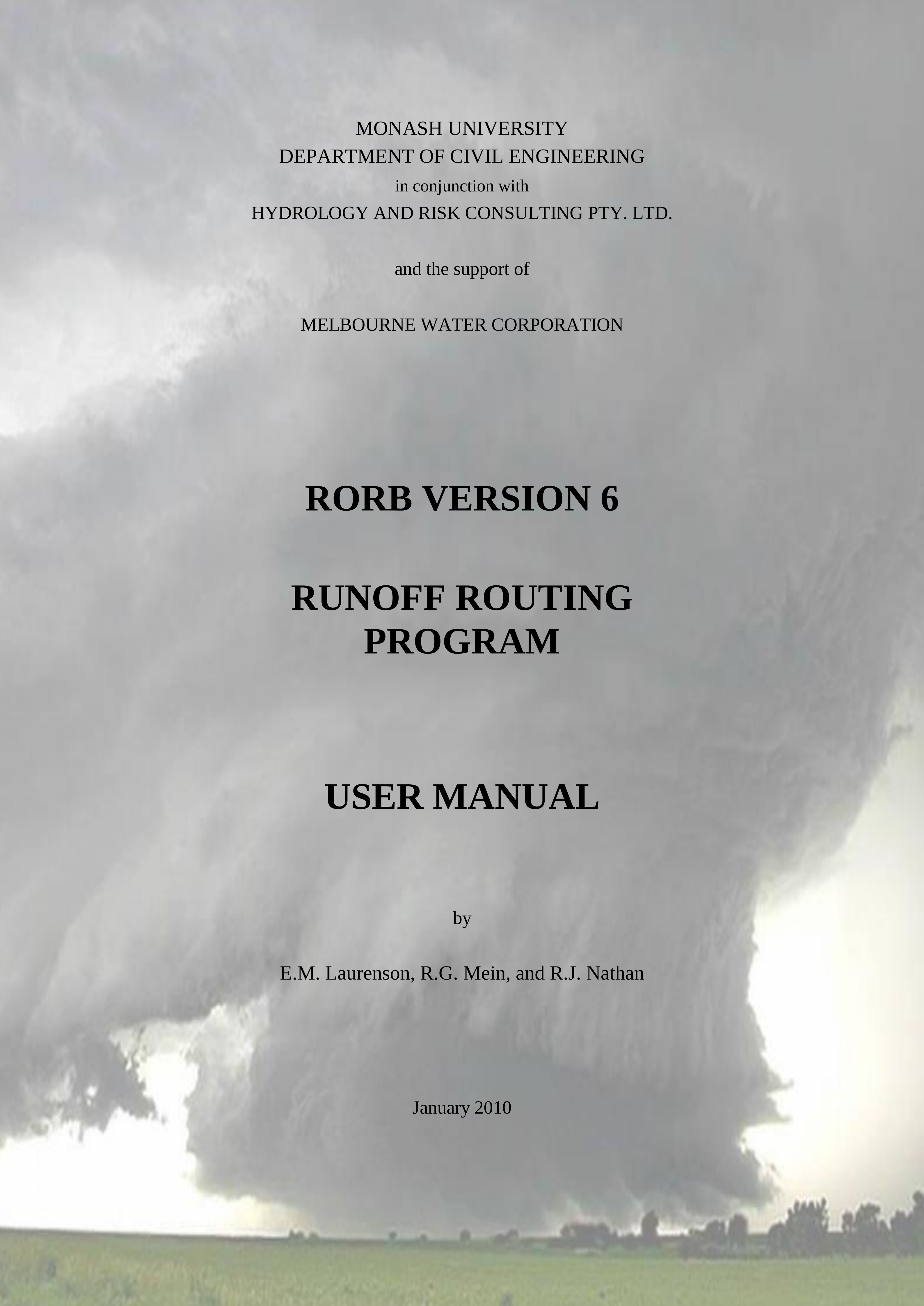

MONASH UNIVERSITY
DEPARTMENT OF CIVIL ENGINEERING
in conjunction with
HYDROLOGY AND RISK CONSULTING PTY. LTD.

and the support of

MELBOURNE WATER CORPORATION

# RORB VERSION 6

# RUNOFF ROUTING PROGRAM

# USER MANUAL

by

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January 2010

## Document history and status

| Date | Revision | Description of Change |
|---|---|---|
| 8 October 2007 | V0.0 | First release of RORB Version 6 |
| 14 December 2007 | V0.1 | Typos fixed for release on web-site |
| 11 January 2010 | V0.2 | First release of RORB Version 6.14 |
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| | | |
| | | |

# Summary

RORB is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall-excess and routes this through catchment storage to produce runoff hydrographs at any location. It can also be used to design retarding basins and to route floods through channel networks.

The program requires a datafile to describe the particular features of the stream network being modelled and is run interactively. It can be used both for the calculation of design hydrographs and for model calibration by fitting to rainfall and runoff data of recorded events.

The model is areally distributed, nonlinear, and applicable to both urban and rural catchments. It makes provision for temporal and areal variation of rainfall and losses and can model flows at any number of gauging stations. In addition to normal channel storage, specific modeling can be provided for retarding basins, storage reservoirs, lakes or large flood plain storages. Base flow and other channel inflow and outflow processes, both concentrated and distributed, can be modeled.

RORB is fully Windows compatible, with considerable enhancements over previous versions. The new features include the provision to vary parameters over sub-catchments, additional features in the specification of design storms and special storages, the facility for multiple (or batch runs) for a range of ARI and/or duration, extended interactive graphics capability, and Monte Carlo simulation.

Version 6 adds a graphical user interface (GUI) for catchment data input/revision on the screen.

# Foreword

RORB Version 6 is the result of a collaboration between Monash University's Department of Civil Engineering and Sinclair Knight Merz, with support from the Melbourne Water Corporation. Hydrology and Risk Consulting is the custodian of the program and makes it freely available in the public domain for download by those who wish to use it.

Essentially the original FORTRAN code for the mainframe version of RORB 4 has been incorporated as a sub-routine to a Windows graphical user interface for Version 5. Hence, 'old' RORB datafiles are fully compatible with this version. The Windows environment considerably enhances fitting the model to recorded data, and for interactive design of storages. There are new features to make file handling easier, and many more options for generated design storms. Model parameters can be adjusted while simultaneously viewing plots of hydrograph response. Batch runs (for a range of ARI and/or duration) are now possible. The graphical environment has been completely overhauled, and there are many options for the viewing and export of hydrograph plots. A new control code (Code 7.2) has been added to create interstation areas for areal variation of model parameters. For the more advanced user, the facility has been provided for Monte Carlo simulations.

Version 6 adds a graphical user interface (GUI) for on-screen data input (Chapter 6).

The User Manual has been revised and updated to reflect these changes, whilst still largely retaining its previous structure. Chapters 6 (GUI) and Chapter 8 (Monte Carlo simulation) are completely new. Chapter 1 (Introduction) and Chapter 10 (Examples) have had major changes, with (mostly) only minor changes of the remaining chapters being necessary. Experienced RORB users will note the incorporation of model testing (the TEST run) into the FIT Run framework to better reflect current practice.

***A note of caution***: *Because this version of RORB is fully Windows compatible, it is much more intuitive for users to run. Nevertheless, it is stressed that this enhanced ease-of-use should not be seen as a substitute for hydrological knowledge and critical judgement, or for less reliance on the User Manual and authoritative sources of information.*

RGM, RJN
August 2007

Eric Laurenson passed away in April 2003. He was the initial driving force for a public-domain flood-estimation program, and a key contributor in the development of Versions 1 to 4 of RORB. We believe he would be much satisfied with its continued evolution from the foundations he did so much to establish.

# Disclaimer

The RORB Version 6 software and its related documentation have been produced collaboratively by a number of parties, with the intellectual property rights now held by the Department of Civil Engineering (Monash University) and Hydrology and Risk Consulting Pty Ltd ABN 87 553 755 959. The program is subject to a licence agreement (reproduced below) that must be agreed to prior to its use.

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# Guide to Manual



The electronic form of this document contains many hypertext links, indicated by either underlined blue text, or by a change in cursor shape when moved over the word of interest. Clicking on these links will “jump” the viewing pane to that location. Adobe Reader software provides a button to enable a jump back.

# Contents

# 1 INTRODUCTION

### 1.1 Purpose of Program

RORB is an interactive runoff and streamflow routing program that calculates catchment losses and streamflow hydrographs resulting from rainfall events and/or other forms of inflow to channel networks. It is used for:

- flood estimation;
- spillway and retarding basin[1] design; and
- flood routing.

In flood estimation applications, the program may be used on rural, urban or partly rural and partly urban catchments. It is mostly used for design flood investigations but, if the user can provide independently a procedure for evaluating the loss parameters in real time, it may also be used in flood forecasting. In retarding basin and spillway design applications, the program calculates the design inflow hydrograph, provides for interactive adjustment of outlet dimensions until a design criterion is met, and can then route the outflow hydrograph further downstream. In flood routing applications, single and multiple reaches, networks of streams and lateral inflow and outflow can be modeled.

The program provides an event-type modeling procedure whose general concept is illustrated in Figure 1-1. Rainfall is operated on by a loss model to produce rainfall-excess. Rainfall and loss are processes that occur on the catchment surface before the water enters the channel network. The rainfall-excess is operated on by a catchment storage model representing the effects of overland flow storage and channel storage to produce the surface runoff hydrograph. Information on the development and background theory of the runoff routing model employed in this program is contained in Appendix A and References 1 to 7.

Water may also enter the channel network otherwise than directly from rainfall, and diversions from the channel may also occur. Such channel inflow and outflow hydrographs may be concentrated at a point or distributed along the channel. The distributed processes include base flow, lateral inflow, transmission loss, and lateral outflow. These features enable the program to be used for flood routing in channel networks without any catchment area, rainfall, or losses being directly involved in the event modeled.

The program is widely used in Australia and, to a more limited extent, overseas (eg. Refs. 16, 23, 24).

[1]Retarding basin is Australian usage for what is elsewhere often referred to as a 'detention basin' or 'detention storage'. The Australian term is used herein.

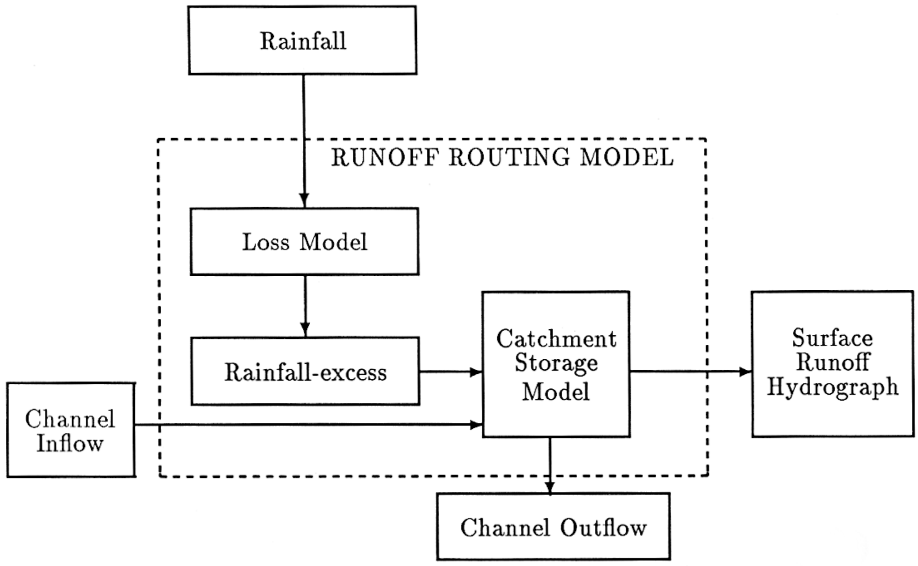


**Figure 1-1 Overall Runoff Routing Model**

## 1.2 Usage of Program

The program is intended to be usable on any catchment or stream network. A general form of catchment model is used by the program and this can be adapted to represent almost any of the less general models extant, if desired. The model is areally distributed, nonlinear, and based upon a storage routing procedure. Although the program is capable of modeling very complicated situations, its application to simple problems requires the minimum of input.

