‘channel trends’ and experience of similar field situations is extremely valuable.
- The environmental impacts of scour countermeasures have to be assessed.
- The designer should personally undertake a field inspection trip of the site and the river and catchment upstream and downstream of the bridge site.
- Any previous evidence of dynamic changes in the vicinity of the site (such as early photographs) is useful.
- Geotechnical and soil characteristics that may impact on the design of countermeasures should be determined and taken into account.
- Many of the countermeasures induce complex interaction with the river and its environment. This means that a physical hydraulic model study may often be justified to study the impact of these complex interactions and to determine potential unforeseen effects.
- Often the dynamic nature of scour processes implies that not all effects may be foreseen at the stage that the bridge is initially designed and constructed. This means that an inspection and maintenance plan is usually essential in order to affect ongoing countermeasures as and when required.

In the following paragraphs some of the scour countermeasures are discussed in more detail.

Table 8.11: Overview of the applicability of various scour countermeasures [8.12]

Type	Local scour		Contraction scour	Instability		River type	Stream size	Bend radius	Velocity	Bed material	Debris load	Bank condition	Flood plain	Resource allocation
	Abutments	Piers		Vertical	Lateral	B=Braided M=Meander S=Straight	W=Wide M=Med S=Small	L=long M=Med S=Short	F=Fast M=Med S=Slow	C=Coarse S=Sand F=Fine	H=high M=Med L=Low	V=Vertical S=Steep F=-Flat	W=Wide M=Med N=Narrow/ None	H= High M=Med L=Low
Transverse structures														
Impermeable spurs	□	□	x	x	■	B,M	W,M	L,M	●	●	●	●	●	M-L
Permeable spurs	□	□	x	x	■	B,M	W,M	L,M	M,S	S,F	L	●	●	H-M
Drop structures	□	□	□	■	x	●	●	●	●	●	●	●	●	M
Longitudinal structures														
Guide banks	■	□	□	x	□	●	W,M	●	●	●	●	●	W,M	M-L
Areal structures														
Canalisation	□	□	x	x	■	B,M	●	●	●	●	●	●	●	M
Sediment detention basin	x	x	x	■	□	●	●	●	●	C,S	●	●	●	H-M
Revetments and bed armour														
Rigid														
Soil cement	■	□	□	□	■	●	●	●	●	S,F	●	S,F	●	L
Concrete pavement	■	□	■	□	■	●	●	●	●	●	●	S,F	●	M
Rigid grout filled mattress/ concrete mat	■	□	□	□	■	●	●	●	●	●	●	S,F	●	M
Flexible														
Riprap	■	□	□	□	■	●	●	●	●	●	●	S,F	●	M
Gabion mattresses	■	□	□	□	■	●	●	●	●	S,F	M,L	●	●	M

Table 8.11: Overview of the applicability of various scour countermeasures [8.12] (continued)

Type	Local scour		Contraction scour	Instability		River type	Stream size	Bend radius	Velocity	Bed material	Debris load	Bank condition	Flood plain	Resource allocation
	Abutments	Piers		Vertical	Lateral	B=Braided M=Meander S=Straight	W=Wide M=Med S=Small	L=long M=Med S=Short	F=Fast M=Med S=Slow	C=Coarse S=Sand F=Fine	H=high M=Med L=Low	V=Vertical S=Steep F=-Flat	W=Wide M=Med N=Narrow/ None	H= High M=Med L=Low
Local scour armouring														
Riprap	■	□	N/A	N/A	N/A	●	●	●	●	●	●	S,F	●	H-M
Gabions	■	□	N/A	N/A	N/A	●	●	●	●	●	●	S,F	●	H-M
Foundation strengthening														
Continuous spans	x	■	■	■	x	●	●	●	●	●	●	●	●	L
Pumped concrete/grout	■	■	□	□	□	●	●	●	●	●	●	●	●	M
Lowering foundations	■	■	■	■	■	●	●	●	●	●	●	●	●	L
Pier geometry modification														
Extended footings	N/A	■	N/A	N/A	N/A	●	●	●	●	●	●	●	●	L
Pier shape modifications	N/A	■	N/A	N/A	N/A	●	●	●	●	●	●	●	●	M

Legend for the notation used in Table 8.11.

■ primary use/well suited

□ secondary use/potential application

x not suitable/ seldom used

N/A not applicable

● suitable for the full range of the characteristic

8.6.3 Spurs, Berms and Dykes

Spurs (groynes) are intended to control the movement of river meanders and erosion of river banks [8.9]. These may be placed either upstream or downstream of the area to be protected. Spurs should preferably be used in groups to either 'repel' or 'attract' flow. Spurs usually require some scour protection themselves. Examples of spur lay-outs are given in **Figures 8.14 to 8.16**.

It is recommended that the following formula for spacing groups of spurs should be used [8.9]:

$$L_s < \frac{Cy^{1,33}}{2gn^2} \qquad \text{…(8.26)}$$

where:

- L_s = spacing between spurs (m), see **Figure 8.16**
- C = a constant (approximately 0,6)
- y = mean depth of flow (m)
- n = Manning's roughness coefficient (s/m$^{1/3}$)
- g = gravity acceleration (m/s^2)

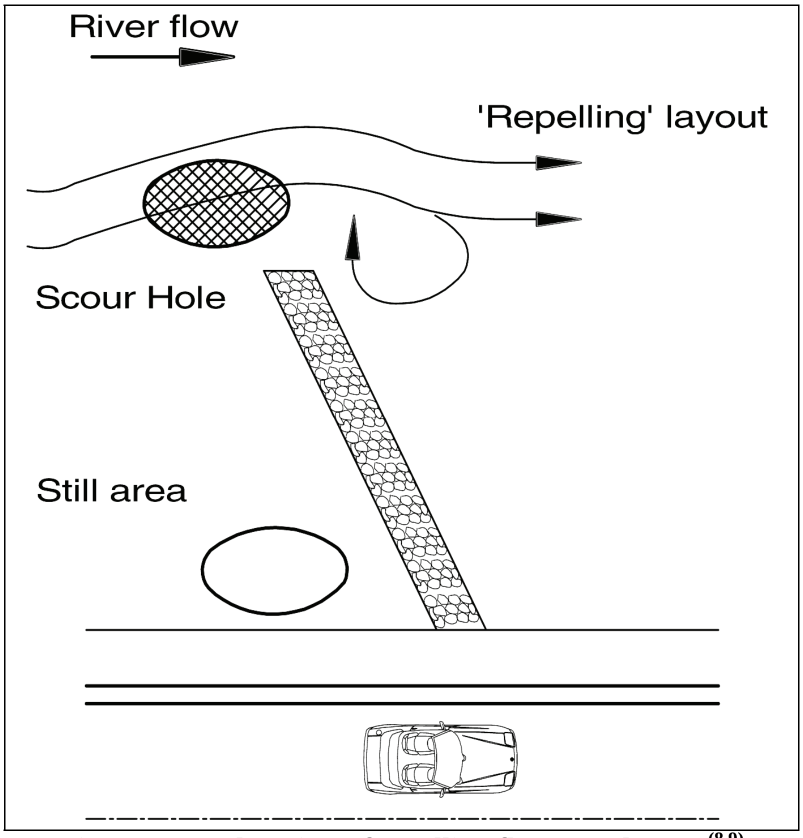

Figure 8.14: Plan view of repelling flow spur layout [8.9]

On smaller narrow rivers it should be carefully considered whether spurs offer the most appropriate form of erosion protection, as the use of these on one bank may result in erosion on the opposite bank.

The interaction of factors influencing the layout and spacing of spurs is complex and a model study is recommended for most cases. An expert should preferably be involved in the design of spurs.

Economic considerations should feature strongly in the final layout decision.

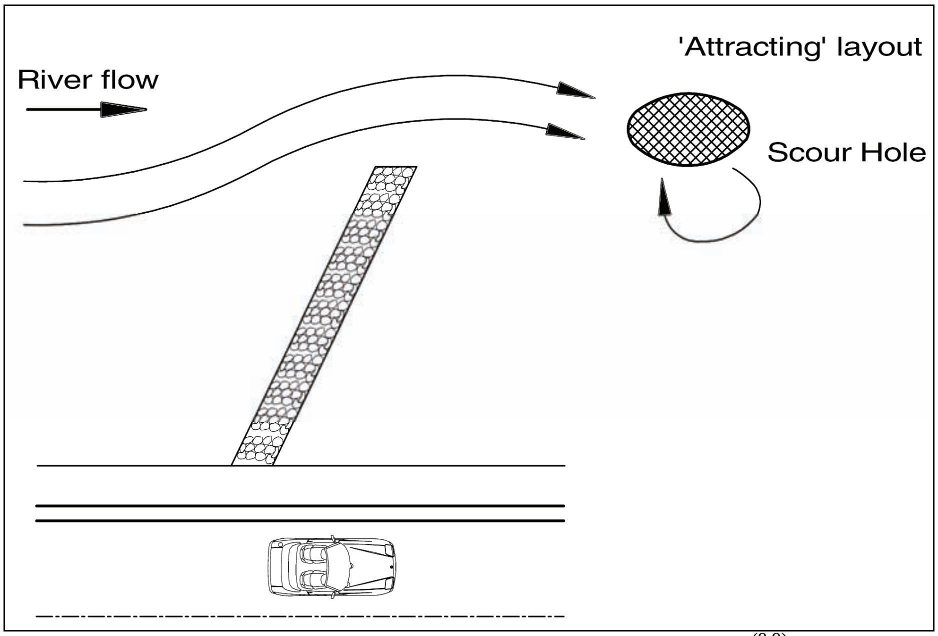

Figure 8.15: Plan view of attracting spur layout (8.9)

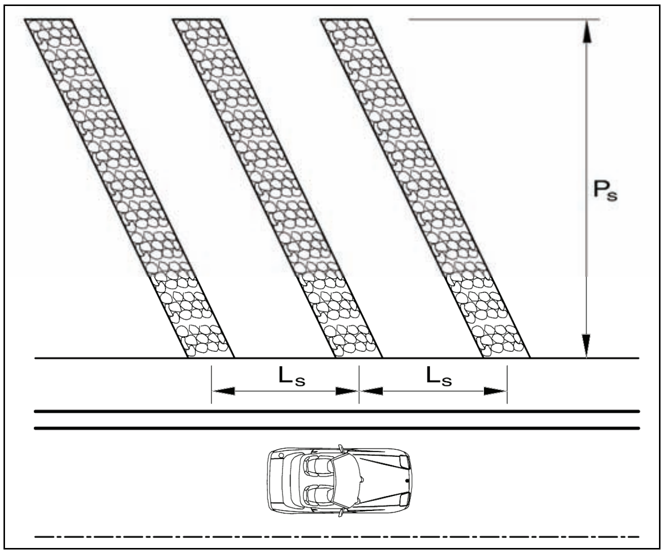

Figure 8.16: Plan view of groups of spur for repelling flow (8.9)

Berms (Guide banks)

Berms may have various uses, including:

- protecting the bridge and approaches from erosion through elimination of lateral stream movement patterns;
- improving flow distribution through the bridge opening and in the process also reducing the effects of contraction scour; and
- changing the angle of the approach flow.

The main design issues are orientation relative to the bridge opening, shape, length upstream and downstream of the bridge and crest levels (8.12).

The plan layout of berms should conform to good hydraulic principles, in addition to providing adequate protection against lateral stream movement. Their height should normally allow for the design water level plus freeboard(8.20). Guide banks normally need some protection against undermining. An expert should preferably undertake or review the design of guide banks.

Dykes

Dykes are linear structures that prevent or control overbank flow. These may typically be used to prevent flood flow from bypassing the bridge opening. Hydraulic model studies to optimise the layout of dykes are usually justified for major bridges [(8.9)].

8.6.4 Revetments: Riprap protection

General comments

Riprap is defined as follows [(8.5)]:

> "A flexible channel or bank lining or facing consisting of a well graded mixture of rock, broken concrete, or other material, usually dumped or hand-placed, which provides protection from erosion."

Riprap protection serves its main purpose in reducing the effects of local scour [(8.12)]. It has been fairly popular because of the availability of materials, ease of construction and low costs [(8.12)].

Riprap layers may fail in various ways, which should be considered during the design process. Some of these failure modes may even cause riprap protection to fail where the stone sizes are adequate [(8.12)]. The failures that might be encountered with riprap layers, are summarised below:

- **Riprap particle erosion**. Various factors may contribute to this type of failure. These include inadequate stone sizing, removal of particles through various causes and steep slopes where the angle of repose of the material could be easily overcome [(8.5)]. This failure mode is limited by sizing of particles to withstand hydraulic and turbulent forces, but factors such as slope, waves and vandalism may not be addressed in this way.

- **Mass failure**. This failure mode happens when a large mass of the riprap slides or slumps because of the effect of gravity. This may happen because of pore water pressure, steep slopes or loss of support (such as undermining of toe support at abutments) that may be ascribed to erosive processes. Undermining and failure of toe support is considered to be a primary reason for revetment failure [(8.5)] and significant attention should be paid to this aspect during design and construction.

- **Substrate particle erosion or base material failure**. In this case, the underlying materials are displaced through erosion or fails and slumps [(8.5)]. This may be countered through the use of filters. However, care has to be exercised in the design of the filters to ensure that water pressure does not build up due to filter blockages.

Special care should be taken with the edges of revetments, such as the head, toe and flanks, to ensure that undermining does not occur [(8.12)].

Riprap stone sizing for revetments

Much of the understanding of riprap design is based on laboratory experiments, with limited field data to verify this understanding. This should be kept in mind when using the design formulae presented for use.

A relationship for the required particle size that was derived from tractive force theory has been recommended for the determination of the riprap size [(8.5)]. The formula was derived for straight channels with uniform flow conditions, but a coefficient in the formula allows for flow conditions that deviate from these ideal conditions. The formula is:

$$D_{50} = \frac{K_u C \bar{v}_a^3}{d_{avg}^{0,5} K_1^{1,5}} \qquad \ldots(8.27)$$

where:

D_{50}	=	median riprap size (m)
K_u	=	0,0059 (SI units)
C	=	coefficient for specific gravity and stability factors
$\bar{v}_a$	=	average velocity in the main channel (m/s)
d_{avg}	=	average depth in the main channel (m)
K_1	=	a factor defined by Equation 8.28

$$K_1 = \left[1 - \left(\frac{\sin^2\theta}{\sin^2\varphi}\right)\right]^{0.5} \qquad \ldots(8.28)$$

where:

θ	=	bank angle with the horizontal (°)
φ	=	riprap angle of repose (°)

The angle of repose for various riprap types may be read from **Figure 5.17**.

The coefficient for specific gravity and stability factors may be obtained from the following equation:

$$C = \frac{1,61(SF)^{1,5}}{(S_s - 1)^{1,5}} \qquad \ldots(8.29)$$

Here the following values for the variables are used:

S_s	=	specific gravity of riprap (usually approximately 2,65)
SF	=	required stability factor to be applied (typically varies between 1,2 and 2,0)

The recommended values for the stability factor, SF, are shown in **Table 8.12**.

Table 8.12: Recommended values for stability factor, SF (8.5)

Flow description	Stability Factor
Uniform flow, no significant bends, little uncertainty regarding parameters involved.	1,0 –1,2
Gradually varied flow, moderate bend curvature, moderate debris impact	1,3 – 1,6
Rapidly varied flow; significant bends; high turbulence (e.g. at abutments); high parametric uncertainty	1,7 – 2,0

Grading of riprap and stone characteristics

Riprap should in general be well graded in order to ensure maximal interlocking of particles. Individual stones should have a length to width ratio of not more than 1:3 (8.5). Also materials should not weather easily or be prone to chemical wear.

Table 8.13 reflects the grading, which is recommended for riprap.

Table 8.13: Recommended grading of riprap [8.12]

Diameter	Weight	Percentage passing
1,5 D_{50} to 1,7 D_{50}	3,0 W_{50} - 5,0 W_{50}	100
1,2 D_{50} to 1,4 D_{50}	2,0 W_{50} – 2,75 W_{50}	85
1,0 D_{50} to 1,15 D_{50}	1,0 W_{50} – 1,5 W_{50}	50
0,4 D_{50} to 0,6 D_{50}	0,1 W_{50} – 0,2 W_{50}	15

It is recommended [8.12] that the 85% requirement be dropped in the case where the above specification would overburden certain smaller quarries.

Riprap layer thickness

Guidelines [8.5] stipulate that riprap layer thickness should generally not be less than the greater of the D_{100} stone diameter or 1,5 times the D_{50} stone diameter. The layer should have a minimum thickness of 300 mm for practical purposes. In cases where the riprap is placed under water, layer thickness should be increased by 50% because of uncertainties involved in the placement process [8.5].

Design of filters and filter materials

Filters have a dual purpose to fulfil as part of a scour protection system. Firstly the filters have to prevent fine materials from leaching out underneath riprap layers or other protection layers. Secondly, the filter material has to provide for drainage to prevent build-up of pore pressure [8.12].

A geotechnical specialist should preferably undertake the design or be consulted on the design results.

As a broad guideline, the following formula may be used for the design of filter material [8.12]:

$D_{15(\text{Coarser Layer})}/D_{85(\text{Finer Layer})} < 5 < D_{15(\text{Coarser Layer})}/D_{15(\text{Finer Layer})} < 40$

The left side of the inequality provides for erosion prevention and the right side for sufficient permeability.

Revetments: Gabions and stone mattresses

Gabions and stone mattresses may be an option where available rock is of too small size and of lesser quality than would be required for riprap. The manufacture of gabions and rock mattresses is labour intensive, which may be considered an advantage in instances where local job creation is considered an important component of projects.

