

The South African National Roads Agency SOC Limited

# DRAINAGE MANUAL

APPLICATION GUIDE

6th Edition



*Creating wealth through infrastructure*

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## FOREWORD

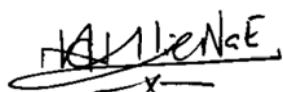
Water is often thought of as the source of civilization; hence the hypothesis that the development of hydraulics is related to the evolution of ancient societies such as those of Mesopotamia and Egypt. Owing to the structure of the early states, which entailed a closed system of absolute monarchy and 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 on-going challenge since the days of the early mathematicians through to modern-day engineers. We trust that this **Application Guide**, 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

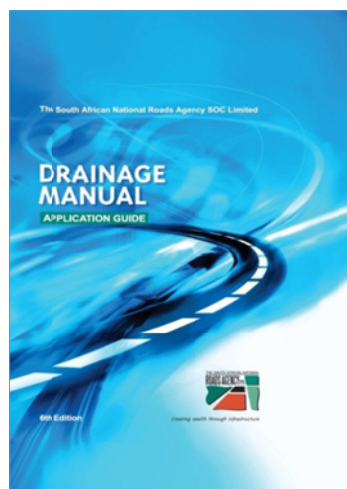
## ACKNOWLEDGEMENTS AND STRUCTURE OF THE DRAINAGE MANUAL

The South African National Roads Agency SOC Ltd (SANRAL) wishes to thank all parties involved in extensively revising and updating the *Road Drainage Manual* (first published in 1981) and now known as the *Drainage Manual*. The previous editors and authors of the original manual, J Bosman, A Rooseboom, MS Basson, CH Loots, JH Wiggett, assisted by ZP Kovács and AM van Vuuren (née Mouton), is hereby also acknowledged.

In realising the goals of producing a manual of high standard the co-operation between authors and reviewers to this and previous editions of the manual has been critical. All contributions, too numerous to mention individually, both big and small is gratefully acknowledged. We have in the manual endeavoured to take differing views into account which at times has proven to be a challenge. The manual, we believe, is a summary of both historical and modern thought pertaining to drainage.

Feedback, comments and suggestions from users of the previous editions of the manual have been incorporated where possible. This edition of the manual still covers all the previous background theory but has been extended to include additional flood calculation methods, the analyses and design of stormwater systems, the hydraulic assessment of existing culverts and the modelling of free surface flows and flood line calculations. With the further expansion of the manual it was deemed necessary to separate the manual into two distinct documents; the first being the **Drainage Manual** and the second being the **Drainage Manual Application Guide**.

The front covers of the two documents are as shown below.



Thank you to my fellow editors, Professor Fanie van Vuuren, Marco van Dijk and Nuno Gomes for their commitment and enthusiasm in updating the manual. The compilation and editing was not an easy task but has been completed with passion and dedication.

  
**Edwin Kruger**

**Editor**

**The South African National Roads Agency SOC 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](mailto:bridges@nra.co.za)

# DRAINAGE MANUAL APPLICATION GUIDE

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## LIST OF SYMBOLS

### Chapter 2

F	=	future value
i	=	annual discount rate as a decimal fraction
IRR	=	internal rate of return technique
n	=	discount period in years
NPV	=	net present value
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 (km <sup>2</sup> )
ARF	=	area reduction factor (%)
ARF <sub>IT</sub>	=	area reduction factor (%)
b	=	constant
C	=	run-off coefficient (dimensionless)
C	=	catchment parameter with regard to reaction time
C <sub>1</sub>	=	run-off coefficient for rural area with a value between zero and one
C <sub>1D</sub>	=	rural run-off coefficient incorporating the effect of dolomites
C <sub>1T</sub>	=	rural run-off coefficient incorporating the effect of dolomites and initial saturation factor
C <sub>100</sub>	=	calibration coefficient (SDF method)
C <sub>2</sub>	=	calibration coefficient (SDF method)
C <sub>2</sub>	=	run-off coefficient for urban area with a value between zero and one
C <sub>3</sub>	=	run-off coefficient for lakes with a value between zero and one
CN	=	Curve Number
CN <sub>f</sub>	=	Final Curve Number
CN <sub>w</sub>	=	Curve Number for wet conditions
CN-II	=	retardance factor approximated by the initial Curve Number unadjusted for antecedent soil moisture
C <sub>p</sub>	=	run-off coefficient according to average soil permeability
C <sub>s</sub>	=	run-off coefficient according to average catchment slope
C <sub>T</sub>	=	combined run-off coefficient for T-year return period (dimensionless)
C <sub>v</sub>	=	run-off coefficient according to average vegetal growth
D	=	storm duration (hours)
F	=	lag coefficient
F <sub>T</sub>	=	adjustment factor for initial saturation for return period T
f <sub>IT</sub>	=	flood run-off factor (%)
H	=	height (m)
H	=	height of most remote point above outlet of catchment (m)
H <sub>0,10L</sub>	=	elevation height at 10% of the length of the watercourse (m)
H <sub>0,85L</sub>	=	elevation height at 85% of the length of the watercourse (m)
he <sub>IT</sub>	=	effective rainfall (mm)
I	=	rainfall intensity (mm/hour)
I <sub>a</sub>	=	initial losses (abstractions) prior to the commencement of stormflow, comprising of depression storage, interception and initial infiltration (mm)
I <sub>T</sub>	=	average rainfall intensity for return period T (mm/h)
K	=	regional constant
K <sub>RP</sub>	=	constant for T-year return period
K <sub>T</sub>	=	constant for T-year return period
K <sub>u</sub>	=	dimensionless factor
l	=	hydraulic length of catchment along the main channel (m)

L	=	hydraulic length of catchment (watercourse length) (km)
L	=	catchment lag time (h)
$L_C$	=	distance from outlet to centroid of catchment area (km)
M	=	2-year return period daily rainfall from TR102
MAP	=	mean annual precipitation (mm/a)
n	=	length of record (years)
P	=	mean annual rainfall (mm/annum)
P	=	probability (%)
P	=	daily rainfall depth (mm), usually input as a one-day design rainfall for a given return period
$P_1$	=	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)
$P_{t,T}$	=	the precipitation depth for a duration of $t$ minutes and a return period of T years
$q_p$	=	peak discharge ( $m^3/s$ )
Q	=	peak discharge ( $m^3/s$ )
Q	=	stormflow depth (mm)
$Q_e$	=	peak discharge of unit hydrograph ( $m^3/s$ )
$Q_{iT}$	=	peak discharge for T-year return period ( $m^3/s$ )
$Q_p$	=	unit hydrograph peak discharge ( $m^3/s$ )
$Q_{RMF}$	=	regional maximum flood peak flow rate ( $m^3/s$ )
$Q_T$	=	peak discharge for T-year return period ( $m^3/s$ )
r	=	roughness coefficient
R	=	average number of days per year on which thunder was heard (days/year)
S	=	potential maximum soil water retention (mm),
$S_{av}$	=	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)
$\alpha$	=	area distribution factor
$\beta$	=	area distribution factor
$\gamma$	=	area distribution factor
$\Delta q_p$	=	peak discharge of incremental unit hydrograph ( $m^3/s$ )
$\Delta Q$	=	incremental stormflow depth (mm)
$\Delta D$	=	unit duration of time, used with the distribution of daily rainfall to account for rainfall intensity variations (hours)
$f_{30}$	=	30-minute rainfall intensity for the 2-year return period (mm/h)

#### Chapter 4

A	=	sectional area ( $m^2$ )
$A_1$	=	upstream sectional area ( $m^2$ )
$A_2$	=	downstream sectional area ( $m^2$ )
B	=	free surface width of cross section (m)
C	=	Chézy constant
E	=	specific energy (m)
Fr	=	Froude number



$g$	=	gravitational acceleration ( $\text{m/s}^2$ )
$h_l$	=	transition loss (m)
$h_f$	=	friction losses (m)
$k_s$	=	measure of absolute roughness (m)
$L$	=	distance (m)
$P$	=	wetted perimeter (m)
$q$	=	discharge per unit width ( $\text{m}^3/\text{s}/\text{m}$ )
$Q$	=	discharge ( $\text{m}^3/\text{s}$ )
$r_c$	=	centre line radius (m)
$R$	=	hydraulic radius i.e. area divided by wetted perimeter (m)
$Re$	=	Reynolds number
$S_o$	=	Bed slope (m/m)
$S$	=	energy slope, which is equal to bed slope only when flow is uniform (m/m)
$v$	=	uniform channel velocity (m/s)
$\bar{v}$	=	average velocity (m/s)
$\bar{v}_c$	=	critical flow velocity (m/s)
$y$	=	depth of flow measured perpendicular to the streambed (m)
$\bar{y}$	=	distance between water surface and centre of gravity of section (m)
$y_c$	=	critical flow depth (m)
$y_n$	=	normal/uniform flow depth (m)
$z$	=	bed level at point where depth of flow = $y$ (m)
$\gamma$	=	specific weight (value for water $9,8 \times 10^3 \text{ N/m}^3$ )
$\Delta x$	=	distance (m)
$\rho$	=	mass density = $1\,000 \text{ kg/m}^3$ for water
$\nu$	=	kinematic viscosity ( $\approx 1,14 \times 10^{-6} \text{ m}^2/\text{s}$ for water)

## Chapter 5

$A$	=	effective cross-sectional plan area of the opening ( $\text{m}^2$ )
$A$	=	cross sectional area ( $\text{m}^2$ )
$B$	=	total flow width (m)
$C$	=	inlet coefficient (0,6 for sharp edges or 0,8 for rounded edges)
$C$	=	Chézy constant
$C_D$	=	discharge coefficient
$d$	=	flow depth of water (mm)
$D$	=	depth of flow (m)
$d_1$	=	particle size (m)
$d_2$	=	side slope particle size (m)
$E$	=	specific energy (m)
$F$	=	blockage factor (say, 0,5)
$Fr$	=	Froude number
$H$	=	total energy head above grid (m)
$H$	=	energy head $\approx$ flow depth for upstream conditions (m)
$H$	=	head (m)
$I$	=	rainfall intensity (mm/h)
$K_L$	=	discharge coefficient
$L_f$	=	length of flow path (m)
$n$	=	Manning roughness value ( $\text{s}/\text{m}^{1/3}$ )
$n_1$	=	road crossfall (%)
$n_2$	=	road gradient (%)
$P$	=	wetted parameter (m)
$Q$	=	discharge ( $\text{m}^3/\text{s}$ )
$R$	=	hydraulic radius i.e. area divided by wetted perimeter (m)
$s$	=	energy gradient (m/m)

$S$	=	bed slope (m/m)
$S_f$	=	slope of flow path (m/m)
$\bar{v}$	=	average velocity (m/s)
$W$	=	width of roadway (m)
$y$	=	depth of flow at deepest point (m)

## Chapter 6

$A_{\text{over}}$	=	area of flow over structure at the flow depth selected ( $\text{m}^2$ )
$A_{\text{eff}}$	=	the effective inlet area through the structure ( $\text{m}^2$ )
$B$	=	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 ( $9,81 \text{ m/s}^2$ )
$L_B$	=	the total width of the deck of the structure (m)
$n$	=	Manning n-value ( $\text{s/m}^{1/3}$ )
$n_{\text{concrete}}$	=	Manning roughness coefficient of concrete ( $\text{s/m}^{1/3}$ )
$n_{\text{river}}$	=	Manning roughness coefficient of the river bed ( $\text{s/m}^{1/3}$ )
$P_{\text{cell}}$	=	the total wetted perimeter of each cell (m)
$P_{\text{concrete}}$	=	the part of the wetted perimeter that has a concrete surface per cell (m)
$P_{\text{eff}}$	=	$\Sigma P_{\text{cell}}$ (effective wetted perimeter for the flow passing through the structure) (m)
$P_{\text{over}}$	=	wetted perimeter at the flow depth selected (m)
$P_{\text{river}}$	=	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 ( $\text{m}^3/\text{s}$ )
$Q_{\text{design}}$	=	design discharge ( $\text{m}^3/\text{s}$ )
$Q_{\text{over}}$	=	discharge over the structure within the selected flow depth ( $\text{m}^3/\text{s}$ )
$Q_{\text{under}}$	=	discharge capacity of the openings through the structure ( $\text{m}^3/\text{s}$ )
$R$	=	hydraulic radius (m)
$S_0$	=	slope in direction of flow (m/m)
$\bar{v}_{\text{under}}$	=	average velocity of flow through the structure (m/s)
$x$	=	thickness of the deck (depending on the structural design outcome) (m)

## Chapter 7

$A$	=	cross sectional area ( $\text{m}^2$ )
$B$	=	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)
$Fr$	=	Froude number
$h_{f1-2}$	=	friction losses between cross-section 1 and 2 (m)
$\Sigma h_{1-2}$	=	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)
$K_{\text{in}}$	=	inlet secondary loss coefficients
$K_{\text{out}}$	=	outlet secondary loss coefficients
$R$	=	hydraulic radius i.e. area divided by wetted perimeter (m)
$S_0$	=	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)

## Chapter 8

$A_1$	=	flow area at section 1 ( $m^2$ )
$A_4$	=	flow area at section 4 ( $m^2$ )
$A_n$	=	flow area at for normal flow conditions ( $m^2$ )
$A_{n2}$	=	projected flow area at constricted section 2 below normal water level ( $m^2$ )
$B$	=	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
$C_b$	=	backwater coefficient
$D$	=	flow depth (m)
$d_{50}$	=	average particle diameter (m)
$D_{50}$	=	median size of bed material (m)
$d_{avg}$	=	average depth in the main channel
$D_c$	=	critical particle size for the critical velocity $V_c$ (m)
$d_s$	=	local scour depth at pier (m)
$E_s$	=	specific energy (m)
$F_D$	=	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 ( $9,81 \text{ m/s}^2$ )
$h^*$	=	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
$L$	=	pier length (m)
$n$	=	Manning's coefficient of roughness ( $s/m^{1/3}$ )
$Q$	=	total discharge ( $m^3/s$ )
$Q$	=	equivalent steady discharge which would generate the channel geometry ( $m^3/s$ )
$Q_T$	=	design flood ( $m^3/s$ )
$Q_{2T}$	=	twice the recurrence interval design flood ( $m^3/s$ )
$S$	=	energy slope (m/m)
$s$	=	specific gravity of soil particles
$SF$	=	required stability factor to be applied
$q$	=	discharge per unit width ( $m^3/s.m$ )
$q$	=	discharge through the sub-channel ( $m^3/s.m$ )
$V$	=	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)
$\bar{y}$	=	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)
$\alpha_1$	=	velocity coefficient
$\alpha_2$	=	velocity head coefficient for the constriction
$\theta$	=	bank angle with the horizontal (°)
$\rho$	=	density of water ( $\text{kg/m}^3$ )
$\rho_d$	=	dry bulk density ( $\text{kg/m}^3$ )
$\rho_s$	=	saturated bulk density ( $\text{kg/m}^3$ )
$\phi$	=	riprap angle of repose (°)
$\tau_c$	=	critical tractive stress for scour to occur ( $\text{N/m}^2$ )
$\nu$	=	kinematic fluid viscosity ( $\text{m}^2/\text{s}$ )

## Chapter 9

$A$	=	full-flow area ( $\text{m}^2$ )
$A_1, A_2$	=	full-flow area for the inlet pipe and outflow pipe ( $\text{m}^2$ )
$D$	=	pipe inner diameter (m)
$g$	=	gravitational acceleration ( $9,81 \text{ m}^2/\text{s}$ )
$h$	=	difference in elevation between the highest incoming pipe invert and the centreline of the outlet pipe (m)
$h_{f1-2}$	=	friction losses between cross-section 1 and 2 (m)
$h_L$	=	minor loss (m)
$h_{l1-2}$	=	secondary losses between cross-section 1 and 2 (m)
$k$	=	minor loss coefficient
$k_s$	=	absolute roughness of conduit (m)
$n$	=	coefficient of roughness ( $\text{s/m}^{1/3}$ )
$P$	=	wetted perimeter (m)
$Q$	=	flow rate ( $\text{m}^3/\text{s}$ )
$R$	=	hydraulic radius (m) – $A/P$
$S$	=	slope of the energy grade line (m/m)
$v_1, v_2$	=	velocity of flow in the inlet pipe and outflow pipes (m/s)
$y_c$	=	critical depth (m)

$z_1, z_2$	=	invert elevations of the inflow pipes relative to the outlet pipe invert (m)
$\gamma$	=	specific weight (value for water $9,8 \times 10^3 \text{ N/m}^3$ )
$\nu$	=	kinematic viscosity ( $\text{m}^2/\text{s}$ )

#### Chapter 10

$D$	=	vertical dimension of the existing culvert.
$\frac{dS}{dt}$	=	the change in storage over the time step of $dt$ ( $\text{m}^3$ )
$\bar{I}$	=	average inflow ( $\text{m}^3/\text{s}$ )
$N$	=	auxiliary function ( $\text{m}^3/\text{s}$ )
$O$	=	outflow through culvert ( $\text{m}^3/\text{s}$ )
$\bar{O}$	=	average outflow ( $\text{m}^3/\text{s}$ )
$Q_{T0}$	=	design flood for the design return period which was obtained from the review of the road classification, $R_{C0}$ and the index flood, $Q_{20}$ ( $\text{m}^3/\text{s}$ ).
$Q_{C1}$	=	maximum calculated existing inlet capacity of the culvert by limiting the total energy head to $1,2D$ ( $\text{m}^3/\text{s}$ )
$Q_{C2}$	=	maximum calculated current existing hydraulic capacity of the culvert by limiting the total energy head to be equal to the shoulder brake point level (SBP) ( $\text{m}^3/\text{s}$ )
$Q_{T1}$	=	design flood for the design return period which was obtained from the review of the road classification, $R_{C-1}$ and the index flood, $Q_{20}$ ( $\text{m}^3/\text{s}$ )
$Q_{20}$	=	index flood for the contributing catchment with a return period of 20 years ( $\text{m}^3/\text{s}$ )
$Q_{2T0}$	=	flow rate related to a return period twice that which was obtained for the design flood, $Q_{T0}$ ( $\text{m}^3/\text{s}$ )
$Q_{2T1}$	=	flood rate related to a return period twice that which was obtained for the design flood, $Q_{T1}$ ( $\text{m}^3/\text{s}$ )
$R_{C0}$	=	original road classification
$R_{C-1}$	=	reflects the selection of a road classification which is one class less than that determined for the road
$S$	=	temporal storage or ponding volume ( $\text{m}^3$ )
$S$	=	sum of the storage volume of the prism and the wedge ( $\text{m}^3$ )
$T$	=	design return period
$T_c$	=	time of concentration (h)
$T_s$	=	total time during the routing of the flood when the upstream energy head is more than $1,2D$ (h)
$V_{T1}$	=	maximum storage volume upstream of the culvert assuming level-pool routing conditions and an inflow hydrograph with a peak flow rate of $Q_{T1}$ and a triangular distribution with a base width of $3T_c$ and the peak discharge occurring at $T_c$ ( $\text{m}^3$ )
$V_{\text{storm}}$	=	calculated storm volume based on the assumption of an inflow hydrograph with a peak flow rate of $Q_{T1}$ and a triangular distribution with a base width of $3T_c$ and the peak discharge occurring at $T_c$ ( $\text{m}^3$ )
$x$	=	a dimensionless weighting factor indicating the relative importance of the inflow (I) and the outflow (O) in determining the storage (S) in the reach
$\Delta S$	=	change in storage volume ( $\text{m}^3$ )
$\Delta t$	=	time step that is used (s)
$\frac{I_1 + I_2}{2} \Delta t$	=	average volumetric inflow ( $\text{m}^3$ )
$\frac{O_1 + O_2}{2} \Delta t$	=	average volumetric outflow ( $\text{m}^3$ )

#### Chapter 11

$A$	=	cross sectional flow area (m)
$B$	=	top width (m)
$dy$	=	change in water depth (m)

$dx$	=	distance over which change occurs (m)
$E_{sc}$	=	specific minimum energy (m)
$Fr$	=	Froude number
$g$	=	gravitational acceleration ( $m/s^2$ )
$h$	=	stage height (m)
$Q$	=	flow rate ( $m^3/s$ )
$S$	=	bed slope (m/m)
$S_c$	=	critical slope (m/m)
$S_f$	=	represents the slope of the total energy line
$S_0$	=	bed slope (m/m)
$\bar{V}$	=	mean cross-sectional velocity
$y$	=	flow depth (m)
$y_c$	=	critical flow depth (m)
$y_n$	=	normal flow depth (m)
$\alpha$	=	velocity coefficient
$\Delta E_s$	=	specific energy (m)

## Chapter 12

$A$	=	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)
$\psi$	=	geotextile permittivity



# 1 CHAPTER 1 - INTRODUCTION

The SANRAL Drainage Manual had a major update in 2006 which included additional chapters, worked examples and links to applicable drainage software. SANRAL decided to revise and upgrade the manual to be used as a user tool for all persons involved in the design of drainage structures and systems. The purpose of the Drainage Manual (cover shown in **Figure 1.1**) 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 numerous worked examples were well received by industry as well as academia whom prescribe the Drainage Manual as part of the Civil engineering curriculum that it was decided to expand this. The document was split into the *Drainage Manual* and the *Drainage Manual Application Guide* (this document).



**Figure 1.1: SANRAL Drainage Manual**

The *Drainage Manual Application Guide* contains the following:

- Hand calculations of typical problems.
- Reference to applicable software utilities and user manuals for the programs.
- Step-by-step worked examples using freeware software programs.

## **1.1 Layout of the Drainage Manual Application Guide**

The *Drainage Manual Application Guide* contains twelve chapters. The focuses in the different chapters are:

Chapter 1: Provides an introduction to the Drainage Manual Application Guide.

Chapter 2: Review the economic considerations.

Chapter 3: Illustrate the various flood calculation methods for different recurrence intervals.

Chapter 4: Reflects basic hydraulic calculations.

Chapter 5: Demonstrates surface drainage design.

Chapter 6: Hydraulic analysis of low-level crossings.

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

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

Chapter 9: Storm water analyses and design.

Chapter 10: Assessment of hydraulic capacity of existing drainage structures and the application of flood routing.

Chapter 11: Free surface flow determination.

Chapter 12: Discusses sub-surface drainage.

It is trusted that the document will provide valuable assistance in the design of drainage systems. References to all the figures and literature can be found in the Drainage Manual

## 2 ECONOMIC EVALUATION OF DRAINAGE SYSTEMS

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.

### 2.1 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:

$$NPV = \frac{F}{(1 + i)^n} \quad \dots(2.1)$$

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.

### 2.2 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.

$$F = P(1 + i)^n \quad \dots(2.2)$$

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

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

$$P = \frac{F}{(1 + i)^n} \quad \dots(2.3)$$

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

### 2.3 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  $NPV_{\text{income}} = NPV_{\text{expenditure}}$

$$NPV_{\text{income}} = \frac{250}{(1 + i)^1} + \frac{350}{(1 + i)^2} + \frac{600}{(1 + i)^3} + \frac{100}{(1 + i)^4} + \frac{400}{(1 + i)^5} \quad \dots(2.4)$$

$$NPV_{\text{expenditure}} = \frac{1300}{(1 + i)^0} \quad \dots(2.5)$$

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”

$$IRR = 9,525\%$$

### 3 FLOOD CALCULATIONS

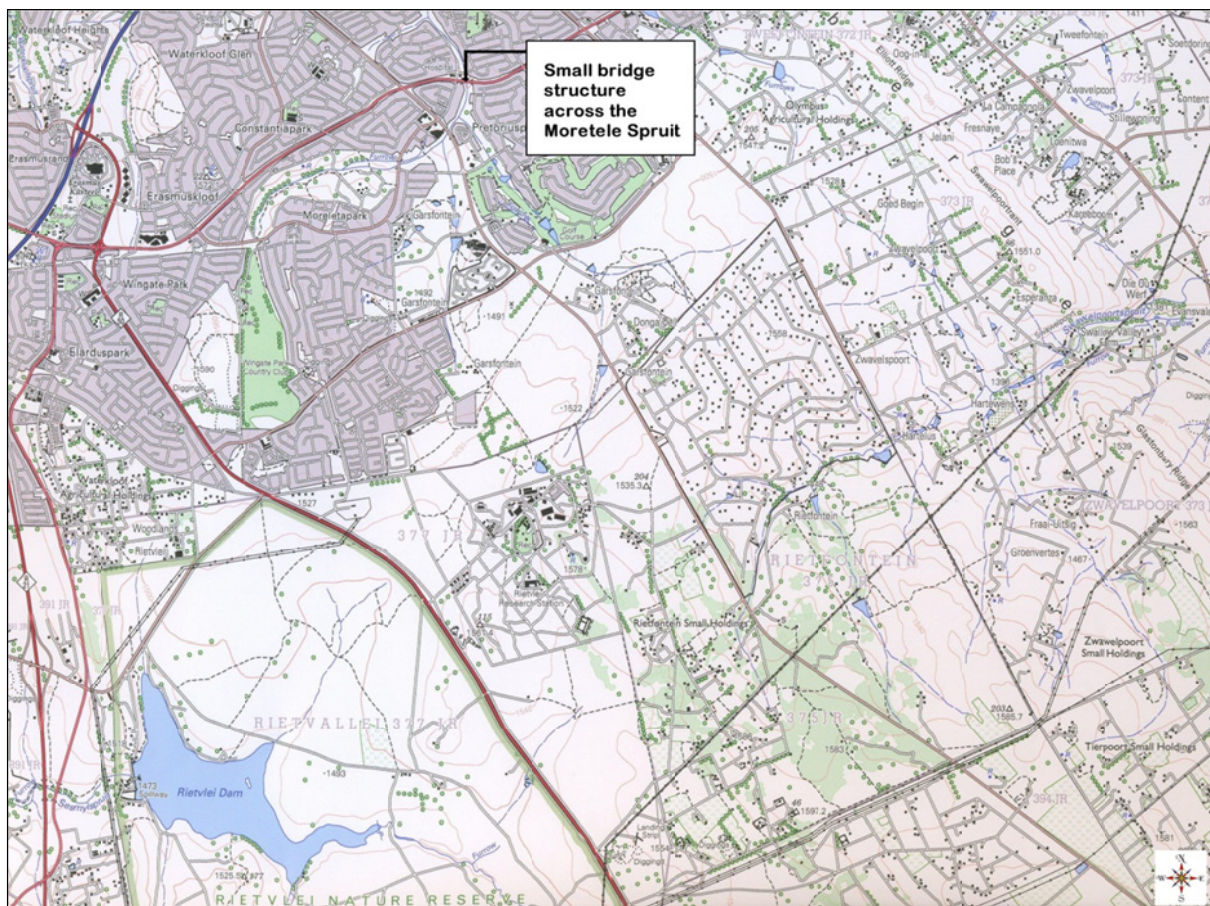
The main aim of this section is to provide the reader with a step-by-step explanation of the procedures used to calculate flood magnitudes for different return periods.

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.

#### 3.1 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 3.1**).

The small bridge, as shown in **Figure 3.2**, is located in Pretoria East (location indicated on **Figure 3.1**). 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 3.1: Moretele Spruit catchment area (shown on a 1:50 000 topographical map)**





**Figure 3.2: Small Bridge across the Moretele Spruit (Hans Strijdom Drive)**

### **3.1.1 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 precipitation and rainfall region to determine average rainfall intensity

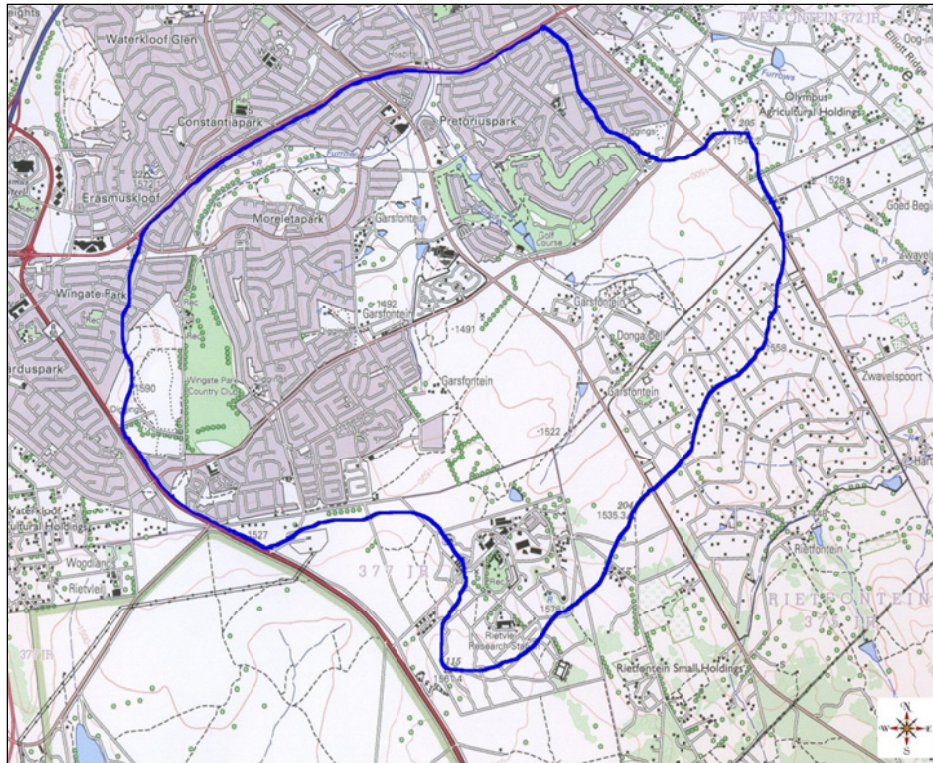
#### *Calculation procedure*

**Step 1:** Determine the catchment area (km<sup>2</sup>).

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 use of Geographical Information Systems (GIS) has permeated almost every field in the engineering, natural and social sciences. GIS do not inherently have all the hydrological simulation capabilities that complex hydrological models do, but are used to determine many of the catchment parameters that hydrological models or design flood estimation methods require. Other software applications such AutoCAD could also be used to determine the catchment area.

The catchment area of the Moretele Spruit up to the small bridge is  $28,5 \text{ km}^2$  as shown in **Figure 3.3**.



**Figure 3.3: 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  $L = 7,25 \text{ km}$ .

**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 3.4**.

$$S_{av} = \frac{H_{0,85L} - H_{0,10L}}{(1\ 000)(0,75L)} \quad \dots (3.1)$$

where:

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

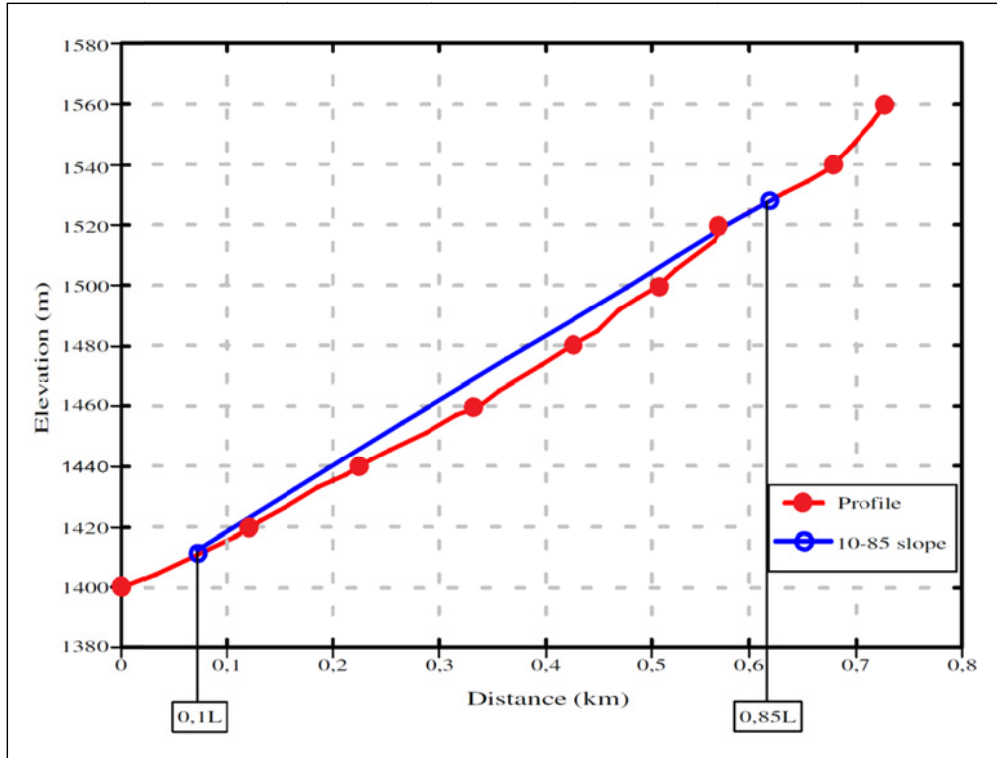
$H_{0,10L}$  = elevation height at 10% of the length of the watercourse (m)

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

The calculated average slope for this example is  $0,02146 \text{ m/m}$ .



**Figure 3.4: Longitudinal profile of the Moretele Spruit**

**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.

$$T_C = \left( \frac{0,87L^2}{1\,000 S_{av}} \right)^{0,385} \quad \dots (3.2)$$

where:

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

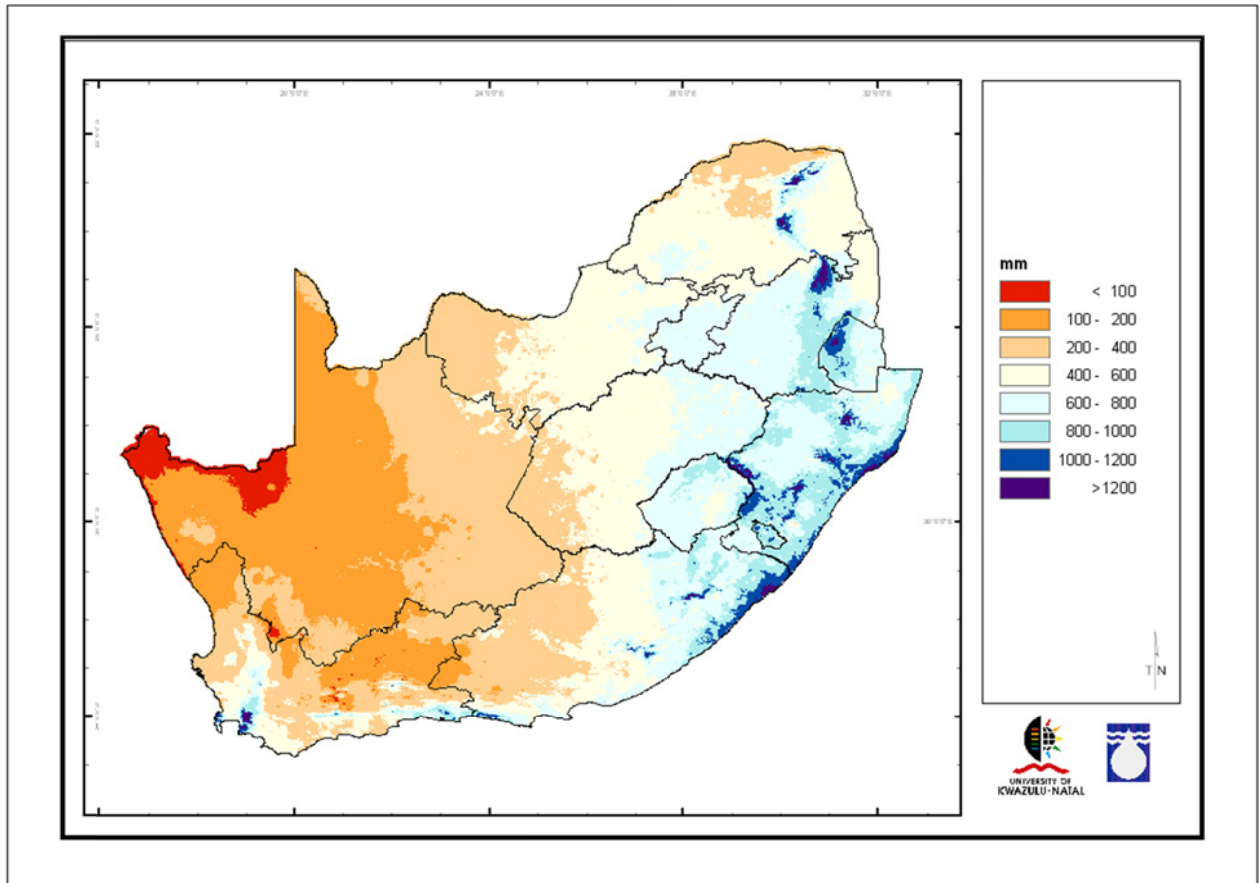
In most cases the longest water path includes both overland and channel flows. In large catchments the channel flow is usually dominant, but in small catchments it may be necessary to determine  $T_C$  as the sum of the flow times for both the overland and channel flow stretches. To obtain a broad indication, 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 km<sup>2</sup>.

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

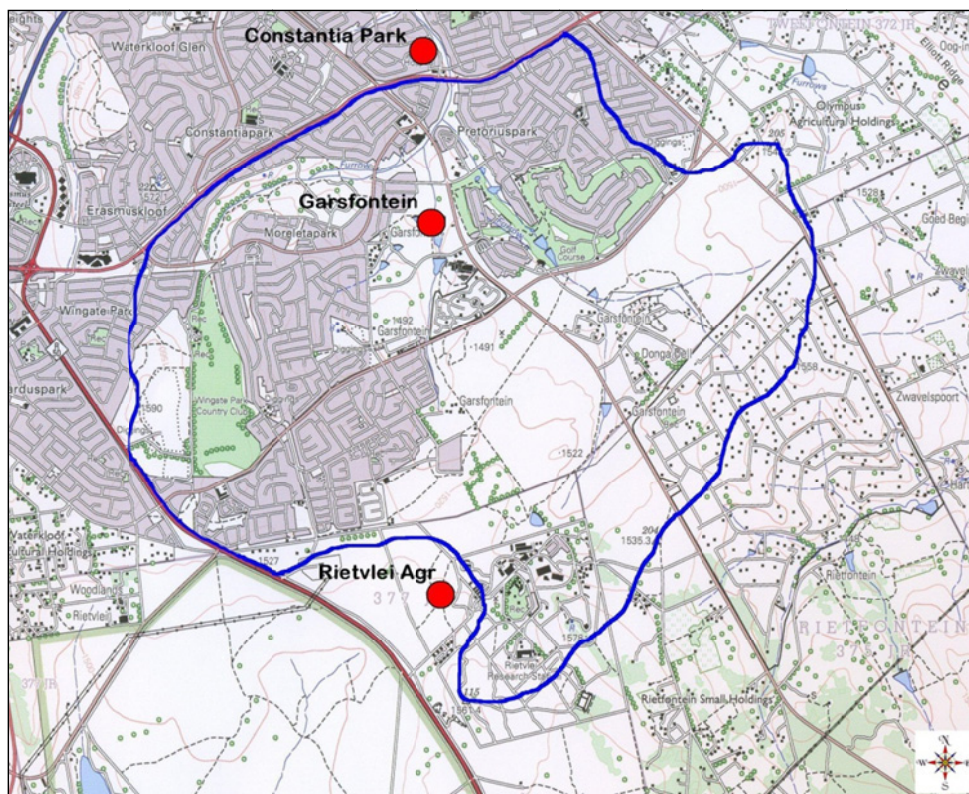
**Step 5:** Obtain the mean annual precipitation (MAP), from the South African Weather Service or from the simplified **Figure 3.5**.

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 3.6**.





**Figure 3.5: Mean Annual Precipitation**



**Figure 3.6: Rainfall stations used in determining the representative MAP**

The weighted area method was used to determine the representative mean annual precipitation 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 3.1: Mean annual precipitation of catchment area**

Weather Service rainfall station	Latitude D M	Longitude D M	MAP (mm)	Area (km <sup>2</sup> )
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
<b>Total</b>			746,6	28,5

Also determine the rainfall region in which the catchment falls.

The mean annual precipitation (MAP) for this catchment is 746,6 mm (see **Table 3.1**) and the catchment is located in the inland region.

**Historically there have been a number of ways in which the rainfall intensity could be determined. These alternative methods have been retained in this document although the latest method using the Design Rainfall Estimation Software is recommended (Alternative 3).**

- **Alternative 1** – Original method using Depth-Duration-Frequency Diagram.
- **Alternative 2** – The TR102 representative rainfall data and the modified Hershfield equation is used (in the past this was referred to as the Alternative Rational Method).
- **Alternative 3** – The design rainfall from the Design Rainfall Estimation software is used to determine the point rainfall of the catchment.

#### **Alternative 1**

**Step 6a:** Determine the point rainfall values ( $P_T$ ) (mm) for the required return periods. Based on the mean annual precipitation (MAP), the rainfall region, the time of concentration ( $T_C$ ) and the required return period, **Figure 3.7** can be used to determine the point rainfall. As shown in **Figure 3.7** the point rainfall for the 1:20 year and 1:50 year return periods are determined using the co-axial Depth-Duration-Frequency diagram.

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

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

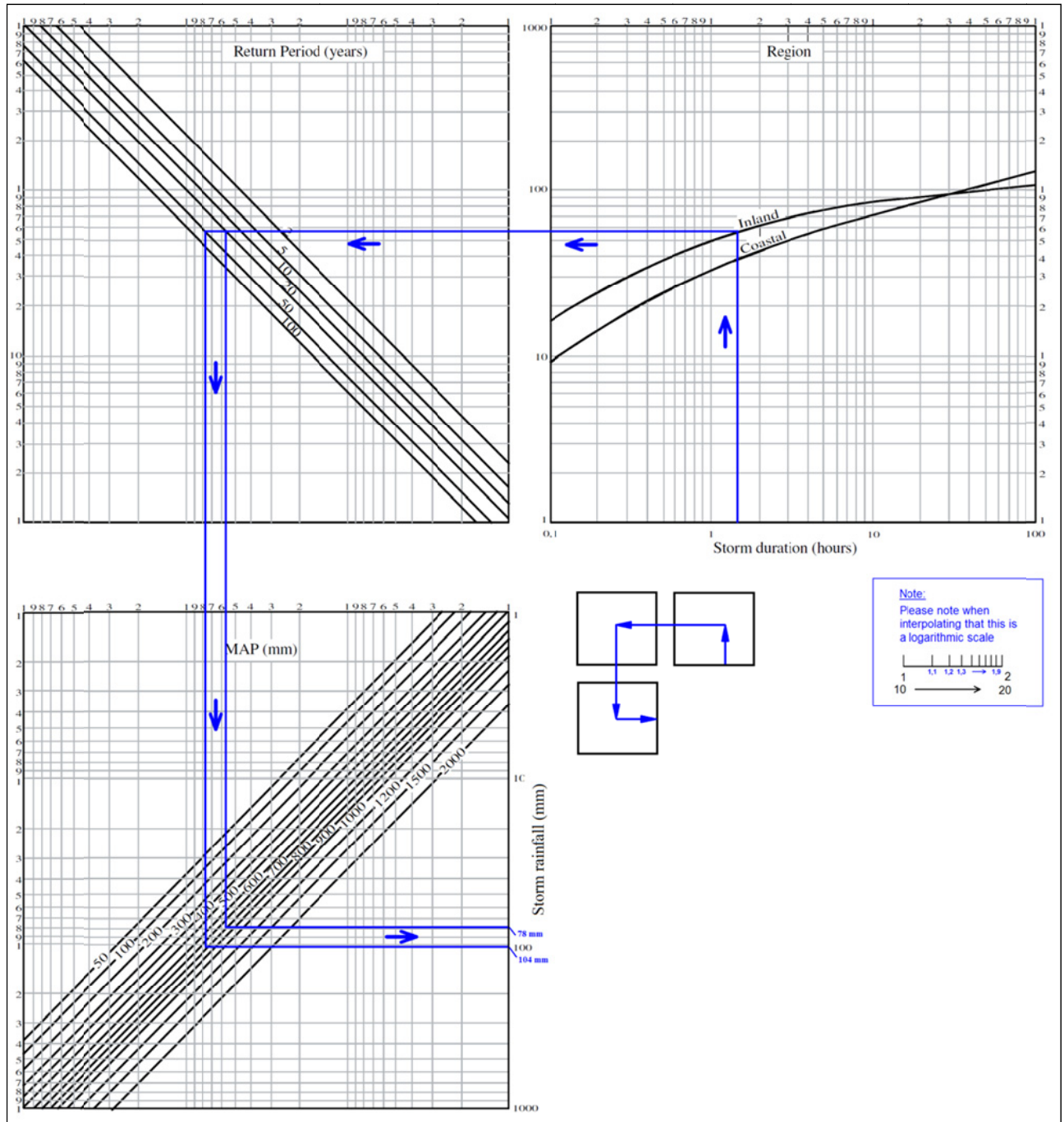
The point intensity ( $P_{iT}$ ) is the point rainfall divided by the time of concentration (if  $T_C > 0,25$  hours). If  $T_C \leq 0,25$  hours divide by 0,25 hours.

$$P_{iT} = \frac{P_T}{T_C} \quad \dots (3.3)$$

where:

$$\begin{aligned} P_{iT} &= \text{point intensity for the different return periods (mm/h)} \\ P_T &= \text{point rainfall (mm)} \\ T_C &= \text{time of concentration (hours)} \end{aligned}$$

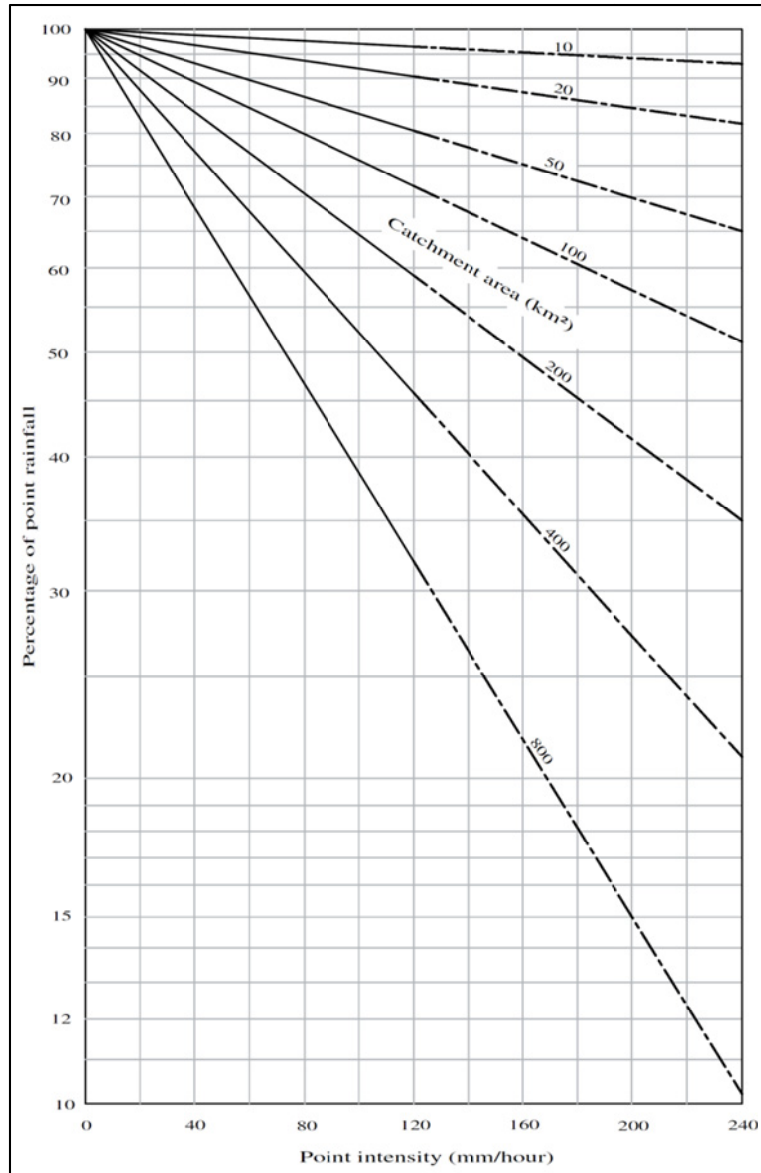
The point intensities for the 1:20 and 1:50 year return periods are  $P_{i20} = 58,3 \text{ mm/h}$  and  $P_{i50} = 77,7 \text{ mm/h}$  respectively.



**Figure 3.7: Determining the point rainfall utilizing Depth-Duration-Frequency diagram**

**Step 8a:** Determine the area reduction factors (ARF) for the different return periods.

In this example the catchment area is small and thus **Figure 3.8** is used. The resulting ARFs are  $ARF_{20} = 94\%$  and  $ARF_{50} = 91\%$  for the 1:20 and 1:50 years return periods respectively.



**Figure 3.8: Expected percentage run-off as a function of point intensity (small areas), ARF**

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

$$I_T = P_{iT} \left( \frac{ARF_T}{100} \right) \quad \dots (3.4)$$

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  $I_{20} = 54,79 \text{ mm/h}$  and  $I_{50} = 70,71 \text{ mm/h}$ .

## Alternative 2

**Step 6b:** Determine the representative rainfall from the available TR102 South African Weather Service stations in and around the catchment (see **Table 3.2**).

**Table 3.2: Representative rainfall station from TR102**

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

**Step 7b:** 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.

$$P_{t,T} = 1,13(0,41 + 0,64\ln T)(-0,11 + 0,27\ln t)(0,79M^{0,69}R^{0,20}) \quad \dots (3.5)$$

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
- $R$  = average number of days per year on which thunder was heard (days/year) (**Figure 3.9**)

The average number of days on which thunder was heard ( $R$ ) is equal to 61 and  $M$  is 60 (from **Table 3.2**). The calculated precipitation depths are:

$$P_{120} = 85,55 \text{ mm and } P_{150} = 107,10 \text{ mm}$$

**Step 8b:** 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.

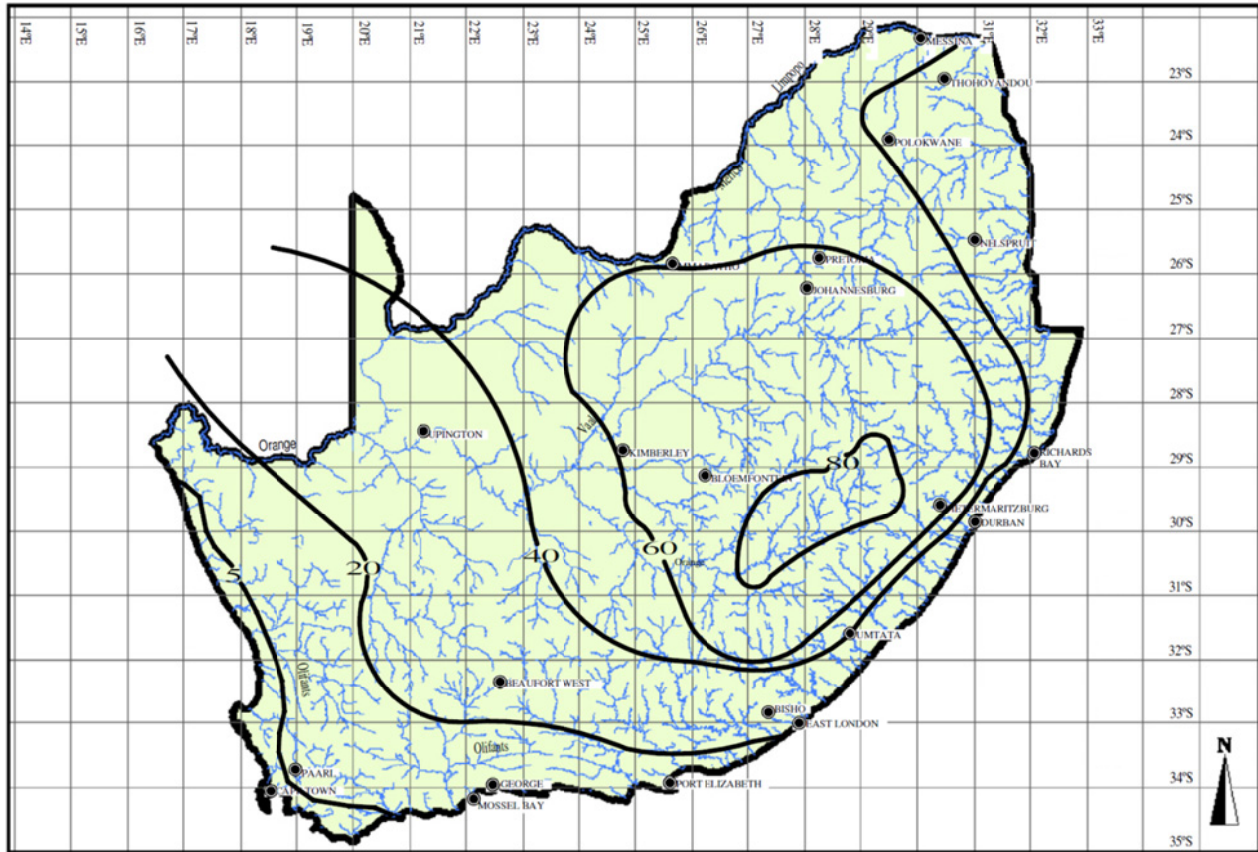
$$P_{iT} = \frac{P_{tT}}{T_C} \quad \dots (3.6)$$

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  $P_{i20} = 63,93 \text{ mm/h}$  and  $P_{i50} = 80,04 \text{ mm/h}$  respectively.





**Figure 3.9: Average number of days per year on which thunder was heard**

**Step 9b:** Determine the area reduction factors (ARF) for the different return periods from **Figure 3.10** or equation 3.7.

$$ARF = (90000 - 12800 \ln A + 9830 \ln(60T_C))^{0.4} \quad \dots (3.7)$$

where:

- ARF = area reduction factor as a percentage (should be less than 100%)
- A = catchment area (km<sup>2</sup>)
- T<sub>C</sub> = time of concentration (hours)

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

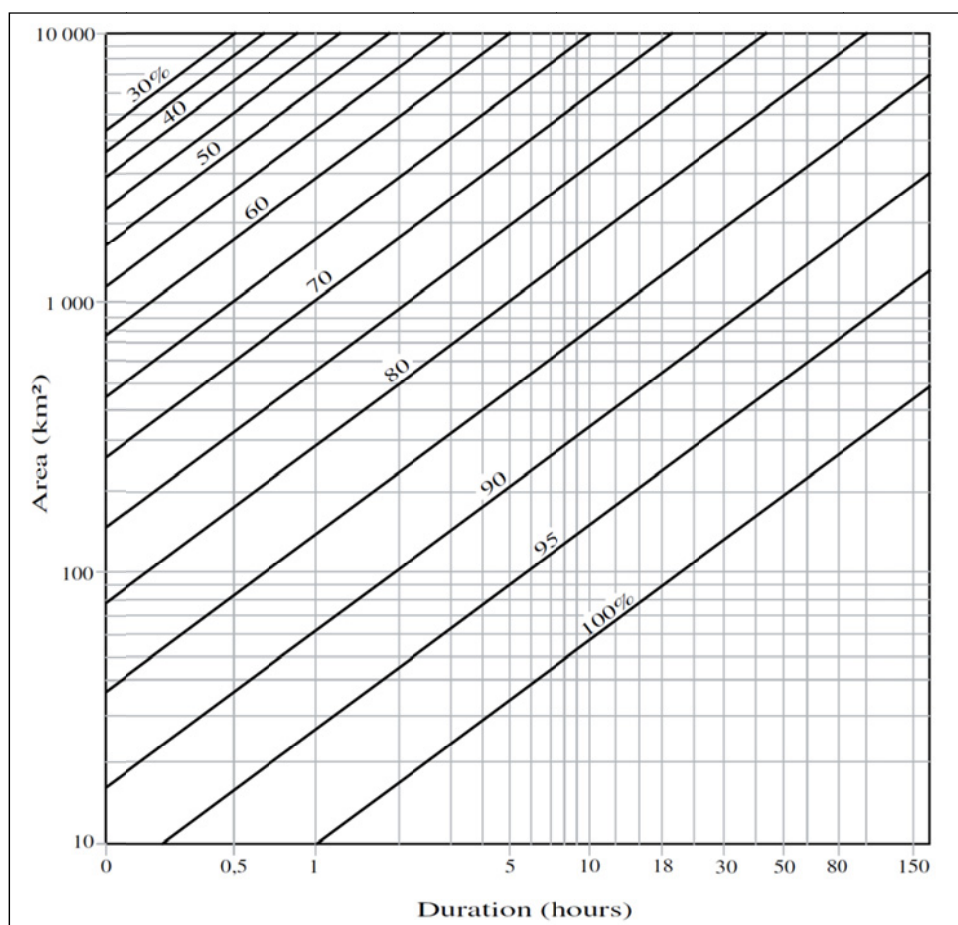
**Step 10b:** Determine the average rainfall intensity or effective catchment precipitation

$$I_T = P_{iT} \left( \frac{ARF_T}{100} \right) \quad \dots (3.8)$$

where:

- I<sub>T</sub> = rainfall intensity averaged over the catchment (mm/h) for the return period T.
- ARF<sub>T</sub> = area reduction factor as a percentage for return period T (should be smaller than 100 %)
- P<sub>iT</sub> = point intensities for the different return periods (mm/h)

The average rainfall intensities are I<sub>20</sub> = 61,4 mm/h and I<sub>50</sub> = 76,9 mm/h.



**Figure 3.10: Area reduction factors**

### **Alternative 3**

**This is the preferred method of obtaining point rainfall data for use in the Rational Method procedure.**

**Step 6c:** The design rainfall from the Design Rainfall Estimation software is the recommended method to determine the point design rainfall of a catchment (see **Figure 3.11**). Utilising the Design Rainfall Estimation in South Africa software application the representative weather station or coordinates as shown in **Figure 3.12** can be entered. A summary of all the closest rainfall stations as well as the n-day rainfall values as shown in **Figure 3.13** is obtained. The software enables the estimation of design rainfall for durations ranging from 5 minutes to 7 days and for 2 to 200 year return periods at any 1' latitude x 1' longitude point in South Africa.

Enter the coordinates or station and click on the proceed button (**Figure 3.12**) to obtain a summary of all the closest rainfall stations as well as the n-day rainfall values (see **Figure 3.13**).

SAWB NUMBER	Station Name	Latitude (°) (')	Longitude (°) (')	MAP (mm)	Altitude (m)	Years	Duration (days)	Return Period (years)																							
								2		5		10		20		50		100		200											
								L	D	U	L	D	U	L	D	U	L	D	U	L	D	U	L	D	U						
0001517 A	DAXGER POINT - VRT.	34	37	19	18	463	46	73	1	38	38	39	54	54	55	65	66	68	77	79	83	93	98	106	107	115	127	122	133	151	
									2	50	51	52	70	71	73	84	86	89	97	101	106	114	122	132	127	139	154	141	156	178	
									3	57	58	59	80	81	83	95	98	101	110	114	120	129	137	148	143	155	171	158	174	198	
									4	62	64	65	87	88	89	102	105	107	116	121	126	135	144	154	148	162	177	166	180	201	
									5	65	66	68	90	91	93	105	108	111	120	125	131	139	148	159	152	166	182	164	185	207	
									6	67	69	70	93	94	96	109	112	115	124	129	135	142	152	163	155	170	186	168	188	211	
									7	70	72	73	96	98	99	112	115	118	127	132	137	146	155	165	159	173	188	171	190	211	
0001517 W	DAXGER POINT (VRT)	34	37	19	18	463	46	93	1	37	38	38	53	53	54	64	65	67	76	78	82	92	97	104	105	113	125	121	131	149	
									2	48	49	50	67	68	70	80	82	85	93	97	102	109	117	126	122	133	147	135	150	171	
									3	55	56	56	76	77	79	91	93	96	105	109	114	123	131	141	136	148	163	150	166	188	
									4	58	60	61	81	82	84	95	98	101	109	114	119	127	135	145	139	152	166	151	169	189	
									5	61	62	64	84	86	87	99	102	104	113	118	122	130	139	149	142	156	171	154	173	195	
									6	63	65	66	87	89	90	102	105	108	116	122	127	134	143	153	146	160	175	158	177	198	
									7	66	68	69	90	92	93	105	108	111	119	125	129	137	146	155	150	162	176	161	179	199	
0001605 W	GANSBAAI	34	35	19	21	543	17	72	1	38	38	39	53	54	55	65	66	68	76	79	83	93	98	106	107	115	126	122	133	151	
									2	49	50	50	68	69	70	81	83	86	94	98	103	111	118	128	123	134	149	137	152	173	
									3	55	56	57	77	78	80	92	94	97	106	110	115	124	132	142	138	150	165	152	168	190	
									4	59	60	62	82	84	85	97	100	102	111	115	120	128	137	146	141	154	168	153	171	191	
									5	62	63	64	85	87	88	100	103	105	114	119	124	132	141	151	144	158	173	156	175	197	
									6	64	66	67	88	90	92	104	107	109	118	123	128	136	145	155	148	162	177	160	179	201	
									7	66	68	69	90	92	93	105	109	111	120	125	130	137	146	155	150	163	177	161	179	199	
0001726 W	UILENKRAAL (BOS)	34	36	19	25	530	9	32	1	37	38	38	53	53	54	64	65	67	75	78	81	92	97	104	105	113	125	120	131	149	
									2	46	47	48	65	66	67	78	80	82	90	93	98	106	113	122	118	128	142	130	145	165	
									3	53	54	55	75	76	77	88	91	93	102	106	111	120	127	137	133	144	159	147	162	184	
									4	59	60	61	82	83	84	96	99	101	110	114	119	127	136	145	139	152	166	151	169	190	
									5	62	63	64	85	86	88	100	103	105	114	119	124	131	141	150	144	158	173	155	175	196	
									6	64	66	67	88	90	92	104	107	109	118	123	128	136	145	155	148	162	177	160	179	201	
									7	67	68	70	92	93	95	107	110	113	121	126	131	139	148	157	152	164	179	163	181	201	
0002069 W	PETERS GATE	34	39	19	32	588	20	35	1	39	39	40	57	58	58	71	72	73	85	88	90	106	111	117	124	131	140	144	154	167	
									2	54	55	55	79	80	80	98	99	101	116	120	123	143	149	157	165	174	186	189	201	219	
									3	60	61	61	88	89	89	108	109	111	128	131	135	157	163	171	180	190	202	205	219	236	
									4	64	65	65	93	94	94	115	116	117	136	139	141	166	171	176	191	199	207	217	228	241	
									5	68	68	68	98	98	99	119	121	122	142	144	147	173	178	183	199	206	215	227	237	250	
									6	70	70	71	100	100	101	121	123	124	143	146	149	173	180	186	198	207	218	225	238	253	
									7	74	74	75	105	105	106	127	128	130	148	152	156	179	186	194	204	214	227	231	245	262	
0002256 W	QUOIN POINT (BOS)	34	46	19	39	571	15	25	1	45	45	46	66	66	67	82	83	84	98	101	104	123	128	135	143	151	161	167	177	193	
									2	57	58	58	84	84	85	103	105	106	122	126	130	151	158	165	174	184	196	200	213	231	
									3	63	63	64	91	92	93	112	114	115	133	137	140	163	170	177	187	197	210	213	227	246	
									4	67	68	68	98	98	99	120	121	122	143	145	148	174	179	185	200	208	217	228	239	252	
									5	70	70	71	101	101	102	123	124	126	146	149	151	178	184	189	205	213	222	234	244	258	
									6	74	74	74	105	106	106	127	129	131	151	154	157	183	189	196	209	219	230	237	250	267	
									7	76	76	77	108	108	109	130	132	134	153	157	160	184	192	199	210	220	233	237	252	270	
0002456 W	ELIDJ (POL)	34	35	19	45	532	53	61	1	39	39	40	57	58	58	71	72	73	85	88	90	107	111	117	125	131	141	145	154	168	
									2	51	51	52	75	75	76	92	93	95	109	112	116	134	140	147	155	164	175	178	190	206	
									3	57	58	58	84	84	85	102	104	106	121	125	128	149	155	162	171	180	192	195	208	225	
									4	62	62	62	90	90	90	110	111	112	131	133	135	160	164	169	183	191	198	206	219	231	
									5	65	65	66	93	94	94	114	115	116	136	138	140	165	170	175	190	202	206	217	227	239	
									6	68	68	69	97	97	98	117	119	121	139	142	145	168	174	181	192	201	212	219	231	246	
									7	70	70	71	99	100	100	120	122	123	141	144	148	170	177	184	194	203	215	219	232	249	

1

L=lower 90% error bound (mm); D=design rainfall depth (mm); U=upper 90% error bound (mm)



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User selection has the following criteria:  
Station Number: 0513529  
Duration requested: 5 m, 10 m, 15 m, 30 m, 45 m, 1 h, 1.5 h, 2 h, 4 h, 6 h, 8 h, 10 h, 12 h, 16 h, 20 h, 24 h, 1 d, 2 d, 3 d, 4 d, 5 d, 6 d  
Return Periods requested: 2 yr, 5 yr, 10 yr, 20 yr, 50 yr, 100 yr, 200 yr  
Block Size requested: 3 minutes

Data extracted from Daily Rainfall Estimate Database File  
The station selected and the five closest stations are listed

Station Name	SAWS Number	Distance (km)	Record (Years)	Latitude (°)	Longitude (°)	MAP (mm)	Altitude (m)	Duration (m/h/d)	Return 2	Period (years)	2L	2U	5		
GARSTFONTEIN	0513529_W	0.0	34	25	49	28	17	741	1440	1 d	55.9	55.6	56.2	77.2	76
										2 d	73.3	72.8	73.8	101.6	101
										3 d	82.0	81.6	82.3	112.9	112
										4 d	90.1	89.7	90.5	122.9	122
										5 d	95.7	95.3	96.2	130.2	129
										6 d	102.9	102.5	103.7	139.8	139
										7 d	108.3	107.8	108.8	146.6	145
RIETVLEI-AGR.	0513531_A	4.0	21	25	51	28	18	743	1510	1 d	61.4	61.1	61.7	84.8	84
										2 d	71.8	71.4	72.3	99.6	99
										3 d	78.5	78.2	78.9	108.2	107
										4 d	85.5	85.2	85.9	116.7	116
										5 d	92.6	92.2	93.1	126.0	125
										6 d	97.4	96.9	98.1	132.2	131
										7 d	101.7	101.3	102.2	137.7	137
PRETORIA-LYNWOOD	0513496_W	5.4	55	25	46	28	17	706	1360	1 d	57.4	57.1	57.6	79.2	78
										2 d	70.3	69.9	70.8	97.6	96
										3 d	79.1	78.7	79.4	109.0	108
										4 d	85.7	85.3	86.0	116.9	116
										5 d	92.5	92.1	93.0	125.8	125
										6 d	99.5	99.1	100.3	135.2	134
										7 d	105.1	104.7	105.6	142.3	141
PRETORIA-WATERKLOOF-CNT	RY 0513437_W	6.5	81	25	47	28	14	741	1470	1 d	62.3	62.0	62.6	86.0	85
										2 d	78.4	77.9	78.9	108.7	108
										3 d	86.5	86.1	86.9	119.2	118
										4 d	93.2	92.8	93.6	127.2	126
										5 d	100.1	99.7	100.7	136.2	135
										6 d	106.3	105.9	107.2	144.4	143
										7 d	110.7	110.2	111.1	149.8	149
PTA-UNIV-PROEFPLAAS.	0513465_A	7.4	50	25	45	28	16	687	1360	1 d	57.8	57.5	58.0	79.8	79
										2 d	70.1	69.6	70.6	97.2	96
										3 d	78.6	78.2	78.9	108.3	107
										4 d	86.1	85.7	86.4	117.4	116
										5 d	94.6	94.2	95.1	128.7	128
										6 d	101.4	101.0	102.2	137.7	137
										7 d	108.1	107.6	108.6	146.4	145
PRETORIA-BROOKLYN-3	0513405_W	7.6	31	25	46	28	14	692	1400	1 d	62.2	61.9	62.5	85.9	85
										2 d	75.6	75.1	76.1	104.9	104
										3 d	85.3	84.9	85.7	117.5	117
										4 d	93.4	93.0	93.8	127.5	126
										5 d	100.1	99.7	100.7	136.2	135
										6 d	104.7	104.2	105.5	142.2	141
										7 d	111.5	111.0	112.0	150.9	150

Figure 3.13: Design Rainfall estimation results

**Step 7c:** Based on the calculated time of concentration and representative rainfall, determine the point rainfall values (see Figure 3.14) for the catchment area.

The calculated precipitation depths are  $P_{120} = 57 \text{ mm}$  and  $P_{150} = 70 \text{ mm}$ .

**Step 8c:** 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.

$$P_{IT} = \frac{P_{IT}}{T_C} \quad \dots (3.9)$$

where:

$P_{IT}$  = point intensity for the different return periods (mm/h)

$P_{IT}$  = 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  $P_{120} = 42,6 \text{ mm/h}$  and  $P_{150} = 52,3 \text{ mm/h}$  respectively.

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Gridded values of all points within the specified block

Latitude Longitude MAP Altitude Duration Return Period (years)

2 2L 2U 5 5L 5U 10 10L 10U 20 2U

25 51 28 17 716 1520

5 m 9.7 8.0 11.3 13.3 11.1 15.6 16.1 13.4 18.9 19.1 15

10 m 14.4 12.1 16.6 19.8 16.7 23.0 24.0 20.2 27.8 28.4 23

15 m 18.1 15.4 20.8 25.0 21.3 28.8 30.2 25.7 34.8 35.8 30

30 m 22.9 19.6 26.3 31.7 27.1 36.3 38.3 32.8 43.8 45.3 38

45 m 26.3 22.6 30.1 36.4 31.2 41.5 44.0 37.8 50.2 52.0 44

1 h 29.0 25.0 33.1 40.1 34.5 45.7 48.5 41.8 55.3 57.4 49

1.5 h 33.3 28.8 37.9 46.0 39.8 52.3 55.7 48.1 63.3 65.9 56

2 h 36.8 31.9 41.7 50.8 44.0 57.6 61.4 53.2 69.6 72.7 62

4 h 43.4 38.1 48.7 59.9 52.6 67.3 72.5 63.6 81.4 85.8 75

6 h 47.9 42.3 53.4 66.1 58.4 73.7 79.9 70.6 89.1 94.5 83

8 h 51.3 45.6 56.9 70.8 62.9 78.6 85.6 76.1 95.1 101.3 89

10 h 54.1 48.3 59.9 74.7 66.7 82.7 90.3 80.6 100.0 106.8 95

12 h 56.5 50.6 62.4 78.0 69.9 86.2 94.3 84.5 104.1 111.6 99

16 h 60.5 54.5 66.5 83.6 75.3 91.9 101.0 91.0 111.1 119.6 107

20 h 63.8 57.7 70.0 88.1 79.7 96.6 106.6 96.4 116.8 126.1 113

24 h 66.7 60.5 72.9 92.1 83.6 100.7 111.3 101.0 121.7 131.8 119

1 d 55.5 50.3 60.6 76.6 69.5 83.7 92.6 84.0 101.2 109.6 99

3 d 68.3 63.5 73.1 94.3 87.7 101.0 114.1 106.0 122.1 135.0 125

4 d 77.2 72.7 81.7 106.6 100.4 112.8 128.9 121.4 136.3 152.5 143

5 d 84.1 78.6 89.6 116.1 108.6 123.8 140.4 131.2 149.6 166.2 154

6 d 89.9 83.5 96.3 124.1 115.4 133.0 150.1 139.4 160.8 177.7 164

7 d 95.0 87.7 102.2 131.1 121.2 141.1 158.5 146.5 170.6 187.6 180

25 52 28 17 702 1495

5 m 9.5 7.9 11.1 13.2 11.0 15.4 15.9 13.2 18.6 18.8 15

10 m 14.3 12.1 16.5 19.7 16.7 22.8 23.9 20.1 27.6 28.2 23

15 m 17.9 15.2 20.6 24.7 21.0 28.4 29.8 25.4 34.3 35.3 29

30 m 22.7 19.4 26.0 31.4 26.9 35.9 37.9 32.5 43.4 44.9 38

45 m 26.2 22.5 29.9 36.1 31.0 41.3 43.7 37.5 49.9 51.7 44

1 h 28.9 24.9 32.9 39.9 34.4 45.5 48.3 41.6 55.0 57.1 49

1.5 h 33.3 28.8 37.8 46.0 39.7 52.2 55.6 48.0 63.1 65.8 56

2 h 36.3 31.4 41.1 50.1 43.4 56.8 60.6 52.5 68.7 71.7 51

4 h 43.0 37.7 48.3 59.4 52.1 66.7 71.8 63.0 80.6 84.9 74

6 h 47.5 42.0 53.0 65.6 58.0 73.2 79.3 70.1 88.5 93.8 82

8 h 51.0 45.3 56.6 70.3 62.5 78.2 85.1 75.6 94.6 100.7 89

10 h 53.8 48.0 59.6 74.3 66.3 82.3 89.8 80.1 99.5 106.3 94

12 h 56.3 50.4 62.2 77.7 69.6 85.9 93.9 84.1 103.8 111.2 99

16 h 60.4 54.3 66.4 83.4 75.0 91.8 100.8 90.7 110.9 119.3 108

20 h 63.8 57.6 70.0 88.1 79.6 96.6 106.5 96.2 116.8 126.0 117

24 h 65.3 59.2 71.4 90.2 81.7 98.7 109.0 98.8 119.3 129.0 116

1 d 54.3 49.2 59.4 75.0 68.0 82.0 90.7 82.2 99.2 107.3 96

2 d 67.8 62.9 72.6 93.6 87.0 100.3 113.2 105.1 121.2 134.0 124

3 d 75.6 71.2 79.9 104.3 98.3 110.4 126.1 118.8 133.4 149.3 132

4 d 82.9 77.5 88.4 114.5 107.0 122.0 138.4 129.3 147.5 163.9 150

5 d 89.2 82.7 95.5 123.1 114.3 131.9 148.8 138.1 159.5 176.2 163

6 d 94.6 87.3 101.8 130.6 120.6 140.6 157.9 145.8 170.0 186.9 172

7 d 97.1 89.2 104.9 134.0 123.2 144.9 162.0 148.9 175.1 191.8 175

25 52 28 18 712 1520

5 m 9.6 8.0 11.2 13.2 11.0 15.5 16.0 13.3 18.7 18.9 15

10 m 14.2 12.0 16.4 19.6 16.5 22.7 23.7 20.0 27.4 28.1 23

15 m 18.0 15.3 20.7 24.8 21.1 28.5 30.0 25.5 34.5 35.5 30

30 m 22.7 19.4 26.0 31.4 26.8 35.9 37.9 32.4 43.4 44.9 38

Figure 3.14: Gridded point rainfall values

Now that the intensities have been calculated using three alternative methods the run-off coefficient can now be determined.

**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 3.3** provides recommended values of C for the calculation of the run-off coefficient.

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 3.3**) i.e.  $\alpha = 0,4$ ;  $\beta = 0,6$  and  $\gamma = 0,0$ .

Based on the available data from the catchment, the following tables were compiled to characterise the catchment (**Table 3.4** and **Table 3.5**).

Rural (C <sub>1</sub> )					Urban (C <sub>2</sub> )	
Component	Classification	Mean annual rainfall (mm)			Use	Factor
		< 600	600 - 900	> 900		
Surface slope (C <sub>s</sub> )	Vleis and pans (<3%)	0,01	0,03	0,05	<i>Lawns</i> - Sandy, flat (<2%) - Sandy, steep (>7%) - Heavy soil, flat (<2%) - Heavy soil, steep (>7%)	0,05 - 0,10 0,15 - 0,20 0,13 - 0,17 0,25 - 0,35
	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 <sub>p</sub> )	Very permeable	0,03	0,04	0,05	<i>Residential areas</i> - Houses - Flats	0,30 - 0,50 0,50 - 0,70
	Permeable	0,06	0,08	0,10		
	Semi-permeable	0,12	0,16	0,20	<i>Industry</i> - Light industry - Heavy industry	0,50 - 0,80 0,60 - 0,90
	Impermeable	0,21	0,26	0,30		
Vegetation (C <sub>v</sub> )	Thick bush and plantation	0,03	0,04	0,05	<i>Business</i> - City centre - Suburban - Streets - Maximum flood	0,70 - 0,95 0,50 - 0,70 0,70 - 0,95 1,00
	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		

Rural (C <sub>1</sub> )		
Component	Classification	%
Surface slope (C <sub>S</sub> )	Vleis and pans (<3%)	20
	Flat areas (3 to 10%)	70
	Hilly (10 to 30%)	10
	Steep areas (>30%)	0
Permeability (C <sub>P</sub> )	Very permeable	0
	Permeable	50
	Semi-permeable	50
	Impermeable	0
Vegetation (C <sub>V</sub> )	Thick bush and plantation	0
	Light bush and farm lands	45
	Grasslands	50
	No vegetation	5

$$C_l = C_s + C_p + C_v \quad \dots (3.10)$$

$C_1$	=	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

3-19

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

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 ( $D_{\text{factor}}$ ) 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

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

$C_{1D}$  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. 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  $F_{50} = 0,83$ .

$$C_{1T} = C_{1D} \times F_T \quad \dots (3.11)$$

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 3.5**) and the most conservative coefficients of **Table 3.5** (for this example).

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

The combined run-off coefficient is calculated as follows:

$$C_T = \alpha C_{1T} + \beta C_2 + \gamma C_3 \quad \dots (3.12)$$

With  $\alpha = 0,4$ ;  $\beta = 0,6$  and  $\gamma = 0,0$ .

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

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

**Table 3.5: Catchment characteristics (Urban)**

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

**Step 11:** Determine the peak flow for each of the required return periods utilising the simple linear relationship and for this example the various methods used to calculate the average rainfall intensity (Alternatives 1 to 3):

$$Q_T = \frac{C_T I_T A}{3,6} \quad \dots (3.13)$$

where:

Q <sub>T</sub>	=	peak flow rate for T-year return period (m <sup>3</sup> /s)
C <sub>T</sub>	=	combined run-off coefficient for T-year return period
I <sub>T</sub>	=	average rainfall intensity over catchment for a specific return period (mm/hour)
A	=	effective area of catchment (km <sup>2</sup> )
3,6	=	conversion factor

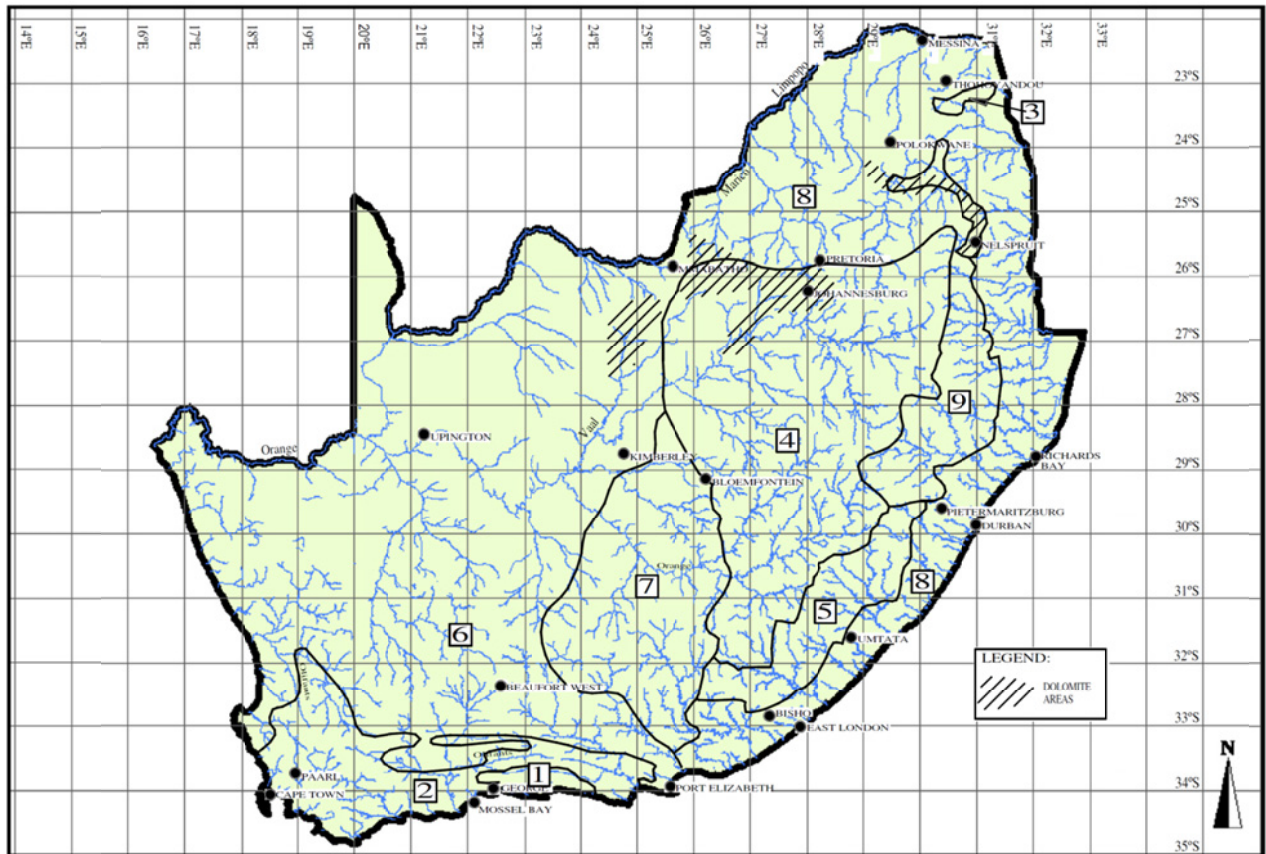
The peak flow rates based on the rational method for the 1:20 year and 1:50 year return periods (calculated by means of equation 3.13) are:

Peak flood	Alternative 1	Alternative 2	Alternative 3
Q <sub>20</sub>	164 m <sup>3</sup> /s	184 m <sup>3</sup> /s	128 m <sup>3</sup> /s
Q <sub>50</sub>	224 m <sup>3</sup> /s	243 m <sup>3</sup> /s	166 m <sup>3</sup> /s

### 3.1.2 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<sup>2</sup>, 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 3.15**. The catchment of the Moretele Spruit falls in *Zone 8*.



**Figure 3.15: Regions with generalised veld types in South Africa**

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

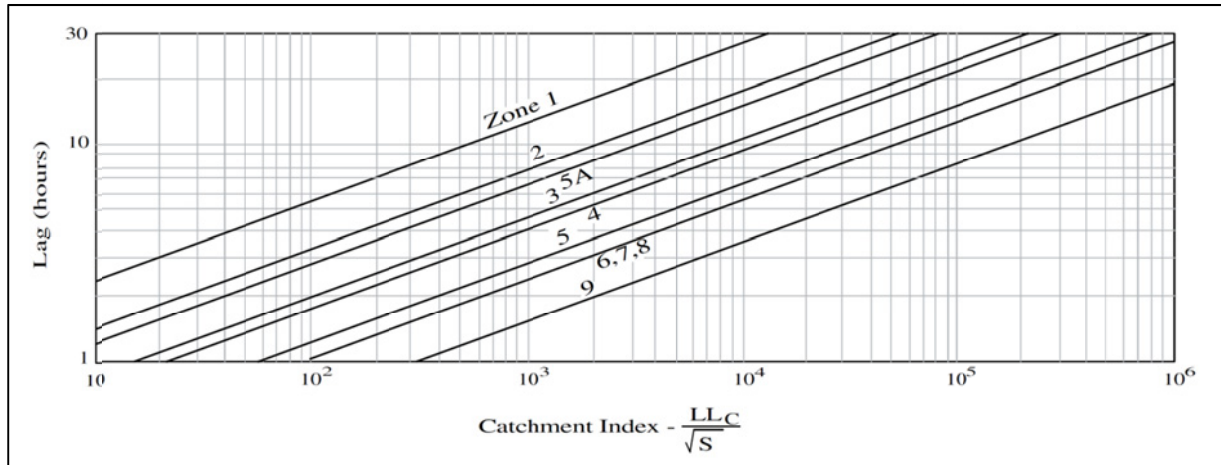
$$\text{Index} = \frac{L L_c}{\sqrt{S}} \quad \dots (3.14)$$

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  $L_c = 4,65 \text{ km}$ . The calculated catchment index is  $230,1$ .

**Step 4:** Determine the lag time in hours from **Figure 3.16** based on the catchment index and veld-type zone. The Lag time ( $T_L$ ) equals  $1,35 \text{ hours}$ .





**Figure 3.16: Ratio of lag time to catchment index**

**Step 5:** From **Table 3.6** obtain the value of  $K_U$  for the specific veld-type. For this example  $K_U = 0,367$ .

**Table 3.6: Values of  $K_U$  for various veld types**

Regional number (Figure 3.15)	Generalised veld type	Factor $K_u$
1	Coastal tropical forest	0,261
2	Schlerophyllous bush	0,306
3	Mountain sourveld	0,277
4	Grasslands of interior plateau	0,386
5	Highland sourveld and Dohne sourveld	0,351
5a	As for Zone 5 – but soils weakly developed	0,488
6	Karoo	0,265
7	False Karoo	0,315
8	Bushveld	0,367
9	Tall sourveld	0,321

**Step 6:** The peak flow rate for the unit hydrograph according to the regional classification given in **Table 3.6** in zone 8 is calculated using the following formula:

$$Q_p = K_u \frac{A}{T_L} \quad \dots (3.15)$$

where:

$Q_p$	=	peak flow rate of unit hydrograph ( $m^3/s$ )
$A$	=	size of catchment ( $km^2$ )
$T_L$	=	Lag time (hours)

The unit hydrograph peak discharge is  $7,75 m^3/s$ .

**Step 7:** Obtain the mean annual precipitation (MAP), as described for the Rational method. The determined MAP 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 ( $he_T$ ) 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 precipitation (MAP), the rainfall region, and the storm duration ( $T_{SD}$ ). Point precipitation for various durations, normally shorter than or equal to the lag time, is obtained. **Figure 3.5** may be used to determine the point rainfall although it is preferred that the Design Rainfall software utility is used (see **Figure 3.12**). The probable maximum flood can also be calculated using the Unit hydrograph method, see the Drainage Manual Chapter 3 for more details.

For this example the point rainfalls for the 0,25 hour, 0,5 hour, 1 hour and 2 hour storms have been determined for the different return periods 1:20 and 1:50 year (see **Table 3.7**).

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

$$P_{iT} = \frac{P_T}{T_{SD}} \quad \dots (3.16)$$

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 3.7**.

**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 **Figure 3.8**. The determined  $ARF_{iT}$  values are shown in **Table 3.7**.

**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 3.7**.

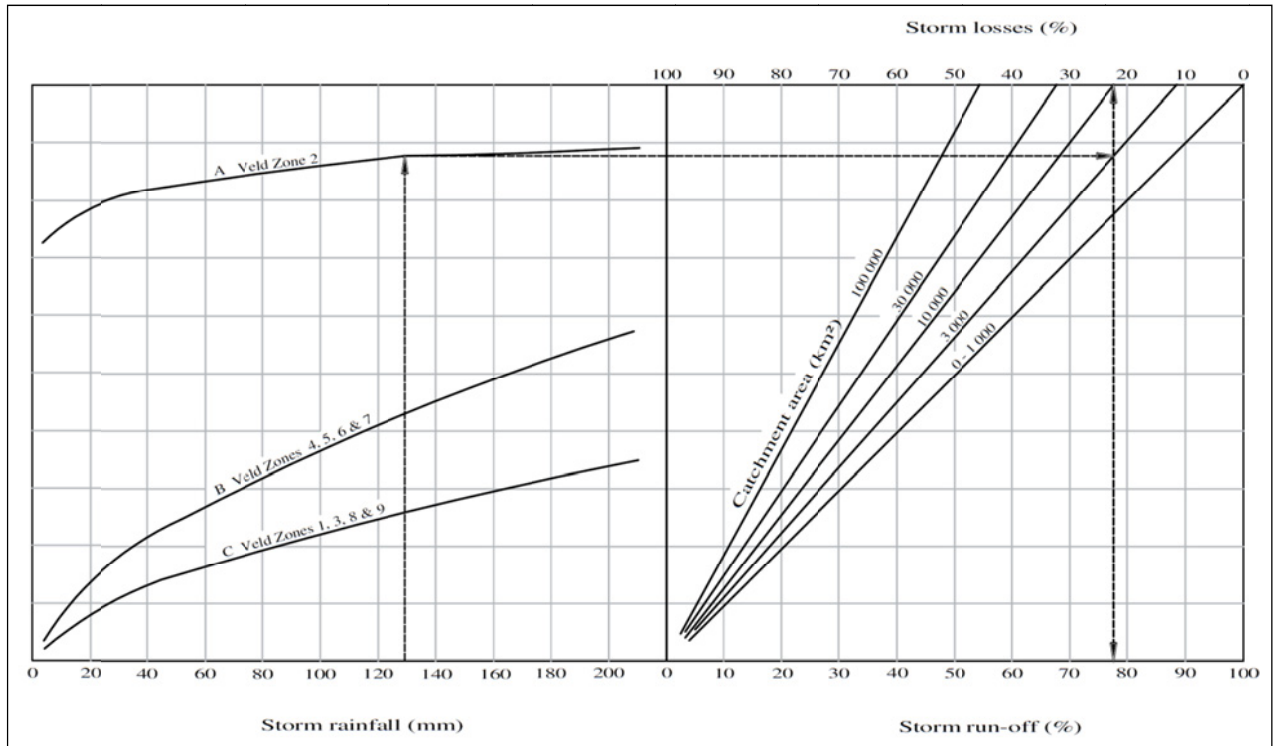
**Step 8.5:** Determine the flood run-off factor from **Figure 3.17**. This factor is based on the average rainfall, veld-type zone and catchment area. The flood run-off factors ( $f_{iT}$ ) are given in **Table 3.7**.

**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 3.7: 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



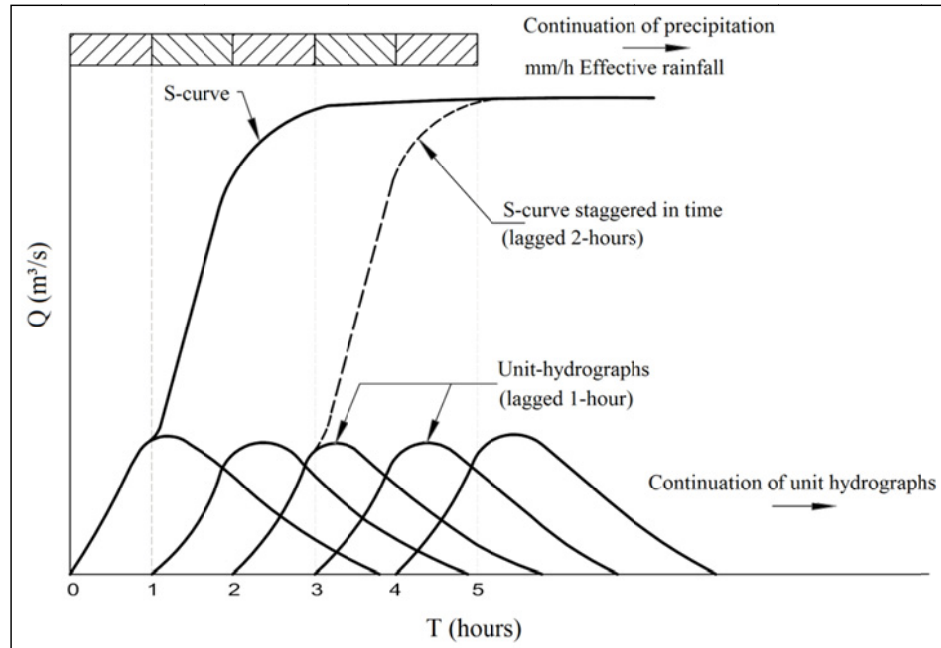


**Figure 3.17: Average storm losses**

**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.

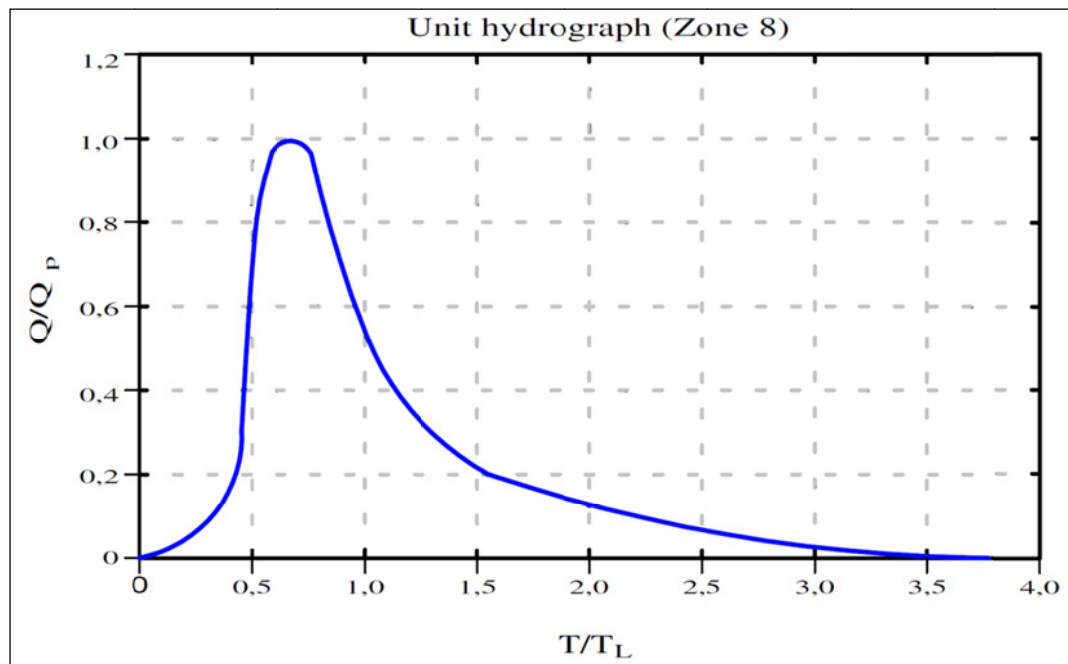
An S-curve is obtained by staggering a number of unit hydrographs by the unit duration and then summing them as shown **Figure 3.18**. It is recommended that an S-curve be constructed in all cases.

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.



**Figure 3.18: Illustration of S-curve**

The dimensionless one-hour unit hydrograph for veld-type zone 8 is shown in **Figure 3.19**, obtained from **Table 3.8**, and the constructed S-curve in **Figure 3.20**. 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 3.20**. 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.



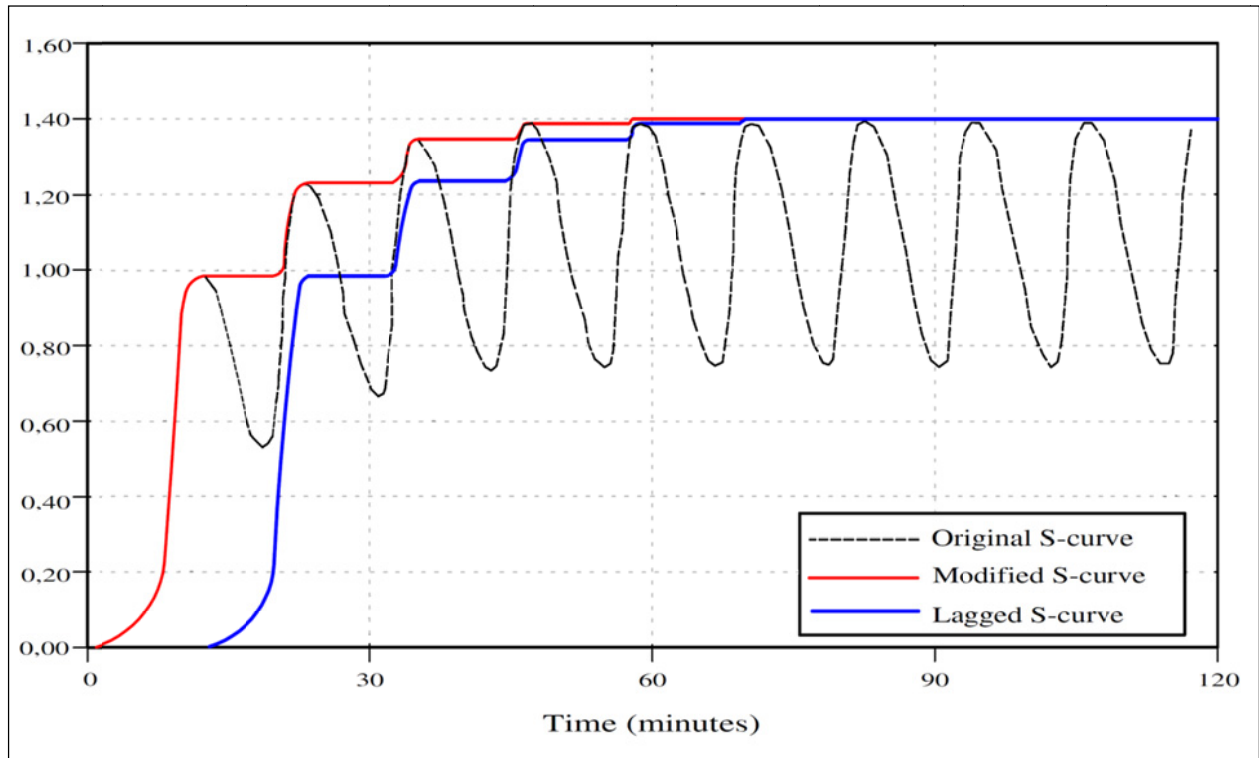
**Figure 3.19: Dimensionless one-hour unit hydrograph for veld type Zone 8**

**Table 3.8: Dimensionless one hour unit hydrographs for various veld zone regions**

Time as T/T <sub>L</sub>	Run-off as Q/Q <sub>P</sub> 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

**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 3.9**.

**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 3.9**. The peak values are adjusted as indicated for  $Q_{pIT}/Q_p < 1$ ; in this case 0,9923.



**Figure 3.20: S-curve for veld type Zone 8 and lagged by 0,25 hour**

**Table 3.9: Peak flows utilising the discharge unit hydrograph method**

Return period		1:20				1:50			
Variable	Unit								
Storm 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 ( $Q_{pIT}$ )	m <sup>3</sup> /s	24,52	14,48	7,68	4,77	24,52	14,48	7,68	4,77
Peak flow ( $Q_{IT}$ )	m <sup>3</sup> /s	101,34	128,42	112,73	96,64	127,50	170,56	155,05	126,91
Adjusted for $Q_{pIT}/Q_p < 1$	m <sup>3</sup> /s	102,12	<b>129,42</b>	113,60	97,39	128,49	<b>171,88</b>	156,26	127,89

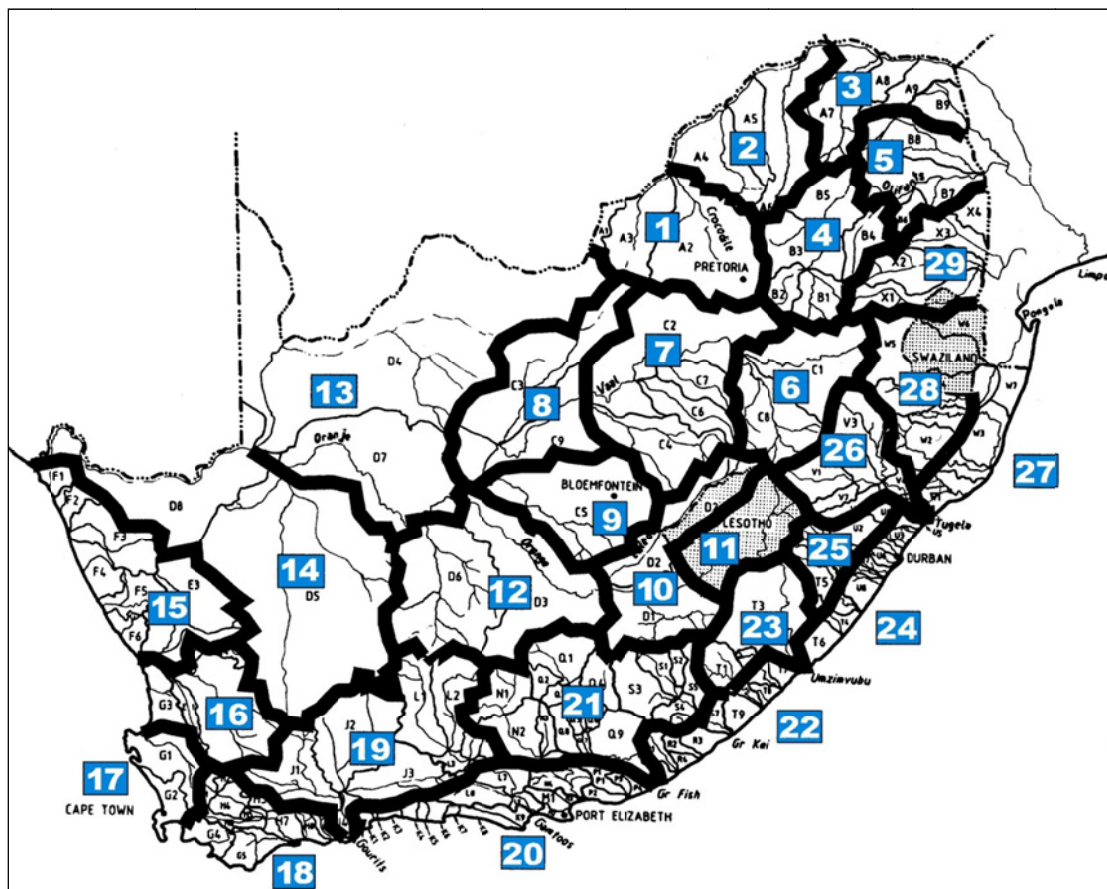
The peak flow rates, based on the unit hydrograph method, for the 1:20 year and 1:50 year return periods are:

$$Q_{20} = 129 \text{ m}^3/\text{s} \text{ and } Q_{50} = 172 \text{ m}^3/\text{s}$$

### 3.1.3 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 3.21**. 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  $A = 28,5 \text{ km}^2$ .
- Step 3:** Determine the length of the main channel. In this example it has already been calculated as  $L = 7,25 \text{ km}$ .
- Step 4:** Determine the average slope of the catchment as described for the Rational method. The calculated average slope (S) for this example is  $0,02146 \text{ m/m}$ .