In typical flood estimation, applications, the catchment is divided into sub-areas bounded by drainage divides. Rainfall on each sub-area is adjusted to allow for infiltration and other losses. A sub-area rainfall-excess is assumed to enter the channel network at a point near the centroid of the sub-area. There, it is added to any existing flow in the channel, and the combined flow is routed through a storage by a linear or nonlinear storage routing procedure based on continuity and a storage function $S = kQ^m$ (see Section 2.2.1 and Appendix A).

The overall catchment storage is represented in the model by a network of such storages arranged like the actual channel network. Each model storage represents the actual storage between two nodes of the model. The nodes represent sub-area inflow points, stream confluences, inflow points to storage reservoirs, and other points of interest on the catchment or channel network.

When the rainfall-excess from the first sub-area has been routed through the first model storage, the rainfall-excess from the second sub-area is added and the total routed through the next storage, and so on. At a confluence, the hydrograph is stored until the hydrograph from the other branch has been calculated in the same way. The two branch hydrographs are then added together and the combined hydrograph routed through the next storage. Each such normal reach storage has a parameter proportional to its length or some other function of known reach properties and also to an empirical coefficient fitted for the catchment. Storage reservoirs, retarding basins and other special storages have their storage-discharge relations defined in a variety of ways independently of the fitted coefficient. The sequence of adding in sub-area inflows, routing through normal and special storages, and storing and adding hydrographs a confluences, proceeds in a downstream direction until the catchment outlet is reached. Calculated hydrographs can be printed out at any time.

It is possible to superimpose on the above processes concentrated inflows to or diversions from the channel and also distributed lateral inflows to or outflows from the channel network. Also, it is not essential to have a catchment area, rainfall and losses; channel networks alone can be modeled. It is even possible to model simply special storages, with no channel network nor catchment area.

In the program, the sequence of operations used to model a particular catchment/stream situation is defined by a series of numerical codes. The data relevant to each code is stored with that code in a data file. Full details of all the above matters are given in succeeding chapters of this manual.

Two routing parameters and the initial loss must be evaluated to fit the catchment model to a particular event on a particular catchment or sub-catchment. If rainfall and streamflow data are available for an event, the program may be used to calibrate the model through an interactive, trial and error fitting procedure and also to provide some testing of the model. It may also be used with known model parameters and assumed rainfall and/or channel input data to calculate design hydrographs.

The functions of fitting/testing and design are associated with different types of run, whose characteristics are summarized in Table 1-1.

**Table 1-1 Types of Run**

| **TYPE OF STUDY** | **ITEM** | | **TYPE OF RUN** | |
|---|---|---|---|---|
| | | | **FIT (Test)** | **DESIGN** |
| CATCHMENT STUDIES & FLOOD ROUTING STUDIES | HYDROGRAPH | | Known | Unknown |
| | MODEL PARAMETERS | | Unknown (or assumed) | Known or assumed |
| | PURPOSE OF RUN | | To determine (or confirm) model parameters | To determine design hydrograph |
| | OUTPUT LABELLED: | | FIT run | DESIGN run |
| | SPECIAL STORAGES | | Existing only | Existing or to be designed |
| CATCH MENT STUDIES ONLY | TRIAL ROUTING PARAMETERS | | Suggested by program | Suggested, but $k_c$ only if $m$ = 0.8 |
| | LOSS PARA-METER | GAUGING STATION | Calculated | Input |
| | | NO GAUGING STATION | Input from terminal | Input from terminal |

On gauged catchments, FIT runs would be used first with data for one or more of the available recorded floods to evaluate the model's parameters. Further runs can be used next with data for the remainder of the recorded floods to test the model. If the fits are regarded as satisfactory, DESIGN runs would then be used to predict the catchment behaviour under various hypothetical design conditions.

At the conclusion of each run, the user has the option of changing the parameters and running again with the new values without re-reading and checking the data and with no unnecessary recomputation or output. The loss model may also be changed in the same way. Re-runs may also use the current parameters (useful for re-designing special storages, and obtaining screen or hard-copy plots). Thirdly, re-runs may use a new data file or the same data file with modified output.

On ungauged catchments, FIT runs are used only under unusual circumstances (see Section 7.2, Item 12); normally only DESIGN runs are used. In these cases, parameter values must be provided by the user. Some suggestions as to appropriate values are made in Sections 2.2.1, 2.2.4 and 3.4.2.

To use the program the user must firstly prepare a data file (Chapters 5 & 6). The program is then run interactively, and functions as shown in Figure 1-2.

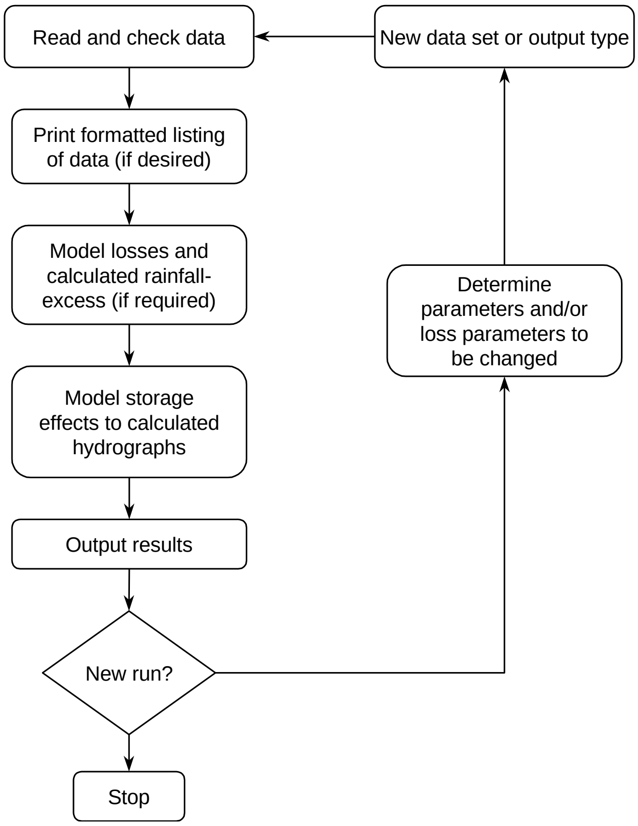


**Figure 1-2 Functional Arrangement of Program**

A large number of checks on the data values have been programmed. To assist the user, the file data are read and checked before any significant computation or output occurs. If a data error is detected, an informative message is printed and the program returns to the main menu. The data file must then be corrected before proceeding.

The extent of the output is controlled by the user, partly through the data file (eg. number of print locations) and partly interactively (eg through the level of detail selected for printed output).

## 1.3 Windows Overview

The aim of this version of RORB is to provide users with:

- a standard Microsoft look-and-feel;
- access to standard Windows features such as network installation, long filenames, directory navigation, the sending of numerical and graphical output to networked printers, and the saving of graphical output to windows metafiles (for import into other Windows programs such as Microsoft Word);
- context sensitive menu and dialogue interaction - this particularly benefits novice users as only those options relevant to the selected configuration are activated;
- access to Windows clipboard (for transferring information between other Windows programs); and,
- access to “on-line” context-sensitive help information – information on dialog-specific functionality is provided via a “help” button on each individual dialogue, and overall help and program guidance is documented in this User Manual.

In addition to the windows environment, the major changes introduced since Version 4 include the ability to:

- perform batch runs (varying ARI, duration, and/or runoff coefficients);
- carry out a Monte Carlo analysis to derive flood frequency curves;
- use up-dated information on areal reduction factors with generated design storms;
- provide users with additional information of regional values, and/or assign different $k_c$ and $m$ parameters to parts of a catchment; and,
- alter model parameters simultaneously while viewing model output
- create a catchment datafile for RORB using a graphical user interface (new with Version 6).

More detailed information on each of these is given in later sections of this manual.

## 1.4 Files Provided

The download (or CD) of RORB6 should include the following files:

| | |
|---|---|
| RORBWin.exe | the application file for RORB |
| WERFIT.DAT | sample data file (Werribee River FIT run) |
| WERDES.DAT | sample data file (Werribee River DESIGN run) |
| TOMFIT.CAT | sample catchment file (Thomson River FIT run) |

| | |
|---|---|
| TOMNOV71.STM | sample storm file (Thomson River FIT run) |
| TOMDES.DAT | sample data file (Thomson River DESIGN run) |
| TOMDES.CAT | sample catchment file (Thomson River DESIGN run – for use with automatically generated design rainfalls and Monte Carlo simulation) |
| SCKFIT.CAT | sample catchment file (South Creek FIT run) |
| SCKMAR56.STM | sample storm file (South Creek FIT run) |
| SCKDES.CAT | sample data file (South Creek DESIGN run) |
| Fig 6-7.wmf | windows metafile of Figure 6-7 (exercise in Section 6.11) |

The above files are used in the examples provided in Chapter 10. Other files used by RORB to compute Design Rainfalls for Australia and to derive inputs for the Monte Carlo simulation include:

| | |
|---|---|
| AUST_IFD.MAP | IFD info for Australian cities |
| MELB_IFD.MAP | IFD info for Melbourne locations |
| MELBOURNE.IFD | Sample user-defined IFD file (using Melbourne ARR IFD) |
| MELBOURNE.DAT | 6 minute pluviograph data (kindly provided by the Bureau of Meteorology) |
| MELBOURNE_MC.PAT | Historic sample of temporal patterns for use in Monte-Carlo simulation |

## 1.5 Computer Requirements and Installation

RORB is designed for users running Windows 95 and above. It is a native 32-bit Windows application based on the Winteracter Graphical User Interface (GUI) and Fortran95 programming environments. In essence, the full functionality of the existing RORB model has been retained, as it runs as a subroutine to the Windows GUI, but combined with extra capabilities as noted above.

Installation is most easily accomplished by running the installation program supplied. It should be noted that this program does not rely on, nor interact with, any other windows programs.

## 1.6 Historical Development of Program

Various runoff routing programs have been written by the authors, even as early as 1959. However, the first version of RORB, specifically designed to provide a general and comprehensive program for widespread use and thereby to facilitate the application of runoff routing by the engineering profession, was released in 1975 as a BASIC program called RORT.

A second version, called RORB[2], was released in 1978 in both a BASIC and a FORTRAN version, the latter being simply a translation of the former. That version extended the usage of the program from rural to urban or partly rural and partly urban catchments and also introduced some other new capabilities.