However, the disadvantages of gabions and stone mattresses have to be carefully considered prior to use [8.5]. These include the higher costs of installation and maintenance relative to riprap and the decrease in flexibility of the system that is created. Whereas riprap particles may move individually to adapt to geometrical deformations, gabions and stone mattress systems are more rigid. More rigid systems generally have a higher potential for catastrophic failure.

The failure mechanisms that need to be considered in the case of gabions and stone mattresses are:

- failure of the wire mesh of the baskets; and
- movement of stones within the baskets that exposes base materials, with the potential of subsequent base material erosion and system failure.

For these reasons, gabions and stone mattresses are only recommended for small streams, preferably with no vertical stability problems [8.12].

8.6.5 Local scour armouring

Sizing of riprap at piers

The standard Isbash formula, recommended by FHWA in HEC-11[8.5] is:

$$D_{50} = \frac{0{,}692(Kv)^2}{2g(S_s - 1)} \qquad \text{…(8.30)}$$

where:

D_{50} = riprap size (m)
v = velocity along the pier (m/s)
S_s = specific gravity of riprap (approximately 2,65)
K = pier shape coefficient (1,5 for round-nosed and 1,7 for rectangular piers)

The velocity along the pier may be calculated by multiplying the average channel velocity by a coefficient that varies between 0,9 for a pier near the bank of a uniform reach to 1,7 for a pier in the main current at a bend in the river.

Riprap mat dimensions at piers

The following may be considered as guidelines to the dimensions of riprap protection that are required (**Table 8.14**):

Table 8.14: Recommended riprap protection dimensions

Dimension	Recommended
Horizontal	Twice pier width on both sides
Thickness	At least three stone diameters (D_{50})
Maximum rock size	Not more than twice D_{50} of riprap

The following should be considered when deciding on the level at which the riprap should be placed:

FHWA [8.5, 8.12] considers it to be disadvantageous to place riprap at such a depth that the top of the riprap is buried below stream level, as this creates problems during inspections to determine the extent to which riprap has been damaged or removed. The top level of the riprap layer should, therefore, be at the invert level of the streambed. This is in contrast with the view of other references [8.9] that the top of the riprap should be below the expected general scour levels.

Riprap should, however, **never be placed at levels higher than the streambed invert** [8.12].

Sizing of riprap at abutments

Two equations are recommended, HEC-11 [8.5], for the sizing of riprap at abutments:

For **Froude numbers less or equal to 0,80 -** the Isbash relationship is recommended:

$$\frac{D_{50}}{y} = \frac{K}{S_s - 1}\left[\frac{\bar{v}^2}{gy}\right] \qquad \text{…(8.31)}$$

where:

D_{50} = median stone diameter (m)
$\bar{v}$ = characteristic average velocity in the contracted section (m/s)
S_s = specific gravity of riprap
g = gravitational acceleration (m/s^2)

y = depth of flow in the contracted bridge opening (m)
K = 0,89 for a spill-through abutment
= 1,02 for a vertical wall abutment

For **Froude numbers in excess of 0,80 -** the recommended equation reads:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{\bar{v}^2}{gy}\right]^{0,14} \qquad \ldots(8.32)$$

with the symbols having similar meanings as in the previous equation, and

K = 0,61 for spill-through abutments or
K = 0,69 for vertical wall abutments

The characteristic average velocity in the contracted section method of calculation is as follows:

A **setback ratio (SBR)** is calculated for each abutment. (This is the ratio of the distance from the near edge of the main channel to the toe of the abutment to the channel flow depth).

If the SBR is less than five for both abutments, the characteristic velocity is based on the entire contracted area through the bridge opening.

If SBR is greater than five for one abutment, the characteristic velocity could be based on the overbank flow only. It should be assumed that the entire overbank flow passes through the bridge section in the overbank section only.

In cases where the SBR on one side is less than 5 and on the other side more than 5, the characteristic velocity for the side less than 5 should be based on a flow area bounded by the abutment and an imaginary boundary along the opposite main channel bank.

It is recommended that the HEC-11 equations described in the previous sections be used as well as a control method before an engineering judgement is made. In this case, the velocity in the vicinity of the abutment should be used, instead of the main channel velocity, with a stability factor of between 1,7 and 2,0 to allow for high turbulence. It should be remembered that all these equations are mainly based on laboratory data and should be treated as such.

In complex cases or where damage may be costly, scale model studies may be an attractive option. It should be remembered that riprap is not generally the preferred method of scour protection for new structures and alternative protection measures should be considered.

Dimensions of riprap protection at abutments

The extent to which riprap should be placed is as follows [(8.12)]:

- The apron at the toe of the abutment should extend along the entire abutment toe around curved portions to the tangent point with the embankment slopes.

- The horizontal dimension of the riprap into the overbank area of the river should be twice the flow depth on the overbank area in the vicinity of the abutment, but not less than 1,5 m [(8.22)] or more than 7,5 m [(8.12)].

- For spill-through abutments, riprap layers should be placed to a height of at least 0,6 m above the high water level for the design flood. Downstream placements should extend back from the abutment for twice the flow depths, or 7,5 m (to 15 m), whichever are greater.

- Riprap layer thickness should not be less than the larger of 1,5 times D_{50} or D_{100}. If the riprap is placed under water, this thickness should be increased 1,5 times. The minimum thickness should be 300 mm.

Abutments should be inspected after every major flood event for scour damage.

8.5.6 Structural countermeasures

Adherence to certain general principles during the design of the bridge may reduce the effects of scour (8.20). These may include:

- Locating the invert levels of pier footings well below the maximum estimated scour levels and preferably on rock, where possible (8.22);
- Putting piers on piles or columns that extend to a great depth below scour levels;
- Designing slender structural elements that do not provide a significant flow obstruction and orienting piers and abutments to ensure good hydraulic flow patterns;
- Reduction of number of piers and widening of bridge opening to reduce contraction scour (8.12);
- Designing the bridge superstructure to be above water (and debris) level at design flood levels will reduce the risk of pressure scour, which could have severe effects.

8.6.7 Maintenance measures

It is important to note that maintenance measures include not only the continuous monitoring of scour patterns at bridges, but also knowing which actions need to be taken once a problem has been identified (8.12).

Maintenance measures include continuous inspection of scour prone bridges after flood events, the removal of debris caught on piers and the repair of any scour damage observed after major flood events (8.20).

8.7 OTHER DESIGN ASPECTS

A number of design aspects are described below (TRH 25: Volume 1, Chapter 5) (8.7).

Piers and pier spacing

The number of piers and alignment of piers should be chosen to minimise contraction effects at the bridge site (8.11). Lateral scour has been identified as the major cause of serious hydraulic problems at bridges in the USA (8.7). Ideally, the total opening width should not be less than the equilibrium channel width corresponding to the design discharge. Alignment should be in the flow direction at the flood stage, which may differ from the flow direction at low flows. Local scour depths are determined by the projected pier widths at the levels where scour occurs.

Pier footings should be located **below the maximum estimated scour depth** and should preferably be placed on piles that extend to a significant depth below the expected scour depth. Piers should be designed to be stable when partially exposed due to scour. Piers on flood plains should be protected against bank scour and meandering (by ensuring sufficient foundation depths). Foundation depths may well be similar to those of the piers in the main river channel (8.11).

Piers should not encroach on the main channel of small rivers, where possible [8.11].

Piers and abutments should be designed to minimise the entrapment of debris, as this may impact on both contraction and local scour.

Pier footings may reduce local scour should they remain below the general scour levels for the bridge and if their horizontal dimensions are sufficient to cover the area over which the local vortex action takes place [8.20]. Should the pier footing be above general scour levels, the effect would be similar to that of a larger pier, with resulting increases in local scour depths.

Provision of additional capacity

In areas with significant bank stability problems, consideration should be given to the provision of extra bridge spans as an alternative to extensive bank and channel stabilisation measures [8.10], but not on the inside of bends, due to sediment build-up.

Monitoring

The complex nature of scour processes may lead to unforeseen scour effects becoming visible after design and construction of a bridge. Monitoring of structures after construction is thus important in order to detect any developing scour problems.

Scour

Where bridges are constructed close to each other, the effect of scour induced by one bridge at another should be carefully considered [8.20].

Scour depths determined by means of the equations presented in this section should always be compared to the depths at which scour-resistant strata are situated, as these may impose limits on the scour depths [8.20]. Good geotechnical knowledge is required to interpret the resistance to scour of different types of bed materials.

In cases where scour impact is deemed significant, but is difficult to predict and in cases where structures are expensive or of strategic importance, it may be useful to build a scale model of the structure and to do laboratory tests to determine scour patterns [8.20].

Drainage of bridge decks

Adequate discharge capacity should be provided to meet the requirements of Section 5.2. Refer to applicable design procedural codes. Water discharged from bridge decks should not be released directly onto railway lines, roads, streets, etc.

Forces acting on bridge structures

Besides provision for the normal hydrostatic and hydrodynamic forces acting on a bridge, provision should be made for forces as a result of debris and impact forces which result from debris colliding with the piers and overtopping forces (if appropriate). The effects of buoyancy should also be considered.

8.8 WORKED EXAMPLES

8.8.1 Worked Example 8.1 – Backwater at a bridge

Problem description Example 8.1

Determine the backwater caused by a proposed bridge across the Broekspruit. Details of the bridge are shown below in **Figure 8.17**.

Design discharge	Q	=	150 m^3/s
Average bed slope	S_o	=	0,00082 m/m
Angle of skew	ø	=	15º
Bridge span on skew	b_s	=	17,6 m
Projected bridge span	b	=	17,0 m
No of rows of piers	N_p	=	1
Projected width of pier	W_p	=	2,00 m

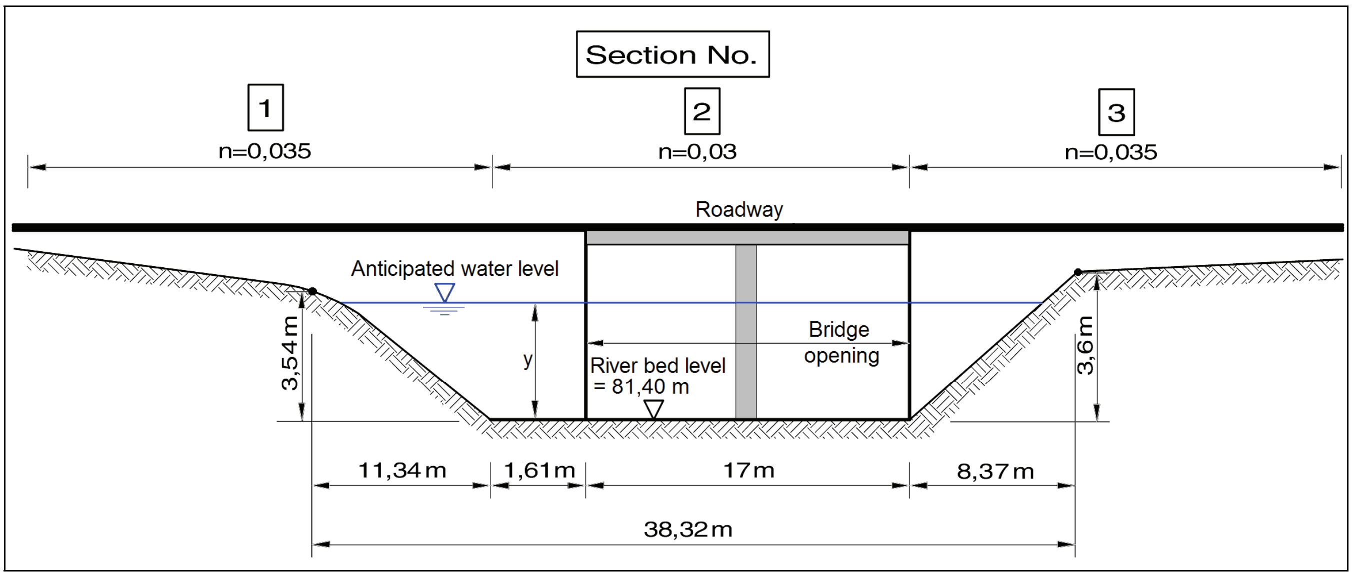

Figure 8.17: Upstream view of the bridge

Determine:

(i) Characteristics of the unconstricted flood state
(ii) The flow type
(iii) Bridge opening ratio
(iv) Velocity head coefficients
(v) Calculate backwater

Solution Example 8.1

(i) Characteristics of the unconstricted flood state:

First determine the normal flow depth, y_n.

Sub-section	n_i	A_i (m^2)	P_i (m)
1	0,035	$1{,}602y^2$	$(y^2 + 10{,}26\ y^2)^{0{,}5}$
2	0,030	$18{,}61y$	18,61
3	0,035	$1{,}163y^2$	$(y^2 + 5{,}41\ y^2)^{0{,}5}$

Utilising the Manning equation the normal flow depth can be calculated, $\mathbf{y_n = 3{,}157\ m}$.

Sub-section	$\mathbf{A_i}$ **($\mathbf{m^2}$)**	$\mathbf{P_i}$ **(m)**	$R_i = \frac{A_i}{P_i}$ **(m)**	$\mathbf{q_i}$ **($\mathbf{m^3/s}$)**	$\bar{v}_i = \frac{q_i}{A_i}$ **(m/s)**
1	15,96	10,59	1,51	17,17	1,08
2	58,75	18,61	3,61	120,69	2,05
3	11,59	7,99	1,45	12,14	1,05
	86,30	37,19		150,00	

This results in the:
Flood stage level = 84,56 m
Width at flood stage = 36,06 m

(ii) Determine flow type:

$$Fr_n = \left(\frac{Q^2 B}{g A_n^3}\right)^{1/2} = \left(\frac{(150)^2 (36{,}06)}{(9{,}81)(86{,}30)^3}\right)^{1/2}$$

$$= 0{,}359 < 1$$

Flow is Type I or Type II.

Calculate specific energy (E_{sn}) of unconstricted normal flow:

with y_n = 3,157 m (Flood stage level – river bed level)

$$\bar{v}_n = \frac{Q}{A_n} = \frac{150}{86{,}30}$$

$$= 1{,}738 \text{ m/s}$$

$$E_{sn} = y_n + \frac{\bar{v}_n^{\,2}}{2g} = 3{,}157 + \frac{(1{,}738)^2}{2(9{,}81)}$$

$$= 3{,}311 \text{ m}$$

Calculate specific energy (E_{sc}) of constricted flow critical depth:

$$y_{2c} = \left(\frac{Q^2}{g b^2}\right)^{1/3} = \left(\frac{(150)^2}{(9{,}81)(17)^2}\right)^{1/3}$$

$$= 2{,}817 \text{ m}$$

$$\bar{v}_{2c} = \frac{Q}{y_{2c} b} = \frac{150}{(2{,}817)(17)}$$

$$= 3{,}132 \text{ m/s}$$

$$E_{sc} = y_{2c} + \frac{\bar{v}_{2c}^{\,2}}{2g} = 2{,}817 + \frac{(3{,}132)^2}{2(9{,}81)}$$

$$= 3{,}317 \text{ m} > E_{sn} \text{ indicating Type II flow.}$$

Because the values of E_{sn} and E_{sc} are fairly close, and other losses are so far ignored, it would be prudent to check Type I and Type II flow.