**Figure 3.21: Standard Design Flood drainage basins**

- Step 5:** Apply the US Soil Conservation Service formula to determine the time of concentration  $T_C$  (hours) as suggested in HRU1/72<sup>(3.23)</sup>.

$$T_C = \left[ \frac{0,87L^2}{1000S_{av}} \right]^{0,385} \quad \dots (3.17)$$

The time of concentration is as calculated earlier, i.e.  $T_C = 1,338 \text{ hours}$ .

**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.

$$P_{tT} = 1,13(0,41 + 0,64\ln T)(-0,11 + 0,27\ln t)(0,79M^{0,69}R^{0,20}) \quad \dots (3.18)$$

where:

- $M$  = mean of the annual daily maxima from **Table 3B.1** (in **Appendix B**) equals *56 mm*.  
 $R$  = average number of days per year on which thunder was heard from **Table 3B.1** equals *30*.

**Table 3.10: Calculating point precipitation**

Return period	1:20	1:50
$P_{tT}$ (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.

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

**Table 3.11: 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 3.12**, using the equation below:

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

**Table 3.12: 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

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:

$$C_{20} = 0,3112 \text{ and } C_{50} = 0,3639$$

**Step 9:** Finally, the flood peak  $Q_T$  ( $m^3/s$ ) for the required return period is calculated as follows:

$$Q_T = \frac{C_T I_T A}{3,6} \quad \dots (3.21)$$

**Table 3.13: Calculating flood peaks**

Return period	1:20	1:50
$C_T$	0,3112	0,3639
$I_T$ (mm/hour)	50,83	63,63
$Q_T$ (m <sup>3</sup> /s)	<i>125</i>	<i>183</i>

**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.

### 3.1.4 SCS method

The SCS method requires a minimum input of catchment area, the catchment response time (Time of Concentration), design rainfalls and soils and land cover classification. The catchment can be divided into sub-areas or Hydrological Response Units (HRUs) which are areas with similar soils and land cover and hence have a relatively similar hydrological response to rainfall.

#### Step 1: Catchment information

The following information is provided or has been calculated above:

**Table 3.14: Previously determined information**

Catchment name	Moretele Spruit	
Latitude	25° 47,75'	
Longitude	28° 17,95'	
Total catchment area - A	28,5	(km <sup>2</sup> )
Catchment time of concentration - $T_c$	1,334	(h)

The catchment is 40% rural and 60 % urban.

#### Step 2: Initial Curve Number determination

From the ISCW images of soil Land Types, the following Land Types are found in the catchment: Ib7, Ba3 and Ba9. For each of the Land Types, the dominant terrain unit was assumed to represent the entire HRU. Similarly, the dominant land class and soil series within a terrain unit was assumed to represent the entire unit and, where more than one soil series is grouped to form the land class, the median SCS soil classification was used. For each soil series representing each HRU, the appropriate SCS soil classification was determined using tables from the *Visual SCS-SA* User Manual. The classification determined as described above is summarised in **Table 3.15**, with the catchment divided into four HRUs. For each HRU in **Table 3.15** the soil and land cover classification is used to determine the initial  $CN$  using information contained in **Table 3E.3** (in **Appendix 3E**). From the initial  $CN$ , values of  $S$  can be determined using Equation 3.22.

$$S = \frac{25400}{CN} - 254 \quad \dots(3.22)$$

**Table 3.15: Determination of initial CN and S**

HRU	Area (%) $A_i$	SOIL				LAND COVER				Initial CN	S (mm)
		Form	Series	Texture	SCS Group	Cover	Cover Class (S/I/D)	Practice/Treatment	Stormflow potential		
1	30	Glenros	Trevanian	SaCILm	B/C	Urban	I	30% imp		77	75,9
2	30	Glenros	Trevanian	SaCILm	B/C	Urban	I	25% imp		75	84,7
3	20	Hutton	Msinga	SaCILm	A	Veld	I		Moderate	49	264,
4	20	Glenros	Trevanian	SaCILm	B/C	Veld	I		Moderate	75	84,7

**Step 3: Lag estimation**

The catchment lag is computed from the time of concentration for the catchment using Equation 3.23.

$$L = (0,6)(1,334) = 0,8 \text{ hours} \quad \dots (3.23)$$

**Step 4: Areal reduction factor (ARF)**

Given that the SCS method is primarily intended to estimate peak discharge from small catchments, over which a uniform daily rainfall may be assumed, the reduction of point to catchment rainfall by means of an ARF is generally not applied. However, to be consistent with the example, an  $ARF_{20} = 94\%$  and  $ARF_{50} = 91\%$  for the 20 and 50 year return periods respectively is used.

**Step 5: One-day design rainfall estimation**

The SCS method requires the input of one-day design rainfalls which can be estimated using a number of different methods. The *Visual SCS-SA* software includes the following options:

- (i) User estimated design rainfalls computed directly from raingauge data.
- (ii) Design rainfall extracted for a selected station from Adamson's TR102 report.
- (iii) Using at-site design rainfall calculated from the representative rainfall stations used to represent the rainfall in each of the 712 hydrological zones.
- (iv) Design rainfall estimated using the regional method developed by Smithers and Schulze (2003).

Given the regional approach used and the longer periods of record from all available stations used in the analysis compared to all previous studies, it is recommended that the regional, scale invariance approach to design rainfall estimation developed for South Africa by Smithers and Schulze (2003) be used to estimate the one-day design rainfall.

The results from the application of this approach at the location of the three rain gauges located in and close to the catchment are summarised in **Table 3.16**. Given the relatively small differences in design rainfall between the values computed from the at-site data and the RLMA&SI methodology, as well as between the different locations, the RLMA&SI values for Garsfontein were used in the estimation of the design floods. These were then reduced by the areal reduction factors of 94 % and 91 % for the 20 and 50 year return period events respectively in order to determine the catchment design rainfalls.



**Table 3.16: Estimation of one-day design rainfalls at stations around the Moretele Spruit catchment used in this example**

Location	Return period (years)			
	20		50	
	At-site (mm)	RLMA&SI (mm)	At-site (mm)	RLMA&SI (mm)
0513529 – GARSFONTEIN Latitude: 25° 49' Longitude: 28° 17'	110,5	118,6	135,4	145,7
0513531 – RIETVLEI Latitude: 25° 51' Longitude: 28° 18'	121,4	114,6	148,8	140,5
0513528 - CONSTANTIA PARK Latitude: 25° 48' Longitude: 28° 18'	*	120,4	*	147,7

\* No at-site values available

**Step 6: Catchment one-day design rainfall**

The ARF is used to calculate the catchment design rainfall, as shown in **Table 3.17**.

**Table 3.17: Calculation of one-day catchment design rainfall**

Return period (years)	20	50
Areal reduction factor - ARF (%)	94	91
Design daily rainfall depth - P <sub>D</sub> (mm)	118,6	145,7
Catchment design rainfall	111,5	132,6

**Step 7: Design stormflow depth estimation**

For each sub-area or HRU which the catchment has been divided into, the stormflow is calculated using Equation 3.24, as shown in **Table 3.18**. For example, the computation for the 20 year return period stormflow from HRU 1 is:

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \quad \text{for } P > I_a \quad \dots(3.24)$$

where

- Q = stormflow depth (mm),
- P = daily rainfall depth (mm), usually input as a one-day design rainfall for a given return period,
- S = potential maximum soil water retention (mm),
- ≡ index of the wetness of the catchment's soil prior to a rainfall event,
- I<sub>a</sub> = initial losses (abstractions) prior to the commencement of stormflow, comprising of depression storage, interception and initial infiltration (mm)
- = 0,1S (recommended for use in South Africa)

$$Q_{20} = \frac{[111,5 - (0,1 \times 75,9)]^2}{[111,5 - (0,1 \times 75,9) + 75,9]} = 60,0 \text{ mm}$$

**Table 3.18: Calculation of stormflow depth (mm)**

Return period (years)	20	50
Sub-Area	Design stormflow depth (mm)	
HRU 1	60,0	77,8
HRU 2	56,5	73,8
HRU 3	20,7	30,4
HRU 4	56,5	73,8

**Step 8: Total stormflow depth**

The stormflow from each sub-area is area weighted to calculate an equivalent stormflow depth from the entire catchment using Equation 3.25 with the results shown in **Table 3.19**.

$$Q = \sum_{i=1}^N Q_i A_i \quad \dots (3.25)$$

where

- Q = average stormflow depth from entire catchment (mm)
- Q<sub>i</sub> = stormflow depth (mm) from i<sup>th</sup> sub-area calculated using Equation 3.24
- A<sub>i</sub> = fraction of sub-area of total catchment area for i<sup>th</sup> sub-area
- N = number of sub-areas

**Table 3.19: Calculation of total stormflow depth**

Return period (years)	20	50
Total runoff depth (mm)	50,4	66,3

**Step 9: Total runoff volume**

The stormflow volume from the catchment is calculated using Equation 3.26 and is given in **Table 3.20**.

$$V = \frac{QA}{1000} \quad \dots (3.26)$$

where

- V = stormflow volume (m<sup>3</sup>x10<sup>6</sup>)
- Q = stormflow depth (mm)
- A = catchment area (km<sup>2</sup>)

**Table 3.20: Calculation of total stormflow volume**

Return period (years)	20	50
Total runoff volume (m <sup>3</sup> x10 <sup>6</sup> )	1,4	1,9

**Step 10: Peak discharge estimation**

The peak discharge from the catchment is calculated using Equation 3.27.

$$q_p = \frac{0,2083AQ}{1,83L} \quad \dots (3.27)$$

For the 20 year return period, the peak discharge is calculated as:

$$q_p = \frac{(0,2083)(28,5)(50,4)}{(1,83)(0,8)} = 204,5 \text{ m}^3/\text{s}$$

## Results

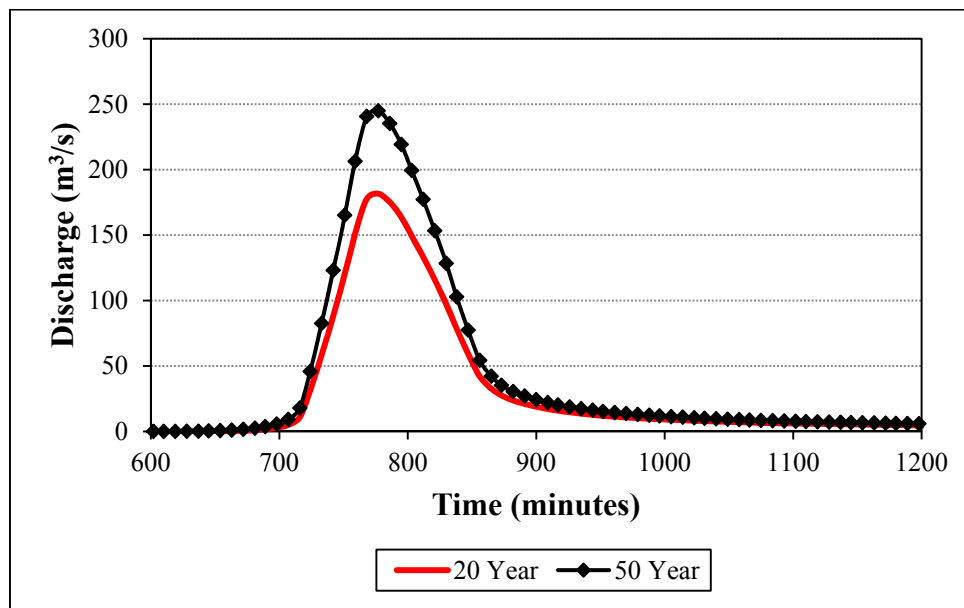
The computed peak discharge for the 20 and 50 year return periods are summarised in **Table 3.21** for both the manual calculation method, as summarised above, and computed by the *Visual SCS-SA* software. The differences in the calculated peak discharge by the two methods are a result of:

- (i) No adjustment of the initial *CNs* into final *CNs* is included in the manual method. In the *Visual SCS-SA* method, the median condition method was used to adjust the initial *CNs* into final *CNs*, thus accounting for typical soil moisture conditions.
- (ii) The manual method does not use the incremental unit hydrograph approach and does not take into account the regionalised typical rainfall hyetographs, both of which are used in the *Visual SCS-SA* software.

**Table 3.21: Computed peak discharge**

Return period (years)	20	50
Peak discharge (m <sup>3</sup> /s): Manual method	204,5	268,9
Peak discharge (m <sup>3</sup> /s): <i>Visual SCS-SA</i>	181,7	245,0

A standard calculation sheet is included in **Appendix 3C** for the manual method. The output from the *Visual SCS-SA* software is shown in **Figure 3.22** and **Figure 3.23**. The manual method outlined in this guide is a simplified version of the adaptations to the SCS method for South Africa. The full adaptations can be performed manually using the SCS-based design runoff user manual developed by Schmidt and Schulze (1987b). The full adaptations are incorporated into the *Visual SCS-SA* software, including the estimation of design rainfall using the RLMA&SI developed by Smithers and Schulze (2003).



**Figure 3.22: Hydrographs generated by Visual SCS-SA**

CATCHMENT NAME	:	Garsfontein
PROJECT NO	:	Example
RUN NO	:	1
TOTAL CATCHMENT AREA (km <sup>2</sup> )	:	28.50
STORM INTENSITY DISTRIBUTION TYPE	:	3
CATCHMENT LAG TIME (h)	:	0,80
COEFFICIENT OF INITIAL ABSTRACTION	:	0,10
CURVE NUMBERS:		
	Initial	Final
Sub-catchment 1	77	72,1
Sub-catchment 2	75	70,4
Sub-catchment 3	49	49,0
Sub-catchment 4	75	70,4
RETURN PERIOD (YEARS)	20	50
DESIGN DAILY RAINFALL DEPTH (mm)	111	132
DESIGN STORMFLOW DEPTH (mm)		
Sub-catchment 1	51,4	67,8
Sub-catchment 2	48,6	64,5
Sub-catchment 3	20,5	30,1
Sub-catchment 4	48,6	64,5
TOTAL RUNOFF DEPTH (mm)	43,8	58,6
DESIGN STORMFLOW VOLUME (thousands m <sup>3</sup> )		
Sub-catchment 1	439,3	579,5
Sub-catchment 2	415,2	551,4
Sub-catchment 3	116,8	171,7
Sub-catchment 4	276,8	367,6
TOTAL STORMFLOW VOLUME (millions m <sup>3</sup> )	1,2	1,7
COMPUTED CURVE NUMBER	67,3	67,1
PEAK DISCHARGE (m <sup>3</sup> /s)	181,7	245,0

**Figure 3.23: Output from Visual SCS-SA**

### 3.1.5 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. The formula reads:

$$Q_T = 0,0377 K_T P A^{0,6} C^{0,2} \quad \dots (3.28)$$

where:

$Q_T$  = peak flow for T return period (m<sup>3</sup>/s)

$K_T$  = coefficient based on veld-type region (see **Figure 3.24** and **Table 3.22**).

P = mean annual precipitation over catchment (mm/a) (see **Figure 3.5** or utilise Design Rainfall software for the catchment (see **Figure 3.11**).

$$\text{and } C = \frac{A \sqrt{S}}{L L_C} \quad (\text{Catchment parameter with regard to reaction time}) \quad \dots (3.29)$$

where:

- A = area of catchment (km<sup>2</sup>)  
S = average slope of stream (m/m)  
L = hydraulic length of catchment (km)  
L<sub>C</sub> = distance between outlet and centre of gravity of catchment (km)

**Table 3.22: Constant values of K<sub>T</sub> for different veld types**

Return period T in years				10	20	50	100
Zone number and Generalized Veld type (Figure 3.15)	1		Coastal tropical forest	0,17	0,23	0,32	0,40
	2	Winter	Schlerophyllous bush	0,42	0,52	0,68	0,80
		All year		0,83	1,04	1,36	1,60
	3		Mountain sourveld	0,29	0,40	0,55	0,70
	4		Grasslands of interior plateau	0,59	0,68	0,95	1,20
	5		Highland sourveld and Dohne sourveld	0,59	0,80	1,11	1,40
	5A		As for Zone 5 – but soils weakly developed	0,59	0,68	0,95	1,20
	6	Winter	Karoo	0,33	0,45	0,63	0,80
		All year		0,67	0,91	1,26	1,60
	7		False Karoo	0,67	0,91	1,26	1,60
	8		Bushveld	0,42	0,57	0,79	1,00
	9		Tall sourveld	0,50	0,68	0,95	1,20

With the catchment being in veld-type zone 8, the K<sub>T</sub> 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 m<sup>3</sup>/s and 109,28 m<sup>3</sup>/s respectively.

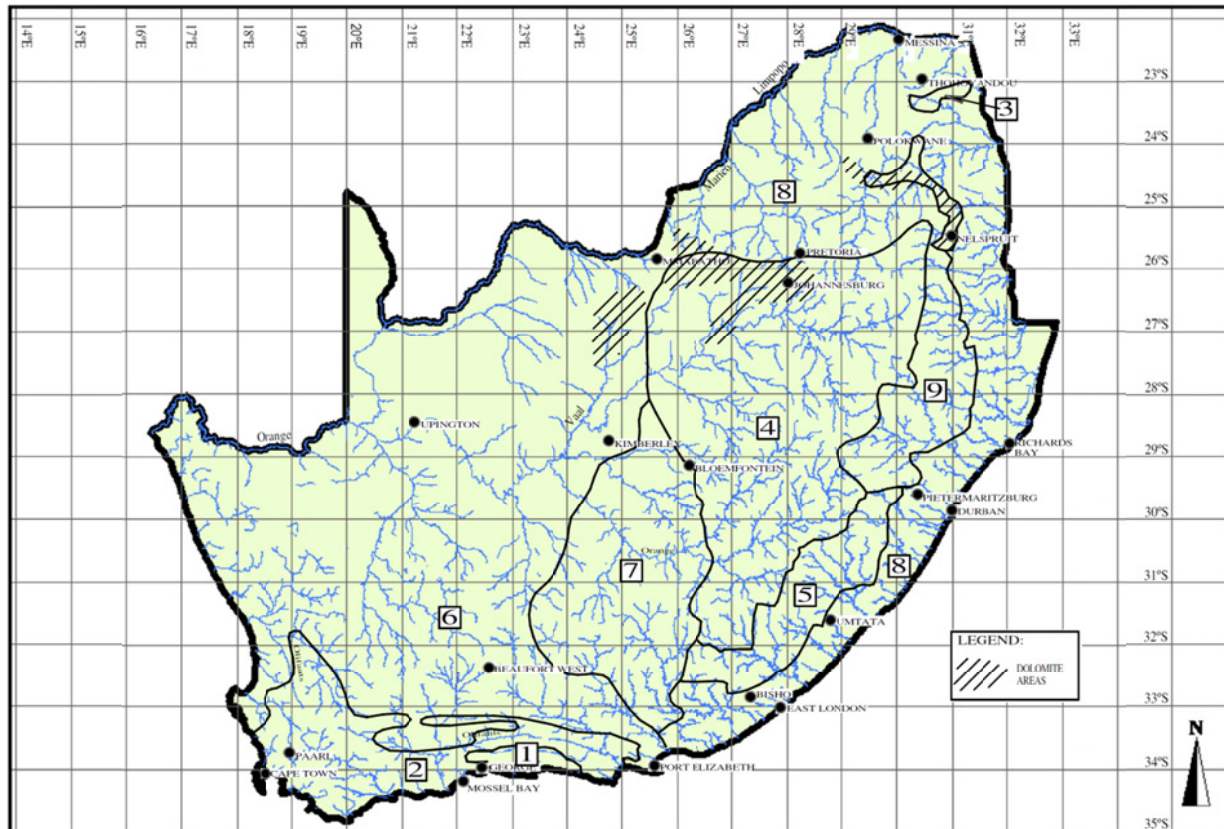


Figure 3.24: Regions with generalised veld types in South Africa

### 3.1.6 Comparison of solutions

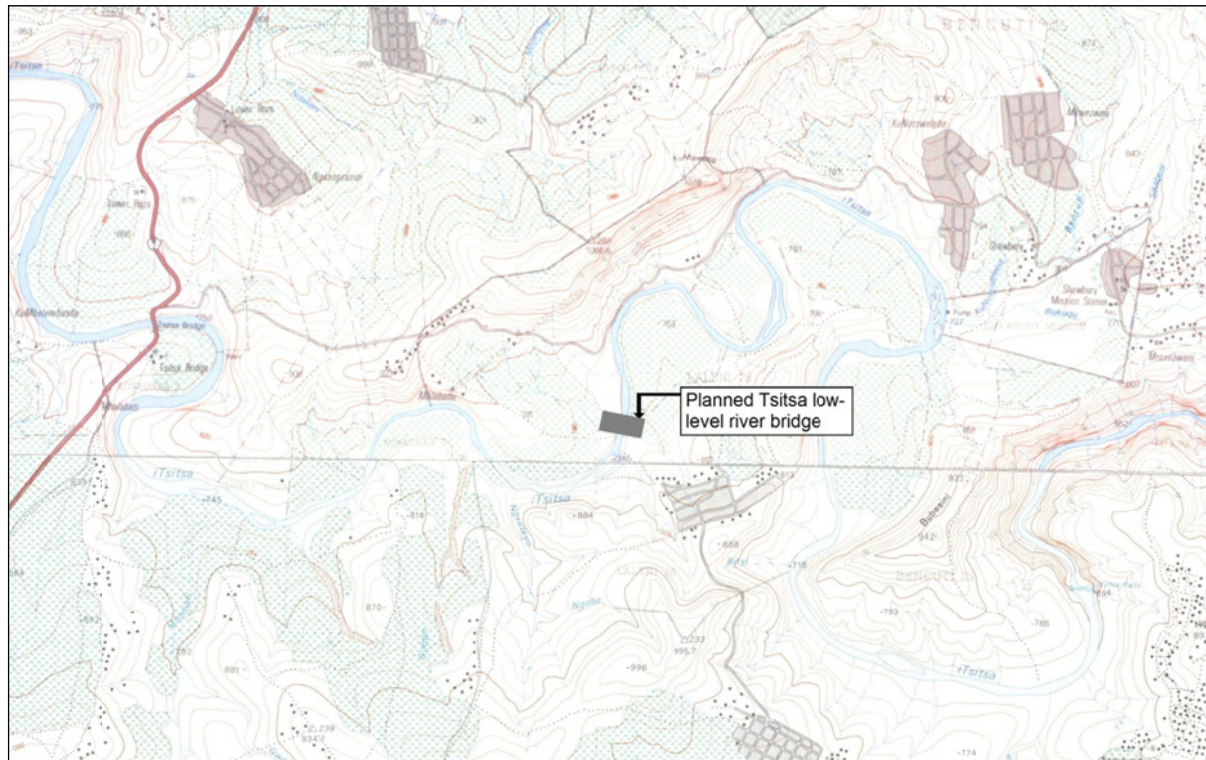
Table 3.23 summarises the results for the five methods used in this example.

Table 3.23: Comparison of solutions

Method	Return period	
	1:20	1:50
Rational (Alternative 1)	164	224
Rational (Alternative 2)	184	243
Rational (Alternative 3)	128	166
Unit Hydrograph	129	172
SDF	125	183
SCS-SA	205	269
Empirical	79	109

### 3.2 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 3.25**). The position of the proposed Tsitsa low-level river bridge is shown on **Figure 3.25**.



**Figure 3.25: Proposed Tsitsa low-level river bridge position**

The catchment characteristics were determined (see **Table 3.24**), and are used in the calculation of the flood peaks for various recurrence intervals.

**Table 3.24: Catchment characteristics**

Description of characteristic	Determined value	Comment
Catchment area (see <b>Figure 3.25</b> )	4318 km <sup>2</sup>	Catchment area may be clearly defined
Length of longest watercourse (see <b>Figure 3.27</b> )	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 <b>Figure 3.28</b> )	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
RMF K-factor	5,0 – 5,2	Catchment area falls within regions K5 and K6 (assume highest value). (See <b>Figure 3.26</b> )



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 3.24)	Zone 2	
Gauging station (see Figure 3.29)	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

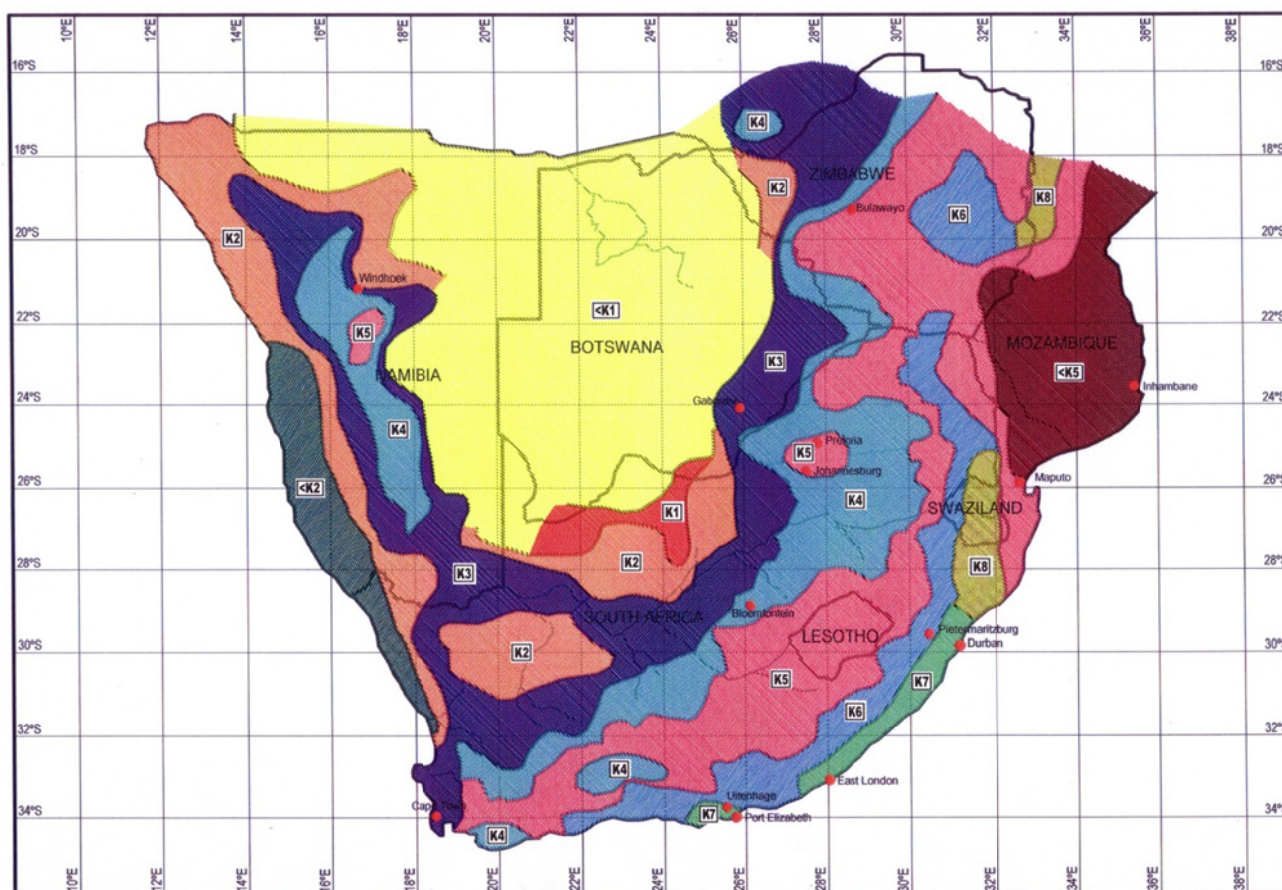


Figure 3.26: Maximum flood peak regions in southern Africa from Kovács



Figure 3.27: Catchment area and longest watercourse (3.21)

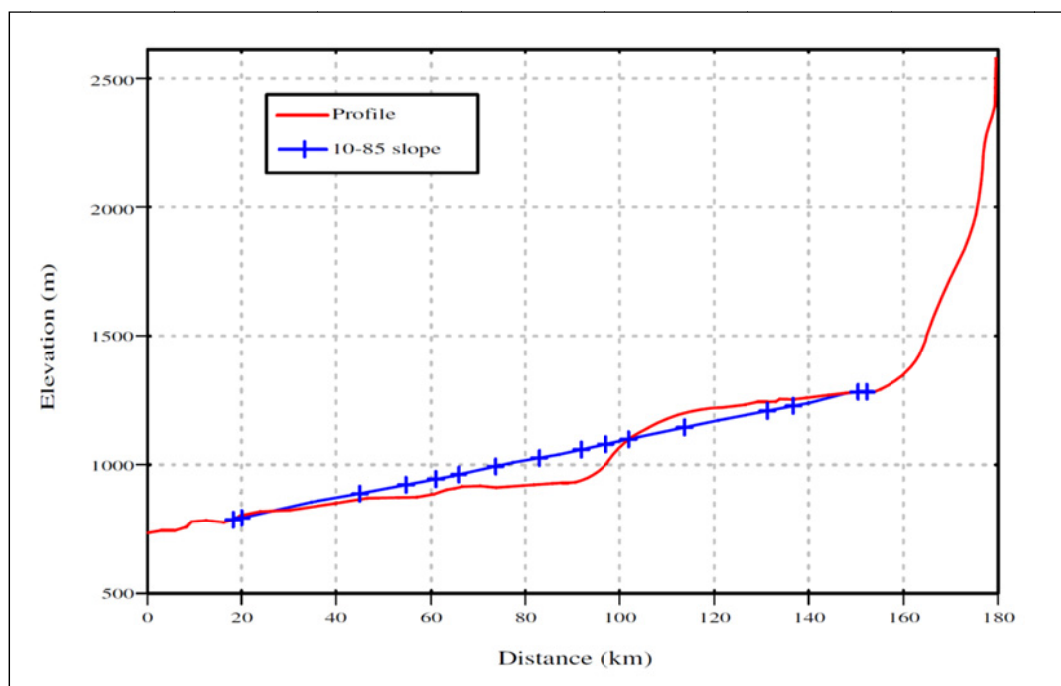


Figure 3.28: Watercourse profile and average slope



**Figure 3.29: Gauging station (T3H016 Tsitsa River @ Xonkonxa)**

### **3.2.1 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. 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 m<sup>3</sup>/s, was exceeded on at least three occasions. The data as used in the statistical analysis are shown in **Table 3.25**.

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 m<sup>3</sup>/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, a peak flow of 1 699 m<sup>3</sup>/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 m<sup>3</sup>/s), and it was estimated as 1 364 m<sup>3</sup>/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 m<sup>3</sup>/s.



**Table 3.25: Historical annual maximum flood peaks for T3H016**

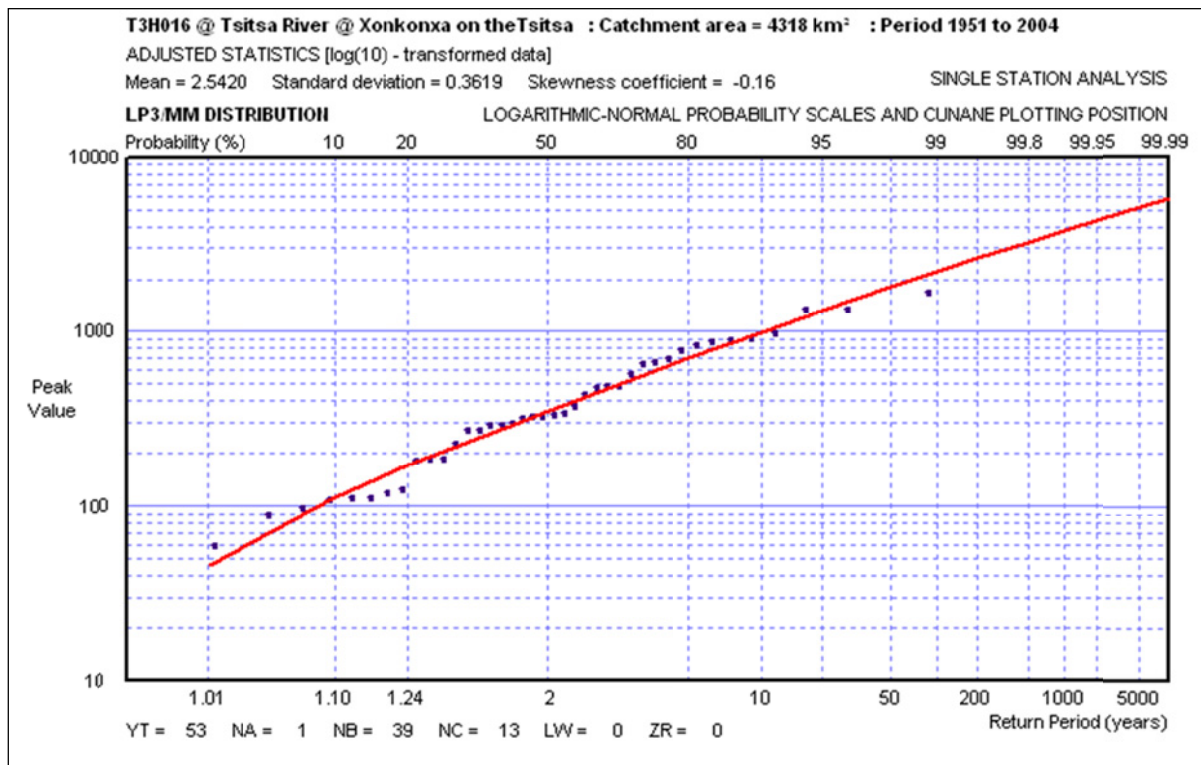
Year	Discharge (m <sup>3</sup> /s)	Comment	Description	Adopted peak flow value (m <sup>3</sup> /s)
1951/52	60			60
1952/53	112			112
1953/54	120	Incomplete year	Includes summer period	120
1954/55	197	Incomplete year	Too short year	-1
1955/56	126			126
1956/57	188			188
1957/58	188			188
1958/59	274			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

Based on the historical flow records the results from statistical analyses are shown in **Table 3.26**. Equations and statistical tables are included in **Appendix 3A**.

**Table 3.26: Statistical analyses for Tsitsa river gauging station T3H016 (missing data excluded)**

Return period	Extreme value Type 1	General extreme value	Log normal	Log Pearson Type 3	Log extreme value
2	421	414	352	358	307
5	766	758	714	719	645
10	994	991	1035	1022	1057
20	1214	1221	1401	1359	1696
50	1497	1527	1980	1850	3126
100	1710	1764	2507	2286	4953

The frequency distribution curve that fitted the data the best was the Log Pearson Type 3 curve (LP3) (see **Figure 3.30**).



**Figure 3.30: Log Pearson Type 3 fit through historical data points**

The following gauge record information relates to **Figure 3.30**.

YT (record length in years) = 53

NA (peaks  $\geq$  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

### 3.2.2 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 3.27**. 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 3.27: Results of SDF calculation**

Description	Answer obtained			
Area (km <sup>2</sup> )	4318			
L (km)	179,5			
S (m/m)	0,0037			
T <sub>C</sub> (hours)	31,16			
M (mm)	60			
R (days)	45			
C <sub>2</sub> (%)	10			
C <sub>100</sub> (%)	80			
Return period	1:10	1:20	1:50	1:100
P <sub>tT</sub>	116,81	144,32	180,69	208,20
ARF (%)	80	80	80	80
P <sub>avgT</sub> (mm)	93,23	115,19	144,21	166,17
I <sub>T</sub> (mm/hour)	2,99	3,70	4,63	5,33
C <sub>T</sub>	0,485	0,593	0,716	0,80
Q <sub>T</sub> (m <sup>3</sup> /s)	1739	2628	3974	5117

### 3.2.3 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. The formula reads:

$$Q_T = 0,0377 K_T P A^{0,6} C^{0,2} \dots (3.30)$$

where:

- $Q_T$  = peak flow for T return period (m<sup>3</sup>/s)  
 $K_T$  = coefficient based on veld-type region (see **Figure 3.24** and **Table 3.22**).  
 $P$  = mean annual precipitation over catchment (mm/a) (see **Figure 3.5** or utilise Design Rainfall database for the catchment (see **Figure 3.11**).

$$\text{and } C = \frac{A \sqrt{S}}{L L_C} \text{ (Catchment parameter with regard to reaction time)} \dots (3.31)$$

where:

- $A$  = area of catchment = 4318 km<sup>2</sup>  
 $S$  = 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

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  $K_{100} = 1,60$

The calculated catchment parameter C with regard to reaction time is 0,01721.

The flood peaks as determined using the empirical method (Equation 3.30) for the different return periods are shown in **Table 3.28**.

**Table 3.28: Flood peaks based on empirical method**

Return period	Peak flows (m <sup>3</sup> /s)
10	1 812
20	2 270
50	2 969
100	3 493

The regional maximum flood may be calculated as follows using the Francou-Rodier relationship:

$$Q_{RMF} = 10^6 \left( \frac{A}{10^8} \right)^{1-0,1K} \quad \dots(3.32)$$

where:

- $Q_{RMF}$  = regional maximum flood peak flow rate (m<sup>3</sup>/s)  
 $K$  = regional constant (Obtainable from the regional classification detailed in **Figure 3.26** and simplified in **Table 3.29**)  
 $10^6$  = total world MAR (m<sup>3</sup>/s)  
 $10^8$  = total world catchment area (km<sup>2</sup>)

**Step 1:** Determine the catchment area: 4318 km<sup>2</sup>.

**Step 2:** Identify the region in which the site is located (**Figure 3.26**). In this example, as shown in **Table 3.24**, 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 3.29** to calculate the RMF as 8059 m<sup>3</sup>/s

**Table 3.29: RMF region classification in southern Africa**

Kovács region	K	Number of floods #	Transition zone		Flood zone	
			Area range (km <sup>2</sup> )	$Q_{RMF}$ (m <sup>3</sup> /s)	Area range (km <sup>2</sup> )	$Q_{RMF}$ (m <sup>3</sup> /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: # Recorded flood data are reflected in the DWAF report TR105

### 3.2.4 Comparisons of solutions

Comparing the calculated flood peaks below (**Table 3.30**) 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 (**Figure 3.31** and **Figure 3.32**). 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 3.30: Comparison of calculated peak flows (m<sup>3</sup>/s)**

Return period	Empirical	Standard design flood	Regional maximum flood (RMF)	Statistical (LP3)
10	1 812	1 739		1 022
20	2 270	2 628		1 359
50	2 969	3 974	3 951*	1 850
100	3 493	5 117	4 796*	2 286
			8 059	

\*Using  $Q_T/Q_{RMF}$  ratios as detailed in **Appendix 3D**.





**Figure 3.31: Tsitsa crossing downstream view**



**Figure 3.32: Tsitsa crossing**

## 4 HYDRAULIC CALCULATIONS

### 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  $477 \text{ m}^3/\text{s}$ . The dimensions and absolute roughness values for the channel are shown below (**Figure 4.1**).

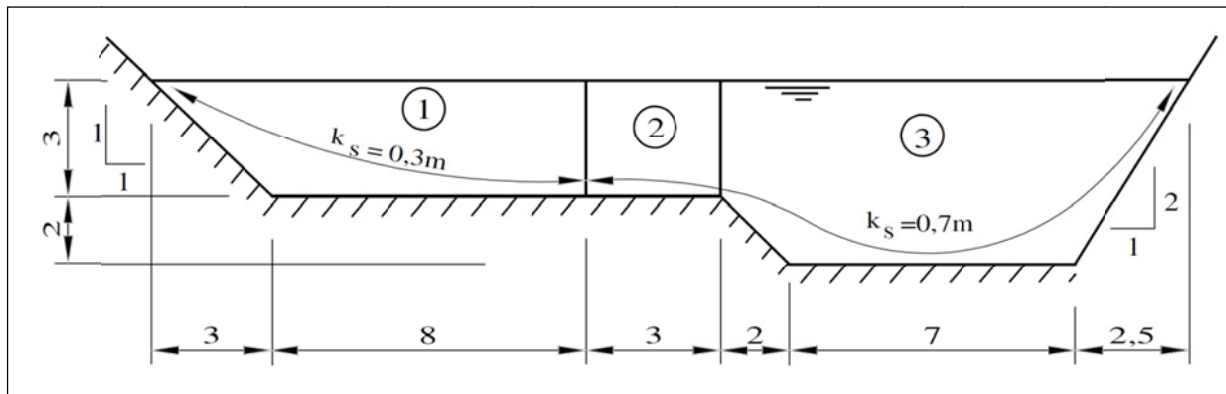


Figure 4.1: Cross-section of channel

**Determine:**

- The energy gradient ( $S_f$ ).
- Whether the flow is sub- or supercritical.
- The average velocity through section 3.
- Whether the flow is laminar or turbulent.
- The normal flow depth.

#### Solution Example 4.1

Divide the channel as shown above and derive the following details (**Table 4.1**):

Table 4.1: Channel characteristics (Example 4.1)

Parameter	Section		
	1	2	3
Area (A)	28,5 m <sup>2</sup>	9,0 m <sup>2</sup>	49,25 m <sup>2</sup>
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 <sup>1/2</sup> /s	30,8 m <sup>1/2</sup> /s	31,3 m <sup>1/2</sup> /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:

$$Q_{\text{total}} = \sum Q$$

$$Q_{\text{total}} = A_1 C_1 \sqrt{R_1 S_f} + A_2 C_2 \sqrt{R_2 S_f} + A_3 C_3 \sqrt{R_3 S_f}$$

$$\therefore Q_{\text{total}} = ((28,5)(35,4)\sqrt{2,33} + (9,0)(30,8)\sqrt{3,0} + (49,25)(31,3)\sqrt{3,19})\sqrt{S_f}$$

$$\therefore Q_{\text{total}} = (1541 + 480 + 2754)\sqrt{S_f}$$

$$\therefore S_f = \frac{Q_{\text{total}}^2}{4776^2} = 0,01 \text{ m/m (Energy gradient)}$$

- **Determine the flow regime**

$$(\text{Froude})^2 = \frac{Q^2 B}{g A^3} = \frac{(477)^2 (25,5)}{(9,81)(86,75)^3} \rightarrow Fr = 0,952 < 1,0 \text{ thus subcritical}$$

- **Determine the average velocity through section 3**

$$\bar{v}_3 = \frac{Q_3}{A_3} = \frac{2754}{4776} \times \frac{477}{49,25} = 5,59 \text{ m/s}$$

- **Identify the flow type**

Calculate the Reynolds Number

$$R_e = \frac{vR}{\nu}$$

$$\bar{v} = \frac{\sum Q}{\sum A} = \frac{477}{28,5 + 9 + 49,25} = 5,50 \text{ m/s}$$

$$\bar{R} = \frac{\sum A}{\sum P} = \frac{86,75}{12,24 + 3,0 + 15,42} = 2,83 \text{ m}$$

$$\nu = 1,14 \times 10^{-6} \text{ m}^2/\text{s} \text{ (kinematic viscosity of water)}$$

$$R_e = \frac{\bar{v}\bar{R}}{\nu} = \frac{(5,50)(2,83)}{1,14 \times 10^{-6}} = 13,65 \times 10^6$$

$$R_e \gg 2000 \therefore \text{Highly turbulent}$$

- **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,  $S_0$ .

$$Q_{\text{total}} = 477,0 = A_1 C_1 \sqrt{R_1 S_f} + A_2 C_2 \sqrt{R_2 S_f} + A_3 C_3 \sqrt{R_3 S_f}$$

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 shown in Table 4.2.

**Table 4.2: Channel characteristics written in terms of unknown flow depth (Example 4.1)**

Parameter	Section		
	1	2	3
Area (A) (m <sup>2</sup> )	$\frac{1}{2}Y^2 + 6Y - 14$	$3(Y - 2)$	$\left(\frac{Y^2}{4} + 9Y - 2\right)$
Wetted perimeter (P) (m)	$(8 + \sqrt{2}(Y - 2))$	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 <sub>s</sub> ) (m)	0,3	0,7	0,7
Chézy $C = 18 \log\left(\frac{12R}{k_s}\right)$ (m <sup>1/2</sup> /s)	*	*	*

*Note: \* The relationship is not shown, 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.

#### 4.2 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 in **Table 4.3**.

$$Q_{50} = 43,3 \text{ m}^3/\text{s}$$

**Table 4.3: Characteristics of the river cross-sections**

Section	Base width (m)	Bed level (m)	Chainage (m)	Manning, n (s/m <sup>1/3</sup> )	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 m<sup>3</sup>/s. It is necessary to determine the type of flow to establish the control, and then to work away from the control.

Although the assumption that uniform flow will be present at the cross-sections is incorrect, calculation of the “normal flow depth” at each section will give an indication of the type of flow. In **Table 4.4** the “normal flow depths” have been calculated. This is not the solution to the problem but merely a way to establish the type of flow!

##### Solution Example 4.2

From **Table 4.4** it could be concluded ( $Fr < 1$ ) that the flow will be subcritical and hence that the control will be downstream.

**Table 4.4: Flow characteristics (Example 4.2)**

Section ID	Position ID	Invert (m)	Calculation based on uniform flow assumption						
			Slope (local) (m/m)	$Y_n$ (m)	A (m <sup>2</sup> )	P (m)	R (m)	Cal Q (m <sup>3</sup> /s)	Fr
a *	b	c	d	e	f	g	h	i	j
1	Downstream	3,02	0,003	1,980	19,718	14,854	1,327	43,3	0,589
2		3,24	0,006	1,653	13,403	12,194	0,973	43,3	0,952
3	Site	3,75	0,004	1,725	15,614	13,316	1,027	43,3	0,792
4		3,99	0,007	1,622	14,026	12,656	0,958	43,3	0,908
5	Upstream	4,42							

Note: \* Refer to the legend table (Table 4.6)

Now start with the assumption of a flow depth at section 1 (downstream) and work upstream by applying the continuity of energy as shown in **Table 4.5**. 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.

**Table 4.5: Flood level calculations**

Section ID	$\Delta X$ (m)	H total energy (m)	Fr	Area (m <sup>2</sup> )	P (m)	Velocity (m/s)	$E_1$ (m)	$S_f$ (m/m)	$(S_0 - S_f)_{avg}$ (m/m)	$E_2$ (m)	$\Delta(E_2 - E_1)$ (m)	Water level (m)
a	k	l	m	n	o	p	q	r	s	t	u	v
1		1205,246	0,589	19,718	14,854	2,196	1205,246	0,00338				1205,00
	65								0,000198	1205,245	-0,001	
2		1205,544	0,675	17,499	13,709	2,474	1205,544	0,00299				1205,23
	82								0,002349	1205,545	0,000	
3		1205,832	0,904	14,105	12,765	3,070	1205,832	0,00475				1205,35
	67								-0,000622	1205,833	0,001	
4		1206,195	0,692	17,282	13,829	2,506	1206,195	0,00366				1205,87
	66								0,002394	1206,195	0,000	
5		1206,505	0,889	14,286	12,832	3,031	1206,505	0,00459				1206,04

Notes: A brief description of the columns content for the above tables is listed in Table 4.6

**Table 4.6: Legend table**

Column ID	Description of the variable
a	Section identification
b	Description of the position
c	Invert level (m)
d	Local slope calculated from the level difference between that of the section and the upstream section divided by the distance between the sections
e	$Y_n$ is the calculated flow depth assuming that uniform flow characteristics will occur
f	Calculated area for the given $Y_n$
g	Calculated wetted perimeter for the given $Y_n$
h	Calculated hydraulic radius for the given $Y_n$
i	Calculated flow rate
j, m	Froude number
k	Distance between the sections
n	Calculated area
o	Calculated wetted perimeter

Table 4.6: Legend table (continued)

Column ID	Description of the variable
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

#### 4.3 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, see **Figure 4.2**. The discharge is 0,1 m<sup>3</sup>/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.

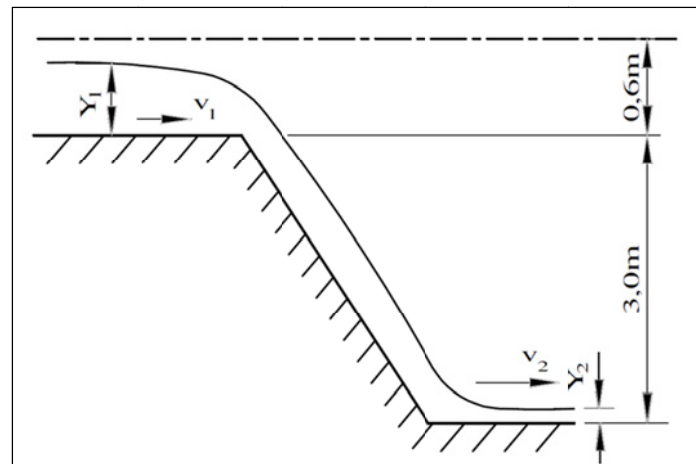


Figure 4.2: Concrete chute down embankment

##### Solution Example 4.3

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:

$$\begin{aligned} Q &= 0,1 \text{ m}^3/\text{s} \\ B &= 0,6 \text{ m} \\ A_c &= B_c y_c = 0,6 y_c \\ g &= 9,81 \text{ m/s}^2 \\ y_c &= 0,141 \text{ m} \\ \bar{v}_c &= 1,178 \text{ m/s} \end{aligned}$$

$$\text{and } E_c = y_c + \frac{\bar{v}_c^2}{2g} = 0,141 + \frac{(1,178)^2}{2(9,81)} = 0,212 \text{ m}$$

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.

$$q = \bar{v}_2 y_2 = 0,167 \text{ m}^3/\text{sm}$$

$$\bar{v}_2 = 7,9 \text{ m/s}; y_2 = 0,021 \text{ m and } Fr_2 = \frac{\bar{v}_2}{\sqrt{gy_2}} = 17$$

The actual velocity will be slightly lower. (If  $H \gg y_2$ , then  $\bar{v}_2 \approx \sqrt{2gH}$ )

#### 4.4 Example 4.4 – 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  $0,02 \text{ s/m}^{1/3}$ . There is a  $90^\circ$  bend with a centre-line radius of 7 m. Calculate the Froude number for uniform flow conditions.

##### Solution Example 4.4

Calculate the flow depths just upstream and just downstream of the  $90^\circ$  bend.

$$Fr_n = \frac{\bar{v}_n}{\sqrt{gy_n}} = \frac{2,0}{\sqrt{(9,81)(2,0)}} = 0,452 \text{ (based on the normal flow depth)}$$

$\therefore$  Downstream control and hence the depth just downstream of the bend will be 2 m.

$$\text{Energy head loss through bend: } h_\ell = \frac{2B}{r_c} \cdot \frac{\bar{v}^2}{2g} = \frac{(2,0)(2,0)(2,0)^2}{(7,0)(2)(9,81)} = 0,1165 \text{ m}$$

Energy equation:

$$\text{Upstream energy head: } H_1 = H_2 + h_\ell = \frac{\bar{v}^2}{2g} + y_2 + h_\ell$$

$$H_1 = \frac{(2,0)^2}{(2)(9,81)} + 2,0 + 0,1165 = 2,321 \text{ m}$$

$$\therefore y_1 + \frac{\bar{v}_1^2}{2g} = 2,321 \text{ m}$$

Write the velocity in terms of the upstream flow depth ( $y_1$ ) using continuity:

$$\bar{v}_1 = \frac{Q}{A_1} = \frac{(2,0)(2,0 + 2,0)}{(y_1)(2,0)} = \frac{4,0}{y_1}$$

$$\therefore y_1 + \frac{\left(\frac{4,0}{y_1}\right)^2}{(2)(9,81)} = 2,321 \text{ m. Solving the only unknown term - } y_1$$

$$y_1 = 2,143 \text{ m (depth upstream of bend)}$$



## 4.5 Example 4.5 – Identification of acting controls

### Problem description Example 4.5

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 4.3.

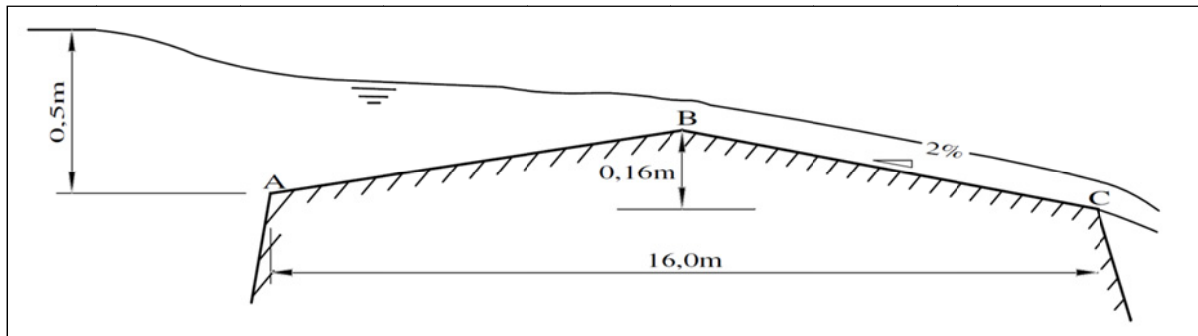


Figure 4.3: Flow across the road

### Solution Example 4.5

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:

$$q = \bar{v}_c y_c = \sqrt{g y_c^3} = 0,338 \text{ m}^3/\text{s.m} \quad \text{Assume that the actual discharge is } 0,3 \text{ m}^3/\text{s.m}$$

$$y_c = \sqrt[3]{\frac{q^2}{g}} = 0,209 \text{ m} \quad \text{and} \quad E_c = \frac{3}{2} y_c = 0,314 \text{ m}$$

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 =  $0,013 \text{ s/m}^{1/3}$

$$0,3 = \frac{y^{\frac{5}{3}} (0,02)^{\frac{1}{2}}}{0,013} \quad y_n = 0,116 \text{ m} < y_c = 0,209 \text{ m}$$

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.

$$Re = \frac{\bar{v} y}{\nu} = \frac{0,3}{1,14 \times 10^{-6}} = 2,6 \times 10^5 \gg 2000, \text{ indicating the flow is turbulent.}$$

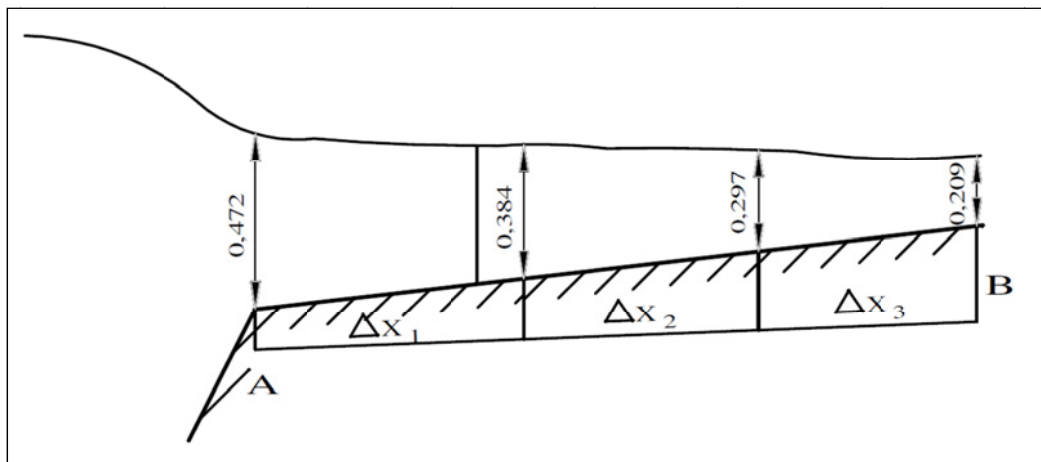
Now determine the depth  $y_A$  at A.

$$0,5 = y_A + \frac{\bar{v}_A^2}{2g} + 0,35 \frac{\bar{v}_A^2}{2g} \quad (\text{Energy equation with provision for transition losses})$$

$$\text{and } y_A \bar{v}_A = 0,3 \text{ (continuity)}$$

$$\text{Thus } y_A = 0,472 \text{ m}$$

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 4.4**), and **Table 4.7**.



**Figure 4.4: Increments of flow depth across the road**

$$\frac{dE}{\Delta x} = S_o - \bar{S}_f \quad (\text{Energy equation for prismatic channels})$$

$$\Delta x = \frac{dE}{S_o - \bar{S}_f} \quad dE = \left( y_1 + \frac{\bar{v}_1^2}{2g} \right) - \left( y_2 + \frac{\bar{v}_2^2}{2g} \right)$$

$$S_o = -0,02 \text{ m/m (uphill)} \quad S_f = \frac{\bar{v}^2 n^2}{y^{\frac{4}{3}}}$$

**Table 4.7: Calculation table (Example 4.5)**

y (m)	E (m)	S <sub>f</sub> (m/m)	S <sub>f(average)</sub> (m/m)	Δx (m)
0,209	0,314	0,002792		
			0,001831	1,60
0,297	0,349	0,000870		
			0,000619	3,23
0,385	0,416	0,000368		
			0,000277	3,81
0,472	0,493	0,000185		

$$\Sigma \Delta x = 8,64 \text{ m}$$

$\Sigma \Delta x = 8,64 > 8,0$  thus the discharge per unit width should be less (i.e. q should be smaller). Choose smaller q and repeat until  $\Sigma \Delta x \approx 8,0$  m for a more accurate answer.

## 5 SURFACE DRAINAGE

A number of typical problems are explained below.

### 5.1 Worked Example 5.1 - Flow depth on the road surface

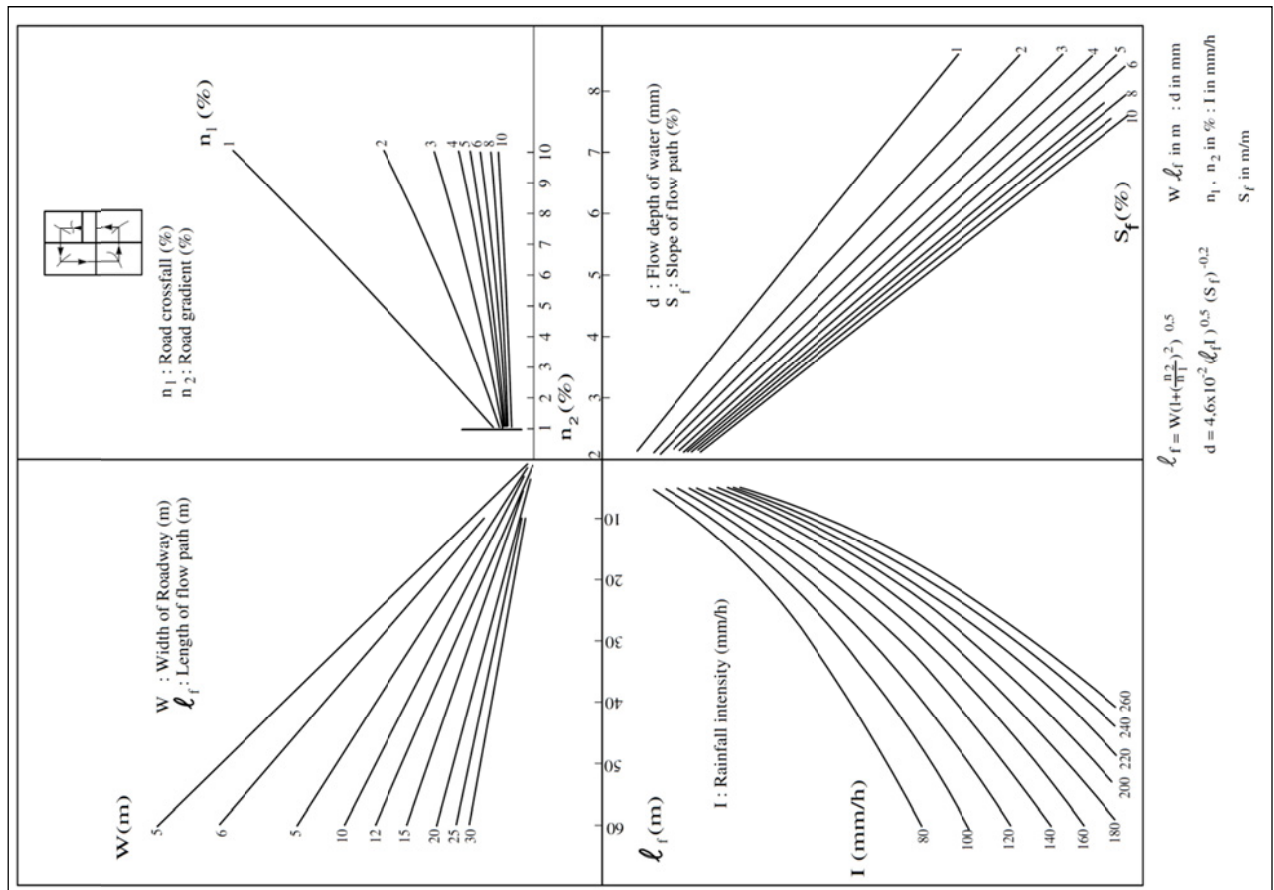
#### Problem description Example 5.1

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: Depth of sheet flow on road surface (Laminar flow conditions assumed)**

**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 dependent on  $n_1$  and  $n_2$  and may be calculated as follows:

$$S_f = \sqrt{n_1^2 + n_2^2} \quad \dots(5.1)$$

$$S_f = \sqrt{2^2 + 6^2}$$

$$S_f = 6,32\%$$

Calculate the flow path length,  $L_f$ :

$$L_f = W \left( 1 + \frac{n_2^2}{n_1^2} \right)^{\frac{1}{2}} = (10) \left( 1 + \frac{(6)^2}{(2)^2} \right)^{\frac{1}{2}} \quad \dots(5.2)$$

$$L_f = 31,62$$

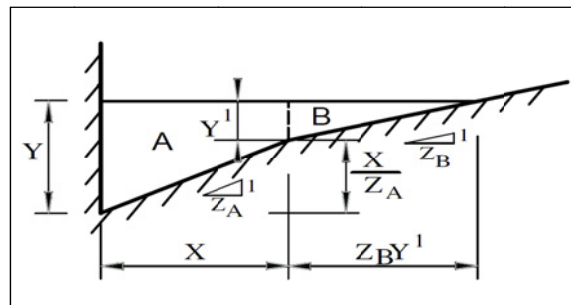
$$d = (4,6 \times 10^{-2}) (L_f I)^{0,5} (S_f^{-0,2}) \quad \dots(5.3)$$

$$d = (4,6 \times 10^{-2}) ((31,62)(100))^{0,5} (0,0632)^{-0,2} = 4,49 \text{ mm}$$

## 5.2 Worked Example 5.2 – Capacity of side channel

### Problem description Example 5.2

Determine the flow capacity in a side channel cross section shown in **Figure 5.2**, and the dimensions provided below.



**Figure 5.2: Channel cross section**

Manning roughness,  $n = 0,015 \text{ s/m}^{1/3}$

Flow depth,  $Y = 100 \text{ mm}$

$Y^1 = 40 \text{ mm}$

$1/Z_A = 1/20$

$1/Z_B = 1/40$

Road gradient = 5 %

### Solution Example 5.2

The following relationships could be obtained for geometry (units in mm):

$$X = Z_A(Y - Y^1) \quad \dots(5.4)$$

$$X = (20)(100 - 40)$$

$$X = 1200$$

$$Z_B Y^1 = (40)(40)$$

$$Z_B Y^1 = 1600$$

$$\text{Top width, } T = 1200 + 1600 = 2800 \text{ mm}$$

$$Z = \frac{T}{Y}$$

$Q = Q_A + Q_B$  see calculations in **Table 5.1**.

$S = 0,05 \text{ m/m}$

Manning  $n = 0,015 \text{ s/m}^{1/3}$

**Table 5.1: Channel flow characteristics**

Parameter	Section A	Section B
Cross-sectional area ( $\text{m}^2$ )	$A_A = (0,5)(0,1 + 0,04)(1,2)$ $A_A = 0,084$	$A_B = (0,5)(0,040)(1,6)$ $A_B = 0,032$
Wetted perimeter (m)	$P_A = 0,1 + [(0,06)^2 + (1,2)^2]^{0,5}$ $P_A = 1,302$	$P_B = 1,6[(0,04)^2 + (0,001)^2]^{0,5} / 0,04$ $P_B = 1,6001$
Hydraulic radius (m)	$R_A = 0,06454$	$R_B = 0,020$
Flow rate from Manning ( $\text{m}^3/\text{s}$ ) $Q = \frac{R^{0,667} S^{0,5}}{n} A$	$Q_A = 0,2015$	$Q_B = 0,03514$

The total capacity of the channel is:

$$Q_{\text{total}} = 0,2366 \text{ m}^3/\text{s}$$

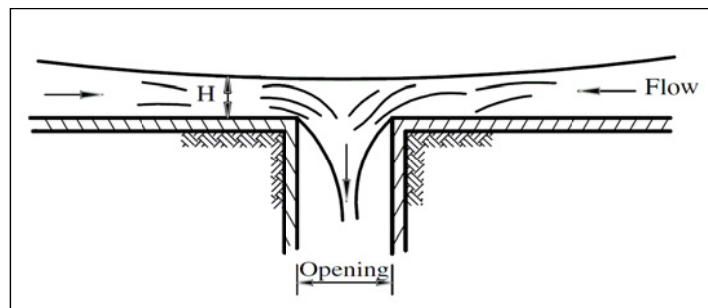
### 5.3 Worked Example 5.3 – Capacity of drop grid inlet

#### 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 inlet coefficient = 0,8 and the blockage factor,  $F = 0,5$ .

#### Solution Example 5.3

**Figure 5.3** provides the relationship of flow rate for an orifice control or a broad-crested weir.



**Figure 5.3: Section through outlet: Drowned conditions**

$$Q = CFA\sqrt{2gH} \quad \dots(5.5)$$

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 ( $\text{m}^2$ )
- $H$  = total energy head above grid (m)



**Figure 5.4: Example of type of grid inlet**

From Equation 5.5:

$$Q = 1,77A \sqrt{H}$$

$$A = (0,9)(0,6) = 0,54 \text{ m}^2$$

$$Q = 1,77 (0,54) \sqrt{0,2}$$

$$Q = 0,427 \text{ m}^3/\text{s}$$

The calculation of the flow rate was based on the orifice equation.

#### **5.4 Worked Example 5.4 – Kerb flow**

##### **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  $0,015 \text{ s/m}^{1/3}$ .

##### **Solution Example 5.4**

The Manning equation may be used.

$$Q = \frac{1}{n} \frac{A^{5/3}}{P^{2/3}} \sqrt{S}$$

$$A = \frac{1}{2} YT = (0,5)(0,1)(5,0) = 0,25 \text{ m}^2$$



$$P = Y + \sqrt{Y^2 + T^2} = 0,1 + \sqrt{(0,1)^2 + (5,0)^2} = 5,101 \text{ m}$$

$$S = \frac{4}{100} = 0,04 \text{ m/m}$$

$$Q = \frac{1}{(0,015)} \frac{(0,25)^{\frac{5}{3}}}{(5,101)^{\frac{2}{3}}} \sqrt{0,04} = 0,446 \text{ m}^3/\text{s}$$

Strictly speaking, the very wide section with variable velocities should be subdivided into narrower sections.

## 5.5 Worked Example 5.5 – Scour velocity

### 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:

$$V = 18 \log \left( \frac{12R}{k_s} \right) \sqrt{RS} \quad \dots(5.6)$$

From Shields (Equation 5.7) it follows that for a stable channel:

$$d_1 > 11 DS \quad \dots(5.7)$$

$$D = y = \frac{(0,25)}{(11)(0,02)} = 1,14 \text{ m}$$

Now the velocity can be calculated using Equation 5.6:

$$v = 18 \log \left( \frac{12(1,14)}{(0,25)} \right) \sqrt{(1,14)(0,02)}$$

$$v = 4,71 \text{ m/s}$$

## 5.6 Worked Example 5.6 – Protection measures

### 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  $25^\circ$  and a flow depth of 1,8 m. The stones are slightly angular and have an angle of repose of  $30^\circ$ . The channel slope is 0,1%.

### Solution Example 5.6

For a stable bed the particle size ( $d_1$ ) should at least be:

$$d_1 = 11DS = (11)(1,8)(0,001) = 0,0198 \text{ m}$$

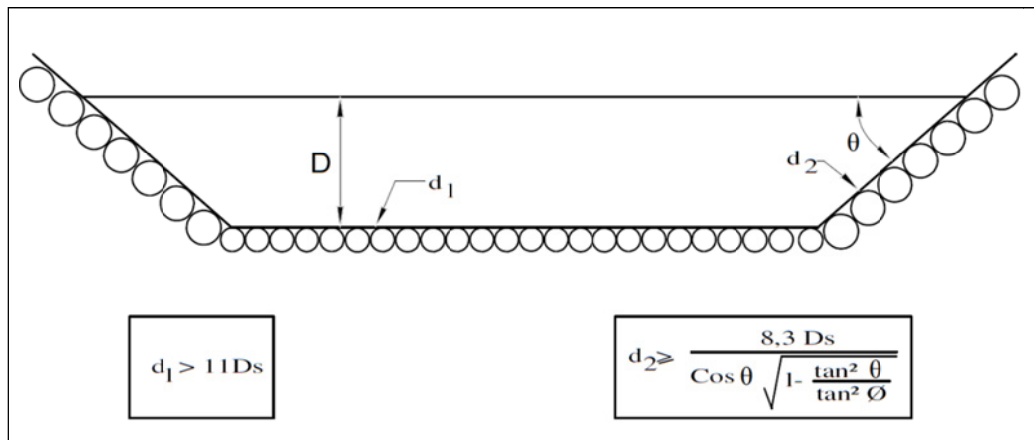
For stable side slopes the particle size ( $d_2$ ) should at least be:

$$d_2 = \frac{8,3Ds}{\cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}}} \quad \dots(5.8)$$

where:

$\theta$  = angle of slope of sides of the channel ( $^\circ$ )

$\phi$  = angle of repose of stone material ( $^\circ$ )



**Figure 5.5: Required sizes of the stone for erosion protection of loose bed channels**  
(The side slope,  $\theta$ , should always be smaller than the angle of repose,  $\phi$ , to ensure stability.)

From Equation 5.8:

$$d_2 = \frac{(8,3)(1,8)(0,001)}{\cos(25) \sqrt{1 - \frac{\tan^2(25)}{\tan^2(30)}}} = 0,028 \text{ m}$$

## 6 LOW LEVEL CROSSINGS

### 6.1 Worked Example 6.1 – Low level crossing

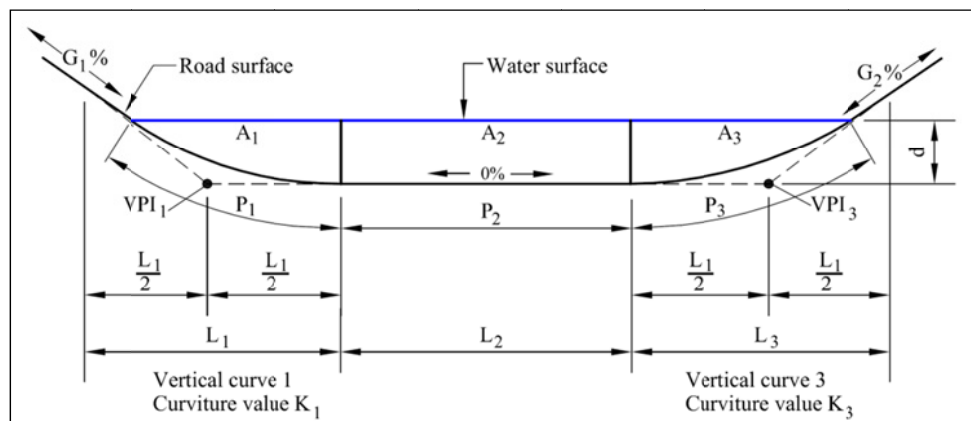
#### Problem description Example 6.1

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 km<sup>2</sup>. The 1:2 year flood has been determined as 120 m<sup>3</sup>/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 6.1**). The slope of the road on the southern bank is - 5,6%, and on the northern bank 7,0%.



**Figure 6.1: Definition of symbols for the flow over the structure**

The deck thickness is taken as 500 mm, and the soffit of the deck is on average 1 400 mm above the riverbed.

#### 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.

The design flow rate is determined from Equation 6.1:

$$Q_{\text{design}} = f_i Q_2 \quad \dots(6.1)$$

From **Table 6.1** follows that  $f_2$  is 0,50 and  $Q_2$  is the discharge with a 1:2 year return period (120 m<sup>3</sup>/s).

**Table 6.1: Levels of design for low-level structures\***

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

Note: \* Based on observed data from the Northern Province

$$\begin{aligned} Q_{\text{design}} &= 0,5 \times 120 \text{ m}^3/\text{s} \\ &= 60 \text{ m}^3/\text{s} \end{aligned}$$

- **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. 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 and through the structure.

*Flow over the structure*

Assume supercritical flow and decide on a maximum flow depth of 0,1 m ( $d$ ). The flow that can be accommodated over the structure is determined from Equation 6.2.

$$Q_{\text{over}} = \frac{A_{\text{over}}^{5/3} S_0^{1/2}}{n P_{\text{over}}^{2/3}} \quad \dots(6.2)$$

$$A_{\text{over}} = A_1 + A_2 + A_3, \text{ or}$$

$$A_{\text{over}} = \frac{1}{3} d \sqrt{800 K_1 d} + d L_2 + \frac{1}{3} d \sqrt{800 K_3 d} \quad \dots(6.3)$$

and

$$P_{\text{over}} = P_1 + P_2 + P_3, \text{ or}$$

$$P_{\text{over}} = \frac{1}{2} \sqrt{800K_1 d} + L_2 + \frac{1}{2} \sqrt{800K_3 d} \quad \dots(6.4)$$

where:

- $Q_{\text{over}}$  = the discharge that could be accommodated over the structure within the selected flow depth ( $\text{m}^3/\text{s}$ )
- $A_{\text{over}}$  = area of flow over structure at the flow depth selected ( $\text{m}^2$ )
- $S_0$  = slope in direction of flow, for example 0,02 or 0,03 m/m
- $n$  = Manning n-value. For a concrete deck  $n_{\text{concrete}}$
- $P_{\text{over}}$  = wetted perimeter at the flow depth selected (m)
- $A_1, A_2, A_3$  = the areas defined in **Figure 6.1** ( $\text{m}^2$ )
- $d$  = depth of flow over the structure (m)
- $K_1$  = the geometric K value for vertical curve 1
- $K_2$  = the geometric 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.

$S_0$  is 0,02 (2% as above) and Manning n roughness parameter for concrete is  $0,016 \text{ s/m}^{1/3}$ . The cross-section area of flow is determined using Equation 6.2.

$$A_{\text{over}} = \frac{1}{3} d \sqrt{800K_1 d} + dL_2 + \frac{1}{3} d \sqrt{800K_3 d} \text{ where } K_1 \text{ and } K_3 \text{ are both 4 and } L_2 \text{ is 20 m (Equation 6.3)}$$

$$A_{\text{over}} = 3,19 \text{ m}^2$$

$$P_{\text{over}} = \frac{1}{2} \sqrt{800K_1 d} + L_2 + \frac{1}{2} \sqrt{800K_3 d} = 37,89 \text{ m (equation 6.4)}$$

From Equation 6.2:

$$Q_{\text{over}} = \frac{A_{\text{over}}^{5/3} S_0^{1/2}}{n P_{\text{over}}^{2/3}} = \frac{(3,19)^{5/3} (0,02)^{1/2}}{(0,016)(37,89)^{2/3}}$$

$$Q_{\text{over}} = 5,42 \text{ m}^3/\text{s}$$

Establish whether flow is indeed supercritical by calculating the Froude number (Equation 6.5):

$$Fr = \sqrt{\frac{Q_{\text{over}}^2 B}{g A_{\text{over}}^3}} \quad \dots(6.5)$$

where:

- $B$  =  $L_1 + L_2 + L_3$  (m), the width of the channel (or the length of the structure)
- $g$  =  $9,81 \text{ m/s}^2$ , the gravity constant

$$\text{and } L_1 = \frac{1}{2} \sqrt{800K_1 d} \text{ and } L_3 = \frac{1}{2} \sqrt{800K_3 d}$$

$B = L_1 + L_2 + L_3$ ,  $L_1 = L_3 = 8,94 \text{ m}$  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:

$$Q_{\text{under}} = \bar{v}_{\text{under}} A_{\text{eff}} \quad \dots(6.6)$$

$\bar{v}_{\text{under}}$  is determined from Equation 6.7, for which the following is required:

$$\bar{v}_{\text{under}} = \sqrt{\frac{H_1 - H_2}{\frac{C}{2g} + \frac{n_{\text{eff}}^2 L_B}{R^{4/3}}}} \quad \dots(6.7)$$

where:

- $A_{\text{eff}}$  = the effective inlet area through the structure ( $\text{m}^2$ ) =  $\Sigma A_{\text{cell}}$  (the effective inlet area through the structure)
- $L_B$  = the total width of the deck of the structure (m)
- $\bar{v}_{\text{under}}$  = the velocity of flow through the structure (m/s)
- $C$  = factor that reflects the transition losses (Equation 6.8)

$$C = \sum (K_{\text{inl}} + K_{\text{out}})_{\text{each cell}} \quad \dots(6.8)$$

$K_{\text{inl}}$  and  $K_{\text{out}}$  are determined as follows for rectangular sections:

- |                                     |                     |   |
|-------------------------------------|---------------------|---|
| $K_{\text{inl}}$ at outlet control: | Sudden transition:  | $K_{\text{inl}} = 0,5$                                    |
|                                     | Gradual transition: | $K_{\text{inl}} = 0,25$                                   |
| $K_{\text{out}}$ at outlet control: | Sudden transition:  | $K_{\text{out}} = 1,0$                                    |
|                                     | Gradual transition: | $K_{\text{out}} = 1,0$ for $45^\circ < \theta < 80^\circ$ |
|                                     |                     | $0,7$ for $\theta = 30^\circ$                             |
|                                     |                     | $0,2$ for $\theta = 15^\circ$                             |

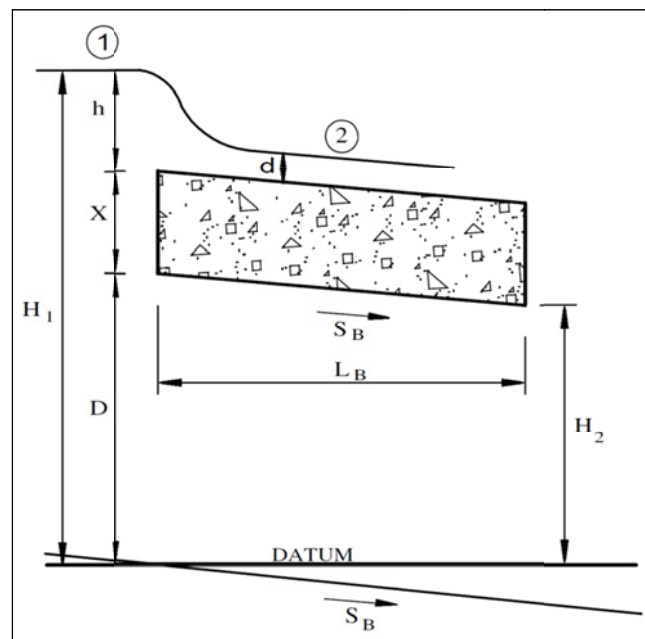


Figure 6.2: Definition of symbols



By applying the conservation of energy principle, determine the depth upstream of the structure,  $h$  as shown in Figure 6.2, that is required to pass the flow rate,  $Q_{\text{over}}$ :

$$h = \frac{\bar{v}_2^2}{2g} + d \quad \dots(6.9)$$

$$H_1 = h + x + D \quad \dots(6.10)$$

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)

$$H_2 = D - L_B S_0 \quad \dots(6.11)$$

where:

- $L_B$  = the total width of the deck of the structure (m)
- $S_0$  = slope of the conduit underneath the structure (m/m)

Determine the depth upstream of the structure,  $h$ , that is required to pass the flow rate,  $Q_{\text{over}}$  using Equation 6.9.

$$h = \frac{\bar{v}_2^2}{2g} + d, \text{ where } \bar{v}_2 = Q_{\text{over}}/A_{\text{over}} = 1,70 \text{ m/s}$$

$$h = 0,247 \text{ m}$$

From Equation 6.10:

$$H_1 = h + x + D, \text{ where } h \text{ is as above, } x = 0,5 \text{ m and } D = 1,4 \text{ m}$$

$$H_1 = 2,147 \text{ m}$$

From Equation 6.11:

$$H_2 = D - L_B S_0, \text{ where } L_B = (4,0) + (2)(0,25) = 4,5 \text{ m (for the guide-blocks)}$$

$$H_2 = 1,4 - (4,5)(0,02)$$

$$H_2 = 1,31 \text{ m}$$

Assume  $K_{\text{inl}} = 0,5$  and  $K_{\text{out}} = 1,0$  (both sudden transitions), then

$C = 6 \times (0,5 + 1,0)$ , see Equation 6.8.

$$C = 9$$

$P_{\text{cell}}$  is the total wetted perimeter of each cell (m).

$$P_{\text{cell}} = (5,7)(2) + (1,4)(2) = 14,2 \text{ m}$$

$$n_{\text{cell}} = \frac{P_{\text{concrete}} n_{\text{concrete}}}{P_{\text{cell}}} + \frac{P_{\text{river}} n_{\text{river}}}{P_{\text{cell}}} \text{ and assume } n_{\text{river}} \text{ to be } 0,03 \text{ s/m}^{1/3}$$

$$n_{\text{cell}} = \frac{(5,7 + (2)(1,4))(0,016)}{14,2} + \frac{(5,7)(0,03)}{14,2}$$

$$n_{\text{cell}} = 0,022 \text{ s/m}^{1/3}$$

$$P_{\text{eff}} = \Sigma P_{\text{cell}} = (6)(14,2) = 85,2 \text{ m}$$

$$n_{\text{eff}} = \frac{\Sigma (n_{\text{cell}} P_{\text{cell}})}{P_{\text{eff}}}$$

$$n_{\text{eff}} = \frac{(6)((0,022)(14,2))}{85,2}$$

$$n_{\text{eff}} = 0,022 \text{ s/m}^{1/3}$$

$$R = A_{\text{eff}}/P_{\text{eff}} \text{ where}$$

$$A_{\text{eff}} = (6)(5,7)(1,4) = 47,88 \text{ m}^2$$

$$R = 0,562 \text{ m}$$

$\bar{v}_{\text{under}}$  from Equation 6.7 is:

$$\bar{v}_{\text{under}} = \sqrt{\frac{H_1 - H_2}{\frac{C}{2g} + \frac{n_{\text{eff}}^2 L_B}{R^{4/3}}}} = \sqrt{\frac{2,147 - 1,31}{\frac{9}{2(9,81)} + \frac{(0,022)^2 (4,5)}{(0,562)^{4/3}}}}$$

$$\bar{v}_{\text{under}} = 1,344 \text{ m/s}$$

$$\text{Also from Equation 6.6 } Q_{\text{under}} = \bar{v}_{\text{under}} A_{\text{eff}} = 64,35 \text{ m}^3/\text{s}$$

### *Design discharge*

$$\text{The capacity of the structure at the design level } Q_{\text{over}} + Q_{\text{under}} = 69,8 \text{ m}^3/\text{s}$$

As  $Q_{\text{over}} + Q_{\text{under}}$  is larger than  $Q_{\text{design}}$  ( $60 \text{ m}^3/\text{s}$ ), 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.

## 7 LESSER CULVERTS AND STORMWATER PIPES

### 7.1 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  $85 \text{ m}^3/\text{s}$  and the road can be assumed to be a Class 3 road. From **Figure 7.1** the design flood frequency is determined as  $T = 15$  years and the calculated 1:15 year flood ( $Q_{15}$ ) is  $44,5 \text{ m}^3/\text{s}$ .

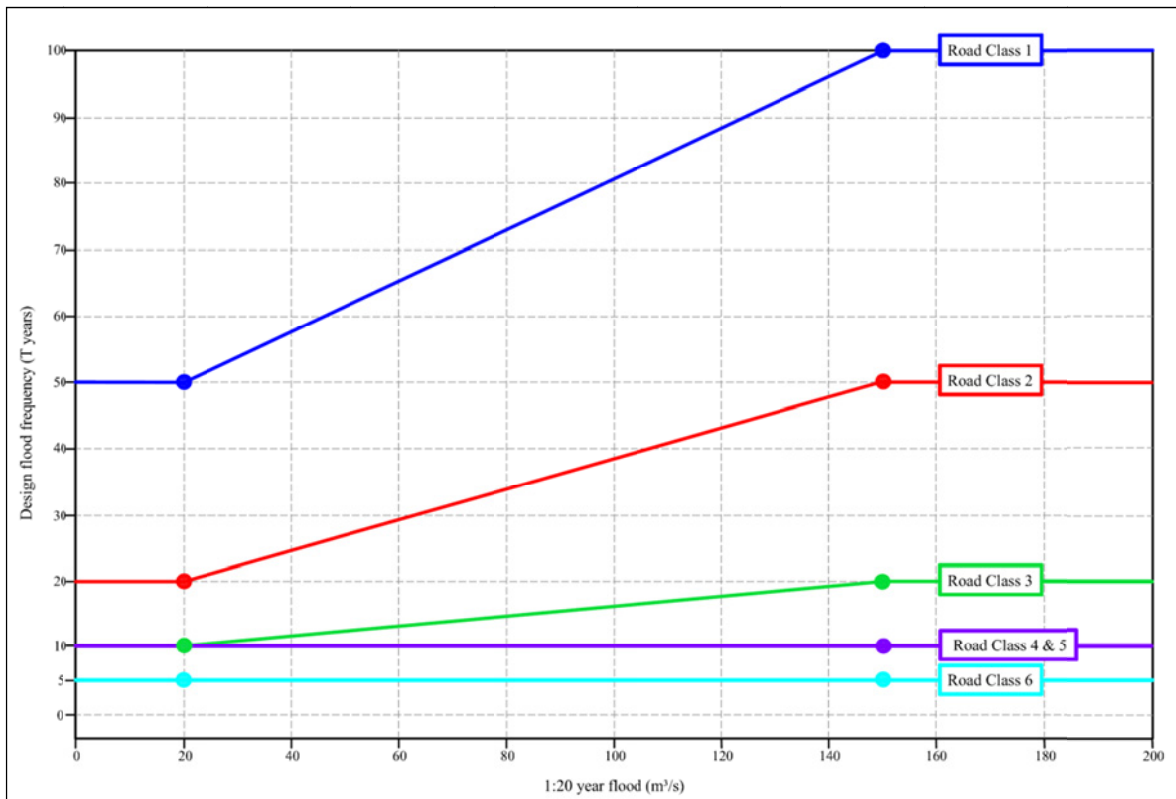


Figure 7.1: Design flood frequency estimate

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 in **Figure 7.2**. 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.

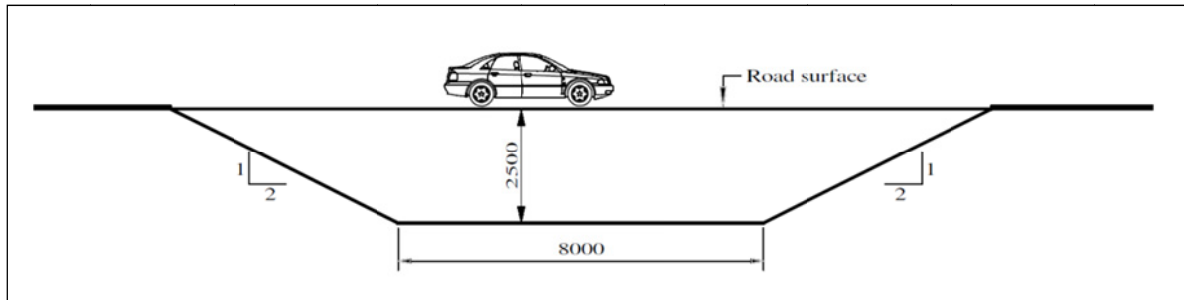


Figure 7.2: Cross-sectional details of the natural channel

### 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.

$$Q = C \sqrt{RS} A \quad \dots(7.1)$$

where:

$Q$	=	flow rate ( $\text{m}^3/\text{s}$ )
$C$	=	Chèzy constant
$R$	=	hydraulic radius (m)
$A$	=	area ( $\text{m}^2$ )
$Y_{n \text{ upstream}}$	=	$Y$ = 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:

$$44,5 = 18 \log \left( \frac{(12)(8Y + (2)(0,5)(Y^2)(2))}{(0,1)(8 + (2)\sqrt{5} Y)} \right) \sqrt{\frac{(8Y + (2)(0,5)(Y^2)(2))(0,0015)}{(8 + (2)\sqrt{5} Y)}} (8Y + (2)(0,5)(Y^2)(2))$$

**Solve the upstream normal flow depth  $Y$  in the above equation.**

**$Y = 2,0 \text{ m}$  and  $A = 24,0 \text{ m}^2$ , hence  $V = 1,851 \text{ m/s}$ .**

The flow type can be determined by calculating the Froude number,

$$\text{Fr}^2 = \frac{Q^2 B}{g A^3} = 0,2327 \text{ and } \text{Fr} = 0,482$$

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,  $Fr_{ds} = 0,841$  and the flow is subcritical.

Total energy head upstream of the culvert,  $H_1 = \frac{\bar{v}^2}{2g} + Y = 2,175$  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 7.4** 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 7.1**).

**Table 7.1: Relationships for the flow rate under inlet control**

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: $0 < H_1/D \leq 1,2$ $Q = \frac{2}{3} C_B B H_1 \sqrt{\frac{2}{3} g H_1}$ Where: $C_B = 1,0$ for rounded inlets ( $r > 0,1B$ ) $C_B = 0,9$ for square inlets
And for: $0,8 < H_1/D \leq 1,2$ * $\frac{Q}{D^2 \sqrt{gD}} = 0,44 \left[ \frac{S_0}{0,4} \right]^{0,05} \left[ \frac{H_1}{D} \right]^{1,5}$ ( $S_0$ = slope of culvert bed with slight effect on capacity) <b>Note:</b> * 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: $H_1/D > 1,2$ $Q = C_h B D \sqrt{2g \left( H_1 - C_h D \right)}$ Where: $C_h = 0,8$ for rounded inlets $C_h = 0,6$ for square inlets

In this example, the maximum  $\frac{H_1}{D} = \frac{2,5}{1,8} = 1,39$  and the flow rate through a culvert can be determined.

**Table 7.1** reflects that for a circular pipe culvert, under submerged conditions with  $H/D \geq 1,2$ , the flow rate can be determined as follows:

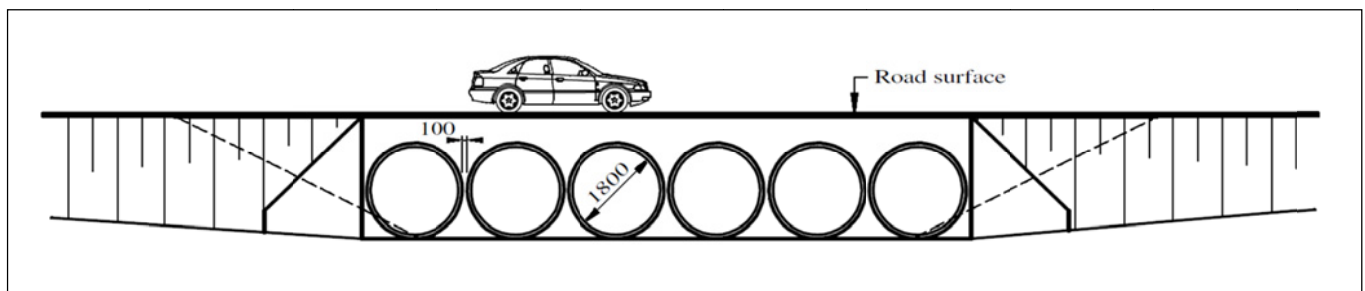
$$Q = C_D A \sqrt{2g \left( H_1 - \frac{D}{2} \right)} \quad \dots(7.2)$$

$$Q = (0,6) \sqrt{(2)(9,81) \left( 2,5 - \frac{1,8}{2} \right)} \left( \pi \frac{(1,8)^2}{4} \right) = 8,55 \text{ m}^3/\text{s}$$

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 =  $(6)[1,8 + (2)(0,078)] + (5)(0,1) = 12,236 \text{ m}$ . With some groundwork it is possible to place the culverts as is shown in **Figure 7.3**.



**Figure 7.3: Positioning of the 6 pipe culverts**

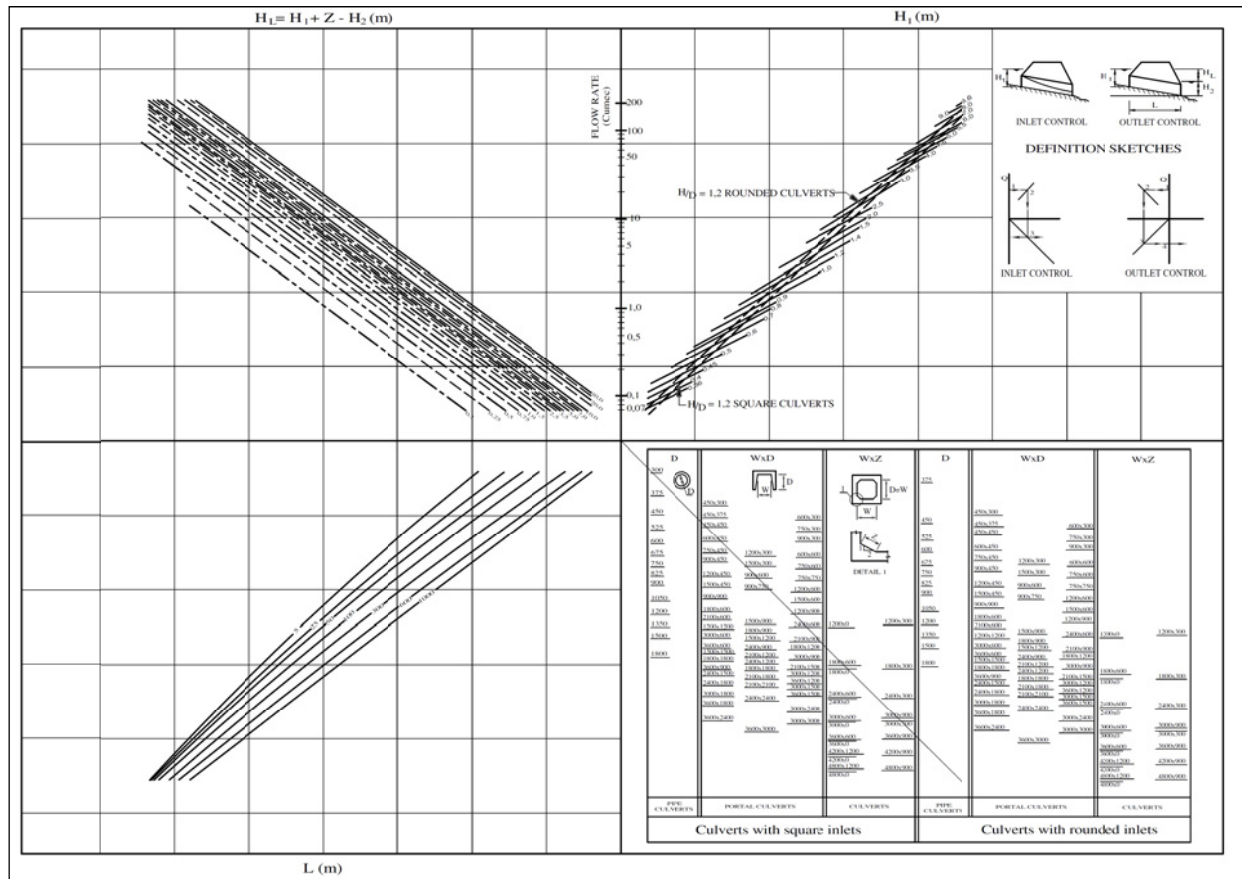
There are, however, also box culverts that could have been used here.

**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 =  $44,5/5 = 8,9 \text{ m}^3/\text{s}$ .





**Figure 7.4: Diagram for the determination of sizes of culverts and storm water pipes**

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  $8,9 \text{ m}^3/\text{s}$ , a  $1800 \times 1800$  mm portal culvert could be selected (as shown in **Figure 7.5**). The  $H/D$  ratio will however be  $1,33$  and the capacity of the 5 culverts needs to be verified.