The third version, released in 1981, further enhanced the program's capabilities, improved its portability, and increased its ease of use. It was completely rewritten in standard FORTRAN 66 code.

Version 4, released in 1987, was functionally the same as Version 3 but made use of the graphical capabilities of personal computers. A Windows Interface, added in 1997, gave significant enhancements, particularly in generation of design storms, and graphical output.

Version 5 removed the DOS dependency completely, by including the FORTRAN code for RORB as a subroutine in a Windows environment, the latter adding much to the usability of the program. New features (eg Monte Carlo simulations, and batch runs) were important additions.

Version 6 features a graphical user interface (GUI) for creation/editing of datafiles. It also allows the generation of rainfall data from user-input IFD information, hence allowing for applications of RORBWin outside Australia and/or for data ranges beyond those provided in ARR.

## 1.7 Arrangement Of Manual

Chapters 2 and 3 present the concepts and theory and define the various entities used by RORB to represent catchment and stream systems and rainfall-runoff events. Chapter 4 gives detailed instructions on how to formulate the model for a particular catchment or stream system, while preparation of the input data file is described in Chapter 5. Instructions on using the RORB Graphical Editor are provided in Chapter 6. Information and advice on the operation of the program in such a way as to achieve successful applications efficiently is contained in Chapter 7. The use of Monte-Carlo simulation is discussed in Chapter 8, and output types and control of output are discussed in Chapter 9. A range of examples in Chapter 10 shows how to use the program and illustrates many of its capabilities.

The basic concepts of runoff-routing, as they relate to RORB, are provided in Appendix A.

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[2]The 'ROR' of 'RORB' stands for 'runoff routing'. The 'B' no longer has significance but at one time indicated that the program was developed and maintained on a Burroughs B6700 computer.

## 1.8 Updates, Errors and Bugs

Updates of the program are provided from time to time to enhance its capabilities, and to fix errors and bugs which may still be present in this version. Updates to the program will be made available for free download from:

http://www.skmconsulting.com/RORB

If users find any errors, or if they have suggestions for further enhancement of RORB, it would be appreciated if these could be forwarded by email to:

RORB@eng.monash.edu.au

# 2 REPRESENTATION OF STREAM SYSTEMS AND CATCHMENTS

This chapter gives details of the way in which stream systems and catchments are represented in the RORB program. Additional material on basic concepts of runoff routing, as they are applied in RORB, is provided in Appendix A.

## 2.1 Stream Channel Network Representation

### 2.1.1 Network Structures

RORB represents the actual channel network by a network of model storages arranged similarly to the actual network. Any form of channel network may be modeled, including single streams, convergent, divergent and looped networks, and any combination of these. Flow in a given channel is assumed always to occur in the same direction, which must be known; the program cannot determine the direction of flow nor change it during an event. Backwater effects are not considered. Branch channels, whether convergent or divergent, may themselves have branches, and the branching may proceed to any order. Divergent branch channels may terminate, so far as the model is concerned, immediately after the diversion, at any point along the branch, or at a confluence with the original or a different channel. Figure 2-1 illustrates these channel network structures.

### 2.1.2 Types of Channel Inflow and Outflow

Water may enter the channel network in the following three ways:

1. **Sub-area Inflow:** This represents the hydrograph of rainfall-excess from a sub-area and it enters the stream network at a point near the centroid of the sub-area.

2. **Concentrated Channel Inflow:** In catchment flood estimation, this may represent streamflow from a part of the catchment not being modeled, or a diversion into the channel from an off-stream storage or from outside the area being modeled. In river or reservoir flood routing, it may represent a main inflow hydrograph or a tributary inflow hydrograph. In either case, it may represent the flow into an effluent stream caused by bank overflow from a channel previously modeled.

3. **Distributed Channel Inflow:** The total inflow is distributed over a specified number of reaches, being added discretely at the ends of the reaches, the amount added in each reach being proportional to its length (or such other function related to travel time as the user may optionally use, as discussed below in Section 2.2.2). This may represent lateral inflow of either base flow or surface flow.

Water may leave the channel network in the following two ways, analogous to 2 and 3 above.

Figure 2-1 illustrates these forms of channel inflow and outflow.

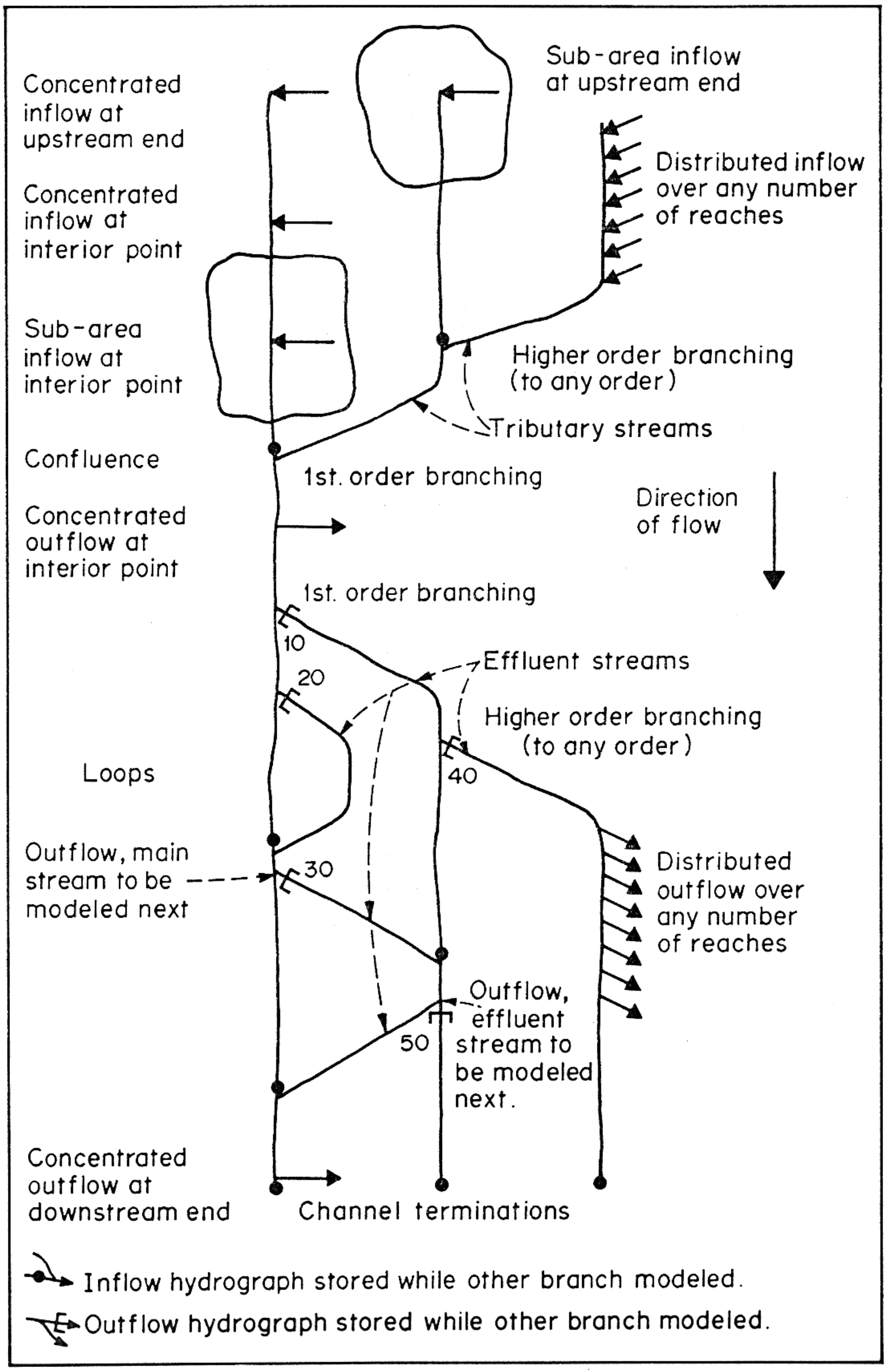


**Figure 2-1 Channel network structures, inflows and outflows**

4. **Concentrated Channel Outflow:** This may represent a controlled diversion into a man-made channel or an uncontrolled diversion into an effluent stream. Such an outflow may subsequently be viewed as the input to an effluent stream to be modeled, as mentioned in 2 above.

5. **Distributed Channel Outflow:** This is treated analogously to distributed channel inflow (3 above) and may represent the processes of transmission loss or lateral outflow.

The program must be informed by an inflow/outflow type flag whether inflow or outflow is to occur; the values of this flag are given in Table 2-1. It will be explained in Section 2.1.3 below why two alternative values are necessary for outflow.

**Table 2-1 Channel Inflow/Outflow Type Flag**

| Flag | Meaning | Conditions |
|---|---|---|
| 1 | Inflow | Concentrated or Distributed |
| 0 | Outflow, main stream modeled next | Concentrated or Distributed |
| -1 | Outflow, effluent stream modeled next | Concentrated only |

Sub-area inflows are identified by the letter designation of the sub-area, which is assigned by the program. Character strings of not more than twenty-eight characters are used to identify the locations of concentrated inflow/outflow and the types of distributed inflow/outflow. In addition, points of concentrated channel inflow/outflow that are subsequently used to define other inflows/outflows must be uniquely identified by a non-zero integer constant (see Figure 2-1).

Channel inflow and outflow hydrographs are added to or subtracted from the hydrograph immediately upstream of the inflow/outflow point by superposition.

Various methods are available for defining the inflow/outflow hydrographs and these are given in Section 3.3.5 below.

### 2.1.3 Modeling Sequence

Simulation of the streamflow commences at the upstream end of any channel and proceeds to the first confluence, where the hydrograph is stored until modeling of the areas and channels contributing to the other branch of the confluence is completed. Any confluence, the two branch hydrographs are superimposed and modeling proceeds downstream.

If an effluent stream that is later to be modeled occurs, either the outflow hydrograph or the main stream hydrograph must be stored while modeling proceeds along the other branch. The channel inflow/outflow type flag (Table 2-1) indicates which branch is being modeled first.

Points at which outflow hydrographs are stored for later use as inflows, whether on an effluent stream or a main stream, can be regarded in the same way as the upstream ends of channels. Furthermore, if the channel network to be modeled has more than one downstream end, as in Figure 2-1, the several channel terminations must be treated in the same way as multiple branches entering a confluence. It is apparent from Figure 2-1 that, by using these two concepts, even the most complex channel network can be treated in the same way as a simple converging tree-like network.

A simple procedure to define the sequence of model operations in accordance with the above principles for any given catchment or stream network is given in Section 4.3.

### 2.1.4 Subdivision of Channel Network

To model the storage effects of the catchment, the stream network is subdivided into a series of reaches and each reach is represented in the model by a reach storage, sometime referred to as a normal storage in contradistinction to reservoirs and retarding basins, which are called special storages. Only the more significant stream channels are explicitly modeled. The storage effects of channels too small to be modeled individually and of the overland flow are lumped in with the storage effects of the more significant channels. This procedure is based on the observation that modeling of the 'sub-area storage', i.e. of the overland flow and minor channel storage, separately and differently from the major channel storage requires more data and computer time but produces almost the same hydrograph at the catchment outlet as does the simpler procedure recommended. Nevertheless, if it is necessary to model the sub-area storage separately from the main channel storage, the model may be adapted to do this in various ways, which are described in Section 4.5.2.