(iii) Calculate bridge opening ratio:

$$Q_b = (120{,}69)\left(\frac{17}{17 + 1{,}61}\right) = 110{,}25 \text{ m}^3\text{/s}$$

$$M = \frac{Q_b}{Q} = \frac{110{,}25}{150} = 0{,}735$$

(iv) Calculate velocity head coefficients:

$$\alpha_1 = \frac{\sum\left(q\bar{v}^2\right)}{Q\bar{v}_n^{\,2}} = 1{,}20$$

α_2 = 1,15 (from **Figure 8.6**)

(v) Calculate backwater

For Type I flow:

Determine secondary energy loss coefficient K^* from **Figure 8.7**:

Projected area of piers in flow direction and projected area below normal water level.

$$A_p = W_p y_n = (2)(3{,}157) = 6{,}314 \text{ m}^2$$

$$A_{n2} = (b\cos ø)(y_n) = (17\cos(15))(3{,}157) = 51{,}84 \text{ m}^2$$

$$J = \frac{A_p}{A_{n2}} = \frac{6{,}314}{51{,}84}$$

$= 0{,}122$ (use J = 0,1 from **Figure 8.7**)

Eccentricity

$$e = 1 - \frac{Q_a}{Q_c} = 1 - \frac{12{,}14}{17{,}17 + (120{,}69)\left(1 - \frac{17}{17 + 1{,}61}\right)} = 0{,}44$$

From **Figure 8.7** and with ø = 15º:

$$K^* = 0{,}73$$

Approximate backwater (to estimate A_1 in Equation 8.2):

$$\bar{v}_{n2} = \frac{Q}{A_{n2}} = \frac{150}{51{,}84} = 2{,}894 \text{ m/s}$$

$$h_1^{*1} = K^*\alpha_2 \frac{\bar{v}_{n2}^{\ 2}}{2g}$$

$$h_1^{*1} = (0{,}73)(1{,}15)\frac{(2{,}894)^2}{2(9{,}81)}$$

$$= 0{,}358 \text{ m}$$

$$A_1 = A_n + h_1^{*1} B = (86{,}3) + (0{,}358)(36{,}06)$$

$$= 99{,}22 \text{ m}^2$$

Final estimate of backwater:

$$h_1^{*1} = h_1^{*1} + \alpha_1\left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right]\frac{\bar{v}_{n2}^{\ 2}}{2g} \qquad \text{(Equation 8.2)}$$

$$= (0{,}358) + (1{,}20)\left[\left(\frac{51{,}84}{86{,}30}\right)^2 - \left(\frac{51{,}84}{99{,}22}\right)^2\right]\left[\frac{(2{,}894)^2}{2(9{,}81)}\right]$$

$$= \mathbf{0{,}403\ m}$$

For Type II flow:

$$b_c = \left(b - \sum W_p\right) = 17{,}0 - 2{,}0$$

$$= 15{,}0 \text{ m}$$

C_b = 0,125 from **Figure 8.9**

$$y_{2c} = \left(\frac{Q^2}{g b_c^{\ 2}}\right)^{\frac{1}{3}} = \left(\frac{(150)^2}{(9{,}81)(15)^2}\right)^{\frac{1}{3}}$$

$$= 2{,}168 \text{ m}$$

$$\bar{y} = \frac{A_{n2}}{b} = \frac{51{,}84}{17}$$

$$= 3{,}050 \text{ m}$$

In 1st iteration, assume

$$\bar{v}_1 = \frac{Q}{A_n} = \frac{150}{86{,}30}$$

$$= 1{,}738 \text{ m/s}$$

for $\bar{v}_{2c}$ based on the net width:

$$\bar{v}_{2c} = (g y_{2c})^{0{,}5} = ((9{,}81)(2{,}168))^{0{,}5}$$

$$= 4{,}612 \text{ m/s}$$

$$h_1^{*1} = \alpha_2 \frac{\bar{v}_{2c}^{\ 2}}{2g}(C_b + 1) - \alpha_1 \frac{\bar{v}_1^{\ 2}}{2g} + y_{2c} - \bar{y}$$

$$= \frac{(1{,}15)(4{,}612)^2(0{,}125+1)}{2(9{,}81)} - \frac{(1{,}20)(1{,}738)^2}{2(9{,}81)} + (2{,}168) - (3{,}050)$$

$$= 1{,}403 - 0{,}185 + 2{,}168 - 3{,}050$$

$$= \mathbf{0{,}336\ m}$$

Adjust result for improved value of $\bar{v}_1$:

$$A_1 = A_n + h_1^{*1} B = (86{,}30) + (0{,}336)(36{,}06)$$

$$= 98{,}43 \text{ m}^2$$

$$\bar{v}_1 = \frac{150}{98{,}43}$$

$$= 1{,}524 \text{ m/s}$$

$$h_1^{*1} = 1{,}403 - \frac{(1{,}20)(1{,}524)^2}{2(9{,}81)} + 2{,}168 - 3{,}050$$

$$= \mathbf{0{,}379\ m}$$

Although the difference in this case is negligible, to be conservative, the higher value should be used. From the calculations h_1^{*1} for Type II flow was **0,379 m** which is less than the backwater calculated for Type I flow, thus Type I flow prevails i.e. h_1^{*1} = **0,403 m**.

Note that this example was also modelled in HEC-RAS (provided on the supporting CD) and that the highest backwater was obtained by the Standard Step Energy Method. The backwater is 300 mm, which is less than the value of 403 mm obtained above. However, in this model the ineffective flow area option had been used. The model was then re-run, with this option switched off and a higher backwater of 580 mm was obtained in the revised model.

Users of HEC-RAS should therefore carefully consider the option where the bridge approach conditions are “smoothed”, thereby reducing the backwater.

8.8.2 Worked Example 8.2 – Scour at a bridge

Problem description Example 8.2

Consideration is being given to construct a bridge across the Sand River, which is some 730 m wide at the proposed bridge site. The **potential scour at the bridge** should be determined. This problem was also evaluated with HEC-RAS, and the data files are included on the supporting CD as Example2.prj.

Figure 8.18 shows a plan view (obtained from the HEC-RAS problem evaluation) and the position of the bridge relative to the other cross-sections. The cross-sectional information for all the sections is available. The bridge will be positioned at cross-section 6.5 (downstream from cross-section 7 and upstream of cross-section 6).

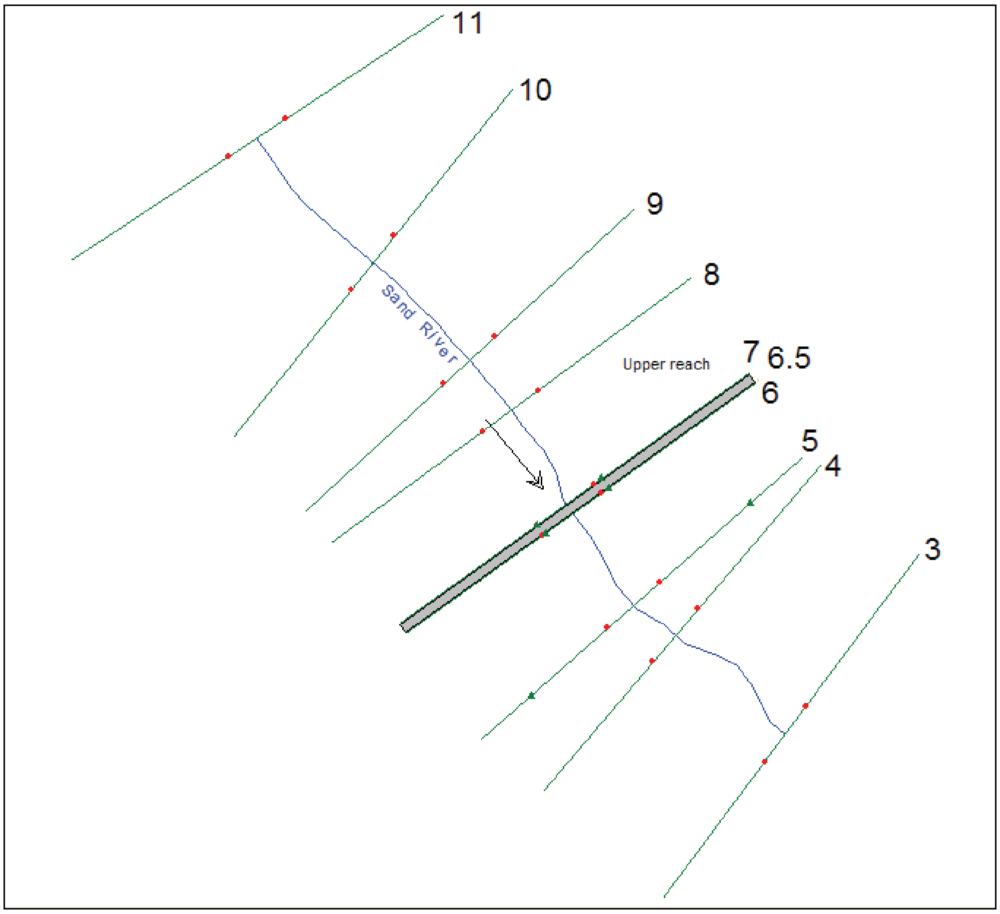

Figure 8.18: General layout of the cross-sections and the position of the bridge

The bridge data is described below and the bridge cross-section is shown in **Figure 8.19**.

The bridge opening between the sloping abutments is approximately 126,61 m wide and the bridge is supported by **five piers, each with a width of 1,5 m (equally spaced)**. The high (road surface) and low cord (bridge soffit) values for the bridge deck on the upstream side are 6,7 and 5,5 m respectively. The user can open the project (Example2.prj) in HEC-RAS and by selecting the appropriate icons, review the bridge data which is not repeated here in detail.

The design flow rate for which the scour analyses have to be conducted is the 1:100 year flood (Q_{100}), which has been determined to be **850 m³/s**.

The flow in the river is downstream control and the normal flow depth, y_n, could be calculated at the bridge, assuming a representative slope of 0,002 m/m.

Bed material characteristics

The sieve analyses (percentage passing) of the bed material revealed the following:

D_{50} = 0,0020 m
D_{90} = 0,0045 m

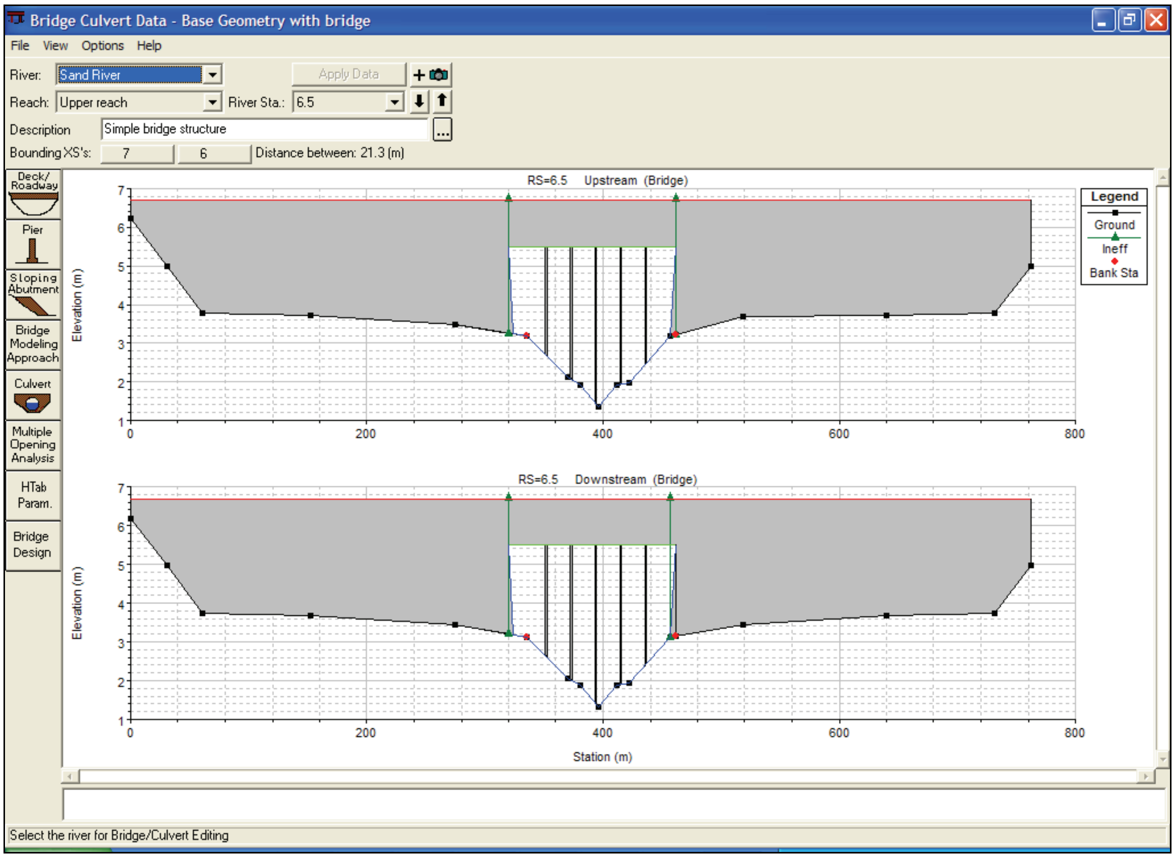

Figure 8.19: Upstream and downstream bridge cross-section from the HEC-RAS analysis

Cross-section details

The cross-section details are given in the **Table 8.16** below. These details can be obtained from analysis of the surveyed cross-section information, using software such as HEC-RAS, or computing the variables by hand, as illustrated in Example 8.1, Section 8.8.1.

Slope of the river

The general slope of the river is 0,2 %.

Determine

(i) Short-term general scour
(ii) Contraction scour
(iii) Local scour at the piers and abutments
(iv) Total scour
(v) Verify the scour depth with the method based on the principle of applied stream power (Section 8.4.4).

Solution Example 8.2

For this analysis the design flood discharge of 850 m^3/s will be used (**Chapter 3** describes procedures to determine the design flood).

The contracted width at the bridge will be 126,61 m. This will result in a discharge per unit width of 850/126,61 = 6,713 m^3/s.

The normal flow depth (fixed bed), y_n, of the river can be determined by the assumption of the energy slope to be equal to the bed slope 0,002 m/m and by using the Chezy or Manning equations.

It is estimated that the bed roughness under flood conditions will be 0,002m, equal to D_{50} the representative sediment material size.

Table 8.16: Details of cross-section 6.5 (Obtained from HEC-RAS analysis)

Section	y_n (m)	Area (m²)	Wetted perimeter (m)	Flow rate (m³/s)
Left bank	2,98	209,97	288,17	168,75
Main channel		258,73	126,67	542,60
Right bank		185,40	283,48	138,65
Total		**654,10**	**698,32**	**850,00**

R = 0,937 m and $\overline{v}$ = 1,299 m/s

Top flow width = 698,2 m for the calculated normal flow depth of 2,98 m.

It is assumed that the bed material consists of deep alluvial sand with no cohesion.

(i) *Short-term general scour*

The **regime equations** are applied to establish equilibrium conditions at the design flow:

From Equation 8.9:

$$B = 14Q^{0,5}D_{50}^{0,25}F_s^{-0,5}$$

With F_s = 0,1 from **Table 8.2** for silty clay loam, the width B can be calculated.

B = 273 m, which is wider than the proposed bridge of 126,61 m.

Use Equation 8.8 to determine the mean flow depth at the equilibrium width:

$$y = 0,38q^{0,67}D_{50}^{-0,17}$$

q = 850/273 = 3,114 m³/s.m

Mean depth y = 2,34 m. The maximum depth, Y_{max} = 1,25y = 2,92 m.

The maximum live-bed depth, Y_{max}, is slightly less than the fixed bed depth, y_n of 2,98 m, which reflects that no general short term scour will occur.

(ii) *Contraction scour*

It has been indicated that contraction scour can be determined by applying either the **regime equations** (Equations 8.8 and 8.10) or the **contraction equations** (Equations 8.14 and 8.16 with 8.15).

First apply the ***regime equation*** on the reduced width of 126,61 m. In this case q = 850/126,61 = 6,714 m³/s.m leading to a mean flow depth, y of 3,915 m. From **Table 8.4**, y_{max} can be determined as follows: y_{max} = 1,25 x 3,915 = 4,893 m. This reflects a scour depth, d_s = 4,893 – 2,98 = 1,913 m

Secondly the ***contraction equations*** are used to determine the scour depth after it has been established if the flow will be sediment laden or not.

V_* can be determined using Equation 8.5. $V_* = \sqrt{gDS} = \sqrt{9{,}81(2{,}98)(0{,}002)}$ = 0,242 m/s and the term, V_*D_{50}/ν = 483 >> 13, thus in turbulent flow region (see **Figure 8.10**).

The critical shear velocity V_{*c} = 0,12 x V_{ss} (Equation 8.4). The settling velocity, V_{ss} can be obtained from **Figure 8.11** for the representative particle, D_{50}, and the relative density of 2,65, it follows:

V_{ss} = 0,24 m/s, and

V_{*c} = 0,029 m/s

From Equation 8.13: $V_c = 5{,}75 V_{*c} \log \frac{12R}{k_s} = 5{,}75\left((0{,}029)\log\left(12\frac{(0{,}937)}{(0{,}002)}\right)\right)$

$V_c = 0{,}625$ m/s

The average approach flow velocity of 1,299 m/s > 'critical' velocity of 0,625 m/s, thus sediment will be entrained and Equation 8.14 together with **Figure 8.12** can be used to estimate contraction scour.

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7}\left(\frac{B_1}{B_2}\right)^{2/3}\left(\frac{n_2}{n_1}\right)^{1/3}$$

$$\frac{y_2}{y_1} = \left(\frac{850}{542{,}6}\right)^{6/7} = 1{,}469$$ (widths and n-values are equal for these sections)

y_2 = (1,469)(2,98) = 4,378 m

Assuming a level bed with a total depth of 4,378 m, the velocity in the contraction can be determined:

$$\bar{v}_2 = \frac{850}{(4{,}378)(126{,}61 - 5(1{,}5))} = 1{,}63 \text{ m/s}$$

Note that in this case the downstream area is 521,5 m², calculated as follows (4,378 x (126,61 – 5(1,5))). This is larger than the upstream main channel area of 258,73 m² (**Table 8.16**), and thus the **flow is expanding**. Equation 8.15 is used to determine the contraction scour depth.

$d_s = (y_2 - y_1) + (1 + K)\left(\frac{\bar{v}_2^2 - \bar{v}_1^2}{2g}\right)$ and with K = 1 for a sudden transition (also see in Equation 4.28)

$$d_s = (4{,}378 - 2{,}98) + (1 + 1{,}0)\left(\frac{1{,}63^2 - 1{,}23^2}{2(9{,}81)}\right)$$

d_s = 1,50 m

This scour depth (1,50 m) is less than that obtained with the regime theory (1,913 m).

(iii) Local scour at piers and abutments

For piers in alluvial cohesionless materials:

Use Equations 8.17 and 8.18 to compute local scour in two different ways. Obtain the factors needed for Equation 8.17 from **Tables 8.4** and **8.5**. Obtain the factors needed for Equation 8.18 from **Tables 8.4, 8.5** and **8.6** and Equations 8.20, 8.21, 8.22 and 8.23. Compare answers obtained from Equations 8.17 and 8.18 and **select a conservative answer using good engineering judgement**.

From Equation 8.17, with depth y_0 in the bridge section as determined from regime Equation 8.8:

$$d_s = 1{,}8 y_0^{0{,}75} b^{0{,}25} - y_0$$
$$d_s = 1{,}8\,(3{,}915^{0{,}75})(1{,}5^{0{,}25}) - 3{,}915$$
$$d_s = 1{,}629 \text{ m}$$

Note that the scour level is (3,915 + 1,629) = **5,544 m below the design flood level**.