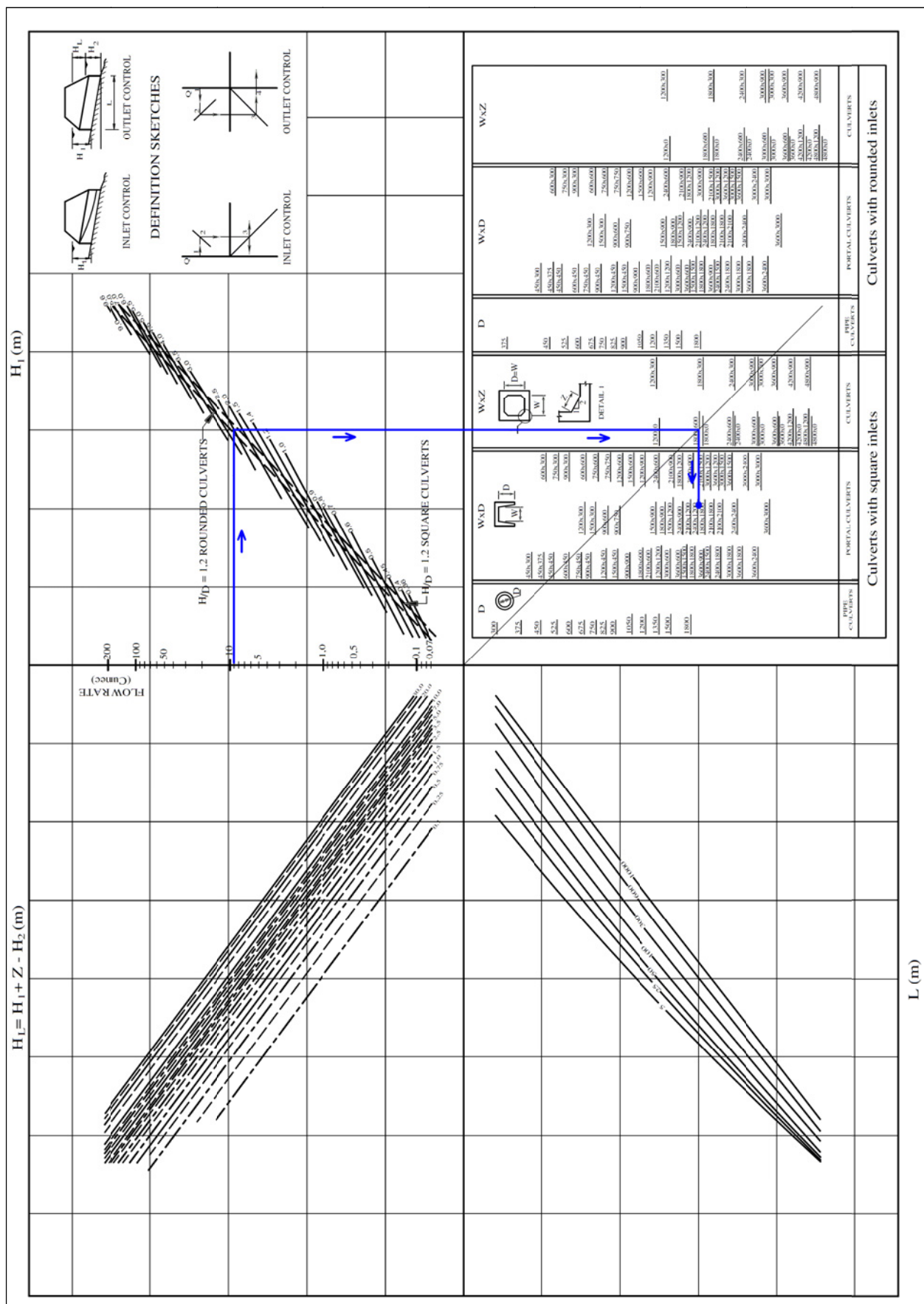
This result can be checked with the following formula (**Table 7.1**):

$$Q = C_h B D \sqrt{2g(H_1 - C_h D)} \quad \dots(7.3)$$

Where  $C_h = 0,8$  for rounded inlets and  $C_h = 0,6$  for square inlets.

$$Q = (0,6)(1,8)(1,8)\sqrt{2(9,81)(2,4 - (0,6)(1,8))} = 9,893 \text{ m}^3/\text{s}$$

and hence 5 culverts will be sufficient.



- **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

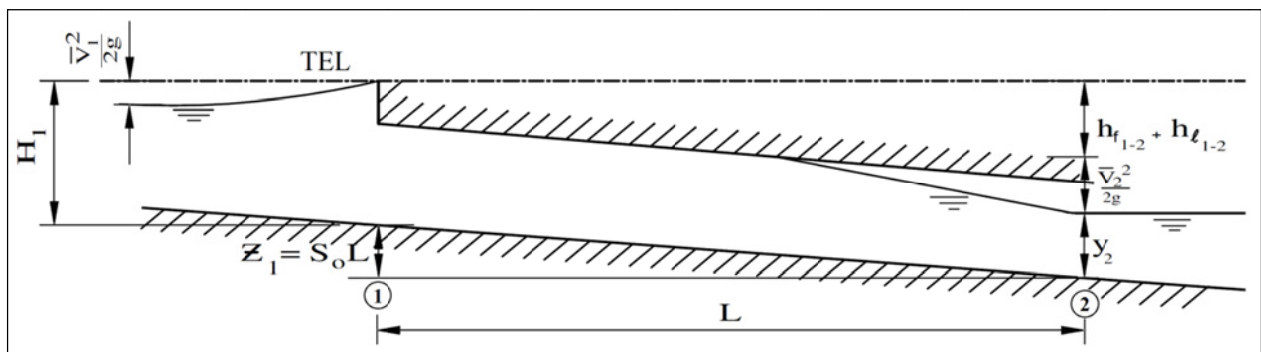
Diameter of the culvert,  $D = 1\ 800$  mm

Length of the culvert,  $L = 25$  m

By assuming that 6 culverts will be used the flow rate per culvert =  $44,5/6 = 7,417$  m<sup>3</sup>/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 7.6**) the required upstream energy,  $H_1$ , can be determined by using the energy principle.



**Figure 7.6: Energy components of flow through a culvert**

$$H_1 + S_0 L = H_2 + h_{1-2} + h_{f1-2} \quad \dots(7.4)$$

(between Position 1 (upstream) and Position 2 (downstream))

For the flow of 7,417 m<sup>3</sup>/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,  $h_{1-2}$ , can be determined as follows:

$$h_{1-2} = h_{l \text{ inlet}} + h_{l \text{ outlet}} \quad \dots(7.5)$$

$$h_{1-2} = (K_{\text{inlet}} + K_{\text{outlet}}) \frac{\bar{v}^2}{2g} = (0,5 + 1) \frac{(2,915)^2}{2(9,81)} = 0,649 \text{ m}$$

The friction losses,  $h_f$ , can also be determined, assuming that full bore flow conditions in the culvert will prevail:

$$h_f = \frac{\lambda L \bar{V}^2}{2gD} \quad \dots(7.6)$$

For rough turbulent flow conditions,

$$\frac{1}{\sqrt{\lambda}} = 2 \log \left( \frac{3,7D}{k_s} \right) \quad \dots(7.7)$$

$$\lambda = 0,02015$$

$$h_{f_{1-2}} = 0,121 \text{ m}$$

For outlet control conditions, the upstream conditions can now be determined (Equation 7.4):

$$H_1 = H_2 - Z_1 + h_{l_{1-2}} + h_{f_{1-2}}$$

$$H_1 = (2,0 + 0,175) - 0,0015(25) + 0,649 + 0,121$$

$$H_1 = 2,908 \text{ m}$$

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  $44,5/7 = 6,357 \text{ m}^3/\text{s}$  ( $\bar{v} = 2,498 \text{ m/s}$ ) and the value of the losses,  $H_L = h_{l_{1-2}} + h_{f_{1-2}} = 0,477 + 0,089 = 0,566 \text{ m}$ , the upstream conditions can now be determined.

$$H_1 = H_2 - Z_1 + H_L$$

$$H_1 = (2,0 + 0,175) - 0,0015(25) + 0,566$$

$$H_1 = 2,704 \text{ m (which is still greater than the allowable upstream damming height)}$$

To illustrate the use of **Figure 7.4**, start at  $Q = 6,357 \text{ m}^3/\text{s}$  (flow rate in each of the 7 culverts) and  $H_L = 0,566 \text{ m}$ , then it can be seen that the 1 800 mm pipe culvert is still insufficient to transport the flow (**Figure 7.7**). **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  $(7)[1,8+(2)(0,078)] + (6)(0,1) = 14,3 \text{ m}$ , 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 (although still marginally undersize). 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.

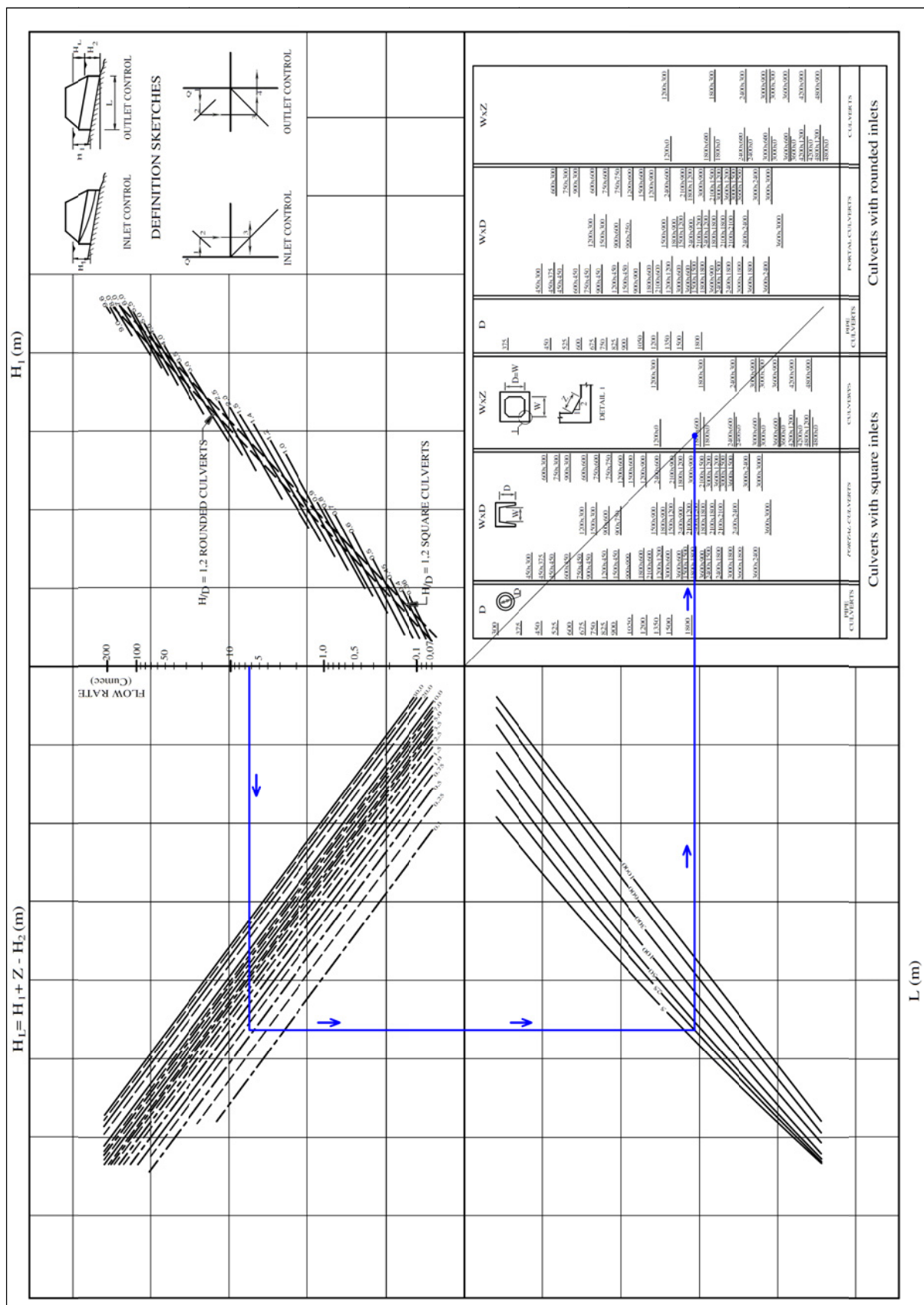


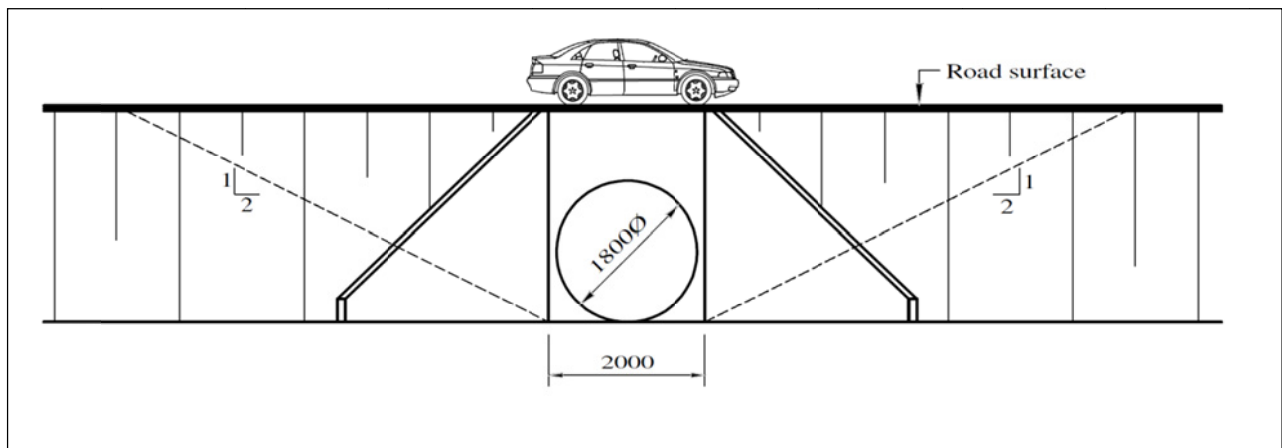
Figure 7.7: Determining culvert size (outlet control)

## 7.2 Example 7.2 - Erosion protection downstream from a culvert

### Problem description Example 7.2

In **Example 7.1** it was indicated that for inlet control it is possible to convey  $8,9 \text{ m}^3/\text{s}$  (for  $H/D = 1,2$ ) through a  $1,8 \text{ m}$  diameter circular culvert. You are now requested to design the protection works for a single  $1,8 \text{ m}$  diameter culvert, functioning under inlet control conditions with  $H/D = 1,2$ . The concrete culvert is  $28 \text{ m}$  long with an estimated absolute roughness of  $0,003 \text{ m}$ . The culvert will be installed at a slope of  $0,01 \text{ m/m}$ .

The flow releases into a natural trapezoidal river section with a base width of  $2,0 \text{ m}$  and side slopes of  $1\text{V}:2\text{H}$ . The natural slope of the river is  $0,004 \text{ m/m}$  and the roughness is  $0,05 \text{ m}$ . Details of the cross-section are given in **Figure 7.8**.



**Figure 7.8: Upstream view of the culvert**

### Solution Example 7.2

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  $28 \text{ m}$  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 than the critical slope,  $S_0 > S_c$ ) or the flow depth will be greater for a subcritical slope ( $S_0 < S_c$ ).

**Firstly** the normal flow depth,  $y_n$ , downstream from the culvert in the river is determined by using the Chezy formula:

$$\bar{v} = C\sqrt{RS} \quad \dots(7.8)$$

$$Q = vA = 18 \left( \log \frac{12R}{k_s} \right) \sqrt{RSA} \quad \dots(7.9)$$

where:

- $Q$  = flow rate ( $\text{m}^3/\text{s}$ )
- $\bar{v}$  = average velocity ( $\text{m/s}$ )
- $R$  = hydraulic radius ( $\text{m}$ )
- $k_s$  = absolute roughness ( $\text{m}$ )
- $S$  = slope of the river section ( $\text{m/m}$ )

Table 7.2 reflects the cross-sectional parameters for a circular pipe.



Firstly the normal flow depth,  $y_n$ , downstream from the culvert in the river is determined by using the Chezy formula:

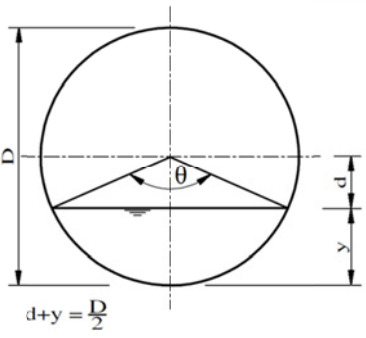
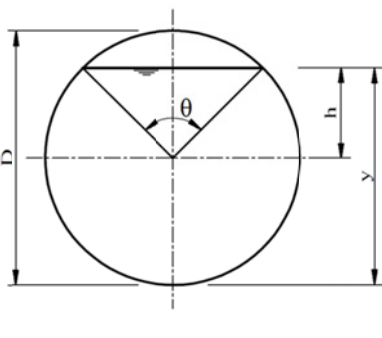
$$\bar{v} = C\sqrt{RS} \quad \dots(7.8)$$

$$Q = vA = 18 \left( \log \frac{12R}{k_s} \right) \sqrt{RS} A \quad \dots(7.9)$$

where:

- $Q$  = flow rate ( $\text{m}^3/\text{s}$ )
- $\bar{v}$  = average velocity ( $\text{m/s}$ )
- $R$  = hydraulic radius ( $\text{m}$ )
- $k_s$  = absolute roughness ( $\text{m}$ )
- $S$  = slope of the river section ( $\text{m/m}$ )

**Table 7.2: Sectional parameters for a circular cross-section**

Variable	$Y < D/2$ ( $\theta < \pi$ radians)	$Y > D/2$ ( $\theta > \pi$ radians)
<b>Cross-sectional view</b>		
<b>Area, A (<math>\text{m}^2</math>)</b>	$R^2 \theta - 2(0,5)(R - y) \sin\left(\frac{\theta}{2}\right) R$	$\pi R^2 - (R^2 \theta) - 2(0,5)(R - y) \sin\left(\frac{\theta}{2}\right) R$
<b>Wetter perimeter, P (m)</b>	$R(\theta)$	$R(2\pi - \theta)$
<b>Hydraulic radius, R (m)</b>	$\frac{A}{P}$	

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  $y_n$ :

$$y_n = 1,068 \text{ m}$$

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:

$$S_c = \frac{\bar{v}^2}{C^2 R} = \frac{Q^2}{C^2 A^3 R} \quad \dots(7.10)$$

$$S_c = 0,00792 \text{ m/m}$$

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:

$y = 1,330 \text{ m}$ ;  $A = 2,015 \text{ m}^2$ ;  $B = 1,582 \text{ m}$  (top width of flow);  $\bar{v} = 4,417 \text{ m/s}$  and  $Fr = 1,25$  (subcritical).

$$y/D = 1,330/1,8 = 0,739$$

**Figure 7.9** can now be used to select the appropriate erosion protection.

In this example the appropriate protection falls in the Type III. **Figure 7.10** can now be used to obtain dimensions for outlet erosion protection.

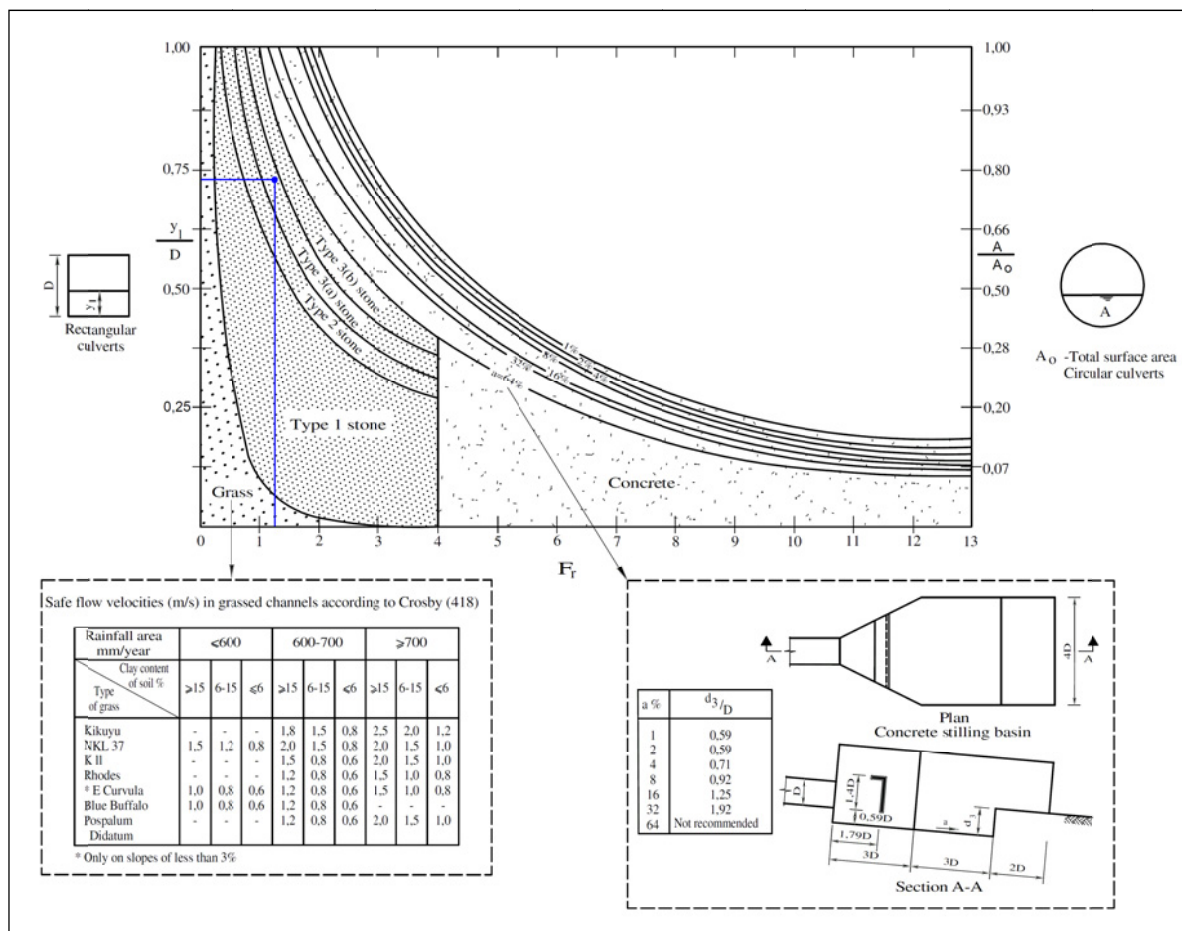
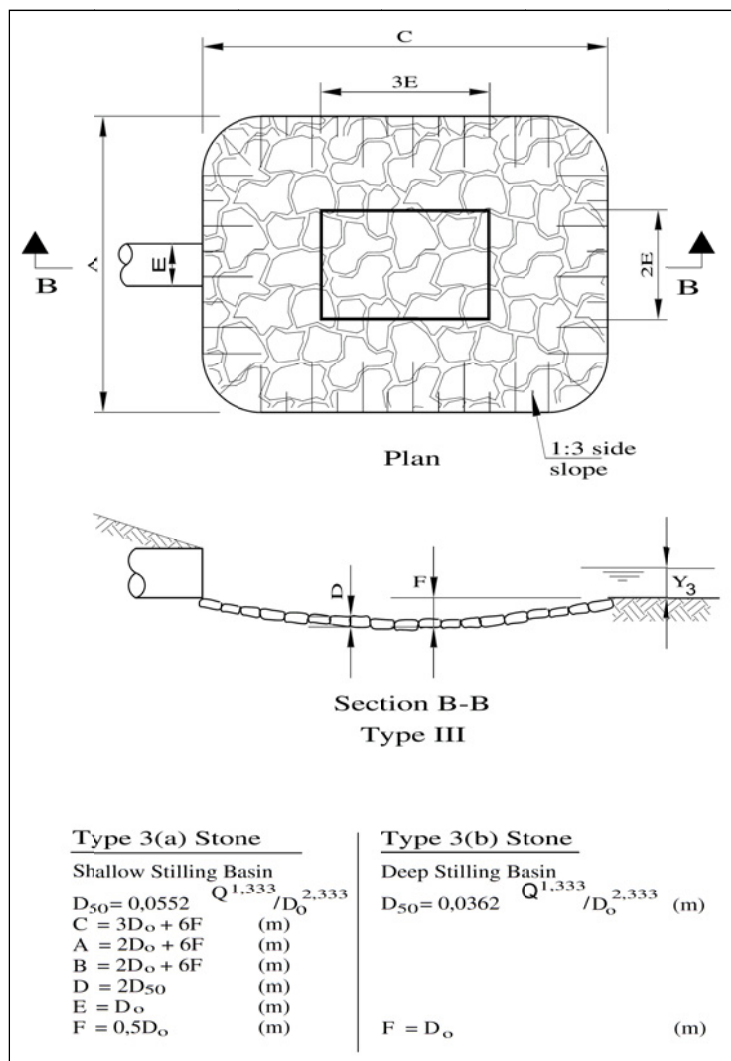


Figure 7.9: Limiting values for different methods of erosion protection at culvert outlets



**Calculated dimensions:**

$$\begin{aligned}
 D_{50} &= 0,258 \text{ m} \\
 C &= 10,8 \text{ m} \\
 A &= 9,0 \text{ m} \\
 B &= 9,0 \text{ m} \\
 D &= 0,517 \text{ m} \\
 E &= 1,8 \text{ m} \\
 F &= 0,9 \text{ m}
 \end{aligned}$$

**Figure 7.10: Dimensions for the outlet erosion protection**

Using **Figure 7.11** with a flow of  $8,9 \text{ m}^3/\text{s}$  in a circular culvert the values of Froude and the flow depth can also be obtained which will correspond with the calculated values.

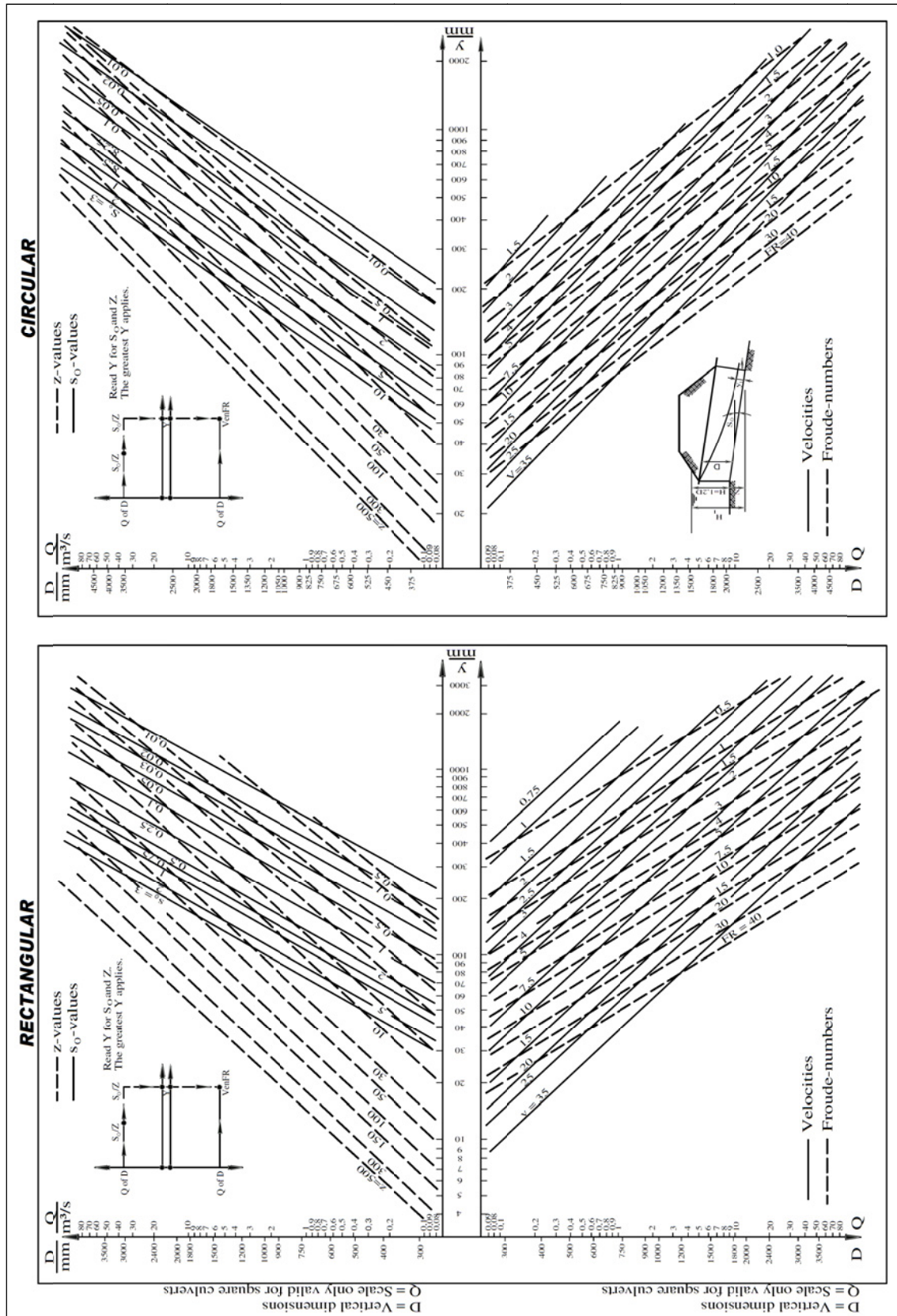


Figure 7.11: Diagram for the determination of outlet velocities for steep culverts and storm water pipes (Only  $H1/D = 1,2$ ,  $S_0 > S_c$ )

## 8 BRIDGES AND MAJOR CULVERTS

### 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 8.1**.

Design discharge	$Q$	=	$150 \text{ m}^3/\text{s}$
Average bed slope	$S_o$	=	$0,00082 \text{ m/m}$
Angle of skew	$\theta$	=	$15^\circ$
Bridge span on skew	$b_s$	=	$17,6 \text{ m}$
Projected bridge span	$b$	=	$17,0 \text{ m}$
No of rows of piers	$N_p$	=	$1$
Projected width of pier	$W_p$	=	$2,00 \text{ m}$

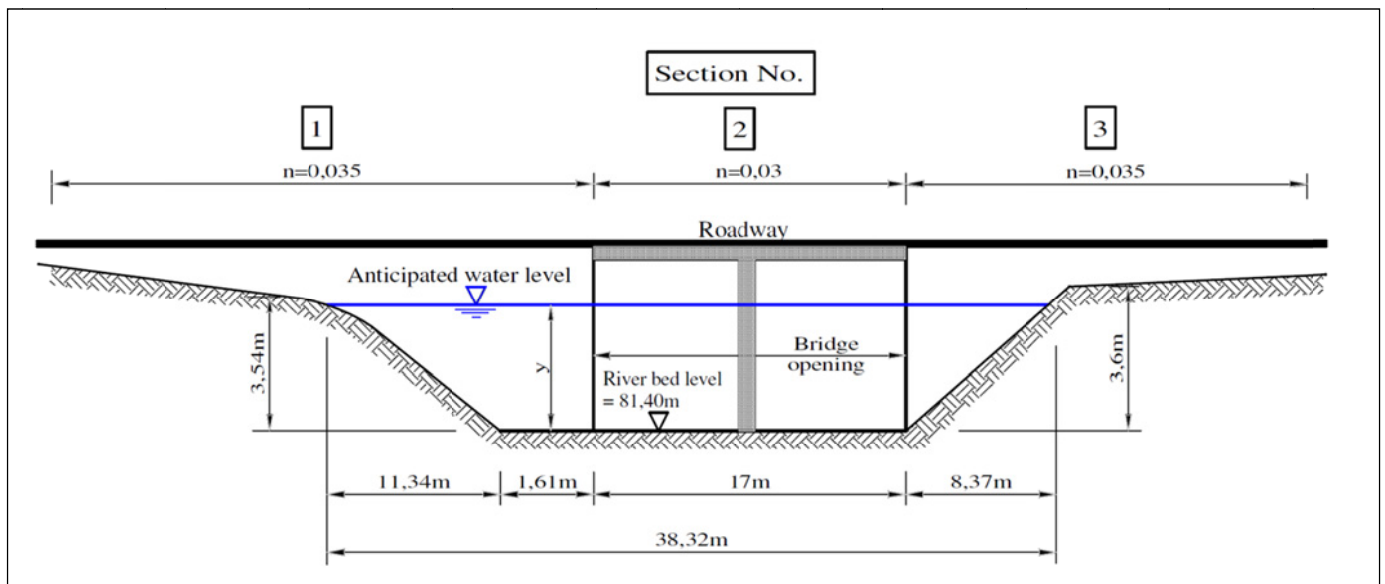


Figure 8.1: Upstream view of the bridge

#### Determine:

- Characteristics of the unconstricted flood state
- The flow type
- Bridge opening ratio
- Velocity head coefficients
- Calculate backwater

#### Solution Example 8.1

- Characteristics of the unconstricted flood state:*

First determine the normal flow depth,  $y_n$ .



**Table 8.1: Sub-section details**

Sub-section	$n_i$	$A_i (\text{m}^2)$	$P_i (\text{m})$
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,  $y_n = 3,157 \text{ m}$ .

**Table 8.2: Sub-section flow characteristics**

Sub-section	$A_i (\text{m}^2)$	$P_i (\text{m})$	$R_i = \frac{A_i}{P_i} (\text{m})$	$q_i (\text{m}^3/\text{s})$	$\bar{v}_i = \frac{q_i}{A_i} (\text{m/s})$
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 result 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 unconfined normal flow:

with  $y_n = 3,157 \text{ m}$  (Flood stage level – river bed level)

$$\bar{v}_n = \frac{Q}{A_n} = \frac{150}{86,30}$$

$$= 1,738 \text{ m/s}$$

$$E_{sn} = y_n + \frac{\bar{v}_n^2}{2g} = 3,157 + \frac{(1,738)^2}{2(9,81)}$$

$$= 3,311 \text{ m}$$

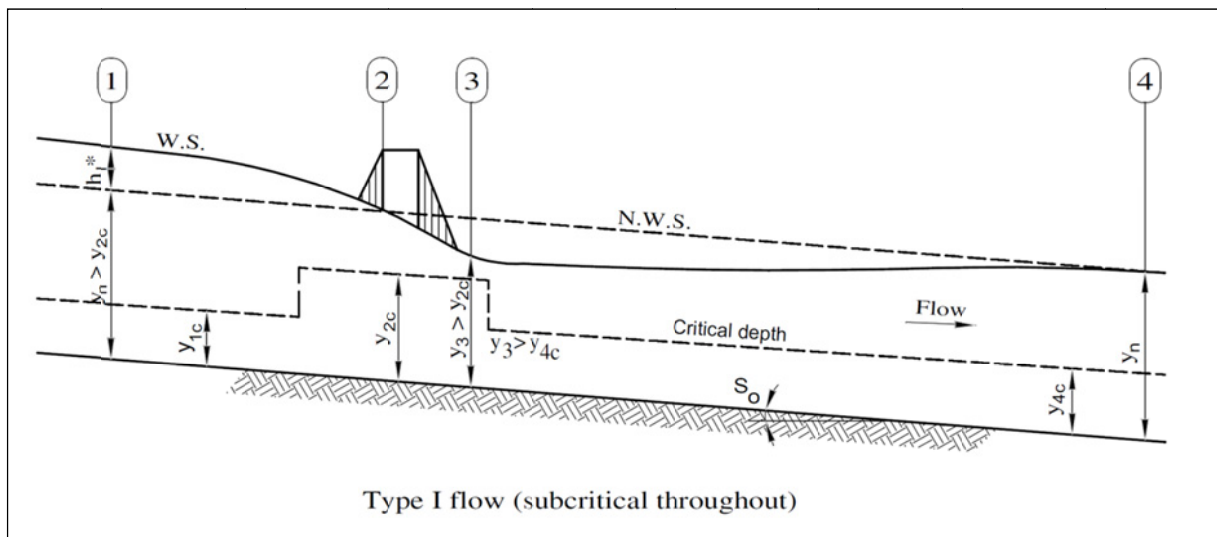
Calculate specific energy ( $E_{sc}$ ) of constricted flow critical depth:

$$y_{2c} = \left( \frac{Q^2}{gb^2} \right)^{1/3} = \left( \frac{(150)^2}{(9,81)(17)^2} \right)^{1/3} = 1,995 \text{ m}$$

$$\bar{v}_{2c} = \frac{Q}{y_{2c}b} = \frac{150}{(2,817)(17)} = 4,424 \text{ m/s}$$

$$E_{sc} = y_{2c} + \frac{\bar{v}_{2c}^2}{2g} = 1,995 + \frac{(4,424)^2}{2(9,81)} = 2,992 \text{ m} < E_{sn} \text{ indicating Type I flow (see$$

**Figure 8.2).**



**Figure 8.2: Type I flow with substantial damming and does not reach critical conditions**

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:

$$\begin{aligned} Q_b &= (120,69) \left( \frac{17}{17 + 1,61} \right) & M &= \frac{Q_b}{Q} = \frac{110,25}{150} \\ &= 110,25 \text{ m}^3/\text{s} & &= 0,735 \end{aligned}$$

(iv) Calculate velocity head coefficients:

$$\begin{aligned} \alpha_1 &= \frac{\sum (q \bar{v}^2)}{Q \bar{v}_n^2} \\ &= 1,20 \\ \alpha_2 &= 1,15 \text{ (from Figure 8.3)} \end{aligned}$$

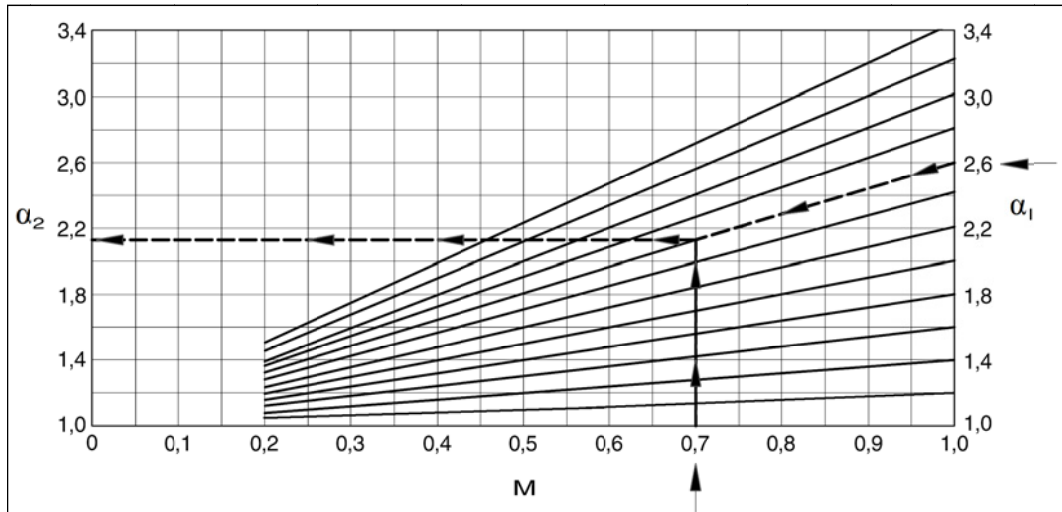


Figure 8.3: Estimation of the velocity coefficient,  $\alpha_2$  (generic example)

(v) Calculate backwater

**For Type I flow:**

Determine secondary energy loss coefficient  $K^*$  from **Figure 8.4**:

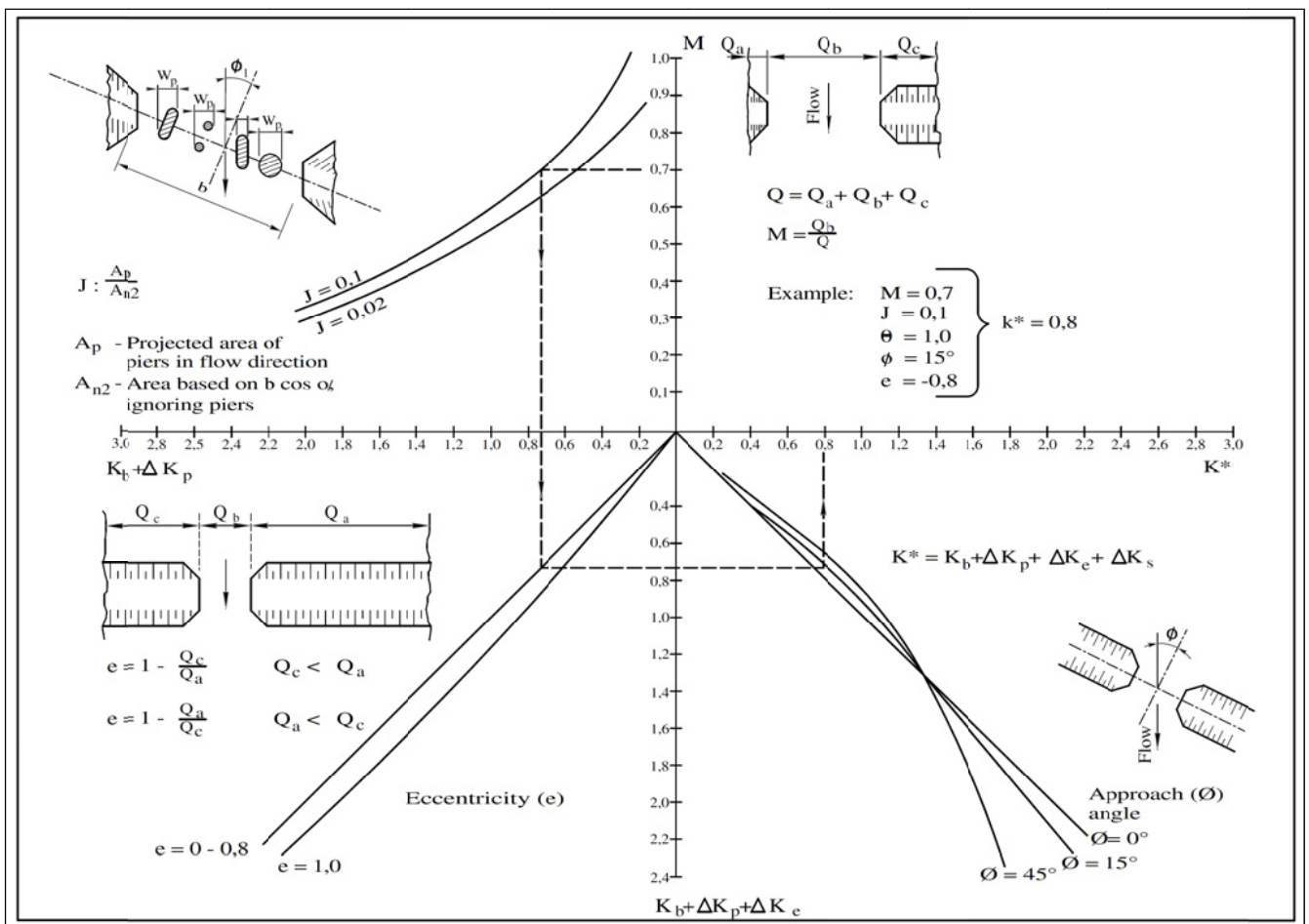


Figure 8.4: Chart to determine the backwater coefficient,  $K^*$

Projected area of piers in flow direction and projected area below normal water level.

$$\begin{aligned} A_p &= W_p y_n & A_{n2} &= (b \cos \theta)(y_n) \\ &= (2)(3,157) & &= (17 \cos(15^\circ))(3,157) \\ &= 6,314 \text{ m}^2 & &= 51,84 \text{ m}^2 \end{aligned}$$

$$\begin{aligned} J &= \frac{A_p}{A_{n2}} = \frac{6,314}{51,84} \\ &= 0,122 \quad (\text{use } J = 0,1 \text{ from Figure 8.4}) \end{aligned}$$

Eccentricity

$$\begin{aligned} e &= 1 - \frac{Q_a}{Q_c} = 1 - \frac{12,14}{17,17 + (120,69) \left( 1 - \frac{17}{17 + 1,61} \right)} \\ &= 0,56 \end{aligned}$$

From **Figure 8.4** and with  $\theta = 15^\circ$ :

$$K^* = 0,80$$

Approximate backwater (to estimate  $A_1$  in Equation 8.1):

$$h_1^* = K^* \alpha_2 \frac{\bar{v}_{n2}^2}{2g} + \alpha_1 \left\{ \left( \frac{A_{n2}}{A_4} \right)^2 - \left( \frac{A_{n2}}{A_1} \right)^2 \right\} \frac{\bar{v}_{n2}^2}{2g} \quad \dots(8.1)$$

where:

$$\begin{aligned} K^* &= \text{secondary energy loss coefficient} \\ \alpha_1, \alpha_2 &= \text{velocity coefficients} \\ \bar{v}_{n2} &= \frac{Q}{A_{n2}} \text{ (m/s) where } Q = \text{design discharge (m}^3/\text{s)} \\ A_{n2} &= \text{projected flow area at constricted section 2 below normal water level of the river section (m}^2\text{)} \\ A_1 &= \text{flow area at section 1, including the influence of the backwater on the flow depth (m}^2\text{)} \\ A_4 &= \text{flow area at section 4 (m}^2\text{)} \end{aligned}$$

$$\begin{aligned} \bar{v}_{n2} &= \frac{Q}{A_{n2}} = \frac{150}{51,84} \\ &= 2,894 \text{ m/s} \end{aligned}$$

$$h_1^{*1} = K^* \alpha_2 \frac{\bar{v}_{n2}^2}{2g} \quad \dots(8.2)$$

$$\begin{aligned} h_1^{*1} &= (0,80)(1,15) \frac{(2,894)^2}{2(9,81)} \\ &= 0,393 \text{ m} \end{aligned}$$

$$\begin{aligned} A_1 &= A_n + h_1^{*1} B = (86,3) + (0,393)(36,06) \\ &= 100,46 \text{ m}^2 \end{aligned}$$

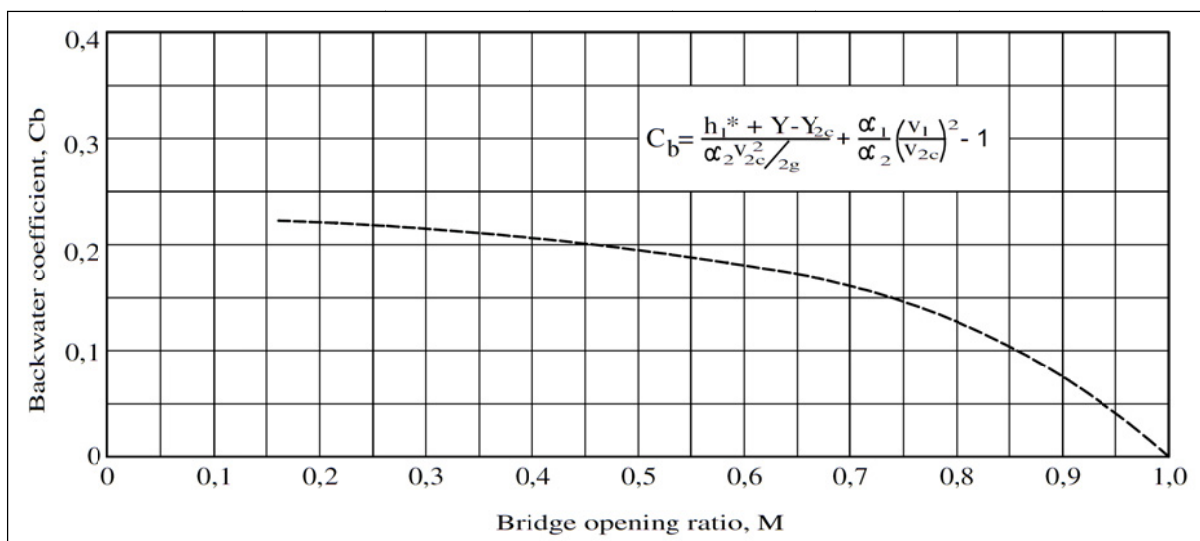
Final estimate of backwater (Equation 8.1):

$$\begin{aligned}
 h_1^{*1} &= h_1^{*1} + \alpha_1 \left[ \left( \frac{A_{n2}}{A_4} \right)^2 - \left( \frac{A_{n2}}{A_1} \right)^2 \right] \frac{\bar{v}_{n2}^2}{2g} \\
 &= (0,393) + (1,20) \left[ \left( \frac{51,84}{86,30} \right)^2 - \left( \frac{51,84}{100,46} \right)^2 \right] \left[ \frac{(2,894)^2}{2(9,81)} \right] \\
 &= \mathbf{0,441 \text{ m}}
 \end{aligned}$$

**For Type II flow:**

$$\begin{aligned}
 b_c &= (b - \sum W_p) = 17,0 - 2,0 \\
 &= 15,0 \text{ m}
 \end{aligned}$$

$$C_b = 0,152 \text{ from Figure 8.5}$$



**Figure 8.5: Estimation of the Backwater Coefficient,  $C_b$**

$$\begin{aligned}
 y_{2c} &= \left( \frac{Q^2}{g b_c^2} \right)^{\frac{1}{3}} = \left( \frac{(150)^2}{(9,81)(15)^2} \right)^{\frac{1}{3}} \\
 &= 2,168 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 \bar{y} &= \frac{A_{n2}}{b} = \frac{51,84}{17} \\
 &= 3,050 \text{ m}
 \end{aligned}$$

In 1<sup>st</sup> iteration, assume

$$\begin{aligned}
 \bar{v}_1 &= \frac{Q}{A_n} = \frac{150}{86,30} \\
 &= 1,738 \text{ m/s}
 \end{aligned}$$

for  $\bar{v}_{2c}$  based on the net width:

$$\begin{aligned}
 \bar{v}_{2c} &= (g y_{2c})^{0,5} = ((9,81)(2,168))^{0,5} \\
 &= 4,612 \text{ m/s}
 \end{aligned}$$

$$\begin{aligned}
h_1^{*1} &= \alpha_2 \frac{\bar{v}_{2c}^2}{2g} (C_b + 1) - \alpha_1 \frac{\bar{v}_1^2}{2g} + y_{2c} - \bar{y} \\
&= \frac{(1,15)(4,612)^2(0,152+1)}{2(9,81)} - \frac{(1,20)(1,738)^2}{2(9,81)} + (2,168) - (3,050) \\
&= 1,436 - 0,185 + 2,168 - 3,050 \\
&= \mathbf{0,371 \text{ m}}
\end{aligned}$$

Adjust result for improved value of  $\bar{v}_1$ :

$$\begin{aligned}
A_1 &= A_n + h_1^{*1} B = (86,30) + (0,371)(36,06) \\
&= 99,67 \text{ m}^2
\end{aligned}$$

$$\begin{aligned}
\bar{v}_1 &= \frac{150}{99,67} \\
&= 1,505 \text{ m/s}
\end{aligned}$$

$$\begin{aligned}
h_1^{*1} &= 1,436 - \frac{(1,20)(1,505)^2}{2(9,81)} + 2,168 - 3,050 \\
&= \mathbf{0,417 \text{ m}}
\end{aligned}$$

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 **0,417 m** which is less than the backwater calculated for Type I flow, thus Type I flow prevails i.e.  $h_1^{*1} = \mathbf{0,441 \text{ m}}$ .

Note that this example was also modelled in HEC-RAS (provided on the supporting flash drive) 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 441 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.

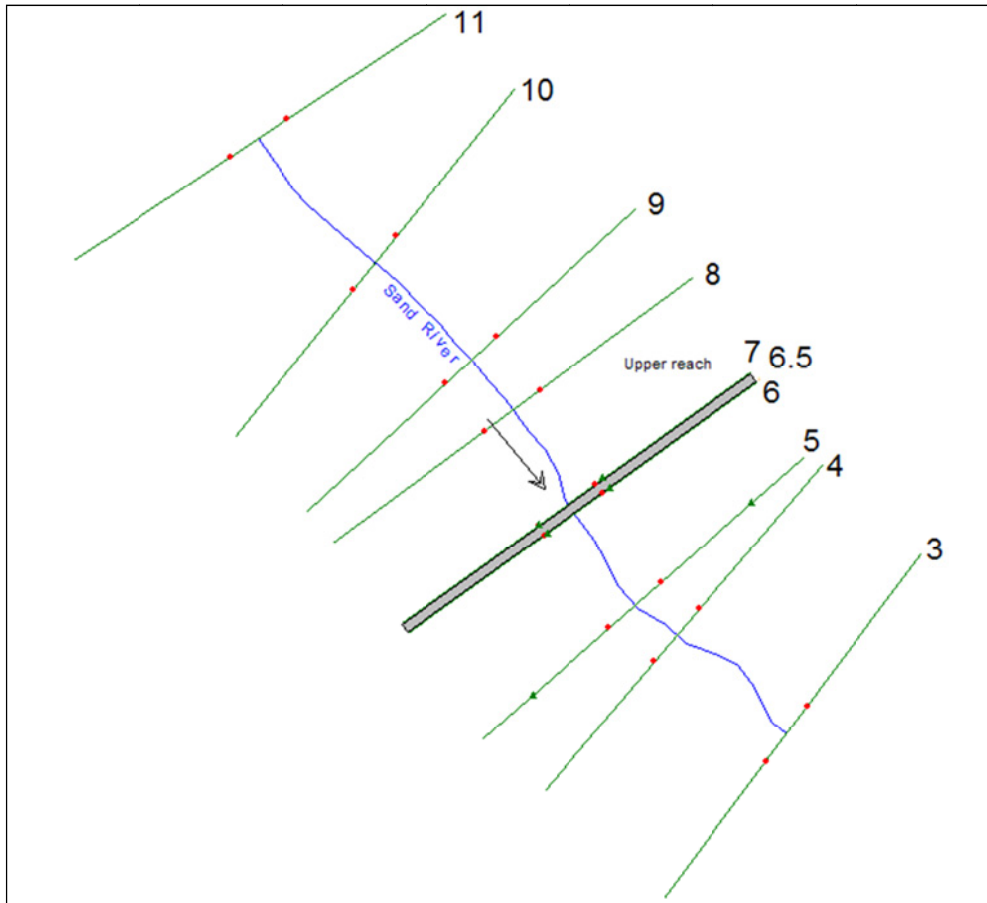
Users of HEC-RAS should therefore carefully consider the option where the bridge approach conditions are “smoothed”, thereby reducing the backwater.

## 8.2 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 with the data files as Example2.prj.

**Figure 8.6** 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 8.6: 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 8.7**.

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 m<sup>3</sup>/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.

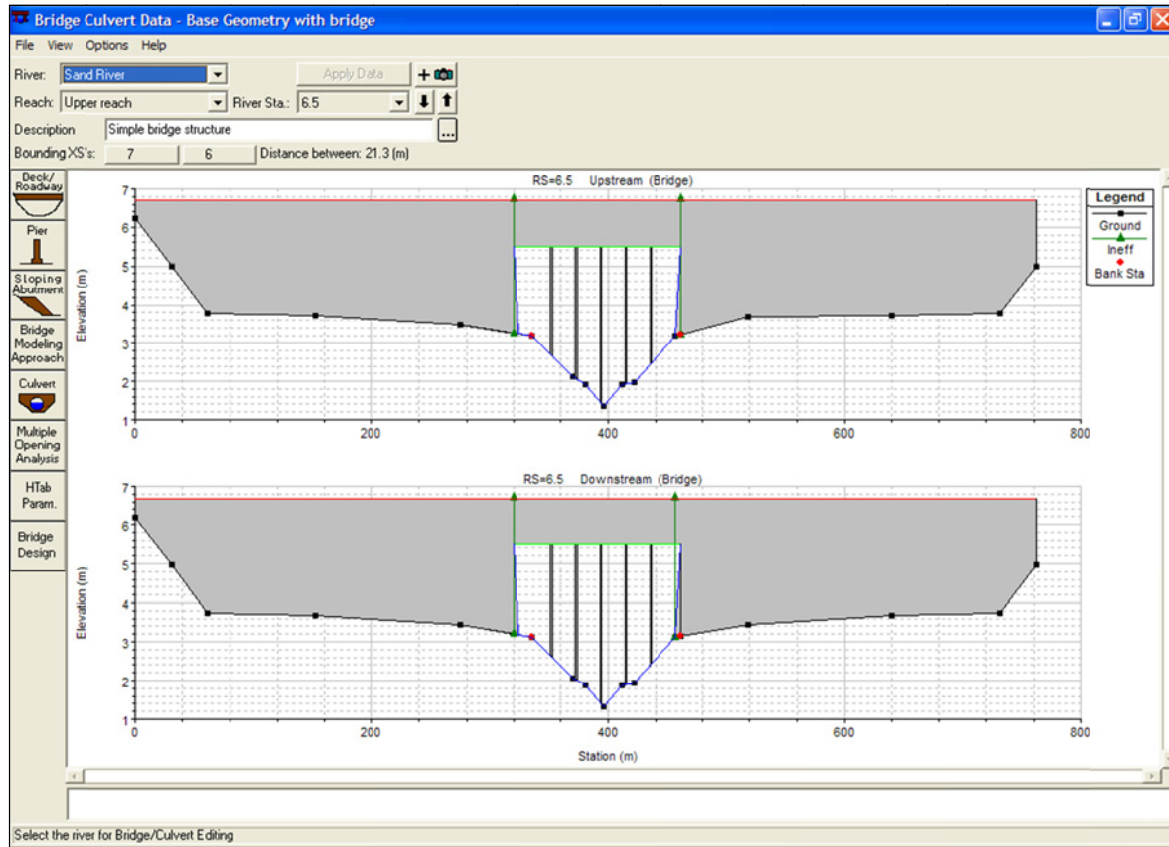
#### ***Bed material characteristics***

The sieve analyses (percentage passing) of the bed material revealed the following:

$$D_{50} = 0,0020 \text{ m}$$

$$D_{90} = 0,0045 \text{ m}$$





**Figure 8.7: Upstream and downstream bridge cross-section from the HEC-RAS analysis**

### *Cross-section details*

*The cross-section details are given in the Table 8.3 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.*

### *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.

### **Solution Example 8.2**

For this analysis the design flood discharge of  $850 \text{ m}^3/\text{s}$  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  $850/126,61 = 6,713 \text{ m}^3/\text{s}$ .

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 8.3: Details of cross-section 6.5 (Obtained from HEC-RAS analysis)**

Section	$y_n$ (m)	Area (m <sup>2</sup> )	Wetted perimeter (m)	Flow rate (m <sup>3</sup> /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
<b>Total</b>		<b>654,10</b>	<b>698,32</b>	<b>850,00</b>

$R = 0,937$  m and  $\bar{v} = 1,299$  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:

$$B = 14Q^{0,5} D_{50}^{0,25} F_s^{-0,5} \quad \dots(8.3)$$

where:

- B = mean channel width (m)
- y = mean depth of flow (m)
- Q = equivalent steady discharge which would generate the channel geometry (m<sup>3</sup>/s)
- q = discharge per unit width (Q/B) (m<sup>3</sup>/s.m) (Note: To estimate channel geometry conditions under flood conditions the design flood flow may be used.)<sup>(8.9)</sup>
- $D_{50}$  = median size of bed material (m)
- $F_s$  = side factor to describe bank resistance to scour (**Table 8.4**)

**Table 8.4: Side factors**

Bank type	Value of $F_s$
Sandy loam	0,1
Silty clay loam	0,2
Cohesive banks	0,3

With  $F_s = 0,1$  from **Table 8.4** for sandy loam, the width B can be calculated.

$B = 273$  m, which is wider than the proposed bridge of 126,61 m.

Use Equation 8.4 to determine the mean flow depth at the equilibrium width:

$$y = 0,38q^{0,67} D_{50}^{-0,17} \quad \dots(8.4)$$

where:

- y = mean depth of flow (m)
- $D_{50}$  = median size of bed material (m)

$$q = 850/273 = 3,114 \text{ m}^3/\text{s.m}$$

Mean depth  $y = 2,34 \text{ m}$ . The maximum depth,  $Y_{\max} = 1,25y = 2,92 \text{ m}$ .

The maximum live-bed depth,  $Y_{\max}$ , is slightly less than the fixed bed depth,  $y_n$  of  $2,98 \text{ m}$ , 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 8.4 for the case of an alluvial channel) or the **contraction equations** (Equations 8.5 with 8.6 or 8.7).

The depth of scour is given by:

$$d_s = (y_2 - y_1) + (1 + K) \left( \frac{\bar{v}_2^2 - \bar{v}_1^2}{2g} \right) \quad \dots(8.5)$$

*Sediment-laden flow*

$$\frac{y_2}{y_1} = \left( \frac{Q_t}{Q_c} \right)^{6/7} \left( \frac{B_1}{B_2} \right)^{2/3} \left( \frac{n_2}{n_1} \right)^{1/3} \quad \dots(8.6)$$

*Clear water flow*

$$y_2 = \left[ \frac{Q^2}{40D_m^{2/3} B_2^2} \right]^{3/7} \quad \dots(8.7)$$

First apply the **regime equation** on the reduced width of  $126,61 \text{ m}$ . In this case  $q = 850/126,61 = 6,714 \text{ m}^3/\text{s.m}$  leading to a mean flow depth,  $y$  of  $3,915 \text{ m}$ . From **Table 8.5**,  $y_{\max}$  can be determined as follows:  $y_{\max} = 1,25 \times 3,915 = 4,893 \text{ m}$ . This reflects a scour depth,  $d_s = 4,893 - 2,98 = 1,913 \text{ m}$

**Table 8.5: Factors to convert mean flow depth (y) to maximum channel depth**

Description	Multiplying factor
Straight reach of channel	1,25 <sup>(*)</sup>
Moderate bend	1,50
Severe bend	1,75
Right-angled abrupt turn	2,00

Note: \* Neill recommends that this factor be increased to 1,50 in cases where dune movement takes place on the riverbed.

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 8.8.

$$V_* = \sqrt{gDS} = \sqrt{9,81(2,98)(0,002)} = 0,242 \text{ m/s} \quad \dots(8.8)$$

and the term,  $V_*D_{50}/\nu = 483 \gg 13$ , thus in turbulent flow region (see **Figure 8.8**).

The critical shear velocity is:

$$\frac{V_{*c}}{V_{ss}} = 0,12 \quad \dots(8.9)$$

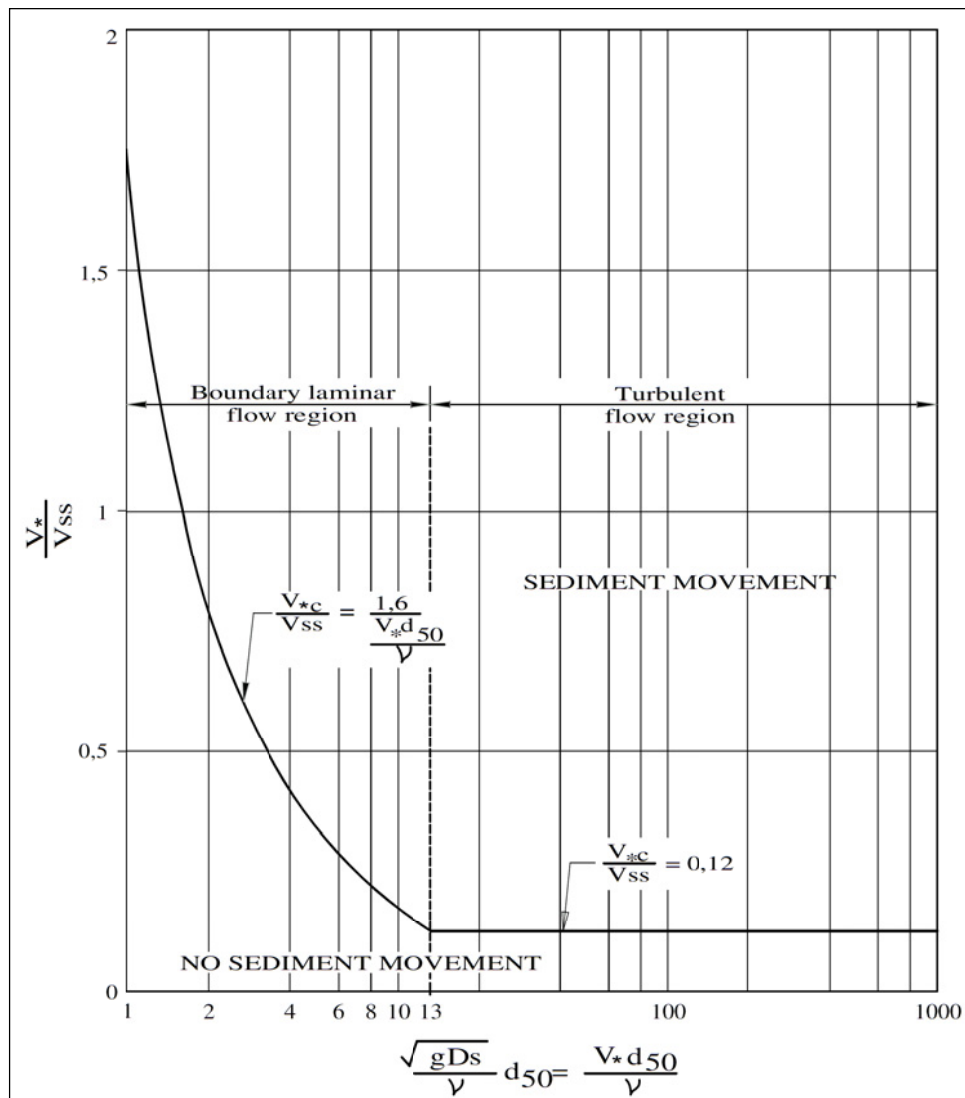
The settling velocity,  $V_{ss}$  can be obtained from **Figure 8.9** for the representative particle,  $D_{50}$ , and the relative density of 2,65, it follows:

$$V_{ss} = 0,24 \text{ m/s, and } V_{*c} = 0,029 \text{ m/s}$$

The velocity at the boundary between sediment movement and no sediment movement (the 'critical' velocity),  $V_c$ , is determined from the logarithmic relationship:

$$V_c = 5,75V_{*c} \log \frac{12R}{k_s} \quad \dots(8.10)$$

$$\text{From Equation 8.10: } V_c = 5,75V_{*c} \log \frac{12R}{k_s} = 5,75 \left( 0,029 \log \left( 12 \frac{(0,937)}{(0,002)} \right) \right) = 0,625 \text{ m/s}$$



**Figure 8.8: Modified Lui Diagram showing the relationships for incipient sediment movement**

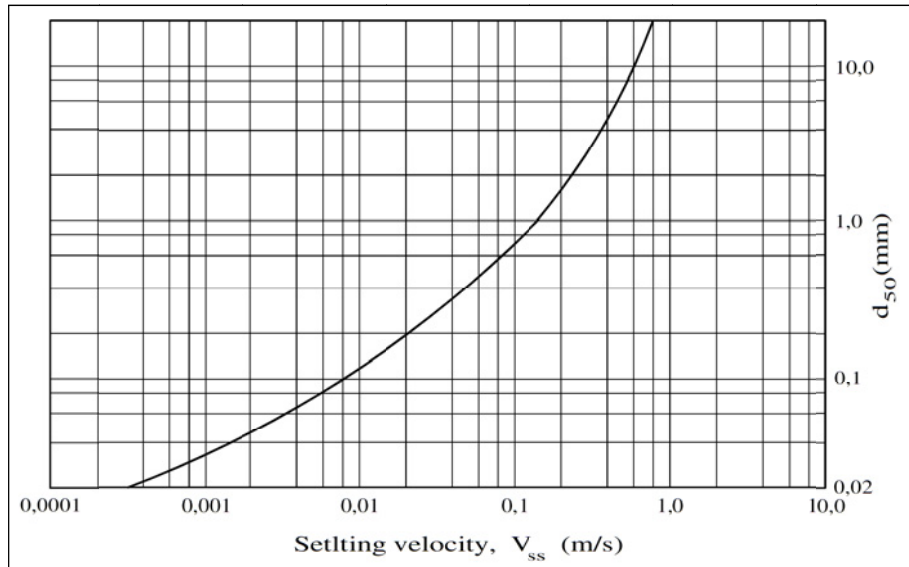


Figure 8.9: Settling velocity as a function of the sediment size (Shape factor not taken into consideration) <sup>(8.13)</sup>

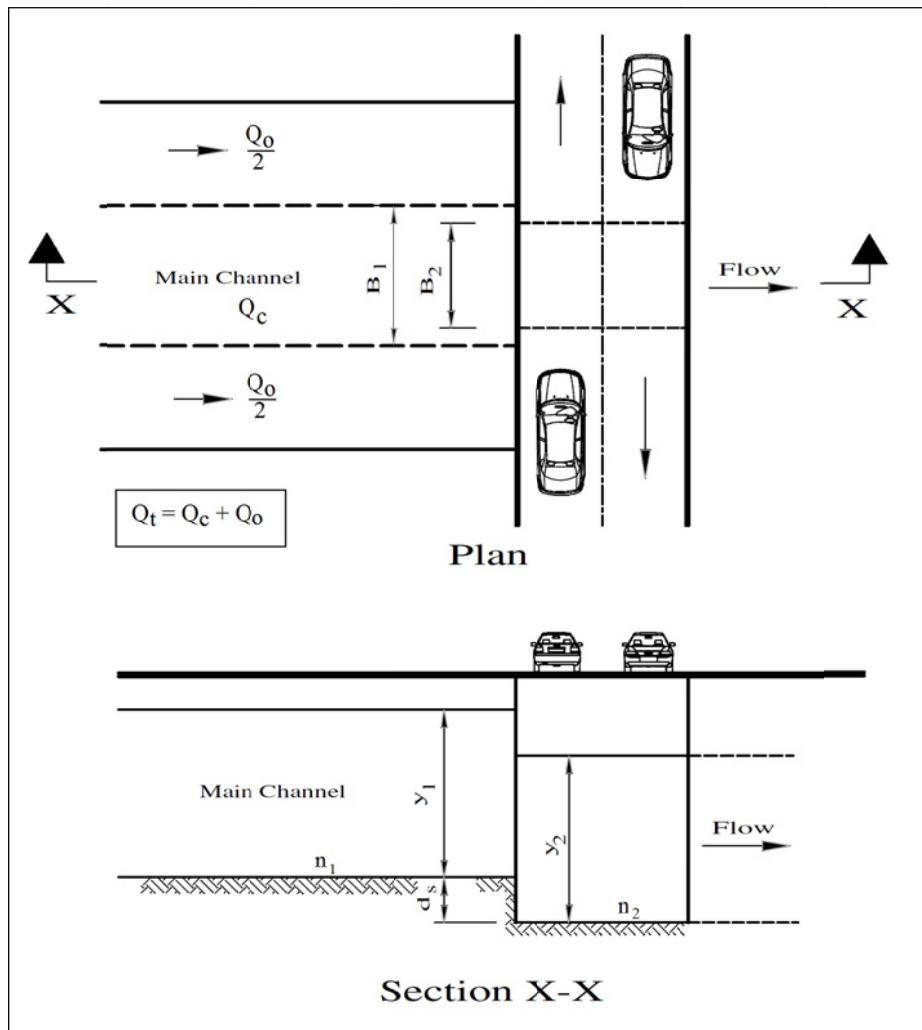


Figure 8.10: Long constriction in sediment-laden flow: definition of terms

The average approach flow velocity of 1,299 m/s > 'critical' velocity of 0,625 m/s, thus sediment will be entrained and Equation 8.11 together with **Figure 8.10** can be used to estimate contraction scour.

$$\frac{y_2}{y_1} = \left( \frac{Q_t}{Q_c} \right)^{6/7} \left( \frac{B_1}{B_2} \right)^{2/3} \left( \frac{n_2}{n_1} \right)^{1/3} \quad \dots(8.11)$$

$$\frac{y_2}{y_1} = \left( \frac{850}{542,6} \right)^{6/7} = 1,469 \quad (\text{widths and } n\text{-values are equal for these sections})$$

$$y_2 = (1,469)(2,98) = 4,378 \text{ m}$$

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 m<sup>2</sup>, calculated as follows (4,378 x (126,61 – 5(1,5))). This is larger than the upstream main channel area of 258,73 m<sup>2</sup> (**Table 8.3**), and thus the **flow is expanding**. Equation 8.12 is used to determine the contraction scour depth.

$$d_s = (y_2 - y_1) + (1 + K) \left( \frac{\bar{v}_2^2 - \bar{v}_1^2}{2g} \right) \text{ and with } K = 1 \text{ for a sudden transition} \quad \dots(8.12)$$

$$d_s = (4,378 - 2,98) + (1 + 1,0) \left( \frac{1,63^2 - 1,299^2}{2(9,81)} \right)$$

$$d_s = 1,468 \text{ m}$$

This scour depth (1,468 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 8.13 and 8.14 to compute local scour in two different ways for alluvial channels (cohesionless material). Obtain the factors needed for Equation 8.13 from **Table 8.6** and **Table 8.7**. Obtain the factors needed for Equation 8.14 from **Table 8.6** to **Table 8.8** and  $K_4$  in cases where armouring is applicable. Compare answers obtained from Equations 8.13 and 8.14 and **select a conservative answer using good engineering judgement**.

**Table 8.6: Correction factor  $K_1$ , for pier nose shape**

Shape of pier in plan #	Length/Width ratio (L/b)	$K_1$
Circular	1,0	1,0
Lenticular	2,0	0,91
	3,0	0,76
	4,0	0,67-0,73
	7,0	0,41
Parabolic nose		0,8
Triangular 60°		0,75
Triangular 90°		1,25
Elliptic	2,0	0,91
	3,0	0,83
Ogival	4,0	0,86-0,92
Rectangular	2,0	1,11
	4,0	1,11 (HEC 18) - 1,40 (F&C)
	6,0	1,11

Note: # Table 8.6 is based on the list by Faraday and Charlton (1983), which is more complete than the list in HEC 18 documentation

**Table 8.7: Correction factor  $K_2$ , for angle of attack of the flow**

Angle (skew angle of flow)	L/b = 4	L/b = 8	L/b = 12
0	1,0	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 8.8: 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	$\geq 9$ m	1,3

From Equation 8.13, with depth  $y_0$  in the bridge section as determined from regime:

$$d_s = 1,8y_0^{0,75}b^{0,25} - y_0 \quad \dots(8.13)$$

where:

- $d_s$  = local scour depth at pier (m)
- $y_0$  = depth upstream of pier (m)
- $b$  = pier width (m)

$$d_s = 1,8(3,915^{0,75})(1,5^{0,25}) - 3,915$$

$$d_s = 1,629 \text{ m}$$

Note that the scour level is  $(3,915 + 1,629) = 5,544$  m below the design flood level.



Alternatively Equation 8.14, for the longest piers close to the minimum river invert could be used to calculate the local scour depth at the pier.

$$\frac{y_s}{b} = 2,0 K_1 K_2 K_3 K_4 \left( \frac{y_1}{b} \right)^{0,35} Fr_1^{0,43} \quad \dots(8.14)$$

with

- $b = 1,5 \text{ m}$
- $y_1 = 2,98 \text{ m}$  (normal flow depth upstream of the bridge, **Table 8.3**)
- $Fr_1 = 0,429$  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), see Drainage Manual for details of calculating  $K_4$  correction factor, then

$$\frac{y_s}{b} = (2,0)(1,0)(1,0)(1,1)(1,0) \left( \frac{2,98}{1,5} \right)^{0,35} (0,429)^{0,43} = 1,943$$

$$y_s = 2,915 \text{ m}$$

Note that the scour level is  $(2,98 + 2,915) = 5,895 \text{ m}$  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 8.13.

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 8.9** reflects a summary of all the calculated scour depths.

**Table 8.9: 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	1,468
Local scour	Piers	2,915
	Abutments	2,102
Total expected scour	Piers	4,828
	Abutments	4,013

## Review of the contraction or short-term scour using different analyses procedures

The potential general scour at the bridge has been determined in (i) using Equations 8.5. 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 1,468 m using Equation 8.12 is too conservative and hence discarded.

Based on the summary in **Table 8.9** 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 + 2,915 = 4,828 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.**

*Verify the scour depth with the method based on the principle of applied stream power*

For total scour at piers in alluvial rivers, check the answer against values obtained by means of Equation 8.15 that is based on the principle of applied stream power.

**Equation 8.15 reflected the following relationship:**

$$\frac{C(Y_t)(v_{ss} k_s)^{1/3}}{q\sqrt{g}} = F \quad \dots(8.15)$$

**With  $F = 0,8$ ;  $k_s = 0,002 \text{ m}$  ;  $v_{ss} = 0,24 \text{ m/s}$ ;  $q = 542,6/126,61 = 4,29 \text{ m}^3/\text{s.m}$  and  $C$  calculated from the Chezy relationship for total section,  $C = 67,5$ ; it follows that:**

**$Y_t = 2,03 \text{ m}$  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 in the supporting data files.

## 9 STORMWATER ANALYSES AND DESIGN

### 9.1 Example 9.1 – Pipe flow

#### Problem description Example 9.1

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 9.1

To solve this problem the conservation of energy principle will be used i.e. Equation 9.1. 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.

$$\frac{\bar{v}_1^2}{2g} + \frac{p_1}{\gamma} + z_1 = \frac{\bar{v}_2^2}{2g} + \frac{p_2}{\gamma} + z_2 + h_{f_{1-2}} + h_{l_{1-2}} \quad \dots(9.1)$$

The velocity in the catch pit ( $\bar{v}_1$ ) 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 9.2) with the C-value determined by Equation 9.3. The absolute roughness value of the pipe was estimated as  $k_s = 0,0004$  m.

$$h_{f_{1-2}} = \frac{\bar{v}^2 L}{C^2 R} \quad \dots(9.2)$$

$$C = 5,75 \sqrt{g} \log\left(\frac{12R}{k_s}\right) \text{ or } C = 5,75 \sqrt{g} \log\left(\frac{12\left(\frac{D}{4}\right)}{k_s}\right) \text{ for circular pipes} \quad \dots(9.3)$$

$$C = 5,75 \sqrt{9,81} \log\left(\frac{12\left(\frac{(0,3)}{4}\right)}{0,0004}\right) = 60,37$$

$$h_{f_{1-2}} = \frac{\bar{v}_2^2 (20)}{(60,37)^2 \left(\frac{0,3}{4}\right)} = 0,07317 \bar{v}_2^2$$

A secondary inlet loss will occur at the entrance. From **Table 9.1**, an inlet coefficient of  $k = 0,5$  will be used for a blunt entrance.

**Table 9.1: Transition losses in pipe flows**

Description	Sketch	k-value
<u>Inlets</u> $h_1 = \frac{k\bar{v}^2}{2g}$ $(\bar{v} = \text{average velocity in conduit})$	Protruding	0,9
	Oblique	0,7
	Blunt	0,5
	Well-rounded	0,2
<u>Diverging sections</u> $h_1 = \frac{k(\bar{v}_1 - \bar{v}_2)^2}{2g}$	Sudden	1,0
	Cone $45^\circ < \theta < 180^\circ$	1,0
	$\theta = 30^\circ$	0,7
	$\theta = 15^\circ$	0,2
<u>Converging sections</u> $h_1 = \frac{k\bar{v}^2}{2g}$	Sudden	0,5
	Cone	0,25
<u>Bends</u> $h_1 = \frac{k\bar{v}_2^2}{2g}$	$\theta = 90^\circ$	0,4
	$\theta = 45^\circ$	0,3
<u>Outlets</u> $h_1 = \frac{k\bar{v}_1^2}{2g} \left(1 - \frac{A_1}{A_2}\right)^2$	Sudden	1,0

The diameter does not change and the velocity where the water enters the pipe can be assumed to be equal to  $\bar{v}_2$ .

$$h_1 = \frac{k\bar{v}_2^2}{2g} = \frac{(0,5)\bar{v}_2^2}{2(9,81)} = 0,02548\bar{v}_2^2$$

The energy equation is simplified as follows:

$$\frac{(0)^2}{2(9,81)} + 0 + (0,3) = \frac{\bar{v}_2^2}{2(9,81)} + 0 + 0 + (0,07317\bar{v}_2^2) + (0,02548\bar{v}_2^2)$$

$$\bar{v}_2 = 1,416 \text{ m/s}$$

$$Q = \bar{v}_2 A_2 = (1,416) \left( \pi \left( \frac{0,3}{2} \right)^2 \right) = 0,1 \text{ m}^3/\text{s}$$

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.

## 9.2 Example 9.2 – Introduction to using EPASWMM

The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model that computes runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of subcatchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps and regulators. SWMM tracks the quantity and quality of runoff generated within each subcatchment and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps.

SWMM was first developed in 1971 and since then has undergone several major upgrades. It continues to be widely used throughout the world for planning, analysis, and design related to storm water runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas and has also been used for modelling non-urban areas. The most current implementation of the model is version 5.0 which was released in 2005. It has modernized both the model's structure and its user interface, making SWMM easier to use and more accessible to a new generation of hydrologists, engineers, and water resources management specialists.

All the practical exercises after this first tutorial are developed for the same catchment area and each one builds in some degree on the results of a previous example. Therefore, it is recommended to begin with Exercise 1 and work sequentially through up to Exercise 6, while hopefully building the required input data files and running them with SWMM for each exercise. These files, as well as the backdrop image files that are needed to complete the exercises, are available on the flash disk.

**Goal:** This first tutorial provides an introduction to using EPA SWMM, Version 5, for modeling the quantity of storm water runoff produced from an urban area. The topics to be covered include:

- Project Setup
- Constructing a SWMM Model
- Saving and Opening Projects
- Setting the Properties of SWMM Objects
- Running a Single Event Analysis
- Viewing Simulation Results

In the exercises the decimal separator will be the decimal comma (,) whilst in the EPA SWMM program the decimal point (.) is used. The EPA SWMM program is constantly upgraded and the screen layouts on newer versions may differ from what is shown in this example.

### PROJECT SETUP

To begin this tutorial, start the EPA SWMM program by double clicking the EPA SWMM icon on the desktop. The main window should appear as shown in **Figure 9.1**.

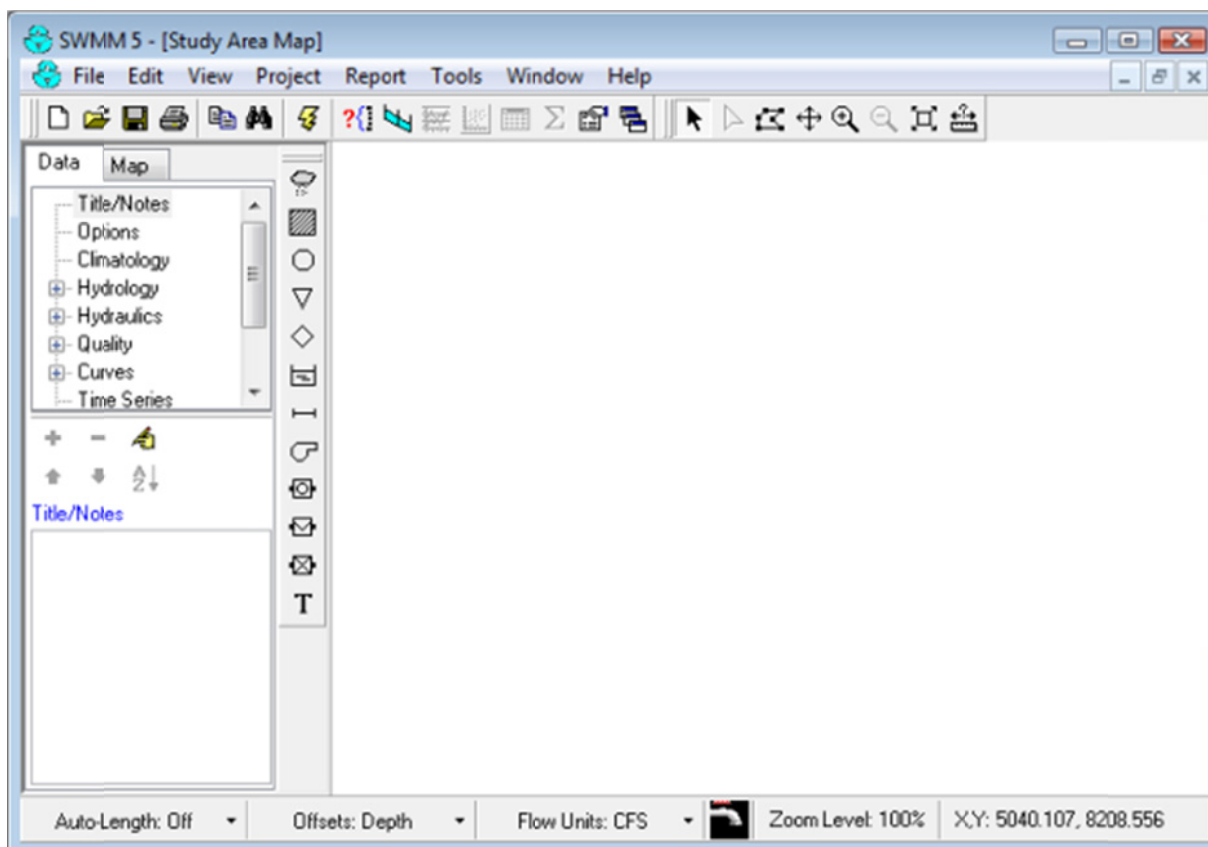


Figure 9.1: SWMM 5 main window

In this tutorial a drainage system serving a 0,48 ha residential area will be modelled. The system layout is shown below in **Figure 9.2** and consists of subcatchment areas S1 through S3, storm sewer conduits C1 through C4, and conduit junctions J1 through J4. The system discharges to a stream at the point labelled Out1. The first steps are to create the objects shown in this diagram on SWMM's Study Area Map and setting the various properties of these objects. The water quantity response to a 75 mm, 6-hour rainfall event, as well as a continuous, multi-year record will then be simulated.

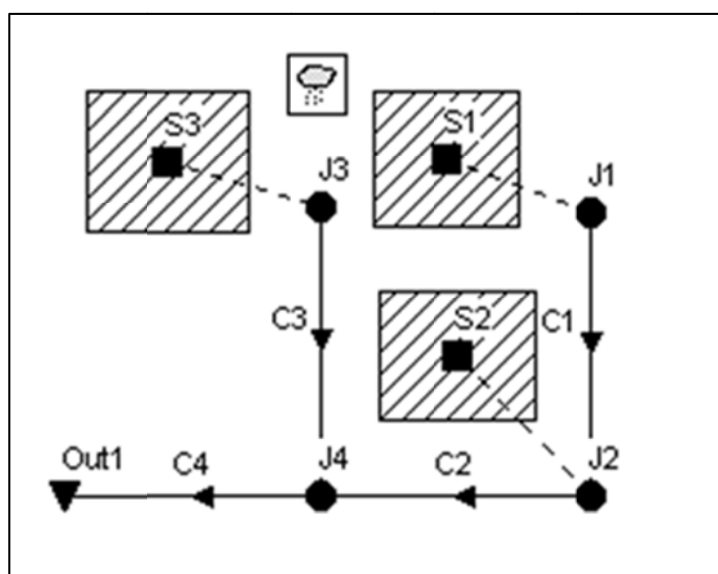


Figure 9.2: System layout

Our first task is to create a new project in EPA SWMM and make sure that certain default options are selected. Using these defaults will simplify the data entry tasks later on. The first step in developing a SWMM application is to start a new project.

1. Go to **File** menu on the main window and select **New**.
2. Select **Project >> Defaults** to open the **Project Defaults** dialog.
3. On the **ID Labels** page, set the **ID Prefixes** as follows (leave the others blank) and shown in **Figure 9.3**:  
 Rain Gages: *Gage*  
 Subcatchments: *S*  
 Junctions: *J*  
 Outfalls: *Out*  
 Conduits: *C*  
 ID Increment: *1*



**Figure 9.3: Project Defaults: ID labels**

This will make EPA SWMM automatically label new objects with consecutive numbers following the designated prefix.

4. On the **Subcatchments** page of the dialog set the following default values (as shown in **Figure 9.4**):  
 Area: *1,619*  
 Width: *120*  
 % Slope: *0,5*  
 % Imperv: *50*  
 N-Imperv: *0,01*  
 N-Perv: *0,10*  
 Dstore-Imperv: *1,3*  
 Dstore-Perv: *1,3*  
 %Zero-Imperv: *25,0*





**Figure 9.4: Project Defaults: Subcatchments**

Infiltration Model <click ... to edit>, as shown in **Figure 9.5**.

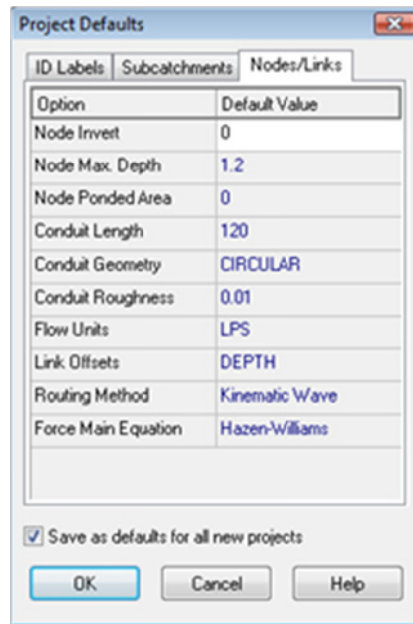
Method: *Green-Ampt*  
 - Suction Head: *90*  
 - Conductivity: *12,5*  
 - Initial Deficit: *0,26*



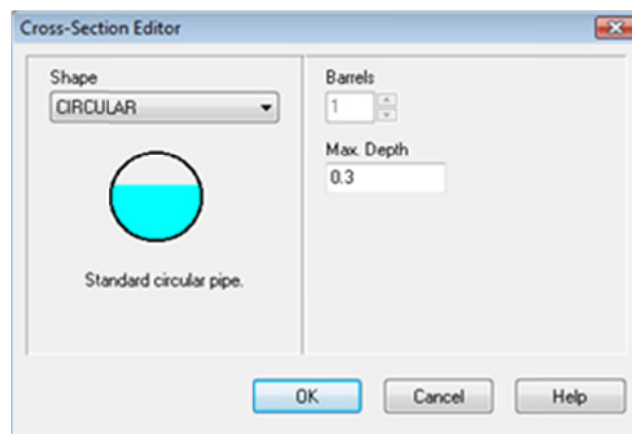
**Figure 9.5: Infiltration Editor**

- On the **Nodes/Links** page set the following default values (**Figure 9.6**):

Node Invert: *0*  
 Node Max. Depth: *1,2*  
 Node Poned Area: *0*  
 Conduit Length: *120*  
 Conduit Geometry: <click ... to edit >, as shown in **Figure 9.7**.  
 - Shape: *Circular*  
 - Max. Depth: *0,3*  
 - Barrels *1*  
 Conduit Roughness: *0,01*  
 Flow Units: *LPS*  
 Routing Model: *Kinematic Wave*  
 Force Main Equation: *Hazen-Williams*



**Figure 9.6: Project Defaults: Nodes/Links**



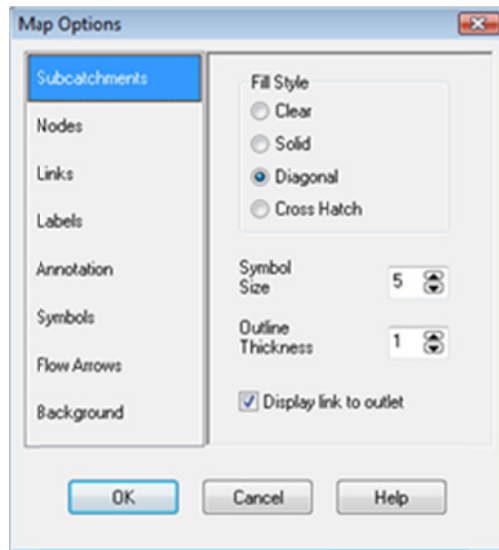
**Figure 9.7: Cross-Section Editor**

6. Select *Save as defaults for all new projects* and click **OK** to accept these choices and close the dialog.

### **Setting Map Options**

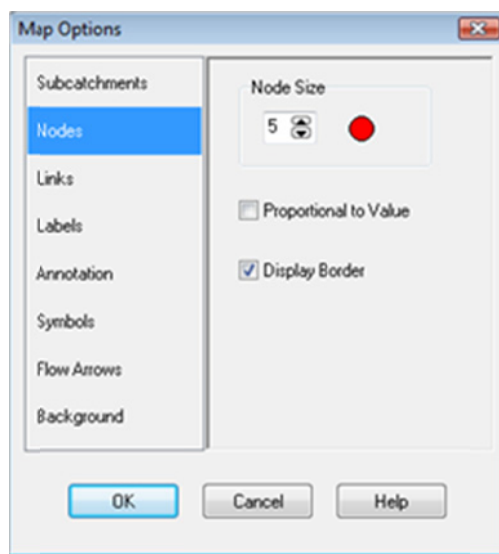
Next we will set some map display options so that ID labels and symbols will be displayed as we add objects to the study area map, and links will have direction arrows.

1. Select **Tools >> Map Display Options** to bring up the **Map Options** dialog (see **Figure 9.8**).



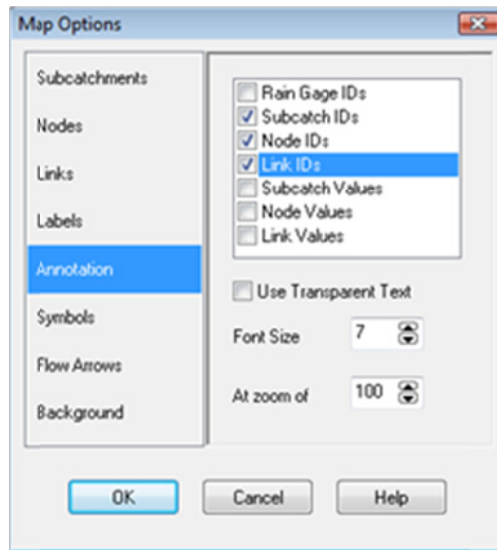
**Figure 9.8: Map Options: Subcatchments**

2. Select the **Subcatchments** page, set the **Fill Style** to *Diagonal* and the **Symbol Size** to 5 (as shown in **Figure 9.8**).
3. Then select the **Nodes** page and set the **Node Size** to 5 (see **Figure 9.9**).



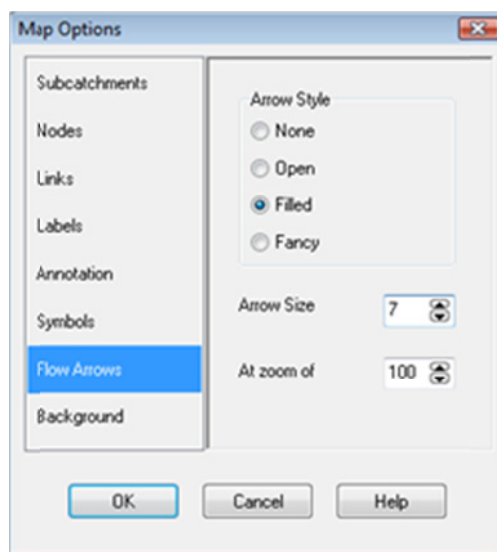
**Figure 9.9: Map Options: Nodes**

4. Select the **Annotation** page and check off the boxes that will display ID labels for **Subcatchments**, **Nodes**, and **Links**. Leave the others un-checked as shown in **Figure 9.10**.



**Figure 9.10: Map Options: Annotation**

5. Finally, select the **Flow Arrows** page; select the *Filled Arrow* style, and set the **Arrow size** to 7 (see **Figure 9.11**).



**Figure 9.11: Map Options: Flow Arrows**

6. Click the **OK** button to accept these choices and close the dialog.

Before placing objects on the map we should set its dimensions.

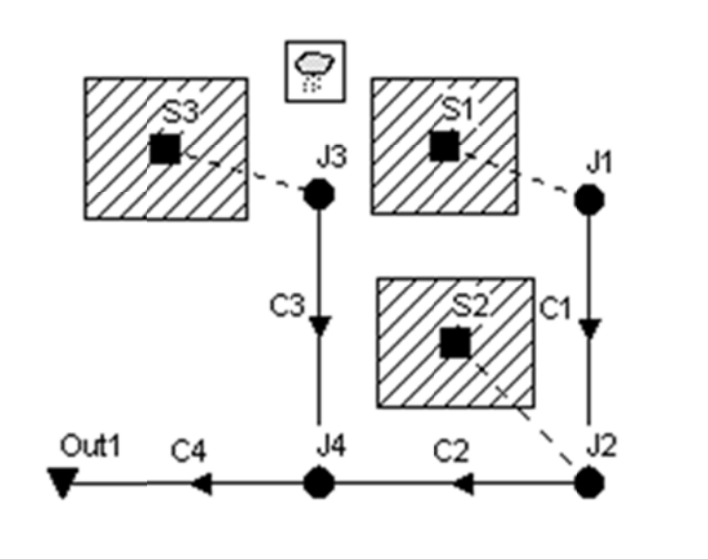
1. Select **View | Dimensions ...** to bring up the **Map Dimensions** dialog.
2. You can leave the dimensions at their default values for this example. Set the **Map Units** to *Meters* by selection this option and click on the **OK** button.

Finally, look in the status bar at the bottom of the main window and check that the **Auto-Length** feature is *Off*. If it is on, then click the down arrow button and select "*Auto-Length: Off*" from the pop-up menu that appears.


## CONSTRUCTING A SWMM MODEL

### Drawing the Drainage Area Subcatchments

We are now ready to begin adding components to the **Study Area** Map to create the model layout as shown in **Figure 9.12**. We will start with the subcatchments.



**Figure 9.12: Model layout**

1. Begin by clicking the  button on the **Object Toolbar**. (If the toolbar is not visible then select **View | Toolbars | Object**). Notice how the mouse cursor changes shape to a pencil.
2. Move the mouse to the map location where one of the corners of subcatchment S1 lies and left-click the mouse.
3. Do the same for the next three corners and then right-click the mouse (or hit the Enter key) to close up the rectangle that represents subcatchment S1. You can press the Esc key if instead you wanted to cancel your partially drawn subcatchment and start over again. Don't worry if the shape or position of the object isn't quite right. We will go back later and show how to fix this.
4. Repeat this process for subcatchments S2 and S3. Note.

Observe how sequential ID labels are generated automatically as we add objects to the map. The model should look similar to the one shown in **Figure 9.13**.

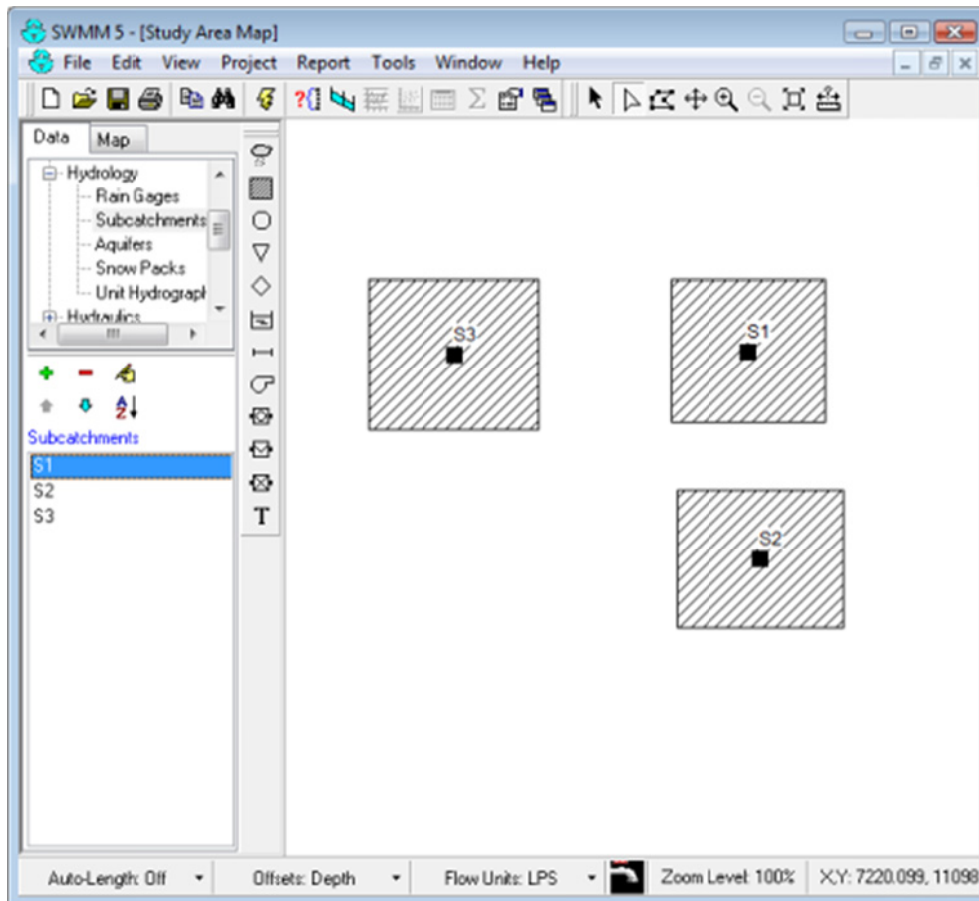




Figure 9.13: Model with the three subcatchments

### Drawing the Drainage System Nodes

Next we will add in the junction nodes and the outfall node that comprise part of the drainage network.

1. To begin adding junctions, click the  button on the **Object Toolbar**.
2. Move the mouse to the position of junction *J1* (as shown in **Figure 9.12**) and left-click it. Do the same for junctions *J2* through *J4*.
3. To add the outfall node, click the  button on the **Object Toolbar**, move the mouse to the outfall's location on the map, and left-click. Note how the outfall was automatically given the name *Out1*.

The model should look similar to the one shown in **Figure 9.14**.