Subdivision of the channel network into reaches serves two purposes. Firstly, it provides for runoff from the catchment to enter the channel system at various points. Secondly, it provides for flow entering the channel system at any point to be routed through a *series* of storages to the catchment outlet, rather than just a single storage. It thereby models the observed translational effects of distributed channel storage as well as the attenuation effects even though storage is treated as concentrated, i.e. as a function of outflow discharge and not of a weighted average of inflow and outflow. (See also Section A.1, Apendix A)

The degree of subdivision of the channel network is a matter demanding subjective judgement by the user and on which little firm advice can be given other than to avoid excessively fine subdivision. In catchment modeling for flood estimation, the subdivision of the channel network is conveniently, though not necessarily, related closely to the subdivision of the catchment into sub-areas as will be described in Section 2.3.2. A catchment subdivision into from five to twenty sub-areas usually seems to lead to a satisfactory subdivision of the channel network.

## 2.2 Reach Storage Representation

### 2.2.1 Storage-Discharge Relations

Reach storages are so called because they mainly represent the storage effects of channel reaches though they normally also include the effects of overland storage. That inclusion is incidental, however, as the storage parameters of reach storages are related to the properties of the reach and not those of overland flow. As discussed in Appendix A, reach storages are assumed to have storage-discharge relations of the form:

$$S = 3600kQ^m \qquad \textit{Equation 2-1}$$

where $S$ is the storage ($m^3$), $Q$ is the outflow discharge ($m^3/s$), $m$ is a dimensionless exponent, and $k$ is a dimensional empirical coefficient.

The exponent $m$ is a parameter to be determined in FIT runs but is subsidiary to $k$. A value of 0.8 is recommended as a first trial value. In the absence of more relevant information, the value of 0.8 may also be used for flood estimation on ungauged catchments. $m$ is a measure of the catchment's non linearity and a value of unity implies a linear catchment. Rarely is the value less than 0.6 or greater than 1.0 for catchment areas up to several thousand square kilometres in area. On the other hand, for flood *routing* in major rivers and through storage reservoirs, values between 1.0 and 1.5 are not unusual.

The coefficient $k$ is formed as the product of two factors:

$$k = k_c k_r \qquad \text{Equation 2-2}$$

where $k_c$ is an empirical coefficient applicable to the catchment (or, more rarely, a sub-catchment) and stream network, and $k_r$ is a dimensionless ratio called the relative delay time applicable to an individual reach storage.

### 2.2.2 Relative Delay Time

For *catchment studies*, in which some area is being modeled, the relative delay time of a storage is defined as the ratio of its delay time at any given discharge to the total delay time at the same discharge of all channel reaches from the centroid of the area being modeled to the downstream end of the channel network, with the total delay time calculated assuming that all reaches are in their natural condition. It is calculated in the program as follows:

$$k_{ri} = F_i \quad (L_i / d_{av}) \qquad \text{Equation 2-3}$$

where $k_{ri}$ is the relative delay time of storage $i$, $L_i$ is the length of reach represented by storage $i$ (km), $d_{av}$ is the average flow distance in the channel network of sub area inflows (km) [this is calculated by the program from the reach length data and printed out with the sub-area data], and $F_i$ is a factor depending upon the type of the reach (evaluated by the program as shown in Table 2-2).

For *river flood routing studies* in which no area is being modeled, the relative delay time of a storage is taken simply as its length in km modified by the factor $F_i$ if appropriate:

$$k_{ri} = F_i L_i \qquad \text{Equation 2-4}$$

In Equation 2-3 and Equation 2-4, reach length is used as an indicator of the storage delay time in the reach. Since average flow distance is calculated from reach lengths, this too is an indicator of a representative flow time. Implicitly it is being assumed that delay time is proportional to flow distance in natural stream channels. This assumption has been justified for many catchments and has the merit of simplicity. If, however, the user prefers to use some other quantity as an indicator of storage delay time in the reach, he or she may put the values of such other quantity in the data file instead of reach length. For example, the quantity $L/\sqrt{S_c}$ where $L$ is the length, and $S_c$ is the slope of the channel reach (in any suitable units) might be used. Use of $L/\sqrt{S_c}$ may be desirable in cases involving extreme slope variations and including very low slopes, say less than 0.05%.

**Table 2-2 Reach Types ($S_c$ = Slope of channel reach, %)**

| Reach Type | Description of Channel reach | $F_i$ in Equation 2-3 |
|---|---|---|
| 1 | Natural | 1.0 |
| 2 | Excavated but unlined | $1/(3S_c^{0.25})$ |
| 3 | Lined or piped | $1/(9S_c^{0.5})$ |
| 4 | Drowned (by reservoir) | 0.0 |

### 2.2.3 Reach Types

Four different types of reach are recognised, having different properties and different relative delay times and identified by reach type codes 1, 2, 3 and 4. These types and their factors $F_i$ in Equation 2-3 are defined in Table 2-2.

The $F_i$ value of 1.0 for Reach Type 1 establishes that type as the standard. The formulas for $F_i$ in the cases of Reach Types 2 and 3 are empirically determined on the basis of parameter fitting for three fully urbanised Australian catchments. They are thought to be reasonable for slopes between about 0.05% and 5%. Outside these limits, the values applying at the limits are used.

In channel reaches that have been drowned by the construction of a reservoir, flood waves are transmitted as dynamic waves rather than as kinematic waves and their celerity is typically high, approximating $\sqrt{gy}$ where $y$ is the depth. (Note $g \approx 10\ m/s^2$ and $y$ is in $m$.) If this leads to a travel time through a reach that is much smaller than the time increment being used, a reach type code of 4 should be used, giving a delay time in the model of zero.

To simplify data file preparation, a reach type flag is included. If this has a value of 1, 2, 3 or 4, it indicates that all reach storages in the model are of the type indicated by the value of the flag and it is then not necessary to include the reach type codes for the individual reach storages. However, if the reach type flag has a value of zero, it indicates that not all reaches in the model are of the same type and consequently the reach type codes must be input for all reach storages.

### 2.2.4 Coefficient $k_c$

#### *2.2.4.1 General*

The empirical coefficient $k_c$ is the principal parameter of the model. Its determination is the main object of FIT runs.

Because relative delay time is defined differently for models that include some area (catchment studies) than for those that don't (flood routing studies), as explained above in Section 2.2.2, $k_c$ also has different values for the two cases.

In both cases, the value of $k_c$ is very dependent on the value of $m$. Consequently, a $k_c$ value determined with one value of $m$ cannot validly be used with another value. If the $m$ value is changed between runs, an approximate adjustment factor for $k_c$ is $(Q_p/2)^{m-m'}$ where $Q_p$ is the peak discharge and $m$ the old and $m'$ the new value of the exponent. Its use may save time taken in interative fitting.

In fitting the model to a range of different floods, a procedure due to Weeks (Ref. 9) can be used to obtain the optimum combination of $k_c$ and $m$. This procedure is described in Section 7.2, Item 5.

#### 2.2.4.2 $k_c$ for Catchment Studies

In initial FIT runs, the program suggests, as one option, the use of a first trial value calculated as follows:

$$k_c = 2.2A^{0.5} (Q_p / 2)^{0.8-m} \qquad \textit{Equation 2-5}$$

where A is the catchment area, (km²), and $Q_p$ is the (maximum) peak discharge of hydrograph(s) ($m^3/s$). It should be noted that the term $Q_p$ has a value of unity when $m$ is at its recommended value of 0.8. $k_c$ is calculated for an entire catchment area, even if the most downstream gauging station is not at the downstream extremity of the catchment. Equation 2-5 is an empirical equation that generally represents a wide range of fitted data for Australian catchments; it is used in the program only as a rough estimate to help users get started in FIT runs on gauged catchments. In the absence of more relevant information, it could be used for ungauged catchments but values so calculated can be in error by a factor of about 2 so great reliance should not be placed on the equation.

Another option provided by the program for users is information on regional relations for ungauged catchments from *Australian Rainfall and Runoff* (Ref. 16), but users should be careful when using regression equations averaged over non-homogeneous regions. In the view of the authors, the surest procedure is that given in Section 7.4, Item 5; to this end, the studies cited in Ref. 16 may be useful sources of $k_c$ and $m$ values for individual, neighbouring catchments.

It is important to note that $k_c$ depends on the size of the catchment area. Consequently, when a $k_c$ value determined in a FIT run is used in a DESIGN run, *the same catchment and channel network must be used for both runs*, even though the main point of interest might be different in the two runs.

#### 2.2.4.3 $k_c$ for Flood Routing Studies

A trial value of $k_c$ is not suggested by the program. However, if an $m$ value of unity is used, if reach lengths are input as the indicator of travel time as suggested in Section 2.2.2 above, and if the channels are in natural condition, $k_c$ should approximately equal the reciprocal of the average wave speed for the channel network in km/h. Any available observations of travel time in the channels being modeled would lead to a good first trial estimate of $k_c$ in these circumstances. [See Section 10.2.1 for an example.]

If a function other than length is used in the data to indicate reach travel time, an appropriate adjustment could be made. For example, if $L/\sqrt{S_c}$ were being used, $k_c$ would be approximated by multiplying the reciprocal of the average wave speed in km/h by the square root of the average reach slope in %. Further adjustments if the channels are not in natural condition are suggested by the $F_i$ values in Table 2-2. Suitable adjustments of $k_c$ to provide for *m* values other than unity would be suggested by the program if it were first run with $m$ = 1 and a new *m* value then adopted.

### 2.2.5 Routing Method

Routing of a hydrograph through a model reach storage is performed by a non-linear storage routing procedure based on continuity and a storage function of the form of Equation 2-1 (see Ref. 1). A linear storage function may be adopted by putting $m$=1, in which case the Muskingum routing method is used. For $m \neq 1$ the routing computation for each time increment is adjusted until the change in storage calculated from the storage-discharge relation is equal to the difference between inflow and outflow assuming linear variation of the hydrographs over the time increment. These two storage changes are assumed equal if they differ by less than 0.1% of the inflow peak discharge over the time increment. The Regula Falsi convergence algorithm is used. A limit on the number of iterations has been programmed and if this limit is exceeded, the user will be so informed and given the option of accepting the average of the last two discharge estimates and proceeding to the next time increment or of terminating execution. [The limit is also used in the iterative calculation of loss parameters.]

Routing computations cover a period defined by the user in a data item called the duration of calculations. This period is independent of the duration of the rainfall and the hydrograph. It should be noted, however, that the hydrograph volumes and centroids are calculated only over the `duration of calculations' period, so this should not be shorter than the actual hydrograph if these quantities are of interest. At each model storage, the entire hydrograph is computed before routing computations for the next storage commence. For each model storage, the outflow discharge at time zero is assumed equal to the inflow discharge at time zero.

## 2.3 Catchment Area Representation

### 2.3.1 Catchment

In flood estimation, but not in flood routing, there must be a catchment, which is defined as the entire area being modeled. The most downstream point in a catchment is called the catchment outlet.