Alternatively Equation 8.18, for the longest piers close to the minimum river invert could be used to calculate the local scour depth at the pier.

$$\frac{y_s}{b} = 2{,}0\,K_1 K_2 K_3 K_4 \left(\frac{y_1}{b}\right)^{0{,}35} Fr_1^{0{,}43}$$

with

b	=	1,5 m
y_1	=	2,98 m (normal flow depth upstream of the bridge, **Table 8.16**)
Fr_1	=	0,468 based on main channel data directly upstream of the pier
K_1	=	1,0 for round nose
K_2	=	1,0 for zero skew angle
K_3	=	1,1 for small dunes
K_4	=	1,0 for uniform sediment (no armouring), then

$$\frac{y_s}{b} = (2{,}0)(1{,}0)(1{,}0)(1{,}1)(1{,}0)\left(\frac{2{,}98}{1{,}5}\right)^{0{,}35}(0{,}468)^{0{,}43} = 2{,}02$$

y_s = 3,03 m

Note that the scour level is (2,98 + 3,03) = **6,01 m below the design flood level associated with the normal flow depth and a fixed bed level**.

For abutments in alluvial cohesionless materials:

Apply factors in **Table 8.9** to the general short-term average scour depth obtained from Equation 8.8.

From **Table 8.9** the factor for flow that impinges at right angles on bank = 2,25; hence the scour at the abutments can be determined as shown below:

$$d_s\,(\text{abutments}) = (2{,}25)(3{,}915 - 2{,}98) = 2{,}10 \text{ m}$$

(iv) Total scour

Total scour is the sum of the long and short-term general scour, contraction scour and local scour. **Table 8.17** reflect a summary of all the calculated scour depths.

Table 8.17: Summary of the calculated scour depths for Worked Example 8.2

Scour type		**Calculated scour depth, d_s (m)**
Short term general scour		No scour
Contraction scour	Regime equation	1,913
	Contraction equation	1,398
Local scour	Piers	3,03
	Abutments	2,10
Total expected scour	**Piers**	**4,943**
	Abutments	**4,013**

Review of the contraction or short-term scour using different analyses procedures

The potential general scour at the bridge has been determined in (i) using Equations 8.8 and 8.9. A more correct approach is to estimate contraction scour separately for the main channel and over banks, as is done in HEC-RAS, where the over bank flows may then reflect clear water scour. The scour depth in the channel calculated in the approach used in HEC-RAS is less than 3,1 m. With the regime theory reflecting a scour depth of 1,91 m and the HEC-RAS result of 3,1 m, the contraction scour calculation of 1,5 m using Equation 8.14 is too conservative and hence discarded.

Based on the summary in **Table 8.17** the total scour can be determined as follows.

Total scour at piers in main channel

Total scour level at piers, below design flood level (not accounting for backwater)
= 1,913 + 3,03 = **4,943 m**

Total scour at abutments

With the right abutment on the edge of the main channel, the scour would be the sum of main channel contraction scour plus abutment scour, thus:

Total scour at abutments = 1,913+ 2,1 = **4,013 m** below design flood level

The scour for the left bank abutment would be less.

(v) Verify the scour depth with the method based on the principle of applied stream power (Section 8.5.4)

For total scour at piers in alluvial rivers, check the answer against values obtained by means of Equation 8.24 that is based on the principle of applied stream power.

Equation 8.24 reflected the following relationship:

$$\frac{C(Y_t)(v_{ss}k_s)^{1/3}}{q\sqrt{g}} = F$$

With F = 0,8; k_s = 0,002 m ; v_{ss} = 0,24 m/s; q = 542,6/126,61 = 4,29 m³/s.m and C calculated from the Chezy relationship for total section, C = 67,5; it follows that:

Y_t = 2,03 m below design flood level, which is **substantially less than obtained above**.

The designer will experience these conflicting results, which reflect amongst other the complexities involved in the mathematical description of scour estimates and the shortcoming in the assumption that the material is cohesionless.

Considering the risk of failure of the structure due to scour and the potential consequences, these cases require further evaluation by experienced persons.

The problem is also evaluated using HEC-RAS, and the data files are contained as Example 2 on the supporting CD.

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CHAPTER 9 – SUB-SURFACE DRAINAGE

A Rooseboom, GM James, J Maastrecht and DW Stipp

9.1 INTRODUCTION

9.1.1 General

Water in the structural layers of a pavement is the chief cause of road failures. This water causes the mechanical properties of the material to weaken, because excessively high pore pressures develop under traffic conditions. This weakening of mechanical properties and the washing out of underlying foundation materials is generally known as "pumping" (**Figure 9.1**). The purpose of sub-surface drainage is to remove from the road structure, as rapidly as possible, any infiltrated water occurring in damaging quantities.

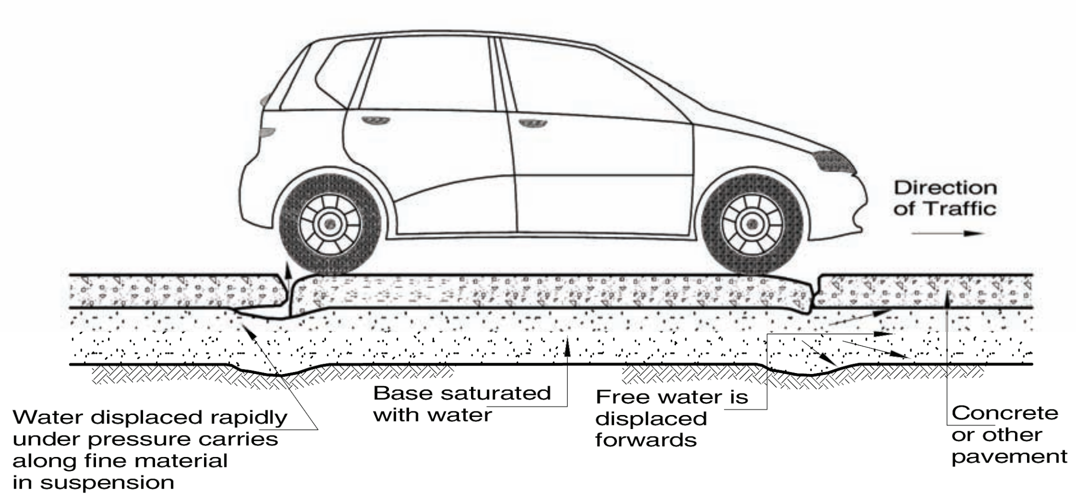

Figure 9.1: Schematic visualisation of the pumping phenomenon under pavements

Table 9.1: Road Map for sub-surface drainage

ROAD MAP 9			
Typical problems		**Input information**	**Worked examples (Example number)**
Topic	**Paragraph**		
Planning of sub-surface drainage	9.2	Road horizontal and vertical alignment, surface drainage and maintenance requirements	Design of a herringbone drainage system (9.1)
Spacing of intercept drains	9.3.2.2	Material size distribution, soil permeability	
Design of underground pipes	9.3.2.4.6	Gradient of pipes, pipe perforations and layout of the drainage system	
Design of layer drainage	9.5	Grade of the subbase	
Outlets for sub-surface drainage	9.6	Topography and erosion potential at the outlets	

Important sources of underground water are:

- the natural water table;
- irrigation, canals and dams; and
- rainfall infiltration.

Such water may enter the pavement layers by infiltration, either sideways or through the road surface.

The following types of underground drainage systems to remove water are generally used:

- interception drainage
- herringbone drainage
- layer drainage
- structural drainage

Photograph 9.1 reflects the typical problems associated with poor sub-surface drainage design, and **Photograph 9.2** shows the installation of sub-surface drainage to improve the situation.

Photograph 9.1: Typical conditions associated with poor sub-surface drainage design

Photograph 9.2: Installation of sub-surface drainage to improve the drainage

9.1.2 Sub-surface interception drainage (Trench drains)

This type of drain intercepts mainly groundwater moving horizontally, and lowers the water table. It is used in the following cases:

- at the toe of a cut where the side slope is stable;
- at the toe of a fill to prevent or limit groundwater inflow into the road prism; and
- across the road on the downhill side of a cut.

Trench drains could consist of conventional drains incorporating granular material. This material could act as a natural filter; as a permeable collector, or as a geocomposite incorporating a synthetic collector.

Permeable collector drains consist of single sized aggregates between 9,5 and 25 mm encapsulated within a geosynthetic filter material.

Geocomposite drains consist of a geospacer synthetic collector encapsulated within a geosynthetic filter material. The main advantages of geocomposite drains are:

- smaller dimensions (narrow trenches);
- speed and ease of construction; and
- pre-assembly before installation into the trench.

The total cost of geocomposite drains are usually lower than conventional drains of the same depth.

9.1.3 Herringbone sub-surface drainage

This type of sub-surface drainage reduces a generally high groundwater table to an acceptable level. It is used in the following cases:

- In areas, cuts or fills where there is a high groundwater table that is undesirable; or
- To stabilise areas where a high water table interferes with construction.

The herringbone sub-surface drainage may be installed below the road prism or within the pavement layers.

9.1.4 Layer sub-surface drainage

This type of sub-surface drainage removes, in particular, water that seeps in through the road surface. In many cases an attempt is made to make the road surface impermeable which, from the point of view of drainage, makes the construction of an open-graded layer unnecessary.

Blanket drains could consist of either natural filters or permeable collectors. A geosynthetic filter material is placed at the interface between the permeable collector (sand, gravel or aggregate) and the subgrade, and functions as a filter and a separator to prevent two dissimilar materials from intermixing. These drains may consist of a thick geosynthetic material where flow may occur both perpendicular to and transversely in the plane of the geotextile.

9.1 5 Structural sub-surface drainage

These drains are used for structures, such as bridge abutments and retaining walls. Geocomposite drains or thick non-woven geotextiles tied into a collector pipe are especially useful in these applications because of the speed and ease of installation, pre-assembly benefits and narrower dimensions.

9.2 PLANNING OF SUB-SURFACE DRAINAGE

In the planning of an underground drainage system, the first step is to undertake a groundwater investigation in the area. There are two basic groundwater conditions for which drainage systems are essential:

- Groundwater with a hydraulic gradient smaller than the slope of the ground. Typical warning signs here are wet patches and visible outflows on the side of a cut. Interception drains are installed in such cases.
- A groundwater table close to the surface. Signs of this are collapsing wet spots in flat areas. This condition arises where water infiltrates from higher areas or through a "leaking" surface layer. Herringbone or layer drainage is installed in such cases.

Observations should be carried out on:

- geology – fractured, fissured or jointed rock, impermeable dykes or alternating layers of permeable and impermeable material
- vegetation – variations in colour and vigorous growth, or hydrophilic vegetation
- topography – shape of the land, depressions, valley lines, catchments, etc
- road surface failures – crocodile cracking, pumping, rutting, tension cracks.

A design method should now be chosen to calculate the capacity of the drainage system. (Note: Storm water may not be discharged into a sub-surface drain.)

In the case of ground water intercepted from cuts, it is generally not practical to carry out a sophisticated calculation, probably because too wide a variety of material is found in a cut, and seasonal changes also have a strong influence on the groundwater discharge. In practice, the capacity of an intercept drain may be determined by *in-situ* flow measurement (during the wet season after the channel has been excavated!) or, if excessive quantities of groundwater have not been observed, or there is no groundwater in the dry season, nominal drainage may be provided. In calculating the capacity of herringbone drainage, **Table 9.3** may be used to determine the depth/spacing of the system. Layer drainage is designed with the aid of formulae such as those described in Section 9.5. During the planning and design of a drainage system the following points should be borne in mind:

- A thorough investigation of the sub-surface drainage requirements is essential. Such an investigation should be performed during the rainy season, because the dry season may present a completely false picture of groundwater conditions.
- A sub-surface drainage system should ensure that the road structure is without free water for about 1 metre below the road surface most of the time.
- It is, therefore, advisable to provide sub-surface drainage in all cuts and underneath some low embankments.
- Intercept drains should be provided at the end of a cut.
- Water should be able to escape from an embankment that drains towards a solid bridge abutment.
- The capacity of the drainage network should be adequate.
- The drainage network should require minimum maintenance.
- The drainage network should not become blocked.
- Repairs and alterations are extremely expensive.
- The drainage network should be economical.

9.3 DESIGN OF AN INTERCEPT DRAIN

9.3.1 General

Intercept drains may be open (above ground) or covered (below ground). Open channels normally occupy greater space and require more maintenance than underground drains.

9.3.2 Sub-surface intercept drains

9.3.2.1 Introduction

Sub-surface drains require less maintenance than surface drains, but blockages are difficult to rectify. Rodding eyes may be provided for long lengths of drains in problem soil areas.

The theory discussed below regarding sub-surface drains also applies in general to open intercept channels. The use of intercept drains is normally recommended where investigations have shown a definite inflow of water, and they are then placed across the direction of flow. The length of the flow-path should also be limited to approximately:

- 200 to 300 m for conventional subsoil drains (9.1); and
- 50 to 150 m for geocomposite drains (9.2).

Unusual formations or groundwater conditions may be responsible for a high water table in certain localities, and likewise sudden changes in the topography may cause a high water table in some areas. These conditions are difficult to describe, and no fixed rules can therefore be laid down.

Sub-surface intercept drains consist of a mechanism for interception, such as a filter and a discharge system, which is usually a system of pipes. **Figure 9.2** shows typical, sub-surface intercept drains.

9.3.2.2 Design capacity of intercept drains

The capacity of an intercept drain may be calculated by using the Darcy methods, but normally not all the accepted assumptions would apply in practice. If visible ground water is present, a guide ditch may be used to measure the yield accurately. When little or no ground water is present, nominal sizes are used (refer to paragraph 9.3.3.2).

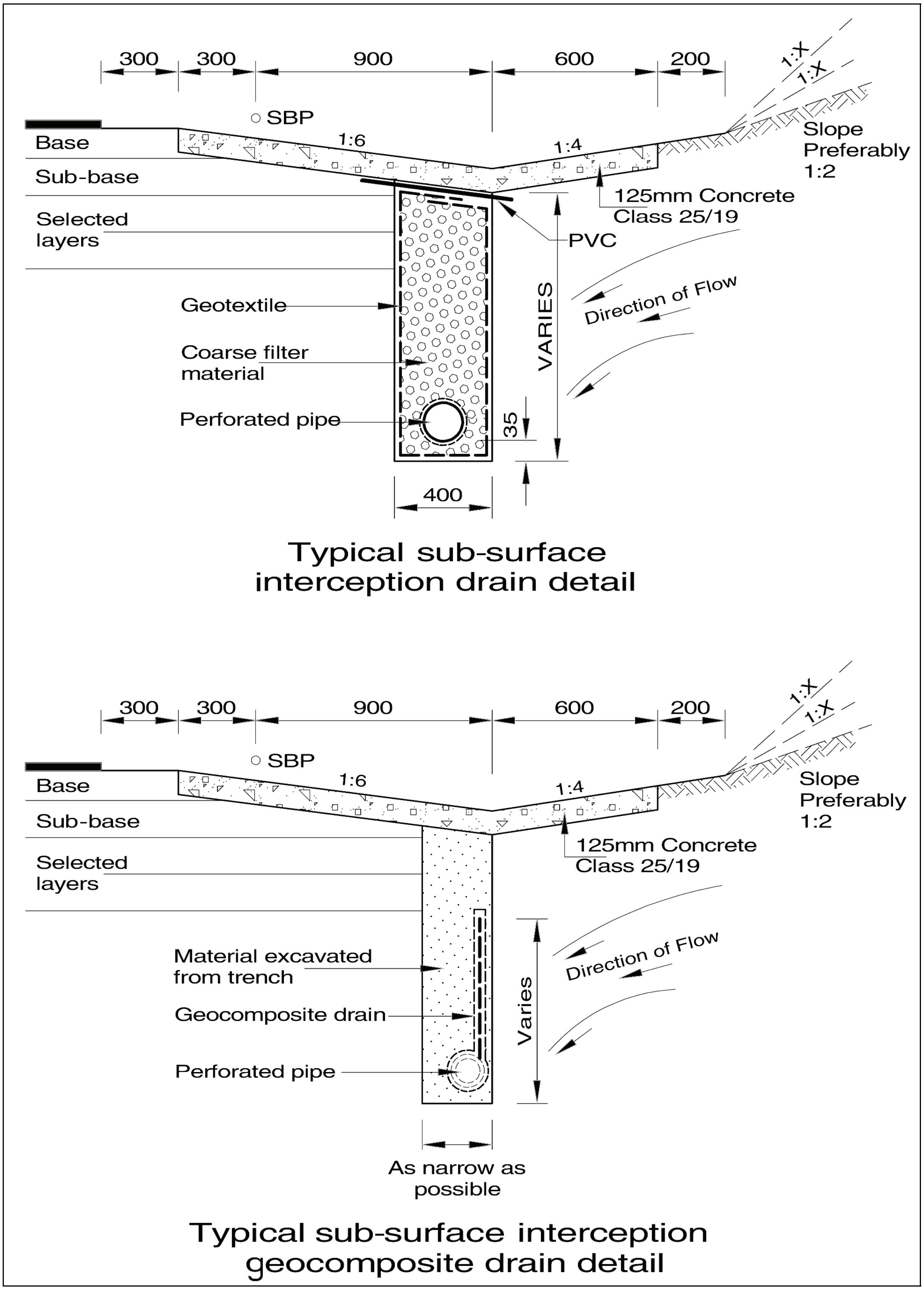

Figure 9.2: Typical sub-surface drain details

9.3.2.3 Designing a granular filter layer

Groundwater moving along a hydraulic gradient is intercepted by a filter layer (coarser material). Water usually flows from the fine to the coarse material, and the following should be ensured:

- Fine material should not wash into the coarse material and so clog it.
- The grading of each layer should be uniform to ensure good quality.
- Adequate through-flow should take place.
- The minimum layer thickness for construction purposes should be 100 mm.