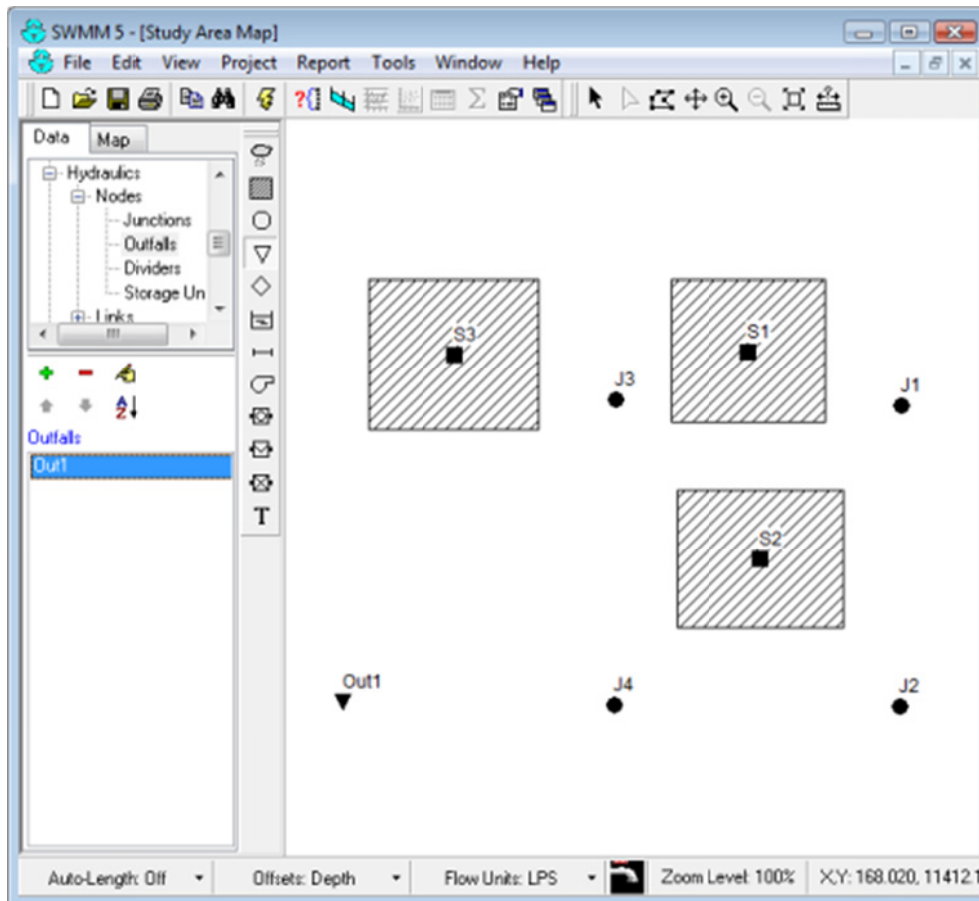



Figure 9.14: Model with added junctions and outlet

### Drawing the Drainage System Links

Now we will add the storm sewer conduits that connect our drainage system nodes to one another. (You must have created a link's end nodes as described in the previous topic before you can create the link.) We will begin with conduit *C1* which connects junction J1 to J2.

1. Click the  button on the **Object Toolbar**. The mouse cursor changes shape to a crosshair.
2. Click the mouse on junction J1. Note how the mouse cursor changes shape to a pencil.
3. Move the mouse over to junction J2 (note how an outline of the conduit is drawn as you move the mouse) and left-click to create the conduit. You could have cancelled the operation by either right-clicking or by hitting the Esc key.

Repeat this procedure for conduits *C2* through *C4*. The model should look similar to the one shown in **Figure 9.15**.

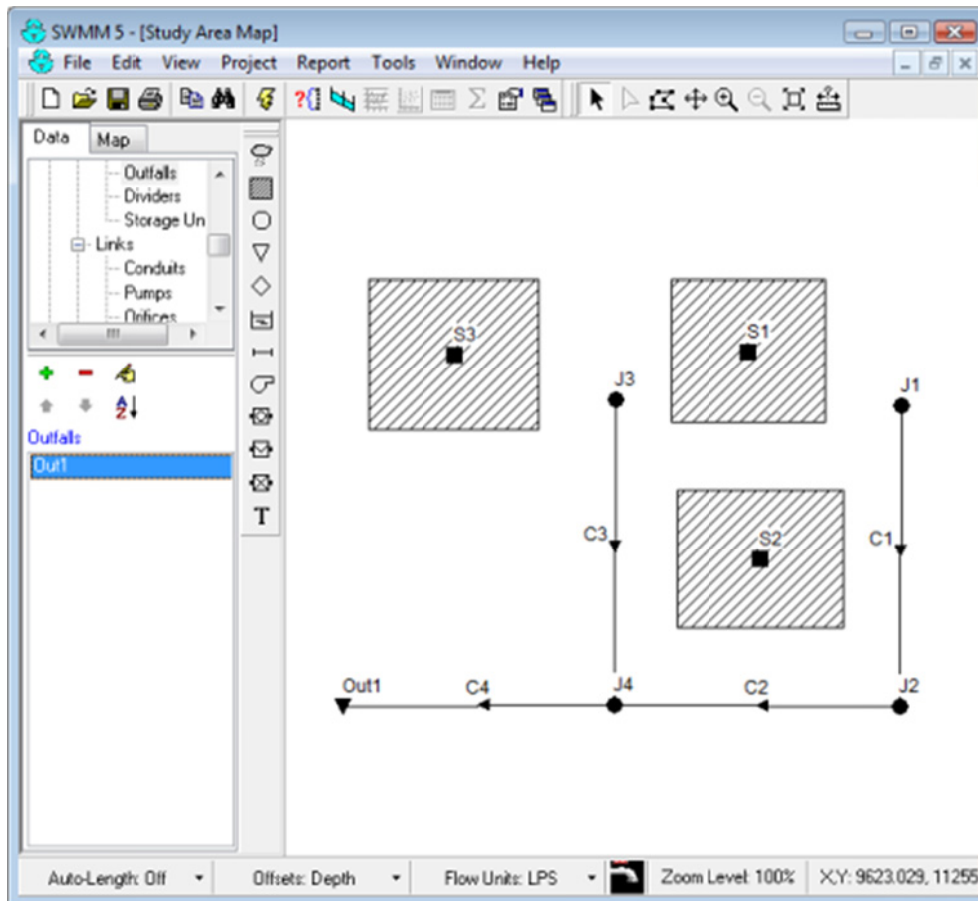



Figure 9.15: Model with added links

### Adding a Rain Gage


To complete the construction of our study area schematic a rain gage need to be added.

1. Click the Rain Gage button  on the **Object Toolbar**.
2. Move the mouse over the **Study Area Map** to where the gage should be located and left-click the mouse.

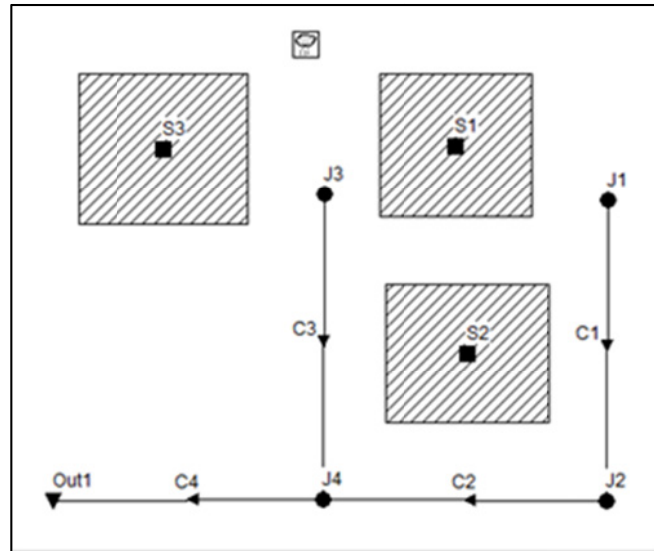
Note in **Figure 9.15** that the subcatchments are not yet linked to a specific junction. This will be done later in the exercise.

### Re-Positioning Objects

At this point we have completed drawing the example study area. Your system should look like the one shown in **Figure 9.16**. If the rain gage, subcatchments or nodes are out of position you can move them around by:



1. Clicking the  button to place the map in **Object Selection** mode;
2. clicking on the object to be moved;
3. dragging the object with the left mouse button held down to its new position.





**Figure 9.16: Model layout**

To re-shape a subcatchment's outline:

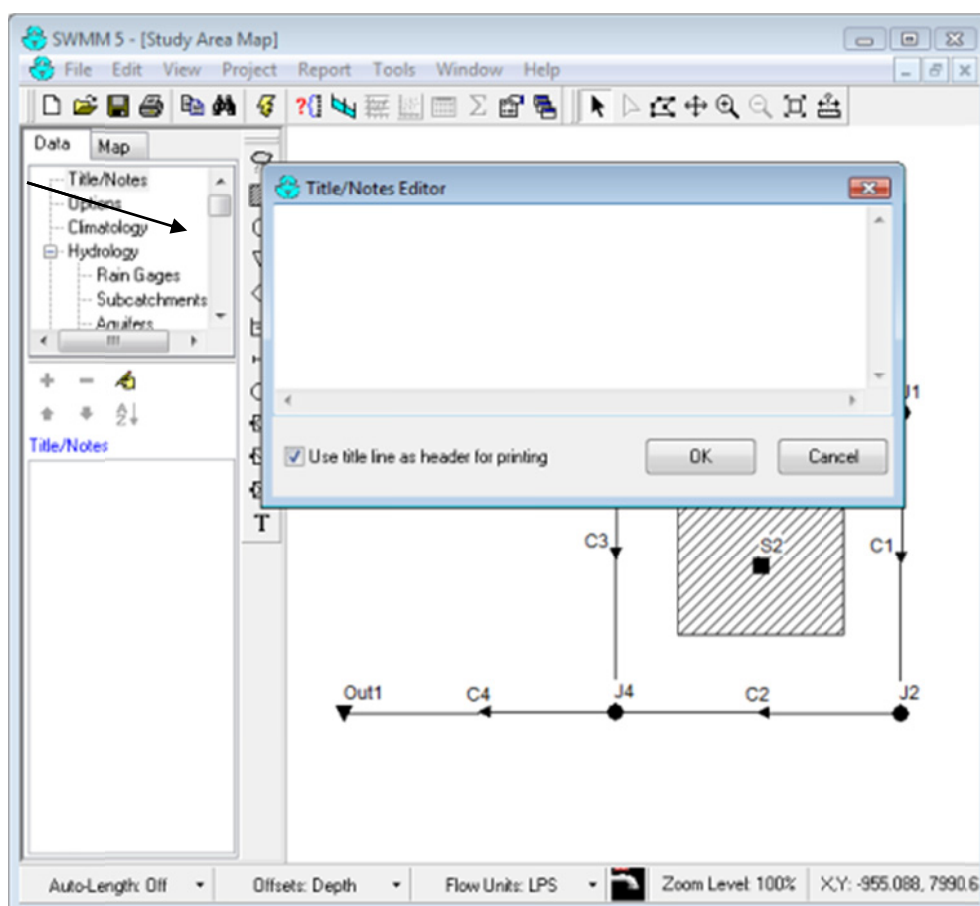
1. With the map in **Object Selection** mode, click on the subcatchment's centroid (indicated by a solid square within the subcatchment) to select it.
2. Then click the  button on the **Map Toolbar** to put the map into **Vertex Selection** mode.
3. Select a vertex point on the subcatchment outline by clicking on it (note how the selected vertex is indicated by a filled solid square).
4. Drag the vertex to its new position with the left mouse button held down.
5. If need be, vertices can be added to or deleted from the outline by right-clicking the mouse and selecting the appropriate option from the popup menu that appears.
6. When finished, click the  button to return to **Object Selection** mode.

This same procedure can also be used to re-shape a link.

### **Saving and opening the project**

Having completed the initial design of our example project it is a good idea to give it a title and save our work to a file at this point. To do this:

1. Select the **Title/Notes** category from the **Data Browser** and click the  button (see **Figure 9.17**).



**Figure 9.17: Title/Notes Editor**

2. In the Project **Title/Notes** dialog that appears, enter "*Practical Exercise E5*" as the title of our project and click the **OK** button to close the dialog.
3. From the **File** menu select the **Save As** option.
4. In the **Save As dialog** that appears (**Figure 9.18**), select a folder and file name under which to save this project. We suggest naming the file *E5.inp*. (An extension of .inp will be added to the file name if one is not supplied.)
5. Click **Save** to save the project to file.

The project data is saved to the file in a readable text format. You can view what the file looks like by selecting **Project | Details** from the main menu. To open our project at some later time, we would select the **Open** command from the **File** menu.

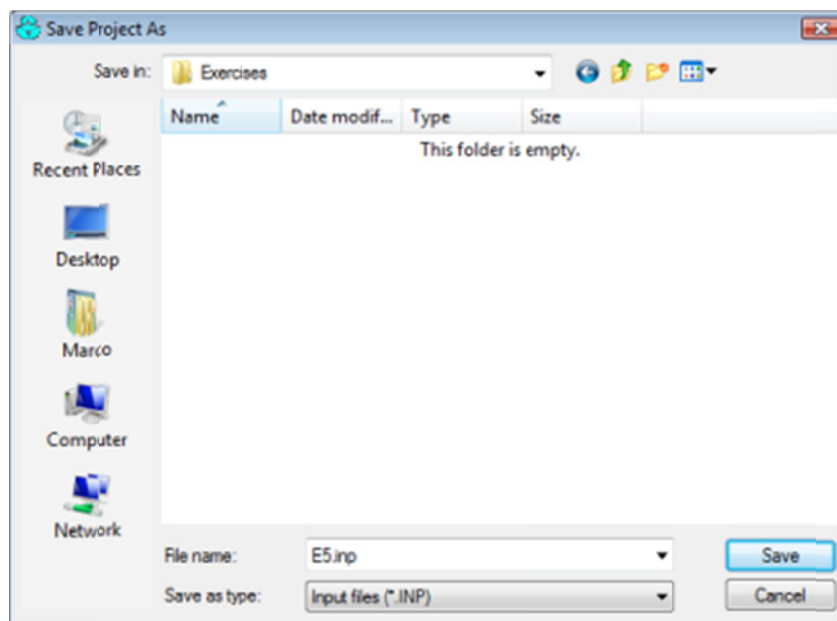



Figure 9.18: Save As dialog box

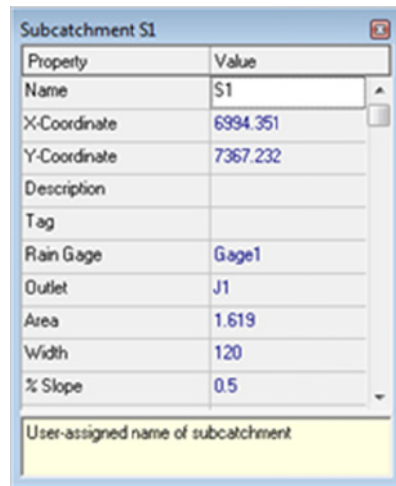
## SETTING THE PROPERTIES OF SWMM OBJECTS

### Setting Properties

As visual objects are added to our project, SWMM assigns them a default set of properties. To change the value of a specific property for an object we must select the object into the **Property Editor** (shown in Figure 9.19). There are several different ways to do this. If the **Property Editor** is already visible then you can simply click on the object or select it from the **Data** page of the **Browser**. If the **Editor** is not visible then you can make it appear by one of the following actions:

- Double-click the object on the map.
- Right-click on the object and select **Properties** from the pop-up menu that appears.
- Select the object from the Data page of the Browser window and then click the **Browser's**  button.

Whenever the **Property Editor** has the focus you can press the F1 key to obtain a more detailed description of the properties listed.

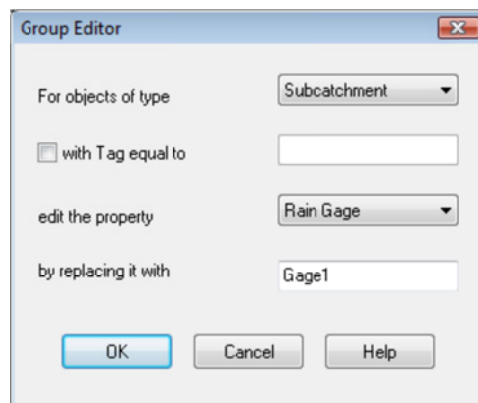


**Figure 9.19: Property Editor**

### **Setting Subcatchment Properties**

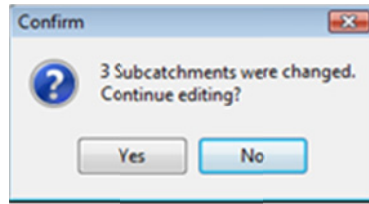
Two key properties of our subcatchments that need to be set are the rain gage that supplies rainfall data to the subcatchment and the node of the drainage system that receives runoff from the subcatchment. Since all of our subcatchments utilize the same rain gage, *Gage1*, we can use a shortcut method to set this property for all subcatchments at once:

1. From the main menu select **Edit | Select All**.
2. Then select **Edit | Group Edit** to make a **Group Editor dialog** appear.
3. Select **Subcatchments** as the class of object to edit, **Rain Gage** as the property to edit, and type in *Gage1* as the new value (as shown in **Figure 9.20**).




**Figure 9.20: Group Editor dialog**

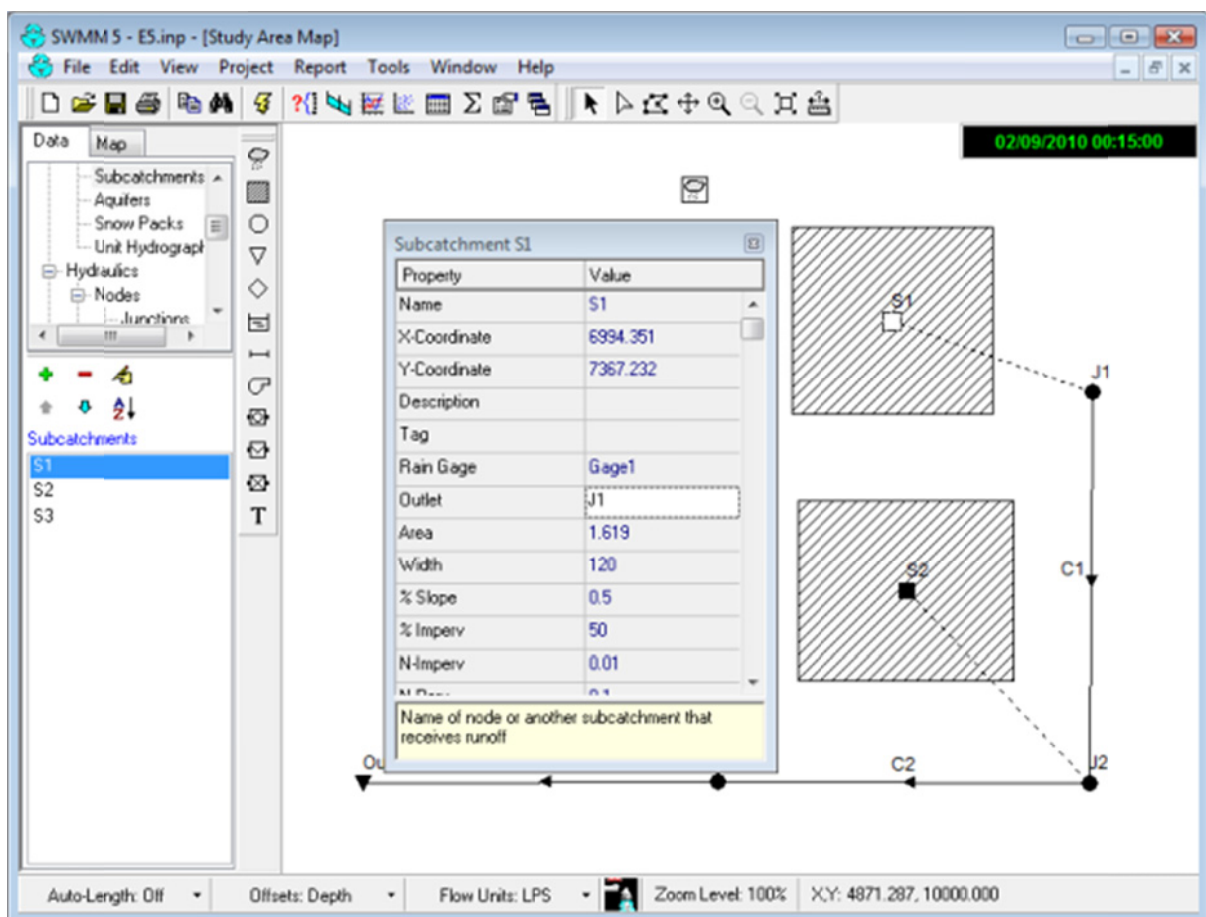
4. Click **OK** to change the rain gage of all subcatchments to *Gage1*. A confirmation dialog will appear noting that 3 subcatchments have changed (**Figure 9.21**). Select *No* when asked to continue editing.



**Figure 9.21: Confirmation dialog**

To set the outlet node of our subcatchments we have to proceed one by one, since these vary by subcatchment:

1. Double click on subcatchment S1 or select it from the **Data Browser** and click the Browser's  button to bring up the **Property Editor**.
2. Type *J1* in the **Outlet** field and press Enter. Note how a dotted line is drawn between the subcatchment and the node (**Figure 9.22**).



**Figure 9.22: Linking subcatchment and junction**

3. Click on subcatchment S2 and enter *J2* as its **Outlet**.
4. Click on subcatchment S3 and enter *J3* as its **Outlet**.

Finally, we wish to represent area S3 as being less developed than the others. Select S3 into the **Property Editor** and set its **% Imperviousness** to 25 (**Figure 9.23**).

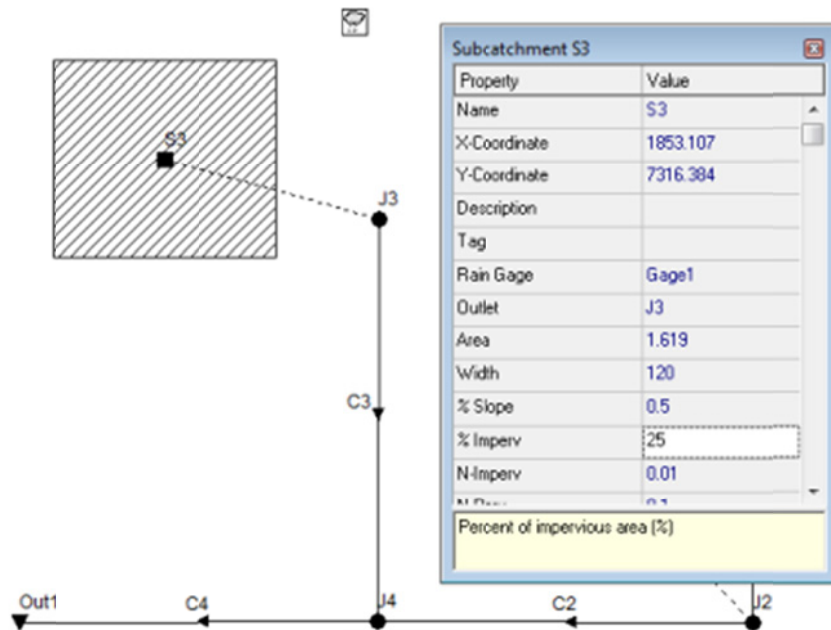


Figure 9.23: Setting %Imperviousness

### Setting Node/Link Properties

The junctions and outfall of our drainage system need to have invert elevations assigned to them. As we did with the subcatchments, select each junction individually into the **Property Editor** and set its **Invert Elevation** to the value shown in **Table 9.2**.

**Table 9.2: Node properties**

Node	Invert (m)
J1	1129,3
J2	1127,4
J3	1128,3
J4	1126,8
Out1	1125,9

Only one of the conduits in our example system has a non-default property value. This is conduit C4, the outlet pipe, whose diameter should be 0,45 m. instead of 0,3 m. To change its diameter (maximum depth), select conduit C4 into the **Property Editor** and set the **Max. Depth** value to 0,45.

### Setting Rain Gage Properties

In order to provide a source of rainfall input to our project we need to set the rain gage properties. Select **Gage1** into the **Property Editor** and set the following properties (see **Figure 9.24**):

Rain Format: *INTENSITY*  
 Rain Interval: *1:00*  
 Data Source: *TIMESERIES*  
 Series Name: *TSI*

Property	Value
Name	Gage1
X-Coordinate	4399.267
Y-Coordinate	9087.594
Description	
Tag	
Rain Format	INTENSITY
Rain Interval	1.00
Snow Catch Factor	1.0
Data Source	TIMESERIES
<b>TIME SERIES:</b>	
- Series Name	TS1
<b>DATA FILE:</b>	
- File Name	*
- Station ID	*
- Rain Units	IN
User-assigned name of rain gage	

**Figure 9.24: Rain Gage property editor**

As mentioned earlier, a simulation of the response of the study area to a 75 mm, 6-hour design storm is required. A time series named TS1 will contain the hourly rainfall intensities that make up this storm. Thus a time series object is created and populate with data. To do this:

1. From the **Data Browser** select the **Time Series** category of objects.
2. Click the **+** button on the **Browser** which will bring up a **Time Series Editor** form.
3. Enter *TS1* in the **Time Series Name** field.
4. Enter the values into the **Time** and **Value** columns of the data entry grid (leave the Date column blank) as shown in **Figure 9.25**.

Time Series Name: TS1

Description:

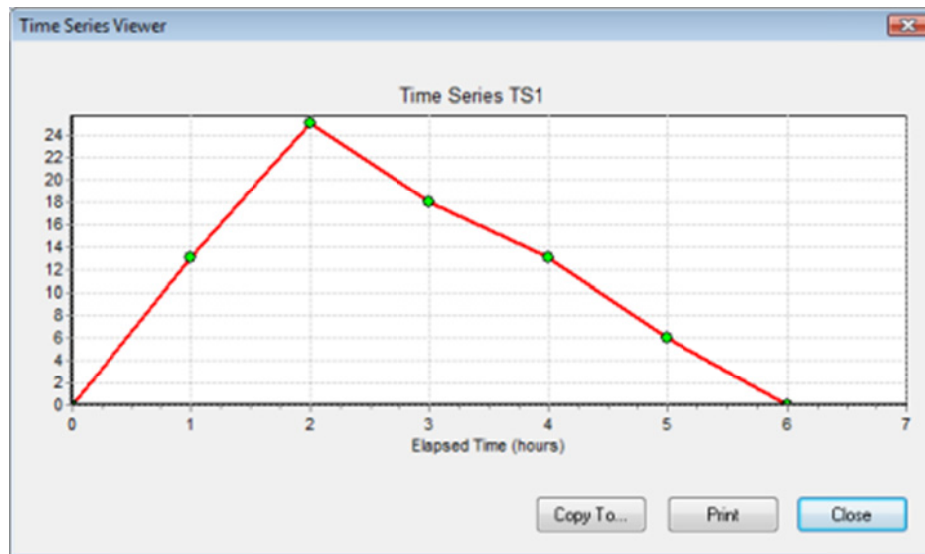
No dates means times are relative to start of simulation.

Date (M/D/Y)	Time (H:M)	Value
	0	0
	1	13
	2	25
	3	18
	4	13
	5	6
	6	0

Buttons: View..., Load..., Save..., OK, Cancel, Help

**Figure 9.25: Time Series Editor**

5. You can click the **View** button on the dialog to see a graph of the time series values (**Figure 9.26**). Click **Close** to return to the **Time Series Editor**.




**Figure 9.26: Time Series Viewer**

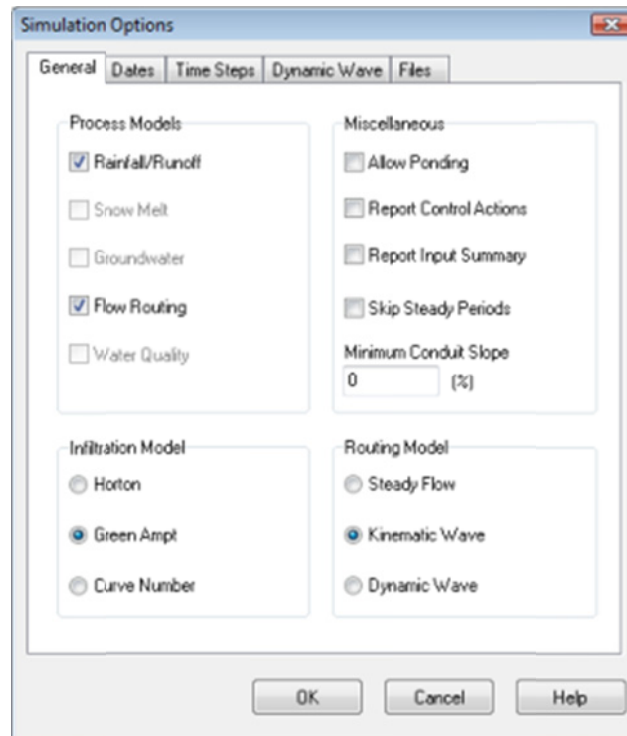
6. Click the **OK** button to accept the new time series.

### **RUNNING A SINGLE EVENT ANALYSIS**

Before analysing the performance of our example drainage system we need to set some options that determine how the analysis will be carried out. To do this:

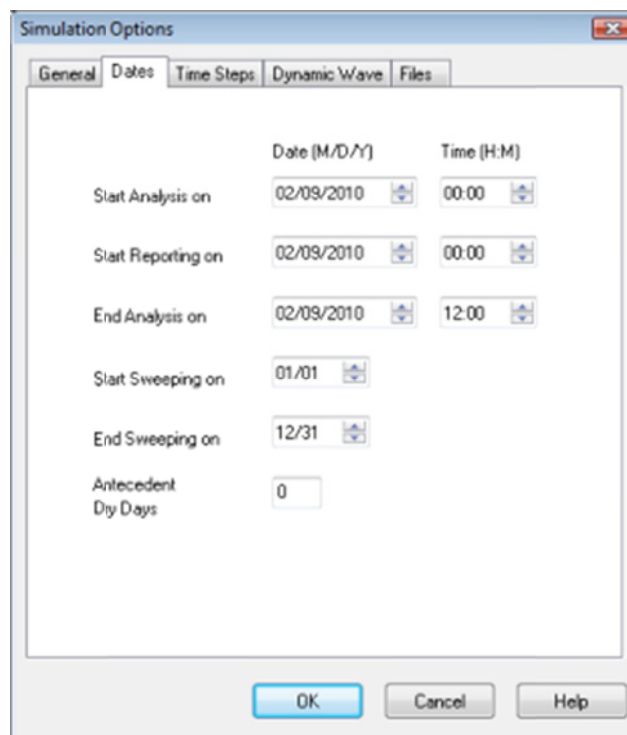
1. From the **Data Browser**, select the **Options** category and click the  button.
2. On the **General** page of the **Simulation Options** dialog that appears, select *Kinematic Wave* as the flow routing method. The **Infiltration model** should already be set to *Green-Ampt*. The **Allow Ponding** option should be *unchecked* (see **Figure 9.27**).





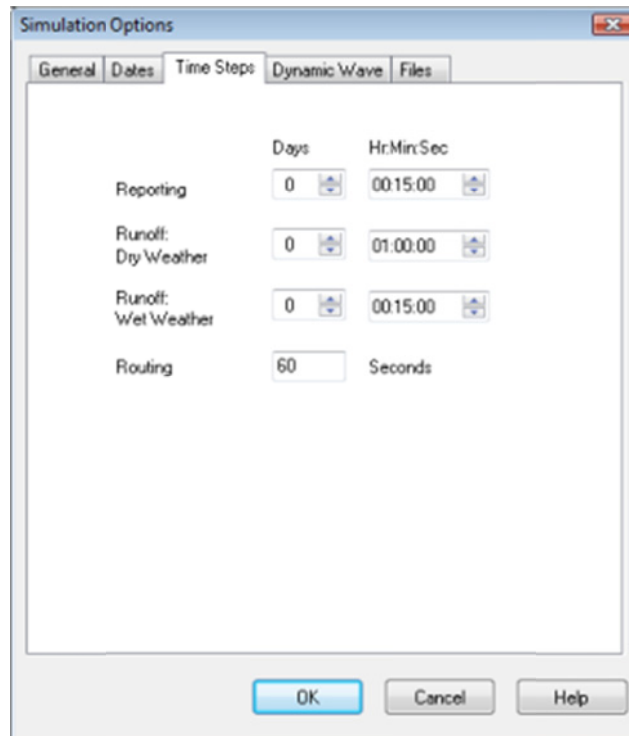
**Figure 9.27: Simulation options – General**

- On the **Dates** page of the dialog, set the **End Analysis** time to **12:00:00** (Figure 9.28) and the Dates and Times as indicated on Figure 9.28.



**Figure 9.28: Simulation options – Dates**

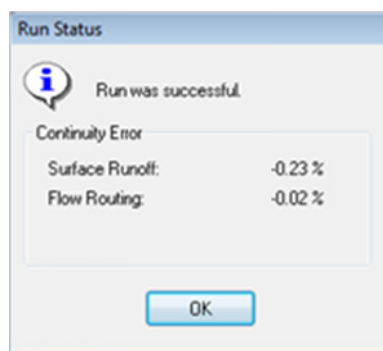
- On the **Time Steps** page, set the **Routing** time step to **60 sec** (Figure 9.29).



**Figure 9.29: Simulation options – Time Steps**

5. Click **OK** to close the **Simulation Options** dialog.

We are now ready to run the simulation. To do so, select **Project | Run Simulation** (or click the ⚡ button). After the run was completed the **Run Status** will be indicated as shown in **Figure 9.30**.



**Figure 9.30: Run Status**

## VIEWING SIMULATION RESULTS

### Viewing Analysis Results

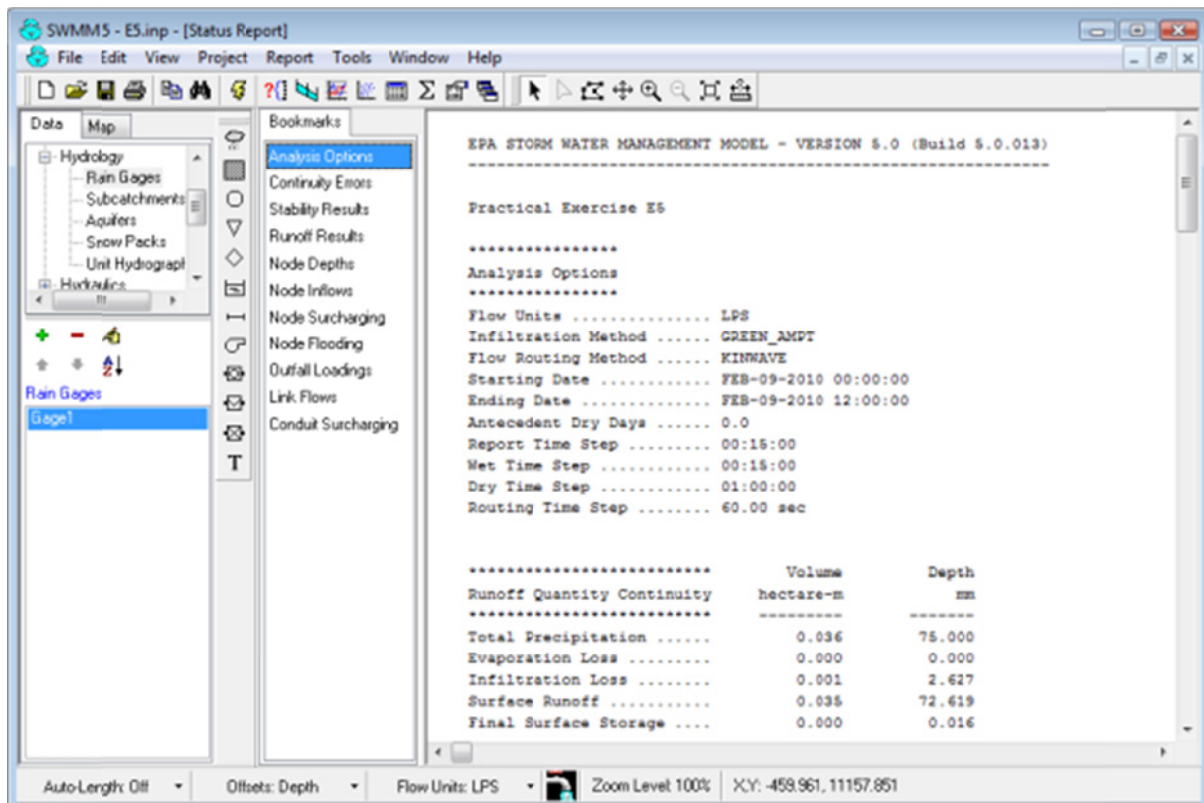
If there was a problem in running the simulation, a **Status Report** will appear describing what errors occurred.

Upon successfully completing a run, there are numerous ways in which to view the results of the simulation. A few will be illustrated:

- Viewing the Status Report
- Viewing results on the map
- Viewing a time series plot
- Viewing a profile plot

### Viewing the Status Report

The **Status Report** contains useful summary information about the results of a simulation run. To view the report, select **Report | Status** (shown in **Figure 9.31**)



**Figure 9.31: Status report**

For the system we just analysed the report indicates the following:

1. The quality of the simulation is quite good, with negligible mass balance continuity errors for both runoff and routing (-0.23% and -0.02%, respectively, if all data were entered correctly).
2. Of the 75 mm of rain that fell on the study area, 43,75 mm infiltrated into the ground and essentially the remainder became runoff.

3. The **Node Flooding Summary** table indicates there was internal flooding in the system at node J2.
4. The **Conduit Surcharge Summary** table shows that Conduit C2, just downstream of node J2, was at full capacity and therefore appears to be slightly undersized.

### Viewing Results on the Map

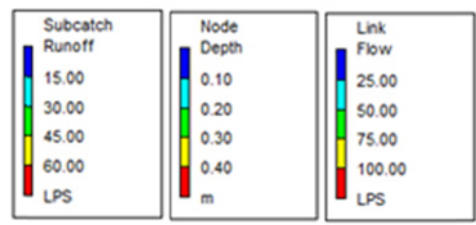
Simulation results (as well as some design parameters, such as subcatchment area, node invert elevation, link maximum depth) can be viewed in colour-coded fashion on the study area map. To view a particular variable in this fashion:

1. Select the **Map** page of the **Browser** panel.
2. Select the variables to view for **Subcatchments**, **Nodes**, and **Links** from the dropdown combo boxes in the **Themes** panel. Try for instance *Runoff*, *Depth* and *Flow* respectively as shown in **Figure 9.32**.



**Figure 9.32: Themes panel**

3. The colour coding used for a particular variable is displayed with a legend on the study area map. To toggle the display of a legend, select **View | Legends** (see **Figure 9.33**).



**Figure 9.33: Legends**

4. To move a legend to another location, drag it with the left mouse button held down.
5. To change the colour coding and the breakpoint values for different colours, select **View | Legends | Modify** and then the pertinent class of object (or if the legend is already visible, simply right-click on it). Attempt to set the breakpoint values as indicated in **Figure 9.33**.
6. To view numerical values for the variables being displayed on the map, select **Tools | Map Display Options** and then select the **Annotation** page of the **Map Options** dialog. Use the check boxes for **Rain Gages**, **Subcatchments**, **Nodes**, and **Links** to specify what kind of annotation to add.
7. The **Date / Time of Day / Elapsed Time** controls on the **Time Period** panel **Map Browser** can be used to move through the simulation results in time. Set the **Time of Day** to 3:00:00 to obtain the view as depicted in **Figure 9.34**.

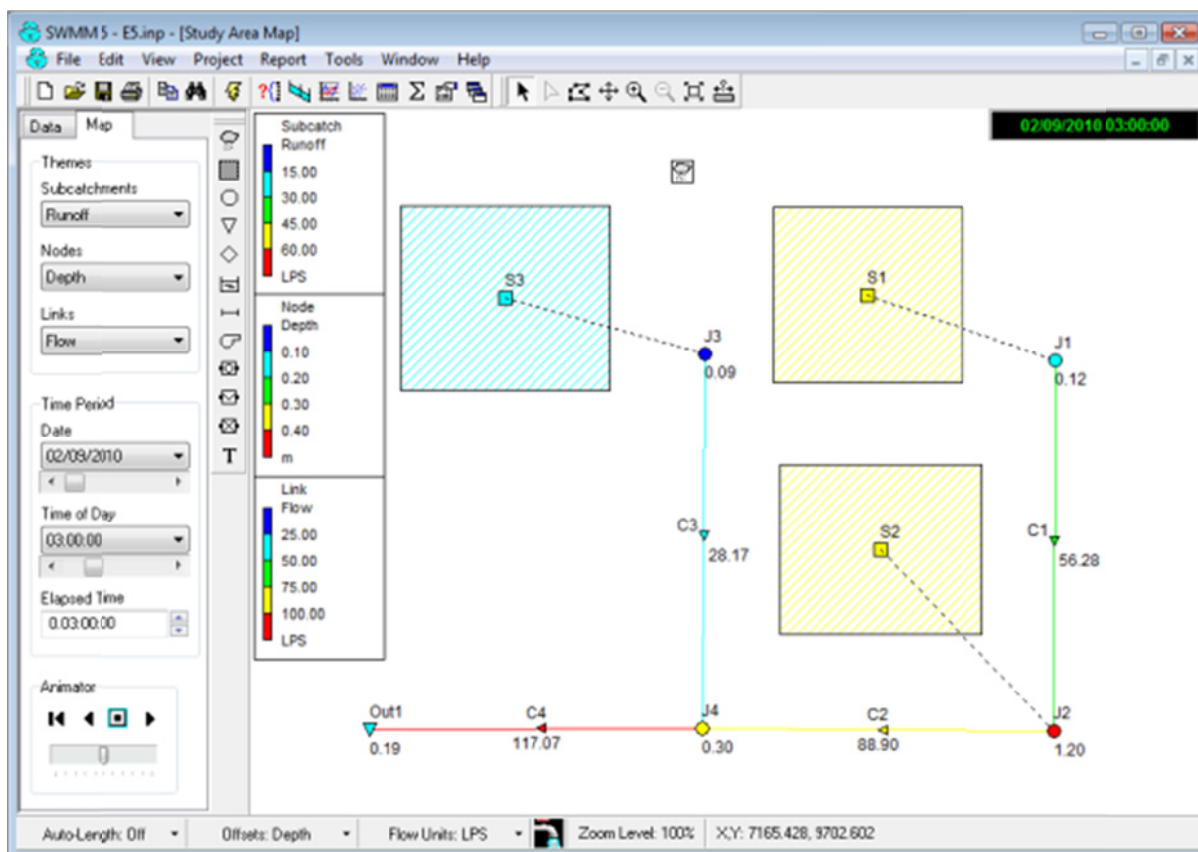




Figure 9.34: Viewing results (using simulation time)

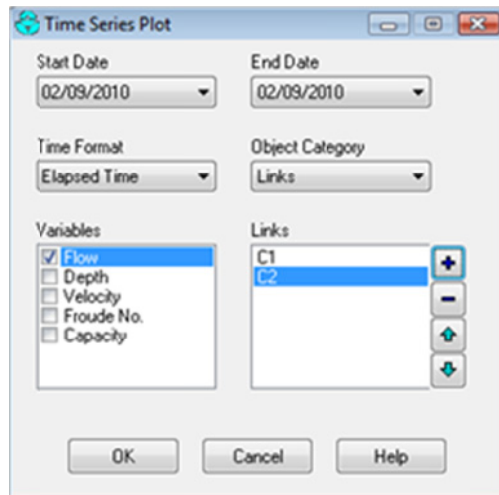
8. You can use the controls in the **Animator** panel of the **Map Browser** to animate the map display through time. For example, pressing the  button will run the animation forward in time.

### Viewing a Time Series Plot


To generate a time series plot of a simulation result:

1. Select **Report | Graph | Time Series** or simply click  on the **Standard Toolbar**.
2. A **Time Series Plot** dialog will appear. It is used to select the objects and variables to be plotted.

For this example, the Time Series Plot dialog can be used to graph the flows in conduits C1 and C2 as follows (shown in **Figure 9.35**):



**Figure 9.35: Time Series Plot**


1. Select **Links** as the **Object Category**.
2. Select **Flow** as the **Variable** to plot.
3. Click on conduit C1 (either on the map or in the **Data Browser**) and then click  in the dialog to add it to the list of links plotted. Do the same for conduit C2.
4. Press **OK** to create the plot as shown in **Figure 9.36**.

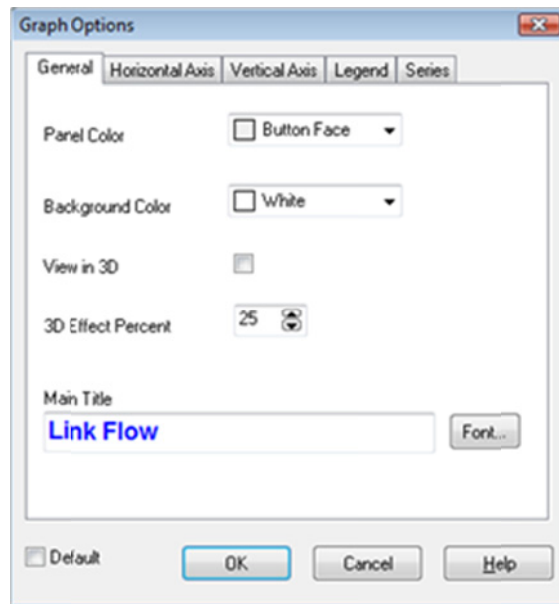


**Figure 9.36: Graph – Link Flow**

After a plot is created you can:

- customize its appearance by selecting **Report | Customize** or right clicking on the plot (see **Figure 9.37**),



- copy it to the clipboard and paste it into another application by selecting **Edit | Copy To** or clicking  on the **Standard Toolbar**
- print it by selecting **File | Print** or **File | Print Preview** (use **File | Page Setup** first to set margins, orientation, etc.).

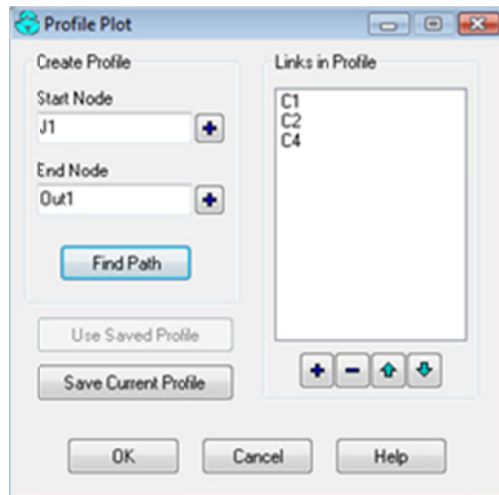


**Figure 9.37: Graph Options**

### **Viewing a Profile Plot**

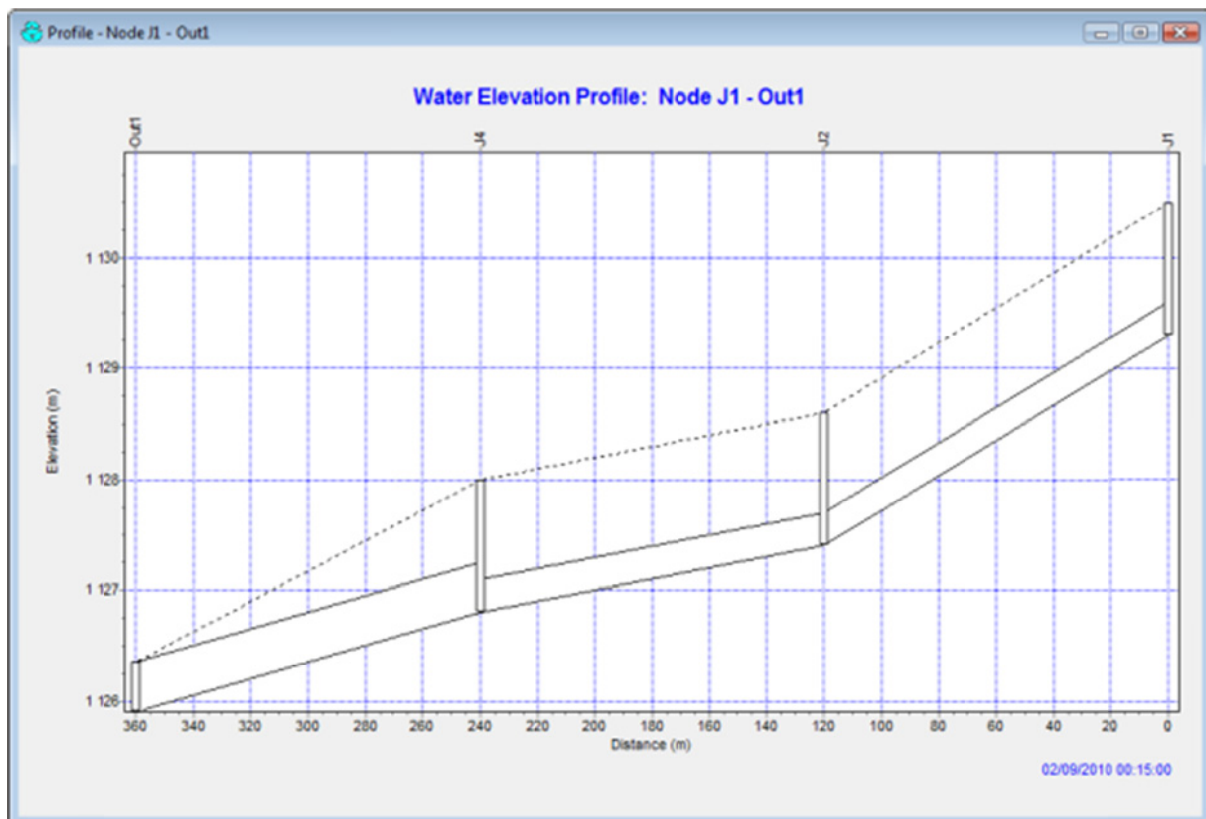
SWMM can generate profile plots showing how water surface depth varies across a path of connected nodes and links. Create such a plot for the conduits connecting junction *J1* to the outfall *Out1* of our example drainage system. To do this:

1. Select **Report | Graph | Profile** or simply click  on the **Standard Toolbar**.
2. Either enter *J1* in the **Start Node** field of the **Profile Plot** dialog that appears or select it on the map or from the **Data Browser** and click the  button next to the field.
3. Do the same for node *Out1* in the **End Node** field of the dialog.
4. Click the **Find Path** button. An ordered list of the links which form a connected path between the specified **Start and End** nodes will be displayed in the **Links in Profile** box see **Figure 9.38**. You can edit the entries in this box if need be.



**Figure 9.38: Profile plot**

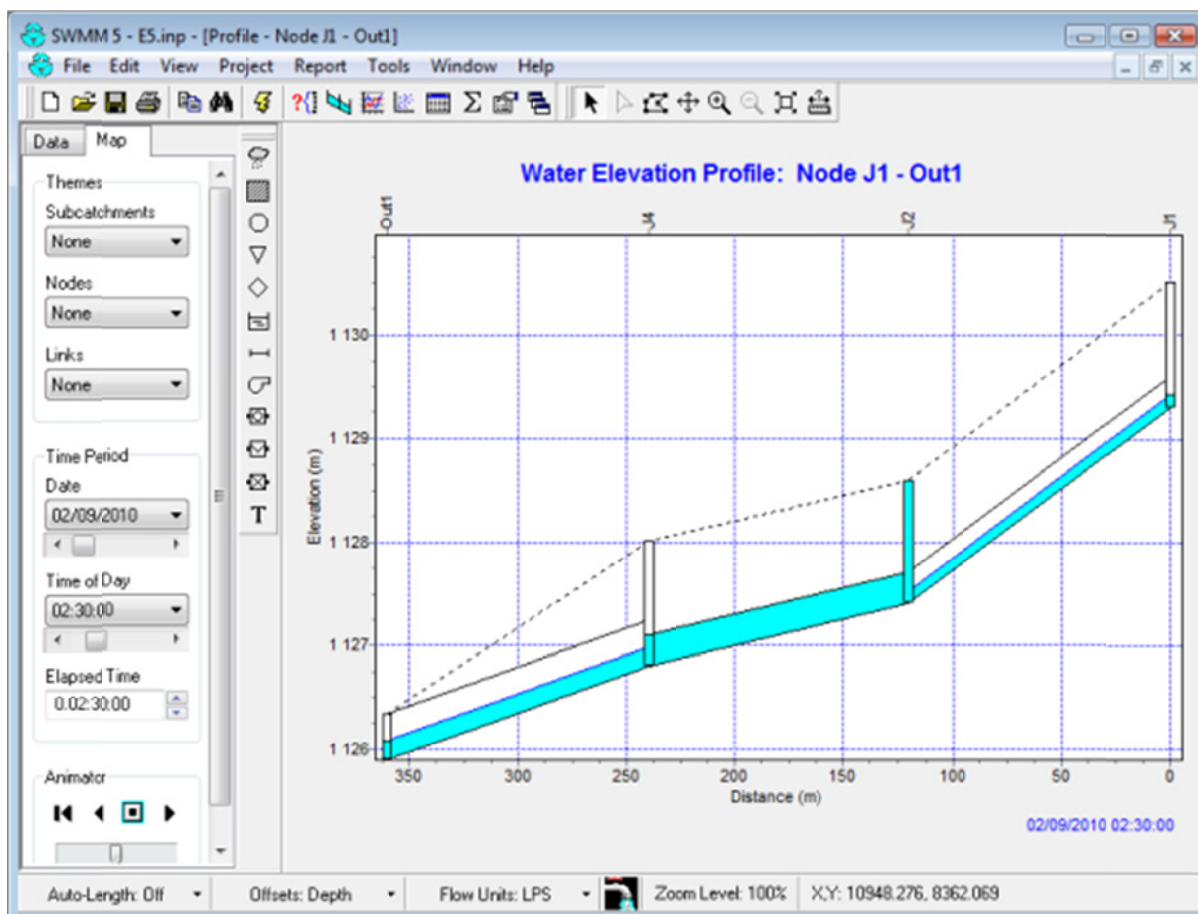
5. Click the **OK** button to create the plot (**Figure 9.39**), showing the water surface profile as it exists at the simulation time currently selected in the **Map Browser**.



**Figure 9.39: Profile – Node J1 – Out1**

As you move through time using the **Map Browser** or with the **Animator control**, the water depth profile on the plot will be updated. Observe how node J2 becomes flooded between hours 2 and 3 of the storm event as shown in **Figure 9.40**.





**Figure 9.40: Flooding shown at junction J2**


The appearance of a profile plot can be customized or it can be copied or printed using the same procedures as for a time series plot.

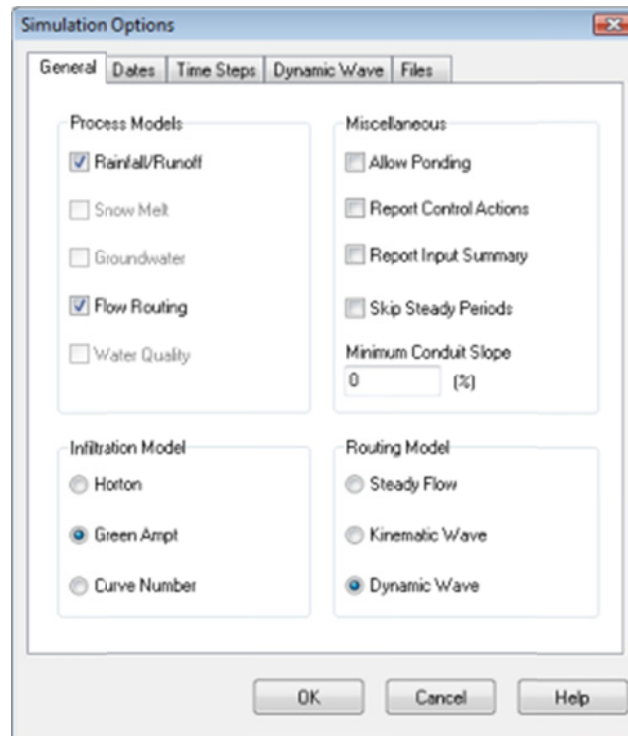
### **Running a Dynamic Wave Analysis**

In the analysis just run we chose to use the Kinematic Wave method of routing flows through our drainage system. This is an efficient but simplified approach that cannot deal with such phenomena as backwater effects, pressurized flow, flow reversal, and non-dendritic layouts. SWMM also includes a Dynamic Wave routing procedure that can represent these conditions. This procedure, however, requires more computation time, due to the need for smaller time steps to maintain numerical stability.

Most of the effects mentioned above would not apply to our example. However we had one conduit, C2, that flowed full and caused its upstream junction to flood. It could be that this pipe is actually being pressurized and could therefore convey more flow than was computed using Kinematic Wave routing. We would now like to see what would happen if we apply Dynamic Wave routing instead.

To run the analysis with Dynamic Wave routing:

1. From the **Data Browser**, select the **Options** category and click the  button.
2. On the **General** page of the **Simulation Options** dialog that appears, select *Dynamic Wave* as the flow routing method (see **Figure 9.41**).



**Figure 9.41: Simulation options**

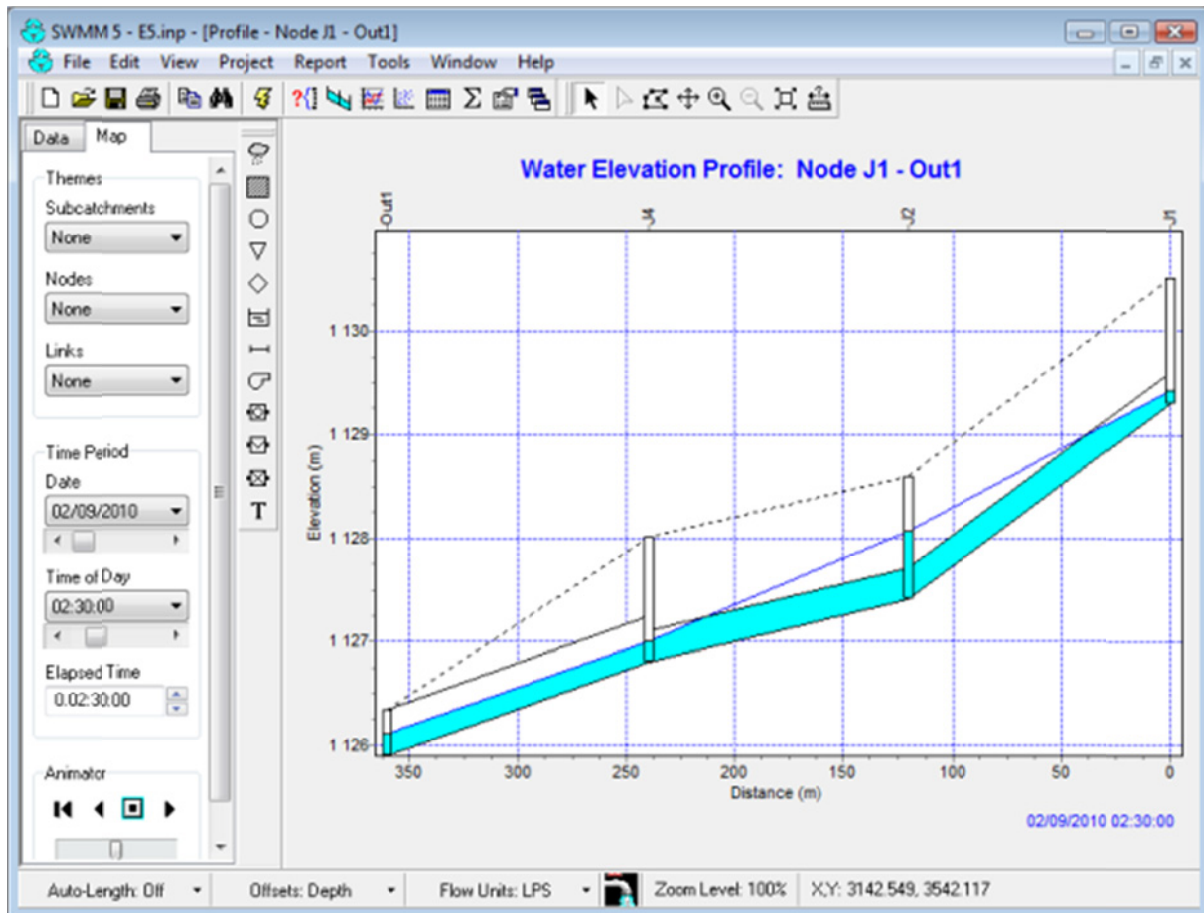
3. Click **OK** to close the form and select **Project | Run Simulation** (or click the ⚡ button) to re-run the analysis.

If you look at the **Status Report** for this run you will see that there is no longer any flooding and that the peak flow carried by conduit C2 has been increased from 95,17 l/s to 112,56 l/s (the conduit now flows pressurized).

A profile can be drawn again which will indicate the pressurized flow conditions experienced in link C2, see **Figure 9.42**.

We have only touched the surface of SWMM's capabilities. Some additional features of the program that you will find useful include:

- Water quality analysis;
- running a continuous simulation;
- performing a frequency analysis;
- utilizing additional types of drainage elements, such as storage units, flow dividers, pumps, and regulators, to model more complex types of systems;
- using control rules to simulate real-time operation of pumps and regulators ;
- employing different types of externally-imposed inflows at drainage system nodes, such as direct time series inflows, dry weather inflows, and rainfall-derived inflow/infiltration;
- modelling groundwater interflow between aquifers beneath subcatchment areas and drainage system nodes;
- modelling snow fall accumulation and melting within subcatchments;
- adding calibration data to a project so that simulated results can be compared with measured values;
- utilizing a background street, site plan, or topo map to assist in laying out a system's drainage elements and to help relate simulated results to real-world locations.



**Figure 9.42: Pressurized flow conditions in link C2**

You can find more information on these and other features in the SWMM User's Manual. Once you have reached this stage of the exercise you should be in a position to answer the following questions based on the model that has been set-up at this point.

1. What is the maximum depth at junction J2 and when does this occur?
2. What is the determined system runoff coefficient for this exercise? Hint: see Status Report
3. What is the maximum flow at the system outlet?
4. What is the peak runoff from catchment S2 and when does it occur?
5. What is the head difference between junctions J1 and J2 at 02:45?
6. What is the velocity and Froude number in link C4 at 04:00? What would be the type of flow at this time step?
7. What capacity is still available in link C1?
8. Draw the flow versus time graph for link C1.
9. What happens to the flow routing continuity if the Routing time step is set to 5 seconds instead of 60 seconds?

## 10 ASSESSMENT OF HYDRAULIC CAPACITY OF EXISTING DRAINAGE STRUCTURES

### 10.1 Example 10.1 – Level pool routing

#### Problem description Example 10.1

You have to determine the attenuation and translation that results from the routing of a given inflow hydrograph through a dam, see **Figure 10.1** (The methodology is the same as for a culvert, using an outflow equation from **Table 7.1** or the continuity of energy relationship). The following is known:

**Outflow stage relationship of the spillway of the dam is given by:**

$$Q = C_d LH^{1.5}$$

where:

$Q$  = discharge ( $\text{m}^3/\text{s}$ )

$C_d$  = discharge coefficient

$L$  = length of the spillway (m)

$H$  = total energy head (measured above the spillway level) (m)

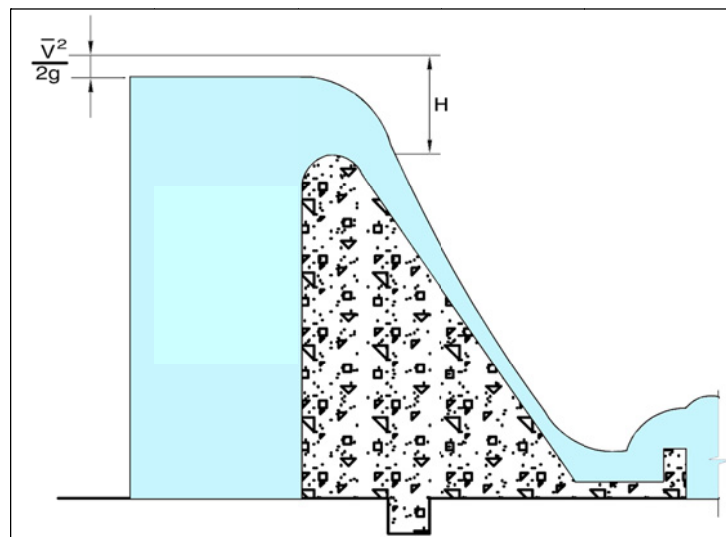
**In this case the outflow can be determined by the following relationship:  $Q = 110H^{1.5}$**

**Area-volume relationship of the storage volume is given as indicated below:**

Surface area at the spill level =  $7,5 \text{ km}^2$

Surface area at a level above spill level =  $7,5 + 1,5H \text{ km}^2$

$H$  = reflects the difference between the free surface level and the spill level, i.e. total energy (m)



**Figure 10.1: Section through the spillway of the dam**

The inflow hydrograph is given in **Figure 10.2**.

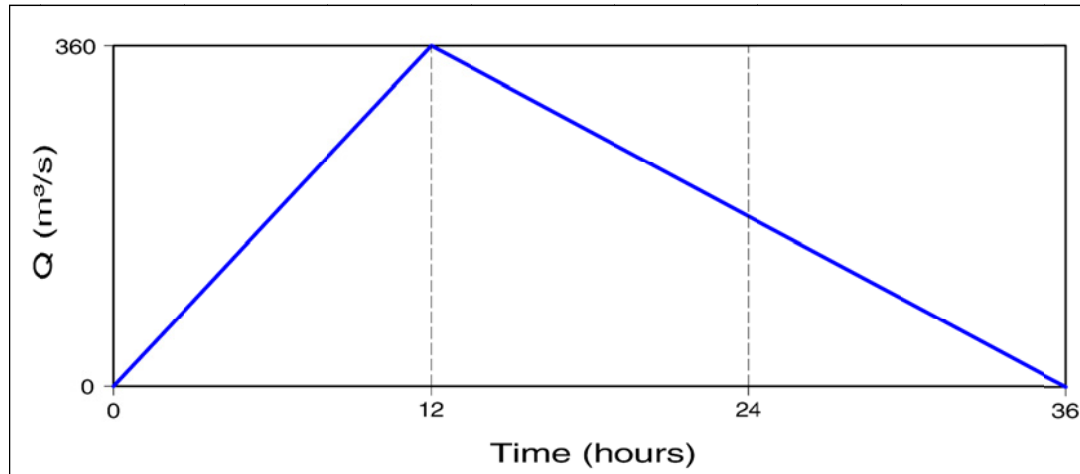


Figure 10.2: Inflow hydrograph

### Solution Example 10.1

It is known that the storage relationship is:

$$S = \int_0^H A dH = 10^6 \int_0^H (7,5 + 1,5H) dH$$

$$S = 10^6 (7,5 H + 0,75 H^2 + k)$$

It is known that  $S = 0$ , when  $H = 0$  and hence the integration constant,  $k = 0$ .

Assume that the time step,  $\Delta t = 2$  hours = 7200 seconds; then it follows that in the auxiliary function:

$$N = \frac{S}{\Delta t} + \frac{O}{2} :$$

$$N_2 = N_1 + \frac{I_1 + I_2}{2} - O_1 \text{ and by substituting the known values, it follows:}$$

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.  $h = H$

$$N = \frac{10^6}{7200} (7,5 H + 0,75 H^2) + 55 H^{1,5}$$

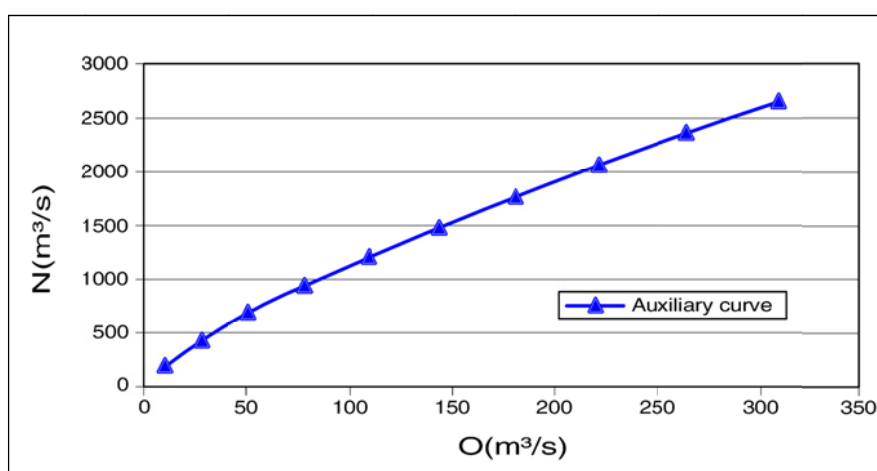
$$N = 1041,7H + 104,17H^2 + 55 H^{1,5}$$

$$N = 104,17H(10 + H + 0,53\sqrt{H})$$

The relationship for  $N$  and  $H$  to be used in the auxiliary function is shown in **Table 10.1** (and graphically in **Figure 10.3**).

**Table 10.1: Relationship of N versus O, for different H-values**

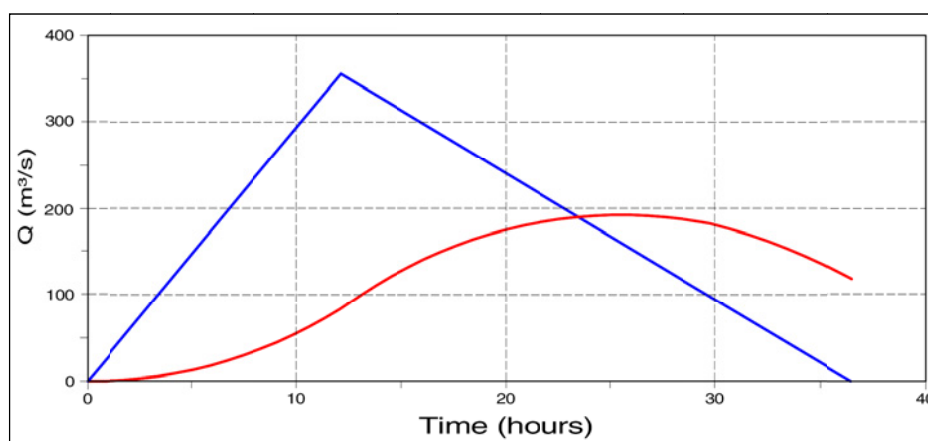
H	O	N
0,2	9,8	217,4
0,4	27,8	447,3
0,6	51,1	688,1
0,8	78,7	939,4
1,0	110,0	1200,9
1,2	144,6	1472,3
1,4	182,2	1753,7
1,6	222,6	2044,7
1,8	265,6	2345,4
2,0	311,1	2655,6



**Figure 10.3: Graphical presentation of the auxiliary function**

If the inflow and outflow hydrographs are plotted (**Figure 10.4**) 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 10.4: The inflow and calculated outflow hydrographs**

**Summary of the results:**

$$\text{Attenuation} = 360 - 180 = \mathbf{180 \text{ m}^3/\text{s}}$$

$$\text{Translation} = 24 - 12 = \mathbf{12 \text{ h}}$$



## 10.2 Example 10.2 – Level pool routing trough a culvert (inlet controlled)

### Problem description Example 10.2

You have to determine the attenuation and translation that results from the routing of a given inflow hydrograph through an existing culvert, **Figure 10.5**. The culvert has a square inlet with dimensions of 3,6 m high and 3m wide. The level of the shoulder break point (SBP) is 5 m as measured from the culvert's invert.

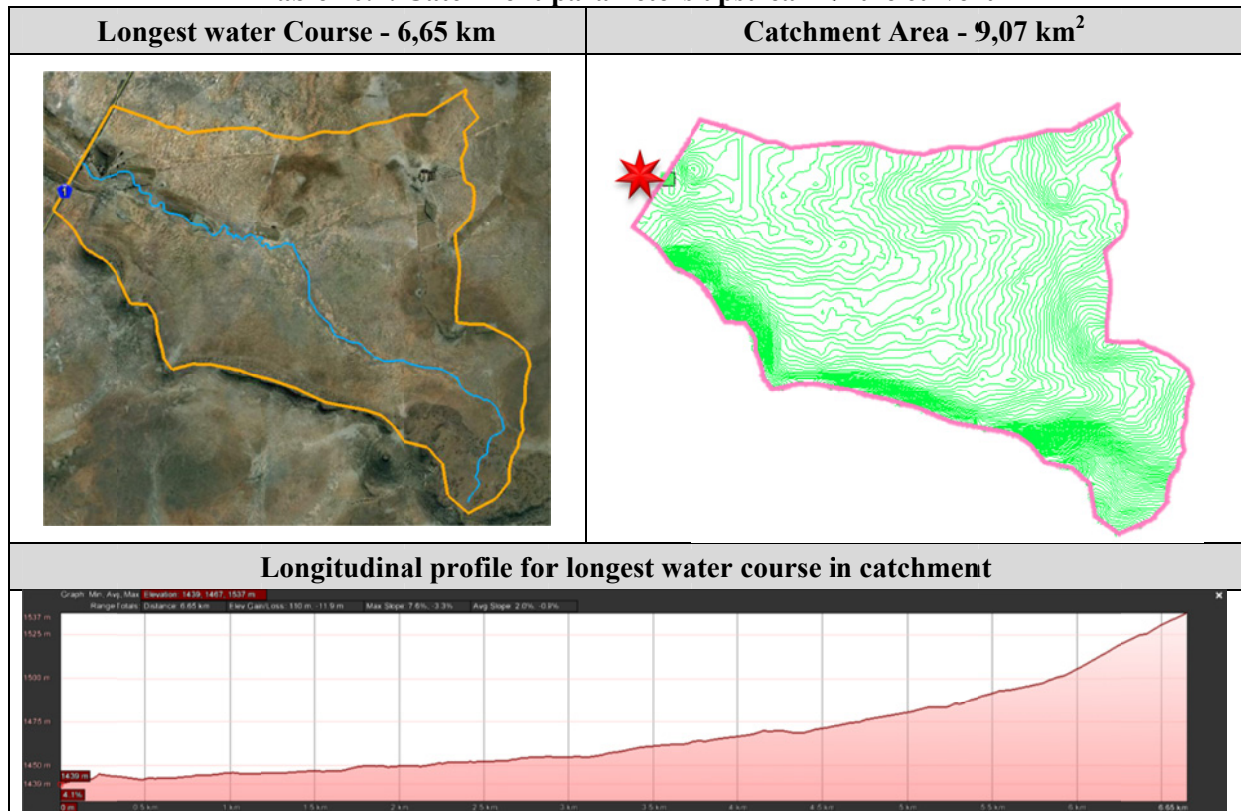
The methodology for applying level pool routing is the same for a culvert as being described from 1<sup>st</sup> principles in **Example 10.1** with the exception of using the outflow characteristics as given in **Table 10.5**. In this example, the Routing Utility given on the accompanying flash drive is used to estimate the effect of upstream storages to calculate the attenuation and translation for an existing culvert structure.



**Figure 10.5: Existing culvert**

The following catchment parameters upstream of the culvert are known and are given in **Table 10.2**.

**Table 10.2: Catchment parameters upstream of the culvert**



Calculation of the time of concentration for flow in a defined watercourse. (This calculation is by default done by the Routing Utility and is also given in **Table 10.3**.)

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.

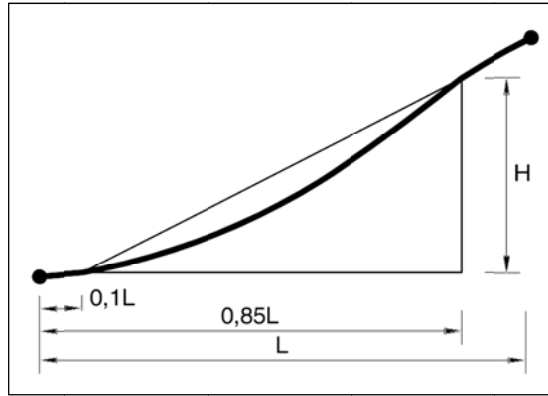
$$T_C = \left( \frac{0,87L^2}{1\,000 S_{av}} \right)^{0,385} \quad \dots(10.1)$$

Where:

$T_C$	=	time of concentration (hours)
$L$	=	hydraulic length of the catchment, measured along the flow path from the catchment boundary to the point where the flood needs to be determined (km)
$S_{av}$	=	average slope (m/m)

The user may calculate the average slope as defined in **Figure 10.6**.





**Figure 10.6: Slope definition for a defined water course**

$$S_{av} = \frac{H_{0,85L} - H_{0,10L}}{(1\,000)(0,75L)} \quad \dots(10.2)$$

Where:

$S_{av}$	=	average slope of the catchment (m/m) (see <b>Figure 10.6</b> )
$H_{0,10L}$	=	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 the watercourse (km)

The catchment parameters required to determine the average slope for the catchment from **Table 10.2** is given in **Table 10.3**.

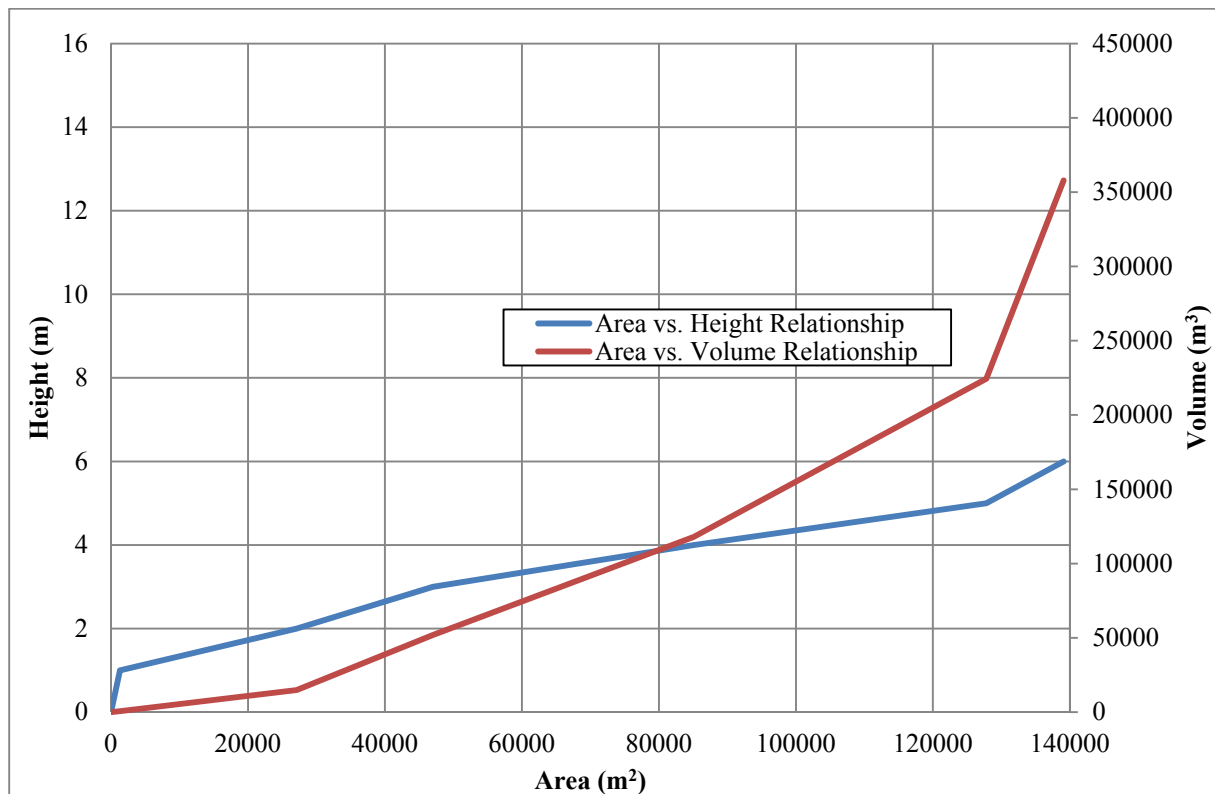
**Table 10.3: Catchment characteristics with respect to the defined water course**

Catchment parameter	Value
Longest Water Course (km)	6,65
Start Elevation (m)	1439
End elevation (m)	1537
10% elevation (m/m)	1443
85% elevation (m/m)	1494
Average slope (m/m)	0,0147
Area (km <sup>2</sup> )	9,07
Time of Concentration, $T_c$ (hr)	1,45

The area-height relationship for the upstream side of the culvert is given in **Table 10.4**. These values can be entered in the blue cells in the Routing Utility in the table titled “*Area/Volume Relationship Upstream of Culvert*”. From this relationship, the Routing Utility will automatically calculate the corresponding volume-height relationship (**Table 10.4**, **Figure 10.7**).

**Table 10.4: Area-height relationship for the upstream side of the culvert**

Elevation (Contour intervals) (m)	Depth measured from culvert Invert (m)	Area (m <sup>2</sup> )
1401	0	0
1402	1	1300
1403	2	27100
1404	3	47000
1405	4	85100
1406	5	127800
1407	6	139080



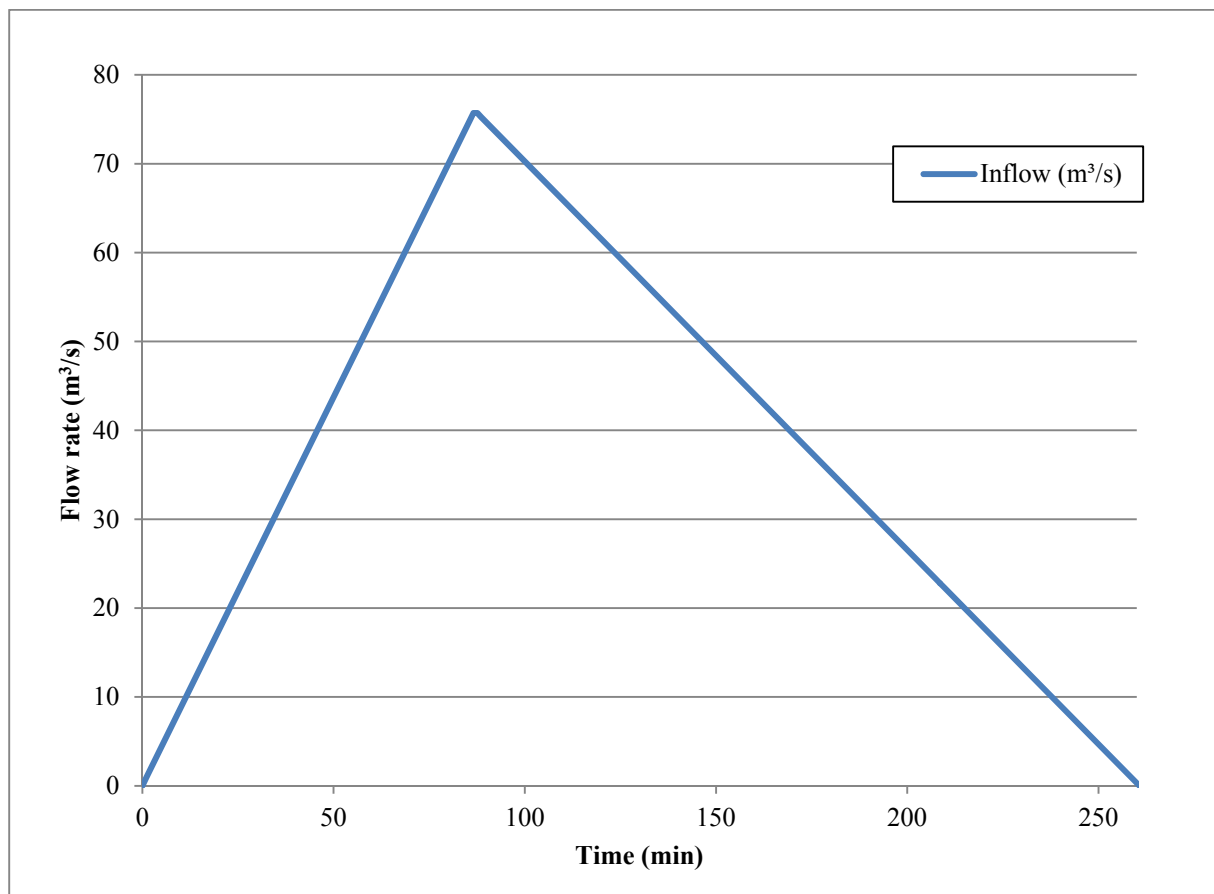
**Figure 10.7: Area-Height and Area-Volume relationship for the upstream side of the culvert**

The discharge capacity of culverts operating as inlet controlled systems and varying upstream water levels, are reflected in **Table 10.5**. The outlet characteristics can be defined for a rectangular culvert by the equations as given below in column 2 for the different ratios of  $H_1/D$

**Table 10.5: The capacity of culverts**

ROUND CULVERTS	RECTANGULAR CULVERTS
D = inside diameter (m)	D = height (inside) (m) B = width (inside) (m)
<p>For :</p> <p style="text-align: center;"><b><math>0 &lt; H_1/D &lt; 0,8</math></b></p> $\frac{Q}{D^2 \sqrt{gD}} = 0,48 \left[ \frac{S_0}{0,4} \right]^{0,05} \left[ \frac{H_1}{D} \right]^{1,9}$	<p>For:</p> <p style="text-align: center;"><b><math>0 &lt; H_1/D \leq 1,2</math></b></p> $Q = \frac{2}{3} C_B B H_1 \sqrt{\frac{2}{3} g H_1}$ <p>Where: <math>C_B = 1,0</math> for rounded inlets (<math>r &gt; 0,1B</math>)  <math>C_B = 0,9</math> for square inlets</p>
<p>And for: <b><math>0,8 &lt; H_1/D \leq 1,2</math> *</b></p> $\frac{Q}{D^2 \sqrt{gD}} = 0,44 \left[ \frac{S_0}{0,4} \right]^{0,05} \left[ \frac{H_1}{D} \right]^{1,5}$ <p>(<math>S_0</math> = slope of culvert bed with slight effect on capacity)  <b>Note:</b>  <i>* For <math>H_1/D &gt; 1,2</math>, the orifice formulae applies</i>  <math display="block">Q = C_D A \sqrt{2g \left( H_1 - \frac{D}{2} \right)}</math> with <math>C_D \approx 0,6</math></p>	<p>And for: <b><math>H_1/D &gt; 1,2</math></b></p> $Q = C_h B D \sqrt{2g (H_1 - C_h D)}$ <p>Where: <math>C_h = 0,8</math> for rounded inlets  <math>C_h = 0,6</math> for square inlets</p>

The estimated peak flow rate for the catchment was anticipated to be 76 m<sup>3</sup>/s for a return period of  $Q_{2T}$ . By assuming a triangular distribution of inflow, the inflow hydrograph is given in **Figure 10.8**.



**Figure 10.8: Inflow hydrograph**

### **Solution Example 10.2**

**Step 1:** Open the Routing Utility by double clicking on the icon.

**Step 2:** Accept the user agreement.

**Step 3:** Enter the site specific catchment and culvert parameters as given in the problem statement above in the allocate blue cells in the Routing Utility:

- Top width (## Times the Time of Concentration) = 0 ;
- Maximum inflow (Design Flow rate for normal routing or  $Q_{2T}$  for the review of existing culverts) = 76 m³/s;
- Number of Culverts = 1 unit;
- Culvert Height = 3,6 m;
- Culvert Width = 3,0 m;
- Culvert Shape Coefficient -  $C_B$  = 0,9;
- Culvert Shape Coefficient -  $C_H$  = 0,6;
- Maximum allowable damming depth measured from invert of culvert = 5 m;
- Longest Water Course 6,65 km;
- Average Slope 0,0147 m/m; and
- Area-Height relationship as given in **Table 10.4**.

After entering these parameters in the *Variable* and *Area/Volume Relationship Upstream of Culvert* tables in the Routing Utility, the screen captures should look like that given in **Table 10.6** and **Table 10.7**.

**Table 10.6: Print screen of entered catchment and culvert parameters**

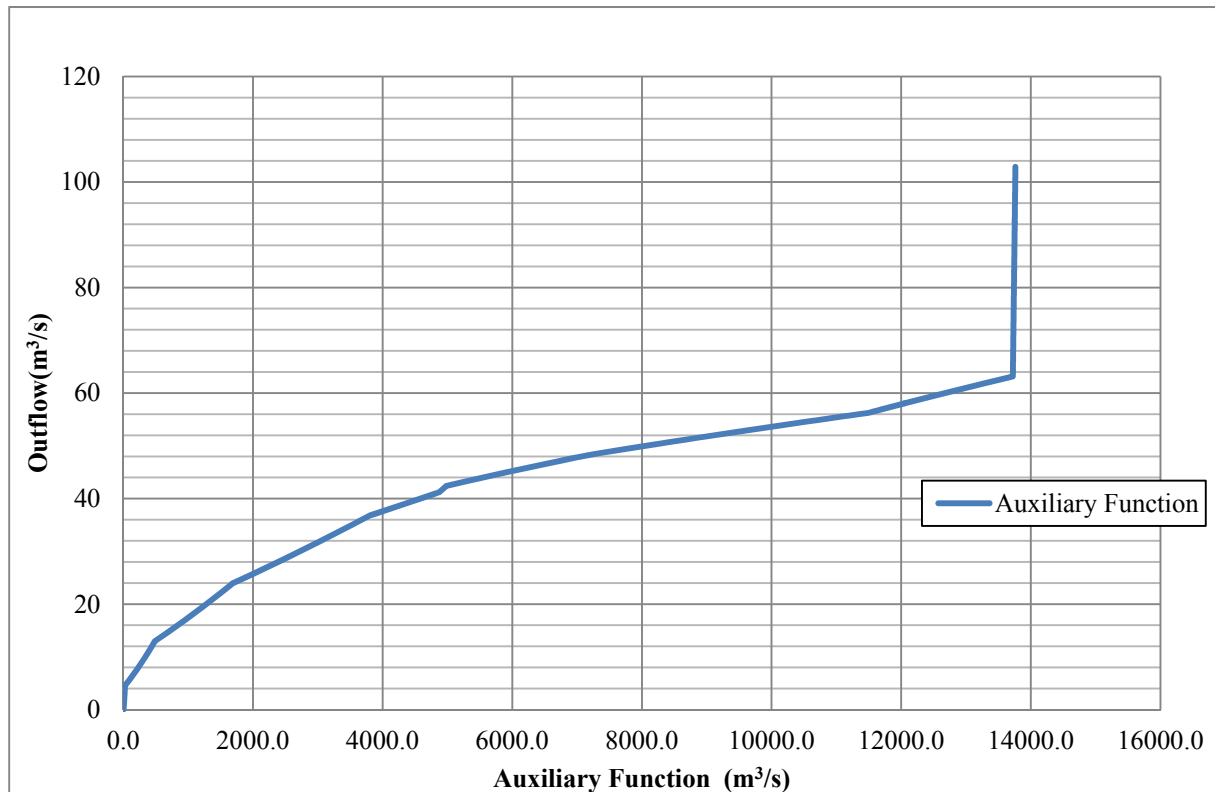
Only fill in values where the cells are blue shaded		
Variable	Value	Units
Top width (## Times the Time of Concentration)	0	
Maximum outflow	45,18	m <sup>3</sup> /s
Volume of Inflow	594389	m <sup>3</sup>
Volume of Outflow	594389	m <sup>3</sup>
Routing volume difference	0,0%	
Maximum flow depth	4,64	m
Volume of Storage at Maximum flow depth	185794	m <sup>3</sup>
Maximum inflow (Design Flow rate for normal routing or Q <sub>2T</sub> for the review of existing culverts)	76,00	m <sup>3</sup> /s
% reduction in peak (Attenuation)	40,3%	
Total Time H <sub>1</sub> /D > 1,2	102	min
Standing water that may cause piping	1,70	hours
Number of Culverts	1,00	units
Culvert Height	3,60	m
Culvert Width	3,00	m
Culvert Shape Coefficient - C <sub>B</sub>	0,9	
Culvert Shape Coefficient - C <sub>h</sub>	0,6	
Maximum allowable damming depth measured from invert of culvert	5,0	m
H/D maximum Ratio	1,29	
Longest Water Course	6,65	km
Average Slope	0,0147	m/m
Calculated T <sub>c</sub>	1,45	hours
Time duration of Outflow (dV/dt≈0)	436	min
	7,26	hours

**Table 10.7: Print screen of entered Area-Height parameters**

Area/Volume Relationship Upstream of Culvert				
Elevation (Contour intervals) (m)	Depth measured from culvert Invert (m)	Area (m <sup>2</sup> )	Delta Volume (m <sup>3</sup> )	Volume (m <sup>3</sup> )
1401	0	0	0	0
1402	1	1300	650	650
1403	2	27100	14200	14850
1404	3	47000	37050	51900
1405	4	85100	66050	117950
1406	5	127800	106450	224400
1407	6	139080	133440	357840
1408	7		69540	427380
1409	8		0	427380
1410	9		0	427380
1411	10		0	427380
1412	11		0	427380
1413	12		0	427380
1414	13		0	427380
1415	14		0	427380
1416	15		0	427380

**Step 4:** View the auxiliary function.

The auxiliary function used to estimate the outflow from the culvert can be viewed under the *Auxiliary Function* tab (**Figure 10.9**). Notice the vertical increase for outflows exceeding 63 m<sup>3</sup>/s. This was due to the Area-Height relationship that was not completed for all 16 available increments. However, this does not have any influence on the outcome of the results since the maximum damming depth is only 4,64 m and that corresponds to an outflow of 45,18 m<sup>3</sup>/s which is lower than the allowable damming depth of 5 m to the SBP.



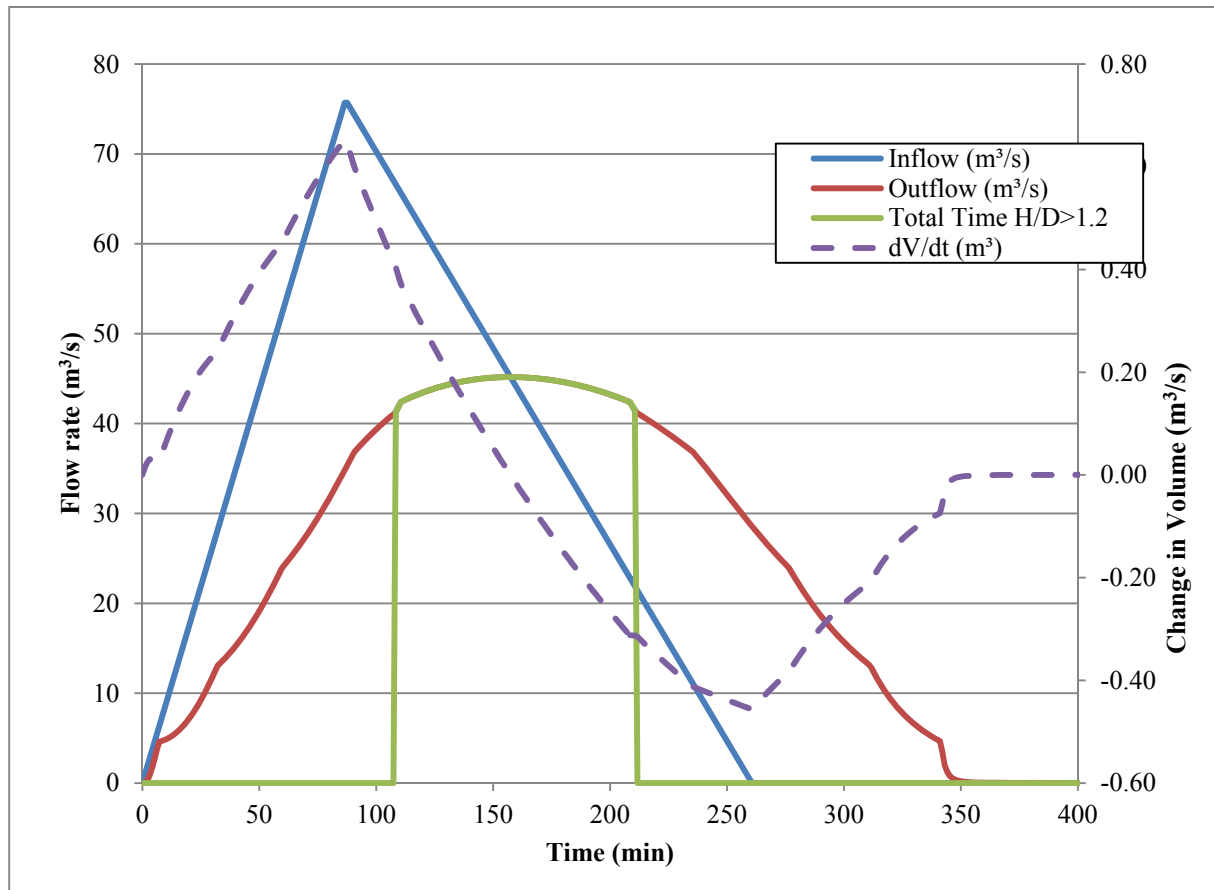
**Figure 10.9: Graphical presentation of the auxiliary function**

**Step 5:** Interpret the inflow and outflow hydrographs.

Depicted on the inflow (blue line) and outflow (red line) hydrographs (**Figure 10.10**) are also the change in volume with time ( $dV/dt$ ) as well as the duration for which the upstream energy head ( $H_1$ ) exceeds a ratio of 1,2D. These values are reflected by the dashed purple line and green solid line respectively.

If the inflow and outflow hydrographs are plotted (**Figure 10.10**) 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 10.10: Inflow and calculated outflow hydrographs**

**Step 6:** The results are summarized under the *Summary of Results* tab and can be printed by clicking on the *Print Summary Sheet* button in the upper right corner of the screen (**Figure 10.11**). Included on the summary sheet is a table that depicts the outcome of the assessment for existing culvert structures. In this case the culvert has not met the criteria as described in **Chapter 10** and thus *Fails* the hydraulic criteria for existing culverts.

$$\text{Attenuation} = 76 - 45.18 = 30.82 \text{ m}^3/\text{s}$$

$$\text{Translation} = 154 - 87 = 67 \text{ min}$$



### Routing Utility for the Investigation of Existing Hydraulic Structures



Print  
Summary  
Sheet

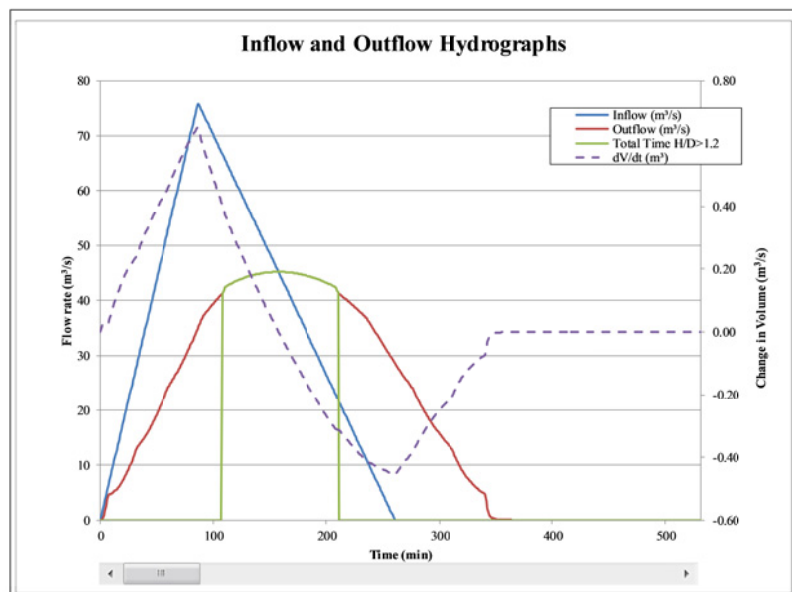
Variable	Value	Units
Maximum outflow	45.18	m <sup>3</sup> /s
Maximum inflow (Design Flow rate for normal routing or Q <sub>2T</sub> for the review of existing culverts)	76.00	m <sup>3</sup> /s
Total Time H <sub>1</sub> /D>1.2	102.19	min
Number of Culverts	1.00	units
Culvert Height	3.60	m
Culvert Width	3.00	m
Maximum allowable damming depth measured from invert of culvert	5.00	m
Calculated Time of Concentration (T <sub>c</sub> )	1.45	hours
Time duration of Outflow (dV/dt=0)	7.26	hours

### Investigation of existing culvert structures (Refer to Figure 10.1 SANRAL DRAINAGE MANUAL)

	V <sub>TI</sub> (m <sup>3</sup> )	0,5 V <sub>storm</sub> (m <sup>3</sup> )	Pass/Fail
Review of maximum temporal storage	185794	297199	Fail

	T <sub>s</sub> (min)	0,5 T <sub>c</sub> (min)	Pass/Fail
Review of the duration of excessive energy (upstream inundation)	102.19	86.90	Fail

Overall culvert structure outcome:	Culvert Structure Fail
------------------------------------	------------------------



Developed by: **Sinotech**

Disclaimer: This program was developed for the convenience of its users. Although every reasonable effort has been made to ensure that the program is accurate and reliable the program developers, Sinotech CC and/or Prof SJ van Vuuren accept no liability of any kind for any results, interpretation thereof or any use made of the results obtained with these programs. All users of this program do so entirely at their own risk

**Figure 10.11: Summary of the results**

**Step 7:** Re-evaluating the system with an upgraded system of 2 culverts of the same dimensions, it can be perceived that the maximum outflow is increased from 45,18 m<sup>3</sup>/s to 60,52 m<sup>3</sup>/s for the same inflow characteristics. However, upstream damming is significantly reduced from 4,64 m to 3,51 m. Now the structure meets the hydraulic criteria as given in **Chapter 10** and thus *Pass* the hydraulic criteria for existing culverts (**Figure 10.12**).