### 2.3.2 Sub-areas

To provide for areal variation of rainfall and losses and the fact that runoff from different parts of the catchment travels different distances to the outlet, the catchment should be divided into sub-areas. Rainfall and losses are averaged over each sub-area.

Although the program can handle sub-areas based on any method of catchment subdivision, in the recommended model sub-areas are based on the stream network and the drainage characteristics of the catchment. Each sub-area is bounded by drainage divides that separate it from other sub-areas or other catchments. Also, each sub-area contains a stream segment called its main stream, which may be part of the catchment main stream or of a tributary stream. A sub-area may or may not have other sub-areas upstream of it. If it does, the main stream passes right through the sub-area and if it does not, the main stream originates within the sub-area. The rainfall-excess from a sub-area is assumed to enter the channel network at a point on the sub-area's main stream adjacent to the centroid of the sub-area.

A procedure for defining the sub-areas of any given catchment is detailed in Section 4.1. The sub-areas are referenced by identifying letters A, B, ... in the order in which their data are input and used by the program. If there are more than 26 sub-areas the letters are repeated as necessary. The ordering of sub-areas is controlled by the drainage network and is defined in a manner explained in Chapter 4.

Subdivision of the catchment area should be fine enough that areal variations of rainfall and losses and the effects of varying flow distance to the catchment outlet are adequately modeled. This will usually be achieved by between five and twenty sub-areas. Finer subdivision is unlikely to improve the accuracy of hydrograph simulation noticeably because of the damping effect of catchment storage.

### 2.3.3 Sub-catchments

A sub-catchment is defined as that part of the catchment upstream of a given point on the stream network, called the sub-catchment outlet. A sub-catchment is identified by the name of its outlet location, expressed in a character string of not more than twenty-eight characters. In RORB, sub-catchments are used in the loss model if there is more than one gauging station on the catchment.

### 2.3.4 Interstation Areas

An interstation area is defined as the area upstream of a given gauging station or the catchment outlet but downstream of any other gauging station. The interstation area of a gauging station that has no other gauging station upstream of it is identical to the sub-catchment of that gauging station. If there is only one gauging station and it is located at the outlet of the catchment, the catchment, sub-catchment, and interstation area are identical.

Users should note the possibility of specifying ‘dummy’ gauging stations (and corresponding dummy hydrograph in FIT runs) to allow for variation of losses in the interstation area so created. To facilitate this option, a Code 7.2 has been added to RORB, as discussed in Section 4.2).

## 2.4 Storage Reservoir and Retarding Basin Representation

### 2.4.1 General Arrangement

Storage reservoirs, lakes and retarding basins are regarded as special storages and are modeled differently from the normal channel reach storages. They are treated in a sequence of model operations at points analogous to their physical locations on the stream system. Each special storage is identified by a name consisting of a character strong of not more than twenty-eight characters.

A retarding basin is a detention storage, usually formed by an embankment across a stream but sometimes by excavation, and having one or more uncontrolled low level pipe outlets and one or more uncontrolled overflow spillways at a higher elevation. Its purpose is to reduce the peak discharges of floods passing through the basin. Storage reservoirs and lakes, on the other hand, have one or more uncontrolled overflow spillways but no uncontrolled pipe outlet. Existing special storages that have neither a spillway nor a pipe outlet but a natural outlet can also be included in the RORB model if their storage-discharge relations can be defined.

### 2.4.2 Elevation Definitions

In Figure 2-2, the general arrangement of embankment, lowest spillway, and lowest pipe outlet is indicated, the various elevations of importance in defining the basis characteristics are defined, and the permissible ranges of these elevations relative to each other are indicated. Attention is drawn to several points:

(i) all elevations are in metres above an arbitrary datum;

(ii) the lowest pipe outlet may be at the elevation of the basin floor, above the basin floor, forming a lake, or below the basin floor in a low flow channel, so that storage does not occur until some threshold discharge has been exceeded;

(iii) there is no upper limit on the pipe diameter;

(iv) the initial water level, at the start of the flood, may be at or below the cease-to-flow level, i.e. the lowest pipe entrance invert if there is one or the lowest spillway crest otherwise;

(v) for pipe full flow, the tailwater elevation is assumed to be at or below the obvert of the lowest pipe outlet at its downstream end.

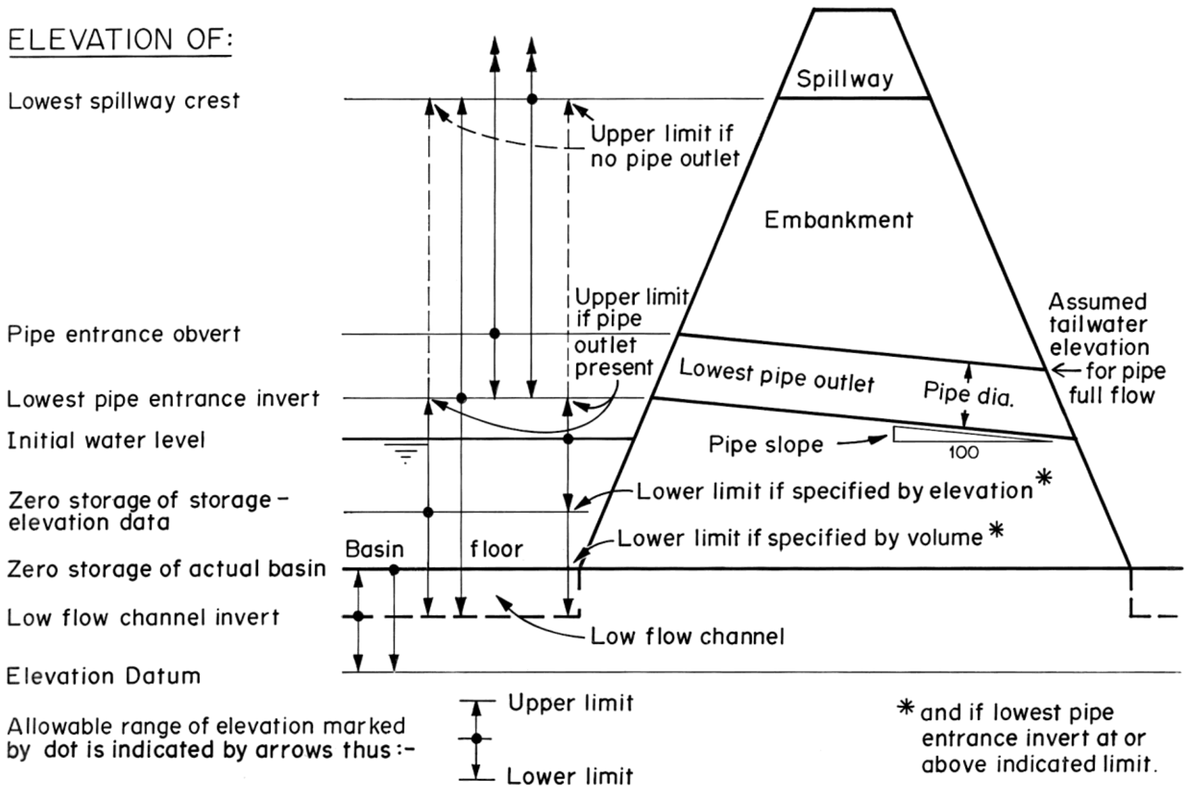


**Figure 2-2 Elevation definitions for reservoirs and retarding basins**

### 2.4.3 Design of Reservoirs and Retarding Basins (Ref. 18)

In RORB, a special storage may be existing or proposed. If proposed, it can be represented only in a DESIGN run, the purpose of which is to design the storage interactively. Design in this context means determining the outlet and storage characteristics in such a way that a design criterion is met under assumed design storm conditions. The outlet characteristics to be determined may include the elevation and effective length of the spillway(s) and the length, gradient, invert elevation, number, and diameter of pipes in the pipe outlet(s). Storage characteristics to be determined may include the parameters of equations defining the storage-discharge and storage-elevation relations. The design criterion can be either:

(i) peak outflow discharge of basin design flood limited to a specified value; or
(ii) peak storage elevation reached by basin design flood limited to a specified value.

In either case, the lowest spillway crest is set at the peak water level reached by the basin design flood. Thus, in the basin design flood, no water flows over the spillway. A larger design flood, the spillway design flood, must then be used to design the length of the lowest spillway, the lengths and elevations of any additional spillways, and the peak surcharge elevation. For both basin and spillway design floods, storms of various durations must be considered, in different runs with different data files, to determine the critical duration.

In retarding basin design, the program calculates the necessary inflow design floods from the design storms and also can be conveniently used to show the effects of the basin on flood hydrographs downstream.

For the purposes of designing the spillway(s) of special storages, a free overfall over a horizontal crest with vertical end walls or no end walls (i.e. a glory hole spillway operating in the weir flow range) is assumed. If there is a pipe outlet, it is assumed to be constructed of circular concrete pipes and if there is more than one pipe in a given outlet group all are assumed to be of the same diameter and at the same elevation (see Figure 2-3). Additional pipe outlet groups may be of different diameter and may be at or above the level of the previous group.

### 2.4.4 Initial Drawdown

As noted in (iv) of Section 2.4.2, a special storage may be assumed to be drawn down below its cease-to-flow level at the commencement of the event being modeled. If the storage is an existing one, the data item included in the data file indicates to the program what assumption is being made, in accordance with Table 2-3. If the storage is being designed, the user inputs initial drawdown from the terminal as part of the design process.

When initial drawdown exists, its volume is subtracted from the start of the inflow hydrograph to the basin. If the total volume of the hydrograph is insufficient to fill the initial drawdown, a message to that effect is printed, the outflow hydrograph is zero, and the program proceeds to the next step in its modeling sequence. If, however, as is usual, there remains a hydrograph after the initial drawdown has been satisfied, it is routed through the storage.