Table 9.2 reflects the requirements that should be satisfied when a filter material is selected.

Table 9.2: Criteria for filter material

Requirement	Criteria *	Objective
1	$\frac{D_{15}\text{ filter material}}{D_{85}\text{ protected soil}} \leq 5$	To ensure retention of fines in the soil by the filter
2	$\frac{D_{60}\text{ filter material}}{D_{10}\text{ filter material}} < 20$	To ensure that the filter is well graded and stable
3	$\frac{D_{15}\text{ filter material}}{D_{15}\text{ protected soil}} \geq 5$	To ensure a permeability differential between the filter and the soil
4	$\frac{D_{50}\text{ filter material}}{D_{50}\text{ protected soil}} < 25$	To ensure uniformity

Note: D_x = the sieve size through which x % of the material passes (mm)

The D_{15} value of a filter material may not be < 0,1 mm, and if D_{15} > 0,4 mm it would not be necessary to meet requirement 1, provided that 2 applies.

9.3.2.4 Designing a geotextile filter

Designing with geotextiles for filtration is essentially the same as designing graded granular filters. A geotextile is similar to a soil in that it contains voids (pores) and particles (filaments and fibres). However, because of the shape and arrangement of the filaments and the compressibility of the structure of geotextiles, the geometric relationships between filaments and voids are more complex than in soils. In geotextiles, pore size is measured directly, rather than using particle size as an estimate of pore size, as is done with soils. Since pore size may be directly measured, relatively simple relationships could be developed between the pore sizes and particle sizes of the soil to be retained. The following three simple filtration concepts are used in the design process:

- If the size of the largest pore in the geotextile filter is smaller than the larger particles of soil, the soil will be retained by the filter. As with graded granular filters, the larger particles of soil will form a filter bridge over the hole which, in turn, filters smaller particles of soil which then retain the soil and prevent “piping”. **Photograph 9.3** indicates the failure of a road surface due to “piping” that occurred around the installed culvert.
- If the smaller openings in the geotextile are large enough to allow smaller particles of soil to pass through the filter, then the geotextile will not blind, block or clog.
- A large number of openings should be present in the geotextile so that proper flow could be maintained even if some of the openings later become blinded, blocked or clogged.

Photograph 9.3: Failure of a road surface due to "piping", occurring around the culvert

These simple concepts, which are synonymous with soil filter design criteria, can also be summarised as follows:

- The geotextile should retain the soil (retention criteria), while
- allowing water to pass (permeability criterion), throughout
- the life of the structure (clogging resistance criterion).

To perform effectively, the geotextile should also survive the installation process (survivability criterion).

These design criteria [9.3] provide an excellent prediction of filter performance, particularly for granular soils (< 50% passing a 0,075 mm sieve).

There are six criteria that should be considered for the design and selection of geotextile filters. These criteria are:

- Retention criteria
- Dynamic (multidirectional) flow conditions
- Permeability/permittivity criteria
- Clogging resistance
- Survivability criteria, and
- Durability criteria.

These criteria are briefly discussed below.

9.3.2.4.1 Retention criteria

Steady state (unidirectional) flow conditions

AOS or $O_{95(geotextile)} \leq B\ D_{85(soil)}$

where:

AOS	=	apparent opening size (mm)
O_{95}	=	opening size in geotextile for which 95% of openings are smaller (mm)
AOS	≈	O_{95}
B	=	a coefficient (dimensionless)
D_{85}	=	soil particle size for which 85% of openings are smaller (mm)

The coefficient B ranges from 0,5 to 2,0, and is a function of the type of soil to be filtered, its density, the uniformity coefficient C_u (if the soil is granular), the type of geotextile (woven or non-woven) and the flow conditions.

For sands, gravelly sands, silty sands and clayey sands (with less than 50% passing the 0,075 mm sieve per the United Soil Classification System), B is a function of the uniformity coefficient, C_u. Thus for:

$C_u \leq 2$ or ≥ 8 : $B = 1$
$2 \leq C_u \leq 4$: $B = 0{,}5\ C_u$
$4 < C_u < 8$: $B = 8 / C_u$
where:
C_u = D_{60} / D_{10} (coefficient of uniformity)

Sandy soils that are gap-graded or well graded tend to bridge across the openings; thus the larger pores may actually be up to twice as large ($B \leq 2$) as the larger soil particles because, quite simply, two particles cannot pass through the same hole at the same time. Consequently use of the criterion B = 1 would be quite conservative for retention, and such a criterion has been used by, for example, the US Corps of Engineers.

If the protected soil contains any fines, only the portion passing the 4,75 mm sieve for selecting the geotextile is used (i.e. scalp off the +4,75 mm material).

For silts and clays (with more than 50% passing the 0,075mm sieve) B is a function of the type of geotextile:

for wovens	$B = 1$; $O_{95} \leq D_{85}$
for non-wovens	$B = 1{,}8$; $O_{95} \leq 1{,}8\ D_{85}$
for both	AOS or $O_{95} \geq 0{,}30$ mm

Due to their random pore characteristics and, in some types, their felt-like nature, non-wovens would generally retain finer particles than a woven geotextile of the same AOS. Therefore, the use of B = 1 would be even more conservative for non-wovens.

In the absence of detailed design, the AASHTO M288 Standard Specification for Geotextiles (2000) [9.4] provides the following recommended maximum AOS values in relation to percent of *in-situ* soil passing the 0,075 mm sieve:

- 0,43 mm for less than 15% passing;
- 0,25 mm for 15 to 50% passing; and
- 0,22 mm for more than 50% passing.

In the case of cohesive soils, with a plasticity index greater than 7, the maximum AOS size is 0,30 mm. These default AOS values are based upon the predominant particle sizes of the *in-situ* soil. The engineer may require performance testing based on engineering design for drainage systems in problematic soil environments. Site-specific testing should be performed, especially if one or more of the following problematic soil environments are encountered:

- unstable or highly erodible soils such as non-cohesive silts;
- gap-graded soils;
- alternating sand/silt laminated soils;
- dispersive soils; and/or
- gold or coal-ash tailings.

9.3.2.4.2 Dynamic (multidirectional) flow conditions

Soil particles may move behind the geotextile, if it is not properly weighted down and intimate contact with the soil to be protected, or if dynamic, cyclic or pulsating loading conditions produce high, localised hydraulic gradients. Thus the use of B = 1 is not conservative because the bridging network would not develop and the geotextile would be required to retain even finer particles. When retention is the primary criterion, B, should be reduced to 0,5 or:

$$O_{50} \leq 0{,}5\ D_{85}$$

Dynamic flow conditions may occur in pavement drainage applications, e.g. a road fill across the upper reaches of a dam. For reversing inflow-outflow or high-gradient situations it is best to maintain sufficient weight or load on the filter to prevent particle movement.

Stable versus unstable soils

The above retention criteria assume that the soil to be filtered is internally stable – it will not pipe internally. If unstable soil conditions are encountered, performance tests should be conducted to select suitable geotextiles. Broadly graded (C_u > 20) soils with concave upward grain size distributions, gap-graded soils and dispersive soils tend to be internally unstable.

9.3.2.4.3 Permeability/permittivity criteria

Permeability requirements

As long as the permeability of the geotextile ($k_{geotextile}$) is greater than the permeability of the soil (k_{soil}) the flow of water will not be impeded at the soil/geotextile interface. To provide for an additional level of conservatism apply the following formula to all critical applications and severe conditions:

$$k_{geotextile} > 10\ k_{soil}$$

By applying a factor of safety of 10 to k_{soil}, allowance is made for the potential reduction in permeability of the geotextile through the pores becoming blinded, blocked or clogged by migrating soil particles.

Permittivity requirements (optional)

$\psi \geq 0,5\ sec^{-1}$ for < 15% passing 0,075 mm
$\psi \geq 0,2\ sec^{-1}$ for 15 to 50% passing 0,075 mm
$\psi \geq 0,1\ sec^{-1}$ for > 50% passing 0,075 mm

In these equations:
k = Darcy coefficient of permeability (m/s)
ψ = geotextile permittivity which is equal to $k_{geotextile} / t_{geotextile}$ (s^{-1}) and is a function of the hydraulic head where $t_{geotextile}$ is the thickness of the geotextile

In very critical applications, to minimise the risk of the geotextile filter not being sufficiently permeable, performance testing should be conducted. For actual flow capacity the permeability criteria for non-critical applications are conservative, since an equal quantity of flow through a relatively thin geotextile takes significantly less time than through a thick, granular filter. Even so, some pores in the geotextile may become blocked, blinded or clogged with time.

The AASHTO M288 Standard Specification for Geotextiles (2000) presents recommended minimum permittivity values in relation to percent of situ soil passing the 0,075 mm sieve. The values are the same as presented in the equations above. The default permittivity values are based upon the predominant particle size of the *in-situ* soil. Again, the engineer may require performance testing based on engineering design for drainage systems in problematic soil environments.

The required flow rate, q, through the system should also be determined, and the geotextile and drainage aggregate selected to provide adequate capacity. As indicated above, flow capacities should not be a problem for most applications, provided the geotextile permeability is greater than the soil permeability. However, in certain situations, such as where geotextiles are used to span joints in rigid structures and where they are used as pipe wraps, portions of the geotextile may be blocked. For these applications the following criteria should be used together with the permeability criteria:

$$q_{required} = q_{geotextile} (A_g / A_t)$$

where:
A_g = geotextile area available for flow (m²);
A_t = total geotextile area (m²)

9.3.2.4.4 Clogging resistance

Less critical / less severe conditions

For less critical / less severe conditions:

$$O_{95\ (geotextile)} \geq 3\ D_{15\ (soil)}$$

This applies to soils with $C_u > 3$. For $C_u \leq 3$ select a geotextile with the maximum AOS value.

In situations where clogging is a possibility (e.g., gap-graded or silty soils) the additional criteria in **Table 9.3** should be considered:

Table 9.3: Additional criteria to be considered when there is a potential for clogging

Material	Additional criteria
Non-woven material	Porosity (void ratio) of the geotextile, n ≥ 30%
Woven monofilament and slit film wovens	Percent open area, POA ≥ 4%

Most common non-wovens have porosities much greater than 70%, while most woven monofilaments easily meet the criteria. **Tightly woven slit films do not, and are thus not recommended for sub-surface drainage applications**.

Performance type filtration tests provide another option for consideration, especially by inexperienced users.

Critical / severe conditions

For critical/severe conditions, select geotextiles that meet the retention and permeability criteria. Then conduct a performance filtration test using samples of on-site soils and hydraulic conditions. One type of performance filtration test is the Gradient ratio test that is well described in literature [9.2].

Although several empirical methods have been proposed to evaluate geotextile filtration characteristics (i.e. the clogging potential) the most realistic approach for all applications is to perform a laboratory test that simulates or models field conditions. This test utilises a rigid-wall soil permeameter (**Figure 9.3**) with piezometer taps that allow for simultaneous measurement of the head losses in the soil and the head loss across the soil/geotextile interface. The ratio of the head loss across this interface (nominally 25 mm) to the head loss across 50 mm of soil is termed the gradient ratio. As fine soil particles adjacent to the geotextile become trapped inside or blind the surface, the gradient ratio will increase. A gradient ratio of less than 3 is recommended.

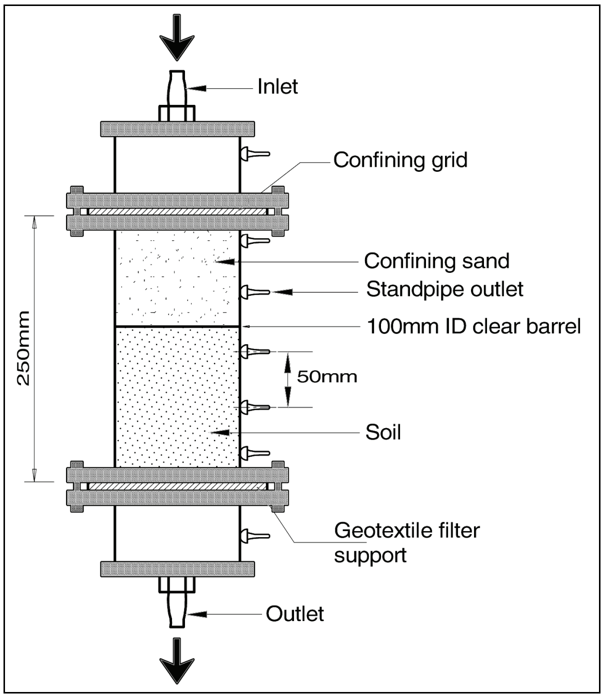

Figure 9.3: Typical gradient ratio permeameter

These filtration tests are performance tests. They should be conducted on samples of project site soil by the specifying agency or its representative.

9.3.2.4.5 Survivability criteria

To ensure that the geotextile will survive the construction process, certain geotextile strength and endurance properties are required for filtration and drainage applications. The geotextile fabric specification falls outside the scope of this manual and as such is not dealt with. Suffice to say it is the duty of the engineer, after doing site-specific evaluation, testing and design, to ensure that an appropriate geotextile is selected for use in the actual construction.

9.3.2.4.6 Durability criteria

Geotextile endurance relates to its longevity. Geotextiles have been shown to be basically inert materials for most environments and applications. However, certain applications may expose the geotextile to chemical or biological activity that could drastically influence its filtration properties or durability. For example, in drains granular filters and geotextiles could become chemically clogged by iron or carbonate precipitates, and biologically clogged by algae, mosses, etc. Biological clogging is a potential problem when filters and drains are periodically inundated and then exposed to air. Excessive chemical and biological clogging may significantly influence filter and drain performance. These conditions are present, for example, in landfills.

Biological clogging potential could be determined with the ASTM D5322-92 Standard Test Method for biological clogging of geotextile or soil/geotextile filters (1991). If biological clogging is a concern, a higher porosity geotextile may be used, and/or the drain design and operation may include an inspection and maintenance programme to flush the drainage system.

9.3.3 Design of underground pipes

9.3.3.1 Slope of pipes

The slope of a pipe is largely influenced by the road alignment or the final contours.

Changes in slope, especially a reduction in slope, should be avoided as far as possible. The following minimum slopes and flow velocities are recommended:

- A minimum slope of 0,5%, with an absolute minimum of 0,2% for laterals and 0,25% for mains.
- The design flow velocities should be between 0,5 and 3,0 m/s.
- The absolute minimum of the half-full velocity should not be less than 0,6 m/s [(9.3)].

For slopes steeper than 2%, pipes with couplings should be used to prevent erosion of the material around the joints caused by excessive eddying at the joints.

9.3.3.2 Pipe diameter and outlet spacing

Under normal to dry conditions the pipe diameters and maximum pipe lengths (without outlets) given below are used if no design data are available:

- Diameter 100 mm, lengths up to 200 m;
- Diameter 150 mm, lengths up to 300 m.

In freeway drainage systems 100 mm diameter pipes are mainly used, but where high inflows of water are encountered, 150 mm diameter pipes are more suitable.

In cases where the maximum discharge is known, the pipe diameter is calculated according to **Figure 9.4**. A sub-surface drainage pipe should preferably not flow more than 70% full under maximum discharge conditions, so that excessive pressure conditions do not develop.

The capacity of different pipes may also be calculated by means of the Manning formula, and typical design values of Manning-n for underground drainage pipes are given in **Table 9.4**.