### Routing Utility for the Investigation of Existing Hydraulic Structures



Print  
Summary  
Sheet

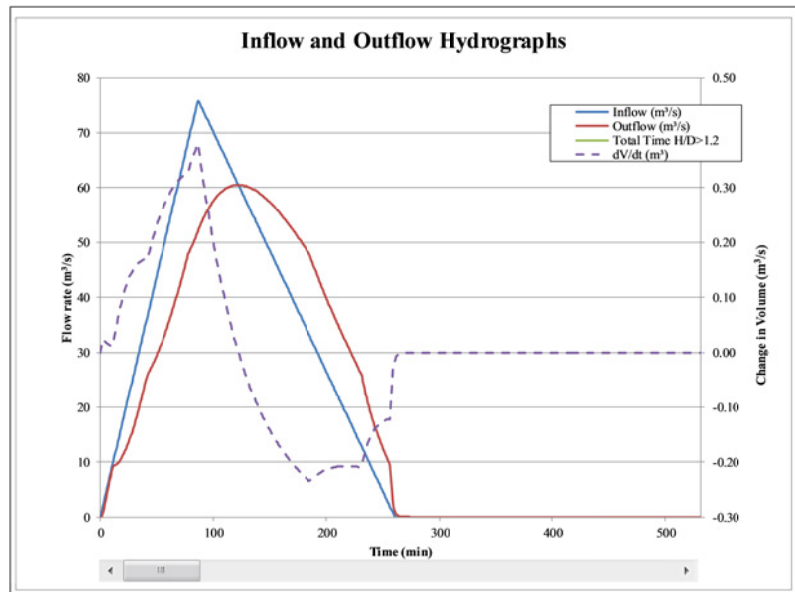
Variable	Value	Units
Maximum outflow	60.52	m <sup>3</sup> /s
Maximum inflow (Design Flow rate for normal routing or Q <sub>2T</sub> for the review of existing culverts)	76.00	m <sup>3</sup> /s
Total Time H <sub>1</sub> /D>1.2	0.00	min
Number of Culverts	2.00	units
Culvert Height	3.60	m
Culvert Width	3.00	m
Maximum allowable damming depth measured from invert of culvert	5.00	m
Calculated Time of Concentration (T <sub>c</sub> )	1.45	hours
Time duration of Outflow (dV/dt≈0)	5.16	hours

### Investigation of existing culvert structures (Refer to Figure 10.1 SANRAL DRAINAGE MANUAL)

	V <sub>T1</sub> (m <sup>3</sup> )	0,5 V <sub>storm</sub> (m <sup>3</sup> )	Pass/Fail
Review of maximum temporal storage	85527	297199	Pass

	T <sub>s</sub> (min)	0,5 T <sub>c</sub> (min)	Pass/Fail
Review of the duration of excessive energy (upstream inundation)	0.00	86.90	Pass

Overall culvert structure outcome:	Culvert Structure Pass
------------------------------------	------------------------



Developed by: **Sinotech**

Disclaimer: This program was developed for the convenience of its users. Although every reasonable effort has been made to ensure that the program is accurate and reliable the program developers, Sinotech CC and/or Prof SJ van Vuuren accept no liability of any kind for any results, interpretation thereof or any use made of the results obtained with these programs. All users of this program do so entirely at their own risk

Figure 10.12: Summary of the results for re-assessed culvert with 2 units of the same dimensions

## 11 FREE SURFACE FLOW DETERMINATION

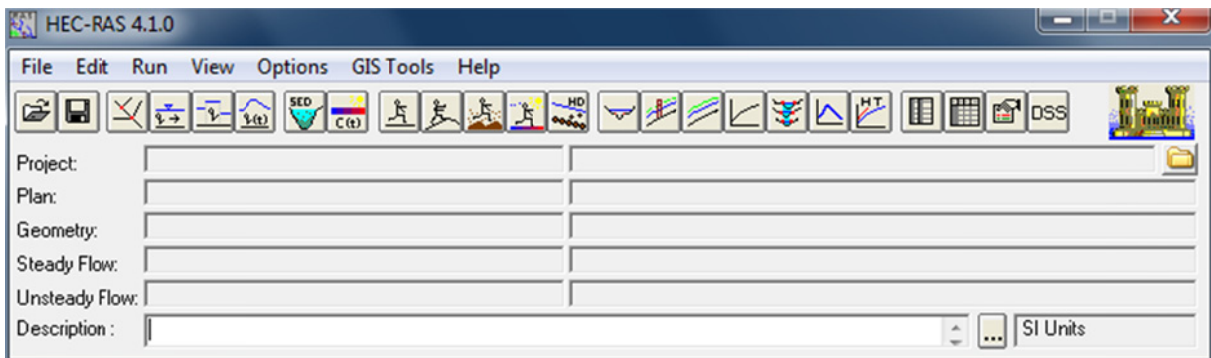
HEC-RAS is constantly being upgraded and the screen layouts on newer versions may differ from what is shown in this chapter.

### 11.1 Basic flood line determination (HEC-RAS)



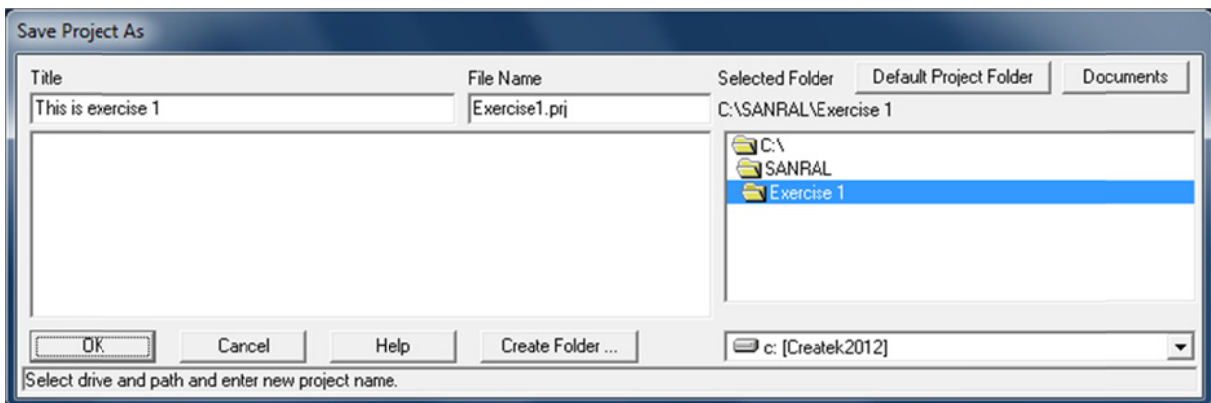
#### STARTING A NEW PROJECT

To begin this exercise, start the HEC-RAS program by double clicking the HEC-RAS icon on the desktop. The main window should appear as shown in **Figure 11.1**.



**Figure 11.1: HEC-RAS main window**

The first step in developing a HEC-RAS application is to start a new project. Go to **File** menu on the main window and select **New Project**. The New Project window should appear as shown in **Figure 11.2**. Set the drive and directory you would like to work in. Enter the *project title* and *file name* as shown in **Figure 11.2**. Once you have entered the information, press the **OK** button to accepted the title and file name and create the new project.



**Figure 11.2: New Project window**

Once back at the HEC-RAS Main window select from the menu bar **Options**, and set the units that you would like to work in to be metric units as well as be the default setting for all new projects (see **Figure 11.3** and **Figure 11.4**).

In the right hand corner of the main screen it will now indicate SI units.

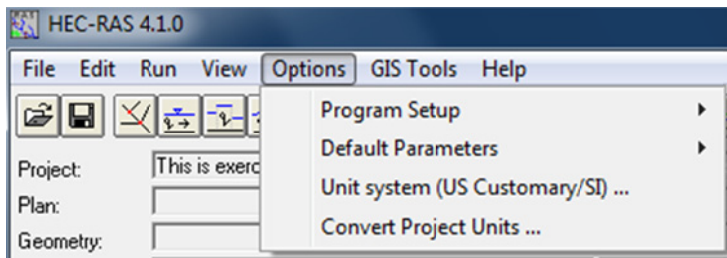


Figure 11.3: Options menu

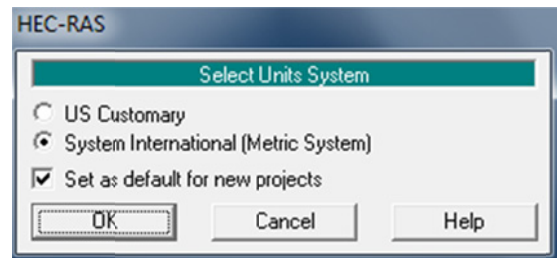




Figure 11.4: Unit systems

### ENTERING GEOMETRIC DATA

First a steady state flow model will be developed:

The next step is to enter the Geometric Data. This is accomplished by selecting **Geometric Data** from the **Edit** menu on the HEC-RAS Main window (Figure 11.1) or clicking the short cut button on the menu bar . Once this option is selected, the geometric data window will be shown (see Figure 5). This screen can be maximized by clicking on the maximize button  in the right hand corner of the window.

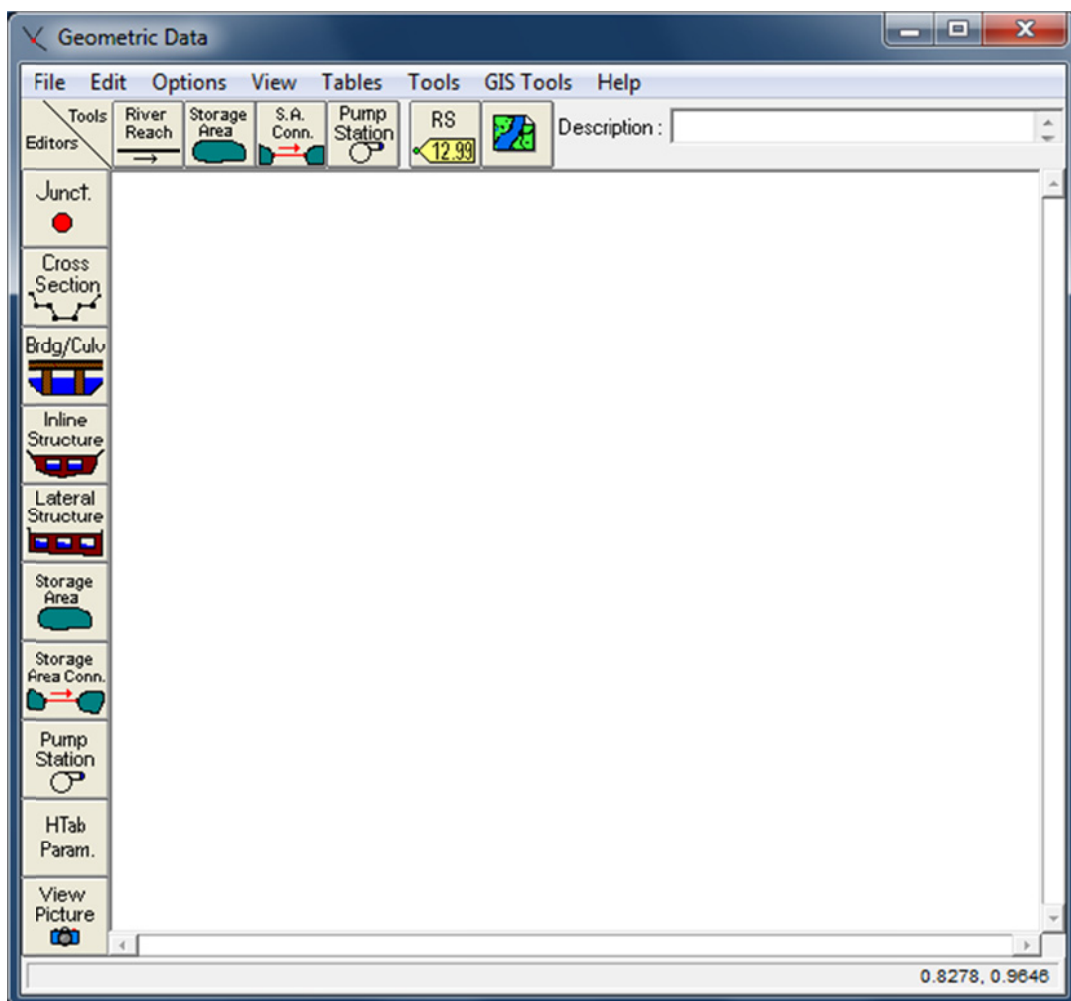
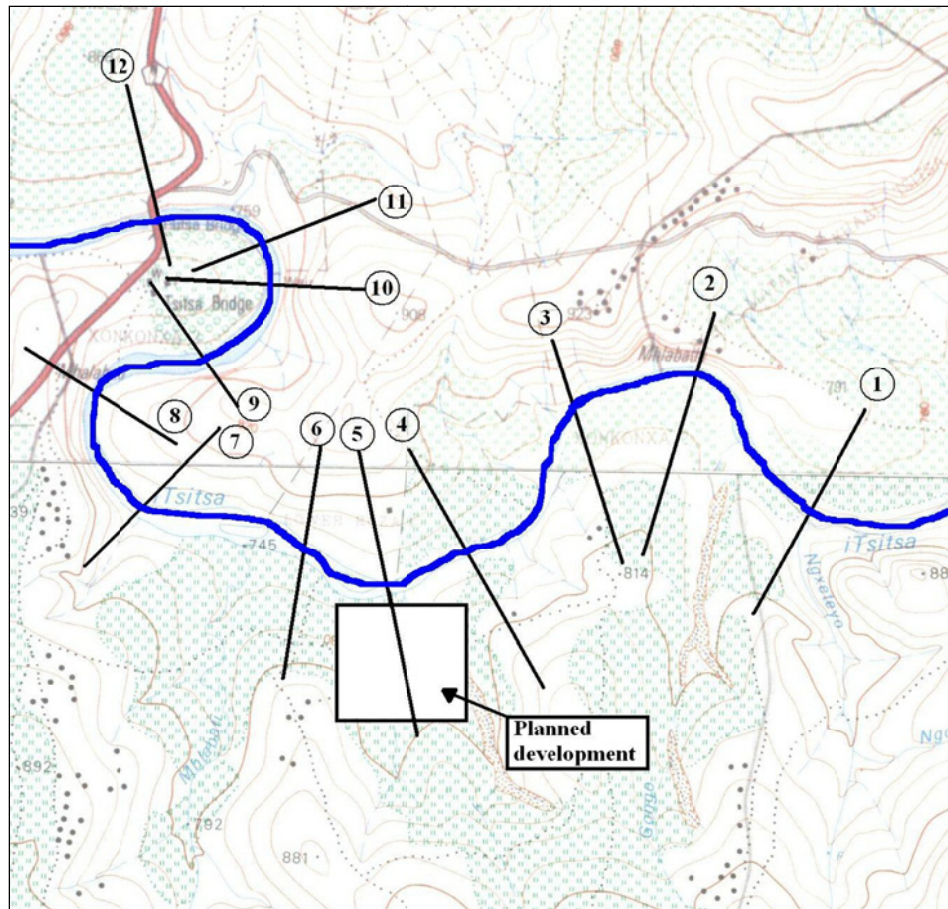


Figure 11.5: Geometric Data window


## Drawing the schematic of the river system

A plan view of the river section with cross sections is shown below in **Figure 11.6**.

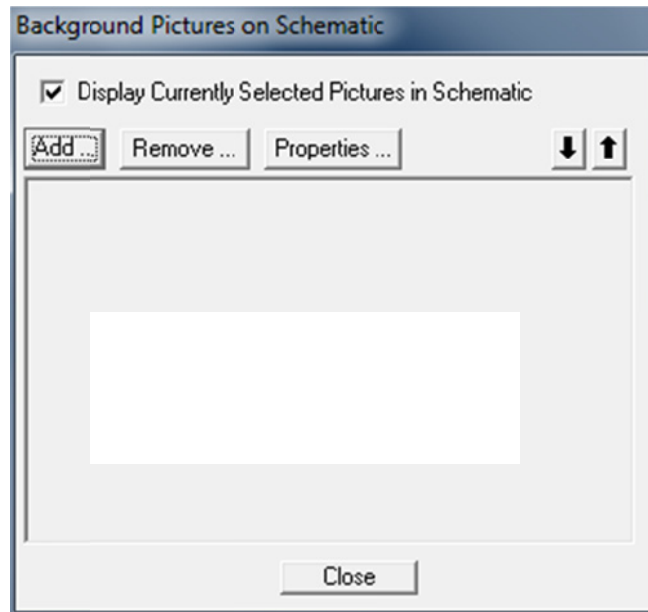


**Figure 11.6: Plan view of river section**

The first step is to draw the river system schematically by performing the following steps:

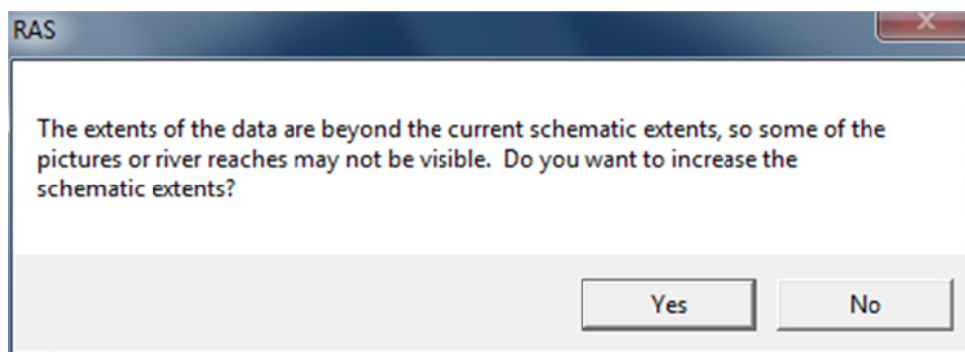
- Although it is only a schematic drawing one would still like to draw it more or less to scale. To assist with this HEC-RAS has an option to import a background picture.
- Click on the **Add/Edit background picture** button  on the menu bar. The program will show the Background Pictures on Schematic window (**Figure 11.7**).





**Figure 11.7: Background Pictures on Schematic window**

- Click on the **Add** button and select the background picture (Exercise 1 – Background picture.jpg)
- HEC-RAS will indicate that the picture extents past the current schematic boundaries and ask you if you would want to increase the schematic extents (**Figure 11.8**). Select **Yes**.



**Figure 11.8: Increasing the schematic extents**

- Once the picture has been selected click on the **Close** button (**Figure 11.7**).
- Use the Schematic view selector box to drag the extents of the inserted picture over the entire screen (**Figure 11.9**).
- Now the river section can be drawn over the background picture. Click the **River Reach** button on the geometric data window.
- Move the mouse pointer over the drawing area and place the pointer at the location in which you would like to start drawing the reach.

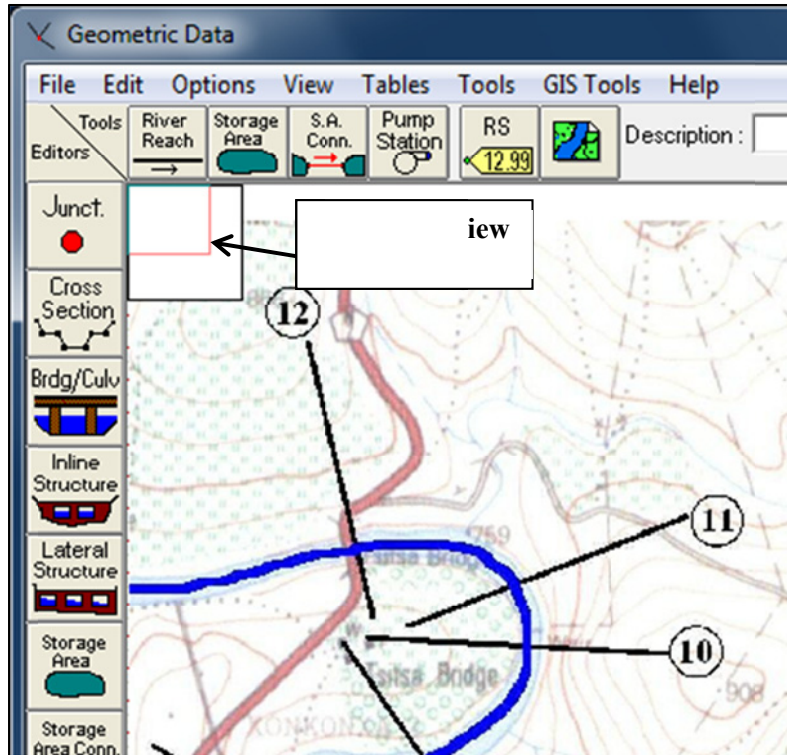



Figure 11.9: Viewing the background picture

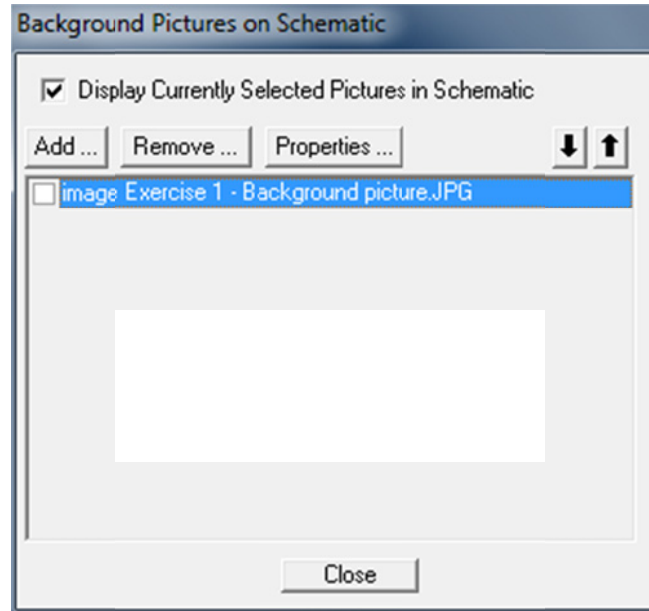
- Press the left mouse button once to start drawing the reach. Move the mouse pointer and continue to press the left mouse button to add additional points to the line segment. To end the drawing of the reach, double click the left mouse button and the last point on the reach will be placed at the current mouse pointer location (right click will remove the last point drawn). All reaches must be drawn from the upstream to downstream (in the positive flow direction) i.e. **start at cross section 12 down to cross section 1** (Decreasing numeric values).
- Once the reach is drawn, the interface will prompt you to enter an identifier for the **River** name and the **Reach** name. The **River** identifier can be up to 32 characters, while the reach name is limited to 12 characters. In this exercise the river will be called, *Tsitsa* and the reach *Lower reach* (see Figure 10).



Figure 11.10: River and reach names

Once you have finished the drawing of the river system, there are several options available for editing the schematic. The options include changing the name, adding points to a reach, removing points from a reach, deleting a reach, and deleting a junction. The editing features are located under the **Edit** menu on the Geometric Data window.

- Since the schematic of the river has now been drawn there is no more need for the background picture. To switch off the background picture click on the **Add/Edit background picture** button  on the menu bar. Deselect the picture as shown in **Figure 11.11**.



**Figure 11.11: Deselecting the background picture**

- When you first draw the schematic there will be no tic marks representing the cross sections as shown in **Figure 11.12**. The tic marks only show up after you have entered cross sections.



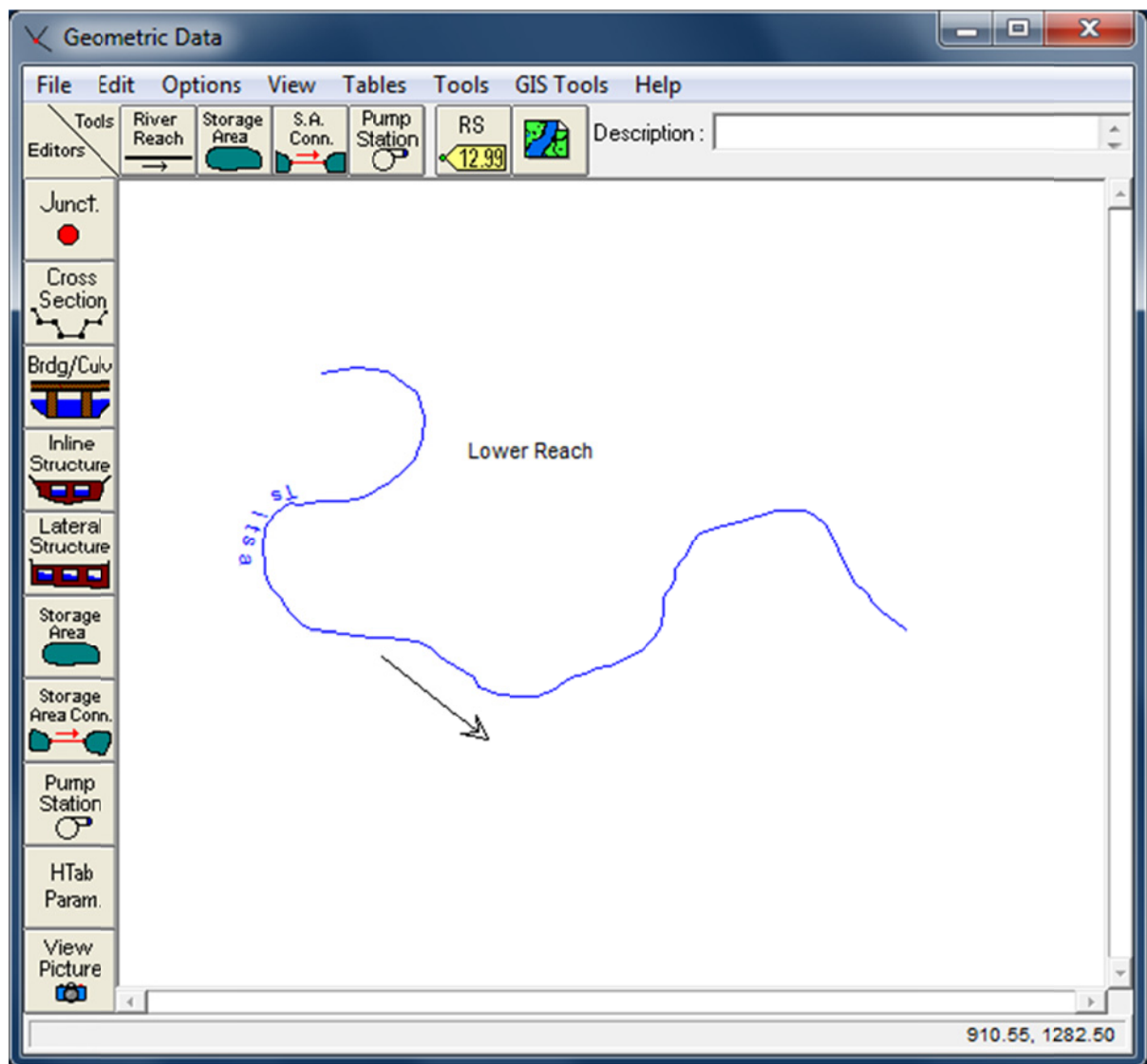


Figure 11.12: Geometric Data window with Tsitsa river schematic

### Entering cross section data



The next step is to enter the cross section data. This is accomplished by clicking the **cross section** button on the Geometric window (**Figure 11.12**). Once this button is clicked, the Cross Section Data editor will appear as shown in **Figure 11.13**.

**Cross Section Data Editor**

Exit Edit Options Plot Help

River: Tsitsa Apply Data

Reach: Lower Reach River Sta.:

Description

Del Row Ins Row

Cross Section Coordinates	
Station	Elevation
1	
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	

Downstream Reach Lengths

LOB	Channel	ROB
-----	---------	-----

Manning's n Values

LOB	Channel	ROB
-----	---------	-----

Main Channel Bank Stations

Left Bank	Right Bank
-----------	------------

Cont'Exp Coefficient (Steady Flow)

Contraction	Expansion
-------------	-----------

**Figure 11.13: Cross section Data Editor**

To enter cross section data follow these steps:

- Select a **River** and a **Reach** to work with. In this exercise there is only one River (Tsitsa) and one Reach (Lower reach).
- Go to the **Options** menu and select **Add a new Cross Section**. An input box will appear to prompt you to enter a river station identifier for the new cross section (see **Figure 11.14**).

**HEC-RAS**

Enter a new river station for the new cross section in reach "Lower Reach"

12

OK Cancel

**Figure 11.14: Add a new river station**

The identifier does not have to be the actual river station, but it must be a numerical value. The numeric value describes where the cross section is located in reference to all other cross sections within the reach. Cross sections are located from upstream (highest river station) to downstream (lowest river station). For this cross section enter a value of 12.

- For this cross section, enter all the data as shown in **Figure 11.15**.

**Cross Section Data - Natural Tsitsa river**

Exit Edit Options Plot Help

River: Tsitsa Apply Data

Reach: Lower reach River Sta.: 12

Description: Upstream boundary of this river section

Del Row Ins Row

Cross Section Coordinates	
	Station Elevation
1	100 100.3
2	103 97.3
3	125 96.1
4	133 94.4
5	135 94.3
6	138 95.6
7	148 96
8	154 99.5
9	
10	
11	

Downstream Reach Lengths

LOB	Channel	ROB
100	95	90

Manning's n Values

LOB	Channel	ROB
0.05	0.035	0.05

Main Channel Bank Stations

Left Bank	Right Bank
125	138

Cont'Exp Coefficient (Steady Flow)

Contraction	Expansion
0.1	0.3


Edit Station Elevation Data (m)

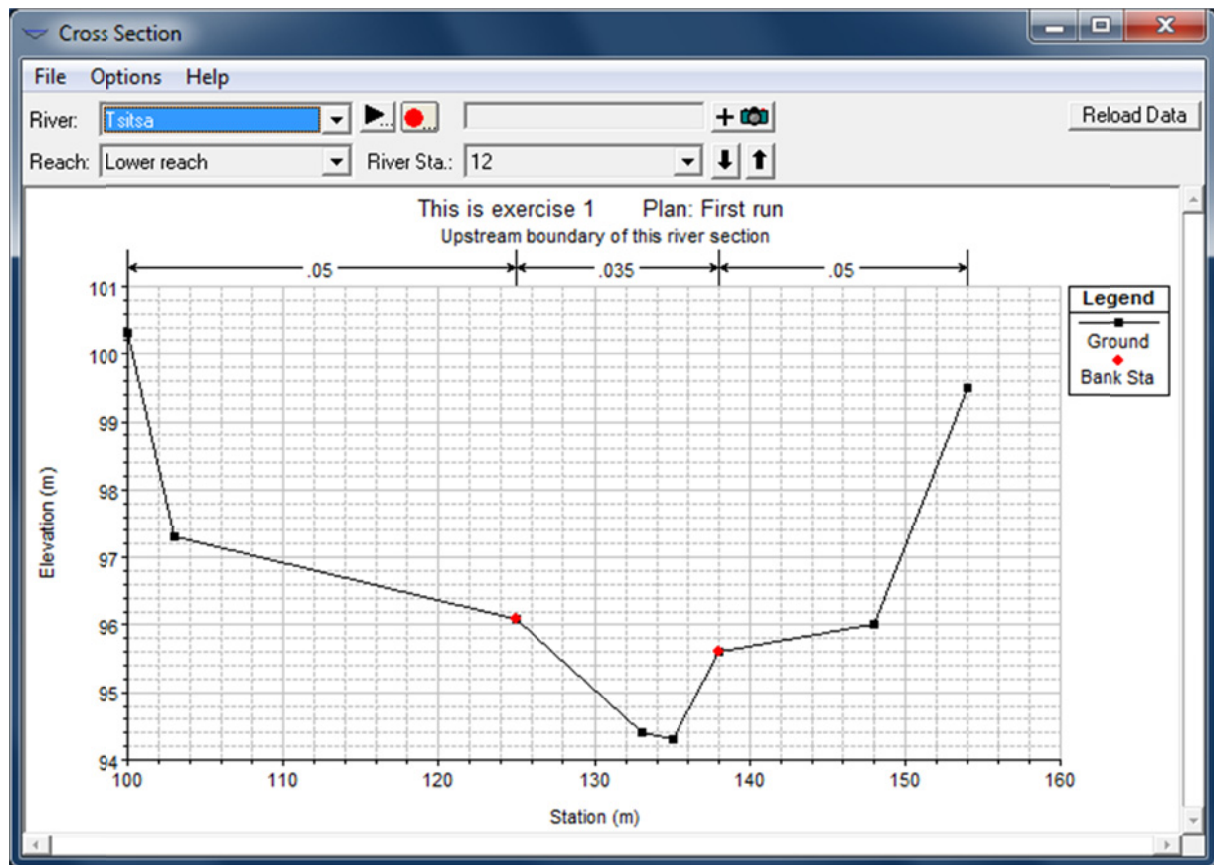
**Figure 11.15: Cross Section Data Editor with data**

- Enter the:  
 Description: *Upstream boundary of this river section*  
 Downstream reach lengths: LOB = 100, Channel = 95 and ROB = 90  
 Manning n-values: LOB = 0,05, Channel = 0,035 and ROB = 0,05  
 Station and elevation details:

Nr	Station	Elevation
1	100	100,3
2	103	97,3
3	125	96,1
4	133	94,4
5	135	94,3
6	138	95,6
7	148	96,0
8	154	99,5

Main channel stations: Left bank = 125 and Right bank = 138  
 Cont'Exp coefficients: Contraction = 0,1 and Expansion = 0,3

- Once all the data is entered press the **Apply Data** button. This button is used to instruct the program to accept the entered data into memory. This button does not save the data to the hard disk. This is done by clicking on **Save Geometry Data** under the **File** menu on the Geometric Data window (which will be explained later).
- Plot the cross section to visually inspect the data. This is accomplished by pressing the **Plot Cross Section** option under the **Plot** menu on the Cross Section Data Editor or by clicking on the **Expand XS editor to include a XS plot** . The cross section should look similar to that shown in **Figure 11.16** below.



**Figure 11.16: Cross section plot for river station 12**

Note the manning roughness values at the top of the cross section and the specified bank stations (red dots) (**Figure 11.16**).

In practice the steps listed above would be repeated for every cross section that is entered. In order to reduce the amount of data entry for this exercise, the current cross section will be copied and adjusted to represent other cross sections within the river system.

the

## METHOD 1

**Cross Section Data - Natural Tsitsa river**

Exit Edit Options Plot Help

River: Tsitsa Apply Data

Reach: Lower reach River Sta.: 11

Description: Tsitsa River 11

Cross Section Coordinates		Downstream Reach Lengths		
Station	Elevation	LOB	Channel	ROB
1	100	48	42	40
2	102.7			
3	122.5			
4	130.5			
5	132.5			
6	135.5			
7	144.5			
8	149.9			
9				
10				
11				

Manning's n Values		
LOB	Channel	ROB
0.05	0.035	0.05

Main Channel Bank Stations	
Left Bank	Right Bank
122.5	135.5

Cont'Exp Coefficient (Steady Flow)	
Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

**Figure 11.17: Cross section 11 data**

**Cross Section Data - Natural Tsitsa river**

Exit Edit Options Plot Help

River: Tsitsa Apply Data

Reach: Lower reach River Sta.: 10

Description: Tsitsa River 10

Cross Section Coordinates		Downstream Reach Lengths		
Station	Elevation	LOB	Channel	ROB
1	100	95	90	85
2	103.24			
3	127			
4	135			
5	137			
6	140			
7	149.9			
8	155.84			
9				
10				
11				

Manning's n Values		
LOB	Channel	ROB
0.05	0.035	0.05

Main Channel Bank Stations	
Left Bank	Right Bank
127	140

Cont'Exp Coefficient (Steady Flow)	
Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

**Figure 11.18: Cross section 10 data**

**Cross Section Data - Natural Tsitsa river**

Exit Edit Options Plot Help

River: Tsitsa Apply Data

Reach: Lower reach River Sta.: 9

Description: Tsitsa River 9

Cross Section Coordinates		Downstream Reach Lengths		
Station	Elevation	LOB	Channel	ROB
1	100	100	102	105
2	103.24			
3	127			
4	135.8			
5	138			
6	141.3			
7	151.2			
8	157.14			
9				
10				
11				

Manning's n Values		
LOB	Channel	ROB
0.05	0.035	0.05

Main Channel Bank Stations	
Left Bank	Right Bank
127	141.3

Cont'Exp Coefficient (Steady Flow)	
Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

**Figure 11.19: Cross section 9 data**

**Cross Section Data - Natural Tsitsa river**

Exit Edit Options Plot Help

River: Tsitsa Apply Data

Reach: Lower reach River Sta.: 8

Description: Tsitsa River 8

Cross Section Coordinates		Downstream Reach Lengths		
Station	Elevation	LOB	Channel	ROB
1	100	100	110	120
2	103.24			
3	127			
4	138.44			
5	141.3			
6	145.59			
7	155.49			
8	161.43			
9				
10				
11				

Manning's n Values		
LOB	Channel	ROB
0.05	0.035	0.05

Main Channel Bank Stations	
Left Bank	Right Bank
127	145.59

Cont'Exp Coefficient (Steady Flow)	
Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

**Figure 11.20: Cross section 8 data**

**Cross Section Data - Natural Tsitsa river**

Exit Edit Options Plot Help

River: Tsitsa Apply Data

Reach: Lower reach River Sta.: 7

Description: Tsitsa River 7

Cross Section Coordinates		Downstream Reach Lengths		
Station	Elevation	LOB	Channel	ROB
1	100	120	120	125
2	104.86			
3	140.5			
4	151.94			
5	154.8			
6	159.09			
7	168.99			
8	174.93			
9				
10				
11				

Manning's n Values		
LOB	Channel	ROB
0.05	0.035	0.05

Main Channel Bank Stations	
Left Bank	Right Bank
140.5	159.09

Cont'Exp Coefficient (Steady Flow)	
Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

**Figure 11.21: Cross section 7 data**

**Cross Section Data - Natural Tsitsa river**

Exit Edit Options Plot Help

River: Tsitsa Apply Data

Reach: Lower reach River Sta.: 6

Description: Tsitsa River 6

Cross Section Coordinates		Downstream Reach Lengths		
Station	Elevation	LOB	Channel	ROB
1	100	80	82	85
2	104.86			
3	140.5			
4	163.38			
5	169.1			
6	177.68			
7	197.48			
8	209.36			
9				
10				
11				

Manning's n Values		
LOB	Channel	ROB
0.05	0.035	0.05

Main Channel Bank Stations	
Left Bank	Right Bank
140.5	177.68

Cont'Exp Coefficient (Steady Flow)	
Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

**Figure 11.22: Cross section 6 data**



Cross Section Data - Natural Tsitsa river

River: Tsitsa

Reach: Lower reach River Sta.: 5

Description: Cross section at site

Del Row	Ins Row	Station	Elevation
1		100	99.55
2		109.72	96.55
3		181	95.35
4		203.88	93.65
5		209.6	93.55
6		218.18	94.85
7		237.98	95.25
8		249.86	98.75
9			
10			
11			

LOB	Channel	ROB
80	80	80

LOB	Channel	ROB
0.05	0.035	0.05

Left Bank	Right Bank
181	218.18

Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

Figure 11.23: Cross section 5 data

Cross Section Data - Natural Tsitsa river

River: Tsitsa

Reach: Lower reach River Sta.: 4

Description: Tsitsa River 4

Del Row	Ins Row	Station	Elevation
1		100	99.5
2		104.86	96.5
3		140.5	95.3
4		163.38	93.6
5		169.1	93.5
6		177.68	94.8
7		197.48	95.2
8		209.36	98.7
9			
10			
11			

LOB	Channel	ROB
180	180	180

LOB	Channel	ROB
0.05	0.035	0.05

Left Bank	Right Bank
140.5	177.68

Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

Figure 11.24: Cross section 4 data

Cross Section Data - Natural Tsitsa river

River: Tsitsa

Reach: Lower reach River Sta.: 3

Description: Tsitsa River 3

Del Row	Ins Row	Station	Elevation
1		100	99.38
2		104.86	96.38
3		140.5	95.18
4		163.38	93.48
5		169.1	93.38
6		177.68	94.68
7		187.58	95.08
8		193.52	98.58
9			
10			
11			

LOB	Channel	ROB
115	105	100

LOB	Channel	ROB
0.05	0.035	0.05

Left Bank	Right Bank
140.5	177.68

Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

Figure 11.25: Cross section 3 data

Cross Section Data - Natural Tsitsa river

River: Tsitsa

Reach: Lower reach River Sta.: 2

Description: Tsitsa River 2

Del Row	Ins Row	Station	Elevation
1		100	99.2
2		102.43	96.2
3		120.25	95
4		131.69	93.3
5		134.55	93.2
6		138.84	94.5
7		148.74	94.9
8		154.68	98.4
9			
10			
11			

LOB	Channel	ROB
155	155	150

LOB	Channel	ROB
0.05	0.035	0.05

Left Bank	Right Bank
120.25	138.84

Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

Figure 11.26: Cross section 2 data

Cross Section Data - Natural Tsitsa river

River: Tsitsa

Reach: Lower reach River Sta.: 1

Description: Downstream boundary of river section

Del Row	Ins Row	Station	Elevation
1		100	99.1
2		101.944	96.1
3		116.2	94.9
4		127.64	93.2
5		130.5	93.1
6		134.79	94.4
7		144.69	94.8
8		150.63	98.3
9			
10			
11			

LOB	Channel	ROB
0	0	0

LOB	Channel	ROB
0.05	0.035	0.05

Left Bank	Right Bank
116.2	134.79

Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

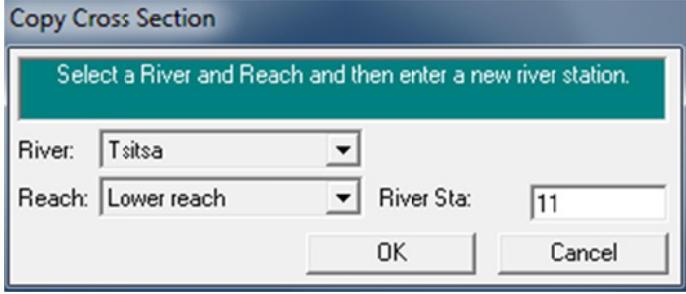
Figure 11.27: Cross section 1 data

If you have completed entering the cross section data as shown in **Figure 11.17** to **Figure 11.27** skip the next two pages.

## METHOD 2

# **Expert users:** Follow the following steps to copy the current cross sections and adjust to look similar to **Figure 11.17** to **Figure 11.27**.

- Go to the **Options** menu on the cross Section Data Editor and select **Copy Current Cross Section**. An input box will appear to prompt you to select a river reach, and then enter a river station for the new cross section, see **Figure 11.28**. For this exercise, keep the river and reach as *Tsitsa River* and *Lower reach*, then enter a new river station of *11*.



**Figure 11.28: Copying and existing cross section**

Press the **OK** button and the new cross section will appear in the editor.

- Change the cross section description to "*Tsitsa River 11*".
- Adjust all the elevations of the cross sections by  $-0,2$  meter. This is accomplished by selecting the **Adjust Elevations** feature from the **Options** menu on the Cross Section Data Editor.
- Adjust the cross section stationing to reduce the overbanks by 10%.
- This is accomplished by selecting the **Adjust Stations** feature from the **Options** menu on the Cross Section Data Editor, then select **Multiply by a Factor**. When the input box appears for this option, three data entry fields will be available to adjust the stationing of the left overbank, channel, and the right overbank separately. Enter values of 0,9 for the right and left overbanks, but leave the main channel field blank. This will reduce the stationing of both overbanks by 10%, but leave the main channel unchanged.
- Downstream reach lengths change to  $LOB = 48$ ,  $Channel = 42$  and  $ROB = 40$  for this cross section.
- Press the **Apply Data** button (The data should be similar to that shown in **Figure 11.17**). Plot the cross section to visually inspect it (see **Figure 11.29**).

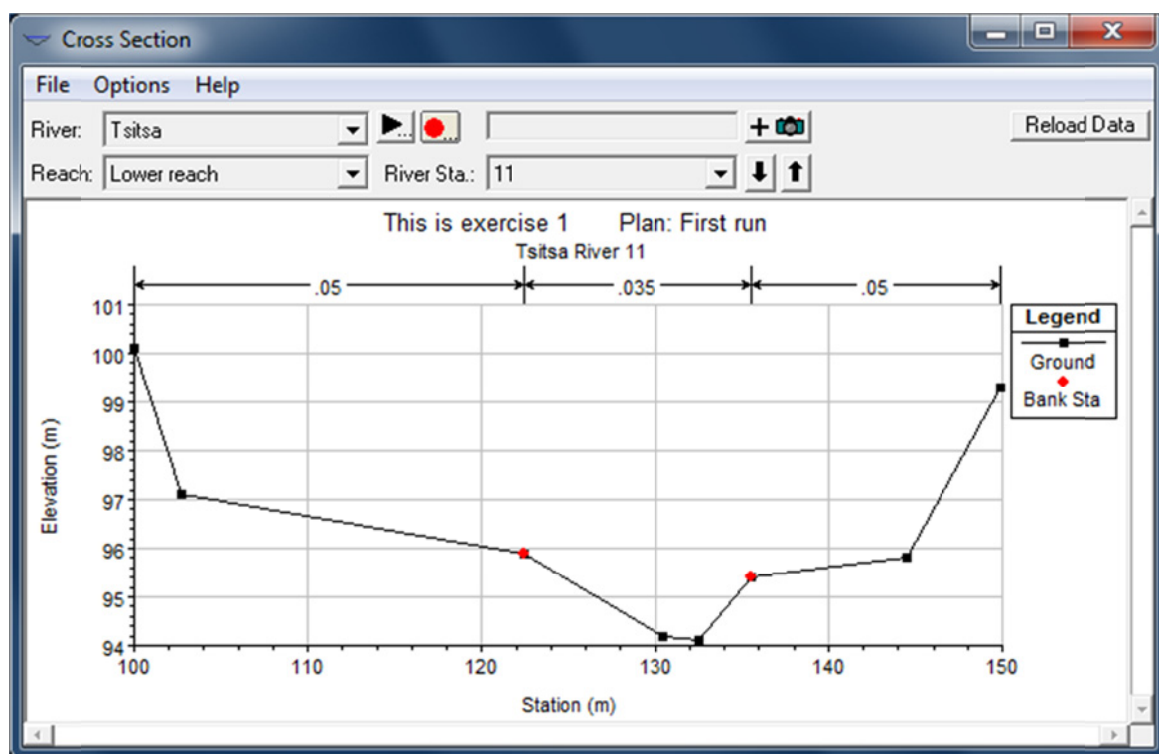


Figure 11.29: Cross section plot (cross section 11)

These seven steps above should be repeated to enter all the data for Tsitsa River (Lower Reach). The necessary adjustments are listed in **Table 11.1**. Perform the cross section duplications in order that they are listed in the table. Make sure to change the description of each cross section, and also press the **Apply Data** button after making the adjustments for each cross section.

Table 11.1: Cross section adjustments for duplicating sections

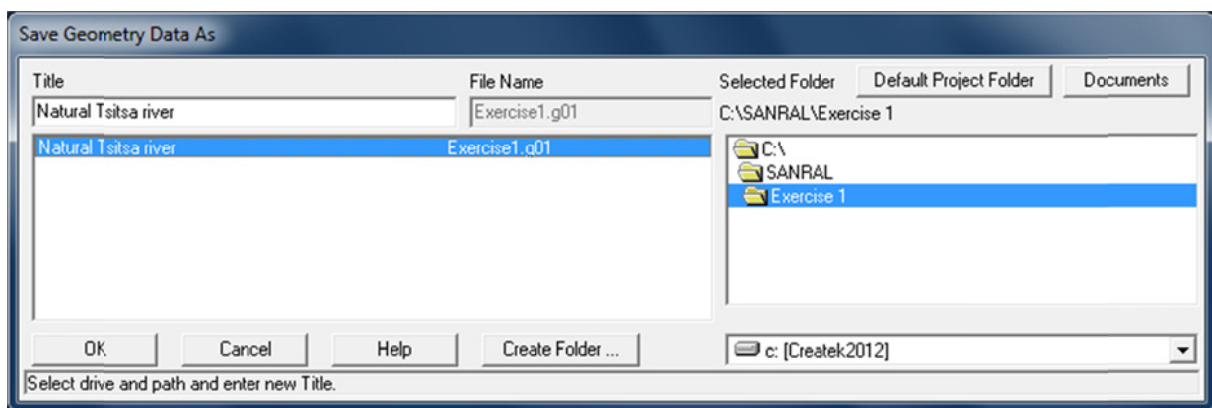
Cross section		Adjusted elevation	Adjusted stationing			Downstream reach lengths		
Description	River station		Left O.B.	Channel	Right O.B.	Left O.B.	Channel	Right O.B.
Tsitsa River 11	11	-0,20	0,9	-	0,9	48	42	40
Tsitsa River 10	10	-0,10	1,2	1,0	1,1	95	90	85
Tsitsa River 9	9	-0,10	-	1,1	-	100	102	105
Tsitsa River 8	8	-0,05	-	1,3	-	100	110	120
Tsitsa River 7	7	-0,10	1,5	-	-	120	120	125
Tsitsa River 6	6	-0,15	-	2,0	2,0	80	82	85
Cross section at site	5	-0,05	2,0	-	-	80	80	80
Tsitsa River 4	4	-0,05	0,5	1,0	1,0	180	180	180
Tsitsa River 3	3	-0,12	-	-	0,5	115	105	100
Tsitsa River 2	2	-0,18	0,5	0,5	-	155	155	150
Downstream boundary of river section	1	-0,10	0,8	-	-	0	0	0



**Table 11.1** is simply a quick way to generate varying cross sections for this exercise. The method however is quite useful when working with a man-made structure such as a channel, which has similar cross sections.

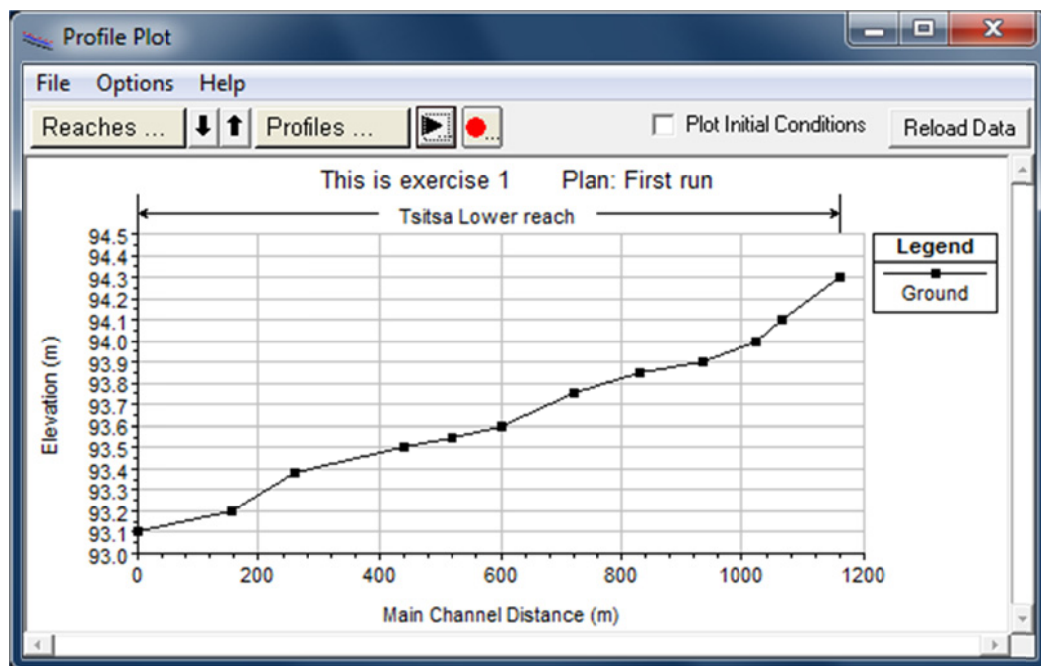
### # Novice & Expert users continue

This completes all the cross section data for the Tsitsa River (Lower reach) save the data file before continuing. Saving the data to a file is achieved by exiting the Cross Section Data editor window and selecting the **Save Geometry Data As** option from the **File** menu on the Geometric Data window. After selecting this option you will be prompted to enter a Title for the geometric data (**Figure 30**). Enter “*Natural Tsitsa River*” for this exercise, and then press the **OK** button. A file name is automatically assigned to the geometry data based on what you entered for the project file name i.e. Exercise1.g01.



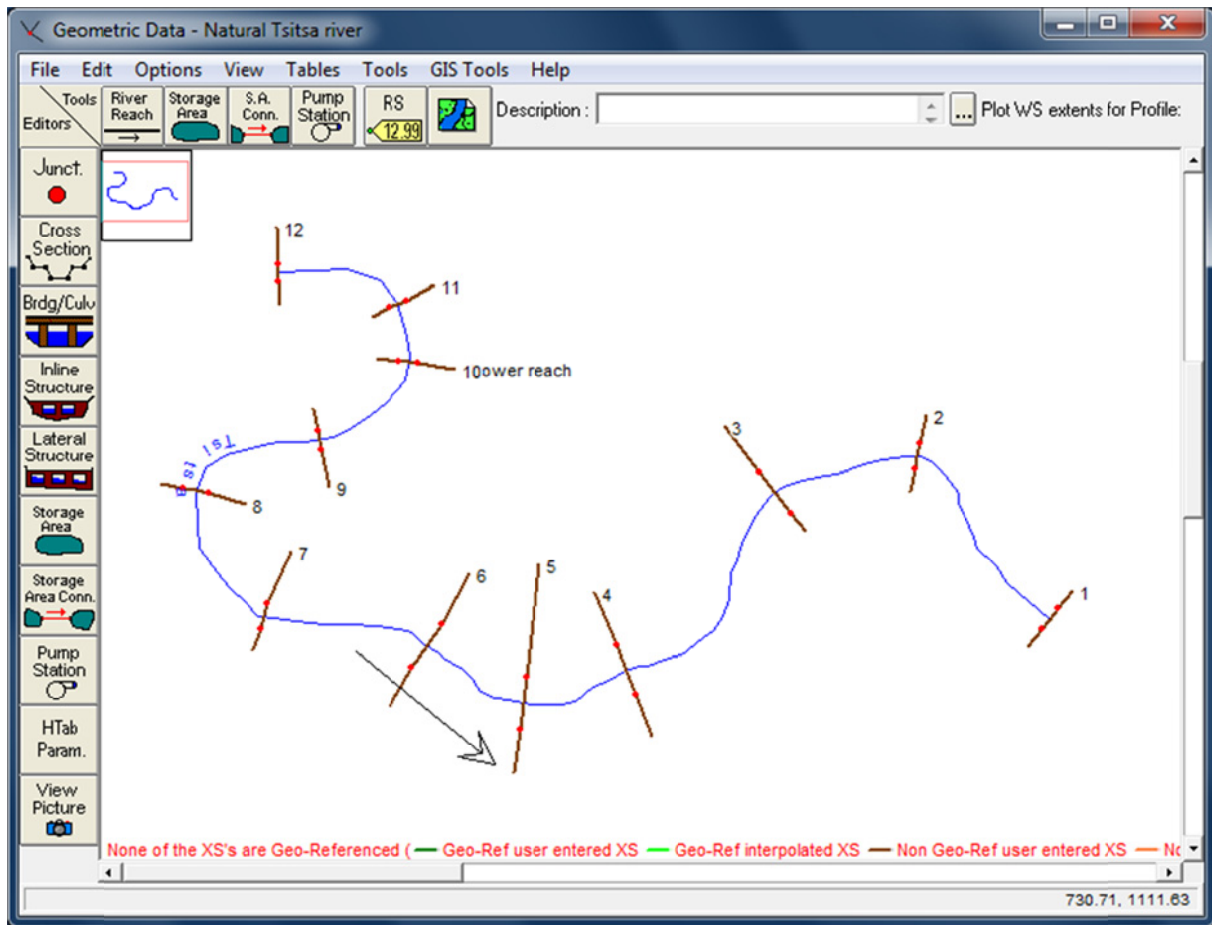
**Figure 11.30: Save Geometry Data As**

- Return to the Cross Section Data editor window and from the **Plot** menu select **Plot Profile...** to view the longitudinal profile of the entered cross sections (see **Figure 11.31**).



**Figure 11.31: Longitudinal profile plot**


When returning to the Geometric Data window the user will notice that the Tsitsa River now also shows the schematic layout with the position of the entered cross sections, see **Figure 11.32**.



**Figure 11.32: Schematic layout of entered system**

We have completed the required geometric data and can now continue and enter the Steady Flow Data.

### **ENTERING STEADY FLOW DATA**

The next step in developing the required data to perform steady flow water surface profile calculations is to enter the steady flow data. To bring up the steady flow data editor, select **Steady Flow Data** from the **Edit** menu on the HEC-RAS main window or click on the **Steady Flow Data** button  on the menu bar. The steady Flow Data editor should appear as shown in **Figure 11.33**.

Steady Flow Data - Calculated flood peak

File Options Help

Enter/Edit Number of Profiles (25000 max):  Reach Boundary Conditions ... Apply Data

Locations of Flow Data Changes

River:  Add Multiple...

Reach:  River Sta.:  Add A Flow Change Location

Flow Change Location			Profile Names and Flow Rates		
	River	Reach	RS	PF 1	PF 2
1	Tsitsa	Lower reach	12		

Edit Steady flow data for the profiles (m3/s)

Figure 11.33: Steady Flow Data window

- The first set of required data to enter is the number of profiles to be calculated. For this exercise enter “3” as shown in **Figure 34** (and click on the **Apply** button). The next step is to enter the flow data. Flow data are entered from upstream to downstream for each reach. At least one flow rate must be entered for every reach in the river system. Once a flow value is entered at the upstream end of a reach, it is assumed that the flow remains constant until another flow value is encountered within the reach. Additional flow values can be entered at any cross section location within a reach. In this exercise there is only 1 reach and thus it will only be required to enter 1 set of flow data. In this exercise, flow data will be entered at the upstream end of the reach i.e. at cross section 12.
- Profile labels will automatically default to “PF1” and “PF2” etc. These labels can be changed to whatever is descriptive of the flow. In this exercise these should be changed to *1:20 yr*, *1:50 yr* and *1:100 yr*. Under the **Options** menu go to **Edit Profile Names** and change the profile names. Click on the **OK** button to accept the names.
- Enter the 1:20, 1:50 and 1:100 year flood peak values of 35, 80 and 150 m<sup>3</sup>/s respectively (see **Figure 34**).

Steady Flow Data - Calculated flood peaks

File Options Help

Enter/Edit Number of Profiles (25000 max): 3 Reach Boundary Conditions ... Apply Data

Locations of Flow Data Changes

River: Tsitsa Add Multiple...

Reach: Lower reach River Sta.: 12 Add A Flow Change Location

Flow Change Location			Profile Names and Flow Rates			
	River	Reach	RS	1:20 yr	1:50 yr	1:100 yr
1	Tsitsa	Lower reach	12	35	80	150

Edit Steady flow data for the profiles (m3/s)

Figure 11.34: Steady flow data

- The next step is to enter any required boundary conditions. To enter boundary conditions, press the **Reach Boundary Conditions** button at the top of the Steady Flow Data editor (see Figure 11.34). The boundary conditions editor will appear as shown in Figure 11.35.

Steady Flow Boundary Conditions

☒ Set boundary for all profiles ☐ Set boundary for one profile at a time

Available External Boundary Condition Types

Known W.S. Critical Depth Normal Depth Rating Curve Delete

Selected Boundary Condition Locations and Types

River	Reach	Profile	Upstream	Downstream
Tsitsa	Lower reach	all		

Steady Flow Reach-Storage Area Optimization ... OK Cancel Help

Enter to accept data changes.

Figure 11.35: Steady Flow Boundary Conditions

- Boundary conditions are necessary to establish the starting water surface at the boundaries of the river system. A starting water surface is necessary in order for the program to begin calculations. In a subcritical flow regime, boundary conditions are only required at the downstream end of the river system.

If a mixed flow regime calculation is going to be performed, then boundary conditions must be entered at all open ends of the river system.

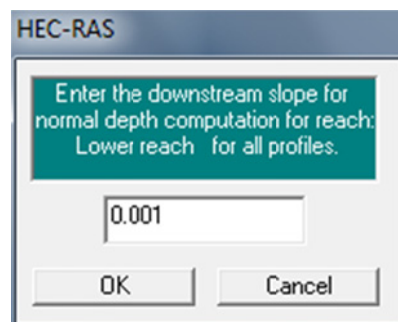
The boundary conditions editor contains a table listing of every river and reach. Each reach has an upstream and a downstream boundary condition. Connections to junctions are considered internal boundary conditions. Internal boundary conditions are automatically listed in the table, based on how the river system is connected in the geometric data editor. The user is only required to enter the necessary external boundary conditions.

In this exercise it is assumed that the flow is subcritical throughout the river system (**Verify if this is correct!!!**). Therefore, it is only necessary to enter a boundary condition at the downstream end of the Tsitsa River, Lower reach. Boundary conditions are entered by first selecting the cell in which you wish to enter a boundary condition. Then the type of boundary condition is selected from the four available types listed above the table see **Figure 11.35** (Known Water Surface, Critical Depth, Normal Depth or a Rating Curve).

**In this exercise it is assumed that there are no control points in the river further downstream and that cross section 1 is far enough downstream and can be assumed to flow at normal flow depth.**

- Click in the cell in the Table (**Figure 11.35**) under the Downstream column and then click on the **Normal Depth** button. In other words the program will start at this downstream boundary, calculate the normal flow depth and work systematically upstream in the river section.

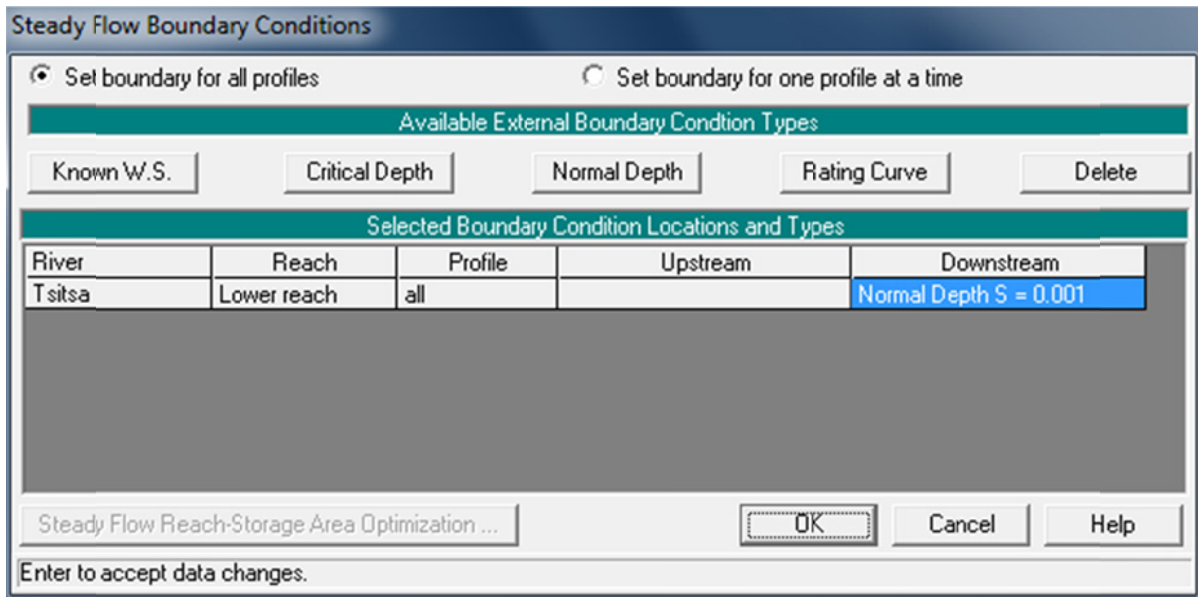
A pop up box will appear (see **Figure 11.36**) requesting you to enter an average slope at the downstream end of the river reach. Enter a value of 0,001 (m/m) then click on the **OK** button.



**Figure 11.36: Pop-up box (enter the average downstream slope)**

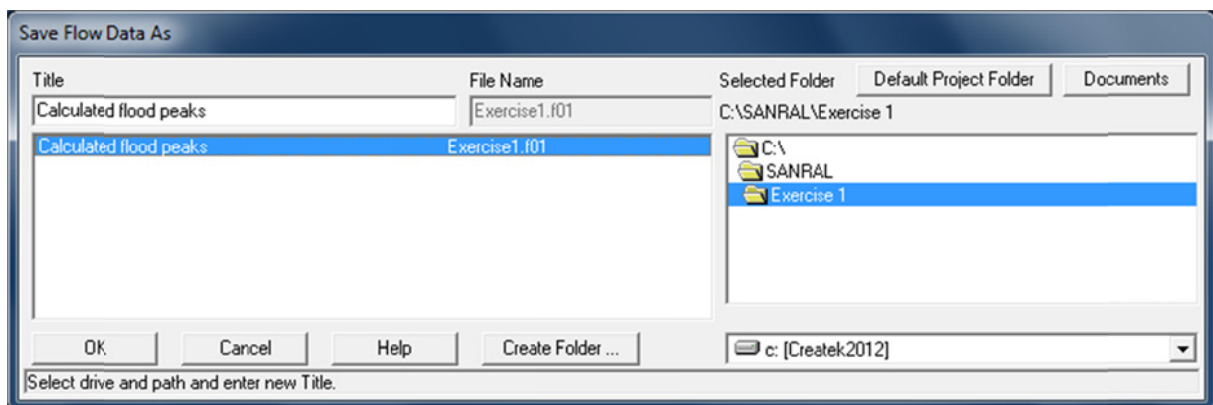
This completes all of the necessary boundary condition data (see **Figure 11.37**).

- Click the **OK** button on the Boundary Conditions window to accept the data and return to the Steady Flow Data screen.



**Figure 11.37: Accepted boundary condition data**

- The last step is to save the data to a file. To save the data, select the **Save Flow Data As** option from the **File** menu on the Steady Flow Data Editor. A pop-up box will prompt you to enter a description/title of the flow data (**Figure 38**). For this exercise enter: *Calculated flood peaks*. A file name is automatically assigned to the steady flow data based on what you entered for the project file name i.e. Assignment1.f01.




**Figure 11.38: Save Flow Data As**

Once the data has been saved, you can close the Steady Flow Data Editor.

### **PERFORMING THE HYDRAULIC CALCULATIONS**

Now that all of the data has been entered, we can calculate the steady water surface profiles. To perform the simulations, go to the HEC-RAS main window and select Steady Flow Analysis from the

**Run** menu or click on the **Steady Flow Analysis** button  on the menu bar. The **Steady Flow Analysis** window should appear as shown in **Figure 11.39**, except yours will not have any plan title or Short ID yet.



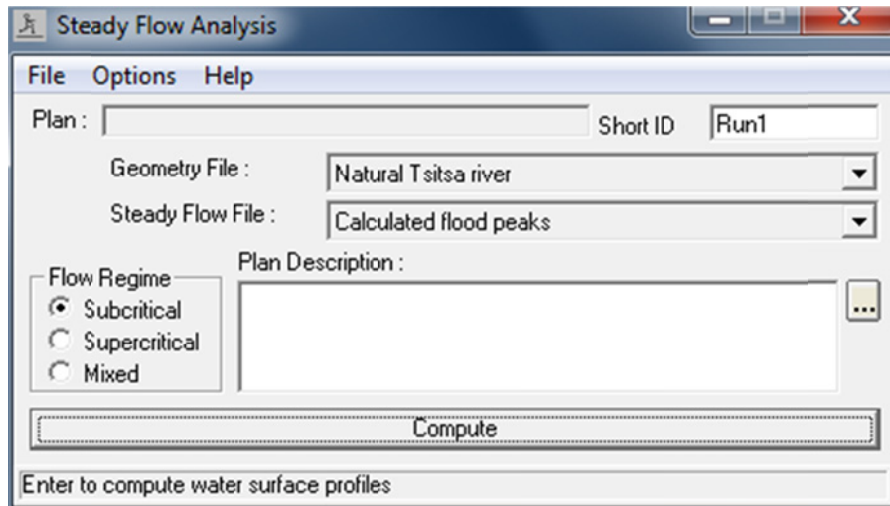


Figure 11.39: Steady Flow Analysis Simulation Window

- The first step is to put together a **Plan**. The **Plan** defines which geometry and flow data are to be used, as well as providing a title and short identifier for the run. To establish a plan, select **New Plan** from the **File** menu on the Steady Flow Analysis window. Enter the plan title as *First run* and then press the **OK** button (Figure 11.40).

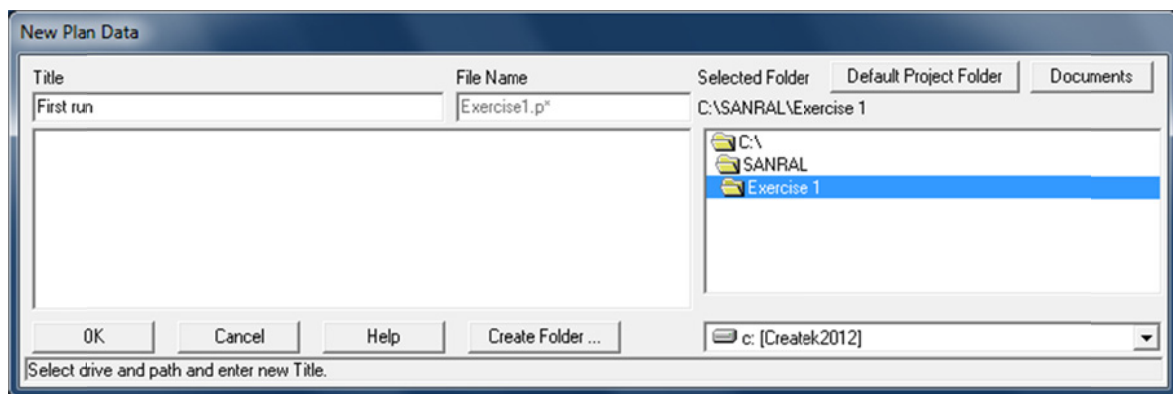


Figure 11.40: Creating new plan

- You will be prompted to enter a short identifier. Enter a title of *Run1* in the **Short ID** box (Figure 11.41) and click on the **OK** button.

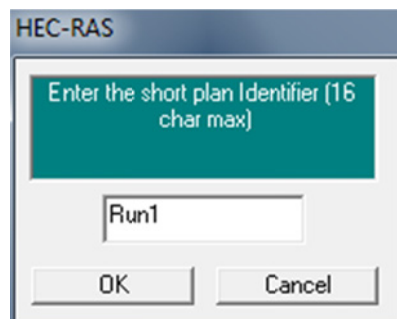
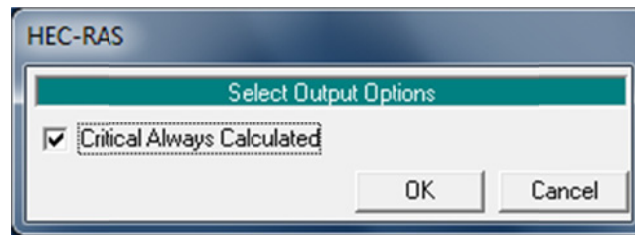


Figure 11.41: Plan identifier

- The next step is to select the desired flow regime for which the model will perform calculations. For this example we will be performing **Subcritical** flow calculations only since only a downstream boundary condition was specified. Make sure **Subcritical** is the selected flow regime.



- Additional job control features are available from the **Options** menu bar. Select **Critical Depth Output Option...** from this menu. A pop-up window will appear in which the user can select the **Critical Always Calculated** option (**Figure 11.42**). The program will then always calculate the critical flow depth for every flow rate. Click on the **OK** button.



**Figure 11.42: Critical depth calculation option**

- Once you have defined a plan and set all the desired job control information, the plan information should be saved. Saving the plan information is accomplished by selecting **Save Plan** from the **File** menu of the Steady Flow Analysis window.

Now that everything has been saved and set, the steady flow computations can be performed by pressing the **Compute** button at the bottom of the Steady Flow Simulation window (**Figure 11.39**). Once the computations have been completed, the computation window can be closed, as well as the Steady Flow Simulation window.

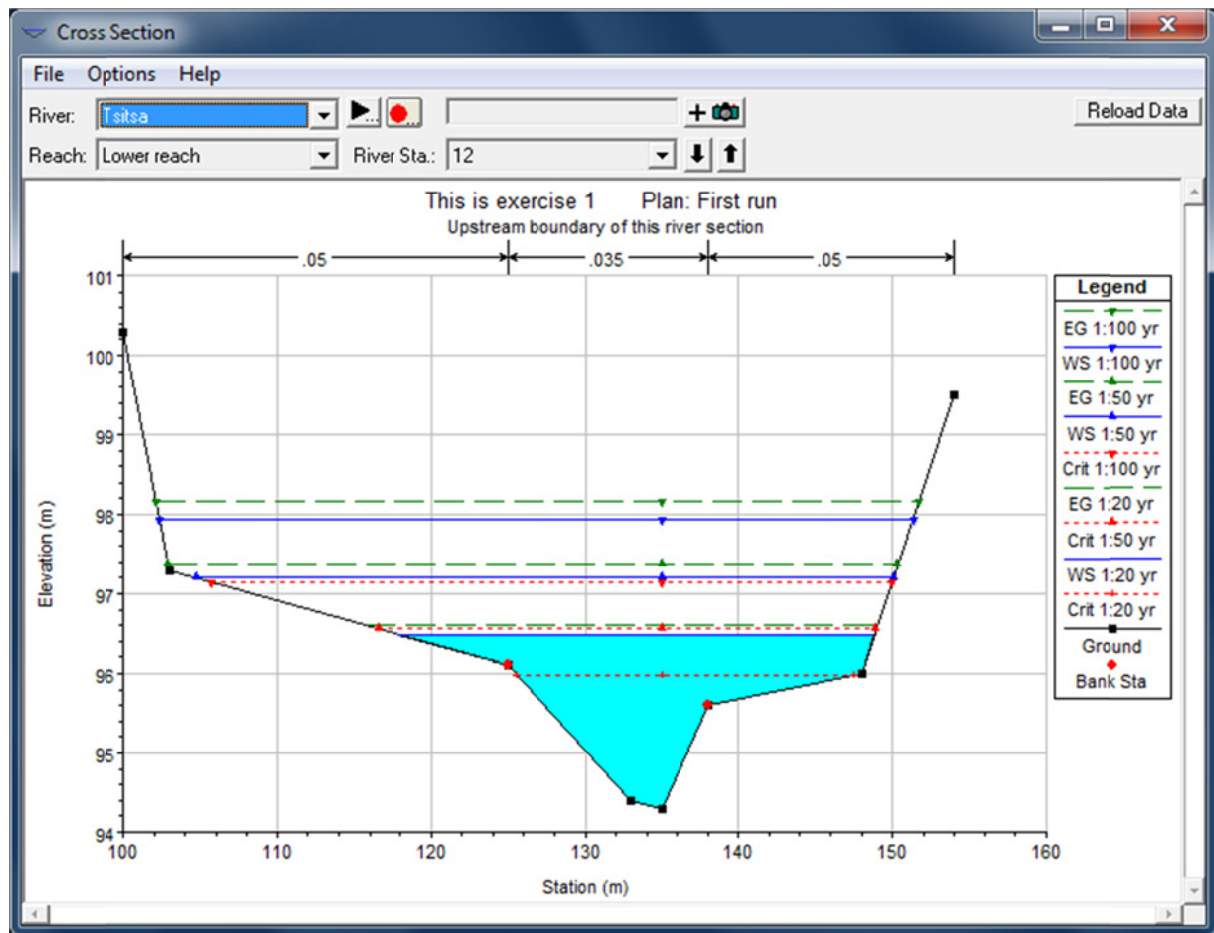
## **VIEWING RESULTS**

Once the model has finished all of the computations successfully, you can begin viewing the results. Several output options are available from the **View** menu bar on the HEC-RAS main window. These options include:

Cross section plots	
Profile plots	
General profile plot	
Rating curves	
X-Y-Z perspective plots	
Detailed tabular output at a specific cross section (cross section table)	
Limited tabular output at many cross sections (profile table)	

### **Cross section plots**

Begin by plotting a cross section. Select **Cross Sections** from the **View** menu on the HEC-RAS main window. Any cross section can be plotted by selecting the appropriate river, reach and river station (see **Figure 11.43**). Several plotting features are available from the **Options** menu bar on the cross section plot window. These options include: zoom in; zoom out; selecting which plans, profiles, variables to plot; and control over lines, labels, symbols, scaling etc.



**Figure 11.43: Cross section (all three profiles)**

Select different cross sections to plot and practice using some of the features available under the options menu bar.

### Profile plot

The second plot, which is of value, is the water surface profile. Select **Water Surface Profiles** from the **View** menu.

This should give you a profile plot as shown in **Figure 11.44**.

Try and obtain the profile plot to look exactly like **Figure 11.44** selecting various options under the **Options** menu (grid, labels, line types, text, profiles etc.).

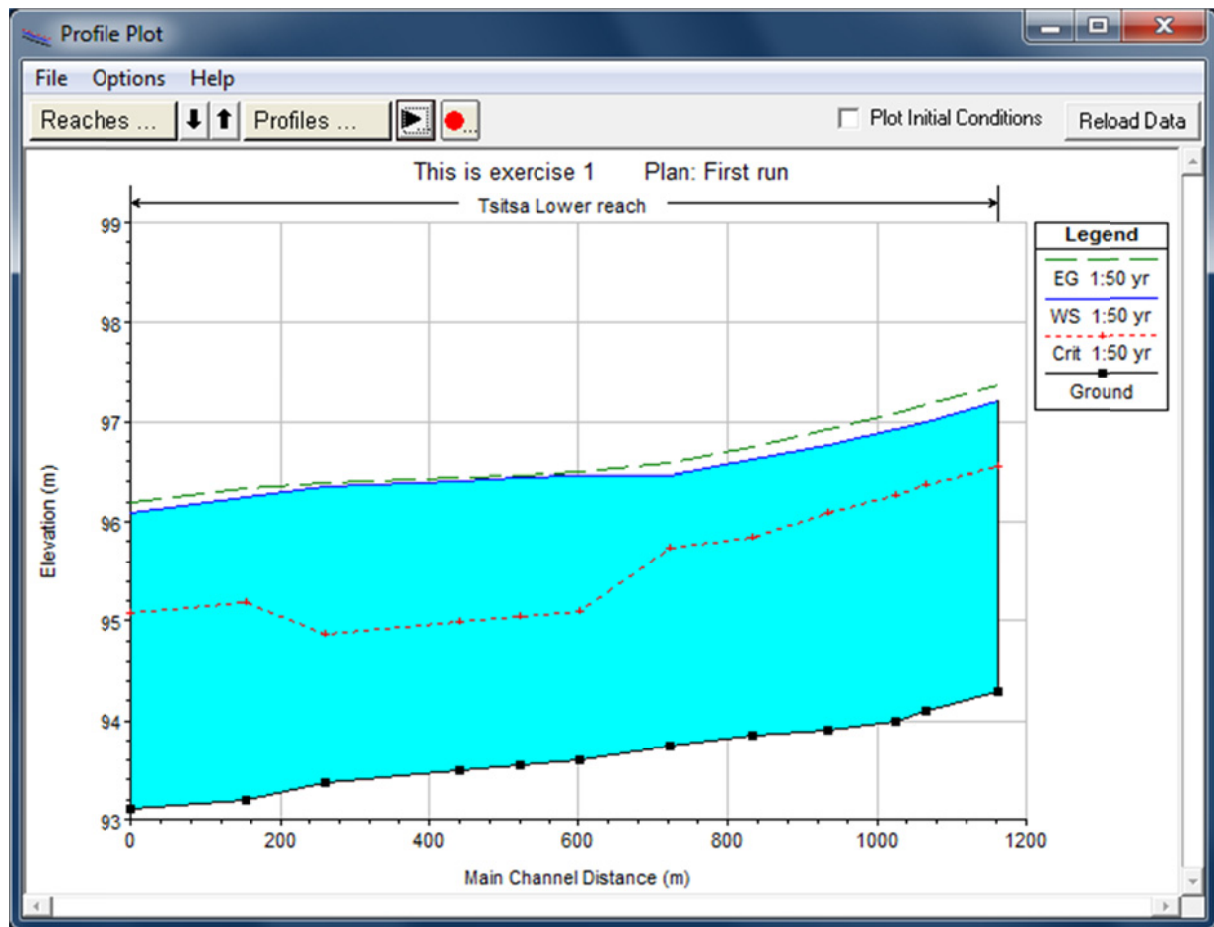


Figure 11.44: Water Surface Profile (for entire reach)

### General profile plots

The third plot option, which is of value, is the General Profile Plot. Select **General Profile Plot** from the **View** menu. This should give you a profile plot as shown in **Figure 11.45**.

From the **Standard Plots** menu various other useful plots can also be made such as Froude numbers (on left bank, main channel and right bank), see **Figure 11.46**.

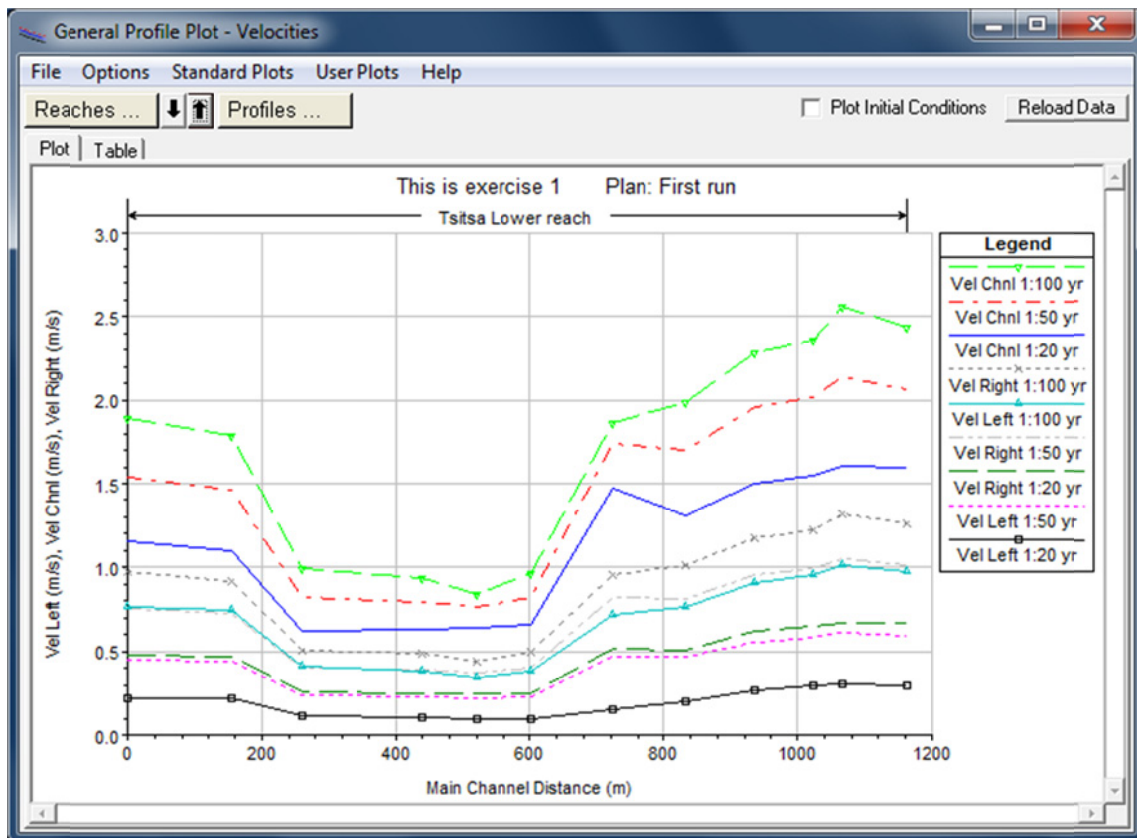


Figure 11.45: General profile plot (velocities)

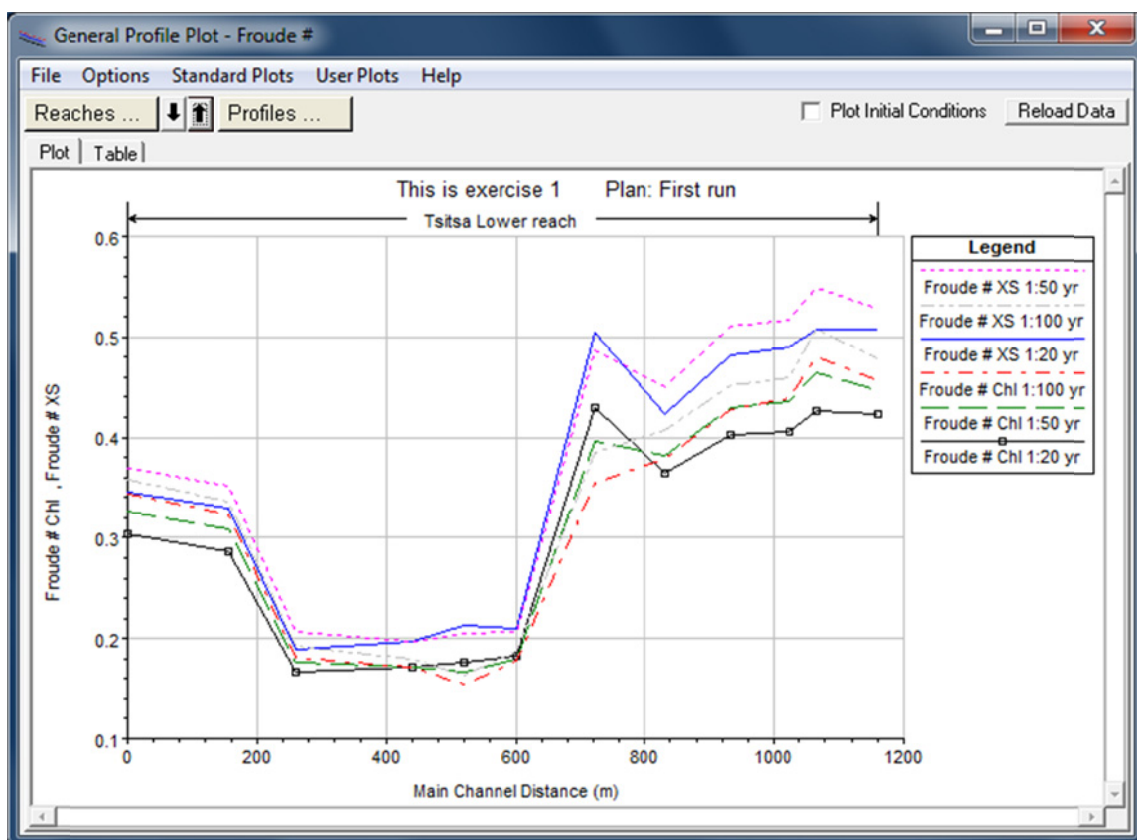
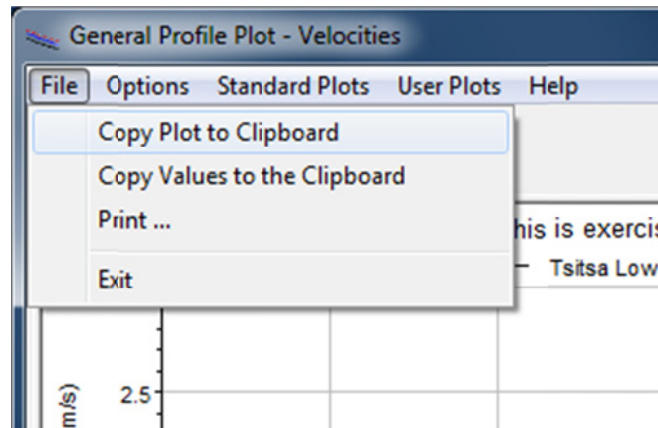
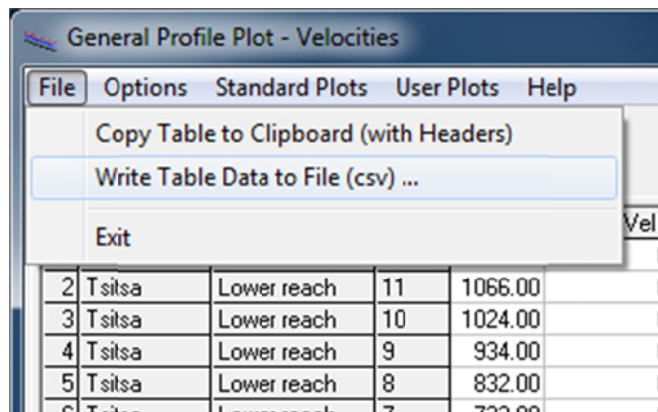


Figure 11.46: General profile plot (Froude numbers)

The data used to generate the plots can also be viewed in Table format by clicking on the **Table** tab next to the **Plot** tab. The generated plot or data can be copied to the clipboard by simply going to the File menu (for that specific tab), see **Figure 11.47** and **Figure 11.48**.



**Figure 11.47: General profile plot (Copying the plot to the clipboard)**

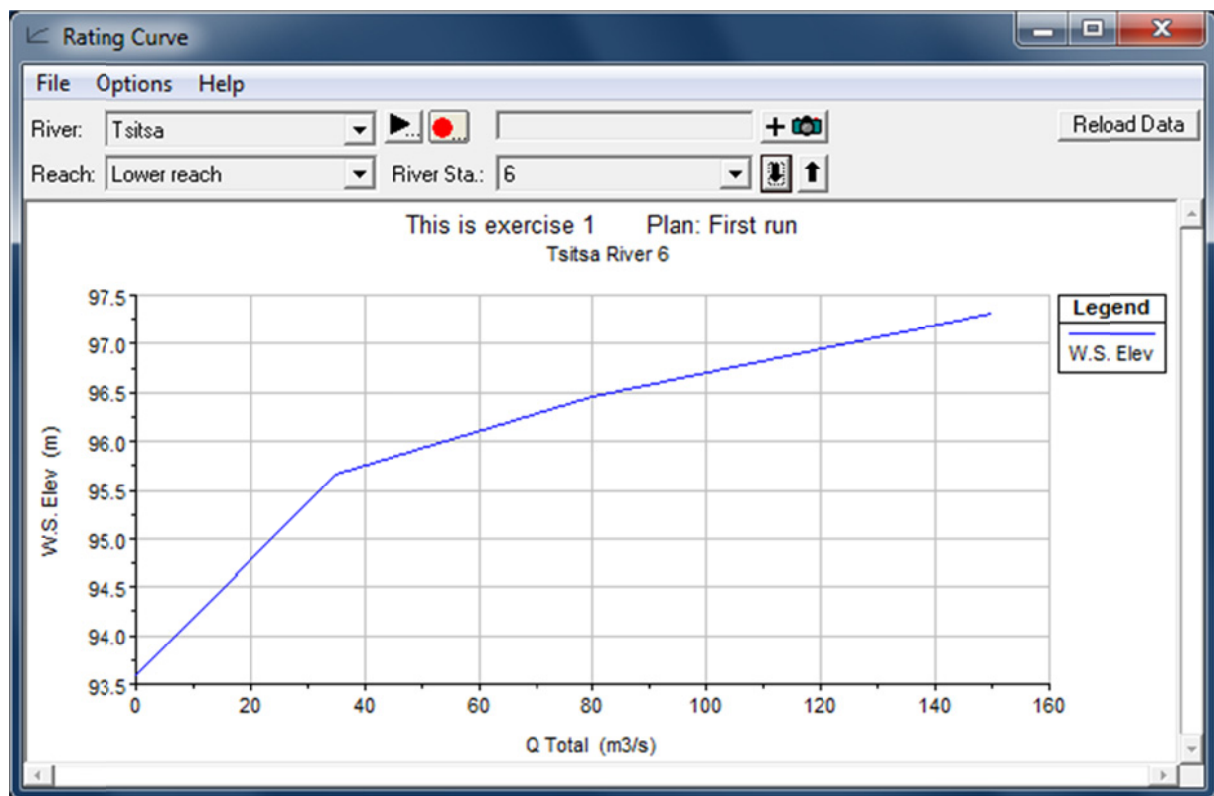


**Figure 11.48: General profile plot (Writing data to a file (csv type))**

This copying of pictures/graphs/data can be done throughout HEC-RAS.

### Rating curve

Select **Rating Curves** from the **View** menu plot a computed rating curve. A rating curve based on the computed water surface profiles will appear as shown in **Figure 11.49**. You can look at the computed rating curve for any location (cross section) by selecting the appropriate river, reach and river station.



**Figure 11.49: Rating Curve (cross section 5)**

HEC-RAS basically plots a curve through the three flows used in the calculation i.e. 35 m³/s, 80 m³/s and 150 m³/s. The more flows used in the calculation the more accurate rating curve will be obtained.

### **X-Y-Z Perspective Plot**

Select **X-Y-Z Perspective Plots** from the **View** menu to plot a 3D view of the river section (**Figure 11.50**). Set the **Rotation Angle** to  $-70$  and the **Asimuth Angle** to  $19$  to obtain the same view as shown in **Figure 11.50**. This type of view gives a clear view of the widening of the river at cross section 5 and 6.

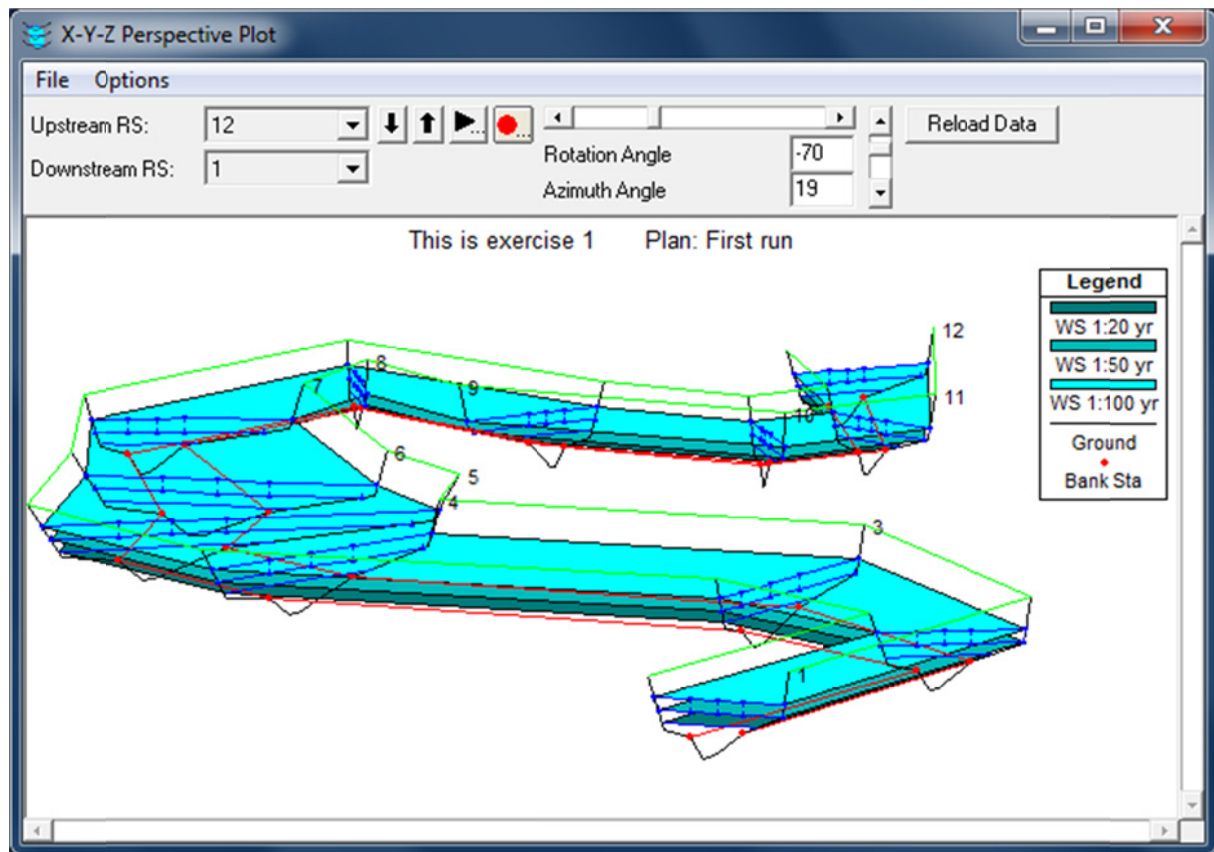


Figure 11.50: X-Y-Z Perspective Plot

## Tabular output

### Detailed tabular output

Now look at some tabular output. Go to the **View** menu on the HEC-RAS main window. There are two types of tables available, a detailed output table and a profile summary table. Select **Detailed Output Tables** to get the first table to appear. The table should be similar to the one shown in **Figure 11.51** (notice the warning at the bottom of the table for river station number 3). This table shows detailed hydraulic information at a single cross section. Other cross sections can be viewed by selecting the appropriate reach and river from the table.



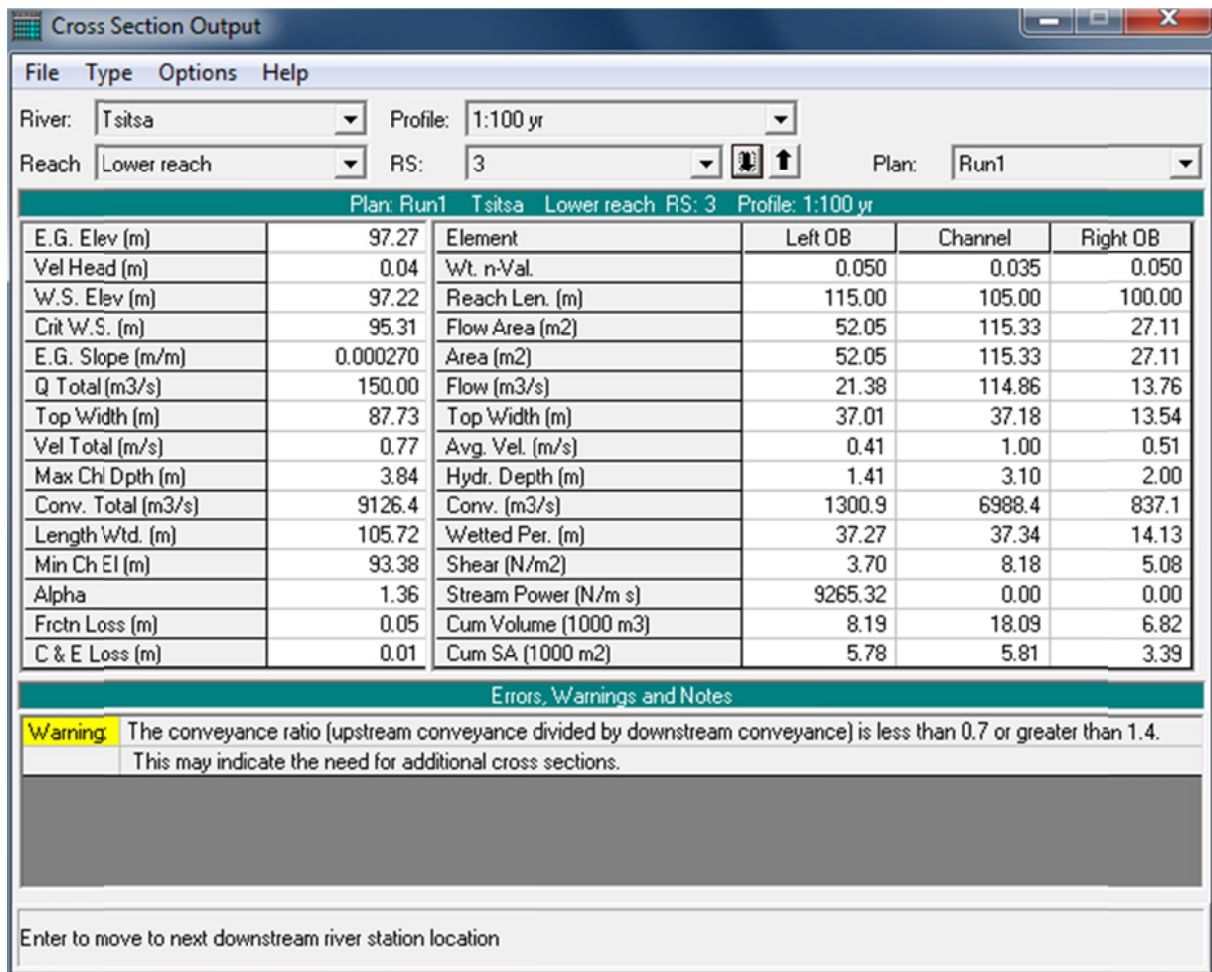



Figure 11.51: Detailed table output

### Summary Errors, Warnings, and Notes

The HEC-RAS software has a system of Errors, Warnings, and Notes that are passed from the computation programs to the user interface. During the computations, the computation programs will set flags for at a particular node (nodes are cross sections, bridges, culverts, or multiple openings) whenever it is necessary. These message flags are written to the standard output file, along with the computed results for that node. When the user interface reads the computed results from the output file, if any errors, warnings, or notes exist, they are interpreted and displayed in various locations from the interface.