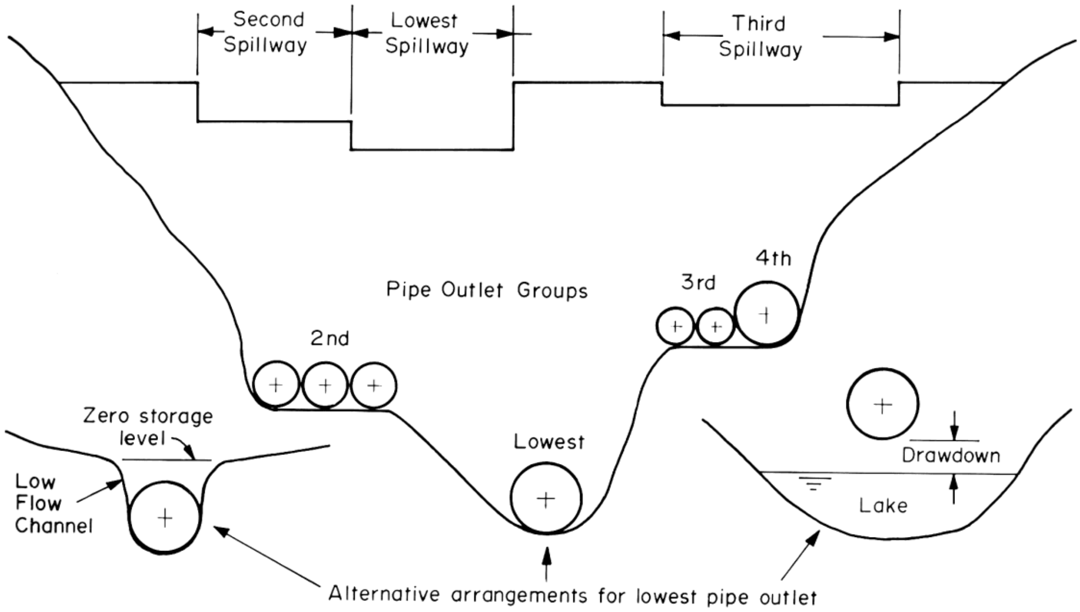


**Figure 2-3 Diagrammatic arrangement of possible pipe outlet and spillway provisions**

**Table 2-3 Initial Drawdown Specification: Existing Basins**

| Assumption | Data Item | Conditions |
|---|---|---|
| No initial drawdown; initial water level at cease-to-flow level | Zero | Nil |
| Initial drawdown specified as volume below cease-to-flow level | Drawdown volume, $m^3$ Negative | Nil |
| Initial water level specified at elevation below cease-to-flow level | Initial W.L., m (Positive) | Permissible only if storage-elevation relation supplied. Initial W.L. within range of storage-elevation data. |

### 2.4.5 Routing Hydrographs through Special Storages

#### *2.4.5.1 General*

To route a hydrograph through the storage, it is necessary to define the relation between discharge from the basin and storage in it. Sometimes this is a direct relationship and sometimes it is achieved indirectly by relating both discharge and storage to the water surface elevation. For convenience and flexibility, the discharge relation may be defined in a variety of ways involving four options for the way discharge is defined and three for the way storage is defined. The way adopted in a particular case is indicated to the program by two data flags called the discharge relation flag and the storage-elevation flag.

**Table 2-4 Forms of Basin Discharge Relation**

| Discharge Relation Flag | Discharge a function of: | Function defined by: | Conditions |
|---|---|---|---|
| 0 | Storage | Equation 2-6 | Nil. |
| 1 | Storage | Storage-discharge table | Existing basins only |
| 2 | W.S. Elev. | Equation 2-10 to Equation 2-14 incl. | Spillway & Pipe outlet<br>If zero pipe outlet groups specified, the case is treated as for spillway only. |
| 3 | W.S. Elev. | Equation 2-10 | Spillway outlet only.<br>Min. no. of spillways = 1 |

#### *2.4.5.2 Discharge Relations*

The four possible values of the discharge relation flag and the conditions associated with them are given in Table 2-4. The equations referred to in that table are used to evaluate discharge under various conditions and are listed below:

***Storage-Discharge Equation - Discharge Relation Flag = 0***

$$S = 3600k_S Q^{m_s} \qquad \textit{Equation 2-6}$$

where $S$ is the storage ($m^3$), $Q$ is the discharge ($m^3/s$), and $k_S$ and $m_S$ are parameters depending upon the topographic and hydraulic characteristics.

The parameters $k_S$ and $m_S$ must be provided to the program in the data file. Their values can be determined as follows:

(i) for various water levels above cease-to-flow level, determine the surcharge storage from contour map or other survey information and the discharge from hydraulic formulae such as Equation 2-11 or from model study results; and,

(ii) plot storage against discharge on log log paper, fit a straight line of best fit, which has the form of Eqn. 2.5, and evaluate is parameters $k_S$ and $m_S$.

A simpler approach that avoids the graphical work can be used if the storage area varies little over the practical range of surcharge elevations and if a simple weir type formula (see Equation 2-11 below) applies to the spillway discharge. In that case:

$$S \approx A_S(H - H_S) \qquad \textit{Equation 2-7}$$

where $A_S$ is the storage area ($m^2$), and $H$ and $H_S$ are defined below Equation 2-10. Combining Equation 2-6 and Equation 2-10 and comparing with Equation 2-5 yields

$$k_S = A_S/[3600(K_W L_S)^{2/3}] \qquad \textit{Equation 2-8}$$

and

$$m_s = 2/3 \qquad \textit{Equation 2-9}$$

***Storage-Discharge Table - Discharge Relation Flag = 1***

Pairs of $S$ and $Q$, determined as in (i) above, are included in the data file. These values must be in increasing order thus:

$$S_i \geq S_{i-1} \text{ and } Q_i > Q_{i-1}, \quad i = 2,3, ...$$

They should commence at the cease-to-flow level ($Q_1$ = 0.0). There is no upper limit on the number of pairs of values in the table; there should be enough to define the relation precisely having in mind that the program interpolates linearly between the defined points.

***Spillway Discharge Equation - Discharge Relation Flag = 2 or 3***

$$Q_s = K_W L_S (H - H_S)^{3/2} \qquad \textit{Equation 2-10}$$

where $Q_S$ is the spillway discharge ($m^3/s$), $K_w$ is the weir coefficient for the spillway (see Table 2-5), $L_S$ is the effective length of the spillway ($m$), $H$ is the water surface elevation ($m$), and $H_S$ is the spillway crest elevation ($m$).

$L_S$ and $H_S$ are input interactively for a basin being designed but are included in the data file for an existing basin. $K_W$ is always included in the data file. A suitable value may be selected from Table 2-5.

**Table 2-5 Weir Coefficients** (Source: Refs. 10 and 11)

| Weir Type | $K_W$ |
|---|---|
| Ogee | 2.15 |
| Broad-crested, with sloping approach | 2.00 |
| Broad-crested, with vertical upstream face | 1.45 |
| Sharp-crested, with vertical upstream face | 1.74 |

***Pipe Flow Equations - Discharge Relation Flag = 2***

Discharge under inlet control is calculated by Equation 2-11 for water levels up to 0.8 pipe diameters above the invert and by Equation 2-12 for higher water levels (Ref. 12, p. 262). Discharge under outlet control is calculated by Equation 2-13 for water levels equal to or greater than one pipe diameter above the invert.

For any water level, the lower of the inlet and outlet control discharges is adopted [see Figure 2-4(a)] except that once outlet control is established on a rising water level, it is assumed to persist for all higher water levels regardless of which condition gives the lower capacity [see Figure 2-4(c)]. If at water levels below pipe full, inlet control discharge exceeds the lowest value of outlet control discharge, the latter is adopted [see Figure 2-4(b)]

$$Q_p = 1.50 N_p (S_p/40)^{0.05} (H - H_O)^{1.9} D^{0.6} \qquad \textit{Equation 2-11}$$

where $Q_p$ is the pipe discharge ($m^3/s$), $N_p$ is the number of pipes, $S_p$ is the average slope of the pipe outlet (%), $H_O$ is the elevation of pipe the entrance invert, and $D$ is the pipe diameter ($m$).

$$Q_p = 1.38\, N_p\, (S_p/40)^{0.05}\, (H - H_o)^{1.5}\, D$$ *Equation 2-12*

$$Q_p = 0.785 N_p D^2 \sqrt{\frac{19.6(H - H_t)}{k_e + k_b + 1.0 + f(L_p / D)}}$$ *Equation 2-13*

where $H_t$ is the tailwater elevation (*m*) calculated by the program as the elevation of the pipe obvert at the exist from the pipe, $k_e$ is the pipe entrance loss coefficient (see Table 2-6), $k_b$ is the pipe bend loss coefficient (see Table 2-7), $L_p$ is the length of pipe (*m*), and *f* is the friction factor calculated by the program from the Colebrook-White equation, Equation 2-14.

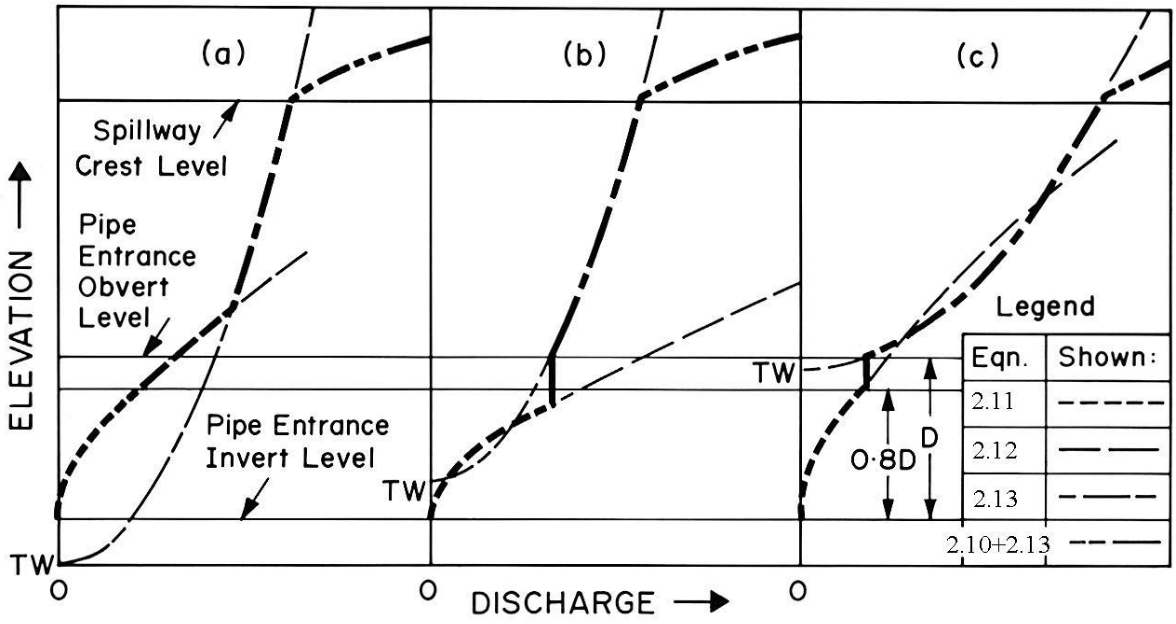


**Figure 2-4 Retarding basin elevation - discharge relations.** [TW = Tailwater level (pipe exit obvert level); Adopted curve shown in bold line in each case.]

$$f = \frac{1.0}{(0.868 \ln(D / 2k_s) + 1.74)^2}$$ *Equation 2-14*

where $k_s$ is the pipe roughness (*m*) assumed by the program to be 0.00015.

**Table 2-6 Pipe Entrance Loss Coefficient** (Source: Ref. 12)

| Type of Entrance | $k_e$ |
|---|---|
| Square-edged (whether projecting from fill, flush with headwall, or headwall with wingwalls) | 0.5 |
| Rounded | 0.2 |
| Mitred to conform to fill slope | 0.7 |

**Table 2-7 Pipe Bend Loss Coefficients ($k_b$)** (Source: Ref. 10, p. 302)

| Radius of Bend ÷ Pipe Diameter | Deflection Angle of Bend | |
|---|---|---|
| | 45° | 90° |
| 1 | 0.37 | 0.50 |
| 2 | 0.22 | 0.30 |
| 4 | 0.19 | 0.25 |
| 6 | 0.11 | 0.15 |
| 8 | 0.11 | 0.15 |

Of the pipe quantities defined above, the number $N_p$, diameter $d$, length $L_p$, slope $S_p$, and entrance invert elevation $H_o$ are input interactively for a basin being designed, but are included in the data file for an existing basin. The entrance loss coefficient, $k_e$, and bend loss coefficient, $k_b$ are always included in the data file. Suitable values for $k_e$ and $k_b$ may be selected from Table 2-6 and Table 2-7 respectively. The pipe roughness cannot be changed in the RORB program.