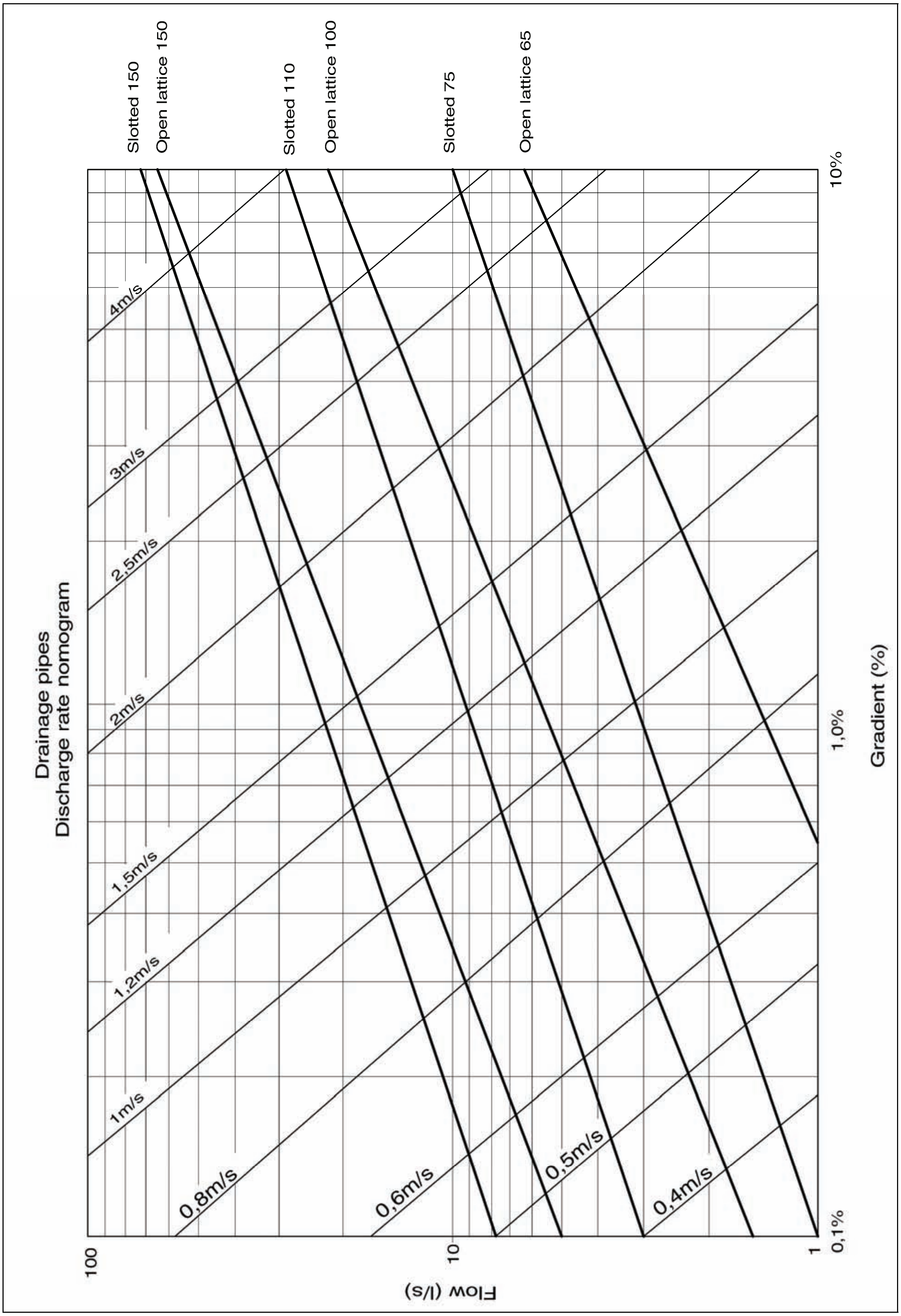

Figure 9.4: Nomogram for the discharge rate of drainage pipes

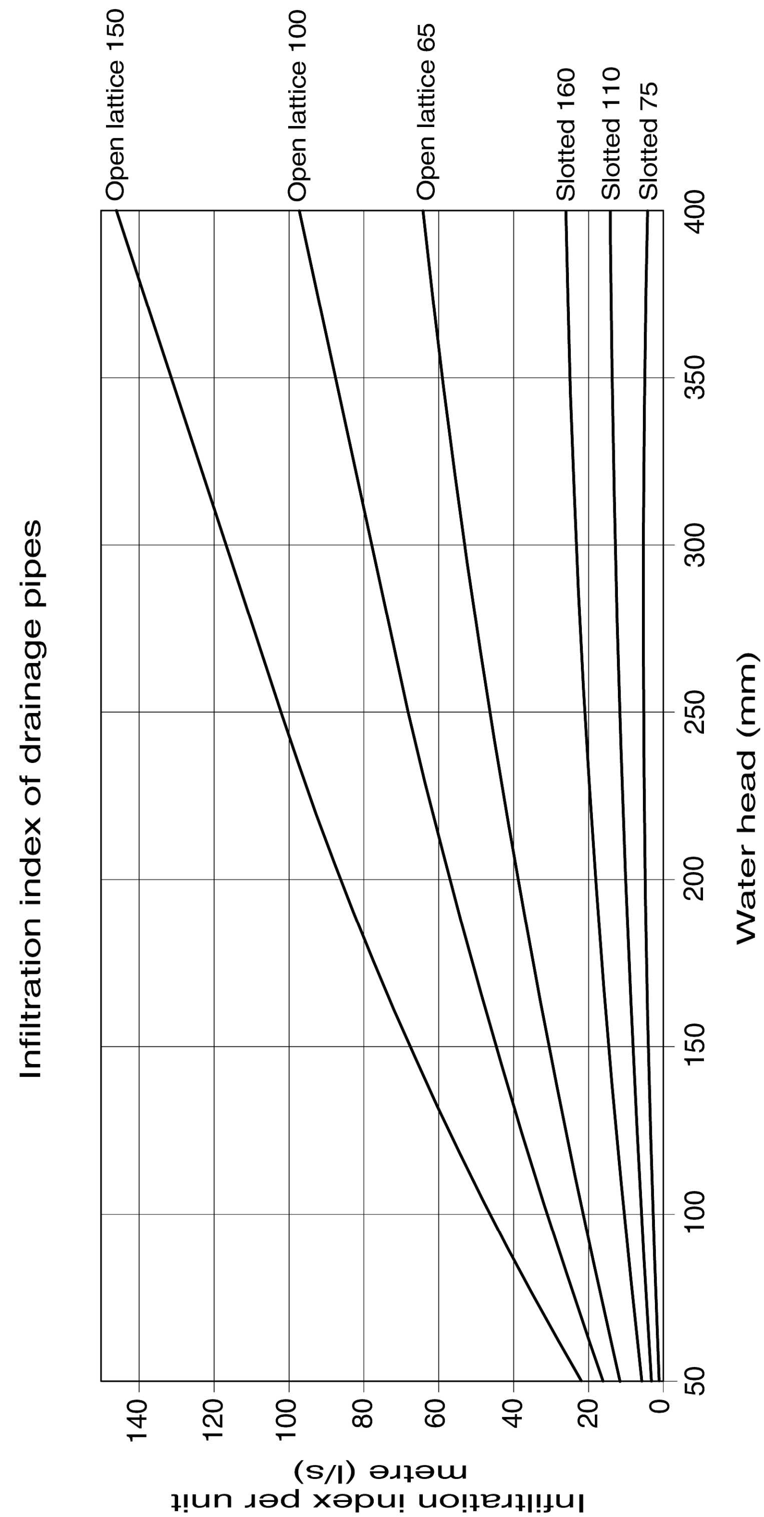

Figure 9.5: Infiltration index of drainage pipes

Table 9.4: Roughness parameters of drain pipes

Pipe type	n-value (s/m$^{1/3}$)
PVC	0,010
Corrugated polyethylene	0,025
Smooth-bore polyethylene (9.6)	0,010
Open-lattice HDPE	0,010

9.3.3.3 Pipe perforations

The pipe perforations should satisfy the following requirements to prevent penetration of filter material:

- Diameter of openings, slots or perforations < D_{85} filter;
- 1,2 x width of openings, slots or perforations < D_{85} filter.

Simply put, the smaller sized particles of the filter material should be larger than the largest size openings, slots or perforations of the pipe.

9.3.3.4 Pipe material

Pipe materials generally used are HDPE and PVC. It is important that the pipes be installed in accordance with manufacturers' instructions.

9.3.3.5 Pipe bedding

Figure 9.6 is a recommended way of laying of sub-surface drainage pipes.

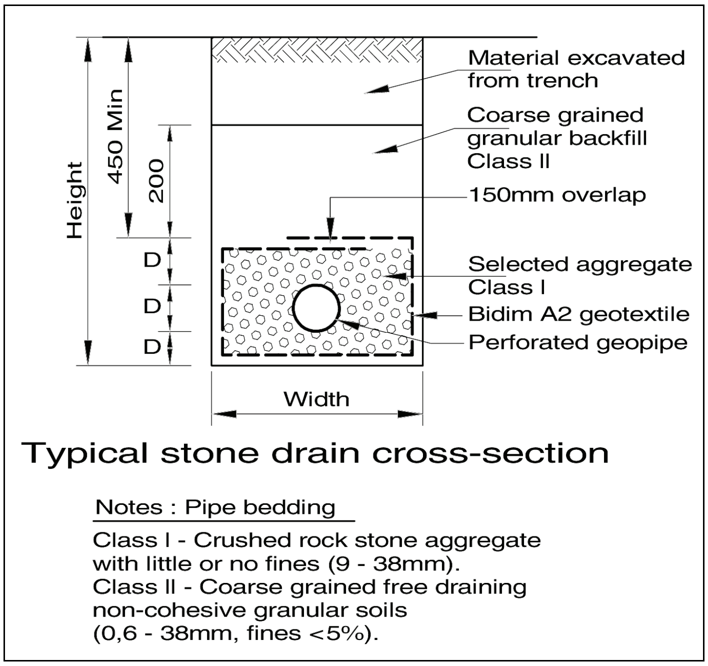

Figure 9.6: Typical drainage pipe bedding details

The purpose of the bedding is to provide a working surface and to ensure that the drainage pipe does not come into contact with rocks or clay material. Drainage pipes are flexible, and consequently their load-bearing capacity depends less on the inherent strength of the pipe, and more on the quality of the backfill surrounding the pipes.

Where the *in-situ* material is reasonable, the bedding thickness should be 100 mm. In softer, saturated conditions the bedding layer should be between 150 and 200 mm thick (SABS 1200).

Proper compaction of the bedding will ensure that the grade (fall) of the drainage pipe is maintained. The side-fill and the compaction thereof are most important and require special care during placement in the haunch areas beneath the drainage pipe. If selected backfill material is not placed beneath the drainage pipe, voids will result and, when loaded, the drainage pipe will deform into these voids. The degree of compaction for bedding should be 90% Mod AASHTO.

Granular soils are normally graded. It gives uniform support to the drainage pipe, and readily accommodates the haunch support material, which is compacted adjacent to and above it, after the drainage pipe is placed.

Two classes of crushed aggregate or selected granular fill recommended for drainage pipe installation are described in **Figure 9.6**.

The drainage pipe being flexible, will deform slightly under vertical load (vertically downwards and horizontally outwards). This outward movement develops horizontal earth pressures against the wall of a pipe and enables the pipe to carry loads far in excess of its crushing strength.

If any trench is left open for too long it may cause local instability or sloughing failures, thereby negatively influencing the drainage pipe placement and/or backfilling. In the event of trench wall collapse or dynamic lateral loading (induced by compaction or other equipment) occurring before proper bedding has been provided, the drainage pipe could well deform (vertically upwards and horizontally outwards). These situations depend largely on the soil type and depth of trench. In some cases trench support systems may be required. Local conditions and safety considerations will govern each situation.

Correct selection and placement of bedding and padding will ensure that deformation is kept within acceptable limits. Also the backfilling should be done in a reasonably symmetric manner so the drainage pipe is not pushed laterally out of alignment. This should preferably be done in 100 mm layers.

Indiscriminate dumping of backfill soil adjacent to and above the drainage pipe will severely jeopardise the integrity of the drainage pipe. Rocks bearing against the drainage pipe will also result in high stress concentrations. **Such practices should not be permitted under any circumstances**.

A layer of selected granular material is placed above the drainage pipe again to protect the pipe from rocks, etc. and to provide support and load transfer to the top of the drainage pipe and support for the subsequent backfilling operations. The thickness of this layer should be at least equal to the diameter of the drainage pipe. It is recommended that hand compaction is carried out up to 300 mm above the drainage pipe; thereafter mechanical compaction may be used. A minimum overall compacted cover layer of 450 mm is recommended (**Figure 9.6**).

Bad compaction of this latter material would not adversely affect the performance of the drainage pipe, but it could lead to consolidation of these layers, settlement of the surface and some deformation of the drainage pipe if not adequately bedded.

Following the placement of this backfill material, natural soil or the soil being used to construct the structure or facility is brought up to final grade in lifts/layers as per the plans and specifications. This aspect follows standard earthworks procedures.

9.4 DESIGN OF A HERRINGBONE SYSTEM

9.4.1 Spacing of laterals

For the general case the spacing of laterals may be determined with the aid of **Table 9.5**, which is self-explanatory.

Table 9.5: Recommended depth and spacing of laterals for different types of soil

Soil classification	Soil composition			Spacing of laterals			
	% sand	% silt	% clay	0,9 m deep	1,2 m deep	1,5 m deep	1,8 m deep
Clean sand	80-100	0-20	0-20	33-45	45-60		
Sandy loam	50-80	0-50		15-30	30-45		
Loam	30-50	30-50		9-18	12-25	15-30	18-36
Clayey loam	20-50	20-50	20-30	6-12	8-15	9-18	12-25
Sandy clay	50-70	0-20	30-50	4-9	6-12	8-15	9-18
Silt/clay	0-20	50-70		3-8	4-9	6-12	8-16
Clay	0-50	0-50	30-100	4 max	6 max	8 max	12 max

In more complex cases, and if feasible, the spacing of the laterals may be designed according to the methods for steady or unsteady groundwater flow.

9.4.2 Design of laterals

The diameter of laterals may be read off **Figure 9.4**. However, if the drainage rate is known, the area that could be drained by a pipe of a certain diameter may be calculated by means of Equation 9.1 below.

$$A = \frac{\left(26{,}92 \times 10^{6}\right) d^{\frac{8}{3}}\ S_o^{\frac{1}{2}}}{nq}\left(0{,}7\right) \qquad \text{(for 70 \% of full capacity)} \qquad \ldots(9.1)$$

where:

- S = spacing (m)
- A = surface area (m^2) = $S\,(L + 0{,}5\,S)$
- d = diameter of pipe (m)
- L = length of the pipe (m)
- q = drainage rate (mm/day)
- n = Manning's n ($s/m^{1/3}$)
- S_o = slope of the pipe (m/m)

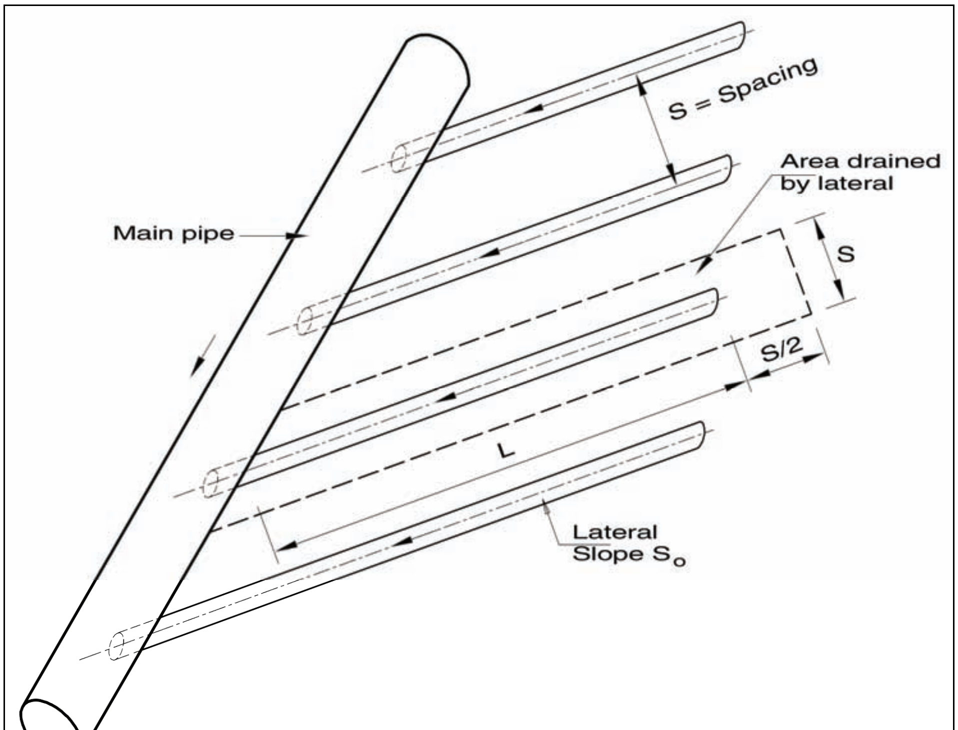

Figure 9.7: General view of a herringbone drainage system

9.4.3 Main drainage pipes

The capacity of the main pipe is not the sum of the capacities of the lateral pipes, unless non-porous pipes are used. The surface drained by a pipe of a certain diameter is calculated in the same way as for lateral pipes.

9.5 DESIGN OF LAYER DRAINAGE

9.5.1 General

Once again it should be stressed that the provision of layer drainage should be regarded not as the basic point of departure, but as a solution to the problem of a permeable road surface.
Layer drainage consists of:

- a permeable, open-graded subbase or base below the surface layer;
- filter layer or impermeable layer;
- a collector drain and perforated pipe;
- outlet pipes; and
- markers.

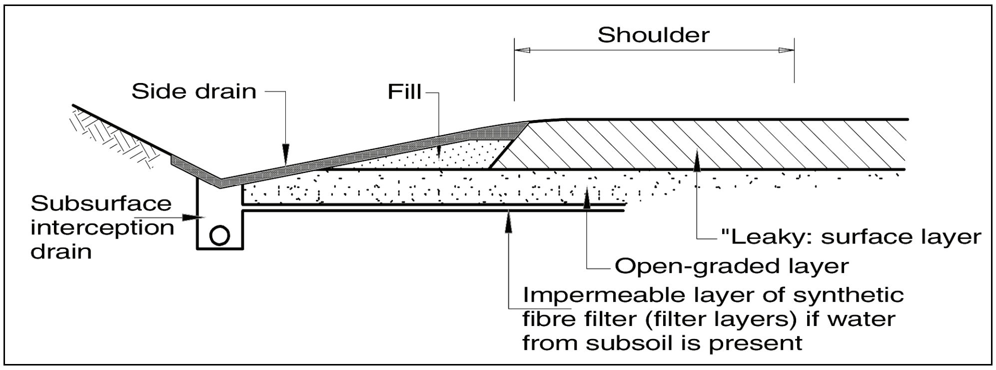

Figure 9.8: Typical section through a layer-drainage

It is essential to investigate the possibility of providing layer drainage whenever:

- The average annual rainfall > 300 mm a year.