The user can request a summary of all the errors, warnings, and notes that occurred during the computations. This is accomplished by selecting **Summary Errors, Warnings, and Notes** from the **View** menu on the main HEC-RAS window or clicking the short cut button .

Once this is selected, a window will pop up displaying all of the messages. The user can select a specific River and Reach, as well as which Profile and Plan to view. The user has the options of expanding the window; printing the messages; or sending them to the windows clipboard.

Besides the summary window, messages will automatically appear on the cross section specific tables. When a cross section or hydraulic structure is being displayed, any errors, warnings, or notes for that location and profile will show up in the Errors, Warnings, and Notes message box at the bottom of the table. An example of this table is shown in **Figure 11.51**.

In general, the errors, warnings, and notes messages should be self-explanatory. The three categories of messages are the following:

**ERRORS:** Error messages are only sent when there are problems that prevent the program from being able to complete the run.

**WARNINGS:** Warning messages provide information to the user that may or may not require action on the user's part. In general, whenever a warning is set at a location, the user should review the hydraulic results at that location to ensure that the results are reasonable. If the hydraulic results are found to be reasonable, then the message can be ignored. However, in many instances, a warning level message may require the user to take some action that will cause the message to disappear on future runs. Many of the warning messages are caused by either inadequate or bad data. Some common problems that cause warning messages to occur are the following:

**Cross sections spaced to far apart.** This can cause several warning messages to be set.

**Cross sections starting and ending stations not high enough.** If a computed water surface is higher than either end point of the cross section, a warning message will appear.

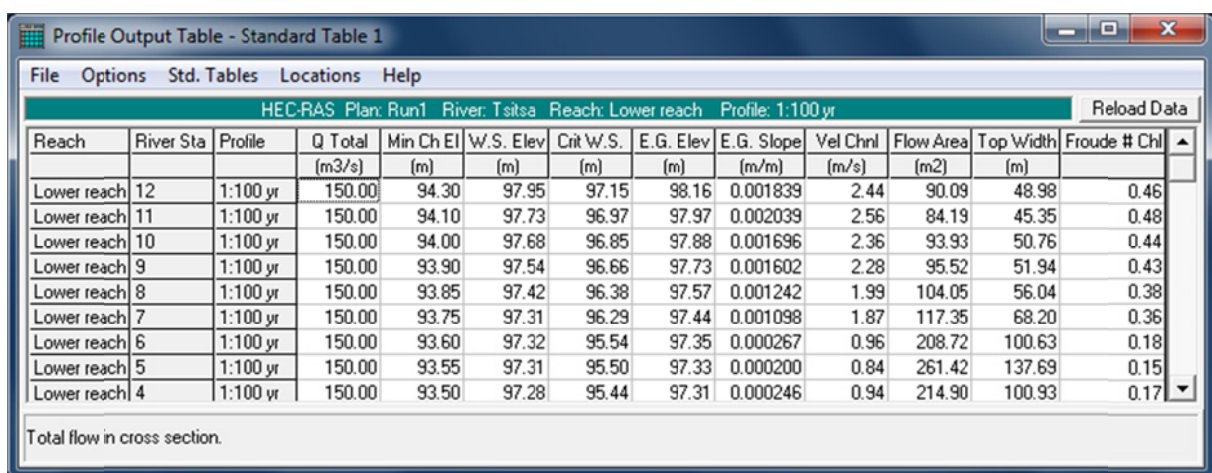
**Bad Starting Water Surface Elevation.** If the user specifies a boundary condition that is not possible for the specified flow regime, the program will take action and set an appropriate warning message.

**Bad Cross Section Data.** This can cause several problems, but most often the program will not be able to balance the energy equation and will default to critical depth.

**NOTES:** Note level messages are set to provide information to the user about how the program is performing the computations.

### Profile Summary Table

Go to the **View** menu on the HEC-RAS main window. There are two types of tables available, a detailed output table and a profile summary table. Select **Profile Summary Table**. This table shows a limited number of hydraulic variables for several cross sections in the selected river reach (see **Figure 11.52**).



The screenshot shows a window titled "Profile Output Table - Standard Table 1". The table displays hydraulic data for the "Lower reach" of the "Tsitsa" River, using a "1:100 yr" profile. The data is organized into columns for Reach, River Sta, Profile, Q Total, Min Ch El, W.S. Elev, Crit W.S., E.G. Elev, E.G. Slope, Vel Chnl, Flow Area, Top Width, and Froude # Chl. The table lists data for reaches 4 through 12, showing a decreasing trend in flow area and top width as the reach number increases.

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Lower reach 12		1:100 yr	150.00	94.30	97.95	97.15	98.16	0.001839	2.44	90.09	48.98	0.46
Lower reach 11		1:100 yr	150.00	94.10	97.73	96.97	97.97	0.002039	2.56	84.19	45.35	0.48
Lower reach 10		1:100 yr	150.00	94.00	97.68	96.85	97.88	0.001696	2.36	93.93	50.76	0.44
Lower reach 9		1:100 yr	150.00	93.90	97.54	96.66	97.73	0.001602	2.28	95.52	51.94	0.43
Lower reach 8		1:100 yr	150.00	93.85	97.42	96.38	97.57	0.001242	1.99	104.05	56.04	0.38
Lower reach 7		1:100 yr	150.00	93.75	97.31	96.29	97.44	0.001098	1.87	117.35	68.20	0.36
Lower reach 6		1:100 yr	150.00	93.60	97.32	95.54	97.35	0.000267	0.96	208.72	100.63	0.18
Lower reach 5		1:100 yr	150.00	93.55	97.31	95.50	97.33	0.000200	0.84	261.42	137.69	0.15
Lower reach 4		1:100 yr	150.00	93.50	97.28	95.44	97.31	0.000246	0.94	214.90	100.93	0.17

Total flow in cross section.

**Figure 11.52: Profile Summary Table (Standard Table)**

There are several types of profile tables listed under the **Std. Tables** menu (see **Figure 11.53**) of the profile table window. Each one of these tables shows typical detail relevant to the specific structure.

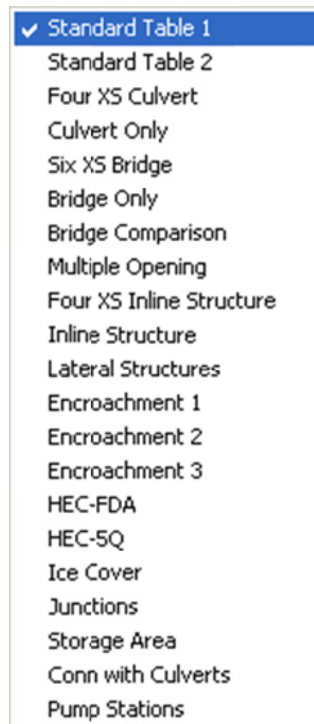


Figure 11.53: Std. Tables menu

A special feature of the profile summary tables is the ability for users to define their own output tables. User defined output tables are available by selecting **Define Table...** from the **Options** menu of the profile table. When this option is selected, a window will appear, as shown in **Figure 11.54**. At the top of the window is a table for the user selected variable headings (Table Column Headings), the units, and the number of decimal places to be displayed for each variable. Below this table is a list containing all of the available variables that can be included in your user-defined table. The variables are listed in alphabetical order. Below the list of variables is a message box that is used to display the definition of the selected variable.

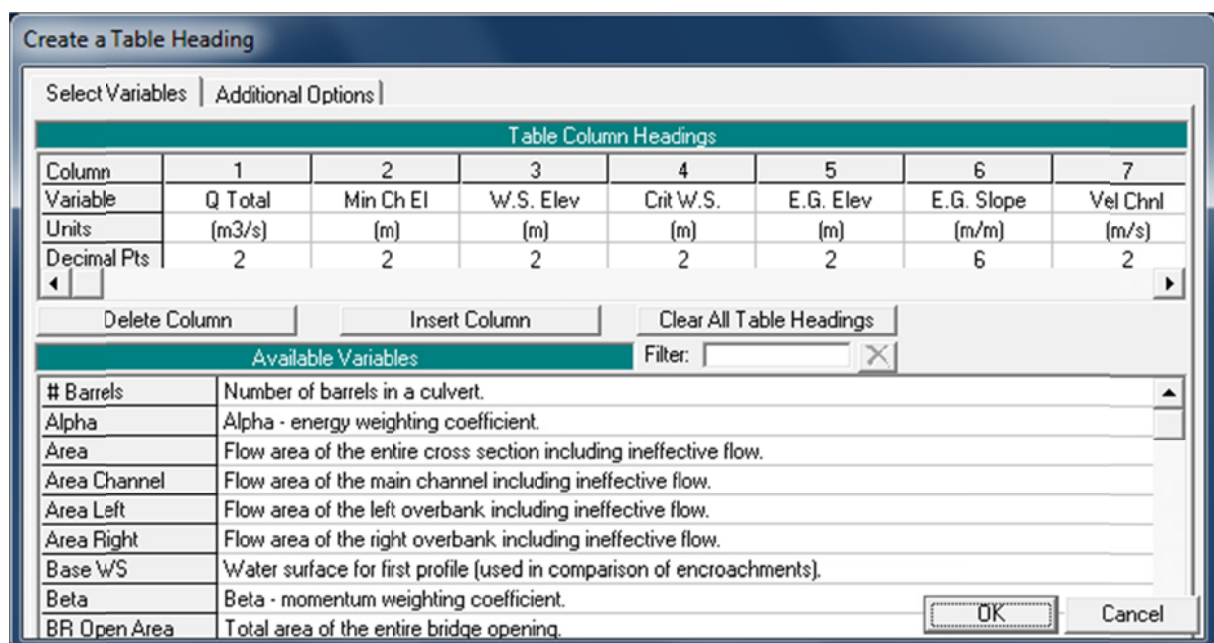


Figure 11.54: Define a table

To get a definition of a particular variable, simply click the left mouse button once while the mouse pointer is over the desired variable. The description of the variable will show up at the bottom of the window. To add variables to the column headings, simply double click the left mouse button while the mouse pointer is over the desired variable. The variable will be placed in the active field of the table column headings. To select a specific column to place a variable in, click the left mouse button once while the mouse pointer is over the desired table column field. To delete a variable from the table headings, double click the left mouse button while the mouse pointer is over the variable that you want to delete. The number of decimal places for each variable can be changed by simply typing in a new value.

User defined tables are limited to 15 variables. Once you have selected all of the variables that you want, press the **OK** button at the bottom of the window. The profile table will automatically be updated to display the new table

Once you have the table displayed in the profile table window, you can save the table headings for future use. To save a table heading, select **Save Table** from the **Options** menu on the profile table window. When this option is selected, a pop up window will appear, prompting you to enter a name for the table. Once you enter the name, press the **OK** button at the bottom of the pop up window. The table name will then be added to a list of tables included under the **User Tables** menu on the profile table window.

Create a user defined table as detailed above with the following columns:

- Q Total (Total flow rate)
- W.S. Elev (Water surface elevation)
- Crit W.S. (Critical water surface elevation)
- Vel Chnl (Velocity in the main channel)
- Froude # Chl (Froude number in the main channel)

Save the table (*Exercise 1 – Table*) and view the summary table thereof with 1:50 and 1:100 year profiles (selected from the **Options** menu) (**Figure 11.55**).

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Run1 River: Tsitsa Reach: Lower reach Reload Data

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Tc
Lower reach	12	1:50 yr	80.00	94.30	97.20	96.56	97.37	0.001952	2.06	54.48	
Lower reach	12	1:100 yr	150.00	94.30	97.95	97.15	98.16	0.001839	2.44	90.09	
Lower reach	11	1:50 yr	80.00	94.10	96.99	96.37	97.18	0.002106	2.14	51.44	
Lower reach	11	1:100 yr	150.00	94.10	97.73	96.97	97.97	0.002039	2.56	84.19	
Lower reach	10	1:50 yr	80.00	94.00	96.93	96.26	97.09	0.001839	2.02	56.41	
Lower reach	10	1:100 yr	150.00	94.00	97.68	96.85	97.88	0.001696	2.36	93.93	
Lower reach	9	1:50 yr	80.00	93.90	96.77	96.09	96.92	0.001792	1.96	56.39	
Lower reach	9	1:100 yr	150.00	93.90	97.54	96.66	97.73	0.001602	2.28	95.52	
Lower reach	8	1:50 yr	80.00	93.85	96.63	95.84	96.75	0.001410	1.70	60.93	
Lower reach	8	1:100 yr	150.00	93.85	97.42	96.38	97.57	0.001242	1.99	104.05	
Lower reach	7	1:50 yr	80.00	93.75	96.46	95.74	96.59	0.001539	1.74	61.75	
Lower reach	7	1:100 yr	150.00	93.75	97.31	96.29	97.44	0.001098	1.87	117.35	
Lower reach	6	1:50 yr	80.00	93.60	96.46	95.09	96.49	0.000305	0.82	124.58	
Lower reach	6	1:100 yr	150.00	93.60	97.32	95.54	97.35	0.000267	0.96	208.72	
Lower reach	5	1:50 yr	80.00	93.55	96.44	95.04	96.46	0.000261	0.77	145.02	
Lower reach	5	1:100 yr	150.00	93.55	97.31	95.50	97.33	0.000200	0.84	261.42	
Lower reach	4	1:50 yr	80.00	93.50	96.42	94.99	96.44	0.000275	0.79	129.78	
Lower reach	4	1:100 yr	150.00	93.50	97.28	95.44	97.31	0.000246	0.94	214.90	
Lower reach	3	1:50 yr	80.00	93.38	96.36	94.86	96.39	0.000286	0.82	119.98	
Lower reach	3	1:100 yr	150.00	93.38	97.22	95.31	97.27	0.000270	1.00	194.50	
Lower reach	2	1:50 yr	80.00	93.20	96.25	95.18	96.34	0.000884	1.46	71.18	
Lower reach	2	1:100 yr	150.00	93.20	97.08	95.73	97.21	0.000873	1.79	112.85	
Lower reach	1	1:50 yr	80.00	93.10	96.09	95.08	96.19	0.001002	1.53	66.34	
Lower reach	1	1:100 yr	150.00	93.10	96.92	95.62	97.06	0.001000	1.89	104.51	

Figure 11.55: User defined table

At the end of this exercise the following **objectives** should have been met:

- Be able to set-up a HEC-RAS project
- Know how to enter geometric data
- Understand the setting of boundaries and controls
- Know how to enter steady flow data
- Know how to analyse a river system
- Know how to extract information

### Questions

1. What is the normal flow depth for the 1:50 year flood at cross section 1?
2. Define the flow type in the river.
3. What is the kinetic energy component at cross section 11 for the 1:50 year flood?
4. What is the kinetic energy component at cross section 5 (the proposed site) for the 1:50 year flood?
5. Why is there a difference between the kinetic energy component at cross section 5 and 11?
6. What are the flood levels (1:20, 1:50 and 1:100) at the proposed site (cross section 5)?
7. What will the water level be at cross section 5 if the flood peak is  $100 \text{ m}^3/\text{s}$ ?
8. What is the energy weighting coefficient ( $\alpha$ ) for cross section 5 for the 1:50 year flood?
9. How wide is the river flowing at cross section 5 during the 1:100 year flood?
10. What is the hydraulic depth in the main channel during the 1:100 year flood at cross section 5?
11. How would you know if cross section 5 or 6 was functioning as a control point in the river section?
12. Are you in a position to draw in the 1:20, 1:50 and 1:100 year flood lines for the proposed site?

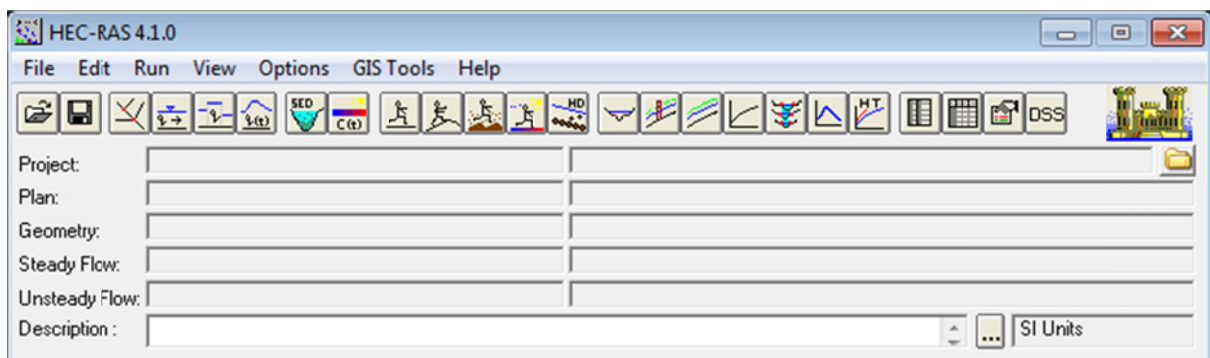


## 11.2 Setting-up a HEC-RAS model (river section, bridge and weir) and performing unsteady flow analysis



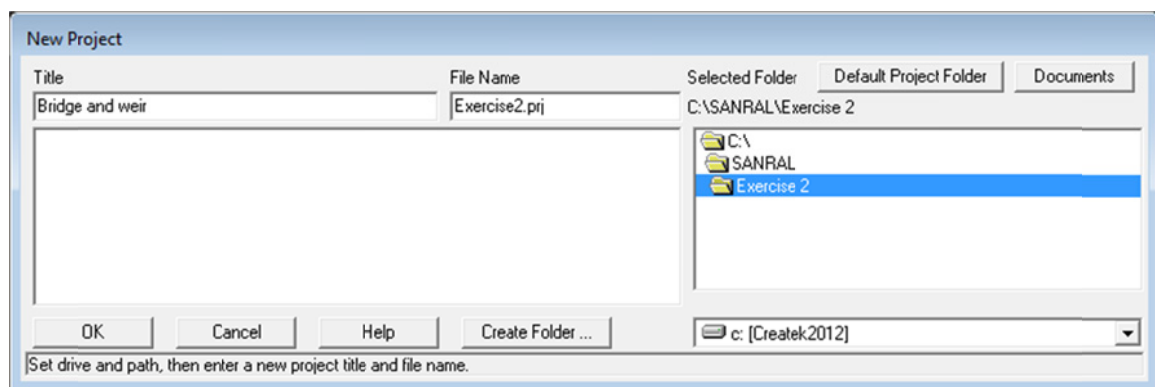
### STARTING A NEW PROJECT

To begin this exercise, start the HEC-RAS program by double clicking the HEC-RAS icon on the desktop. The main window should appear as shown in **Figure 11.56**.



**Figure 11.56: HEC-RAS main window**

The first step in developing a HEC-RAS application is to start a new project. Go to **File** menu on the main window and select **New Project**. The New Project window should appear as shown in **Figure 11.57**. Set the drive and directory you would like to work in. Enter the *project title* and *file name* as typically shown in **Figure 11.57**. Once you have entered the information, press the **OK** button to accepted the title and file name and create the new project.




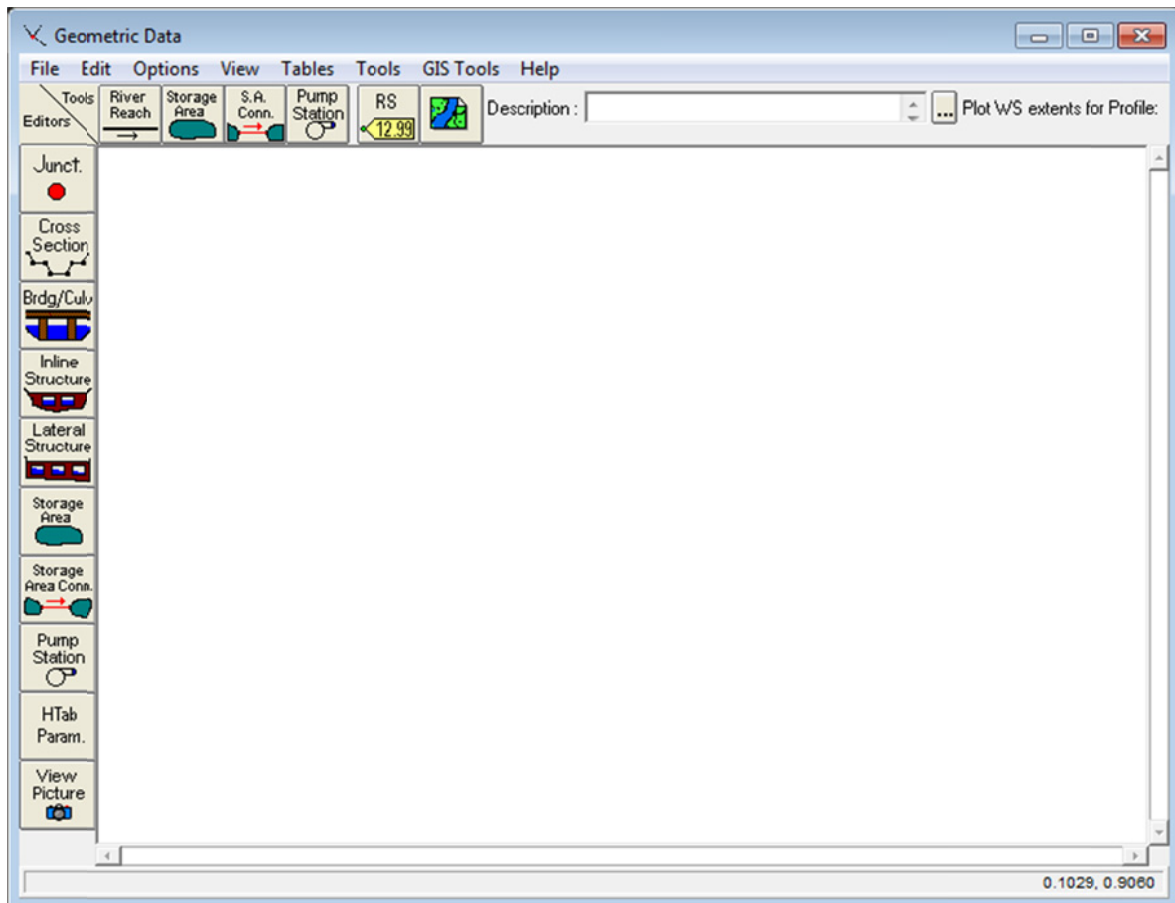
**Figure 11.57: New Project window**

Once back at the HEC-RAS Main window select from the menu bar **Options**, and set the units that you would like to work in to be metric units as well as be the default setting for all new projects (assignments). In the right hand corner of the main screen it will now indicate SI units.



## ENTERING GEOMETRIC DATA

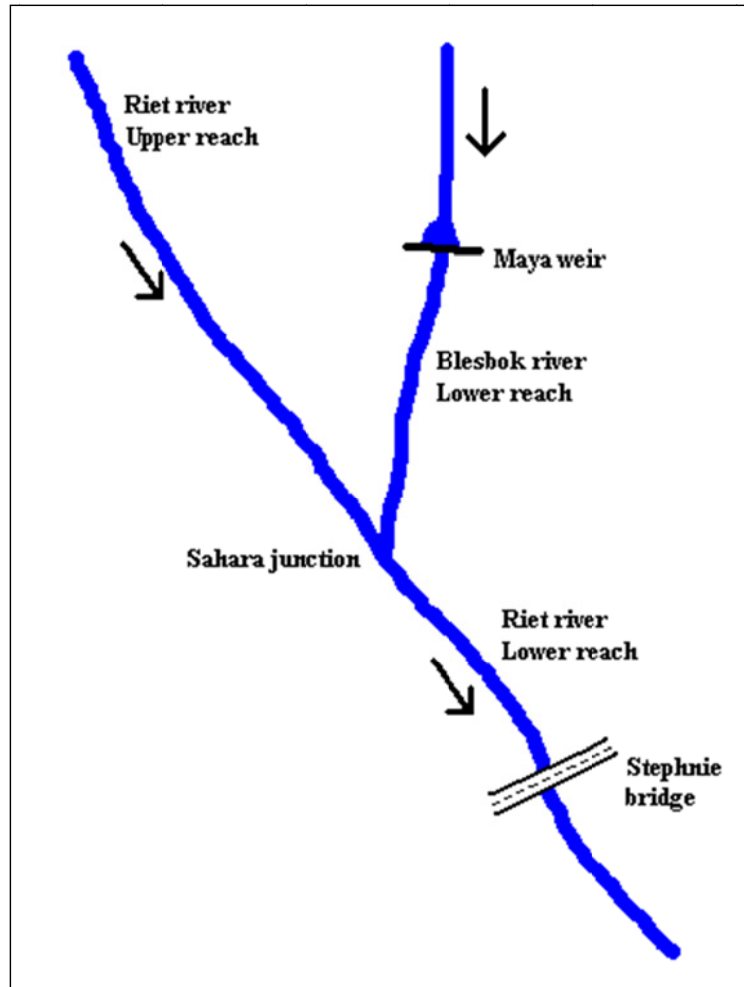
The next step is to enter the Geometric Data. This is accomplished by selecting **Geometric Data** from the **Edit** menu on the HEC-RAS Main window (**Figure 11.56**) or clicking the short cut button on the menu bar . Once this option is selected, the geometric data window will be shown (see **Figure 11.58**).



**Figure 11.58: Geometric Data window**

### **Drawing the schematic of the river system**

A plan view of the river section with cross sections is shown below in **Figure 11.59**.



**Figure 11.59: Plan view of river section**

The first step is to draw the river system schematically by performing the following steps:

- Click the **River Reach** button on the geometric data window.
- Move the mouse pointer over the drawing area and place the pointer at the location in which you would like to start drawing the reach.
- Press the left mouse button once to start drawing the reach. Move the mouse pointer and continue to press the left mouse button to add additional points to the line segment. To end the drawing of the reach, double click the left mouse button and the last point on the reach will be placed at the current mouse pointer location (right click will remove the last point drawn).

All reaches must be drawn from the upstream to downstream (in the positive flow direction) i.e. start at cross section 100 down to cross section 70, 65 down to cross section 40 and from cross section 8 to 3 (see **Figure 11.60**).

- Once a reach is drawn, the interface will prompt you to enter an identifier for the **River** name and the **Reach** name. The **River** identifier can be up to 32 characters, while the reach name is limited to 12 characters. In this exercise the rivers will be called, *Riet* and *Blesbok* and the reaches *Upper* and *Lower reach* (see **Figure 11.60**).

- Once you enter the identifiers for *Blesbok River* or for the *lower reach* of the *Riet River*, you will be prompted to enter an identifier for the junction, in this case *Sahara*. Junctions in HEC-RAS are locations where two or more reaches join together or split apart.
- When you first draw the schematic there will be no tic marks representing the cross sections. The tic marks only show up after you have entered cross sections.

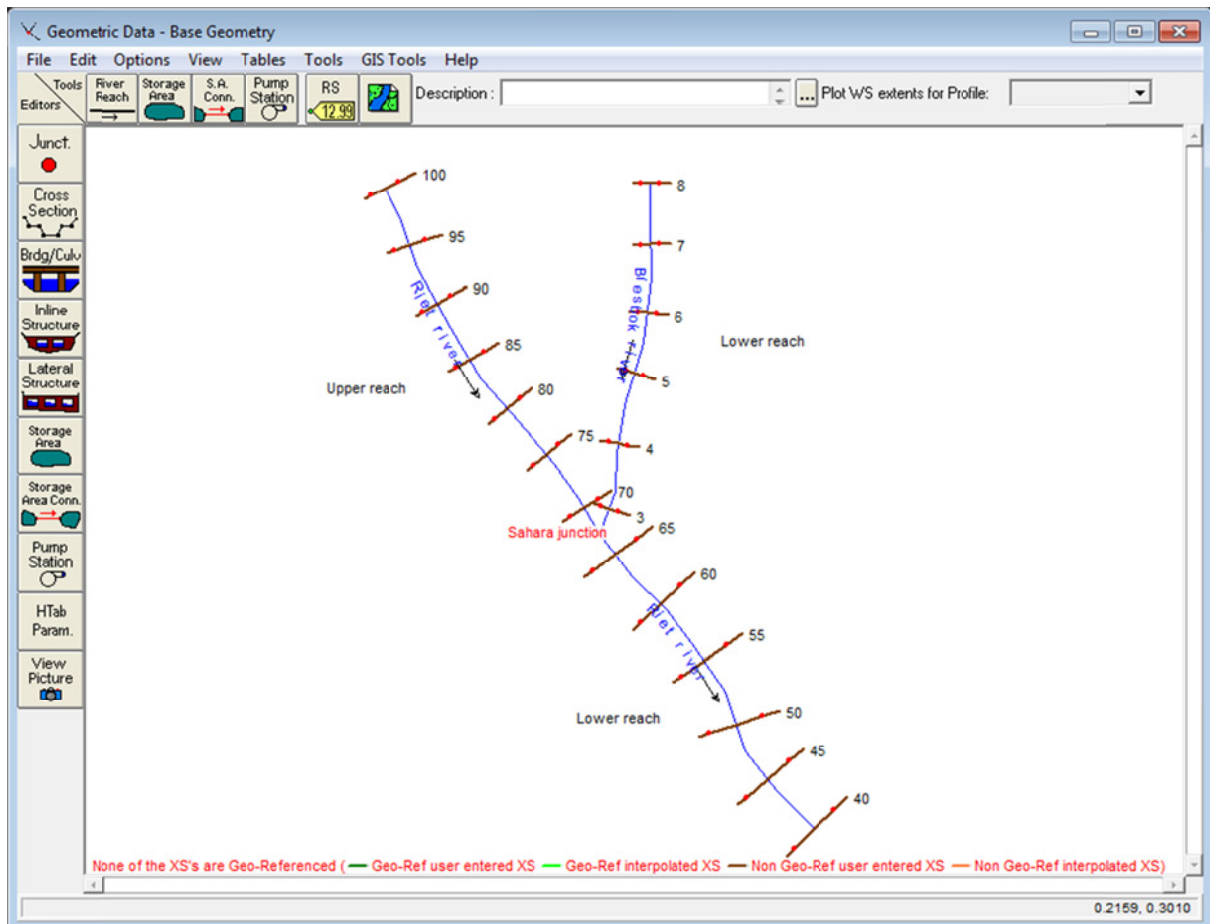


Figure 11.60: Geometric Data window (schematic)

### Entering cross section data



The next step is to enter the cross section data. This is accomplished by clicking the **cross section** button on the Geometric window (**Figure 11.60**). Once this button is clicked, the Cross Section Data editor will appear as shown in **Figure 11.61**.

Figure 11.61: Cross section Data Editor

To enter cross section data follow these steps:

- Select a **River** and a **Reach** to work with (from the drop down lists).
- Go to the **Options** menu and select **Add a new Cross Section**. An input box will appear to prompt you to enter a river station identifier for the new cross section (see Figure 11.62).

Figure 11.62: Add a new river station

The identifier does not have to be the actual river station, but it must be a numerical value. The numeric value describes where the cross section is located in reference to all other cross sections within the reach. Cross sections are located from upstream (highest river station) to downstream (lowest river station). For this cross section enter a value of 100.

- For this cross section, enter all the data as shown in Figure 11.63.

**Cross Section Data - Base Geometry**

Exit Edit Options Plot Help

River: Riet river Apply Data

Reach: Upper reach River Sta.: 100

Description: Upstream River Station

Del Row Ins Row

Cross Section Coordinates	
1	0
2	3.05
3	4.57
4	8.53
5	9.75
6	10.67
7	12.19
8	15.24
9	18.29
10	19.81
11	21.34

Downstream Reach Lengths		
LOB	Channel	ROB
25	28	30.5

Manning's n Values		
LOB	Channel	ROB
0.035	0.025	0.035

Main Channel Bank Stations	
Left Bank	Right Bank
8.53	22.56

Cont\Exp Coefficient (Steady Flow)	
Contraction	Expansion
0.1	0.3

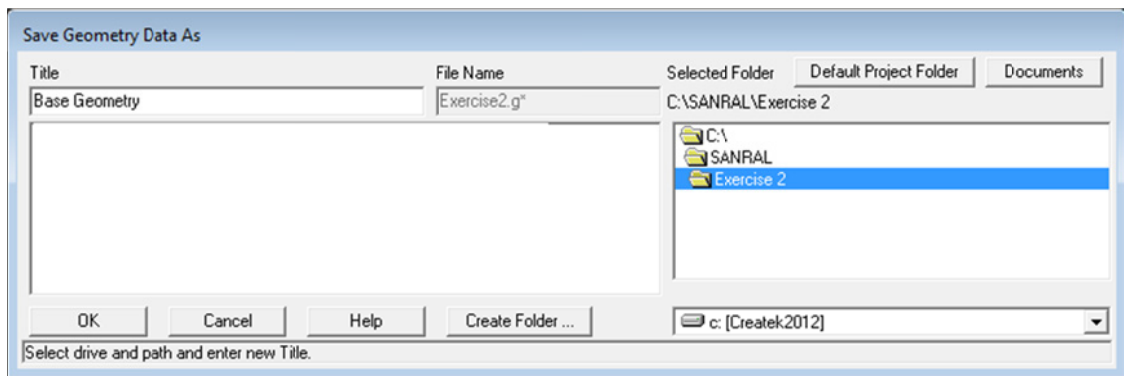
**Figure 11.63: Cross Section Data Editor with data**

- Enter the:  
 Description: *Upstream River Station*  
 Downstream reach lengths: LOB = 25, Channel = 28 and ROB = 30,5  
 Manning n-values: LOB = 0,035, Channel = 0,025 and ROB = 0,035  
 Station and elevation details:

Nr	Station	Elevation
1	0,00	26,82
2	3,05	25,60
3	4,57	24,69
4	8,53	23,77
5	9,75	22,25
6	10,67	21,03
7	12,19	20,42
8	15,24	19,51
9	18,29	19,45
10	19,81	20,73
11	21,34	22,25
12	22,56	24,23
13	24,08	24,38
14	25,30	25,91
15	26,21	28,04

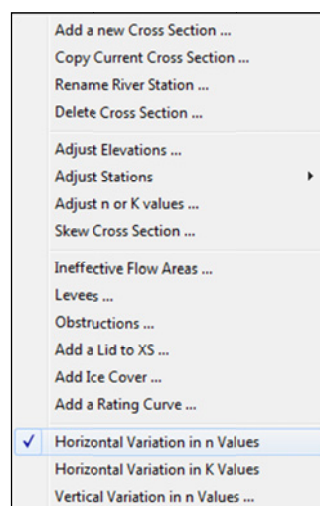
Main channel stations: Left bank = 8,53 and Right bank = 22,56  
 Cont\Exp coefficients: Contraction = 0,1 and Expansion = 0,3

- Once all the data is entered press the **Apply Data** button. This button is used to instruct the program to accept the entered data into memory. This button does not save the data to the hard disk (click on **Exit** on the **Cross Section Data** editor window). This is done by clicking on **Save Geometry Data** under the **File** menu on the **Geometric Data** window. After selecting this option you will be prompted to enter a Title for the geometric data (**Figure 11.64**). Enter “*Base Geometry*” for this exercise, and then press the **OK** button. A file name is automatically assigned to the geometry data based on what you entered for the project file name i.e. Exercise2.g01.



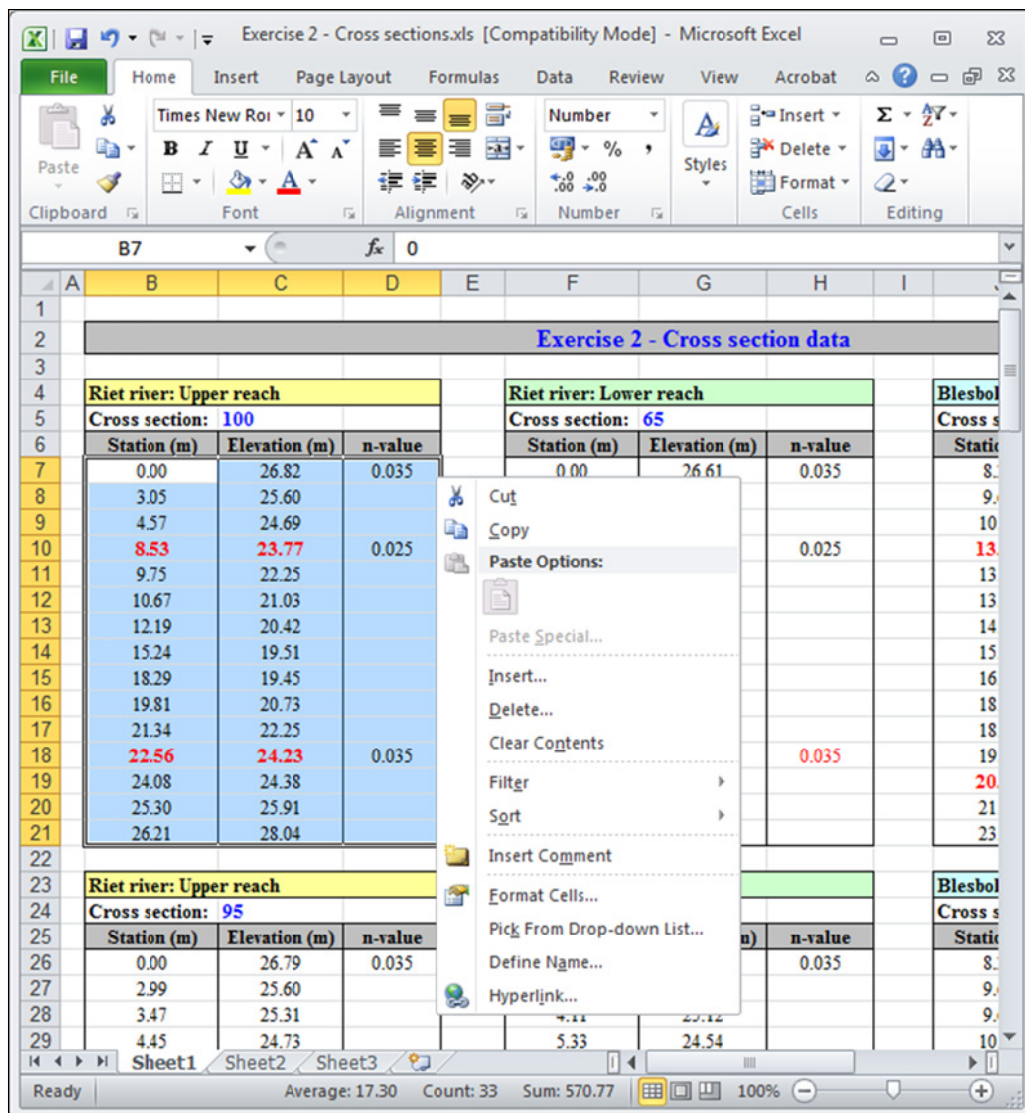
**Figure 11.64: Save Geometry Data As**

- Instead of retyping all the cross section data, an Excel spreadsheet containing all the stations and elevation data is provided. To quickly enter all the cross sectional data follow the following 6 steps
  - Step 1: Select a **River** and a **Reach** to work with (from the drop down lists).
  - Step 2: Go to the **Options** menu and select **Add a new Cross Section**. An input box will appear to prompt you to enter a river station identifier for the new cross section (see **Figure 11.62**).
  - Step 3: Set the **Horizontal Variation in n Values** under the **Options** menu to true (ticked) see **Figure 11.65**.



**Figure 11.65: Horizontal Variation in n Values (Options menu)**

- Step 4: Go to the Excel spreadsheet (Exercise 2 - Cross sections.xls) and copy the Station, Elevation and n-value data for the specific cross section/river station (see **Figure 11.66**).



**Figure 11.66: Copying cross section data from Excel**

- Step 5: Return to HEC-RAS, and select same number (or more) number of rows in the **Cross Section Data** editor window (see **Figure 11.67**). From the **Edit** menu click on **Paste**, to insert the data in the table.
- Step 6: Enter the rest of the other characteristic data:  
 Description: ""  
 Downstream reach lengths: LOB = 22,9, Channel = 29,0 and ROB = 33,5  
 Main channel stations: Left bank = 8,32 and Right bank = 22,56  
 Cont/Exp coefficients: Contraction = 0,1 and Expansion = 0,3  
 All the other characteristic data is provided in **Table 11.2**.

The spreadsheet printout of all the cross section data is attached in **Appendix A** for reference.



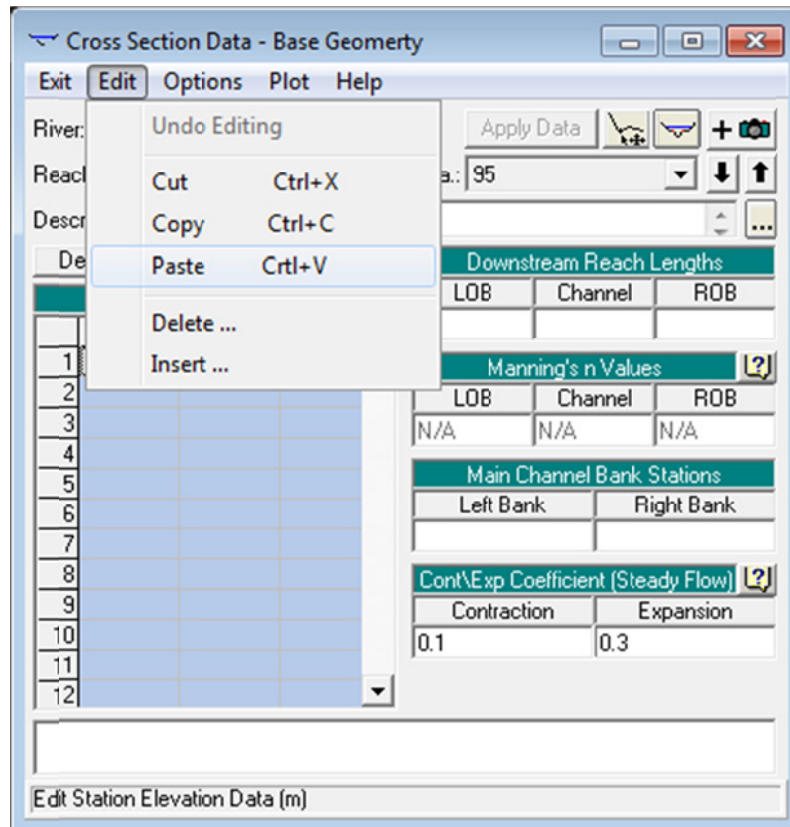


Figure 11.67: Pasting the cross section data from Excel into the Cross Section Data Editor table

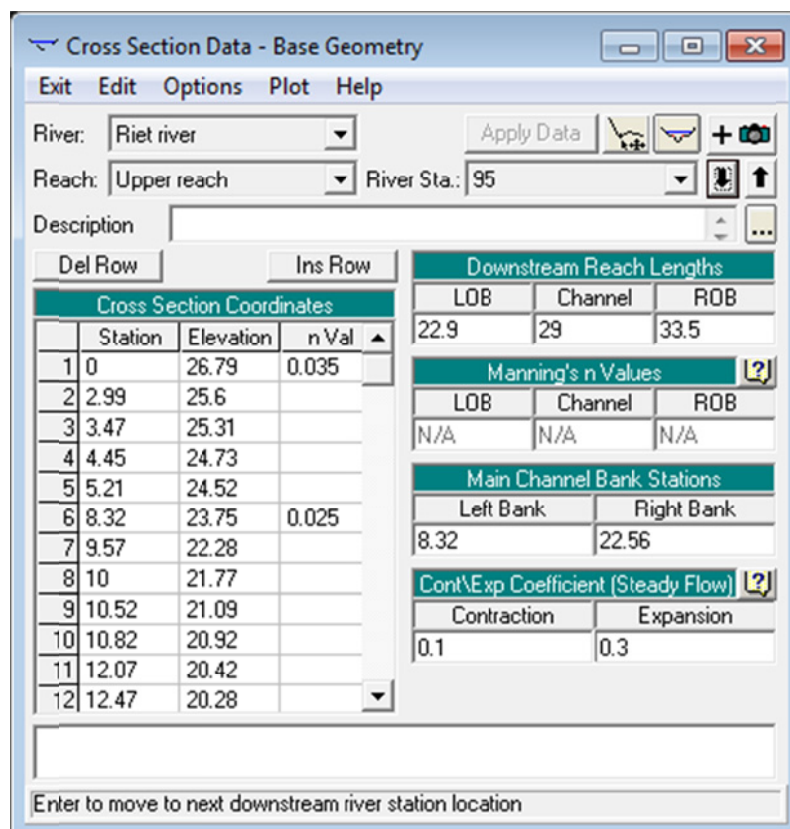


Figure 11.68: Completed Cross Sectional Data (Cross section 95)

Table 11.2: Cross section data

River	Reach	River station	Main channel bank stations		Downstream reach length		
			LOB	ROB	LOB	Channel	ROB
Riet	Upper	100	8,53	22,56	25,0	28,0	30,5
		95	8,32	22,56	22,9	29,0	33,5
		90	8,14	22,56	25,9	29,0	30,5
		85	7,92	22,56	23,8	28,0	31,4
		80	7,71	22,56	25,6	27,4	30,5
		75	7,53	22,56	24,4	29,0	33,5
		70	7,32	22,56	0,0	0,0	0,0
Riet	Lower	65	8,78	31,64	25,0	29,0	33,5
		60	8,53	31,64	25,9	30,5	36,6
		55	8,32	31,64	27,4	30,5	30,5
		50	8,08	31,64	24,4	27,4	30,5
		45	7,86	31,64	25,9	29,0	32,0
		40	7,62	31,64	0,0	0,0	0,0
Blesbok	Lower	8	13,17	20,18	22,9	24,4	27,4
		7	13,23	20,30	24,4	26,8	29,9
		6	13,26	20,42	21,3	24,4	27,4
		5	13,32	20,54	25,9	29,0	30,5
		4	13,35	20,67	24,8	25,9	29,9
		3	13,41	20,82	0,0	0,0	0,0

For the last cross section of the river system (cross section 40) you can enter the following description: *Downstream River Station*

- Remember to save the Geometry data at regular intervals in case of a power failure or human error. To assist with this you could also go to the main HEC-RAS screen and set the Backup saving function at a short interval (**Options** menu, **Program Setup**, **Set Time for Automatic Backup**... and enter 5 minutes). Make sure the **Automatically Backup Data** is selected (see **Figure 11.69**).

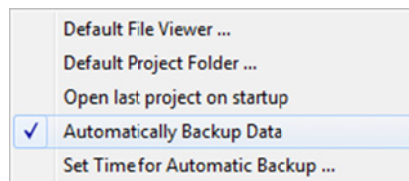


Figure 11.69: Automatically Backup Data

### Entering junction data



The next step is to enter the junction data. This is accomplished by clicking the **Junction** button on the Geometric window (**Figure 11.58**). Once this button is clicked, the Junction Data editor will appear as shown in **Figure 11.70**.

**Junction Data - Base Geometry**

Junction Name:

Description:

Length across Junction	Junction Length (m)	Tributary Angle (Deg)
From: Riet river - Lower reach	24.4	
To: Riet river - Upper reach	21.3	
To: Blesbok river - Lower reach		

Steady Flow Computation Mode:  
☒ Energy  
☐ Momentum  
☐ Add Friction  
☐ Add Weight


Unsteady Flow Computation Mode:  
☒ Force Equal WS Elevations  
☐ Energy Balance Method

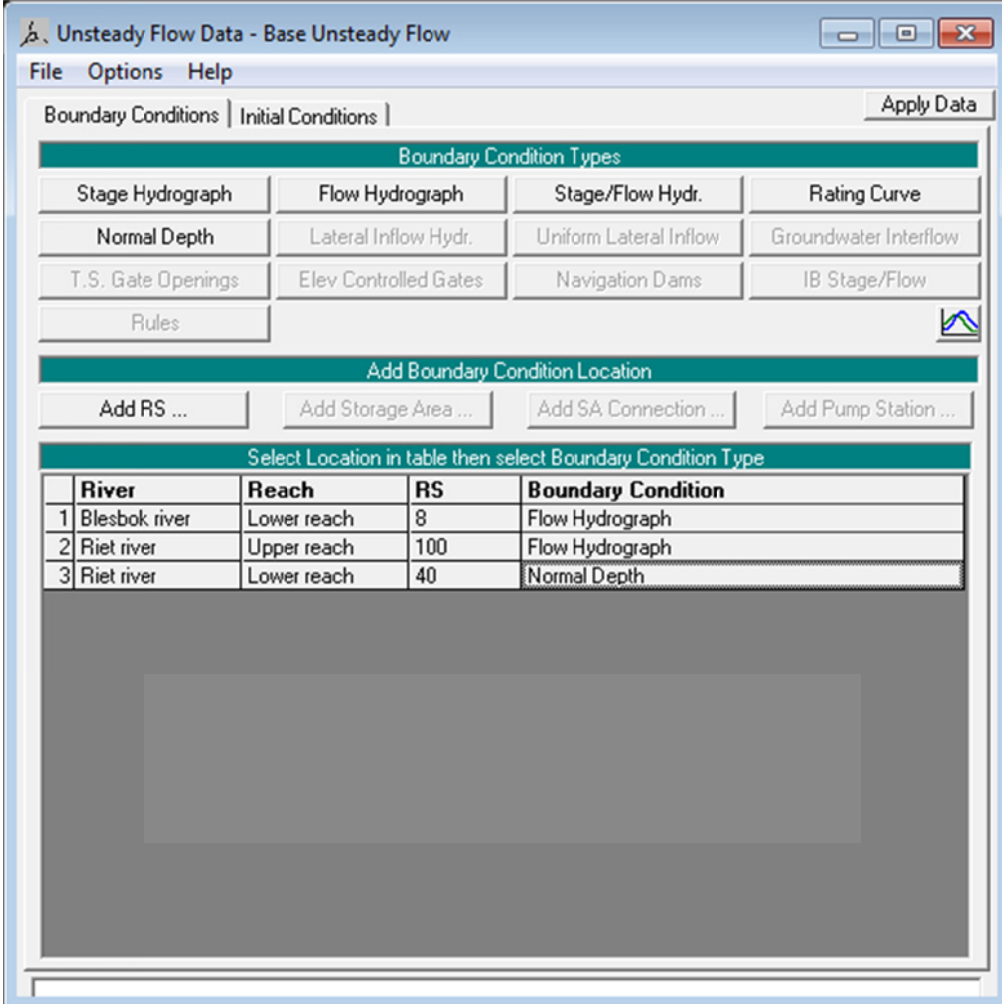
Select Junction to Edit:

**Figure 11.70: Junction Data Editor**

- There is only one junction in the river system. For this junction add the following description: *Division of Upper Reach and Lower Reach at Confluence with Blesbok River* (see **Figure 11.70**)
- Reach lengths across the junction are entered in the junction editor, rather than in the cross section data. This allows for the lengths across very complicated confluences (i.e. flow splits) to be accommodated. This is the reason why the reach lengths in the cross section data for the last cross section of each reach was left blank or set to zero. Enter the junction lengths of 24,4 m and 21,3 m for the Riet River – Upper reach and Blesbok River – Lower reach respectively (see **Figure 11.70**).
- In this exercise the energy equation will be used to compute the water surface profiles through the junction for steady flow computations i.e. select the *Energy* option. If the momentum equation was selected, then an angle can be entered for one or more of the reaches flowing into or out of a junction. The momentum equation is set up to account for the angle of the flow entering the junction.
- For Unsteady flow computations the *Force Equal WS Elevations* option is selected.
- Once you have entered all the data for the junction, click on the **Apply** button and close the window by pressing the OK button.
- Remember to save the Geometry data by clicking on **Save Geometry Data** from the **File** menu.
- Once all the data has been successfully entered your system should look like the schematic layout shown in **Figure 11.60**.

## ENTERING AND EDITING UNSTEADY FLOW DATA

Once all of the geometric data are entered, the modeler can then enter the unsteady flow data that is required. To bring up the unsteady flow data editor, select **Unsteady Flow Data** from the **Edit** menu on the HEC-RAS main window or clicking the short cut button on the menu bar . The **Unsteady Flow** data editor should appear as shown in **Figure 11.71**, for this exercise.



The screenshot shows the 'Unsteady Flow Data - Base Unsteady Flow' window. It has a menu bar with 'File', 'Options', and 'Help'. Below the menu bar are two tabs: 'Boundary Conditions' (selected) and 'Initial Conditions'. An 'Apply Data' button is in the top right. The main area is divided into two sections. The top section, 'Boundary Condition Types', contains a grid of buttons for various boundary conditions: Stage Hydrograph, Flow Hydrograph, Stage/Flow Hydr., Rating Curve, Normal Depth, Lateral Inflow Hydr., Uniform Lateral Inflow, Groundwater Interflow, T.S. Gate Openings, Elev Controlled Gates, Navigation Dams, IB Stage/Flow, and a 'Rules' button. The bottom section, 'Add Boundary Condition Location', contains four buttons: 'Add RS ...', 'Add Storage Area ...', 'Add SA Connection ...', and 'Add Pump Station ...'. Below these buttons is a table with the caption 'Select Location in table then select Boundary Condition Type'. The table has four columns: 'River', 'Reach', 'RS', and 'Boundary Condition'. It contains three rows of data.

	River	Reach	RS	Boundary Condition
1	Blesbok river	Lower reach	8	Flow Hydrograph
2	Riet river	Upper reach	100	Flow Hydrograph
3	Riet river	Lower reach	40	Normal Depth

**Figure 11.71: Unsteady Flow Data Editor**

The user is required to enter boundary conditions at all of the external boundaries of the system, as well as any desired internal locations, and set the initial flow conditions at the beginning of the simulation.

Boundary conditions are entered by first selecting the **Boundary Conditions** tab from the **Unsteady Flow Data** editor. River, Reach, and River Station locations of the external bounds of the system will automatically be shown in the table. Boundary conditions are entered by first selecting a cell in the table for a particular location, then selecting the boundary condition type that is desired at that location. Not all boundary condition types are available for use at all locations. The program will automatically gray-out the boundary condition types that are not relevant when the user highlights a particular location in the table.

Users can also add locations for entering other internal boundary conditions. To add additional boundary condition location, select the desired **River**, **Reach**, and **River Station** from the drop down lists and press the **Add a Boundary Condition Location** button. In this exercise this is however not required.

## Boundary Conditions

There are several different types of boundary conditions available to the user. The following is a short discussion of each type:

- **Flow Hydrograph (this is the type that you can enter for this exercise)**

A flow hydrograph can be used as either an upstream boundary or downstream boundary condition, but is most commonly used as an upstream boundary condition. When the flow hydrograph button is pressed, the window shown in **Figure 11.72** will appear. As shown, the user can either read the data from a HEC-DSS (HEC Data Storage System) file, or can enter the hydrograph ordinates into a table (Since River station 8 was selected as shown in **Figure 11.71** the flow hydrograph detail for this stations is first required).

**Flow Hydrograph**

River: Blesbok river Reach: Lower reach RS: 8

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Hour

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date:  Time:

☐ Fixed Start Time: Date:  Time:

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(m3/s)
1		00:00	
2		01:00	
3		02:00	
4		03:00	
5		04:00	
6		05:00	
7		06:00	

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow:  Multiplier:

**Figure 11.72: Flow Hydrograph (River station 8 – Blesbok River)**

- The user also has the option of entering a flow hydrograph directly into a table, as shown in **Figure 11.72**. The first step is to select a **Data time interval** from the drop down list. Currently the program only supports regular interval time series data.

A list of allowable time intervals is shown in the drop down window of the data interval list box. For this exercise select *15 Minute*.

- To enter data into the table, the user is required to select either **Use Simulation Time** or **Fixed Start Time**. For this exercise select *Use Simulation Time*.
- Instead of entering the data points of the flow hydrographs one-by-one by hand open the Excel spreadsheet entitled: *Exercise 2 - Flow hydrographs.xls*. **Copy** the flow rate values for the Blesbok River flow hydrograph (only the flow rates). Return to the HEC-RAS **Flow Hydrograph** data editor, select at least 25 rows in the hydrograph table and press **Ctrl V** on the keyboard to paste the copied data (see **Figure 11.73**).

**Flow Hydrograph**

River: Blesbok river Reach: Lower reach RS: 8

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 15 Minute

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date:  Time:

☐ Fixed Start Time: Date:  Time:

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(m3/s)
1		00:00	0.5
2		00:15	0.5
3		00:30	0.5
4		00:45	2.
5		01:00	7.
6		01:15	14.
7		01:30	20.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

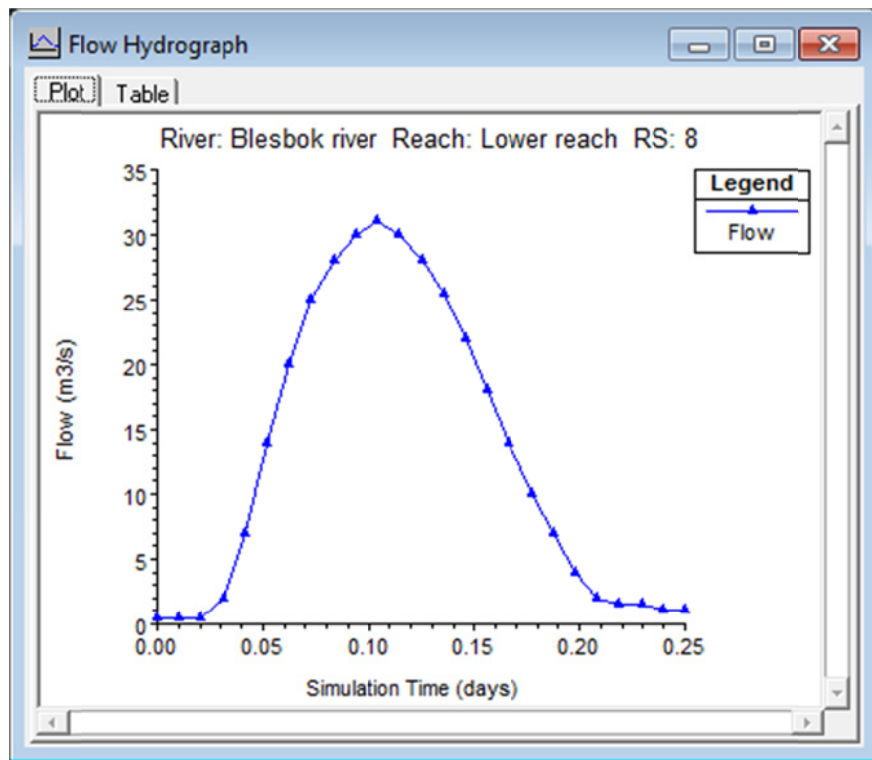
Min Flow:  Multiplier:

**Figure 11.73: Flow Hydrograph (River station 8 – Blesbok River)**

- An option listed at the bottom of the flow hydrograph boundary condition is to make this boundary a **Critical Boundary Condition**. When you select this option, the program will monitor the inflow hydrograph to see if a change in flow rate from one time step to the next is exceeded.

If the change in flow rate does exceed the user entered maximum, the program will automatically cut the time step in half until the change in flow rate does not exceed the user specified max.

- The other options at the bottom of this editor are **Min Flow** and **Multiplier**. Both of these options apply to user entered hydrographs or hydrographs read from HEC-DSS. The “Min Flow” option allows the user to specify a minimum flow to be used in the hydrograph. This option is very useful when too low of a flow is causing stability problems.
- The flow hydrograph for this exercise can be plotted by clicking on the **Plot Data** button (see **Figure 11.74**).



**Figure 11.74: Flow hydrograph (Plot of River station 8 flow hydrograph)**

The maximum peak flow value is 31,0 m³/s (time 2:30)

- Similarly a flow hydrograph can be entered for River station 100, which is the upstream boundary of the Riet River – Upper reach.
- The completed **Flow Hydrograph** editor screen and plot screen is shown in **Figure 11.75** and **Figure 11.76** respectively. The maximum peak flow value is 84,0 m³/s (time 2:00).

The spreadsheet (*Exercise 2 - Flow hydrographs.xls*) containing the flow hydrographs is provided on the supporting flash drive.



**Flow Hydrograph**

River: Riet river Reach: Upper reach RS: 100

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 15 Minute

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date:  Time:

☐ Fixed Start Time: Date:  Time:

Hydrograph Data		
	Date	Simulation Time
		(hours)
1		00:00
2		00:15
3		00:30
4		00:45
5		01:00
6		01:15
7		01:30

Flow (m3/s)

1.  
10.  
20.  
30.  
48.  
66.  
76.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow:  Multiplier:

Figure 11.75: Flow Hydrograph (River station 100 – Riet River)

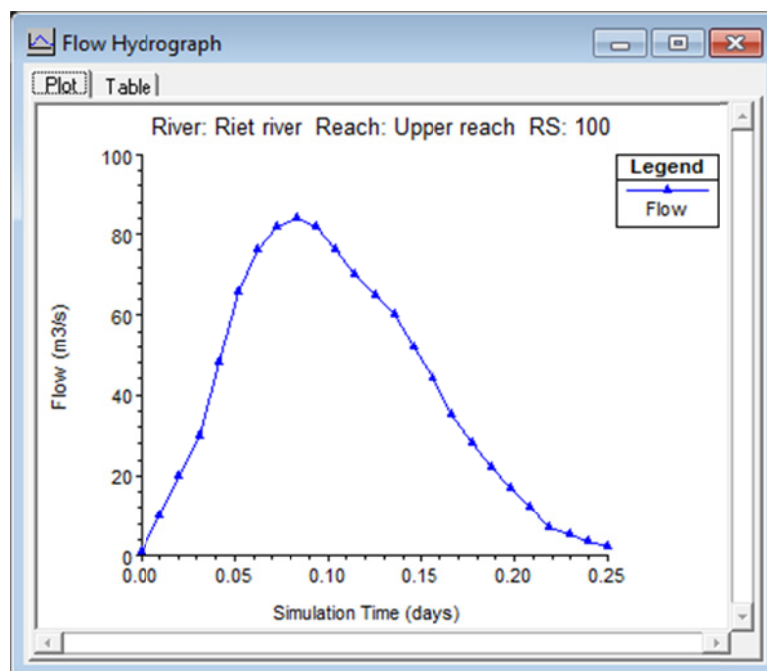
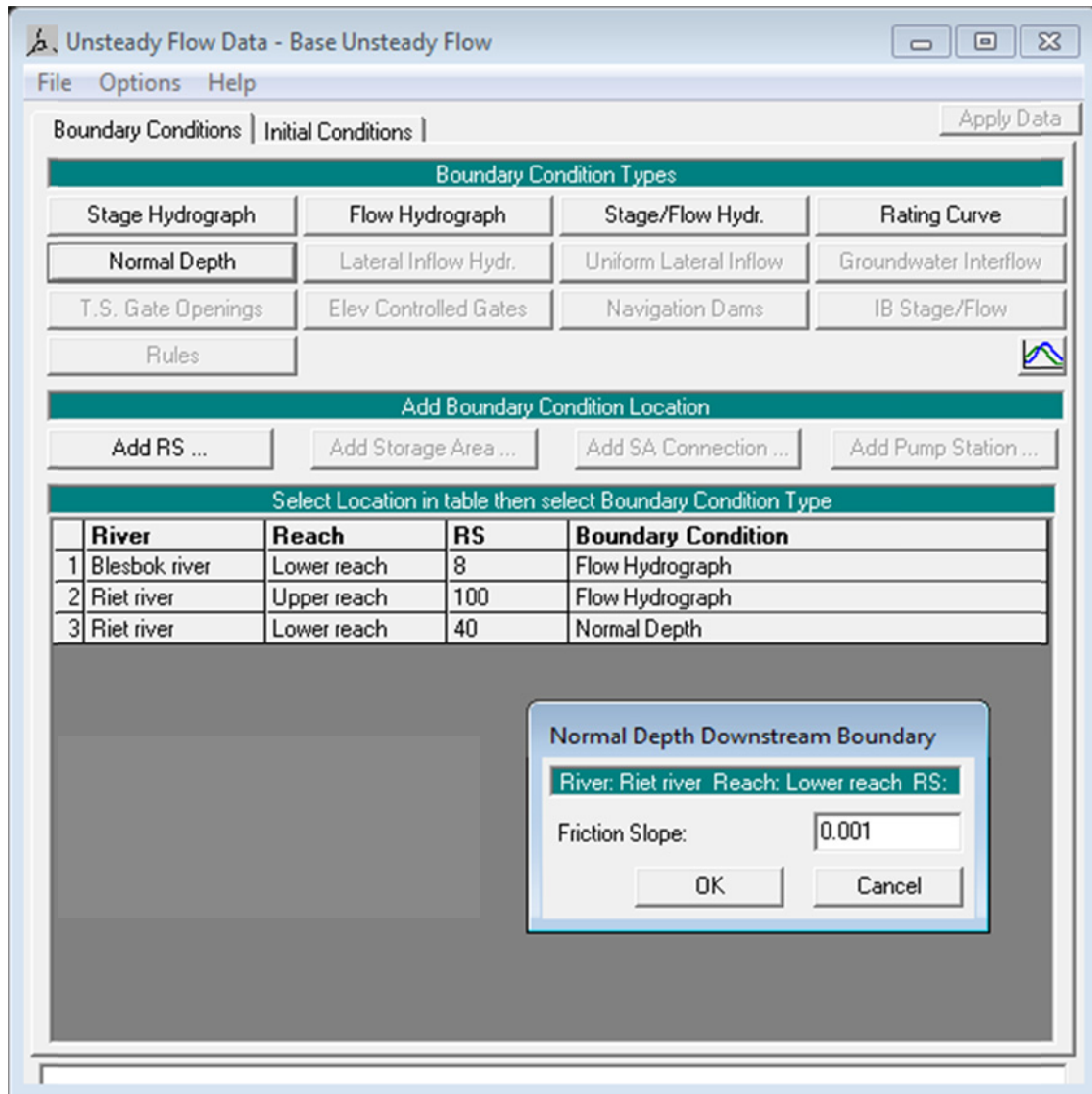


Figure 11.76: Flow hydrograph (Plot of River station 100 flow hydrograph)

- **Normal Depth (this is the type that you can enter for this exercise)**

The **Normal Depth** option can only be used as a downstream boundary condition for an open-ended reach. This option uses Manning's equation to estimate a stage for each computed flow. Select River station 40's cell on the **Unsteady Flow Data** editor window. Click the **Normal Depth** button. A pop-up window will prompt you to enter the downstream friction slope, which should be used to calculate the flow depth at this downstream boundary for each flow (in the time series arriving at this river station), see **Figure 11.77**. Enter a slope of  $0,001\text{ m/m}$ .



**Figure 11.77: Normal flow depth boundary (River station 40)**

Other boundary types not used in this example:

### Stage Hydrograph

A stage hydrograph can be used as either an upstream or downstream boundary condition. The editor for a stage hydrograph is similar to the flow hydrograph editor. The user has the choice of either attaching a HEC-DSS file and path name or entering the data directly into a table.

### Rating Curve

The rating curve option can be used as a downstream boundary condition. The user can either read the rating curve from HEC-DSS or enter it by hand into the editor. The downstream rating curve is a single valued relationship, and does not reflect a loop in the rating, which may occur during an event.

### Elevation Controlled Gate

This option allows the user to control the opening and closing of gates based on the elevation of the water surface upstream of the structure. A gate begins to open when a user specified elevation is exceeded. The gate opens at a rate specified by the user.

### Initial Conditions

- In addition to the boundary conditions, the user must establish the initial conditions of the system at the beginning of the unsteady flow simulation. Initial conditions consist of flow and stage information at each of the cross sections, as well as elevations for any storage areas defined in the system. Initial conditions are established from within the **Unsteady Flow Data** editor by selecting the **Initial Conditions** tab. After the **Initial Conditions** tab is selected, the **Unsteady Flow** editor will appear as shown in **Figure 11.78**.

Unsteady Flow Data - Base Unsteady Flow

File Options Help

Boundary Conditions: Initial Conditions Apply Data

Initial Flow Distribution Method

☐ Use a Restart File Filename:

☒ Enter Initial flow distribution

Add RS...

Locations of Flow Data Changes				
	River	Reach	RS	Initial Flow
1	Blesbok river	Lower reach	8	0.5
2	Riet river	Upper reach	100	1
3	Riet river	Lower reach	65	1.5

Initial Elevation of Storage Areas

Storage Area	Initial Elevation
1	

Figure 11.78: Unsteady Flow Data (Initial Conditions)

- As shown in **Figure 11.78**, the user has two options for establishing the initial conditions of the system. The first option is to enter flow data for each reach and have the program perform a steady flow backwater run to compute the corresponding stages at each cross section.

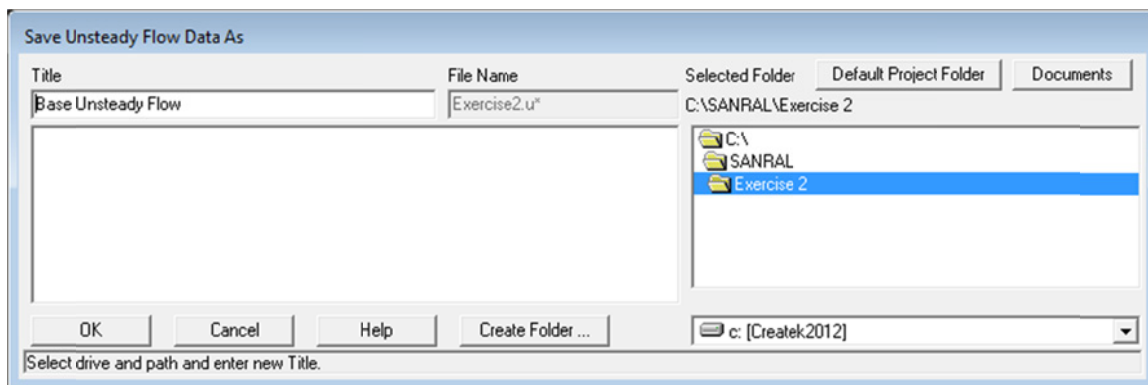
This option also requires the user to enter a starting elevation for any storage areas that are part of the system. This is the most common method for establishing initial conditions. Flow data can be changed at any cross section, but at a minimum the user must enter a flow at the upper end of each reach.

The second option is to read in a file of stages and flows that were written from a previous run, which is called a “Restart file”. The first option is used in this exercise.

Enter the initial flows as indicated in **Figure 11.78**,  $0,5 \text{ m}^3/\text{s}$ ,  $1,0 \text{ m}^3/\text{s}$  and  $1,5 \text{ m}^3/\text{s}$  for cross sections 8, 100 and 65 respectively.

### Saving the Unsteady Flow Data

- The last step in developing the unsteady flow data is to save the information to a file. To save the data, select the **Save Unsteady Flow Data As** from the **File** menu on the **Unsteady Flow Data** editor. A pop-up window will appear prompting you to enter a title for the data as shown in **Figure 11.79**. Enter “*Base Unsteady Flow*” for this exercise, and then press the **OK** button. A file name is automatically assigned to the Unsteady Flow Data based on what you entered for the project file name i.e. Exercise1.u01.



**Figure 11.79: Saving Unsteady Flow Data**

### Other unsteady Flow Data Options

Several options are available from the **Unsteady Flow Data** editor to assist users in entering and viewing data. These features can be found under the Options menu at the top of the window. The following options are available:

#### Delete Boundary Condition

This option allows the user to delete a boundary condition from the table. To use this option, first select the row to be deleted with the mouse pointer. Then select **Delete Boundary Condition** from the options menu. The row will be deleted and all rows below it will move up one. Only user inserted boundary conditions can be deleted from the table. If the boundary condition is an open end of the system, the system will not allow that boundary to be deleted. There must always be some type of boundary condition at all the open ends of the system.

#### Internal RS Initial Stages

This option allows the user to specify starting water surface elevations for any internal cross section within the system. A common application of this would be to specify the starting pool elevation for the first cross section upstream of a dam (modeled with the inline weir/spillway

option). The user specifies locations and water surface elevations, which are then used to establish the initial conditions for the system at the beginning of a run.

### **Observed Data In DSS**

This option allows the user to attach observed data pathnames from a HEC-DSS file to specific river stations within the model.

When an observed data path name is attached to a specific river station location, the user can get a plot of the observed flow or stage hydrograph on the same plot as the computed flow and stage hydrographs. Additionally, the observed data will show up on profile and cross section plots. To use this option, the user selects **Observed Data In DSS** from the **Options** menu of the **Unsteady Flow Data** editor.


### **Minimum Flow and Flow Ratio Table**

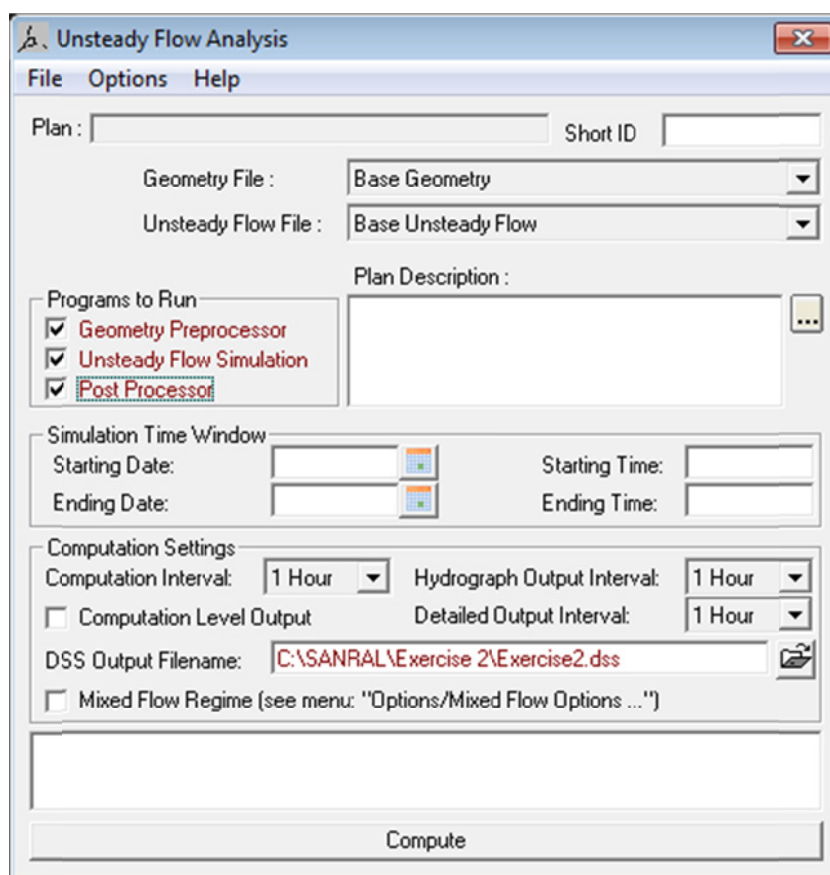
This option brings up a global editor that will show all the locations in which flow hydrographs have been attached as boundary conditions. The editor allows the user to enter a minimum flow or a flow factor for each flow hydrograph boundary condition.

## **UNSTEADY FLOW ANALYSIS**

### **Performing Unsteady Flow Calculations**

Once all of the geometry and unsteady flow data have been entered, the user can begin performing the unsteady flow calculations. To run the simulation, go to the HEC-RAS main window and select **Unsteady Flow Analysis** from the **Run** menu or click on the **Unsteady**

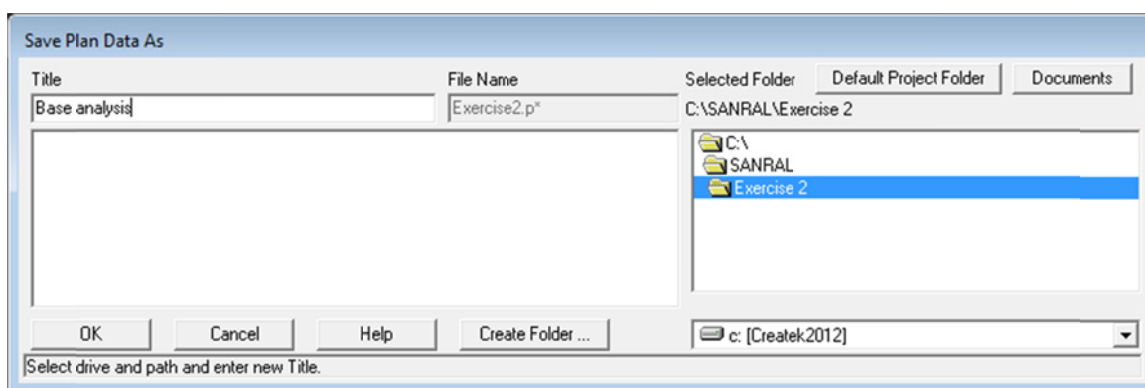
**Flow Analysis** button  on the menu bar. The **Unsteady Flow Analysis** window will appear as shown in **Figure 11.80**.



**Figure 11.80: Unsteady Flow Analysis**

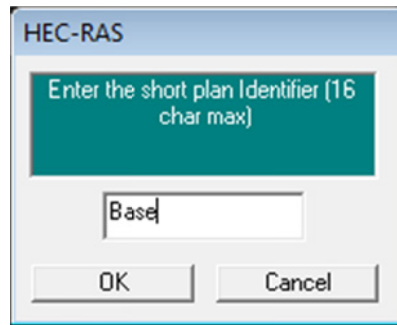
- The first step is to put together a **Plan**. The **Plan** defines which geometry and flow data are to be used, as well as providing a title and short identifier for the run.

To establish a plan, select **New Plan** from the **File** menu on the **Unsteady Flow Analysis** window. Enter the plan title as *Base analysis* and then press the **OK** button (Figure 11.81).



**Figure 11.81: Creating new plan**

- You will be prompted to enter a short identifier. Enter a title of *Base* in the **Short ID** box (Figure 11.82) and click on the **OK** button.



**Figure 11.82: Plan identifier**

- **Selecting Programs to Run**

There are three components used in performing an unsteady flow analysis within HEC-RAS. These components are: a geometric data pre-processor; the unsteady flow simulator; and an output post-processor (see **Figure 11.80**).

- **Geometric Pre-Processor**

The pre-processor is used to process the geometric data into a series of hydraulic properties tables, rating curves, and family of rating curves. This is done in order to speed up the unsteady flow calculations. Instead of calculating hydraulic variables for each cross-section, during each iteration, the program interpolates the hydraulic variables from the tables. The pre-processor must be executed at least once, but then only needs to be re-executed if something in the geometric data has changed.

- **Unsteady Flow Simulation**

The unsteady flow computations within HEC-RAS are performed by a modified version of the UNET (Unsteady NETwork model) program, developed by Dr. Robert Barkau (Barkau, 1992) and modified by HEC. The unsteady flow simulation is actually a three-step process. First, a program called RDSS (Read DSS data) runs.

This software reads data from a HEC-DSS file and then converts all of the boundary condition time series data into the user specified computation interval. Next, the UNET program runs. This software reads the hydraulic properties table computed by the pre-processor, as well as the boundary conditions and flow data from the interface and the RDSS program.

The program then performs the unsteady flow calculations. The final step is a program called TABLE. This software takes the results from the UNET unsteady flow run and writes them to a HEC-DSS file.

- **Post-Processor**

The post-processor is used to compute detailed hydraulic information for a set of user specified time lines during the unsteady flow simulation periods. In general, the unsteady flow computations only compute stage and flow at all of the computation nodes, as well as stage and flow hydrographs at user specified locations. If the post-processor is not run, then the user will only be able to view the stage and flow hydrographs and no other output from HEC-RAS. By running the post-processor, the user will have all of the available plots and tables for unsteady flow that HEC-RAS normally produces for steady flow.

- **Simulation time window**

The user is required to enter a time window that defines the start and end of the simulation period. The time window requires a starting date and time and an ending date and time.



In this exercise the flow hydrograph shown in **Figure 11.73** and **Figure 11.75** starts at 0:00 and has data until 06:00 and this is used in the simulation time window. The date can be anything since the option to **Use Simulation Time** on the Flow Hydrograph window was selected (see **Figure 11.73** and **Figure 11.75**). Enter the **Starting date** as *01JAN2013* and the **Ending date** as *01JAN2013*. Enter the **Starting time** as *0000* and the **Ending time** as *0600*.

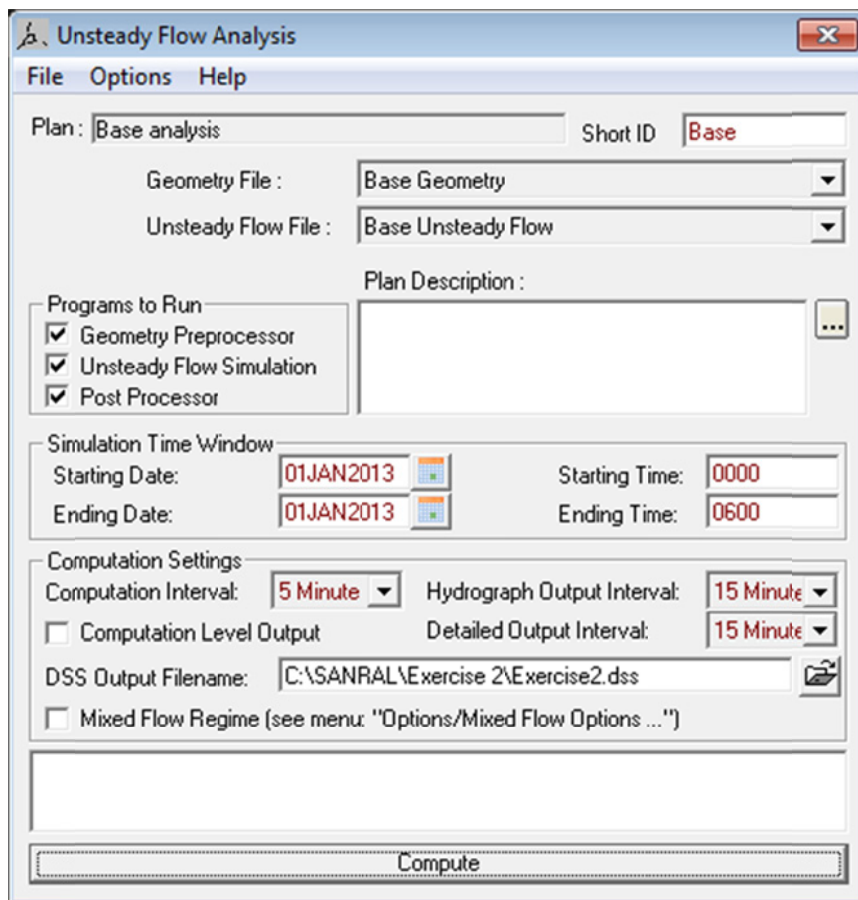
- **Computational settings**

The computational Settings area contains the **Computational Interval**, **Hydrograph Output Interval**, **Detailed Output Interval**, the name and path of the output DSS file, and whether or not the program is run in a mixed flow regime. The computation interval is probably one of the most important parameters entered into the model. It should be small enough to accurately describe the rise and fall of the hydrographs being routed but not small to take forever to compute.

For this exercise set the **Computational Interval** at *5 minutes* (from the drop down list). Set the **Hydrograph Output Interval** to *15 minutes* and the **Detailed Output Interval** also at *15 minutes*.

The **DSS Output filename** is the file that contains all the calculated data in a format that can be read by HEC-RAS and used in displaying all the results (tables and graphs). The default will be *.....\Exercise2.dss* and does not have to be changed for this exercise.

The completed **Unsteady Flow Analysis** screen is shown in **Figure 11.83**.



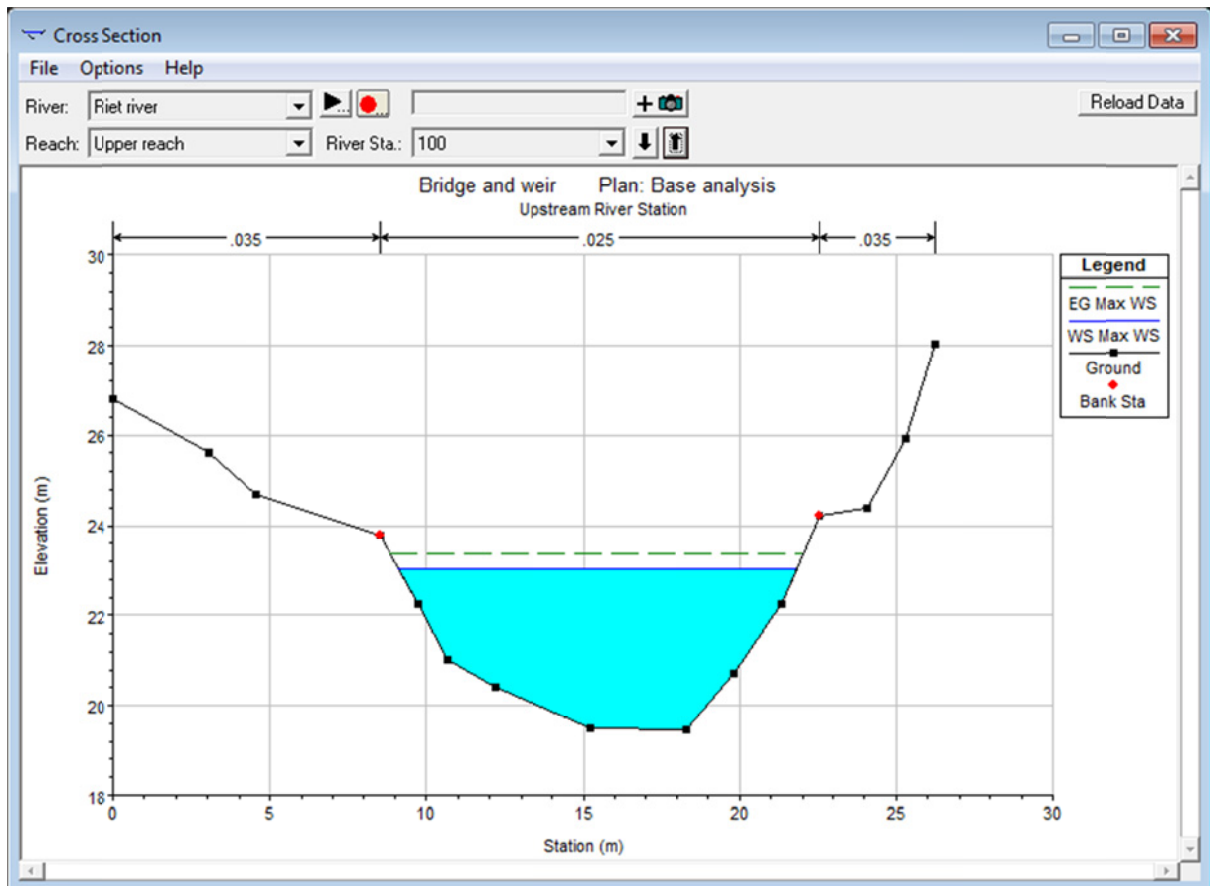
**Figure 11.83: Unsteady Flow Analysis (completed)**

Click on the **Compute** button to run the **Unsteady Flow Analysis**.



## VIEWING THE RESULTS

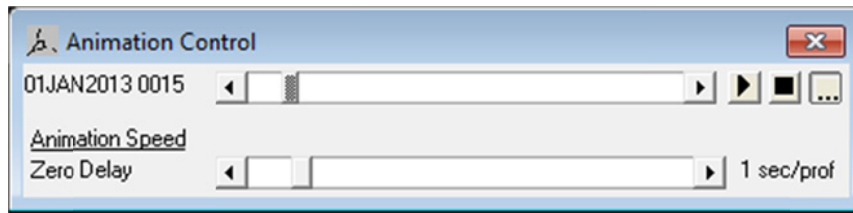
Once the model has finished all of the computations successfully, you can begin viewing the results. Several output options are available from the **View** menu bar on the HEC-RAS main window. These options include:

- a) Cross section plots
  - b) Profile plots
  - c) General profile plot
  - d) Rating curves
  - e) X-Y-Z perspective plots
  - f) Detailed tabular output at a specific cross section (cross section table)
  - g) Limited tabular output at many cross sections (profile table)
- Begin by plotting a cross section. Select **Cross Sections** from the **View** menu bar on the HEC-RAS main window. Any cross section can be plotted by selecting the appropriate river, reach and river station (See **Figure 11.84**). Several plotting features are available from the **Options** menu bar on the cross section plot window. These options include: zoom in; zoom out; selecting which plans, profiles, variables to plot; and control over lines, labels, symbols, scaling etc.





**Figure 11.84: Cross section (Riet River: Upper reach – River station 100)**

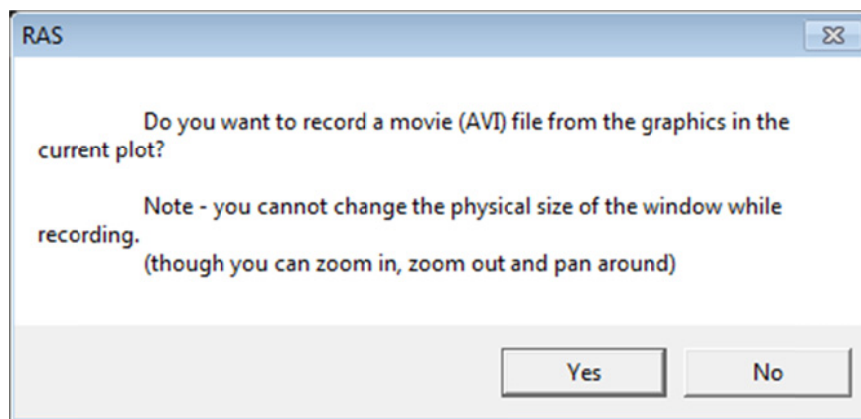
- Select different cross sections to plot and practice using some of the features available under the options menu bar. First try and view the change in water level at this cross section. Click on the **Play** button, , on the **Cross Section** window. An **Animation Control** window will appear (see **Figure 11.85**). Click on the **Expand** button, , to see the entire control.




**Figure 11.85: Animation Control**

Set the **Zero Delay** horizontal scroll bar as indicated (in **Figure 11.85**) and click on the play button, , to view the changing water surface levels and energy grade lines at this cross section. Click the **Stop** button, , to stop the animation.

- Try and make a movie clip by viewing the change in flow levels with time (**Figure 11.84**), this can be done by clicking on the **Record** button (red dot **Figure 11.84**) to start recording and clicking it again will stop recording. You will be prompted as shown in **Figure 11.86**, whether or not you would like to record a movie (AVI). Click on the **Yes** button, remember the program starts recording as soon as you click on the **Yes** button.



**Figure 11.86: Confirming that a movie clip should be recorded**

Click on the Play button, , to start playing the animation as explained earlier. Once the animation ends click on the **Record** button (red dot **Figure 11.84**) again to stop recording. The recorded screen captures will be shown as indicated in **Figure 11.87**. You now have the option of saving the screens after performing some editorial work to an AVI file by clicking on the **Write AVI** button.

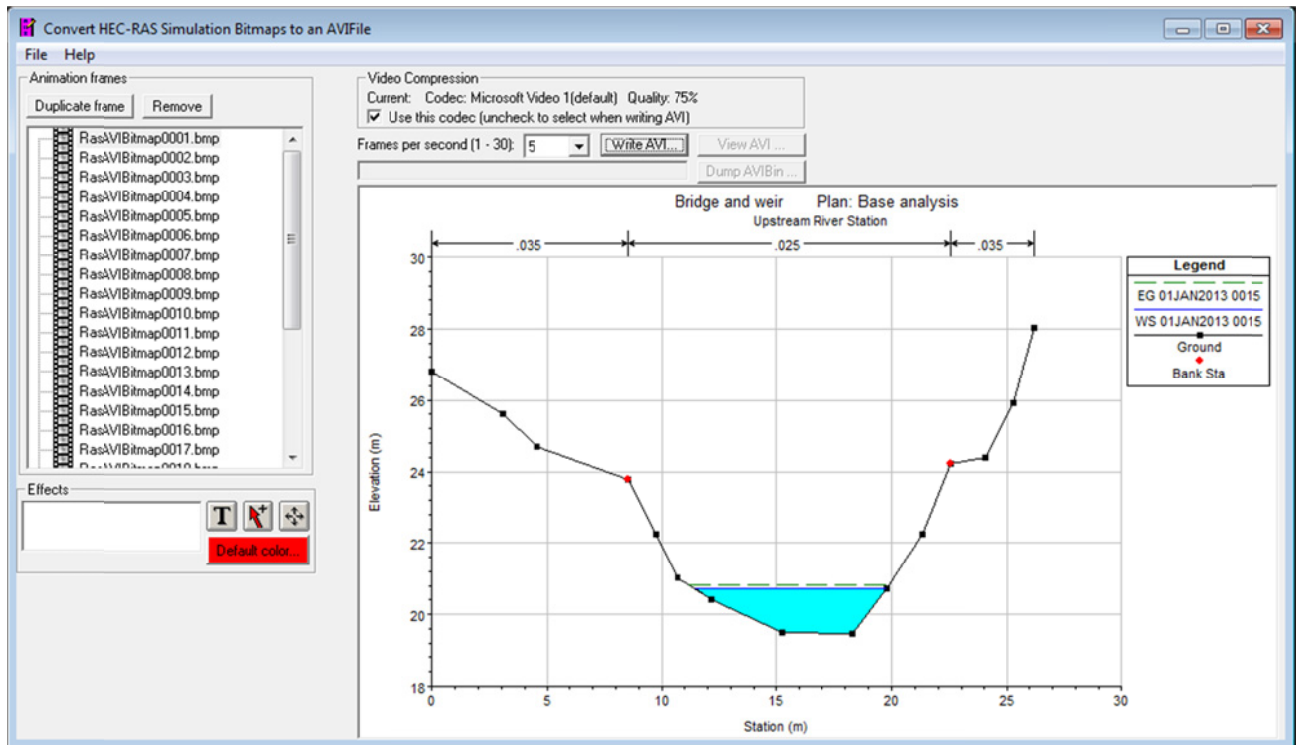


Figure 11.87: Recorded screens

- Next plot a water surface profile. Select **Water Surface Profiles** from the **View** menu bar. This should give you a profile plot as shown in **Figure 11.88**.

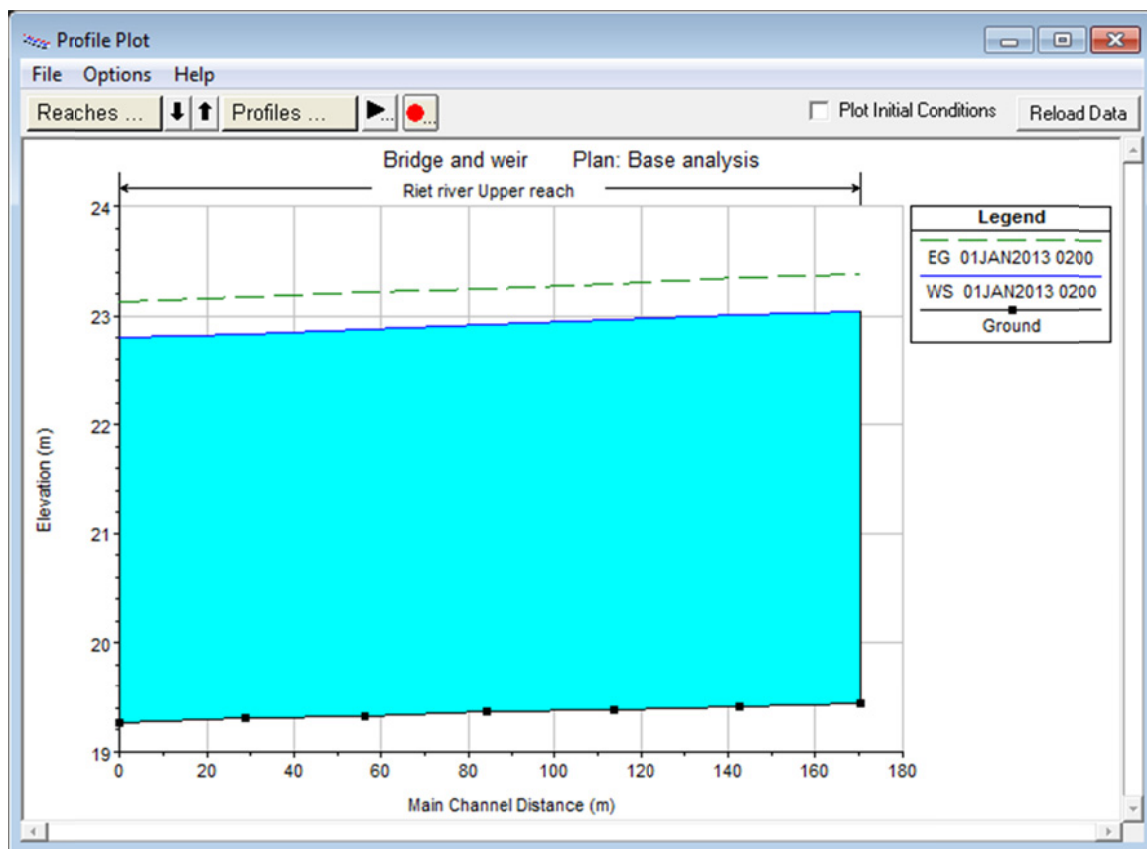
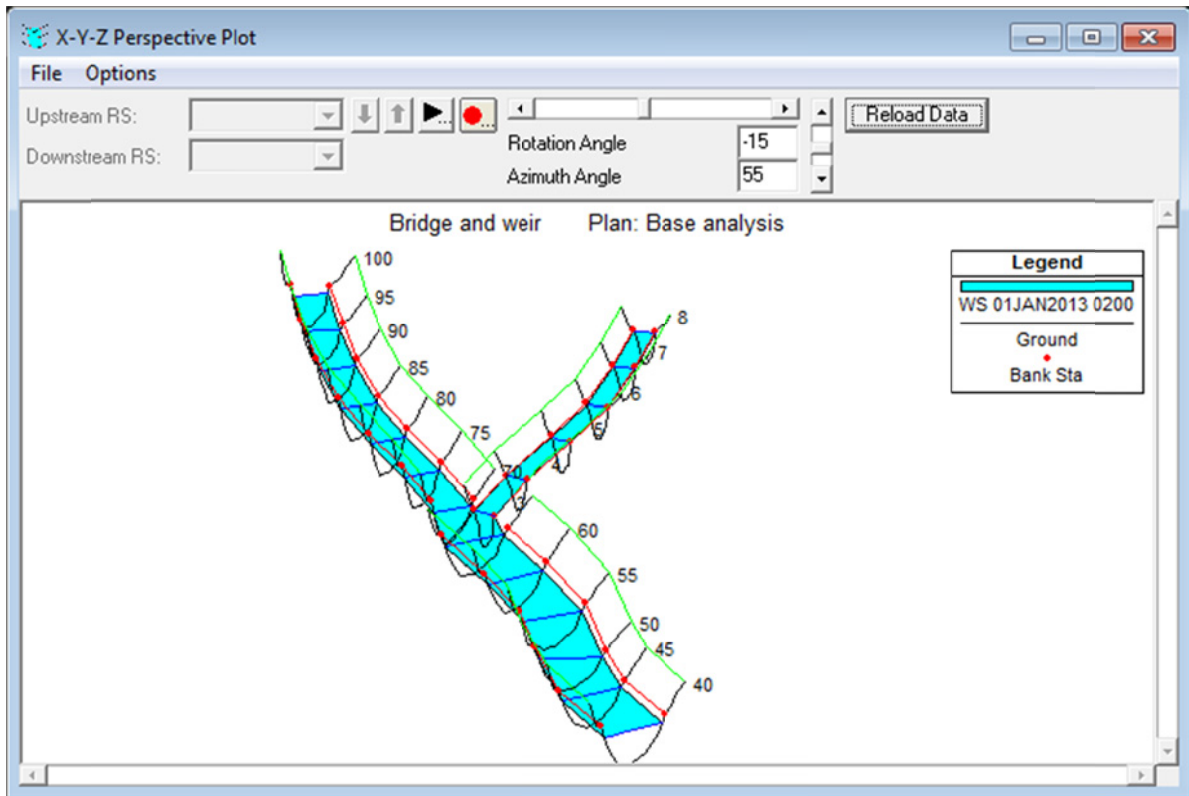


Figure 11.88: Water Surface Profile (for Riet River: Upper reach)

- Also have a look at a **General Profile Plot** and the **X-Y-Z Perspective Plot (Figure 11.89)**. Also look at some tabular output. Go to the **View** menu bar on the HEC-RAS main window. There are two types of tables available, a detailed output table and a profile summary table. Select **Detailed Output Tables** to get the first table to appear. This table shows detailed hydraulic information at the cross section. Other cross sections can be viewed by selecting the appropriate reach and river from the table. A table with all the errors, warnings and comments can also be viewed, by selecting **Summary, Err Warn, Notes...** from the **View** menu on the HEC-RAS main window.