**Table 2-8 Forms of Storage-Elevation Relation**

| Flag | Relation defined by: |
|---|---|
| 0 | No storage-elevation relation |
| 1 | Storage-Elevation Table |
| 2 | Equation 2-15 |

#### *2.4.5.3 Storage-Elevation Relations*

If the special storage is being designed or is an existing basin for which the storage-discharge relation expresses discharge as a function of water surface elevation, a storage-elevation relation for the basin must be input. In other cases, a storage-elevation relation may be input if the user would like to have the peak elevation reached by the flood printed.

A data flag called the storage-elevation flag must be included in the data to indicate to the program whether a storage-elevation relation is being supplied and, if so, whether as a formula or as a table. Table 2-8 indicates the values of this flag for different circumstances.

***Storage-Elevation Table - Storage-Elevation Flag = 1***

Pairs of values of $H$ and $S$ in that order are included in the data file. These values must be in increasing order thus:

$$H_i > H_{i-1} \text{ and } S_i \geq S_{i-1} \quad i = 2,3, \ldots$$

There is no upper limit on the number of pairs of values in the table; there should be sufficient to define the relation precisely, having in mind that the program interpolates linearly between the defined points. It is not necessary for the storage-elevation table to coincide with those of the storage-discharge table if one is input (see Section 2.4.5 above).

The elevations in this table must be to the same datum as those used in the spillway discharge and pipeflow equations. When discharge is expressed as a function of storage (discharge relation flag = 0 or 1, see Section 2.4.5), the storage datum here must be the same as in that relation. For discharge relation flag = 0, zero storage occurs at zero discharge. In that case, if an initial drawdown (Section 2.4.4) is to be specified by elevation, negative values of the storage are required at the lower end of the storage-elevation table.

***Storage-Elevation Equation - Storage-Elevation Flag = 2***

$$S = a(H - H_o)^b \qquad H \geq Ho \qquad \text{Equation 2-15}$$

$$S = 0.0 \qquad H < Ho \qquad \text{Equation 2-16}$$

where $S$ is the storage volume ($m^3$), $H$ is the water surface elevation ($m$), $H_o$ is the elevation ($m$) corresponding to zero storage, and $a$ and $b$ are constants.

Constants $a$ and $b$ can be evaluated by plotting storage against ($H$ - $H_O$) on log-log paper, fitting a straight line of best fit to the plotted points and evaluating its parameters $a$ and $b$. For a storage reservoir assumed full at the commencement of the flood, $H_O$ and zero storage can be at the spillway crest elevation only if $b \approx 1$; otherwise they should be at the bottom of the reservoir. In all cases, the elevations used here must be to the same datum as those used in the spillway discharge and pipe flow equations.

Note that the program allows these parameters to be changed at run time to facilitate design of basins where storage characteristics can be changed (eg. excavated basins).

### *2.4.5.4 Routing Procedure*

If the discharge is defined by Equation 2-6 (i.e. discharge relation flag = 0) the routing procedure is as described in Section 2.2.5 except for a superimposed algorithm to avoid significant, and spurious, oscillations in the outflow hydrograph that can otherwise occur when the storage-discharge relation contains sharp bends. This algorithm involves internal subdivision of the user-defined time increment when necessary, but hydrograph ordinates are output only at the user-defined intervals.

If the discharge is defined by a storage-discharge table (i.e. discharge relation flag = 1), the table input is used instead of an equation of the form of Equation 2-1 or Equation 2-6 to define the storage-discharge relation. In other respects, the routing procedure is as described in Section 2.2.5 above.

Also if the discharge is defined by discharge-elevation relations for the spillway (flag = 3) or the spillway and pipe outlet (flag = 2) and related to storage through a storage-elevation relation, a table routing procedure is used. In these cases, regardless of whether storage-elevation relation is a formula or a table, an elevation-storage-discharge table is computed by the program. The user is given the option of printing out this table when routing through the basin is completed. In other respects, the routing procedure is as described above. All interpolated values obtained from input or calculated tables are determined by linear interpolation.

## 2.5 Translations

Any point in the channel system, the current hydrograph may be translated in time, either advanced or retarded by an integral number of time increments, without change in its shape. This feature is used regularly in some types of catchment model that can be represented by RORB. Though it is not normally a part of the model recommended in this manual, it has been found in some cases that the model produces a calculated hydrograph of similar shape to the actual one but preceding or lagging behind it (usually the former) by a constant translation time. The reasons for this are not understood and no firm guidance can be given on values of translation time for design purposes.

In the light of the above, the translation capability of the program has not been greatly used in the past and if it is to be used, the user must make his or her own interpretation of the physical factors that are being modeled. In some cases, this might be just a synchronization error in the data but more often it is likely to be a routing effect not otherwise taken into account by the model.

It should be realized that in a translation of N time increments with N positive, the last N ordinates of the calculated hydrograph are lost and N zero ordinates are added to the front of the hydrograph. For N negative, the early ordinates are lost and the zeros are added at the end.

# 3 REPRESENTATION OF RAINFALL-RUNOFF EVENTS

## 3.1 Events

### 3.1.1 Processes Modeled

Since RORB does not model evapo-transpiration nor soil moisture and groundwater redistribution, its intended use is for events short enough that the effects of those processes on streamflow are negligible, say, up to about a week in duration. It can be applied to longer events only if the user provides for the effects mentioned through an independently determined base flow and/or evapotranspiration and/or seepage input to the model.

Streamflow is always modeled in RORB and streamflow hydrographs are the principal output of the program. The term `streamflow' is interpreted to include free-surface flow in man-made channels and conduits as well as in natural streams. In flood routing applications, only streamflow is modeled but in flood estimation applications, rainfall and losses are also modeled. The modeling of rainfall and losses implies the modeling of catchment area in addition to a channel system; their absence implies that no area but only a channel system is modeled.

### 3.1.2 Time Representation

To represent rainfall and streamflow, a constant time increment, of any value, expressed in hours, is adopted. This increment is used for both rainfall and streamflow, one ordinate per time increment, and is constant throughout the period being modeled. All times are expressed as the number of time increments measured from an initial time selected by the user to serve as a time datum. If a real event is being modeled, the initial time is entered into the program as character string data. It is not used in calculations but is used in the headings of the output. If a hypothetical event is being modeled (e.g. as a design storm), there is no real initial time and the string data entered should be a label descriptive of the hypothetical conditions.

Time subdivision should be fine enough to define the shape of the hydrograph and the major variations in rainfall intensity. A time increment of around one fifth the time of hydrograph rise is often satisfactory. A shorter increment increases the processor time and memory requirements for the computation but gives no significant increase in accuracy, because of the damping effects of the catchment.

### 3.1.3 Bursts (& Rises)

In catchment studies, separate bursts may optionally be defined if the rainfall occurs in distinct bursts that result in a multi-peaked hydrograph. These bursts are defined by their starting and finishing times and are numbered chronologically. Successive bursts may not overlap but may be separated by periods of no rainfall.

If a loss parameter is to be derived, as in most FIT runs, and the rainfall is defined to occur in more than one burst, the streamflow is assumed to occur in corresponding `rises' and the proportions of the total streamflow occurring in each rise must be input. These proportions may be indicated by the volumes, in any units, of the successive rises. The program will calculate the proportions and later the volumes of the several rises. The purpose of defining bursts is to allow the loss parameters to vary with time. A different loss parameter is derived or input for each burst.

## 3.2 Rainfall Representation

Rainfall hyetographs consist of rainfall depths in *mm* within time increments, the first ordinate being for the first time increment after the defined start of the rainfall burst. The times at which the rainfall starts and finishes, measured in time increments from the initial time, must be input. If more than one burst is defined (see Section 3.1.3), starting and finishing times must be input for each burst. If hyetographs are input for more than one pluviograph (recording rain gauge), the same starting and finishing times must be used for them all.

If the rainfall is assumed to be uniform in depth and temporal pattern over the entire catchment, the `uniform rainfall flag' in the data is set equal to zero and the only rainfall input is one hyetograph, which is applied by the program to the whole catchment.

If the rainfall is not areally uniform, it is necessary that the catchment be divided into sub-areas (such subdivision is recommended in any case, even if rainfall is uniform). The procedure for this is given in Section 4.1. The uniform rainfall flag in the data is set to one and the rainfall inputs to the program consist of:

(i) hyetographs at one or more pluviographs (recording rain gauges);

(ii) total storm rainfall in *mm* on each sub-area;

and, if there is more than one pluviograph,

(iii) pluviograph reference numbers associating each sub-area with a particular pluviograph, usually the nearest one.

The pluviographs are identified by names not exceeding twenty-eight characters in length and are assumed to be numbered consecutively in the order in which their data are input. It is these numbers that are used in (iii) to associate the sub-areas with the pluviographs. The pluviograph hyetographs define the temporal pattern of the rainfall on their associated sub-areas, while the total depth of rainfall on the sub-areas is defined by (ii).

In a DESIGN run, there may be no actual pluviograph, in which case the `pluviograph' data input represent a hypothetical design storm. A name associated with such a hypothetical hyetograph is still required.

If the rainfall is assumed to occur in more than one defined burst (see Section 3.1.3), then items (i), (ii) and (iii) above must be provided for each burst.

## 3.3 Streamflow Representation

### 3.3.1 Hydrographs

Streamflow hydrographs are expressed as discharge values in $m^3/s$ at a constant spacing of one time increment. The number of ordinates is one more than the number of time increments in the hydrograph's duration. Hydrographs calculated by the program start at the initial time (see Section 3.1.2) and have a duration, referred to as the duration of calculations, expressed in time increments (see also Section 2.2.5). Hydrographs input as data may start at any time at or after the initial time and finish at any time after the starting time.

Hydrographs, whether input or output, are identified by names of not more than twenty-eight characters. Hydrographs input are used by the program in the order they are input so this order is governed by the sequence of operations used to model the catchment. The various types of hydrograph input to or calculated by the program are described below.

### 3.3.2 'Running' Hydrograph

RORB represents the stream channel network by a series of model storages and calculates the outflow hydrographs from all storages in a defined sequence, finishing at the downstream end of the channel system. At any step during this sequence, the hydrograph so calculated is called the 'running' hydrograph. Information on the running hydrograph at any step may be viewed or printed by including an appropriate item in the data file. At gauging stations, the running (calculated) hydrograph may be compared with the gauging station (actual) hydrograph.

The running hydrograph is initialized as zero. It may be augmented by rainfall-excess hydrographs, concentrated channel inflow hydrographs, and distributed (lateral) channel inflow hydrographs. It may be depleted by concentrated or distributed channel outflow hydrographs. Its shape may be modified by routing through normal reach storages or special storages, and it may be translated in time without change of shape.

### 3.3.3 Gauging Station Hydrographs

A gauging station hydrograph is an actual, recorded hydrograph, which is input as data. In FIT runs for catchment studies, gauging station hydrographs are used to calculate the loss parameters. Whenever input, in any type of run, it is used as a standard against which to compare the calculated running hydrograph at that point.