- The horizontal drainage coefficient (HDC) < 100 times the design infiltration rate (DIR). The DIR = 0,33 to 0,67 times the one-hour yearly rainfall (T=1) intensity and the HDC = the layer thickness x the permeability of the drainage layer.

- More than 250/10 t wheel loads per day would use the road during its design life.

9.5.2 Open-graded base and subbase

Open-graded layers should be constructed directly underneath the surface layer and should extend approximately 300 mm and 600 mm past the inner and outer shoulders respectively. Over these parts and underneath the layer, the open-graded layer should be protected from penetration by fine material. The minimum thickness of this drainage layer is 75 mm, and with such thin layers strict care should be taken that undesirable material is not mixed in.

Open-graded layers should be designed to convey water. The inflow consisting of water that has infiltrated through the surface and water from other sources such as channels, cuts, and so on, should be estimated.

The effectiveness of a drainage layer is determined by the permeability of the layer. Adequate permeability may be ensured by setting the following requirements:

- Minimum sieve size : 4,75 mm (no 4 sieve)
- Maximum sieve size : 19 – 37,5 mm
- Permeability (laboratory): > 3 000 m/day
- Design permeability: 0,33 to 0,5 of laboratory permeability.

9.5.3 Filter layer and collector drain

The filter layer and collector drain are similar to those described in Section 9.3.

9.5.4 Design formulae

For the purposes of this manual only formulae from Cedergren[9.5] are given. The symbols that are used are defined below.

q	=	discharge per metre width (m^3/s.m)
B	=	width of collector drain (m)
L	=	length of paving (1 m wide) subject to infiltration (m)
S	=	cross-slope of a drainage layer (m/m)
W	=	width of the drainage layer (m)
g	=	longitudinal slope of the road (m/m)
k_b	=	permeability of an open-graded layer (m/day)
k_t	=	permeability of the channel backfill (m/day)
n_b	=	porosity of an open-graded layer $= \frac{V_e}{V} = \frac{\text{Volume of pores}}{\text{Total volume of material}}$
P	=	1h duration/1 year return period rainfall intensity (mm/h)
I	=	design infiltration rate (mm/h)
t_b	=	thickness of drainage layer (mm)
t_b'	=	effective thickness of drain layer (mm)
T	=	drainage period for layer (h)
i	=	hydraulic gradient, (m/m)
k_s	=	permeability of material (m/day)
t	=	depth of flow in material (mm)

The following formulae are used:

- The minimum thickness of a drainage layer:

  t_b = Effective thickness + say 25 mm

$$t_b = \frac{24\,IW}{k_b S} + 25 \qquad \ldots(9.2)$$

- The time required for the drainage layer to drain:

$$T = \frac{24\ Wn_b}{k_b\left[S + \frac{t_b'}{2000W\left(1+(g/s)^2\right)}\right]} \qquad \ldots(9.3)$$

- The minimum width of the collector drain

$$B = \frac{0,48\,IW}{k_t} \qquad \text{…(9.4)}$$

- The inflow into a drainage layer, according to Darcy:

$$q = k_s\,i\,t \qquad \text{…(9.5)}$$

The symbols are defined above in paragraph 9.5.4.

9.6 OUTLETS, MARKERS AND INSPECTION REQUIREMENTS

9.6.1 Outlets

Outlets should be provided at regular intervals so that the system will always flow freely, and to facilitate maintenance. The following general principles should also be observed:

- Outlets should be provided at the lowest points on the pipes.
- Water should flow freely from an outlet, since a pump outlet is generally not economical.
- The outlet should be about 200 mm above the invert of the outlet channel to prevent silting up, and should be protected from penetration by plants.
- The area directly surrounding the outlet should be pitched to limit damming resulting from the growth of vegetation at the outlet.
- The final 6 m of the drainage pipe should be back-filled and compacted with an impermeable material, or should be provided with a cut-off or a storm water manhole outlet to prevent flow and erosion alongside the pipe.
- Wire netting or gauze should be provided at the outlet to prevent rats and mice from entering.
- When an excavation passes from a wet area to a dry area, a cut-off should be provided to force water from the wet area up to the pipe so that the dry area remains dry.

Figure 9.9 shows a typical outlet.

9.6.2 Markers

The start and end of sub-surface drainage pipes should be clearly marked or visible to ensure proper maintenance. They should be clearly numbered on galvanised plates that are conspicuously mounted on the road fencing. The construction of a conspicuous outlet is also acceptable as a proper marker.

9.6.3 Inspection requirements

Sound construction and maintenance practices ensure the effective working of a drainage system.

At the upper end of a filter drain an inspection eye could be installed. The inspection eye may be built into a storm water inlet or may rise to the surface at an angle of 45° by means of a rising limb. Inspection eyes are not normally used to clean seepage drains, but are very useful during construction for testing through-flow.

9.6.4 Manholes

Manholes should be placed not more than 150 m apart, if there are no other outlets.

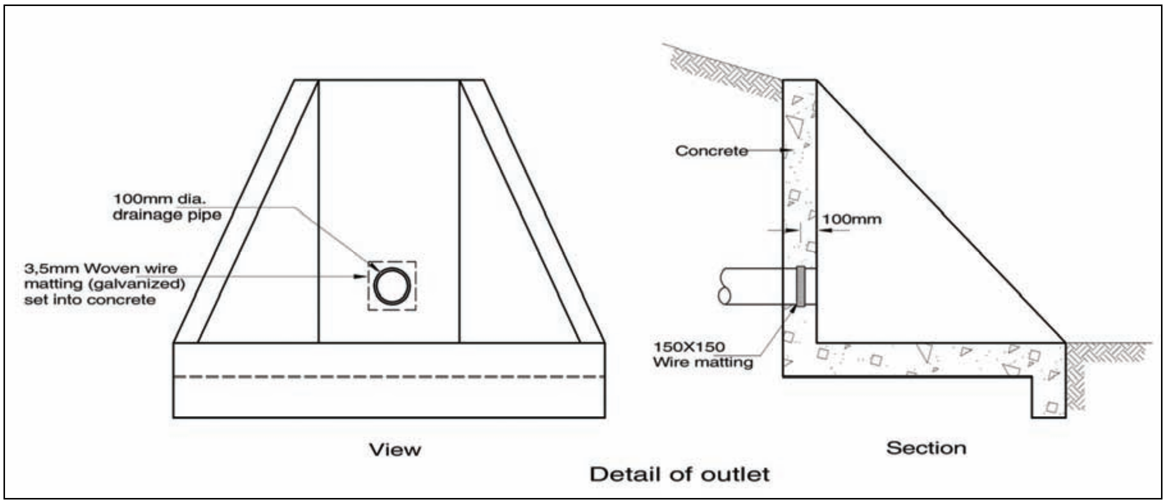

Figure 9.9: Detail of the outlet for sub-surface drainage collector

9.7 WORKED EXAMPLE: HERRINGBONE DRAINAGE DESIGN

9.7.1 Example 9.1 - Herringbone drainage system

Problem description Example 9.1:

Calculate the maximum infiltration rate (mm/day), which may be discharged via the depicted sub-surface herringbone system to a main drainage pipe. The diameter of the central pipe is 150 mm and its slope 1:500. The diameter of the laterals is 100 mm and their slopes 1: 100. The Manning n-value for the pipes is 0,011s/m$^{1/3}$. **Figure 9.10** reflects the layout.

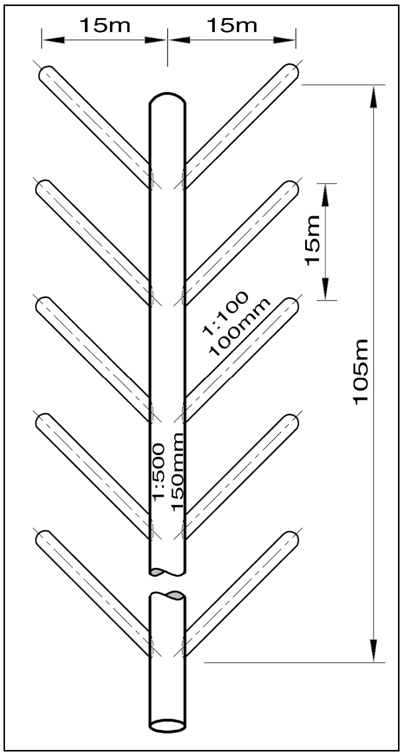

Figure 9.10: Layout of the herringbone drainage system

Solution Example 9.1

For each lateral, flowing 70% full:

$$q = \frac{\left(26{,}92 \text{ x } 10^6\right) d^{8/3} \left(S_0^{\ 1/2}\right)(0{,}7)}{n\ A}$$

$$A = S\ (L + \frac{S}{2})$$ Refer to **Figure 9.7**

$$L = \sqrt{15^2 + 15^2} = 21{,}21\text{m}$$

$$S = \frac{L}{2} = 10{,}61\text{m}$$ (Valid in this example since laterals are placed at 45º and dimensions are equal 15 m x 15 m)

$$\therefore \quad A = 10{,}61\left((21{,}21) + \frac{10{,}61}{2}\right)$$

$$= 281{,}3\ \text{m}^2$$

$$\therefore \quad q = \frac{\left(26{,}92 \text{ x } 10^6\right)\left(0{,}1^{8/3}\right)\left(0{,}01^{1/2}\right)(0{,}7)}{(0{,}011)(281{,}3)}$$

$$\therefore \quad q = 1312 \text{ mm/day}$$

Flow rate, Q, for 14 laterals:

$$Q = (1{,}312)(281{,}3)(14)$$

$$Q = 5167 \text{ m}^3\text{/day}$$

Hence $Q \approx 0{,}06 \text{ m}^3\text{/s}$

Capacity of central pipe: (Manning Formula)

$$Q = \frac{\frac{\pi}{4}\left(0{,}15^2\right)\left(\frac{0{,}15}{4}\right)^{2/3}\left(\frac{1}{500}\right)^{1/2}}{0{,}011}$$

$Q = 0{,}00805 \text{ m}^3\text{/s}$

But this << $0{,}06 \text{ m}^3\text{/s}$!

$Q \approx 695 \text{ m}^3\text{/day}$

$$\therefore \ q_{max} = \frac{695}{(281{,}3)(14)}$$

q_{max} = 177 mm/day (<< 1312 mm/day)

9.8 REFERENCES

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9.3 Holtz, R.D., Christopher, B.R. and Berg, R.R. (1998). *Geosynthetic Design and Construction Guidelines*. FHWA.

9.4 AASHTO. (2000). *Standard Specification for Highway Applications*. Designation M288-00

9.5 Cedergren, H.R. *et al*. (1974). D*evelopment of Guidelines for the Design of Sub-surface Drainage Systems for Highway Structural Sections*. Report No FHWA-RD-72-30.

CHAPTER 10 – WEB-BASED LINKS AND SUPPORTING SOFTWARE

M van Dijk and SJ van Vuuren

10.1 INTRODUCTION

The purpose of this chapter is to provide relevant web-based links and details of the supporting material. A supporting CD at the back of the Drainage Manual contains some additional material, supporting documentation and computer software. These items are briefly described in the following paragraphs.

10.2 WEB-BASED LINKS

Due to the evolving nature of the internet only a single link is provided in this document which will be kept unchanged. On this website a list of useful links will be provided relevant to this manual and issues relating to drainage.

http://www.sinotechcc.co.za

10.3 SUPPORTING CD

The CD contains additional material and supporting documentation on a number of the aspects that can be viewed in pdf format or downloaded. The visual material will aid in describing some drainage structures, potential problems and should emphasize the importance of these structures ensuring an optimal road infrastructure. **Figure 10.1** reflects the supporting CD "Intro Screen" which could be used to link to the supporting documentation, software and links. **Figures 10.2** to **10.4** indicate the contents covered in each of the sub-sections.