**Figure 11.89: X-Y-Z Perspective plot**

**You will now be in a position to answer some of the questions at the back of this exercise (Questions 1 to 5).**

## **BRIDGES**

In the next section a bridge structure will be added downstream of the confluence of the two rivers. HEC-RAS computes energy losses caused by structures such as bridges and culverts in three parts. One part consists of losses that occur in the reach immediately downstream from the structure where an expansion of flow takes place. The second part is the losses at the structure itself, which can be modeled with several different methods. The third part consists of losses that occur in the reach immediately upstream of the structure where the flow is contracting to get through the opening.

### **Cross section locations**

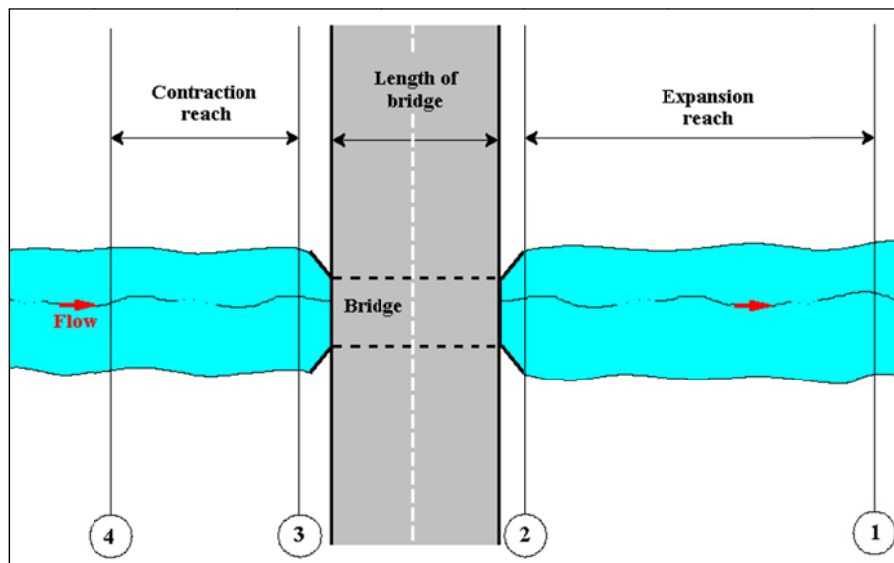
The bridge routine utilizes four user defined cross sections in the computation of energy losses due to the structure.

**Cross section 1** is located sufficiently downstream from the structure so that the flow is not affected by the structure (i.e. the flow has fully expanded)

**Cross section 2** is located immediately downstream from the bridge (i.e. within a short distance). This cross section should represent the natural ground just outside the bridge.

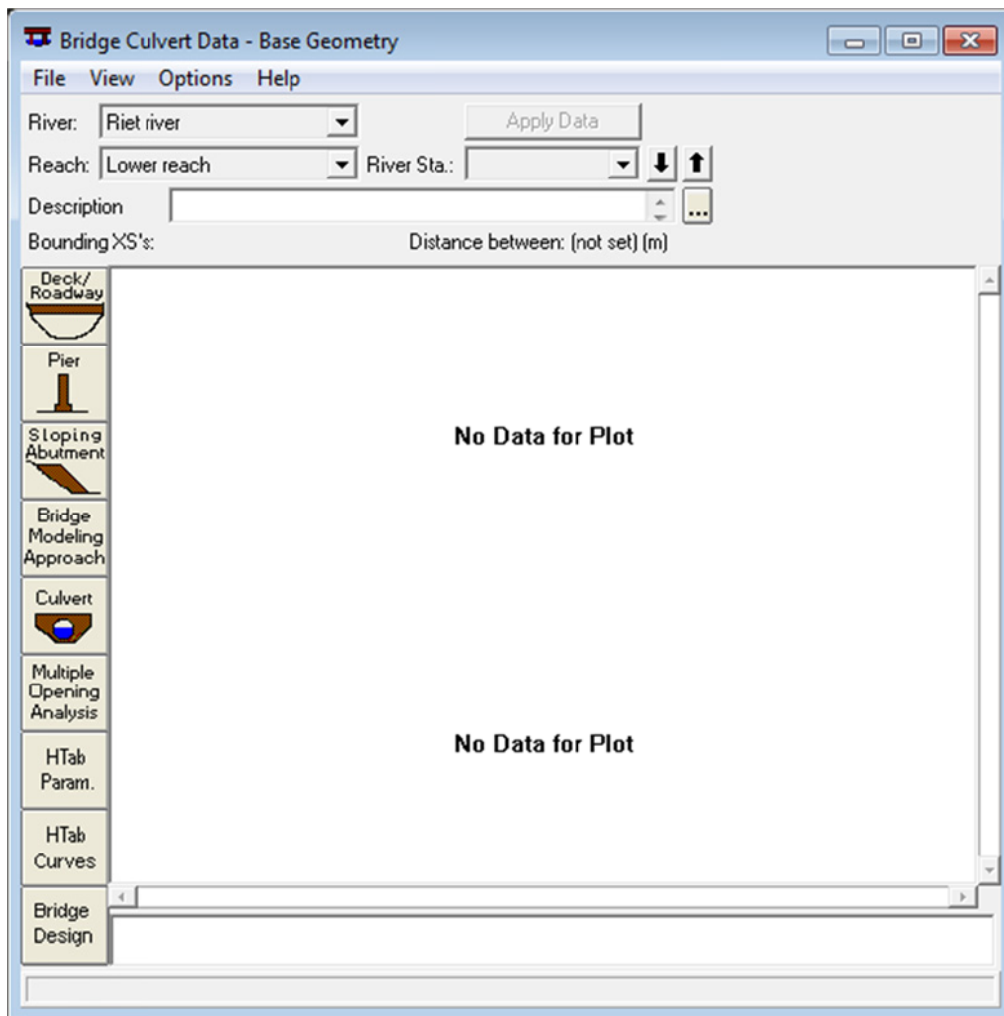
**Cross section 3** should be located just upstream of the bridge. The distance between cross section 3 and the bridge should be relatively short. This distance should only reflect the length required for the abrupt acceleration and contraction of the flow that occurs in the immediate area of the opening.

**Cross section 4** is an upstream cross section where the flow lines are approximately parallel and the cross section is fully effective.



#### Entering bridge data

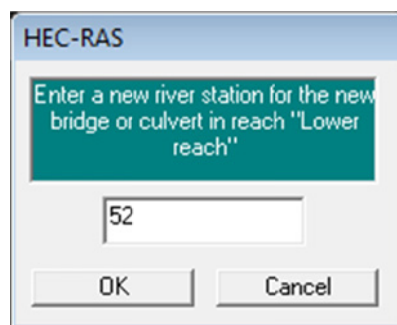
- To enter bridge data the user presses the **Bridge/Culvert** button on the **Geometric data** window (**Figure 11.60**). Once the **Bridge/culvert** button is pressed, the **Bridge/Culvert Data** Editor will appear as shown in **Figure 11.90**.



**Figure 11.90: Bridge/Culvert Data window**

To add a bridge to the model, take the following steps:

- Select the river and reach that you would like to place the bridge in (from the drop down lists) i.e. *Riet river* and *Lower reach*.
- From the **Options** menu, select **Add a Bridge and/or Culvert** from the list. An input box will appear prompting you to enter a river station identifier for the new bridge. Enter 52 as shown in **Figure 11.91**.



**Figure 11.91: Bridge river station (Riet River: Lower reach)**

- Enter the **Description** of the bridge: *Stephnie bridge* (see **Figure 11.92**)



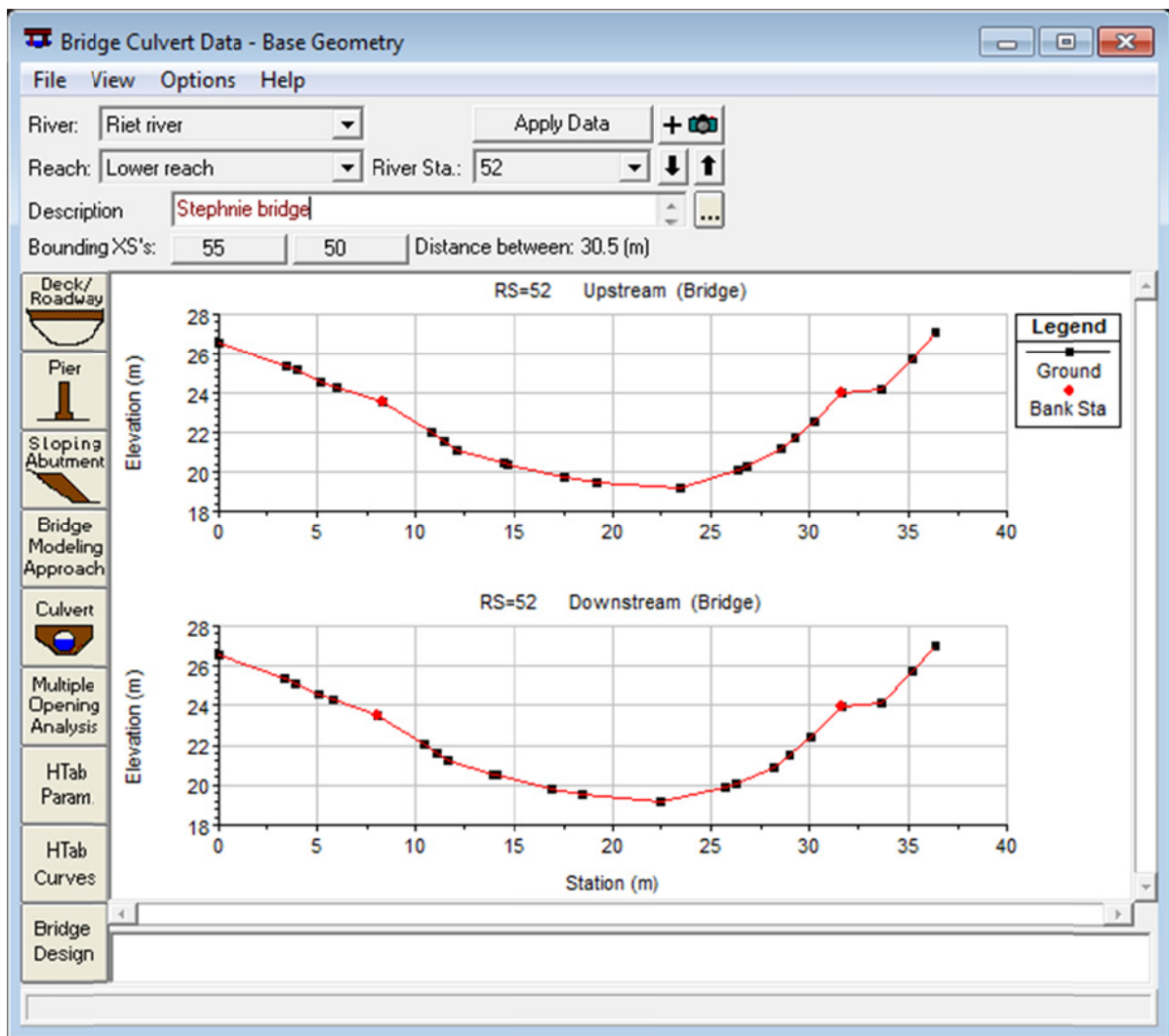


Figure 11.92: Upstream and downstream view of cross sections at bridge

- Enter all of the required data for the new bridge. This includes:
  - a. Bridge deck
  - b. Sloping abutments (optional)
  - c. Piers (optional)
  - d. Bridge modeling approach information
- From the **Bridge/Culvert Data** Editor select the **Deck/Roadway** icon to activate the **Deck/Roadway Data** Editor as shown in **Figure 11.93**.
- The first input at the top of the editor is the distance from the upstream side of the bridge deck to the cross section immediately upstream from the bridge (i.e. river station 55). This distance is 5 m.
- The bridge deck itself will have a width of 7,5 m. The weir flow coefficient selected for this analysis is 1,44.
- The bridge deck will be 0,9 m high and will have a slope across it of 0,05 m (for road drainage).

**Deck/Roadway Data Editor**

Distance	Width	Weir Coef
5	7.5	1.44

Upstream				Downstream			
	Station	high chord	low chord		Station	high chord	low chord
1	5.	23.5	20.	5.	23.45	20.	
2	12.5	23.5	20.	12.5	23.45	20.	
3	12.5	23.5	22.6	12.5	23.45	22.6	
4	27.5	23.5	22.6	27.5	23.45	22.6	
5	27.5	23.5	20.	27.5	23.45	20.	
6	32.5	23.5	20.	32.5	23.45	20.	
7							
8							

U.S Embankment SS: 
 D.S Embankment SS:

**Weir Data**  
 Max Submergence: 
 Min Weir Flow El:

**Weir Crest Shape**  
☒ Broad Crested  
☐ Ogee

Enter distance between upstream cross section and deck/roadway. (m)

**Figure 11.93: Bridge/Deck and Roadway data editor window**

- At every station position the high chord and low chord of the bridge should be entered as shown in **Figure 11.93** to provide a bridge shape as shown in **Figure 11.94**.
- The US and DS Embankment SS (upstream and downstream embankment side slope) values should be entered as 2 (horizontal to 1 vertical). These values are used for the graphical representation on the profile plot.
- At the bottom of the **Deck/Roadway Data Editor**, there are three additional fields of data entry. The first is the **Max Allowable Submergence**. This input ratio of downstream water depth to upstream energy, as measured above the minimum weir elevation. When the ratio is exceeded, the program will no longer consider the bridge deck to act as a weir and will switch the computation mode to energy (standard step) method. For this exercise the default value of 0.95 (95%) should be selected.
- The second field at the bottom of the editor is the **Min Weir Flow Elevation**. This is the elevation that determines when weir flow will start to occur over the bridge. If this field is left blank (as in this exercise), the program will default to use the lowest high cord value on the upstream side of the bridge. The last field at the bottom of the editor is the selection of the **Weir Crest Shape**. This selection will determine the reduction of the weir flow coefficient due to submergence. For this exercise, a **Broad Crested** weir shape should be selected.
- Click the **OK** button

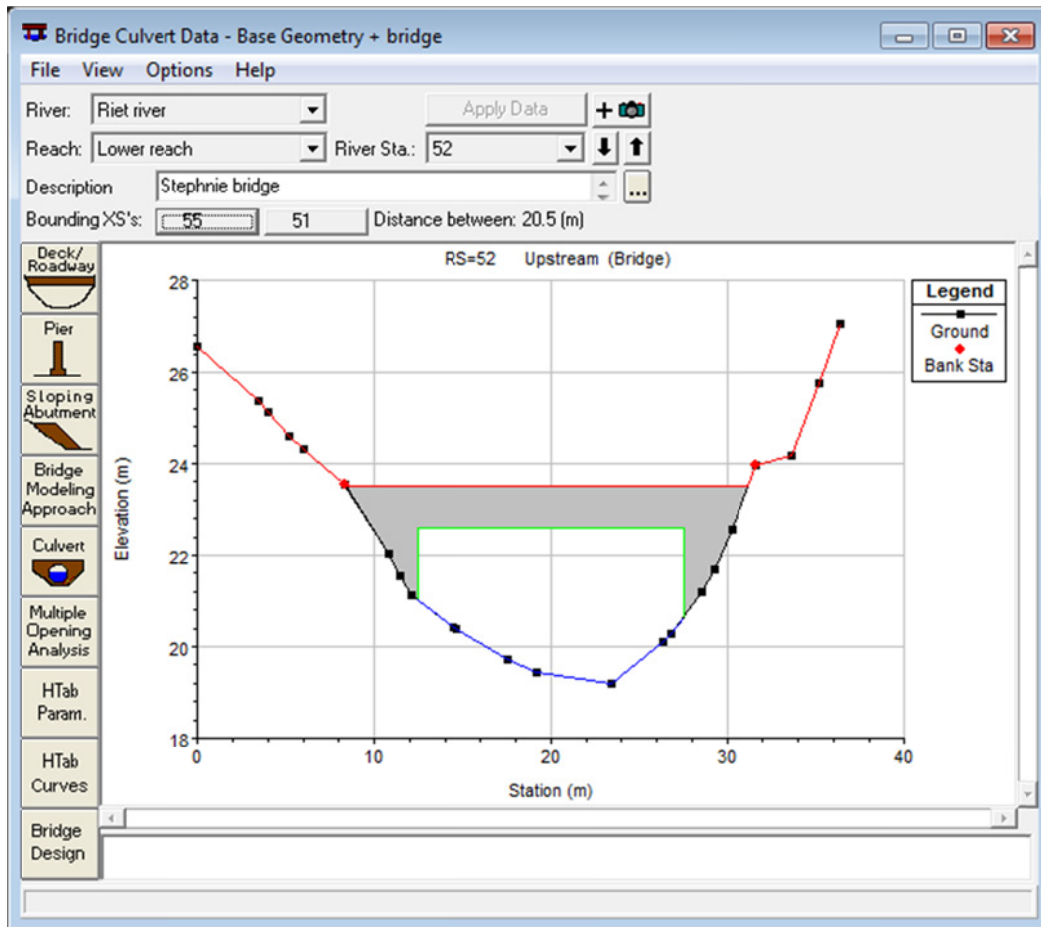


Figure 11.94: Bridge data (View of bridge)

- This bridge has three piers that should be entered. From the **Bridge/Culvert Data** Editor select the **Pier** icon to activate the **Pier Data** Editor as shown in **Figure 11.95**. The three piers are entered by specifying the **Centerline station** at the upstream side as well as the downstream side. The first pier is positioned at 15 m. It has a width of 0,5 m and it starts at a level below the ground profile and ends at a level inside the bridge defined cords i.e. 20 and 23 m.
- Click on the **Add** button to add a pier (Pier #2). The second pier is at centerline 20 m, has a width of 0,5 m and starts at elevation 19 m and ends at elevation 23 m.
- Click on the **Add** button to add a pier (Pier #3). The third pier is at centerline 25 m, has a width of 0,5 m and starts at elevation 19 m and ends at elevation 23 m.
- Click on the **OK** button to return to the **Bridge Culvert Data** editor to view the specified piers (see **Figure 11.96**).

**Pier Data Editor**

Add Copy Delete Pier # 1 ↓ ↑

Del Row Centerline Station Upstream 15

Ins Row Centerline Station Downstream 15

Floating Pier Debris

All On ... All Off ... ☐ Apply floating debris to this pier

Set Wd/Ht for all ... Debris Width:

Debris Height:

	Upstream		Downstream	
	Pier Width	Elevation	Pier Width	Elevation
1	0.5	20.	0.5	20.
2	0.5	23.	0.5	23.
3				
4				
5				
6				

OK Cancel Help Copy Up to Down

Select the Pier to Edit

Figure 11.95: Pier Data Editor

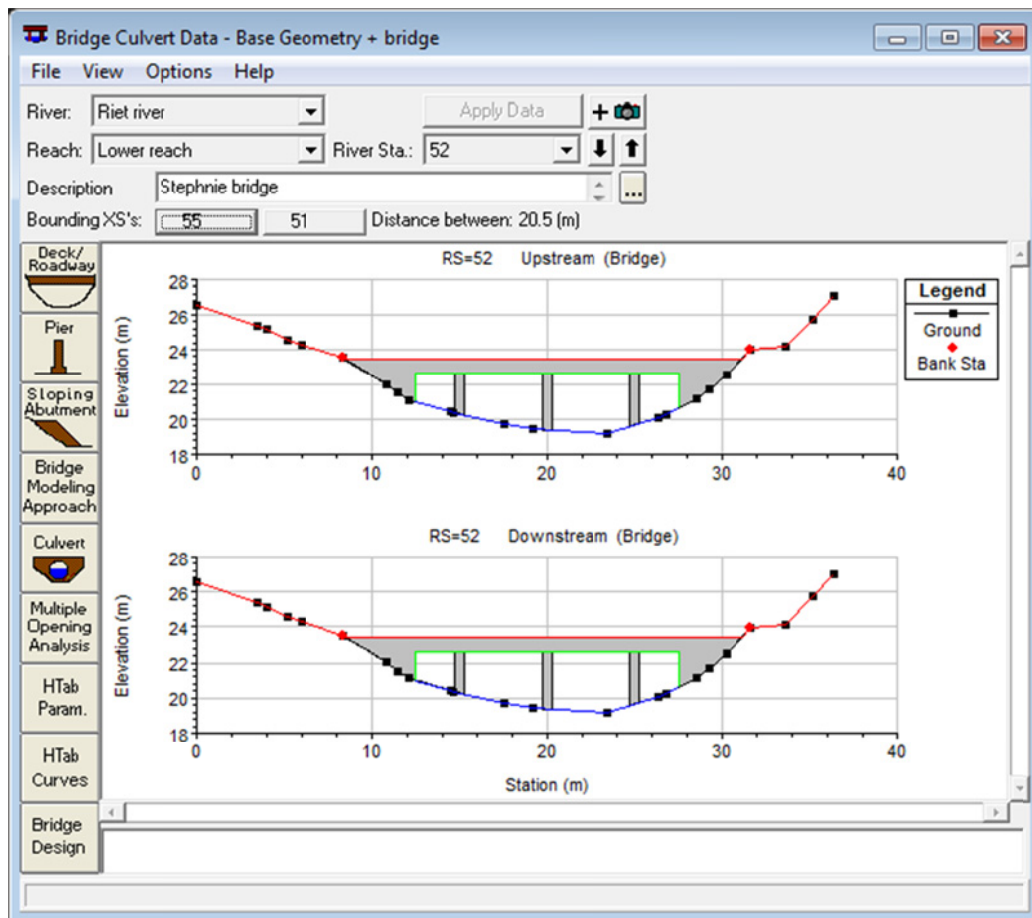

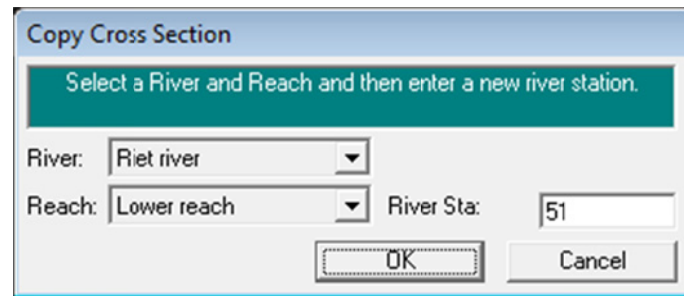


Figure 11.96: Bridge data (View of bridge with piers)

- Another cross section within a short distance downstream of the bridge is also required. This cross section should represent the natural ground where the flow is not affected by the structure (fully expanded). Return to the **Geometric Data** Editor by exiting the **Bridge Culvert Data** window (**Exit** under the **File** menu). Click on the **Cross Section** button,  on the Geometric window (**Figure 11.60**). Go to cross section 55, just upstream of the new bridge (Riet River: Lower reach). Under the **Options** menu, click on **Copy current cross section**. Enter the new River station name, 51, as shown in **Figure 11.97**.



**Figure 11.97: New cross section (River station 51)**

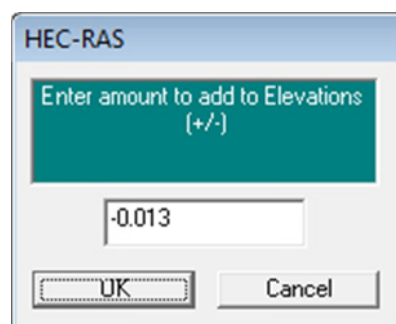
This will position the new cross section downstream of the newly added bridge (which is at cross section 52). The reach lengths should also be corrected. This new cross section is 8 m downstream of the bridge, thus the distance from River station 55 to the bridge is 5 m, the width of the bridge is 7,5 m and this cross section is a further 8 m downstream of the bridge. The total distance from River station 55 to River station 51 is 20,5 m. Go to River station 55 and change all the reach lengths to 20,5 m. Click on the **Apply** button to accept the changes.

Now go to the newly created River station 51 and change the reach lengths (by subtracting the 20,5 m from the previous lengths) to:

Downstream reach lengths: LOB = 6,9 Channel = 10,0 and ROB = 10,0

The total reach lengths between River station 55 and River station 50 will thus still remain the same.

- The average slope between River stations 55 and 50 is 0,000656 m/m. The newly added River station 51 currently has the same elevations as River station 55 (since it is a copy of RS 55). To adjust the elevation of this River station (RS 51) with a value of the distance multiplied with the average slope i.e.  $20,5 \text{ m} \times 0,000656 = \pm 13 \text{ mm}$  click on **Adjust Elevations** under the **Options** menu. Enter an adjustment of -0,013 m (as shown in **Figure 11.98**) and click on the **OK** button.



**Figure 11.98: Adjusting the elevation (River station 51)**

- The new River station 51 will then have the values as shown in **Figure 11.99**.

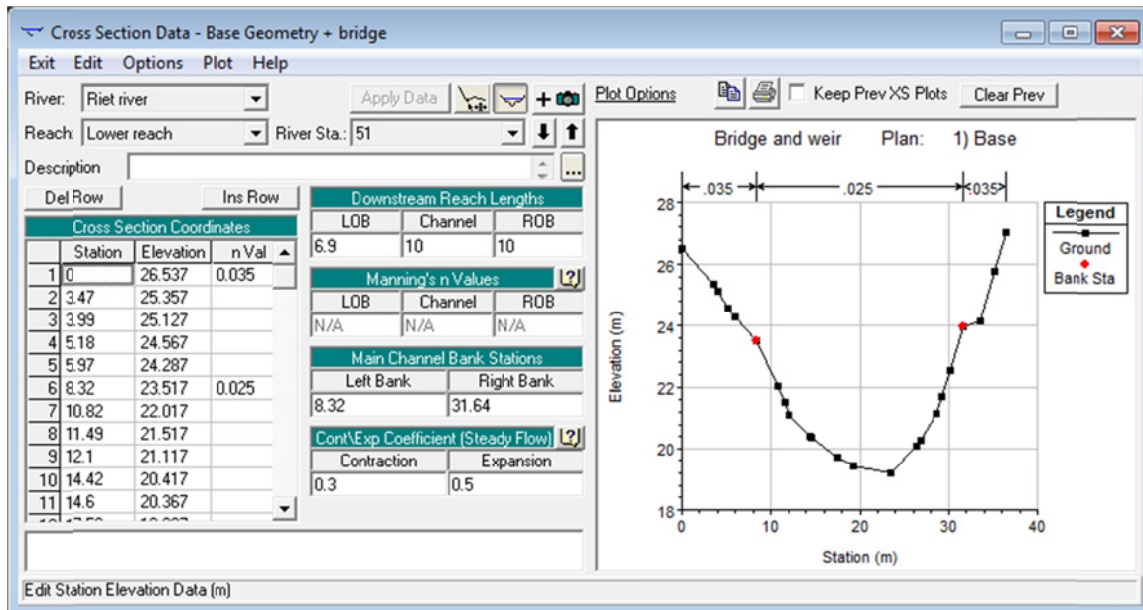


Figure 11.99: River station 51 data

- Before we continue we need to save the Geometric data. This is done by clicking on **Save Geometry Data As** under the **File** menu on the **Geometric Data** window. After selecting this option you will be prompted to enter a Title for the geometric data (Figure 11.100). Enter “*Base Geometry + bridge*” for this exercise, and then press the **OK** button. A file name is automatically assigned to the geometry data based on what you entered for the project file name i.e. Exercise2.g02.

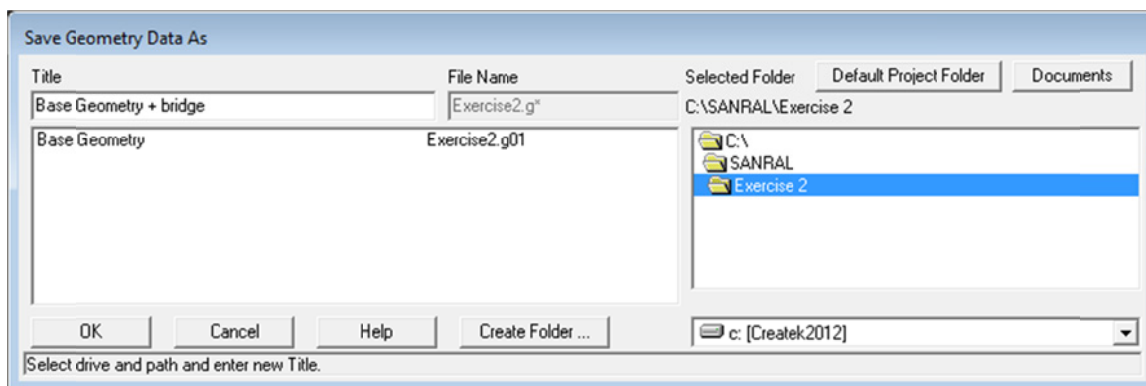
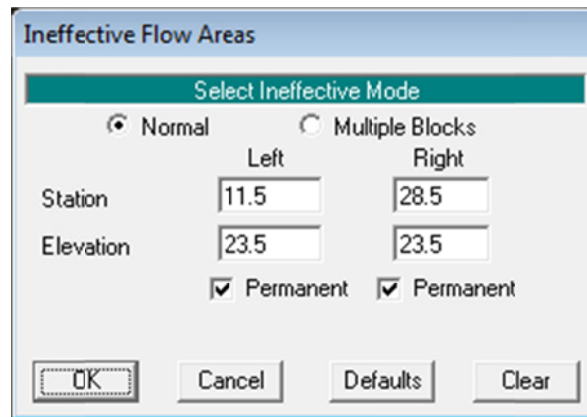


Figure 11.100: Save Geometry Data As

- The next step is to enter the ineffective flow areas. Any ineffective flow areas that exist due to the bridge should be entered. At a bridge ineffective flow areas normally occur just upstream and downstream of the road embankment, away from the bridge opening. It was for this reason that River station 51 was added just downstream of the bridge.
- At River station 55 we need to enter the ineffective flow area by selecting **Ineffective Flow Areas** from the **Options** menu under the **Cross Section Data Editor** window. An initial estimate of the stationing of the ineffective flow areas, a ratio of 1:1 of distance from the bridge to the cross section was used (as an example). In this exercise the upstream cross section is 5 m upstream of the bridge. Therefore, the left and right ineffective flow areas should be set at 5 m left and right of the bridge opening. This however indicates that no ineffective flow area has to be specified since this will already be within the natural profile.



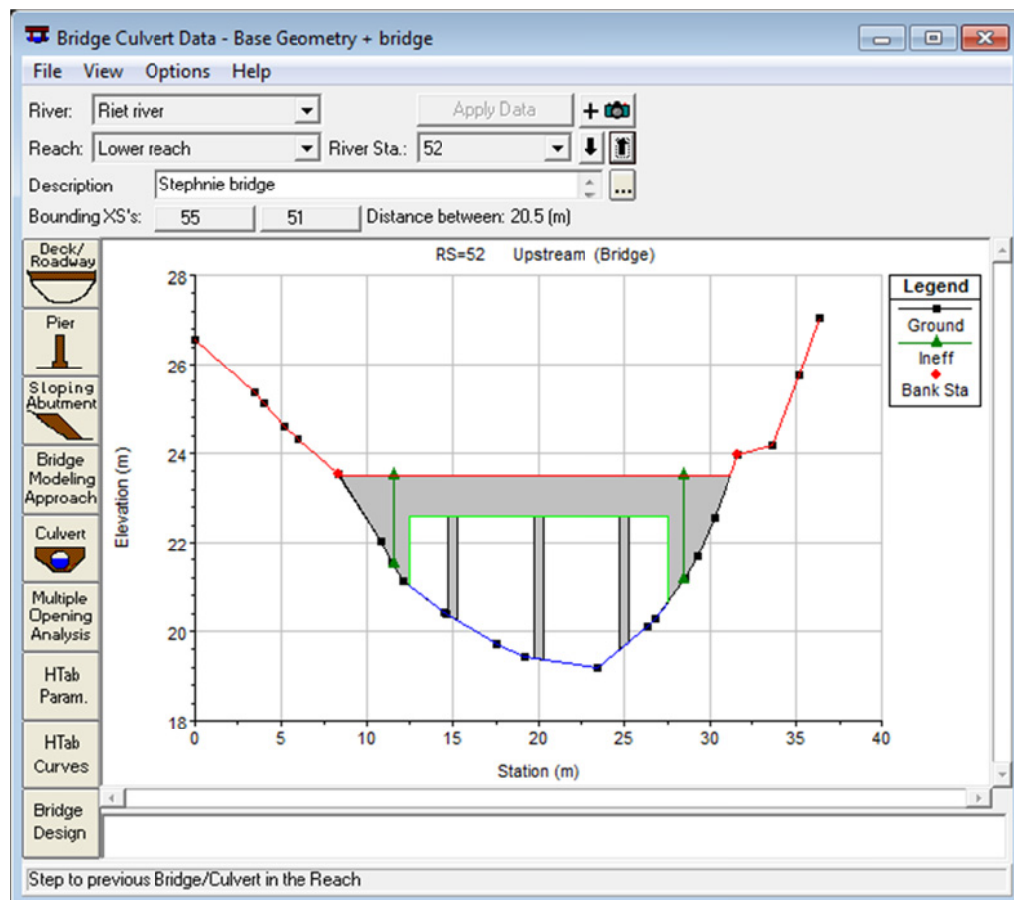
As an example the ineffective flow areas for this exercise were specified at 1 m left and right of the bridge opening at the upstream river station (RS 55), see **Figure 11.101**. The bridge opening starts at station 12,5 m and ends at 27,5 m. These will both be permanent ineffective areas up to the level of the bridge deck.



The dialog box titled "Ineffective Flow Areas" contains a "Select Ineffective Mode" section with two radio buttons: "Normal" (selected) and "Multiple Blocks". Below this, there are two columns for "Left" and "Right" side specifications. Each column has input fields for "Station" and "Elevation", and a checked "Permanent" checkbox. The "Left" side values are Station: 11.5, Elevation: 23.5. The "Right" side values are Station: 28.5, Elevation: 23.5. At the bottom are buttons for "OK", "Cancel", "Defaults", and "Clear".

**Figure 11.101: Ineffective flow areas for river station 55**

- No ineffective flow areas were specified for the newly created downstream river station 51. The ineffective flow areas could also be set by clicking on the **Bounding XS's 55** button on the **Bridge Culvert Data Editor** screen (see **Figure 11.96**).
- The entered bridge should now look similar to that shown in **Figure 11.102**.



**Figure 11.102: Bridge data (View of bridge with piers and ineffective flow areas)**



- The contraction and expansion coefficients are used by the program to determine the transition energy losses between two adjacent cross sections. Typical bridge contraction and expansion coefficients are 0,3 and 0,5 respectively. Select **Contraction\Expansion Coefficients (Steady Flow)** from the **Tables** menu on the **Geometric Data** Editor window and change the contraction and expansion coefficients as indicated in **Figure 11.103**. These coefficients could also be changed by clicking on the **Cross Section Data** editor and changing the cross sections coefficients individually there. Please note these coefficients shown in **Figure 11.103** are for the Steady Flow analysis only. Similarly coefficients can be entered for an Unsteady Flow Analysis by selecting **Contraction\Expansion Coefficients (Unsteady Flow)** from the **Tables** menu on the **Geometric Data** Editor window.

**Edit Contraction/Expansion Coefficients (Steady Flow)**

River: **Riet river** ☐ Edit Interpolated XS's

Reach: **Lower reach**

Selected Area Edit Options:

	River Station	Contraction	Expansion
1	65	0.1	0.3
2	60	0.1	0.3
3	55	0.3	0.5
4	52	Bridge	
5	51	0.3	0.5
6	50	0.1	0.3
7	45	0.1	0.3
8	40	0.1	0.3

**Figure 11.103: Contraction and Expansion coefficients**

- Bridge Modeling Approach**

The bridge routines allow the modeler to analyse the bridge flows by using different methods with the same geometry. The different methods are: low flow, high flow and combination flow.

From the **Geometric Data** Editor, select the **Bridge/Culvert** icon and then the **Bridge Modeling Approach** button. This will activate the **Bridge Modeling Approach** Editor as shown in **Figure 11.104**.

- For this exercise select the energy, momentum and Yarnell equations.

The Energy equation method considers the bridge as just being part of the natural channel and requires Manning's "n" values for the friction losses through the bridge and coefficients of contraction and expansion.

The Momentum Balance method performs a momentum balance through the bridge area and requires the selection of drag coefficient,  $C_d$ . Set the drag coefficient to 2,0 (Square nose piers).

Yarnell Class A flows exists when the water surface through the bridge is completely subcritical (i.e. above the critical depth). The flow regime without the bridge was subcritical and thus **Yarnell** should also be selected. Enter the Yarnell pier coefficient K as 1,25 (Square nose and tail).

Select the **Highest Energy Answer** option. The program will show the results for the answer for the method with the greatest energy loss as the final solution.

For high flows **Energy Only (Standard Step)** should be selected.

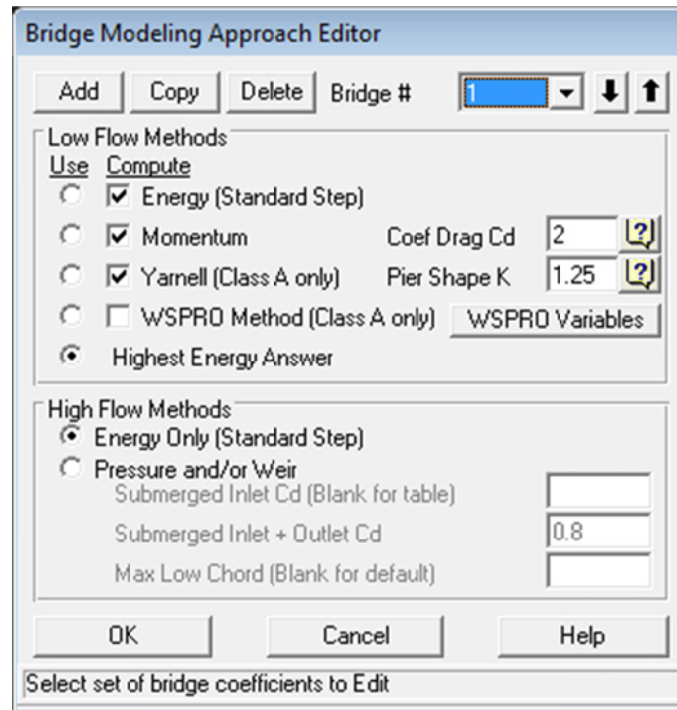



Figure 11.104: Bridge Modeling Approach Editor

- Go to the **Hydraulic Table Parameters Editor** by clicking on the **HTab Param** icon .
- Set the **Head Water Maximum Elevation** value to 24,5 m (see Figure 11.105) and click on the **OK** button.

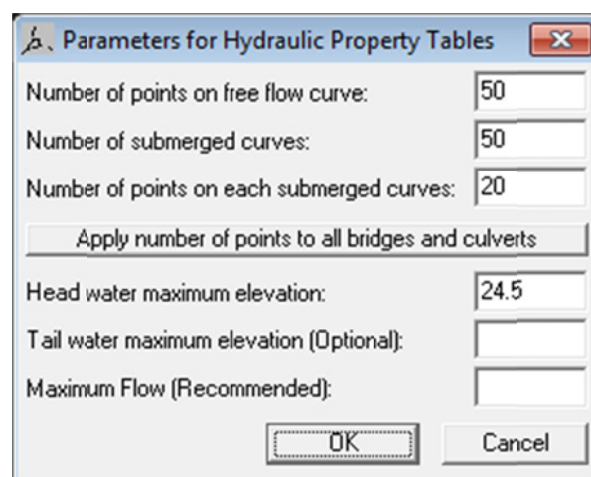



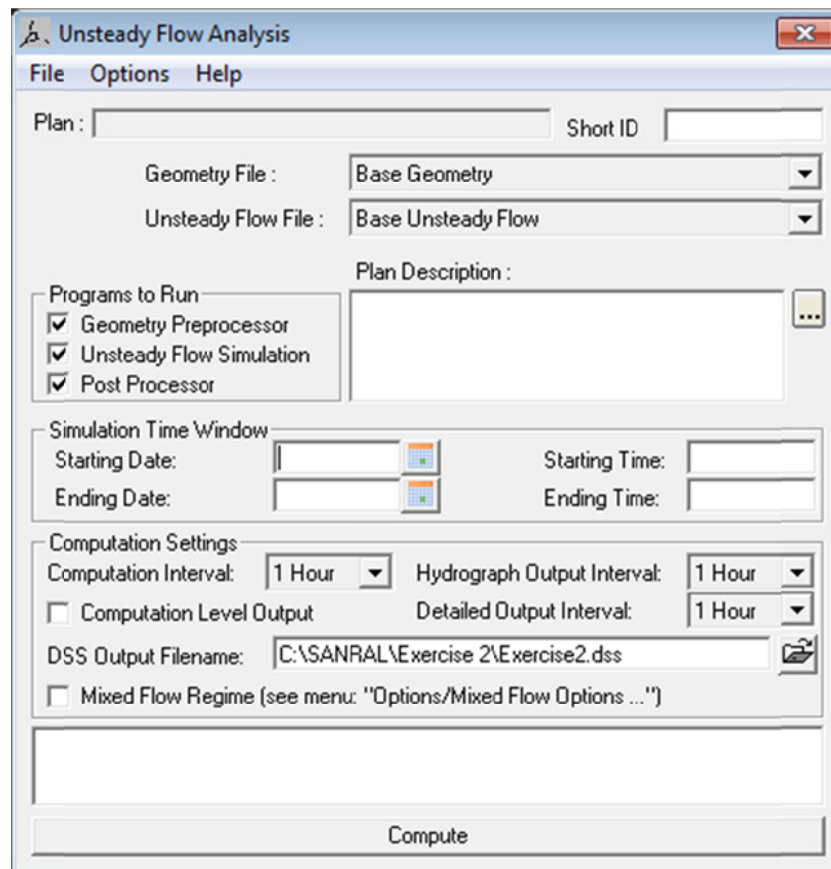
Figure 11.105: Bridge Modeling Approach Editor

- Before we continue we need to save the Geometric data. This is done by clicking on **Save Geometry Data** under the **File** menu on the **Geometric Data** window.

## UNSTEADY FLOW ANALYSIS WITH BRIDGE

### Performing Unsteady Flow Calculations with the Bridge

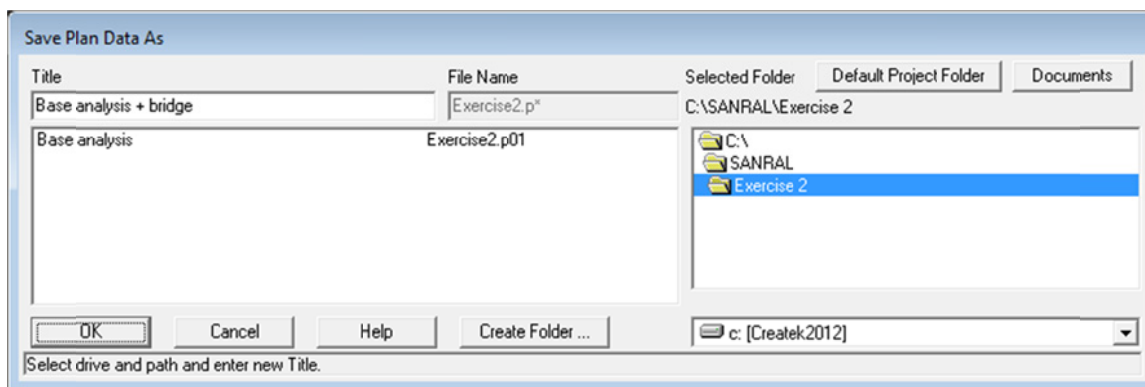
Once all of the geometry and unsteady flow data have been entered, the user can begin performing the unsteady flow calculations. To run the simulation, go to the HEC-RAS main window and select **Unsteady Flow Analysis** from the **Run** menu or click on the **Unsteady Flow Analysis** button  on the menu bar. The **Unsteady Flow Analysis** window will appear as shown in **Figure 11.106**.



**Figure 11.106: Unsteady Flow Analysis**

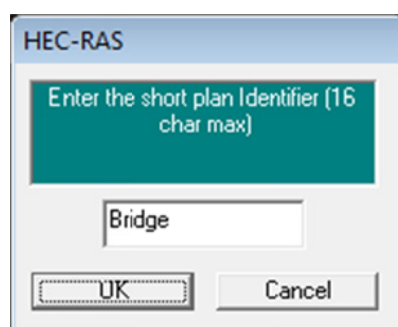
- The first step is to put together a new **Plan**. The **Plan** defines which geometry and flow data are to be used, as well as providing a title and short identifier for the run.

To establish a plan, select **New Plan** from the **File** menu on the **Unsteady Flow Analysis** window. Enter the plan title as *Base analysis + bridge* and then press the **OK** button (**Figure 11.107**).



**Figure 11.107: Creating new plan**

- You will be prompted to enter a short identifier. Enter a title of *Bridge* in the **Short ID** box (Figure 11.108) and click on the **OK** button.



**Figure 11.108: Plan identifier**

- Select the correct Geometry file and Unsteady flow file from the drop down list.
- **Selecting Programs to Run**  
There are three components used in performing an unsteady flow analysis within HEC-RAS. These components are: a geometric data pre-processor; the unsteady flow simulator; and an output post-processor (see Figure 11.106).
- **Simulation time window**  
The user is required to enter a time window that defines the start and end of the simulation period. The time window requires a starting date and time and an ending date and time.

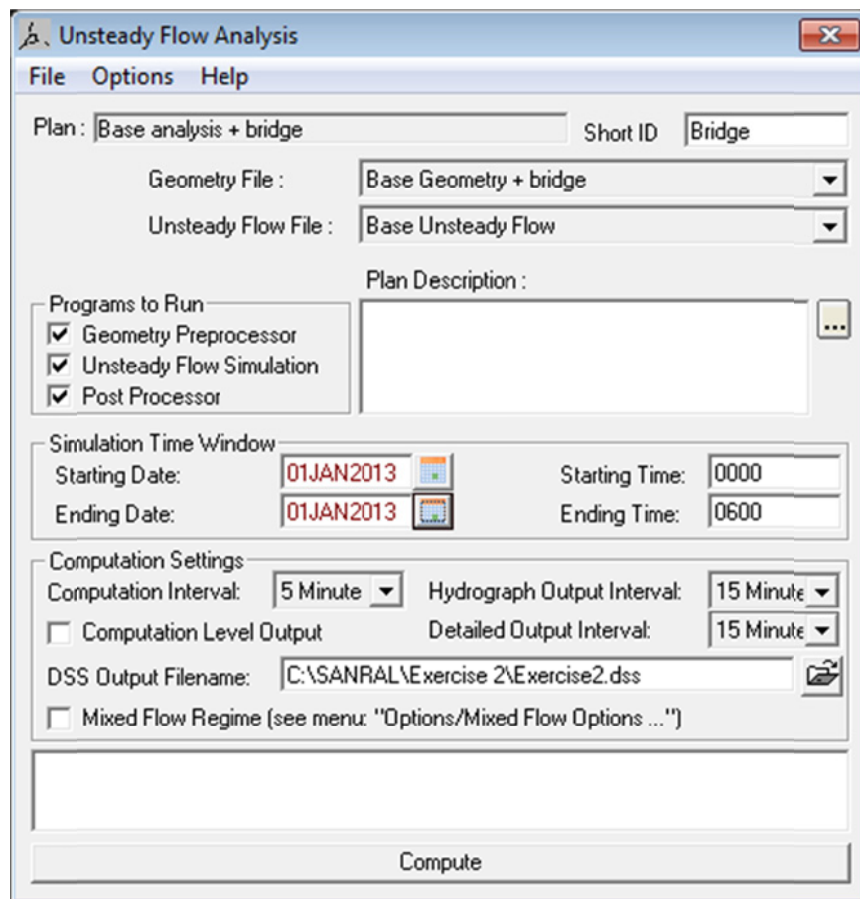
In this exercise the flow hydrograph shown in Figure 11.73 and Figure 11.75 starts at 0:00 and has data until 06:00 and this is used in the simulation time window. The date can be anything since the option to **Use Simulation Time** on the Flow Hydrograph window was selected (see Figure 11.73 and Figure 11.75). Enter the **Starting date** as *01JAN2013* and the **Ending date** as *01JAN2013*. Enter the **Starting time** as *0000* and the **Ending time** as *0600*.

- **Computational settings**  
The computational Settings area contains the **Computational Interval**, **Hydrograph Output Interval**, **Detailed Output Interval**, the name and path of the output DSS file, and whether or not the program is run in a mixed flow regime. The computation interval is probably one of the most important parameters entered into the model. This should be small enough to accurately describe the rise and fall of the hydrographs being routed but not small to take forever to compute.

For this exercise set the **Computational Interval** at *5 minutes* (from the drop down list). Set the **Hydrograph Output Interval** to *15 minutes* and the **Detailed Output Interval** also at *15 minutes*.

The **DSS Output filename** is the file that contains all the calculated data in a format that can be read by HEC-RAS and used in displaying all the results (tables and graphs). The default will be .....*Exercise2.dss* and does not have to be changed for this exercise.

The completed **Unsteady Flow Analysis** screen is shown in **Figure 11.109**.



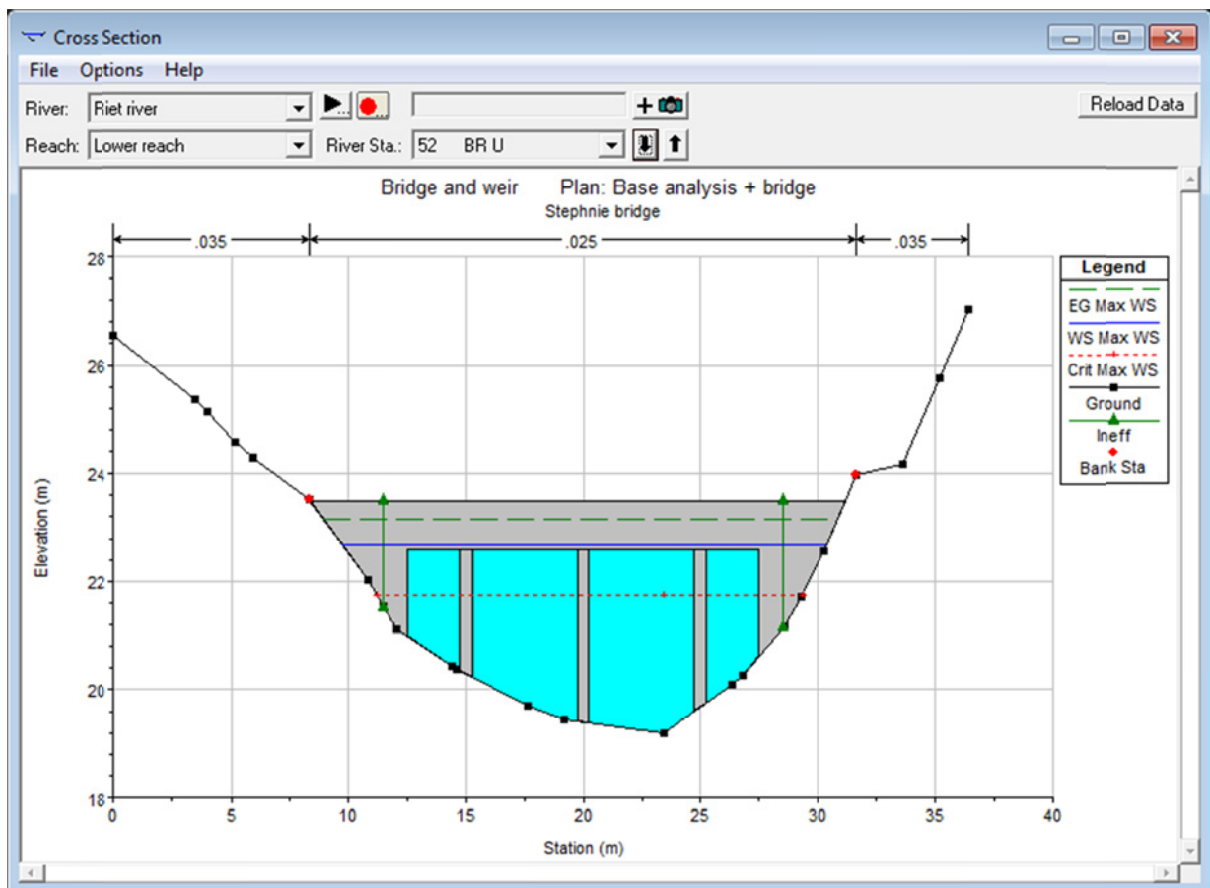
**Figure 11.109: Unsteady Flow Analysis (completed)**

- Click on the **Compute** button to run the **Unsteady Flow Analysis**.



## VIEWING THE RESULTS

Once the model has finished all of the computations successfully, you can begin viewing the results. Several output options are available from the **View** menu bar on the HEC-RAS main window. These options include:

- a) Cross section plots
  - b) Profile plots
  - c) General profile plot
  - d) Rating curves
  - e) X-Y-Z perspective plots
  - f) Detailed tabular output at a specific cross section (cross section table)
  - g) Limited tabular output at many cross sections (profile table)
- Begin by plotting a cross section. Select **Cross Sections** from the **View** menu bar on the HEC-RAS main window. Any cross section can be plotted by selecting the appropriate river, reach and river station (See **Figure 11.110**). Several plotting features are available from the **Options** menu bar on the cross section plot window. These options include: zoom in; zoom out; selecting which plans, profiles, variables to plot; and control over lines, labels, symbols, scaling etc.



**Figure 11.110: Cross section view (Riet River: Lower reach – River station 52 (upstream of bridge))**

- Next plot a water surface profile. Select **Water Surface Profiles** from the **View** menu bar. Click on the Play button, , to view the **Animation Control** window (see **Figure 11.111**). Click on the **Expand** button, , to see the entire control.

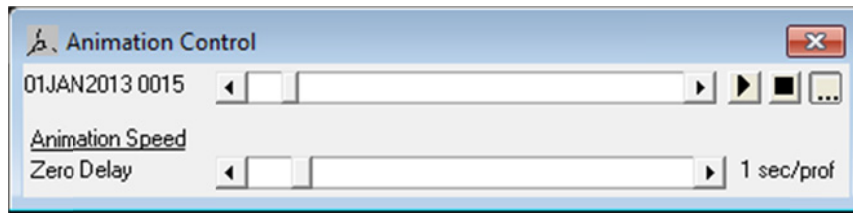



Figure 11.111: Animation Control

- Set the **Zero Delay** horizontal scroll bar as indicated (in Figure 11.111) and click on the play button, , to view the changing water surface levels and energy grade lines of the river reach profile plot. This should give you a profile plot as shown in Figure 11.112 for the maximum water surface.

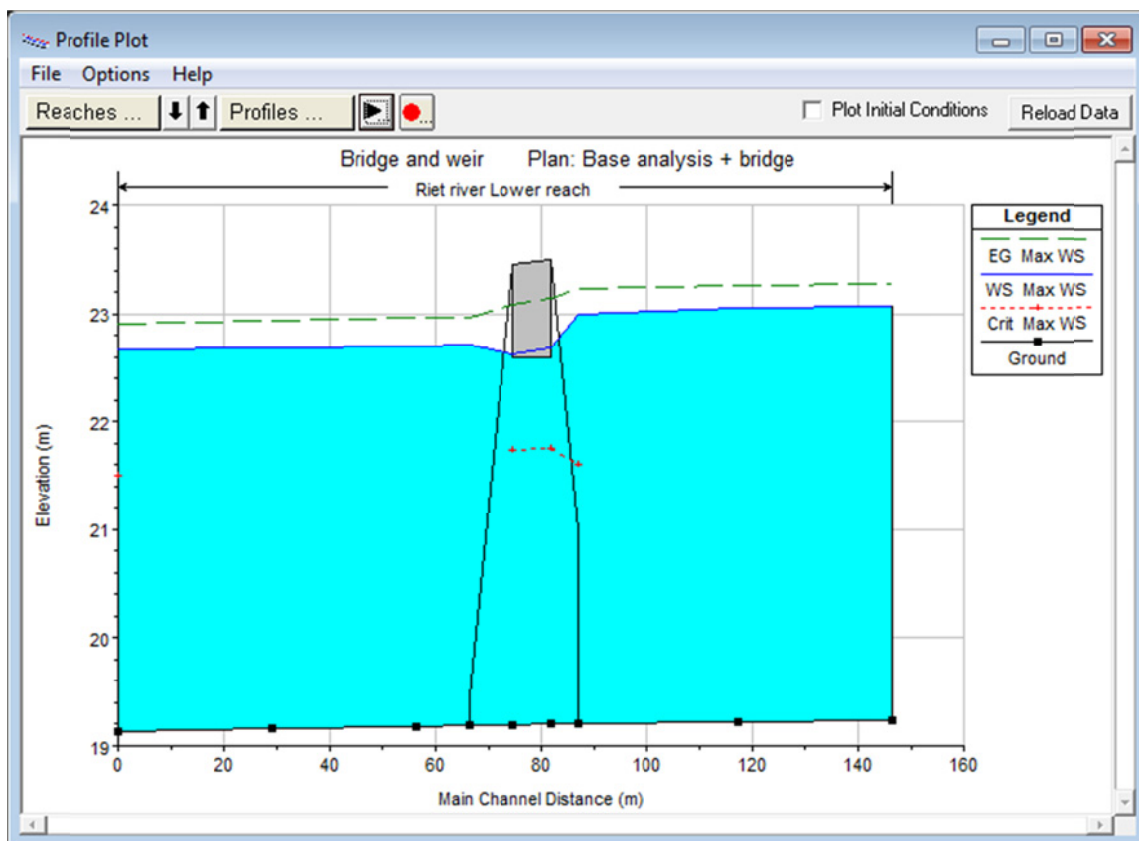
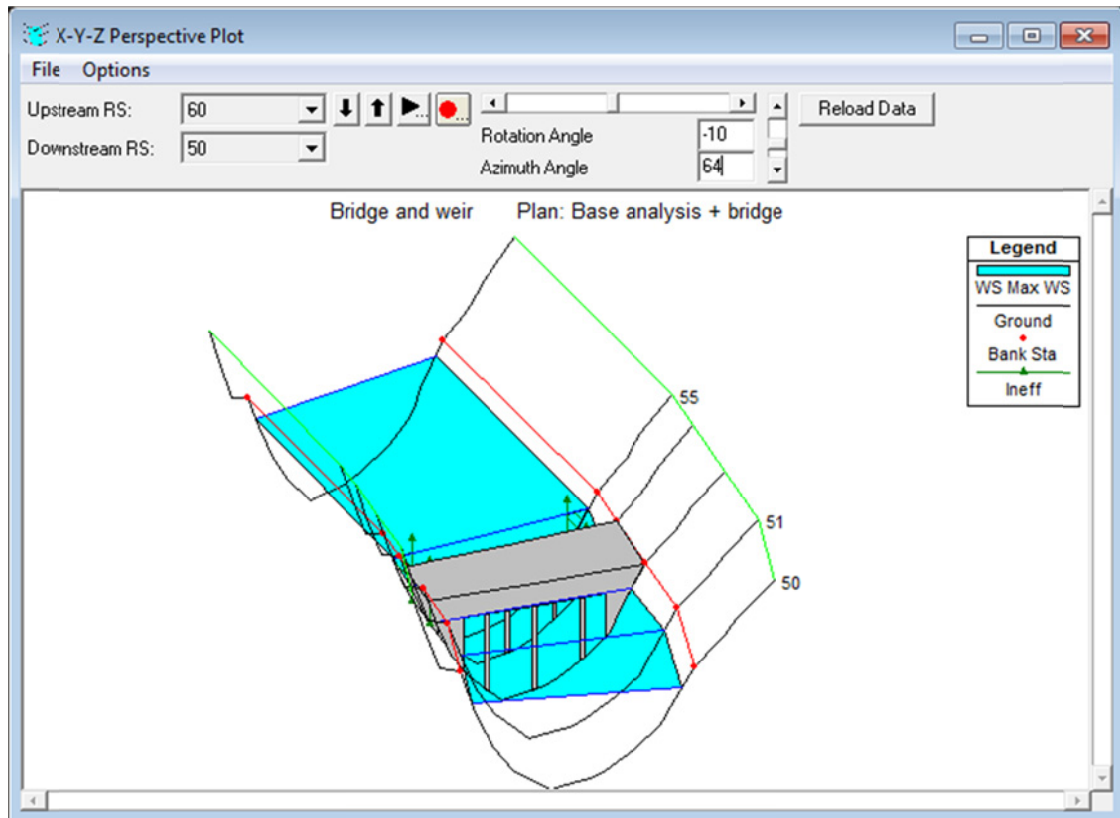


Figure 11.112: Water Surface Profile (for Riet River: Lower reach- Maximum WS)

- Also have a look at a **General Profile Plot** and the **X-Y-Z Perspective Plot** (Figure 11.113). Also look at some tabular output. Go to the **View** menu bar on the HEC-RAS main window. There are two types of tables available, a detailed output table and a profile summary table. Select **Detailed Output Tables** to get the first table to appear. This table shows detailed hydraulic information at the bridge. Other cross sections can be viewed by selecting the appropriate reach and river from the table. A table with all the errors, warnings and comments can also be viewed, by selecting **Summary, Err Warn, Notes...** from the **View** menu on the HEC-RAS main window.





**Figure 11.113: X-Y-Z Perspective plot (Riet river: Lower reach, selected stations)**

You will now be in a position to answer some more of the questions at the end of this exercise (Questions 6 to 10).

### **ADDING AN INLINE STRUCTURE (WEIR)**

In the next section an inline weir will be added in the Blesbok River – Lower reach.

#### **Entering weir data**

- To enter inline weir data click on the **Inline structure** button on the **Geometric data** window (**Figure 11.58**). Once the **Inline structure** button is pressed, the **Inline structure Data** Editor will appear as shown in **Figure 11.114**.

To add the inline weir in the model, take the following steps:

- Select the river and reach that you would like to place the weir in (from the drop down lists) i.e. *Blesbok River* and *Lower reach*.
- From the **Options** menu, select **Add an Inline structure** from the list. An input box will appear prompting you to enter a river station identifier for the inline structure. Enter *6.5* as shown in **Figure 11.115**.

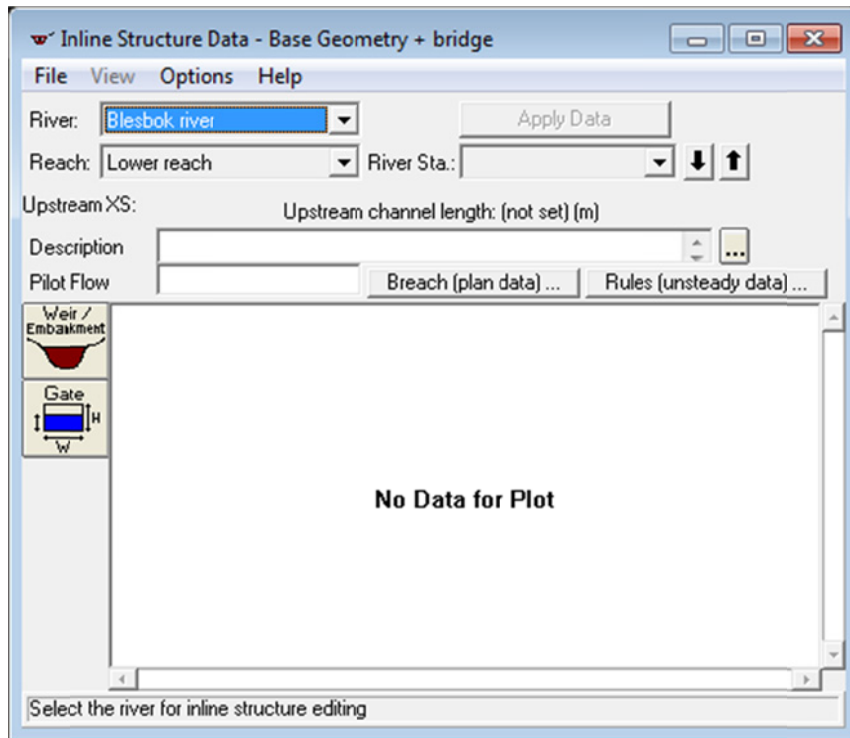


Figure 11.114: Inline weir Data window

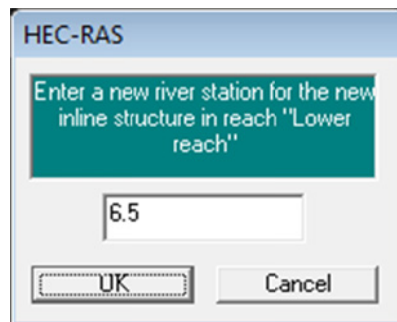


Figure 11.115: Inline weir river station (Blesbok River: Lower reach)

- Enter the **Description** of the inline structure: *Maya weir* (see Figure 11.116).
- Enter all of the required data for the new bridge. This includes:
  - a. Weir/Embankment details
  - b. Gate detail
- From the **Inline Structure Data** editor select the **Weir/Embankment** icon to activate the **Inline Structure Weir Station Elevation Editor** as shown in Figure 11.117.

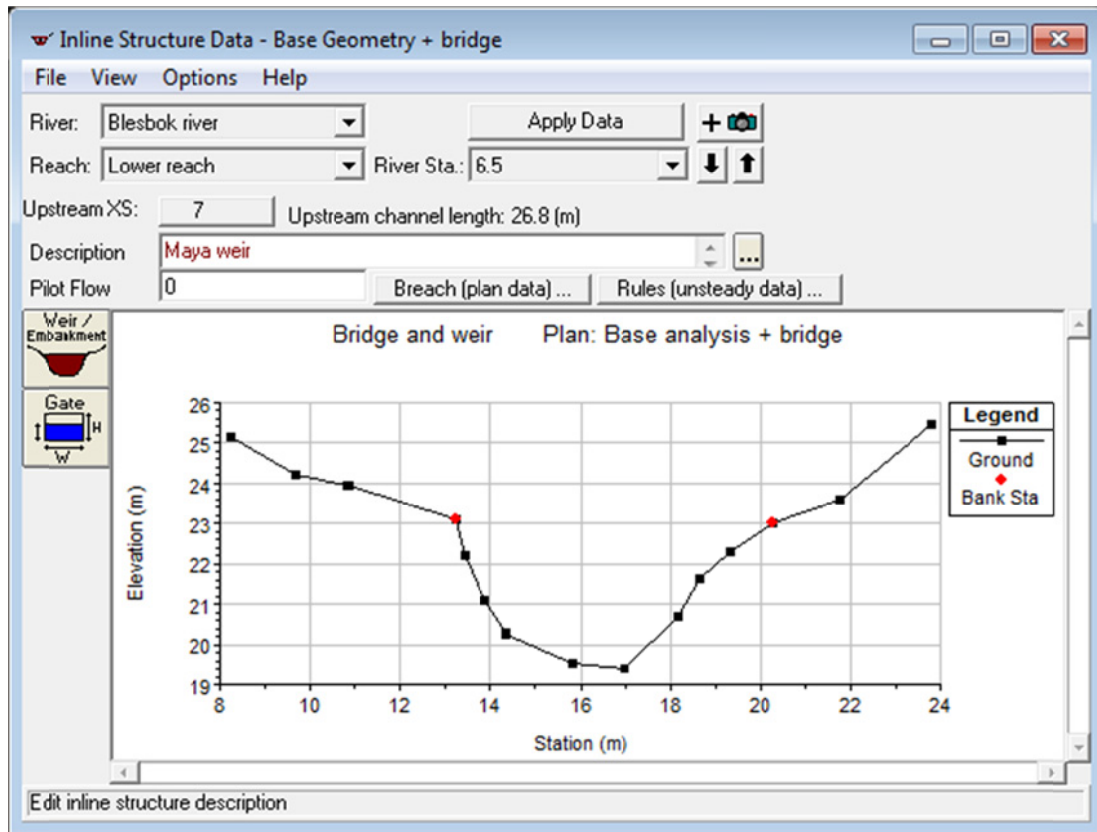


Figure 11.116: View of the cross section at the weir (RS 6.5)

Distance	Width	Weir Coef
10	5	1.44

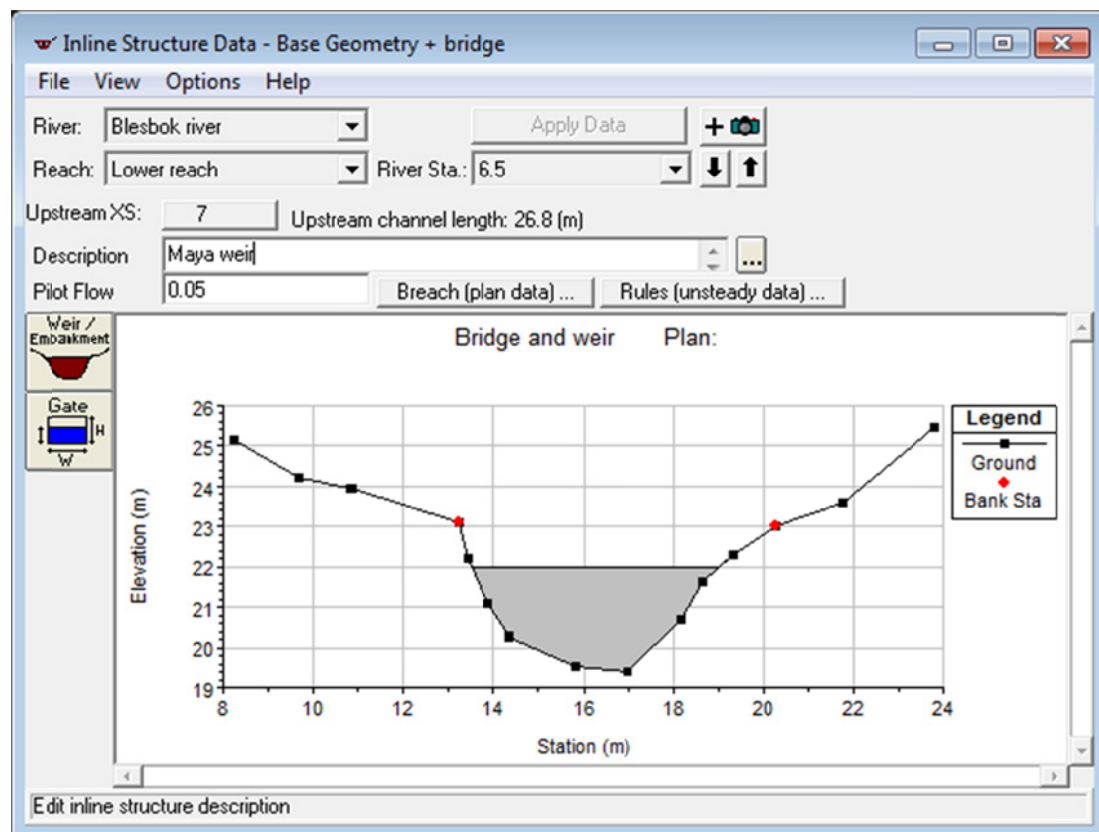
  

Station	Elevation
13	22
20	22
3	
4	
5	
6	
7	
8	

Figure 11.117: Inline weir (Inline Structure Weir Station Elevation Editor)

- The first input at the top of the editor is the distance from the upstream cross section and the deck (i.e. river station 7). This distance is 10 m.

- The deck itself will have a width of 5 m.
- The weir flow coefficient selected for this analysis is  $1,44$ .
- At every station position enter station and elevation of the top of the weir embankment shown in **Figure 11.117** to provide the weir as shown in **Figure 11.118**.
- The US and DS Embankment SS (upstream and downstream embankment side slope) values should be entered as  $1$  (1 horizontal to 1 vertical). These values are used for the graphical representation on the profile plot.
- The last field at the bottom of the editor is the selection of the **Weir Crest Shape**. This selection will determine the reduction of the weir flow coefficient due to submergence. For this exercise, a **Broad Crested** weir shape should be selected.
- Click the **OK** button.



**Figure 11.118: Inline weir (Viewing the weir)**

- Enter the **Pilot flow** of 50 l/s i.e.  $0,05 \text{ m}^3/\text{s}$  (Pilot discharge for leakage or to keep the downstream channel wet at low flows) (see **Figure 11.118**).

## Gated Spillway Data

In addition to uncontrolled overflow weirs, the user can add gated spillways (this is optional). To add gated spillways to the structure, press the **Gate** button on the **Inline Structure Data** editor. Once this button is pressed, the gated editor will appear as shown in **Figure 11.119** (except yours will be blank until you have entered some data).

Centerline Stations	
	Station
1	16.25
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	

**Figure 11.119: Gated Spillway Editor**

The **Gated Spillway** editor is similar to the Culvert editor in concept. The user enters the physical description of the gates, as well as the required coefficients, in the Gated Spillway editor. The functionality of the gates is defined as part of the Unsteady Flow data. The following is a list of the data contained on this editor:

- **Gate Group** - The **Gate Group** is automatically assigned to "Gate #1" the first time you open the editor. The user can enter up to 10 different Gate Groups at each particular river crossing, and each gate group can have up to 25 identical gate openings. If all of the gate openings are exactly the same, then only one gate group needs to be entered. If the user has gate openings that are different in shape, size, elevation, or have different coefficients, then additional Gate Groups must be added for each Gate type.
- **Height** - This field is used to enter the maximum possible height that the gate can be opened in meters (Enter 1,2 m).
- **Width** - This field is used for entering the width of the gate in meters (Enter 1,2 m).

- **Invert** - This field is used for entering the elevation of the gate invert (sill elevation of the spillway inside of the gate) in meters. (Enter 20 m).
- **Gate Type** - This field is used for selecting the type of gate. A number of different gate types are available. Select from the drop down list *Sluice*.
- **Sluice Discharge Coefficient** - This field is used for entering the coefficient of discharge for the gate opening. This coefficient ranges from 0,5 to 0,7 for sluice gates. For this sluice gate enter 0,6.
- **Orifice Coefficient** - This field is used to enter an orifice coefficient, which will be used for the gate opening when the gate becomes more than 80 percent submerged. Between 67 percent and 80 percent submerged, the program uses a transition between the fully submerged orifice equation and the free flow equations. When the flow is less than 67 percent submerged, the program uses the free flow gate equations. Enter an **Orifice Coefficient** of 0,8.
- **Head Reference** – This field is used to select the reference point for which the upstream energy head will be computed from. The default is the gate sill (invert), which is normally used when the flow through the gate goes out into a channel. If the gate causes the flow to jet out freely into the atmosphere, then the head reference should be selected as the centerline elevation of the gate opening. If the gate crest is an ogee spillway crest, then the center of the gate opening should be used. Ogee spillway crests are normally designed to follow the shape of water jetting freely into the atmosphere. For this exercise select from the drop down list *Sill (invert)*.
- **Weir Shape** - This parameter allows the user to select between a Broad Crested, Sharp Crested shape weir and an Ogee shaped weir. Select *Broad Crested*.
- **Weir Coefficient** - This field is used for entering a weir coefficient that will be used for the gate opening. This coefficient will only be used when the gate is opened to an elevation higher than the upstream water surface elevation. When this occurs, the flow through the gate is calculated as weir flow. Enter the Weir Coefficient of 1,67.
- **Centerline Stations** - This table is used for entering the centerline stationing of the identical gate openings. The user should enter a different centerline stationing for each gate opening that is part of the current gate group. All gate openings within the same gate group are exactly identical in every way, except their centerline stationing. As a user adds new centerline stationing values, the number of identical gates in the group is automatically incremented and displayed in the field labelled "# Openings". Enter the **Centerline Station** as 16,25 m.
- Once all of the data for the gates has been entered (as shown in **Figure 11.119**), the user needs to press the **OK** button for the data to be accepted. If the user presses the **OK** button, this does not mean that the data is saved to the hard disk; it is only stored in memory and accepted as being good data. This data is part of the geometry data, and is stored in the geometric data file. The data can be stored to the hard disk by selecting from the **File** menu of the **Geometric Data** window **Save Geometry Data As**.
- After selecting this option you will be prompted to enter a Title for the geometric data (**Figure 11.120**). Enter "*Base Geometry + bridge + weir*" for this exercise, and then press the **OK** button. A file name is automatically assigned to the geometry data based on what you entered for the project file name i.e. Exercise2.g03.

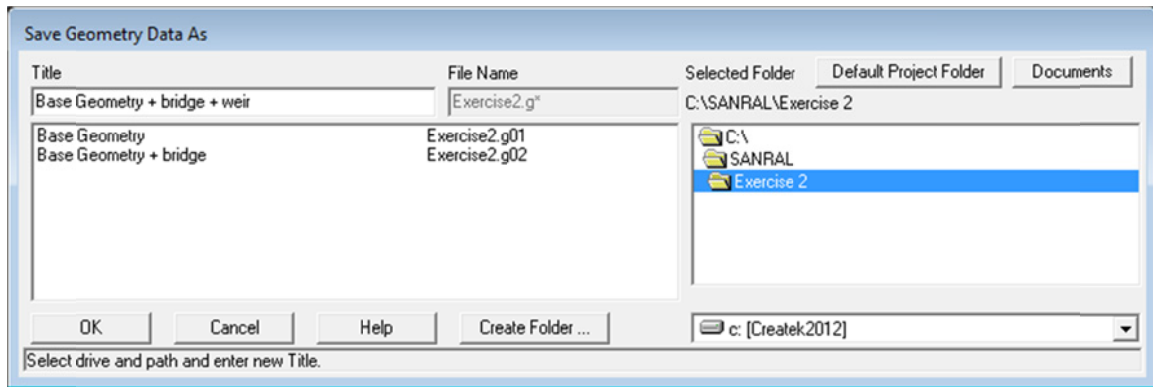


Figure 11.120: Save Geometry Data As

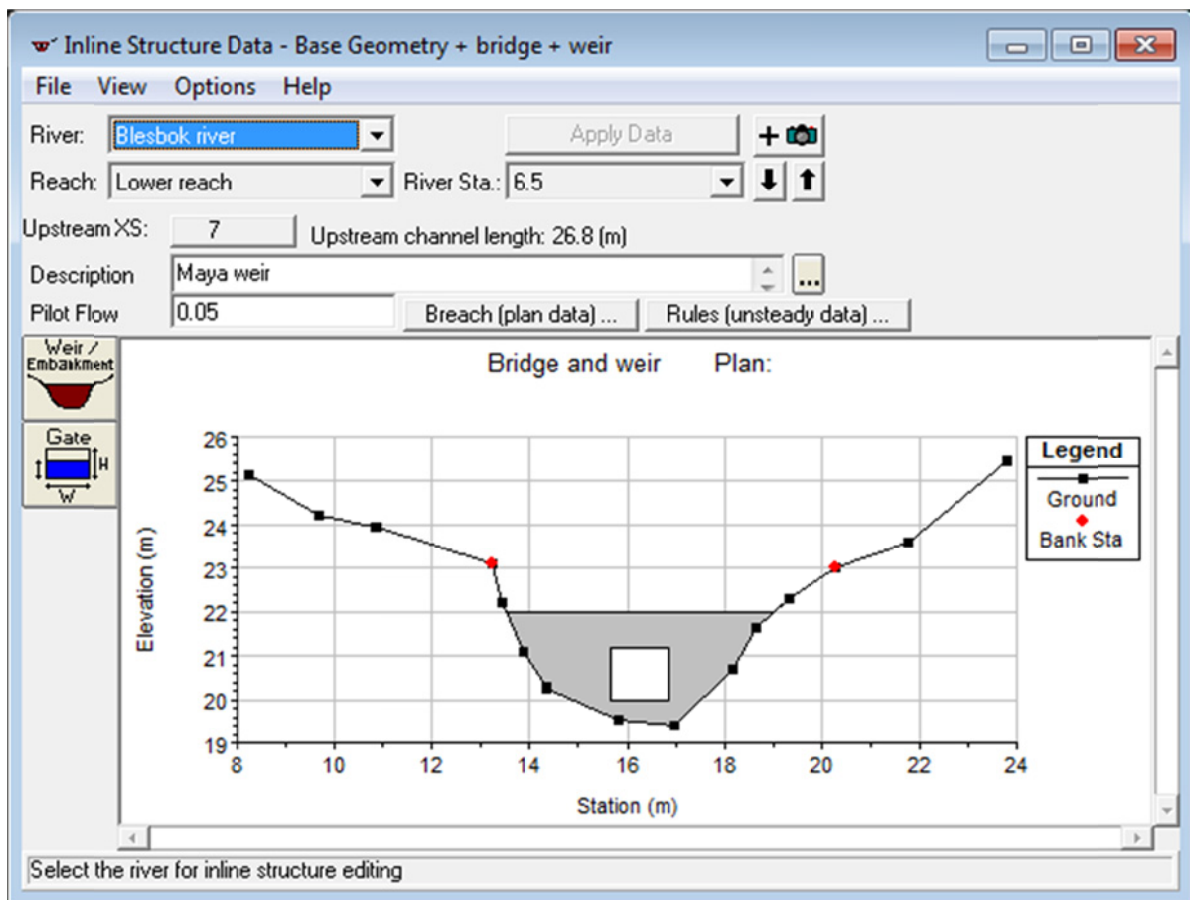


Figure 11.121: Inline weir (Viewing the weir + gate)

- The newly added weir will be shown on the **Geometric Data** editor window as shown in Figure 11.122.



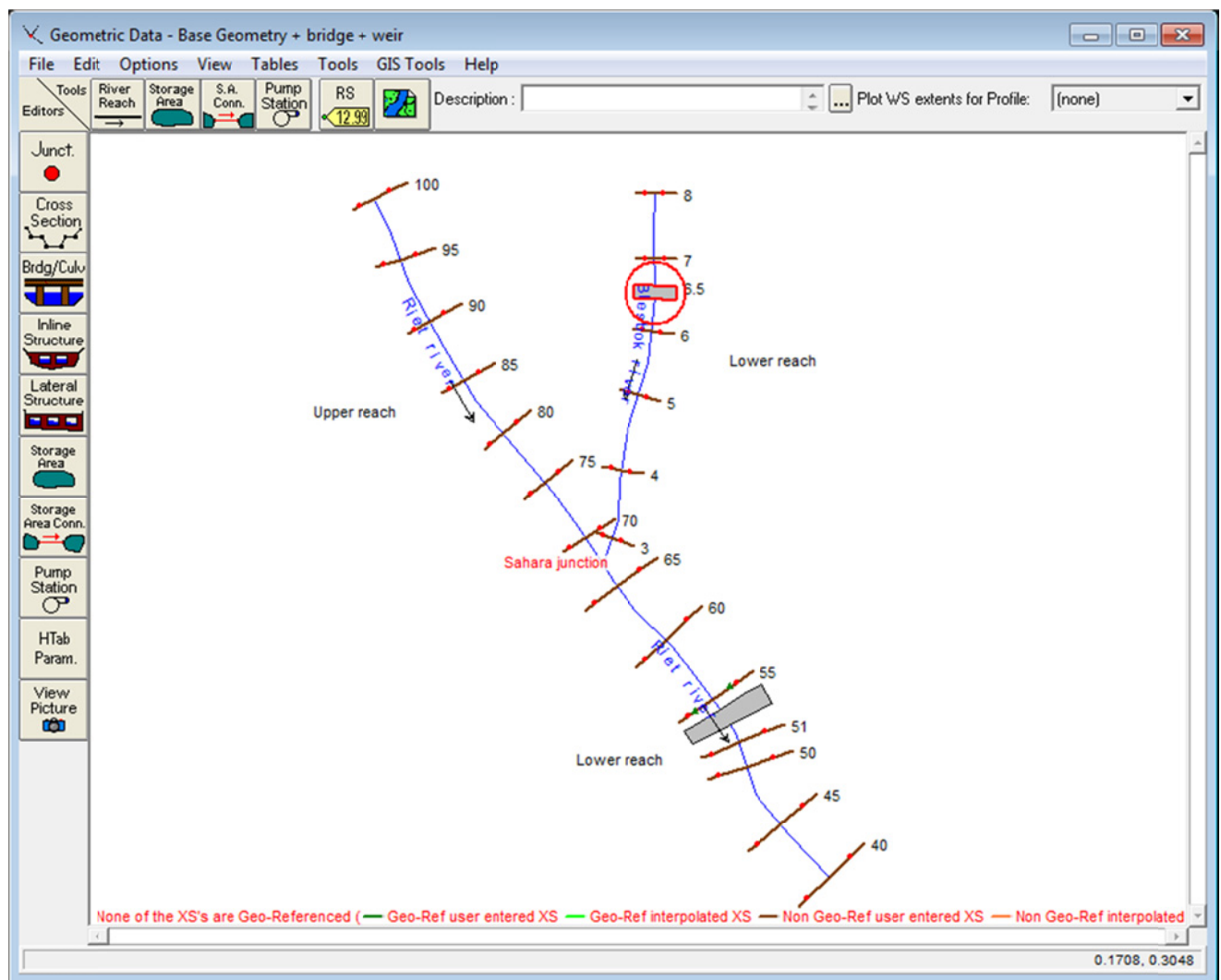

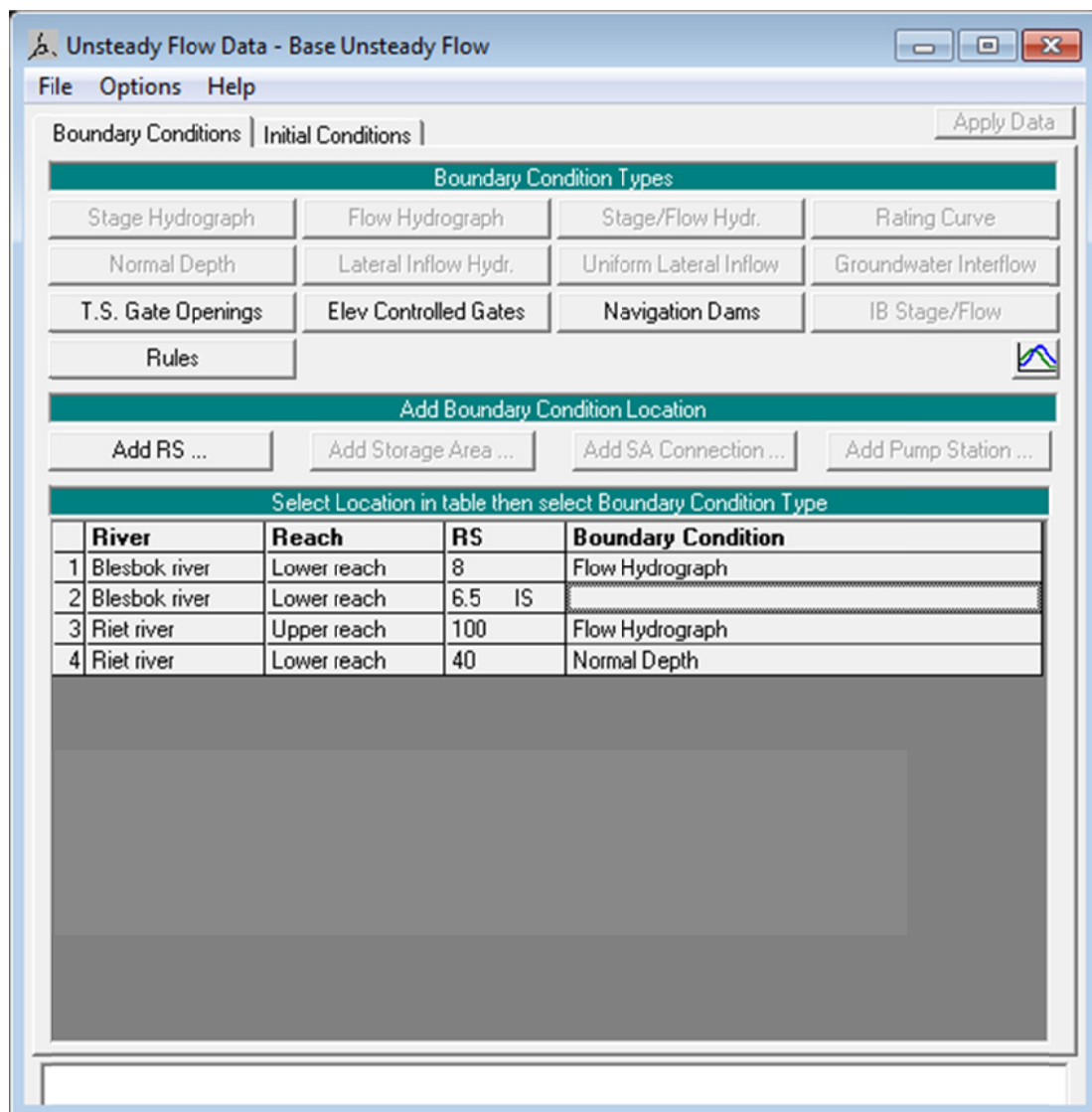


Figure 11.122: Layout of river system (Bridge + weir)

### Entering the gate opening characteristics

Once all of the geometric data are entered, the modeller can then enter the unsteady flow data that is required for the gate. To bring up the unsteady flow data editor, select **Unsteady Flow Data** from the **Edit** menu on the HEC-RAS main window or clicking the short cut button on the menu bar . The **Unsteady Flow** data editor should appear as shown in **Figure 11.123**, for this exercise.



**Figure 11.123: Unsteady Flow Data Editor**

The boundary conditions at all the external boundaries of the system have already been entered. The internal control at the weir will now also be entered.

- If the River station 6.5 IS is not in the list of Boundary Conditions click on the **Add RS** button and add this internal boundary.
- Click in the **Boundary Condition** column next to the River station 6.5 IS. HEC-RAS allows the user to select the type of boundary required from the available buttons: T.S. Gate Opening, Elev Controlled Gates, Navigation Dams or Rules.
- For this exercise the gates will be controlled, opening and closing by means of the water surface elevation. With the Boundary Condition Type block highlighted click on the **Elev Controlled Gates** button. This will show the **Elevation Controlled Gates** data editor as shown in **Figure 11.124**.

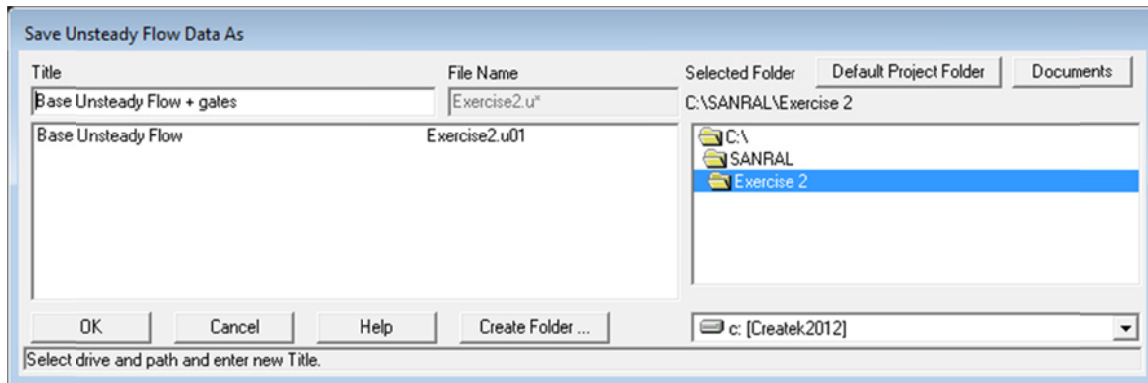
Figure 11.124: Elevation Controlled Gates data editor

- This option allows the user to control the opening and closing of gates based on the elevation of the water surface upstream of the structure. A gate begins to open when a user specified elevation is exceeded in this case 23,5 m (see **Figure 11.125**). The gate will begin to close again when the water surface elevation reaches 22,5 m.
- The gate opens at a rate specified by the user (0,1 m/min).
- The closing of the gate is at a user specified rate (also 0,1 m/min.).
- The user must also enter a maximum and minimum gate opening. For this exercise the **Maximum Gate Opening** is set at 1,1 m and the **Minimum Gate Opening** is set at 0 m.
- The **Initial gate opening** is closed and thus this value must be entered as 0. **Figure 11.125** shows the completed data for this gate.

Figure 11.125: Elevation Controlled Gates data editor

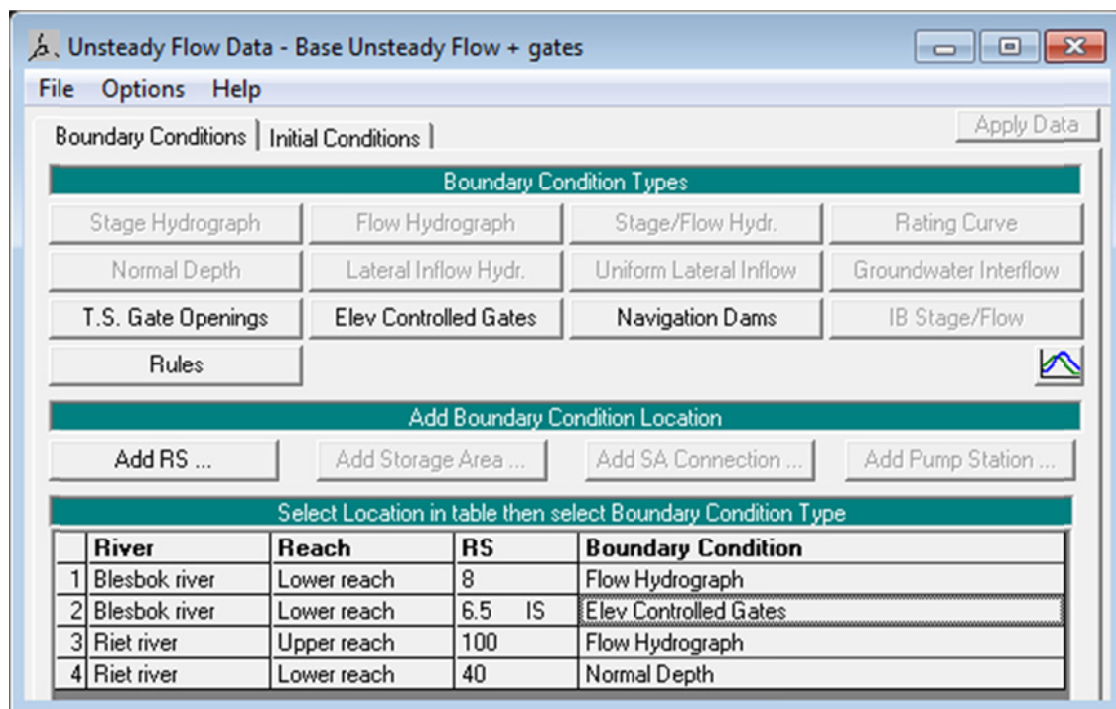
## Saving the Unsteady Flow Data

- The last step in developing the unsteady flow data is to save the information to a file. To save the data, select the **Save Unsteady Flow Data As** from the **File** menu on the **Unsteady Flow Data** editor. A pop-up window will appear prompting you to enter a title for the data as shown in **Figure 11.126**. Enter “*Base Unsteady Flow + gates*” for this exercise, and then press the **OK** button. A file name is automatically assigned to the Unsteady Flow Data based on what you entered for the project file name i.e. Exercise2.u02.



**Figure 11.126: Saving Unsteady Flow Data**

The completed **Unsteady Flow Data** screen should now include the additional boundary at the inline structure (RS 6.5) as shown in **Figure 11.127**.




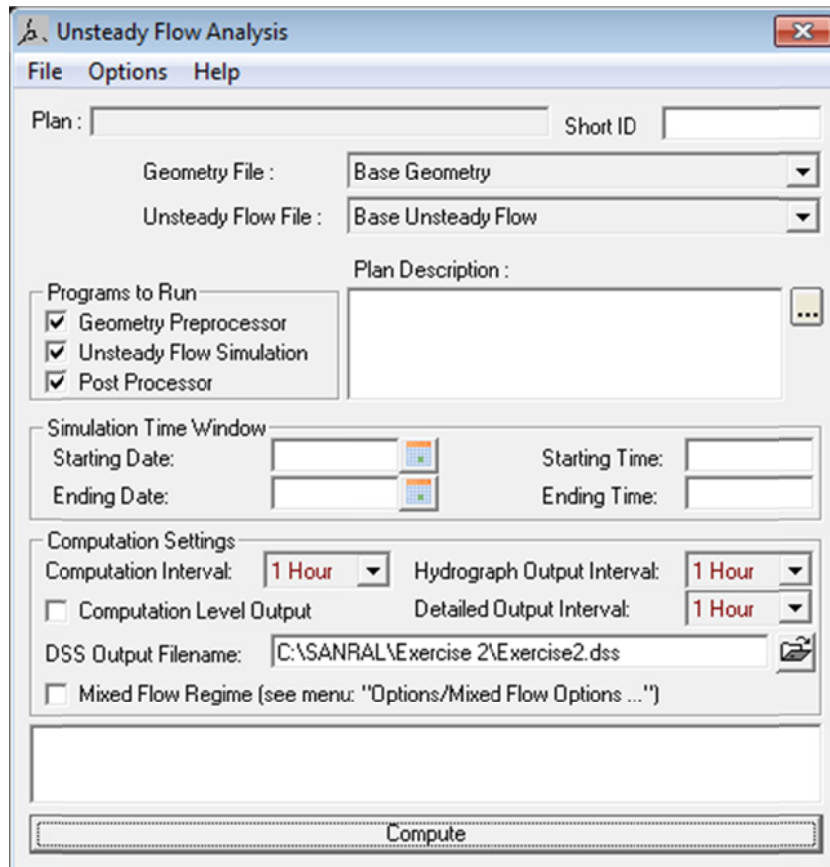
**Figure 11.127: Unsteady Flow Data**

- Initial conditions.** There is no need to add any initial flow conditions at the new internal control since the flow entered at River station 8 will also flow over the weir at the start of the analysis.

## UNSTEADY FLOW ANALYSIS WITH INLINE STRUCTURE AND BRIDGE

### Performing Unsteady Flow Calculations with the Inline Structure and the Bridge

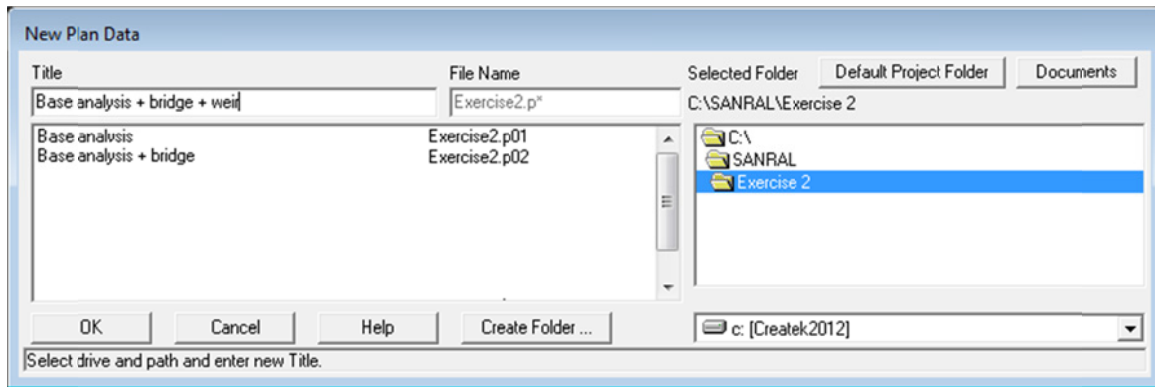
Once all of the geometry and unsteady flow data have been entered, the user can begin performing the unsteady flow calculations. To run the simulation, go to the HEC-RAS main window and select **Unsteady Flow Analysis** from the **Run** menu or click on the **Unsteady Flow Analysis** button  on the menu bar. The **Unsteady Flow Analysis** window will appear as shown in **Figure 11.128**.



**Figure 11.128: Unsteady Flow Analysis**

- The first step is to put together a new **Plan**. The **Plan** defines which geometry and flow data are to be used, as well as providing a title and short identifier for the run.

To establish a plan, select **New Plan** from the **File** menu on the **Unsteady Flow Analysis** window. Enter the plan title as *Base analysis + bridge + weir* and then press the **OK** button (**Figure 11.129**).



**Figure 11.129: Creating new plan**

- You will be prompted to enter a short identifier. Enter a title of *Base+b+w* in the **Short ID** box and click on the **OK** button.
- Select the correct Geometry file and Unsteady flow file from the drop down list i.e. *Base Geometry + bridge + weir* and *Base Unsteady Flow + gates* respectively.

- **Selecting Programs to Run**

There are three components used in performing an unsteady flow analysis within HEC-RAS. These components are: a geometric data pre-processor; the unsteady flow simulator; and an output post-processor (see **Figure 11.128**).

- **Simulation time window**

The user is required to enter a time window that defines the start and end of the simulation period. The time window requires a starting date and time and an ending date and time.

In this exercise the flow hydrograph shown in **Figure 11.73** and **Figure 11.75** starts at 0:00 and has data until 06:00 and this is used in the simulation time window. The date can be anything since the option to **Use Simulation Time** on the Flow Hydrograph window was selected (see **Figures 18** and **20**). Enter the **Starting date** as *01JAN2013* and the **Ending date** as *01JAN2013*. Enter the **Starting time** as *0000* and the **Ending time** as *0600*.

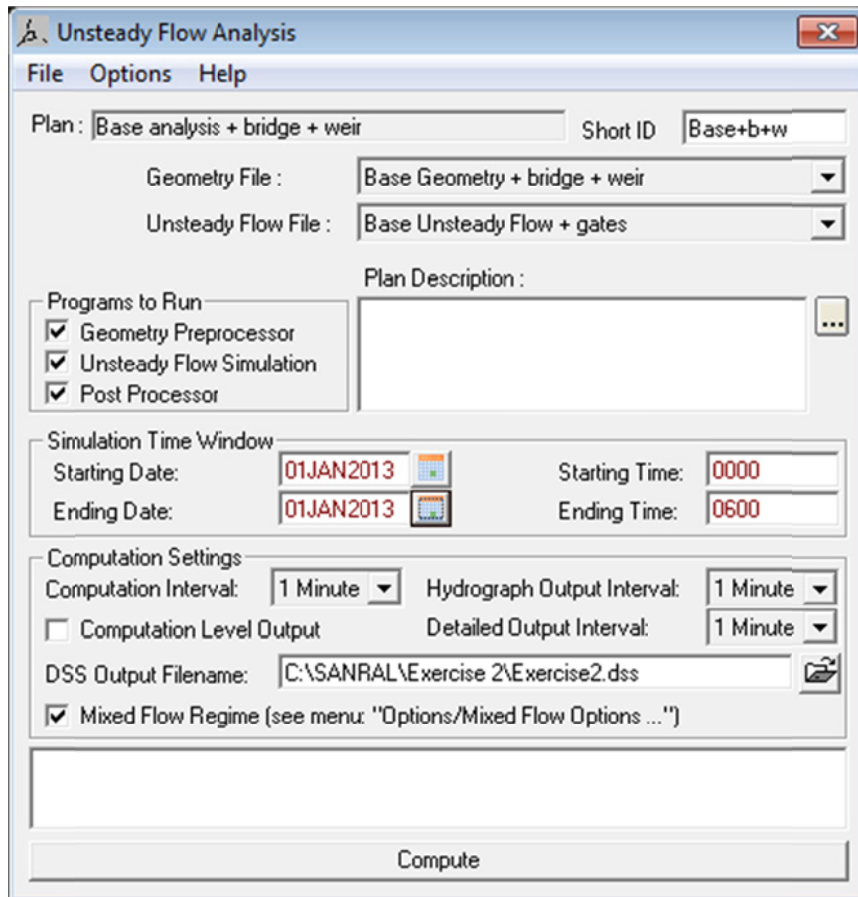
- **Computational settings**

The computational Settings area contains the **Computational Interval**, **Hydrograph Output Interval**, **Detailed Output Interval**, the name and path of the output DSS file, and whether or not the program is run in a mixed flow regime. The computation interval is probably one of the most important parameters entered into the model. This should be small enough to accurately describe the rise and fall of the hydrographs being routed but not small to take forever to compute.

In this exercise the newly added gates close within 1 minute and thus a smaller **Computational Interval** of *1 minute* (from the drop down list) should be selected. Set the **Hydrograph Output Interval** to *1 minute* and the **Detailed Output Interval** also at *1 minute*.

The **DSS Output filename** is the file that contains all the calculated data in a format that can be read by HEC-RAS and used in displaying all the results (tables and graphs). The default will be *.....\Exercise2.dss* and does not have to be changed for this exercise.

- Also select the **Mixed Flow Regime** option since mixed flow might occur at the flow over the weir. The completed **Unsteady Flow Analysis** screen is shown in **Figure 11.130**.



**Figure 11.130: Unsteady Flow Analysis (completed)**

Click on the **Compute** button to run the **Unsteady Flow Analysis**.

### **VIEWING THE RESULTS**

Once the model has finished all of the computations successfully, you can begin viewing the results. Several output options are available from the **View** menu bar on the HEC-RAS main window. These options include:

- a) Cross section plots
  - b) Profile plots
  - c) General profile plot
  - d) Rating curves
  - e) X-Y-Z perspective plots
  - f) Detailed tabular output at a specific cross section (cross section table)
  - g) Limited tabular output at many cross sections (profile table)
- Begin by plotting a cross section. Select **Cross Sections** from the **View** menu bar on the HEC-RAS main window. Any cross section can be plotted by selecting the appropriate river, reach and river station (See **Figure 11.131** and **Figure 11.132**). Several plotting features are available from the **Options** menu bar on the cross section plot window. These options include: zoom in; zoom out; selecting which plans, profiles, variables to plot; and control over lines, labels, symbols, scaling etc.



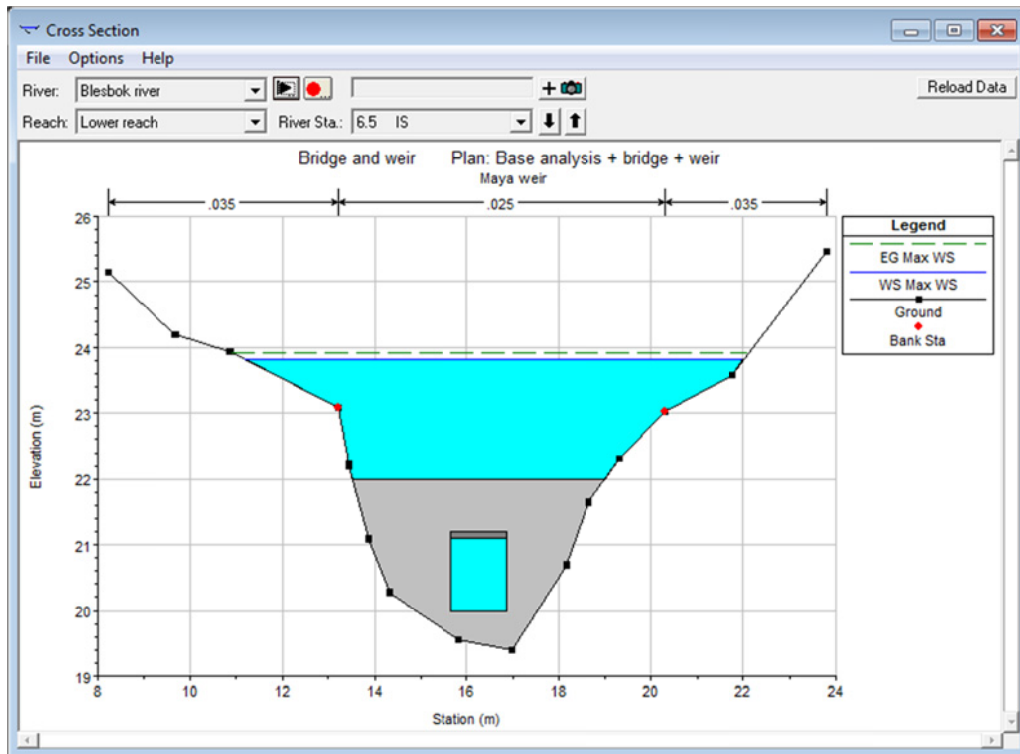


Figure 11.131: Cross section view (Blesbok river: Lower reach – River station 6.5 (Inline structure) at maximum water surface (gates open)

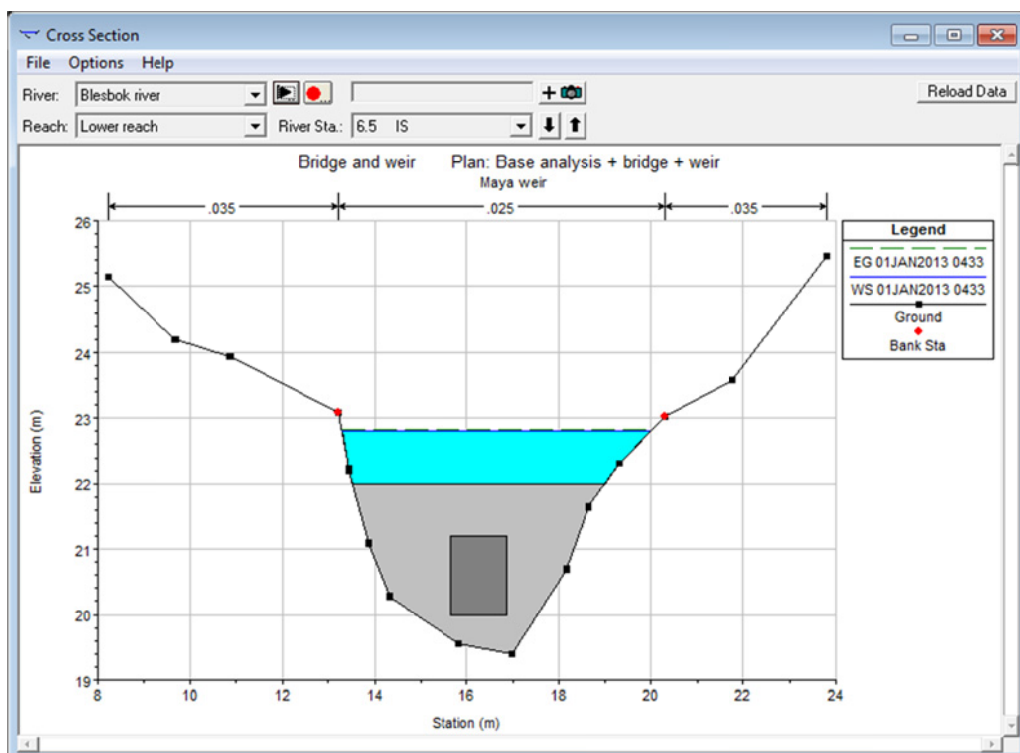




Figure 11.132: Cross section view (Blesbok river: Lower reach – River station 6.5 (Inline structure) at time step when gates just closed completely

- Next plot a water surface profile. Select **Water Surface Profiles** from the **View** menu bar. Click on the Play button, , to view the **Animation Control** window (see Figure 11.133). Click on the **Expand** button, , to see the entire control.

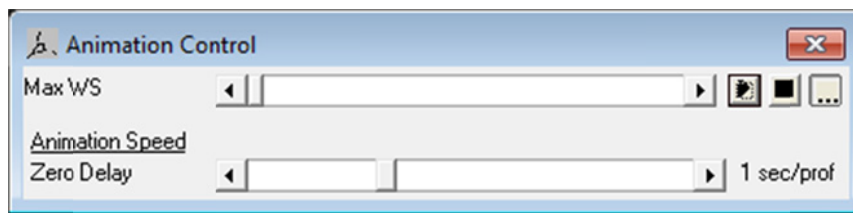



Figure 11.133: Animation Control

- Set the **Zero Delay** horizontal scroll bar as indicated (in Figure 11.133) and click on the play button, , to view the changing water surface levels and energy grade lines of the river reach profile plot. This should give you a profile plot as shown in Figure 11.134 for the maximum water surface.

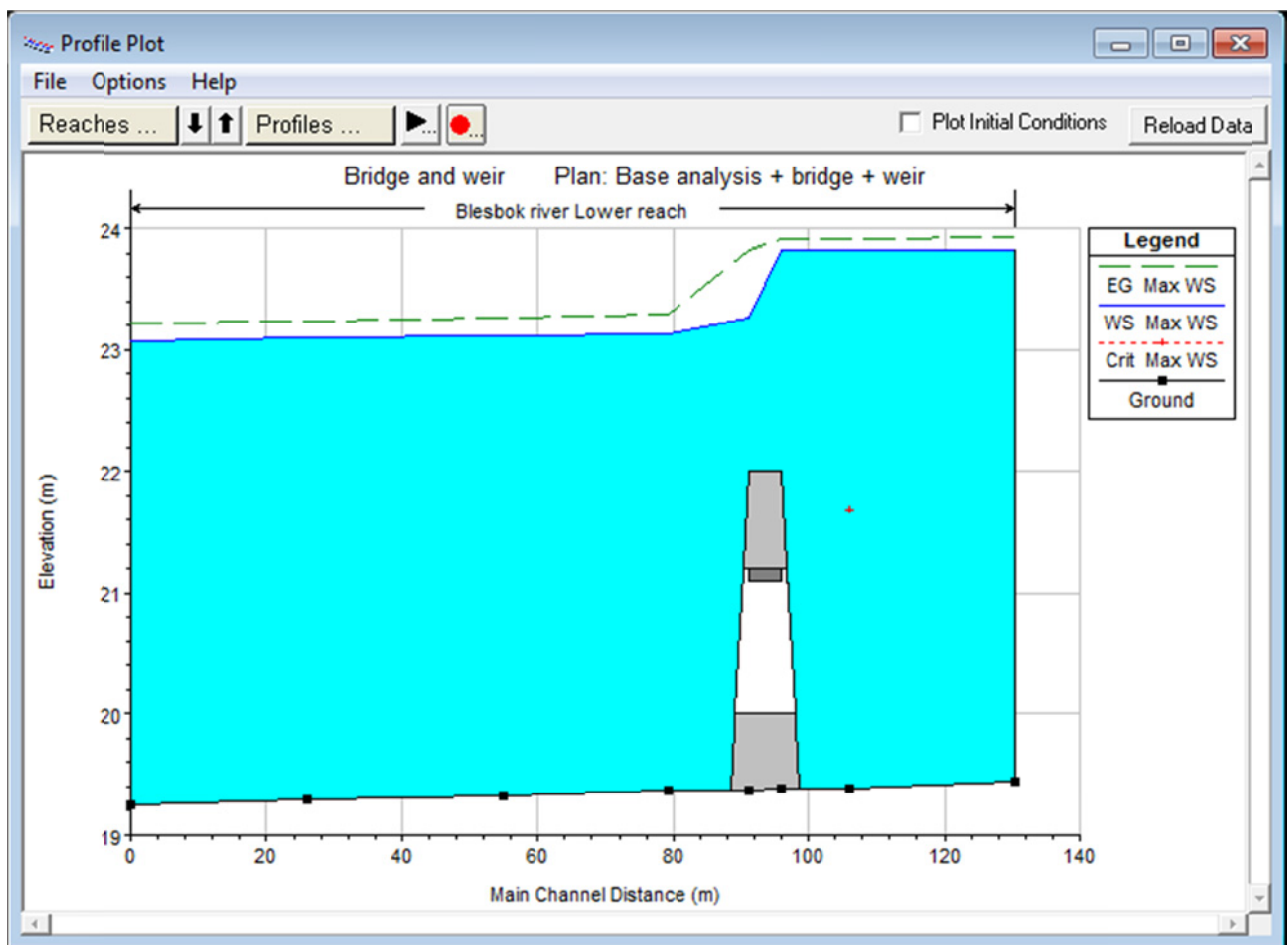
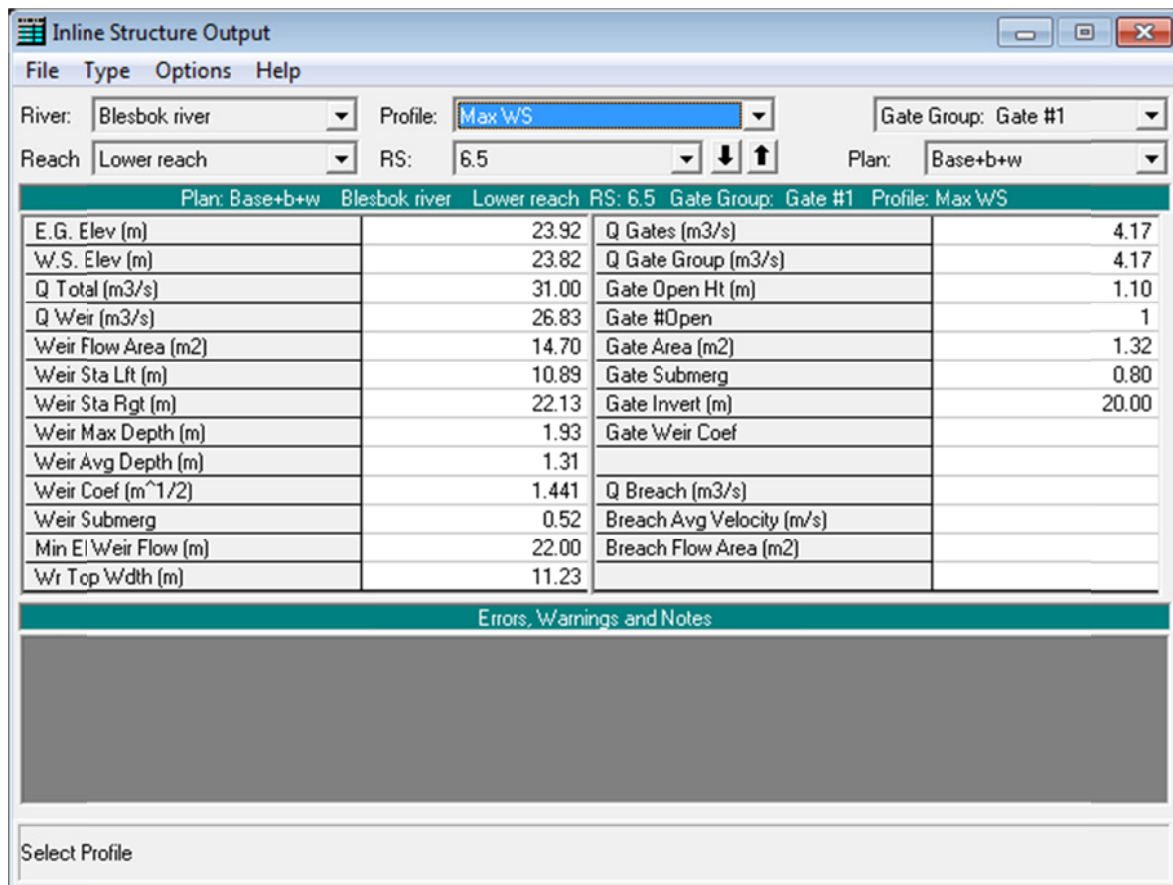


Figure 11.134: Water Surface Profile (for Blesbok river: Lower reach- Maximum Water surface and gate open)

- Also have a look at the tabular output at the inline structure (RS 6.5). Go to the **View** menu bar on the HEC-RAS main window and select **Detailed Output Tables**. From the **Type** menu select **Inline structures**. This table shows detailed hydraulic information at the inline structure. Other cross sections or structures can be viewed by selecting the appropriate reach and river from the table and select the type of data to view.



**Figure 11.135: Detailed Tabular Output (Blesbok river: Lower reach, Inline structure)**

At the end of this exercise the following **objectives** should have been met:

- Be able to a river system containing more than one river reach
- Know how to enter unsteady flow data and defining and entering the boundary controls
- Be able to analyse the river system (unsteady flow)
- Know how to define a bridge structure and a weir
- Know how to set the boundaries for unsteady flow with opening and closing gates
- Know how to obtain results from an unsteady flow analysis

### Questions

1. Define the flow type in the river.
2. Does the flow at any of the cross section flow onto the banks of the river system?
3. At what time does the maximum flow at cross section 40 occur and what is the peak flow? What is the normal flow depth during this peak flow?

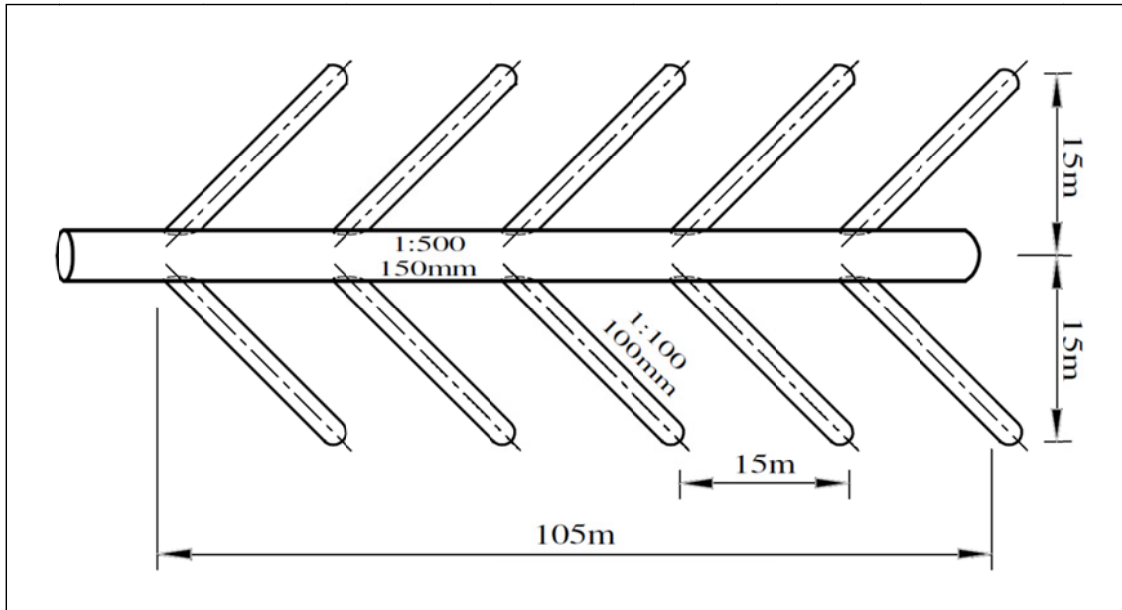
4. What will be the effect on the obtained water level at cross section 3 during maximum flow, if the Junction was analysed using Momentum instead of energy with the Blesbok river joining at an angle of  $45^\circ$ .
5. Describe the reason for the differences in the Rating curves (Flow rate versus Water Surface Elevation) for cross section 40 and cross section 100.
6. What effect does the bridge have on the maximum water surface level at cross section 70?
7. What is the maximum damming upstream of the bridge?
8. Is the bridge function as a control?
9. What will happen if the flow hydrographs of the Upper reach of the Riet river is increased with 100%?
10. What increase in flow velocity is experienced due to the bridge compared to the before scenario?
11. What is the maximum flow through the gate at the inline structure and at what time step does this occur?
12. What effect does the added inline structure have on the flow conditions at the bridge (flow depth, velocity etc.)?
13. What will be the effect if the Computational Interval, Hydrograph Output Interval and Detailed Output Interval are also set to 15 minutes and the rate of opening and closing of the gates are set to 1,2 m/min.

## 12 SUB-SURFACE DRAINAGE

### 12.1 Example 12.1 - Herringbone drainage system

#### Problem description Example 12.1:

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  $0,011 \text{ s/m}^{1/3}$ . **Figure 12.1** reflects the layout.



**Figure 12.1: Layout of the herringbone drainage system**

#### Solution Example 12.1:

For each lateral, flowing 70% full:

$$q = \frac{(26,92 \times 10^6) d^{8/3} (S_0^{1/2}) (0,7)}{n A}$$

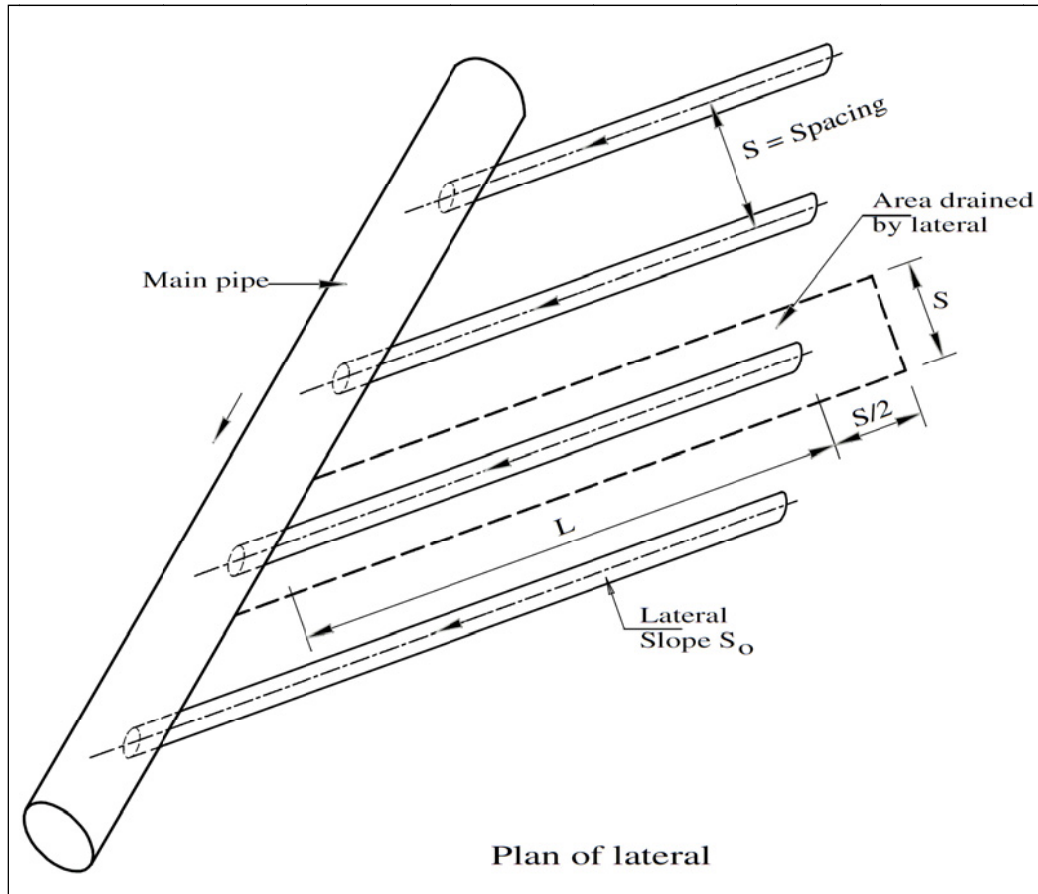
$$A = S \left( L + \frac{S}{2} \right) \quad \text{Refer to Figure 12.2}$$

$$L = \sqrt{15^2 + 15^2} = 21,21 \text{ m}$$

$$S = \frac{L}{2} = 10,61 \text{ m} \quad (\text{Valid in this example since laterals are placed at } 45^\circ \text{ and dimensions are equal } 15 \text{ m} \times 15 \text{ m})$$

$$\begin{aligned} \therefore A &= 10,61 \left( (21,21) + \frac{10,61}{2} \right) \\ &= 281,3 \text{ m}^2 \end{aligned}$$

$$\therefore q = \frac{(26,92 \times 10^6) (0,1^{8/3}) (0,01^{1/2}) (0,7)}{(0,011) (281,3)} = 1312 \text{ mm/day}$$



**Figure 12.2: General view of a herringbone drainage system**

Flow rate,  $Q$ , for 14 laterals:

$$Q = (1,312)(281,3)(14)$$

$$Q = 5167 \text{ m}^3/\text{day}$$

$$\text{Hence } Q \approx 0,06 \text{ m}^3/\text{s}$$

**Capacity of main pipe: (Manning Formula)**

$$Q = \frac{\frac{\pi}{4} (0,15^2) \left( \frac{0,15}{4} \right)^{2/3} \left( \frac{1}{500} \right)^{1/2}}{0,011}$$

$$Q = 0,00805 \text{ m}^3/\text{s}$$

But this  $\ll 0,06 \text{ m}^3/\text{s}$  !

$$Q \approx 695 \text{ m}^3/\text{day}$$

$$\therefore q_{\max} = \frac{695}{(281,3)(14)}$$

$$q_{\max} = 177 \text{ mm/day } (\ll 1312 \text{ mm/day})$$

## **APPENDICES**

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## **APPENDIX 3A**

### **STATISTICAL ANALYSIS**

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### *Single station direct statistical analysis*

The following frequency distributions are discussed for untransformed and log<sub>10</sub>-transformed data:

- **Untransformed data**  
Normal, Extreme Value Type 1 and General Extreme Value
- **Log<sub>10</sub>-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<sub>10</sub> transformed data as follows:

$$\text{Mean} \quad \bar{x} = \frac{\sum x}{N} \quad \dots(3A.1)$$

$$\text{Standard deviation} \quad s = \left[ \frac{\sum (x - \bar{x})^2}{N - 1} \right]^{0.5} \quad \dots(3A.2)$$

$$\text{Skewness coefficient} \quad g = \left( \frac{N}{(N-1)(N-2)} \right) \left( \frac{\sum (x - \bar{x})^3}{s^3} \right) \quad \dots(3A.3)$$

$$\text{Coefficient of variation} \quad c_v = \frac{s}{\bar{x}} \quad \dots(3A.4)$$

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. 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:

$$G(y) = \int_{-\infty}^y \frac{1}{\sqrt{2\pi}} e^{-0.5[y^2]} dy \quad \dots(3A.5)$$

where  $y$  is the standardized variable and is related to  $x$  by:

$$y = \frac{(x - \bar{x})}{s} \quad \dots(3A.6)$$

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**.

$$Q_T = \bar{x} + sy \quad \dots(3A.7)$$

- **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:

$$Q_T = \bar{x} + s(0,780W_T - 0,450) \quad \dots(3A.8)$$

- **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  $\text{var}(y)$  from **Table 3A.2** by using linear interpolation.

For EV2 distribution:

$$Q_T = \bar{x} + \sqrt{\frac{s^2}{\text{var}(y)}} (1 - E(y) - kW_T) \quad \dots(3A.9)$$

For EV3 distribution:

$$Q_T = \bar{x} + \sqrt{\frac{s^2}{\text{var}(y)}} (-1 - E(y) + kW_T) \quad \dots(3A.10)$$

- **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**.

$$Q_T = \text{antilog}[\overline{\log(x)} + s_{\log} W_T] \quad \dots(3A.11)$$

where:

$s_{\log}$  = the standard deviation of the logarithms of the observed values as shown in equation 3A.11

$$s_{\log} = \left[ \frac{\sum (\log(x) - \overline{\log(x)})^2}{N - 1} \right]^{0,5} \quad \dots(3A.12)$$

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.

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:

$$Q_{T(95\%)} = \text{antilog}[\overline{\log(x)} + s_{\log} (W_T \pm W_\alpha)] \quad \dots(3A.13)$$

where:

$$W_\alpha = \text{displacement of the confidence band (column 5 in Table 3A.3)}$$

- **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:

$$Q_T = \text{antilog}[\overline{\log(x)} + s_{\log} (0,780W_T - 0,450)] \quad \dots(3A.14)$$

- **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.

$$Q_T = \text{antilog}[\overline{\log(x)} + s_{\log} W_T] \quad \dots(3A.15)$$

Based on an example from *Flood Risk Reduction Measures* by WJR Alexander 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 ( $g_h$ ).

$$\bar{x}_h = \frac{((WT)\sum x_b + \sum x_a)}{(YT - (WT)(LW))} \quad \dots(3A.16)$$

$$s_h = \left[ \frac{((WT)\sum d_b^2 + \sum d_a^2)}{(YT - (WT)(LW) - 1)} \right]^{0,5} \quad \dots(3A.17)$$

$$g_h = \left[ \frac{(YT - (WT)(LW))((WT)\sum d_b^3 + \sum d_a^3)}{s^3} \right] \quad \dots(3A.18)$$

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 <sup>(3.1)</sup>.

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 WJR Alexander.

Evaluating Example 3.2 for the Tsitsa River, 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 <sub>v</sub>	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	813	766	758	714	645	719
10	985	994	991	1035	1057	1022
20	1127	1214	1221	1401	1696	1359
50	1287	1497	1527	1980	3126	1850
100	1394	1710	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 <sub>v</sub>	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	795	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**.

$$T = \frac{n_i + a}{m - b} \quad \dots (3A.19)$$

where:

$T$	=	return period in years
$n_i$	=	length of record in years
$m$	=	number, in descending order, of the ranked annual peak floods
$a$	=	constant (see <b>Table 3A.7</b> )
$b$	=	constant (see <b>Table 3A.7</b> )

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

$$P = \frac{1}{T} \quad \dots (3A.20)$$

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**

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. Alternatively utilize software such as *Utility Programs for Drainage* or *HEC-SSP* included on the supporting flash drive.

**Table 3A.1: Properties of the standardized normal distribution**

**Table 3A.1a**

Standardized normal distribution			
y	G(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	32,64
0,50	69,14	-0,50	30,86
0,55	70,88	-0,55	29,12
0,60	72,57	-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	91,94	-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 <sub>T</sub>
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**

<b>Standardized general extreme value distribution</b>			
<b>g</b>	<b>k</b>	<b>E(y)</b>	<b>var(y)</b>
-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.3a: Values of the standardized variate  $W_T$  for the normal and exponential distributions**

Return period (years)	Non-exceedance probability	Normal distribution			Exponential distribution
		W <sub>T</sub>	Confidence limits W <sub>a</sub>		
			75%	95%	
2	0,50	0,00	1,63/√2N	2,77/√2N	0,69
5	0,80	0,84	1,89/√2N	3,23/√2N	1,61
10	0,90	1,28	2,20/√2N	3,74/√2N	2,30
20	0,95	1,64	2,49/√2N	4,25/√2N	3,00
50	0,98	2,05	2,87/√2N	4,89/√2N	3,91
100	0,99	2,33	3,13/√2N	5,34/√2N	4,61
200	0,995	2,58	3,38/√2N	5,76/√2N	5,30
500	0,998	2,88	3,69/√2N	6,27/√2N	6,21
1000	0,999	3,09	3,91/√2N	6,66/√2N	6,91
10000	0,9999	3,72	4,58/√2N	7,80/√2N	9,21

**Table 3A.3b: Values of the standardized variate  $W_T$  for the Pearson Type III distribution**

Return period (years)	Pearson Type III distribution (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
2	0,16	0,13	0,10	0,07	0,03	0,00	-0,03	-0,70	-0,10	-0,13	-0,16
5	0,85	0,87	0,86	0,86	0,85	0,84	0,83	0,82	0,80	0,78	0,76
10	1,13	1,17	1,20	1,23	1,26	1,28	1,30	1,32	1,33	1,34	1,34
20	1,32	1,39	1,46	1,52	1,59	1,64	1,70	1,75	1,80	1,84	1,88
50	1,49	1,61	1,72	1,83	1,94	2,05	2,16	2,26	2,36	2,45	2,54
100	1,59	1,73	1,88	2,03	2,18	2,33	2,47	2,62	2,76	2,89	3,02
200	1,66	1,84	2,02	2,20	2,39	2,58	2,76	2,95	3,13	3,31	3,49
500						2,88					
1000	1,79	2,02	2,27	2,53	2,81	3,09	3,38	3,67	3,96	4,24	4,53
10000	1,88	2,18	2,53	2,90	3,30	3,72	4,15	4,60	5,05	5,50	5,96

**Table 3A.4a: Values of the standardized variate  $W_T$  for the general extreme value distribution (EV1 & EV2)**

Return period (years)	General extreme value (Values of $W_T$ )													
	g													
	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
	EV1	EV2												
2	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	1,50	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	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	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	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	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	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	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	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	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

**Table 3A.4b: Values of the standardized variate  $W_T$  for the general extreme value distribution (EV3)**

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
	EV3										
2		0,33	0,34	0,34	0,34	0,35	0,35	0,36	0,36	0,36	0,37
5		1,01	1,06	1,12	1,17	1,23	1,28	1,33	1,38	1,43	1,47
10		1,28	1,37	1,46	1,57	1,67	1,78	1,89	2,00	2,10	2,19
20		1,44	1,57	1,71	1,86	2,02	2,19	2,37	2,54	2,71	2,86
50		1,58	1,74	1,93	2,15	2,38	2,64	2,90	3,18	3,45	3,72
100		1,64	1,83	2,05	2,31	2,60	2,92	3,26	3,62	3,99	4,35
200		1,68	1,89	2,14	2,44	2,77	3,16	3,58	4,02	4,49	4,97
500		1,72	1,95	2,23	2,56	2,96	3,42	3,94	4,51	5,13	5,76
1000		1,74	1,98	2,27	2,64	3,07	3,59	4,19	4,86	5,58	6,35
10000		1,76	2,02	2,36	2,78	3,32	4,00	4,83	5,81	6,96	8,24

**APPENDIX 3B**  
**STANDARD DESIGN FLOOD METHOD**

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**Table 3B.1: Information required for the calculation of the SDF**

<b>Basin</b>	<b>SAWS station number</b>	<b>SAWS site</b>	<b>M (mm)</b>	<b>R (days)</b>	<b>C<sub>2</sub> (%)</b>	<b>C<sub>100</sub> (%)</b>	<b>MAP (mm)</b>	<b>MAE (mm)</b>
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	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	LETJESBOS	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

**APPENDIX 3C**  
**STANDARD FLOOD CALCULATION FORMS**

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# RATIONAL METHOD (ALTERNATIVE 1)

<b>Description of catchment</b>							
<b>River detail</b>							
<b>Calculated by</b>				<b>Date</b>			
<b>Physical characteristics</b>							
Size of catchment (A)				km <sup>2</sup>	Rainfall region		
Longest watercourse (L)				km	<b>Area distribution factors</b>		
Average slope (S <sub>av</sub> )				m/m	Rural (α)	Urban (β)	Lakes (γ)
Dolomite area (D <sub>%</sub> )				%			
Mean annual precipitation (MAP)⑥				mm			
<b>Rural①</b>				<b>Urban②</b>			
<b>Surface slope</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>s</sub></b>	<b>Description</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>2</sub></b>
Vleis and pans				<b>Lawns</b>			
Flat areas				Sandy, flat (<2%)			
Hilly				Sandy, steep (>7%)			
Steep areas				Heavy soil, flat (<2%)			
Total	100	-		Heavy soil, steep (>7%)			
<b>Permeability</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>p</sub></b>	<b>Residential areas</b>			
Very permeable				Houses			
Permeable				Flats			
Semi-permeable				<b>Industry</b>			
Impermeable				Light industry			
Total	100	-		Heavy industry			
<b>Vegetation</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>v</sub></b>	<b>Business</b>			
Thick bush and plantation				City centre			
Light bush and farm-lands				Suburban			
Grasslands				Streets			
No vegetation				Maximum flood			
Total	100	-		Total (C <sub>2</sub> )	100	-	
<b>Time of concentration (T<sub>C</sub>)</b>				<b>Notes:</b>			
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					
<b>Run-off coefficient</b>							
<b>Return period (years), T</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>Max</b>
Run-off coefficient, C <sub>1</sub> (C <sub>1</sub> = C <sub>s</sub> + C <sub>p</sub> + C <sub>v</sub> )							
Adjusted for dolomitic areas, C <sub>1D</sub> (= C <sub>1</sub> (1 - D <sub>%</sub> ) + C <sub>1D</sub> (Σ(D <sub>factor</sub> x C <sub>S%</sub> )))④							
Adjustment factor for initial saturation, F <sub>i</sub> ⑤							
Adjusted run-off coefficient, C <sub>1T</sub> (= C <sub>1D</sub> x F <sub>i</sub> )							
Combined run-off coefficient C <sub>T</sub> (= αC <sub>1T</sub> + βC <sub>2</sub> + γC <sub>3</sub> )							
<b>Rainfall</b>							
<b>Return period (years), T</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>Max</b>
Point precipitation (mm), P <sub>T</sub> ⑥							
Point intensity (mm/hour), P <sub>IT</sub> (= P <sub>T</sub> /T <sub>C</sub> )							
Area reduction factor (%), ARF <sub>T</sub> ⑦							
Average intensity (mm/hour), I <sub>T</sub> (= P <sub>IT</sub> x ARF <sub>T</sub> )							
<b>Return period (years), T</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>Max</b>
Peak flow (m <sup>3</sup> /s), Q <sub>T</sub> = $\frac{C_T I_T A}{3,6}$							

Note: # Reference to the appropriate figures and tables is made in the legend table of this method.

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## RATIONAL METHOD (ALTERNATIVE 1)



<b>LEGEND TABLE</b> <b>Rational method (Alt 1)</b>	
<b>ID</b>	<b>Reference</b>
①	Figure 3.5 or SA Weather Services
①	Table 3C.1
②	Table 3C.2
③	Table 3C.3
④	Table 3C.4
⑤	Table 3C.5
⑥	Figure 3.7
⑦	Figure 3.8 (or Figure 3.26 DM)

<b>Table 3C.1</b>				
<b>Rural (C<sub>1</sub>)</b>				
<b>Component</b>	<b>Classification</b>	<b>Mean annual precipitation (mm)</b>		
		<b>600</b>	<b>600 - 900</b>	<b>900</b>
<b>Surface slope (C<sub>s</sub>)</b>	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
<b>Permeability (C<sub>p</sub>)</b>	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
<b>Vegetation (C<sub>v</sub>)</b>	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

<b>Table 3C.2</b>	
<b>Urban (C<sub>2</sub>)</b>	
<b>Use</b>	<b>Factor</b>
<b>Lawns</b>	
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
<b>Residential areas</b>	
Houses	0,30 - 0,50
Flats	0,50 - 0,70
<b>Industry</b>	
Light industry	0,50 - 0,80
Heavy industry	0,60 - 0,90
<b>Business</b>	
City centre	0,70 - 0,95
Suburban	0,50 - 0,70
Streets	0,70 - 0,95
Maximum flood	1,00

<b>Table 3C.3</b>	
<b>Surface description</b>	<b>Recommended value of r</b>
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

<b>Table 3C.4</b>	
<b>Adjustment factor to C<sub>s</sub></b>	
<b>Surface slope classification</b>	<b>D<sub>factor</sub></b>
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

<b>Table 3C.5</b>						
<b>Return period (years)</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>
Adjustment factor (F <sub>i</sub> ) for steep and impermeable catchments	0,75	0,80	0,85	0,90	0,95	1,00
Adjustment factor (F <sub>i</sub> ) for flat and permeable catchments	0,50	0,55	0,60	0,67	0,83	1,00

## RATIONAL METHOD (ALTERNATIVE 2)

<b>Description of catchment</b>							
<b>River detail</b>							
<b>Calculated by</b>				<b>Date</b>			
<b>Physical characteristics</b>							
Size of catchment (A)				km <sup>2</sup>	Days of thunder per year (R)②		days/year
Longest watercourse (L)				km	Weather Service station⑩		
Average slope (S <sub>av</sub> )				m/m	Weather Service number⑩		
Dolomite area (D%)				%	<b>Area distribution factors</b>		
Mean annual precipitation (MAP)⑩ <sup>#</sup>				mm	Rural (α)	Urban (β)	Lakes (γ)
2-year return period rainfall (M) ①				mm			
<b>Rural③</b>				<b>Urban④</b>			
<b>Surface slope</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>s</sub></b>	<b>Description</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>2</sub></b>
Vleis and pans				<b>Lawns</b>			
Flat areas				Sandy, flat (<2%)			
Hilly				Sandy, steep (>7%)			
Steep areas				Heavy soil, flat (<2%)			
Total	100	-		Heavy soil, steep (>7%)			
<b>Permeability</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>p</sub></b>	<b>Residential areas</b>			
Very permeable				Houses			
Permeable				Flats			
Semi-permeable				<b>Industry</b>			
Impermeable				Light industry			
Total	100	-		Heavy industry			
<b>Vegetation</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>v</sub></b>	<b>Business</b>			
Thick bush and plantation				City centre			
Light bush and farm-lands				Suburban			
Grasslands				Streets			
No vegetation				Maximum flood			
Total	100	-		Total (C <sub>2</sub> )	100	-	
<b>Time of concentration (T<sub>C</sub>)</b>				<b>Notes:</b>			
Overland flow⑤		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					
<b>Run-off coefficient</b>							
<b>Return period (years), T</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>Max</b>
Run-off coefficient, C <sub>1</sub> (C <sub>1</sub> = C <sub>s</sub> + C <sub>p</sub> + C <sub>v</sub> )							
Adjusted for dolomitic areas, C <sub>1D</sub> (= C <sub>1</sub> (1 - D%) + C <sub>1</sub> D%(Σ(D <sub>factor</sub> x C <sub>S%</sub> )))⑥							
Adjustment factor for initial saturation, F <sub>i</sub> ⑦							
Adjusted run-off coefficient, C <sub>1T</sub> (= C <sub>1D</sub> x F <sub>i</sub> )							
Combined run-off coefficient C <sub>T</sub> (= αC <sub>1T</sub> + βC <sub>2</sub> + γC <sub>3</sub> )							
<b>Rainfall</b>							
<b>Return period (years), T</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>Max</b>
Point precipitation (mm), P <sub>T</sub> ⑧							
Point intensity (mm/hour), P <sub>IT</sub> (= P <sub>T</sub> /T <sub>C</sub> )							
Area reduction factor (%), ARF <sub>T</sub> ⑨							
Average intensity (mm/hour), I <sub>T</sub> (= P <sub>IT</sub> x ARF <sub>T</sub> )							
Peak flow (m <sup>3</sup> /s) Q <sub>T</sub> = $\frac{C_T I_T A}{3,6}$							

Note: # Reference to the appropriate figures and tables is made in the legend table of this method.

## RATIONAL METHOD (ALTERNATIVE 2)

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							

LEGEND TABLE Rational method (Alt 2)	
ID	Reference
①	Figure 3.5 or SA Weather Services
①	TR102 or other
②	Figure 3.9
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 3.10
⑩	TR102

Table 3C.6	
Selection criteria	Calculation method
$T_C < 6$ hours	Modified Hershfield equation $P_{t,T} = 1,13(0,41 + 0,64\ln T)(-0,11 + 0,27\ln t)(0,79M^{0,69}R^{0,20})$
$6 \text{ hours} \leq T_C < 24 \text{ hours}$	Linear interpolation between calculated modified Hershfield equation point rainfall and 1-day point rainfall from TR102
$T_C \geq 24 \text{ hours}$	Linear interpolation between n-day point rainfall values from TR102

### RATIONAL METHOD (ALTERNATIVE 3)

<b>Description of catchment</b>							
<b>River detail</b>							
<b>Calculated by</b>				<b>Date</b>			
<b>Physical characteristics</b>							
Size of catchment (A)				km <sup>2</sup>	Weather Service station <sup>⑧</sup>		
Longest watercourse (L)				km	Weather Service number <sup>⑧</sup>		
Average slope (S <sub>av</sub> )				m/m	<b>Area distribution factors</b>		
Dolomite area (D <sub>%</sub> )				%	Rural (α)	Urban (β)	Lakes (γ)
Mean annual precipitation (MAP) <sup>⑩</sup>				mm			
<b>Rural<sup>①</sup></b>				<b>Urban<sup>②</sup></b>			
<b>Surface slope</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>s</sub></b>	<b>Description</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>2</sub></b>
Vleis and pans				<b>Lawns</b>			
Flat areas				Sandy, flat (<2%)			
Hilly				Sandy, steep (>7%)			
Steep areas				Heavy soil, flat (<2%)			
Total	100	-		Heavy soil, steep (>7%)			
<b>Permeability</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>p</sub></b>	<b>Residential areas</b>			
Very permeable				Houses			
Permeable				Flats			
Semi-permeable				<b>Industry</b>			
Impermeable				Light industry			
Total	100	-		Heavy industry			
<b>Vegetation</b>	<b>%</b>	<b>Factor</b>	<b>C<sub>v</sub></b>	<b>Business</b>			
Thick bush and plantation				City centre			
Light bush and farm-lands				Suburban			
Grasslands				Streets			
No vegetation				Maximum flood			
Total	100	-		Total (C <sub>2</sub> )	100	-	
<b>Time of concentration (T<sub>C</sub>)</b>				Notes:			
Overland flow <sup>③</sup>		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					
<b>Run-off coefficient</b>							
<b>Return period (years), T</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>Max</b>
Run-off coefficient, C <sub>1</sub> (C <sub>1</sub> = C <sub>s</sub> + C <sub>p</sub> + C <sub>v</sub> )							
Adjusted for dolomitic areas, C <sub>1D</sub> (= C <sub>1</sub> (1 - D <sub>%</sub> ) + C <sub>1</sub> D <sub>%</sub> (Σ(D <sub>factor</sub> x C <sub>S%</sub> ))) <sup>④</sup>							
Adjustment factor for initial saturation, F <sub>i</sub> <sup>⑤</sup>							
Adjusted run-off coefficient, C <sub>1T</sub> (= C <sub>1D</sub> x F <sub>i</sub> )							
Combined run-off coefficient C <sub>T</sub> (= αC <sub>1T</sub> + βC <sub>2</sub> + γC <sub>3</sub> )							
<b>Rainfall</b>							
<b>Return period (years), T</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>Max</b>
Point precipitation (mm), P <sub>T</sub> <sup>⑥</sup>							
Point intensity (mm/hour), P <sub>IT</sub> (= P <sub>T</sub> /T <sub>C</sub> )							
Area reduction factor (%), ARF <sub>T</sub> <sup>⑦</sup>							
Average intensity (mm/hour), I <sub>T</sub> (= P <sub>IT</sub> x ARF <sub>T</sub> )							
Peak flow (m <sup>3</sup> /s) Q <sub>T</sub> = $\frac{C_T I_T A}{3,6}$							

Note: # Reference to the appropriate figures and tables is made in the legend table of this method.

## RATIONAL METHOD (ALTERNATIVE 3)

<b>LEGEND TABLE</b> <b>Rational method (Alt 3)</b>	
<b>ID</b>	<b>Reference</b>
①	Figure 3.5 or SA Weather Services
①	Table 3C.7
②	Table 3C.8
③	Table 3C.9
④	Table 3C.10
⑤	Table 3C.11
⑥ <sup>#</sup>	Figure 3.12 and Figure 3.13
⑦	Figure 3.8 (or Figure 3.26 DM)
⑧	Figure 3.10, Figure 3.13 or other

<b>Table 3C.7</b>				
<b>Rural (C<sub>1</sub>)</b>				
<b>Component</b>	<b>Classification</b>	<b>Mean annual rainfall (mm)</b>		
		<b>600</b>	<b>600 - 900</b>	<b>900</b>
<b>Surface slope (C<sub>s</sub>)</b>	Wetlands 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
<b>Permeability (C<sub>p</sub>)</b>	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
<b>Vegetation (C<sub>v</sub>)</b>	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

<b>Table 3C.8</b>	
<b>Urban (C<sub>2</sub>)</b>	
<b>Use</b>	<b>Factor</b>
<b>Lawns</b>	
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
<b>Residential areas</b>	
Houses	0,30 - 0,50
Flats	0,50 - 0,70
<b>Industry</b>	
Light industry	0,50 - 0,80
Heavy industry	0,60 - 0,90
<b>Business</b>	
City centre	0,70 - 0,95
Suburban	0,50 - 0,70
Streets	0,70 - 0,95
Maximum flood	1,00

<b>Table 3C.9</b>	
<b>Surface description</b>	<b>Recommended value of r</b>
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

<b>Table 3C.10</b>	
<b>Adjustment factor to C<sub>s</sub></b>	
<b>Surface slope classification</b>	<b>D<sub>factor</sub></b>
Steep areas (slopes >30%)	0,50
Hilly (10 to 30%)	0,35
Flat areas (3 to 10%)	0,20
Wetlands and pans (slopes <3%)	0,10

<b>Table 3C.11</b>						
<b>Return period (years)</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>
Adjustment factor (F <sub>i</sub> ) for steep and impermeable catchments	0,75	0,80	0,85	0,90	0,95	1,00
Adjustment factor (F <sub>i</sub> ) for flat and permeable catchments	0,50	0,55	0,60	0,67	0,83	1,00

**Note:** # Calculate the point intensity rainfall by making use of the provided *Design Rainfall* estimation software. The exact point intensity can be calculated by means of linear interpolation between two consecutive values considering the time of concentration.



### LEGEND TABLE

Unit Hydrograph method

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# STANDARD DESIGN FLOOD METHOD

<b>Description of catchment</b>							
<b>River detail</b>							
<b>Calculated by</b>						<b>Date</b>	
<b>Physical characteristics</b>							
Size of catchment (A)		km <sup>2</sup>	Time of concentration (T <sub>C</sub> )	$T_C = \left( \frac{0,87L^2}{1000S_{av}} \right)^{0,385}$			hours
Longest watercourse (L)		km					
Average slope (S <sub>av</sub> )		m/m					
SDF basin <sup>①</sup>			Time of concentration, t (= 60T <sub>C</sub> )				minutes
2-year return period rainfall (M) <sup>①</sup>		mm	Days of thunder per year (R) <sup>①</sup>				days/year
<b>TR102 n-day rainfall data</b>							
Weather Service station			Mean annual precipitation (MAP)				mm
Weather Service station no.			Coordinates			&	
<b>Duration (days)</b>	<b>Return period (years)</b>						
	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>200</b>
1 day							
2 days							
3 days							
7 days							
<b>Rainfall</b>							
<b>Return period (years), T</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>200</b>
Point precipitation depth (mm), P <sub>t,T</sub> <sup>②</sup>							
Area reduction factor (%), ARF (= (90000 – 12800lnA + 9830lnt) <sup>0,4</sup> )							
Average intensity (mm/hour), I <sub>T</sub> (= P <sub>t,T</sub> x ARF / T <sub>C</sub> )							
<b>Run-off coefficients</b>							
Calibration factors <sup>①</sup>	C <sub>2</sub> (2-year return period) (%)		C <sub>100</sub> (100-year return period) (%)				
<b>Return period (years)</b>	<b>2</b>	<b>5</b>	<b>10</b>	<b>20</b>	<b>50</b>	<b>100</b>	<b>200</b>
Return period factors (Y <sub>T</sub> )	0	0,84	1,28	1,64	2,05	2,33	2,58
Run-off coefficient (C <sub>T</sub> ), $C_T = \frac{C_2}{100} + \left( \frac{Y_T}{2,33} \right) \left( \frac{C_{100}}{100} - \frac{C_2}{100} \right)$							
Peak flow (m <sup>3</sup> /s), Q <sub>T</sub> = $\frac{C_T I_T A}{3,6}$							

LEGEND TABLE Standard Design Flood method			
ID	Reference	ID	Reference
①	Figure 3.21	②	Table 3C.12
③	Table 3B.1		

Table 3C.12	
Criteria	Calculation method
T <sub>C</sub> < 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 <sub>C</sub> < 24 hours	Linear interpolation between calculated modified Hershfield equation point rainfall and 1-day point rainfall from TR102
T <sub>C</sub> ≥ 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.

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## SCS-SA METHOD

<b>Description of catchment</b>											
<b>River detail</b>											
<b>Calculated by</b>					<b>Date</b>						
<b>Physical characteristics</b>											
Size of catchment (A)			km <sup>2</sup>	<b>Time of Concentration (T<sub>C</sub>)</b>							
				<b>Defined watercourse</b>			hours				
Longest watercourse (L)		km	$T_C = \left( \frac{0,87L^2}{1000S_{av}} \right)^{0,385}$								
Average slope (S <sub>av</sub> )		m/m			<b>Overland flow</b>						
Lag estimation	L = 0,6T <sub>C</sub>		hours	$T_C = 0,604 \left( \frac{rL}{\sqrt{S_{av}}} \right)^{0,467}$							
<b>Return period (years)</b>		<b>1:2</b>	<b>1:5</b>	<b>1:10</b>	<b>1:20</b>	<b>1:50</b>	<b>1:100</b>				
Daily rainfall depth (one-day design rainfall, P) (mm) ⑩											
Area reduction factor (only applied for large catchments, ARF) (%) ⑪											
Catchment design rainfall (P x ARF/100) (mm)											
<b>HRU</b>	<b>Area (A<sub>i</sub>) (%)</b>	<b>SOIL ②</b>					<b>LAND COVER ③</b>				
		<b>Form</b>	<b>Series</b>	<b>Typical Textural Class</b>	<b>Depth (m)</b>	<b>SCS Grouping</b>	<b>Land Cover Class</b>	<b>Cover Category (S/I/D)</b>	<b>Practice/Treat-ment</b>	<b>Storm-Flow Potential ④</b>	
1											
2											
3											
4											
5											
		<b>HRU 1</b>		<b>HRU 2</b>		<b>HRU 3</b>		<b>HRU 4</b>		<b>HRU 5</b>	
Initial Curve Number (CN) ④											
Final Curve Number ⑤											
Potential maximum soil water retention (S, mm) ⑥											
Initial losses (mm)		I <sub>a</sub> = 0,12S									
<b>Return period (years)</b>		<b>1:2</b>	<b>1:5</b>	<b>1:10</b>	<b>1:20</b>	<b>1:50</b>	<b>1:100</b>				
<b>HRU</b>		<b>Design Stormflow depth (Q<sub>i</sub>) ⑦</b>									
1											
2											
3											
4											
5											
Total stormflow depth (Σ $\frac{Q_i A_i}{100}$ ) (mm)											
Total runoff volume (V, m <sup>3</sup> x 10 <sup>6</sup> ) ⑧											
<b>Peak discharge (q<sub>p</sub>, m<sup>3</sup>/s) ⑨</b>											

*Note: # Reference to the appropriate figures and tables is made in the legend table of this method.*

## SCS-SA METHOD

### LEGEND TABLE SCS-SA method

ID	Reference
①	Figure 3.12 or Figure 3E.1 to Figure 3E.6
①	Figure 3.25 or 3.26
②	Table 3E.1 or 3E.2
③	Table 3E.3
④	Table 3E.3
⑤	Table 3C.8
⑥	Table 3C.9
⑦	Table 3C.10
⑧	Table 3C.11
⑨	Table 3C.12

**Table 3C.8**  
**Adjustment of Curve Numbers**

Median Condition Method 
$$CN_f = \frac{1100}{\frac{1100}{CN - II} - \frac{\Delta S}{25,4}}$$

Wet/saturated Conditions 
$$CN_w = \frac{CN - II}{0,4036 + 0,0059CN - II}$$

**Table 3C.9**

Potential maximum soil water retention 
$$S = \frac{25400}{CN} - 254$$

**Table 3C.10**

Stormflow depth 
$$Q = \frac{(P - I_a)^2}{P - I_a + S} \text{ for } P > I_a$$

**Table 3C.11**

Stormflow volume 
$$V = \frac{QA}{1000}$$

**Table 3C.12**

Peak discharge estimation 
$$q_p = \frac{0,2083AQ}{1,83L}$$

## EMPIRICAL METHODS

Description of catchment						
River detail						
Calculated by				Date		
Physical characteristics						
Size of catchment (A)		km <sup>2</sup>	Veld type①			
Longest watercourse (L)		km	Catchment parameter (C) with regard to reaction time		$C = \frac{A\sqrt{S}}{LL_c}$	
Length to catchment centroid (L <sub>C</sub> )		km				
Average slope (S <sub>av</sub> )		m/m	Kovács region②			
Mean annual precipitation (P)③		mm				
Return period (years), T			10	20	50	100
Constant value for K <sub>T</sub> ③						
Peak flow (m³/s), Q <sub>T</sub> based on Midgley & Pitman $Q_T = 0,0377K_T PA^{0,6} C^{0,2}$						
Peak flow (m³/s), Q <sub>RMF</sub> based on Kovács④						
Return period (years), T			50	100	200	
Q <sub>T</sub> /Q <sub>RMF</sub> ratios⑤						
Peak flow (m³/s) based on Q <sub>T</sub> /Q <sub>RMF</sub> ratios						

### LEGEND TABLE Empirical methods

ID	Reference	ID	Reference	ID	Reference
①	Figure 3.5 or SA Weather Services	②	Figure 3.26	④	Table 3C.14
①	Figure 3.15	③	Table 3C.13	⑤	Table 3D.1 or 3D.2

**Table 3C.13**

Constant values of K<sub>T</sub>

Return period T in years	Constant values of K <sub>1</sub>										
	Veld type (Figure 3.15)										
	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.14**

RMF region classification in southern Africa

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

Notes:

\* RMF K value as used in Equation 3.32

# Recorded flood data are reflected in the DWAF report TR105 – Maximum flood peak discharges in South Africa: An empirical approach

**APPENDIX 3D**  
 **$Q_T/Q_{RMF}$  RATIOS FOR DIFFERENT CATCHMENT AREAS**

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**Table 3D.1:  $Q_T/Q_{RMF}$  ratios for different catchment areas in South Africa, Lesotho and Swaziland<sup>(3,13)</sup>**

Region	Return period (years)	$K_T$	Effective catchment area - $A_e$ (km <sup>2</sup> )									
			$\leq 10^*$	30*	100	300	1 000	3 000	10 000	30 000	100 000	300 000
<b>K8</b> (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			
<b>K7</b> (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			
<b>K6</b> (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		
<b>K5</b> (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	
<b>K5</b> (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				
<b>K4</b> (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	
<b>K3</b> (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
<b>K2</b> (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

Note: \* Estimated ratios

\*\* Ratios of this region may also be used in region **K1** (2,8)

**Table 3D.2:  $Q_T/Q_{RMF}$  ratios for different catchment areas in Namibia and Zimbabwe<sup>(3,13)</sup>**

Region	Return period (years)	K <sub>T</sub>	Effective catchment area - A <sub>e</sub> (km <sup>2</sup> )									
			≤ 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		

Note: \* Estimated ratios

\*\* In region **K5** use the same ratios as those applicable to South Africa

**APPENDIX 3E**  
**SCS-SA ADDITIONAL INFORMATION**

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**Table 3E.1: Example of classification of soils in southern Africa into hydrological soil groups by soil form, family and textural class (taxonomic classification)**

Soil Form	Code	Soil Family	Typical Textural Class	SCS Grouping	Soil Form	Code	Soil Family	Typical Textural Class	SCS Grouping
ADDO B	Ad 1111	Glenconnor	LmSa	A/B	ARCADIA	Ar 1100	Lonehill	Cl	C/D
	Ad 1111	Glenconnor	SaLm	B	C/D	Ar 1100	Lonehill	Cl	C/D
	Ad 1111	Glenconnor	SaClLm	B		Ar 1200	Rustenburg	Cl	C/D
	Ad 1111	Glenconnor	SaCl	B/C		Ar 1200	Rustenburg	Cl	C/D
	Ad 1112	Dalby	LmSa	A/B		Ar 2100	Minerva	Cl	C/D
	Ad 1112	Dalby	SaLm	B		Ar 2100	Minerva	Cl	C/D
	Ad 1112	Dalby	SaClLm	B		Ar 2200	Diepsloot	Cl	C/D
	Ad 1112	Dalby	SaCl	B/C		Ar 2200	Diepsloot	Cl	C/D
	Ad 1121	Centlivres	LmSa	B		Ar 3100	Bospoort	Cl	C/D
	Ad 1121	Centlivres	SaLm	B/C		Ar 3100	Bospoort	Cl	C/D
	Ad 1121	Centlivres	SaClLm	B/C		Ar 3200	Deercroft	Cl	C/D
	Ad 1121	Centlivres	SaCl	C		Ar 3200	Deercroft	Cl	C/D
	Ad 1122	Kentvale	LmSa	B	ASKHAM	Ak 1000	Aroab	LmSa	A/B
	Ad 1122	Kentvale	SaLm	B/C	B	Ak 1000	Aroab	SaLm	B
	Ad 1122	Kentvale	SaClLm	B/C		Ak 1000	Aroab	SaClLm	B
	Ad 1122	Kentvale	SaCl	C		Ak 1000	Aroab	SaCl	B/C
	Ad 1211	Spekboom	LmSa	A/B		Ak 2000	Noenieput	LmSa	B
	Ad 1211	Spekboom	SaLm	B		Ak 2000	Noenieput	SaLm	B/C
	Ad 1211	Spekboom	SaClLm	B		Ak 2000	Noenieput	SaClLm	B/C
	Ad 1211	Spekboom	SaCl	B/C		Ak 2000	Noenieput	SaCl	B/C
	Ad 1212	Gorah	LmSa	A/B	AUGRABIE	Ag 1110	Hefnaar	LmSa	A/B
	Ad 1212	Gorah	SaLm	B	B	Ag 1110	Hefnaar	SaLm	B
	Ad 1212	Gorah	SaClLm	B		Ag 1110	Hefnaar	SaClLm	B
	Ad 1212	Gorah	SaCl	B/C		Ag 1110	Hefnaar	SaCl	B/C
	Ad 1221	Walkraal	LmSa	B		Ag 1120	Giyani	LmSa	B
	Ad 1221	Walkraal	SaClLm	B/C		Ag 1120	Giyani	SaLm	B/C
	Ad 1221	Walkraal	SaCl	C		Ag 1120	Giyani	SaClLm	B/C
	Ad 1222	Sylvania	LmSa	B		Ag 1120	Giyani	SaCl	C
	Ad 1222	Sylvania	SaLm	B/C		Ag 1210	Khubus	LmSa	A/B
	Ad 1222	Sylvania	SaClLm	B/C		Ag 1210	Khubus	SaLm	B
	Ad 1222	Sylvania	SaCl	C		Ag 1210	Khubus	SaClLm	B
ADDO B	Ad 2111	Maurmond	LmSa	A/B		Ag 1210	Khubus	SaCl	B/C
	Ad 2111	Maurmond	SaLm	B		Ag 1220	Shilowa	LmSa	B
	Ad 2111	Maurmond	SaClLm	B	<b>LEGEND</b>				
	Ad 2111	Maurmond	SaCl	B/C	A	-	low runoff potential		
	Ad 2112	Airedale	LmSa	A/B	B	-	moderately low potential		
	Ad 2112	Airedale	SaLm	B	C	-	moderately high potential		
	Ad 2112	Airedale	SaClLm	B	D	-	high runoff potential		
	Ad 2112	Airedale	SaCl	B/C	Sa	-	sand		
	Ad 2121	Felsenheim	LmSa	B	Cl	-	clay		
	Ad 2121	Felsenheim	SaLm	B/C	Lm	-	loam		
	Ad 2121	Felsenheim	SaClLm	B/C					
	Ad 2121	Felsenheim	SaCl	C					
	Ad 2122	Longhill	LmSa	B					
	Ad 2122	Longhill	SaLm	B/C					
	Ad 2122	Longhill	SaClLm	B/C					
	Ad 2122	Longhill	SaCl	C					
	Ad 2211	Mimosa	LmSa	A/B					
	Ad 2211	Mimosa	SaLm	B					
	Ad 2211	Mimosa	SaClLm	B					
	Ad 2211	Mimosa	SaCl	B/C					
	Ad 2212	Peperboom	LmSa	A/B					
	Ad 2212	Peperboom	SaLm	B					
	Ad 2212	Peperboom	SaClLm	B					
	Ad 2212	Peperboom	SaCl	B/C					
	Ad 2221	Suttondale	LmSa	B					
	Ad 2221	Suttondale	SaLm	B/C					
	Ad 2221	Suttondale	SaClLm	B/C					
	Ad 2221	Suttondale	SaCl	C					
	Ad 2222	Tregaron	LmSa	B					
	Ad 2222	Tregaron	SaClLm	B/C					
	Ad 2222	Tregaron	SaCl	C					

**Table 3E.2: Example of classification of soils in southern Africa into hydrological soil groups by soil form and series (binomial classification)**

Soil Form	Code	Soil Series	Typical Textural Class	SCS Grouping	Soil Form	Code	Soil Series	Typical Textural Class	SCS Grouping
ARCADIA C/D	Ar 40	Arcadia	Cl	C/D	BAINSVLEI	Bv 14	Wykeham	SaLm	A/B
	Ar 11	Bloukrans	Cl	C/D	BONHEIM C	Bo 41	Bonheim	LmSa	C/D
	Ar 21	Clerkness	Cl	C/D		Bo 20	Bushman	SaClLm	C
	Ar 41	Eenzaam	Cl	C/D		Bo 30	Dumasi	SaClLm	C
	Ar 20	Gelykvlake	Cl	C/D		Bo 31	Glengazi	SaCl	C/D
	Ar 10	Mngazi	Cl	C/D		Bo 10	Kiora	SaClLm	C
	Ar 32	Nagana	Cl	C/D		Bo 21	Rasheni	SaCl	C/D
	Ar 12	Noukloof	Cl	C/D		Bo 11	Stanger	SaCl	C/D
	Ar 31	Rooibdraai	Cl	C/D		Bo 40	Weenen	SaClLm	C
	Ar 30	Rydalvale	Cl	C/D	CARTREF C	Cf 10	Amabele	LmSa	B/C
	Ar 42	Wanstead	Cl	C/D		Cf 12	Arrochar	SaClLm	C
	Ar 22	Zwaarkrygen	Cl	C/D		Cf 13	Byrne	SaCl	C/D
AVALON B	Av 13	Ashton	SaLm	A/B		Cf 21	Cartref	SaLm	C
	Av 26	Avalon	SaClLm	B		Cf 22	Cranbrook	SaClLm	C
	Av 12	Banchory	Sa	A		Cf 30	Grovedale	Sa	B/C
	Av 27	Bergville	SaCl	B/C		Cf 31	Kusasa	SaLm	B/C
	Av 37	Bezuidenhout	SaCl	C		Cf 32	Noodhulp	SaClLm	C
	Av 33	Bleeksand	SaLm	B/C		Cf 11	Rutherglen	SaLm	C
	Av 34	Heidelberg	SaLm	B/C		Cf 20	Waterridge	LmSa	B/C
	Av 20	Hobeni	LmSa	A/B	CHAMPAGNE D	Ch 11	Champagne	SaLm	D
	Av 14	Kanhym	SaLm	A/B		Ch 21	Ivanhoe	SaClLm	D
	Av 24	Leksand	SaLm	B		Ch 10	Mposa	SaLm	D
	Av 10	Mastaba	LmSa	A		Ch 20	Stratford	SaClLm	D
AVALON B	Av 32	Middelpoos	Sa	B	CLOVELLY A/B	Cv 33	Annandale	SaLm	B
	Av 31	Mooiveld	LmSa	B		Cv 18	Balgowan	Cl	B
	Av 25	Newcastle	SaLm	A/B		Cv 40	Bleskop	LmSa	A
	Av 17	Normandien	SaCl	B		Cv 36	Blinkklip	SaClLm	B
	Av 22	Rosdale	Sa	A/B		Cv 17	Clovelly	SaCl	B
	Av 16	Ruston	SaClLm	B		Cv 28	Clydebank	Cl	B
	Av 36	Soetmelk	SaClLm	B/C		Cv 35	Denhere	SaLm	A/B
	Av 21	Uithoek	LmSa	A/B		Cv 46	Dudfield	SaClLm	A/B
	Av 30	Viljoenskroon	LmSa	B		Cv 11	Geelhout	LmSa	A
	Av 23	Villiers	SaLm	B		Cv 25	Gutu	SaLm	A
	Av 11	Welverdien	LmSa	A		Cv 47	Klippan	SaCl	B
	Av 35	Windmeul	SaLm	B		Cv 38	Klipputs	Cl	B/C
	Av 15	Wolweberg	SaLm	A		Cv 10	Lismore	LmSa	A
BAINSVLEI A/B	Bv 23	Ashkelon	SaLm	A/B	<b>LEGEND</b>				
	Bv 36	Bainsvlei	SaClLm	B	A	-	low stormflow potential		
	Bv 12	Camelot	Sa	A	B	-	moderately low potential		
	Bv 20	Chelsea	LmSa	A	C	-	moderately high potential		
	Bv 30	Delwery	LmSa	A/B	D	-	high stormflow potential		
	Bv 13	Dunkeld	SaLm	A/B	Sa	-	sand		
	Bv 16	Elysium	SaClLm	A/B	Cl	-	clay		
	Bv 10	Hlatini	LmSa	A	Lm	-	loam		
	Bv 34	Kareekuul	SaLm	B					
	Bv 31	Kingston	LmSa	A/B					
	Bv 26	Lonetree	SaClLm	A/B					
	Bv 25	Maanhaar	SaLm	A					
	Bv 11	Makong	LmSa	A					
	Bv 27	Metz	SaCl	B					
	Bv 22	Oosterbeek	Sa	A					
	Bv 37	Ottosdal	SaCl	B/C					
	Bv 24	Redhill	SaLm	A/B					
	Bv 32	Trekboer	Sa	A/B					
	Bv 15	Tygerkloof	SaLm	A					
	Bv 33	Vermaas	SaLm	B					
	Bv 21	Vungama	LmSa	A					
	Bv 35	Wedgewood	SaLm	A/B					
	Bv 17	Wilgenhof	SaCl	B					

**Table 3E.3 Initial Curve Numbers for selected land cover and treatment classes, stormflow potentials and hydrological soil groups (various sources)**

Land Cover Class	Land Treatment/ Practice/Description	Stormflow Potential	Hydrological Soil Group						
			A	A/B	B	B/C	C	C/D	D
Fallow	1 = Straight row		77	82	86	89	91	93	94
	2 = Straight row + conservation tillage	High	75	80	84	87	89	91	92
	3 = Straight row + conservation tillage	Low	74	79	83	85	87	89	90
Row Crops	1 = Straight row	High	72	77	81	85	88	90	91
	2 = Straight row	Low	67	73	78	82	85	87	89
	3 = Straight row + conservation tillage	High	71	75	79	83	86	88	89
	4 = Straight row + conservation tillage	Low	64	70	75	79	82	84	85
	5 = Planted on contour	High	70	75	79	82	84	86	88
	6 = Planted on contour	Low	65	69	75	79	82	84	86
	7 = Planted on contour + conservation tillage	High	69	74	78	81	83	85	87
	8 = Planted on contour + conservation tillage	Low	64	70	74	78	80	82	84
	9 = Conservation structures	High	66	70	74	77	80	82	82
	10 = Conservation structures	Low	62	67	71	75	78	80	81
	11 = Conservation structures + conservation tillage	High	65	70	73	76	79	80	81
	12 = Conservation structures + conservation tillage	Low	61	66	70	73	76	78	79
Garden Crops	1 = Straight row	Low	45	56	66	72	77	80	83
	2 = Straight row	High	68	71	75	79	81	83	84
Small Grain	1 = Straight row	High	65	71	76	80	84	86	88
	2 = Straight row	Low	63	69	75	79	83	85	87
	3 = Straight row + conservation tillage	High	64	70	74	78	82	84	86
	4 = Straight row + conservation tillage	Low	60	67	72	76	80	82	84
	5 = Planted on contour	High	63	69	74	79	82	84	85
	6 = Planted on contour	Low	61	67	73	78	81	83	84
	7 = Planted on contour + conservation tillage	High	62	68	73	77	81	83	84
	8 = Planted on contour + conservation tillage	Low	60	66	72	76	79	81	82
	9 = Planted on contour - winter rainfall region	Low	63	66	70	75	78	80	81
	10 = Conservation structures	High	61	67	72	76	79	81	82
	11 = Conservation structures	Low	59	65	70	75	78	80	81
	12 = Conservation structures + conservation tillage	High	60	67	71	75	78	80	81
	13 = Conservation structures + conservation tillage	Low	58	64	69	73	76	78	79
Close Seeded Legumes or Rotational Meadow	1 = Straight Row	High	66	72	77	81	85	87	89
	2 = Straight Row	Low	58	65	72	75	81	84	85
	3 = Planted on contour	High	64	70	75	80	83	84	85
	4 = Planted on contour	Low	55	63	69	74	78	81	83
	5 = Conservation structures	High	63	68	73	77	80	82	83
	6 = Conservation structures	Low	51	60	67	72	76	78	80
Sugarcane	1 = Straight row: trash burnt		43	55	65	72	77	80	82
	2 = Straight row: trash mulch		45	56	66	72	77	80	83
	3 = Straight row: limited cover		67	73	78	82	85	87	89
	4 = Straight row: partial cover		49	60	69	73	79	82	84
	5 = Straight row: complete cover		39	50	61	68	74	78	80
	6 = Conservation structures: limited cover		65	70	75	79	82	84	86
	7 = Conservation structures: partial cover		25	46	59	67	75	80	83
	8 = Conservation structures: complete cover		6	14	35	59	70	75	79

**Table 3E.3 Initial Curve Numbers for selected land cover and treatment classes, stormflow potentials and hydrological soil groups (various sources) (continued)**

Land Cover Class	Land Treatment/ Practice/Description	Stormflow Potential	Hydrological Soil Group						
			A	A/B	B	B/C	C	C/D	D
Veld (range) and Pasture	1 = Veld/pasture in poor condition	High	68	74	79	83	86	88	89
	2 = Veld/pasture in fair condition	Moderate	49	61	69	75	79	82	84
	3 = Veld/pasture in good condition	Low	39	51	61	68	74	78	80
	4 = Pasture planted on contour	High	47	57	67	75	81	85	88
	5 = Pasture planted on contour	Moderate	25	46	59	67	75	80	83
	6 = Pasture planted on contour	Low	6	14	35	59	70	75	79
Irrigated Pasture		Low	35	41	48	57	65	68	70
Meadow		Low	30	45	58	65	71	75	81
Woods and Scrub	1 = Woods	High	45	56	66	72	77	80	83
	2 = Woods	Moderate	36	49	60	68	73	77	79
	3 = Woods	Low	25	47	55	64	70	74	77
	4 = Brush - Winter rainfall region	Low	28	36	44	53	60	64	66
Orchards	1 = Winter rainfall region, understory of crop cover		39	44	53	61	66	69	71
Forests & Plantations	1 = Humus depth 25mm; Compactness:	compact	52	62	72	77	82	85	87
	2 = " " "	moderate	48	58	68	73	78	82	85
	3 = " " "	loose/friable	37	49	60	66	71	74	77
	4 = Humus depth 50mm; Compactness:	compact	48	58	68	73	78	82	85
	5 = " " "	moderate	42	54	65	70	75	78	81
	6 = " " "	loose/friable	32	45	57	62	67	71	74
	7 = Humus depth 100mm; Compactness:	compact	41	53	64	69	74	77	80
	8 = " " "	moderate	34	47	59	64	69	72	75
	9 = " " "	loose/friable	23	37	50	56	61	64	67
	10 = Humus depth 150mm; Compactness:	compact	37	49	60	66	71	74	77
	11 = " " "	moderate	30	43	56	61	66	69	72
	12 = " " "	loose/friable	18	33	47	52	57	61	65
Urban/Sub-urban Land Uses	1 = Open spaces, parks, cemeteries	75% grass cover	39	51	61	68	74	78	80
	2 = Open spaces, parks, cemeteries	75% grass cover	49	61	69	75	79	82	84
	3 = Commercial/business areas	85% grass cover	89	91	92	93	94	95	95
	4 = Industrial districts	72% impervious	81	85	88	90	91	92	93
	5 = Residential: lot size 500m <sup>2</sup>	65% impervious	77	81	85	88	90	91	92
	6 = " " 1000m <sup>2</sup>	38% impervious	61	69	75	80	83	85	87
	7 = " " 1350m <sup>2</sup>	30% impervious	57	65	72	77	81	84	86
	8 = " " 2000m <sup>2</sup>	25% impervious	54	63	70	76	80	83	85
	9 = " " 4000m <sup>2</sup>	20% impervious	51	61	68	75	78	82	84
	10 = Paved parking lots, roofs, etc.		98	98	98	98	98	98	98
	11 = Streets/roads: tarred, with storm sewers, curbs		98	98	98	98	98	98	98
	12 = " gravel		76	81	85	88	89	90	91
	13 = " dirt		72	77	82	85	87	88	89
	14 = " dirt-hard surface		74	79	84	88	90	91	92

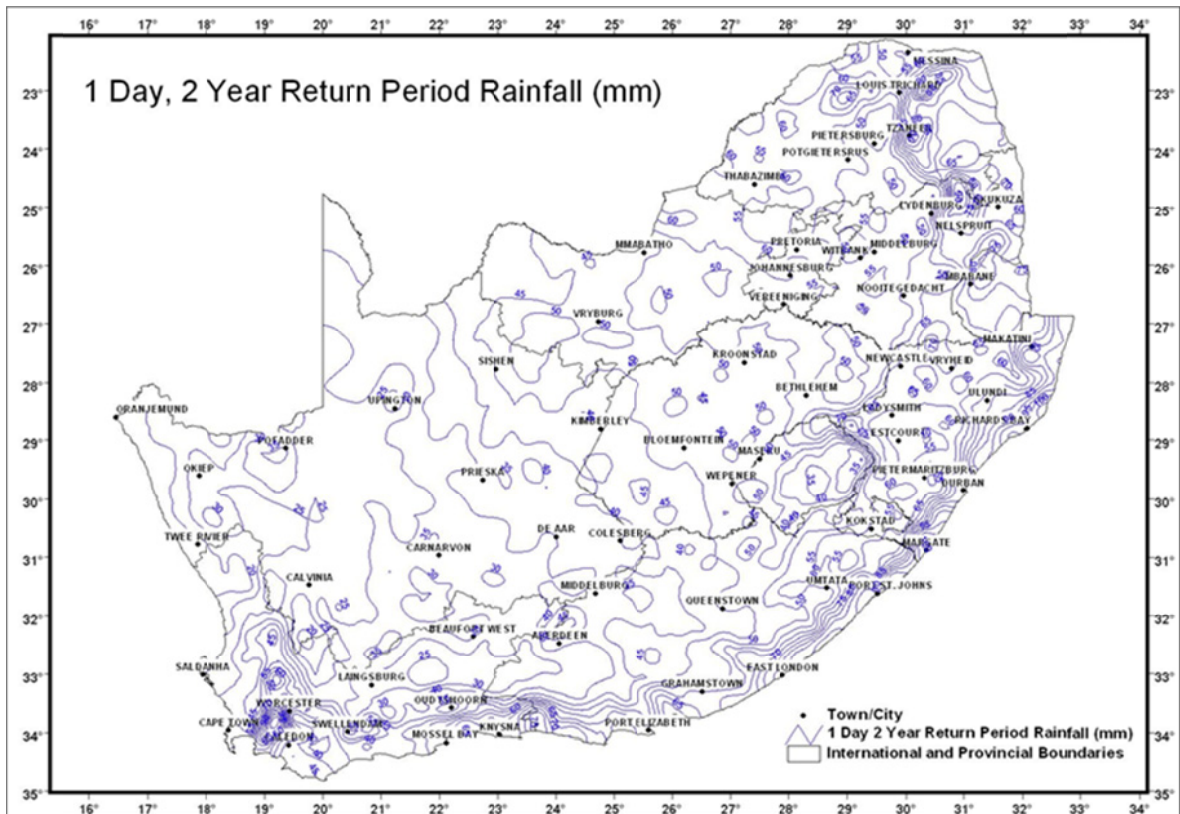


Figure 3E.1: One-day design rainfall distribution over southern Africa for 2 year return period

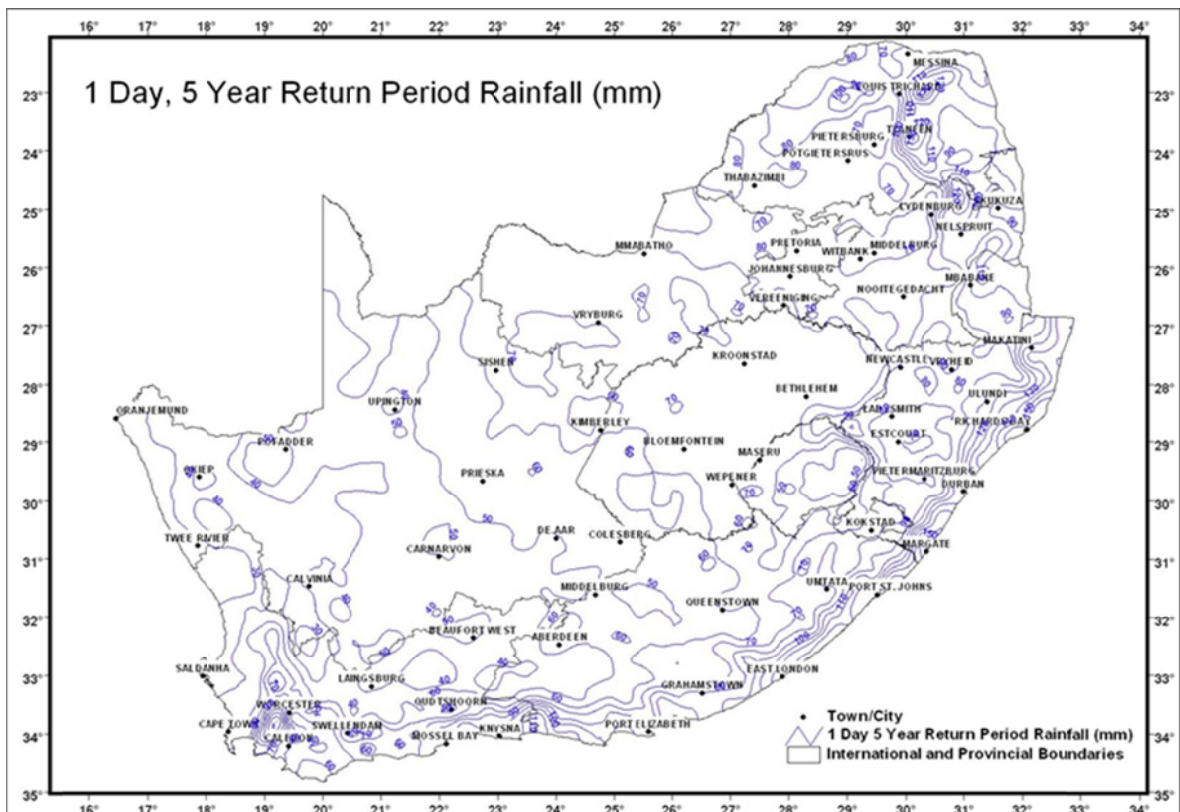


Figure 3E.2: One-day design rainfall distribution over southern Africa for 5 year return period

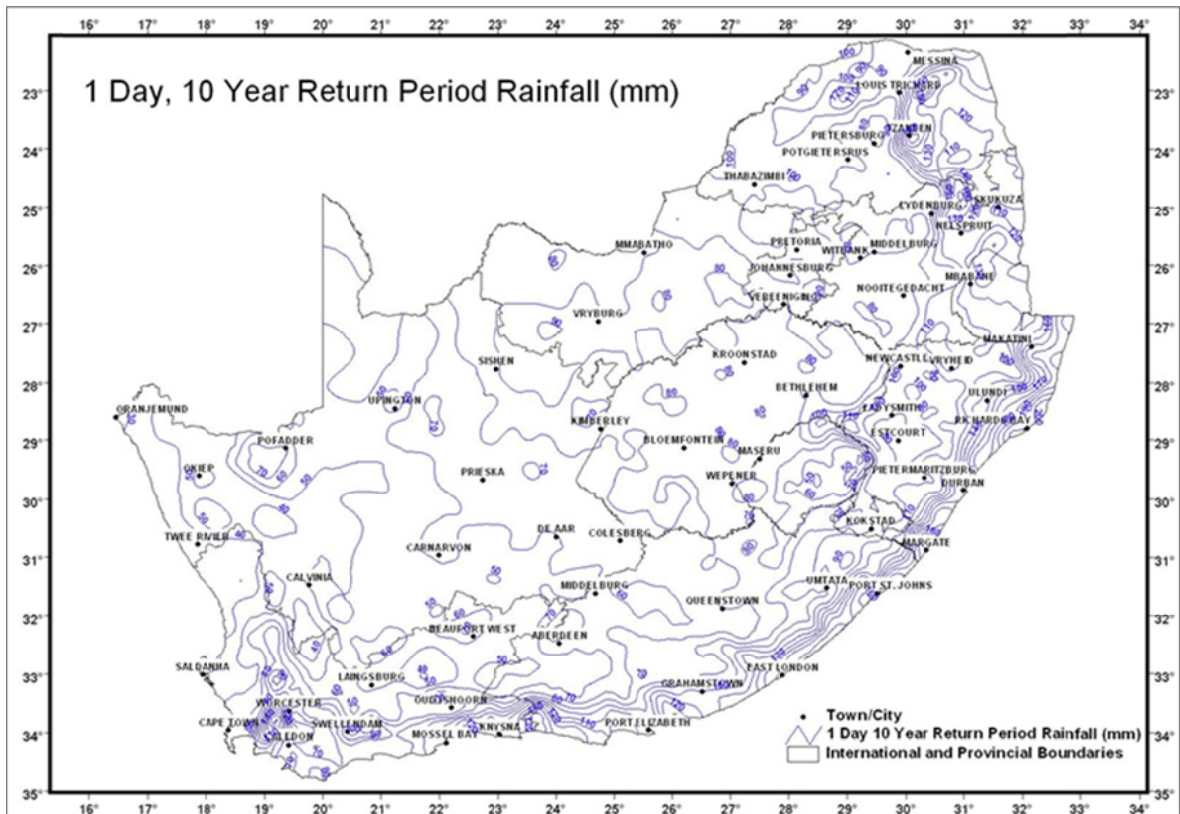


Figure 3E.3: One-day design rainfall distribution over southern Africa for 10 year return period

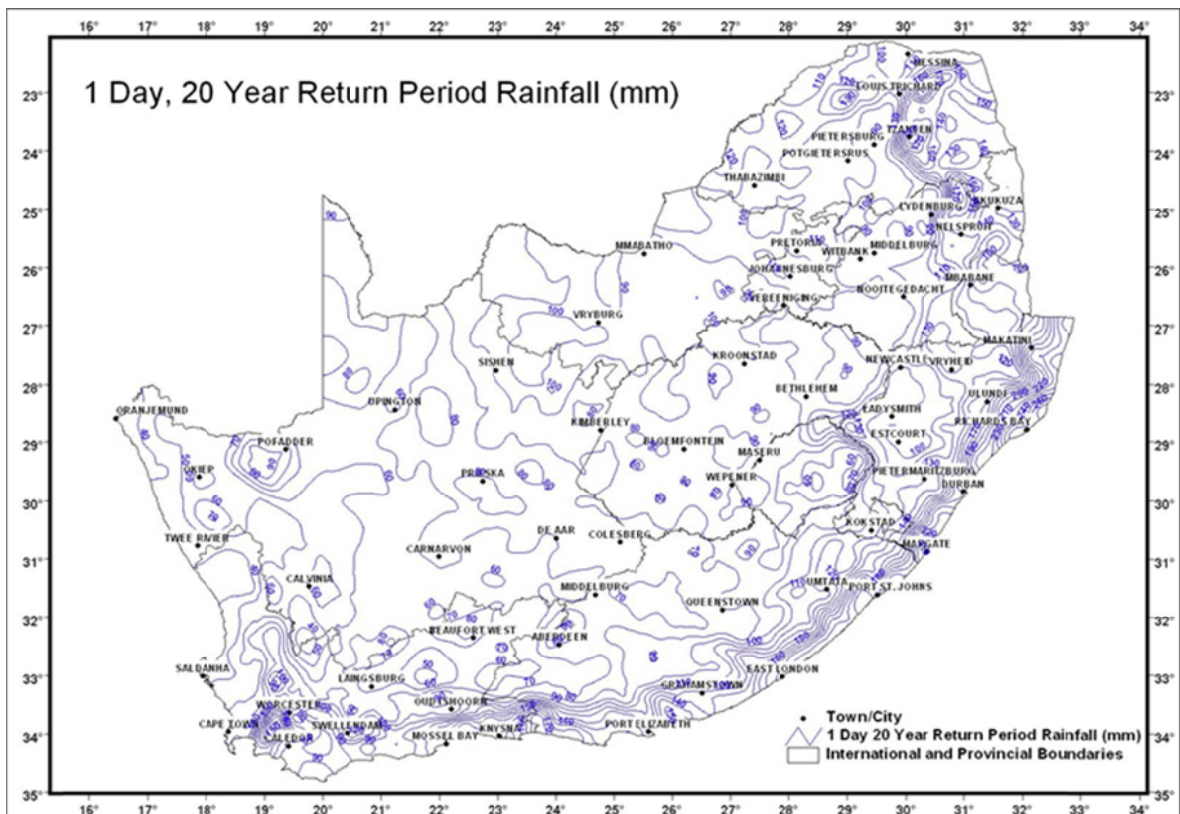


Figure 3E.4: One-day design rainfall distribution over southern Africa for 20 year return period



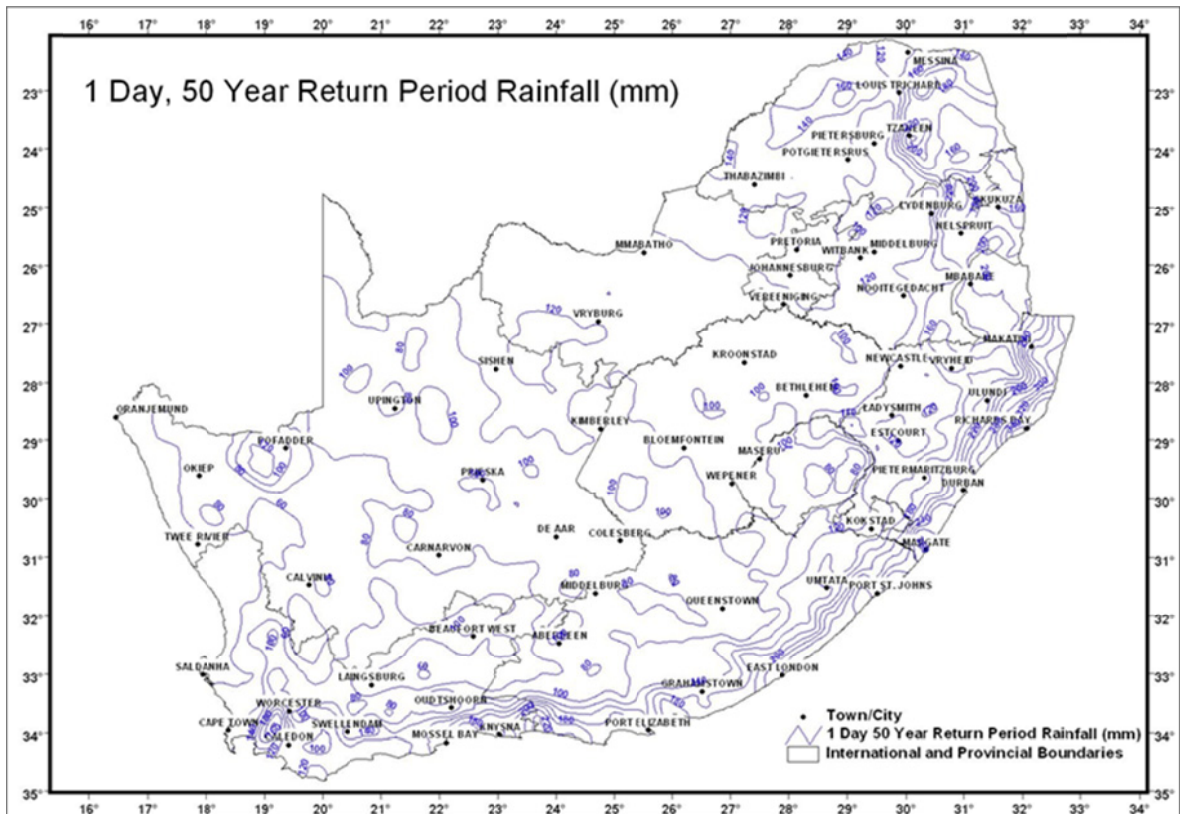


Figure 3E.5: One-day design rainfall distribution over southern Africa for 50 year return period

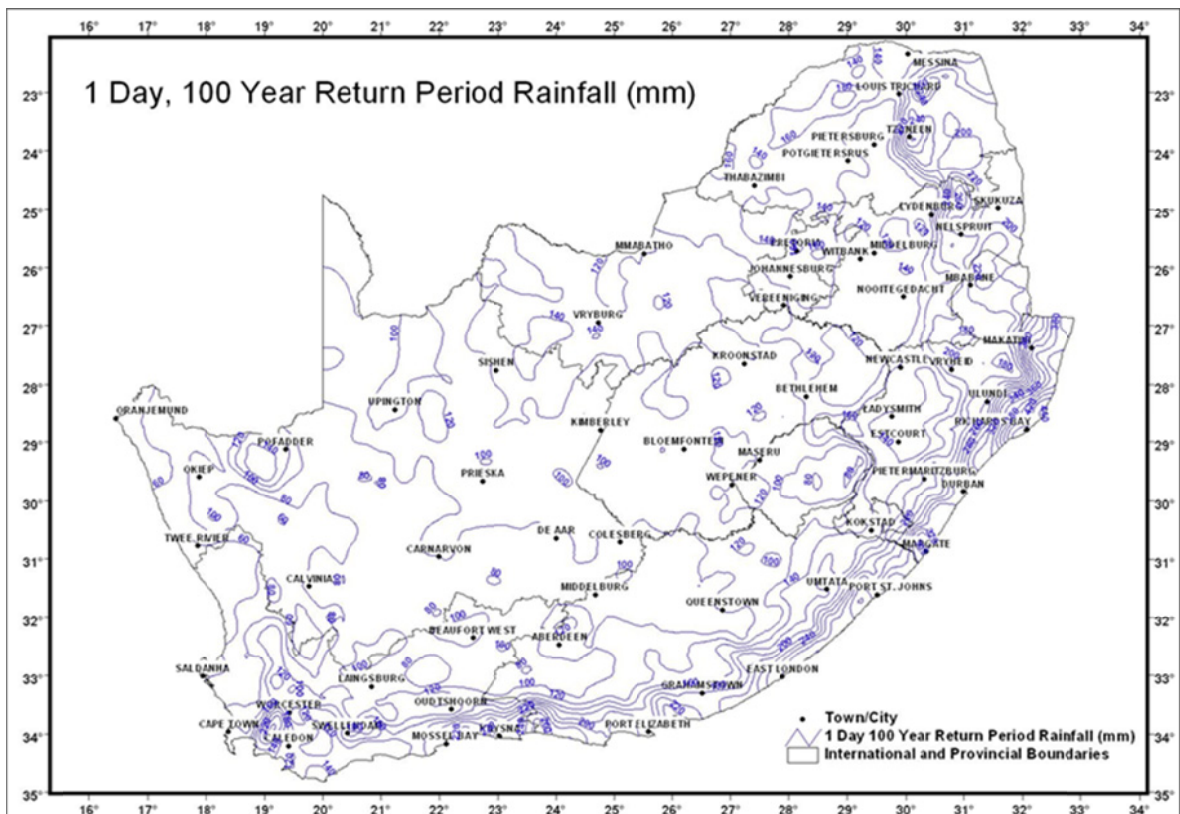


Figure 3E.6: One-day design rainfall distribution over southern Africa for 100 year return period