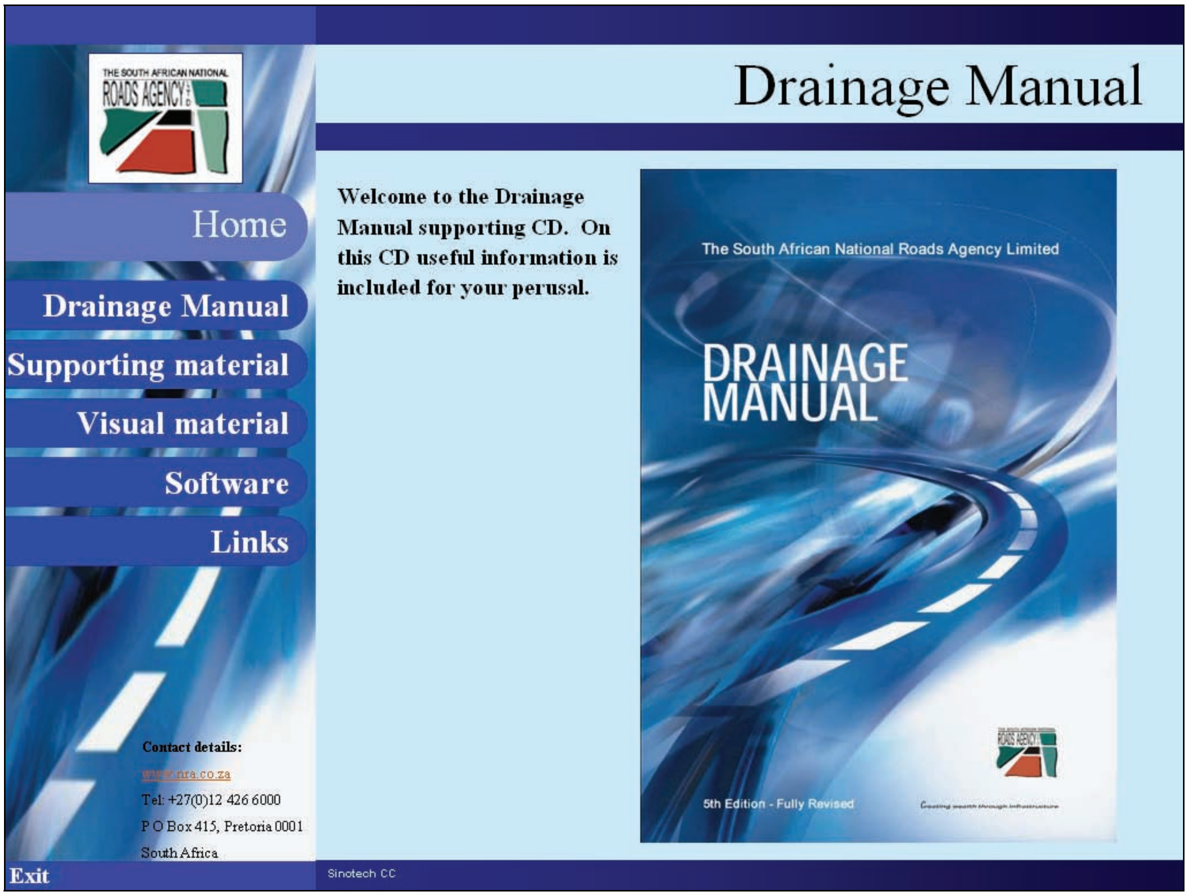

Figure 10.1: Supporting CD

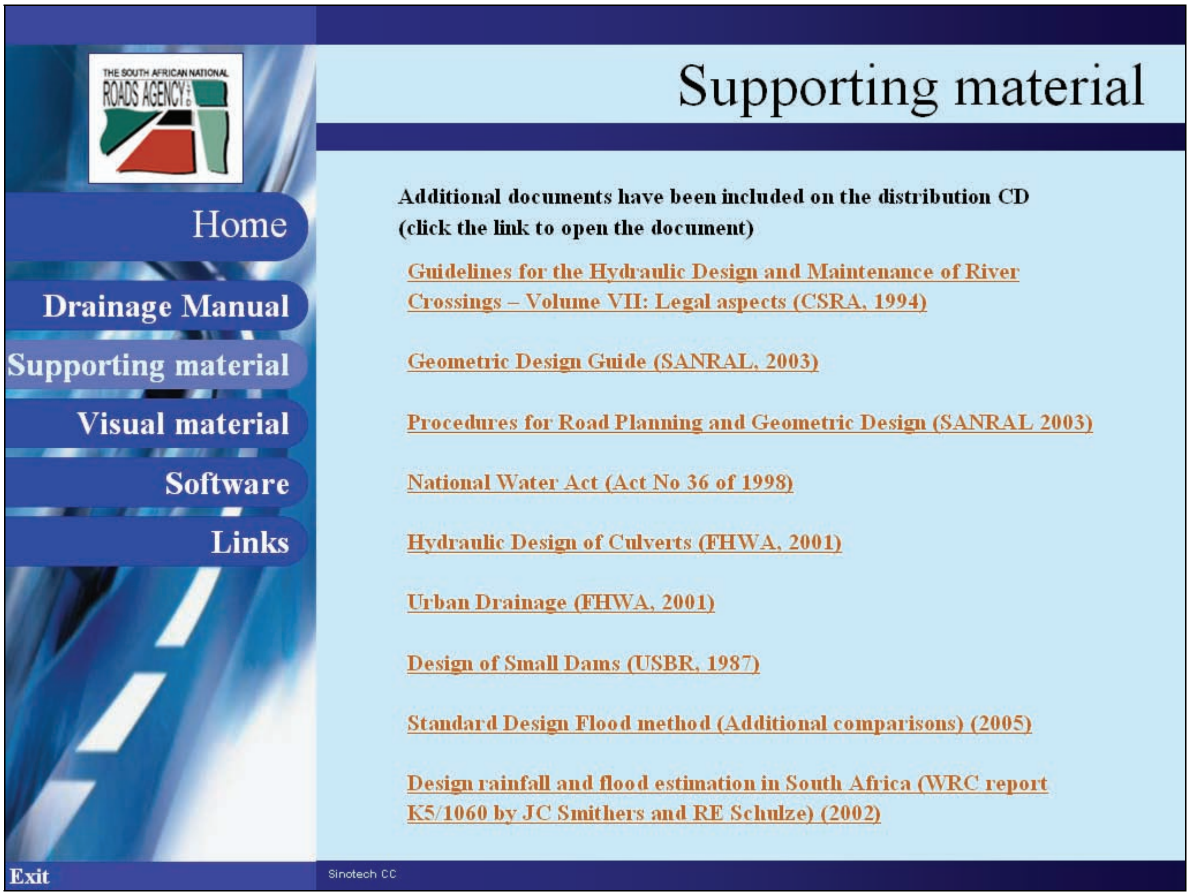

Figure 10.2: Supporting material

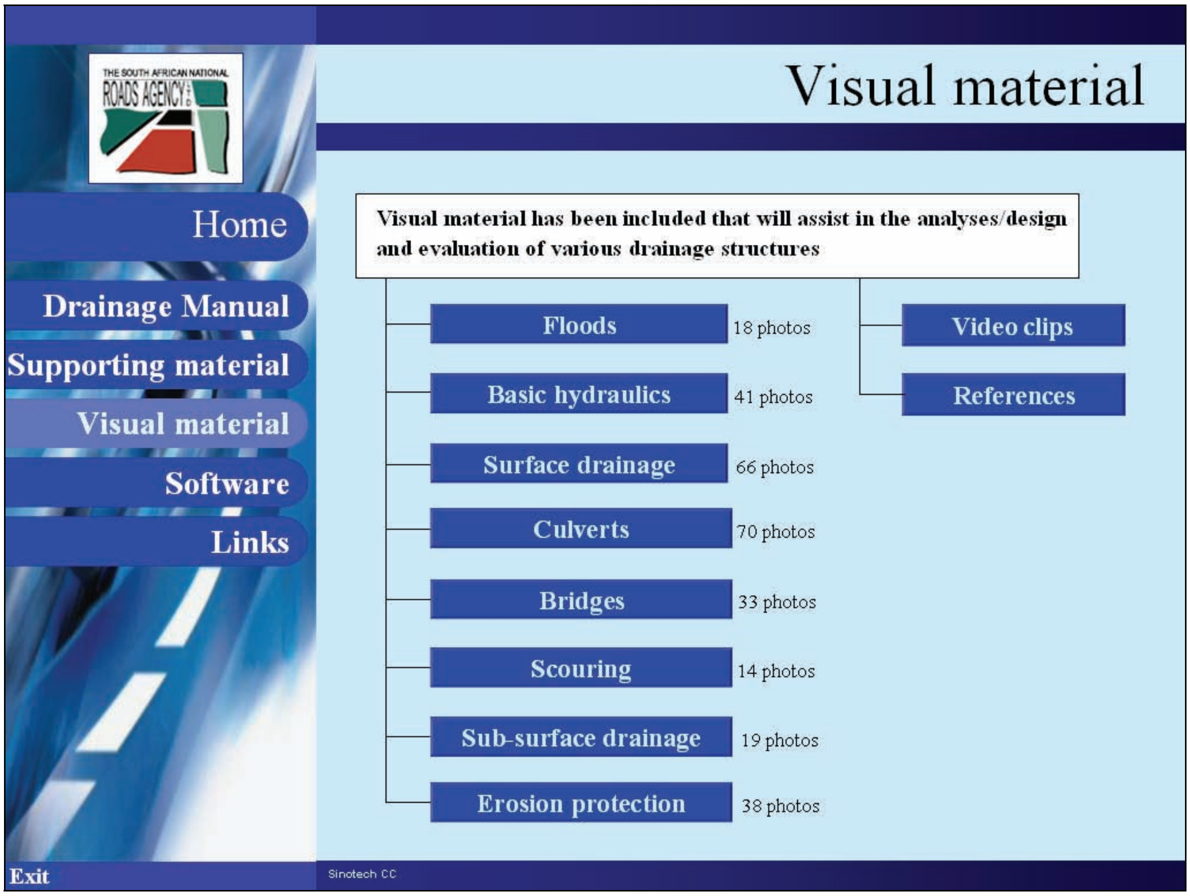

Figure 10.3: Visual material

Figure 10.4: Software

Software programs were developed in parallel to the *Drainage Manual* to assist engineers/designers in the analyses and design of road drainage structures. Examples based on the material of each chapter have been included to demonstrate the use of the supporting software.

The supporting software includes the following:

- Utility Programs for Drainage
- HEC-RAS
- EPA Storm Water Management Model
- BridgeLCC

These software programs are briefly discussed in the following paragraphs.

10.3.1 Utility programs for Drainage (UPD)

Utility Programs for Drainage is distributed as a demo-version on the supporting CD. The CD contains all the files and libraries necessary to install and review the program, UPD. The Utility Programs for Drainage is a suite of programs that will assist in the design and analyses of drainage structures. It consists of the following components:

- Economic calculations
- Flood calculations (Deterministic, empirical and statistical methods)
- Water surface profiles
- Basic hydraulics
- Surface drainage
- Culverts and bridges

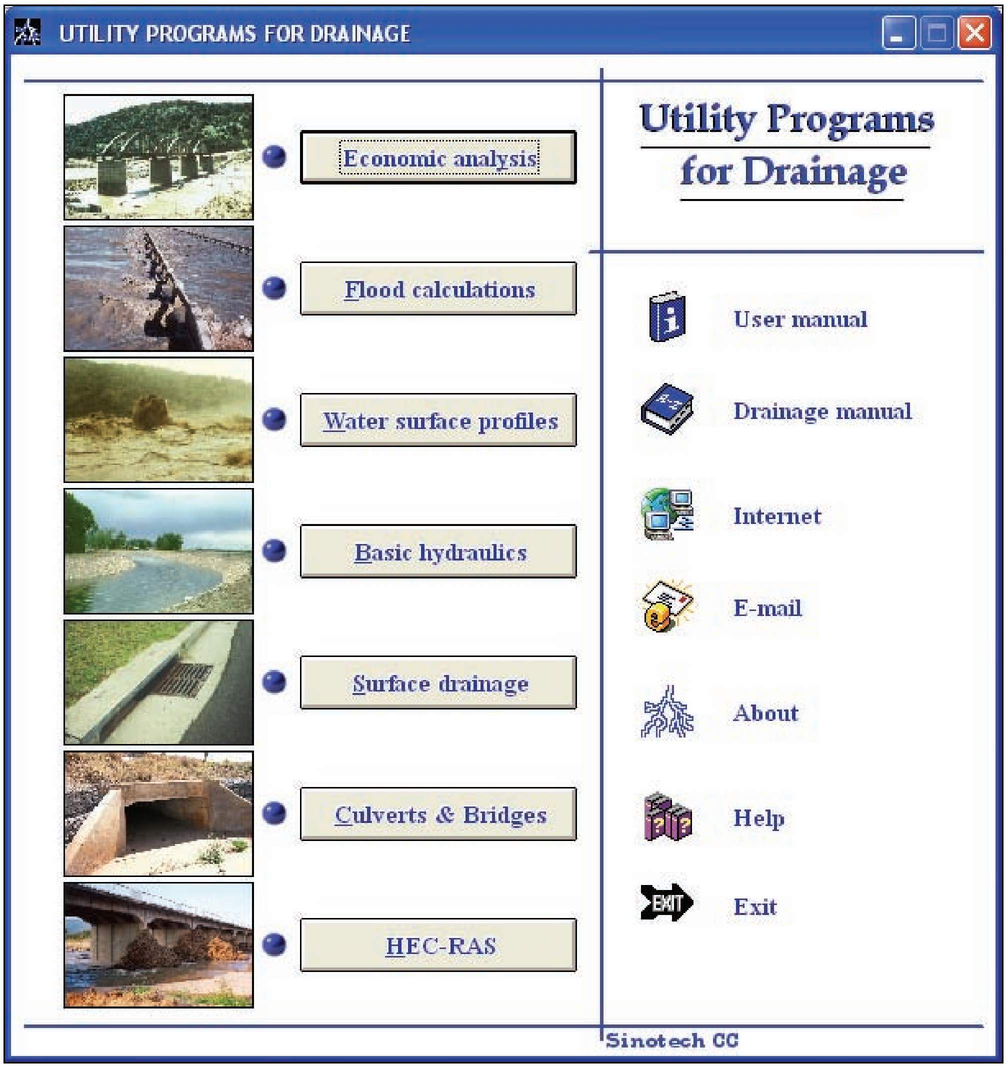

Figure 10.5: Utility Programs for Drainage

The software's user manual is distributed with the installation CD in Adobe Acrobat format and the software has a user-friendly help file.

10.3.2 HEC-RAS

HEC-RAS is public domain software i.e. freeware and is also included on the supporting CD. HEC-RAS was developed by the Hydrological Engineering Centre and allows for the performing of one-dimensional steady flow and unsteady flow calculations, as well as sediment transport calculations. The HEC-RAS User's Manual, Hydraulic Reference Manual and Application Guide are all included in the package.

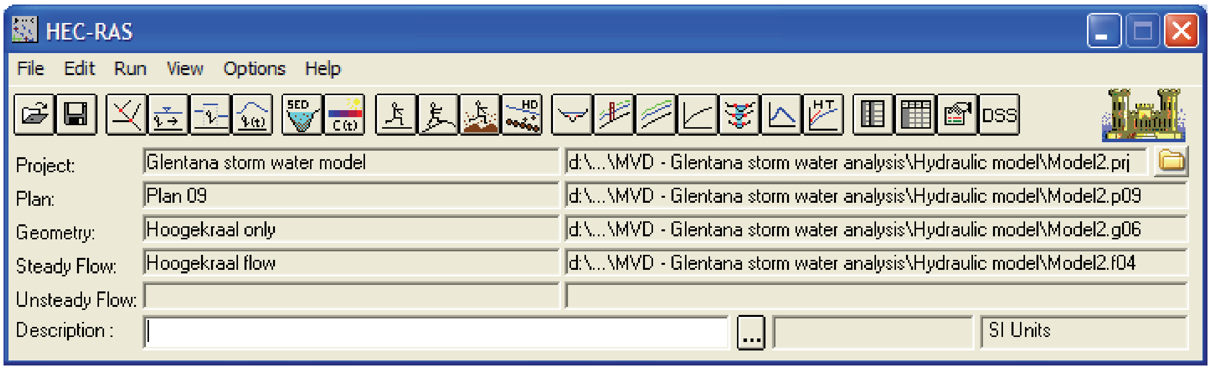

Figure 10.6: HEC-RAS software

10.3.3 EPA Storm Water Management Model (SWMM)

The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of run-off quantity and quality from primarily urban areas. The run-off component of SWMM operates on a collection of sub-catchment areas that receive precipitation and generate run-off and pollutant loads. The routing portion of SWMM transports this run-off through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of run-off generated within each sub-catchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprise of multiple time steps. This software is freeware.

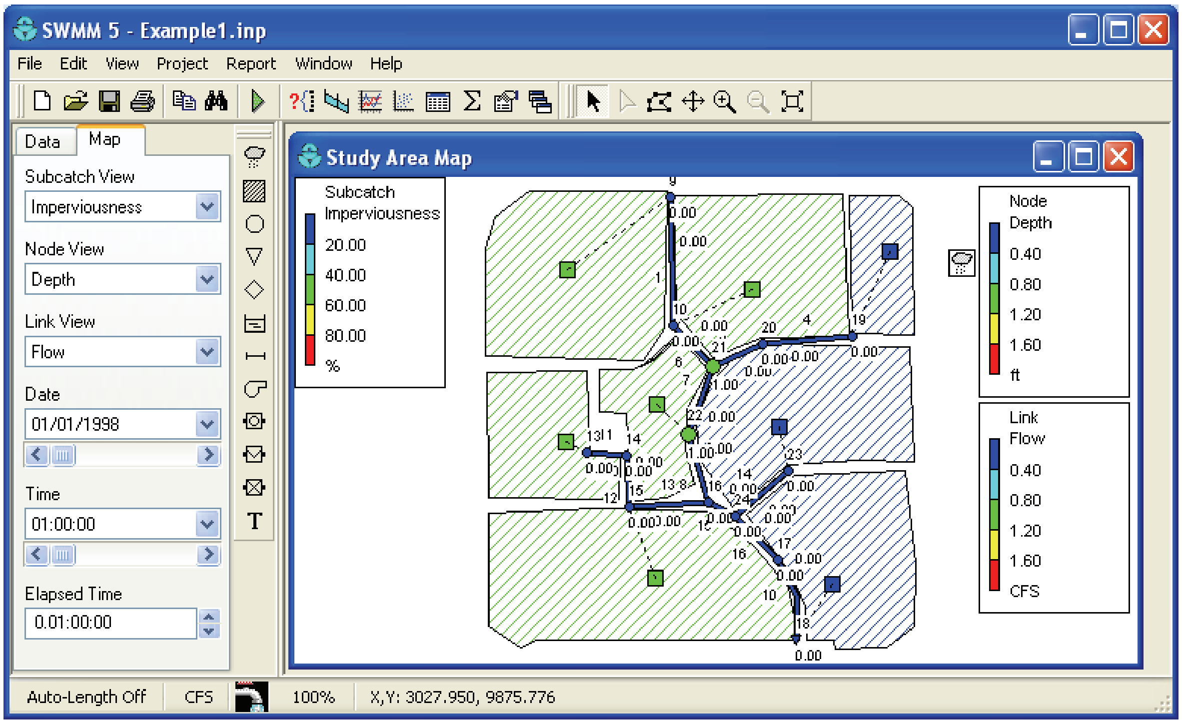

Figure 10.7: EPA Storm Water Management Model (SWMM)

10.3.4 BridgeLCC

BridgeLCC is user-friendly, life-cycle costing software developed by the National Institute of Standards and Technology (NIST) to assist bridge engineers in assessing the cost-effectiveness of new, alternative construction materials. The software uses a life-cycle costing methodology based on both ASTM standard E 917 and a cost classification developed at NIST.

BridgeLCC is specifically tailored for comparing new and conventional bridge materials, for example high-performance concrete versus conventional concrete, but works equally well when analysing alternative conventional materials. Also, it can be used to analyze pavements, piers and other civil infrastructure. This software is freeware.

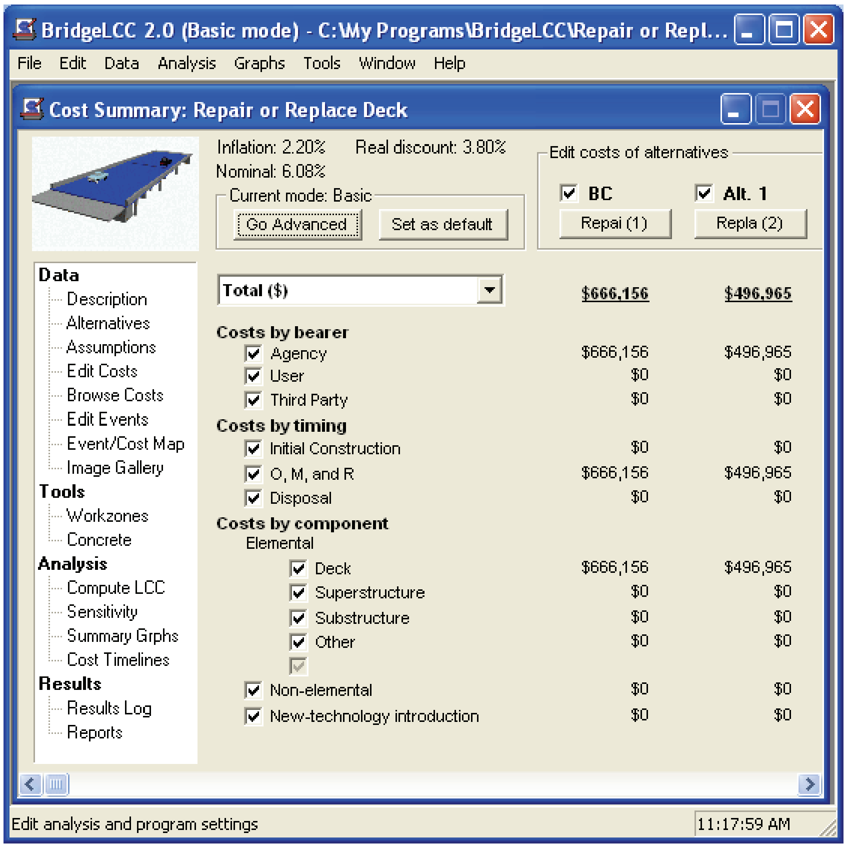

Figure 10.8: BridgeLCC

10.4 FEEDBACK

The project team would like to receive your comments and feedback on this chapter. Please visit the following website and voice your opinion, comments or suggestions for future software updates.

http://www.sinotechcc.co.za

ADDENDA

SANRAL may from time to time issue addenda to this manual which is to be inserted in this space below. Such addenda will be placed on SANRAL's website www.nra.co.za and it remains the user's responsibility to download these and update the Drainage Manual. The 2nd print includes all corrections up to 15 October 2007.

WEB2P

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