Any number of gauging stations may be modeled. It is not necessary to have a gauging station at the downstream extremity of the channel system.

In catchment studies, because of the use of gauging station hydrographs in calculating the loss parameter(s), the entire hydrograph of a rainfall-runoff event must be input even if the routing calculations are truncated to save time by using a relatively short duration of calculations. This is so that the correct volume of flow for the event at the gauging station will be calculated.

Also to ensure proper calculation of loss parameters, the base flow must be handled correctly. This may be done in either of two ways. The base flow may be deducted from the gauging station hydrograph by the user so that only the surface runoff hydrograph is input. Alternatively, the total hydrograph at the gauging station may be input and an appropriate base flow hydrograph also input as a distributed channel inflow upstream of the gauging station. More information on the treatment of base flow is given in Section 3.3.7.

### 3.3.4 Rainfall-excess Hydrographs

In catchment studies, the program calculates hyetographs for all sub-areas. After deducting losses, it converts the hyetograph ordinates to `hydrographs' of rainfall-excess on the sub-areas, in $m^3/s$, and interprets the average `discharge' during a time increment as an instantaneous discharge at the end of the time increment. In this step, a translation of half a time increment occurs, but this is of no great concern. The sub-area rainfall-excess hydrographs are added, in correct sequence, to the running hydrograph.

### 3.3.5 Channel Inflow and Outflow Hydrographs

Permissible types of channel inflow and outflow were described in Section 2.1.2; the various means of defining the hydrographs of inflow and outflow are described here. Four methods of definition are available, each identified by an Input/Output (I/O) definition flag, as indicated in Table 3-1.

**Table 3-1 Methods of Channel Input/Output Definition**

| I/O Defn. Flag | Method of Definition |
|---|---|
| 0 | User-defined hydrograph |
| 1 | Formula relating I/O to discharge ( Equation *3-1*, Equation *3-2*) |
| 2 | Hydrograph previously identified by program |
| 3 | Table relating I/O to discharge |

***User-defined Hydrograph - I/O Definition Flag = 0***

The hydrograph is input as data in accordance with Section 3.3.1 above.

***Formula Relating I/O to Discharge - I/O Definition Flag = 1***

$$D = a + c(Q - b)^d$$
$$D \geq a \qquad \text{Equation 3-1}$$

where $D$ is the channel inflow or outflow ($m^3/s$), $Q$ is the running hydrograph discharge immediately upstream of the inflow or outflow point ($m^3/s$), and $a,b,c$, and $d$ are constants.

This function is illustrated in Figure 3-1. Appropriate values of $a$, $b$, $c$, and $d$ must be determined by the user by hydraulic or hydrologic calculations relevant to the particular circumstances. For outflow but not inflow, the following condition also applies:

$$D \leq Q \qquad \text{Equation 3-2}$$

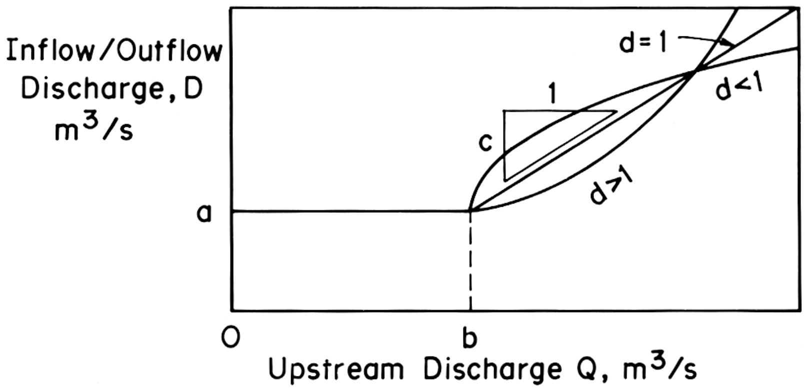


**Figure 3-1 Definition of Channel Inflow/Outflow by Formula**

***Hydrograph Previously Identified by the Program - I/O Definition Flag = 2***

A concentrated channel inflow/outflow hydrograph or the mainstream hydrograph left after a concentrated outflow, if uniquely identified by a non-zero integer, will, as well as serving its initial purpose, be stored for re-use, any number of times, as a concentrated or distributed channel inflow/outflow hydrograph. Such re-use is achieved by an I/O Definition Flag of 2, accompanied by the identifying non-zero integer of the required hydrograph. This facility can be used anywhere but is most useful at a point where one stream diverges from another, for defining the inflow hydrograph to the branch being modeled second. (Note that the stored hydrograph may not itself have an I/O Definition Flag of 2.)

***Table Relating I/O to Discharge - I/O Definition Flag = 3***

Any number of pairs of values of discharge (*Q*) and inflow/outflow (*D*) may be input, in that order. The first Q value is a threshold below which *D* equals zero. Subsequent *Q* values may not decrease ($Q_i \geq Q_{i-1}$, $i$ = 2,3, ...). The program interpolates linearly between data points in the table. For outflow, a warning is issued by the program if, for any step in the table, $\Delta D > \Delta Q$. An error occurs if the table is called with a discharge greater than its maximum.

Concentrated I/O is distinguished from distributed I/O by the number of reaches over which the total I/O hydrograph is distributed. A value of zero indicates concentrated I/O, while a value greater than zero indicates distributed I/O. For convenience, a value of -1 here indicates distribution of the I/O over all remaining reaches in the model. The sequence of reaches to which a distributed I/O definition applies starts at the point where the I/O is defined and is the sequence in which the reaches are modeled. At a confluence, modeling skips to the upstream end of a branch channel so the sequence defined may not always be a sequence of physically adjacent reaches.

A distributed channel inflow or outflow is distributed among the specified number of channel reaches in proportion to their lengths (unless, as discussed in Section 2.2.2, the user has input some function other than reach length as an indicator of travel time, in which case the distribution among reaches is proportional to the values of that function). For a particular reach, the distributed channel inflow/outflow, however defined, is calculated in two parts.

Firstly, the I/O for the upstream half of the reach is calculated and added to or subtracted from the running hydrograph. Lastly, after the running hydrograph has been routed through the reach, the distributed I/O for the downstream half of the reach is determined and added to or subtracted from the running hydrograph. If the distributed I/O has been defined as a function of discharge (I/O Defn. Flag = 1 or 3), the function is always applied to the discharge at the current point in the model, not to that at the point where the distributed I/O commenced.

The definition of distributed channel inflow/outflow may be changed as often as desired. However, a new definition of distributed channel inflow/outflow may not occur while the previous definition is still operative; the ranges of the two definitions may not overlap. It is not feasible, for example, to define both a distributed inflow and a distributed outflow over the same reaches. On the other hand, concentrated inflow/outflow may occur at any model node regardless of whether or not distributed inflow/outflow is operative.

At every point where a concentrated inflow/outflow occurs, the location must be indicated to the program by a character string of not more than twenty-eight characters. Likewise, at every point where a distributed inflow/outflow commences, the nature of that inflow/outflow must be similarly indicated.

### 3.3.6 Limitation on Channel Inflow/Outflow Definition in Catchment Studies

In FIT runs, loss parameters are calculated for any interstation areas that exist in the model if gauging station hydrographs are input (see Table 1-1). This calculation is based on a volumetric balance, so the volumes of all inflows to and outflows from each interstation area must be known. If any such inflow or outflow is a function of discharge, the volume cannot be determined because the discharges can only be calculated after the loss parameters have been determined and the loss parameters cannot be determined until the discharges are known. It follows that channel inflow or outflow within a gauged interstation area may not be a function of discharge in a FIT run [unless it is an outflow that returns to the stream, or an inflow which originates as an outflow, within the interstation area].

The above restriction includes both concentrated I/O within an interstation area and distributed I/O that could have been defined upstream of the interstation area but is still operative. It does not affect inflows nor outflows that are not within an interstation area and does not apply in DESIGN runs at all. By virtue of the Inequality 3.2, channel outflow is always a function of discharge, however defined. In the case of inflow, the only permissible definitions in the relevant circumstances are a user-defined hydrograph and a constant in Equation 3-1; all other definitions of inflow are also functions of discharge.

### 3.3.7 Base Flow

Base flow is conceived of as the portion of total streamflow derived from seepage into the stream channels from underground. If the distributed inflow facility of RORB is to be used to model base flow, the user must have some conception of both the areal and the time distribution of the base flow. Little rational basis exists for defining these distributions and the suggestions below are not wholly rational but provide a simple procedure that is sometimes useful in FIT runs.

Firstly, the base flow component of the gauging station hydrograph at the catchment outlet must be identified. Procedures (necessarily arbitrary) for doing this are contained in most hydrology text books (e.g. Ref. 14). Chapter 4 of Reference 15 may also be useful. The base flow hydrograph for the catchment outlet can then be entered as a user-defined distributed channel inflow hydrograph, distributed over all reaches of the model. If this simplified and partly irrational procedure is unhelpful in fitting the total output hydrograph, the user may define the base flow at different points in the model more rationally or avoid the problem by modeling only the surface or direct runoff component of the hydrograph.

The latter procedure is valid for linear models and is satisfactory even for nonlinear models if the base flow is small relative to the surface runoff. However, it is more correct, especially for nonlinear models and most especially for high base flows, to include the base flow in the modeling process. The feature discussed in this section enables that to be done in a convenient and reasonable fashion but leaves the onus on the user to define the base flow properly.

### 3.3.8 Lateral Inflow/Outflow and Transmission Loss

In flood routing FIT runs, lateral inflow or outflow must generally be provided to maintain continuity between the volumes of the hydrographs at the upstream and downstream ends of the reach or series of reaches. The volume of this inflow or outflow is the difference between the hydrograph volumes, which can be obtained from a preliminary run without lateral inflow or outflow.

The time distribution of the lateral inflow/outflow usually has to be assumed arbitrarily. If it is taken as proportional to the hydrograph ordinates, the parameters of Equation 3-1 should be set as follows:

$$a = b = 0.0$$

$$d = 1.0$$

$$c = \frac{V_o - V_I}{NV_I} \qquad \textit{Equation 3-3}$$

where $V_O$ is the volume of the downstream hydrograph ($m^3$), $V_I$ is the volume of the upstream hydrograph ($m^3$), and $N$ is the number of reaches between the upstream and downstream hydrographs.

This formula for $c$ is satisfactory as long as the volume of lateral inflow or outflow does not exceed about 25% of the inflow hydrograph volume. For larger quantities of intermediate inflow or outflow, a more specific modeling of that process is justified.

An alternative assumption for lateral inflow or outflow is that it occurs at a constant rate. For this assumption:

$$b = c = d = 0.0$$

$$a = \frac{V_o - V_I}{3600 T \Delta T} \qquad \textit{Equation 3-4}$$

where $T$ is the duration of calculations (incs), and $\Delta T$ is the time increment (h).

Transmission loss, the depletion of flow often observed as streamflow hydrographs progress downstream due to processes of seepage into the bed and banks of the channel and the filling of depressions either in the channel bed or on the flood plain, presumably occurs mostly near the start of the hydrograph and may reasonably be modeled by a user-defined distributed outflow hydrograph starting at a high rate and falling rapidly to zero.

## 3.4 Loss Modeling

### 3.4.1 General

For catchment studies, in which area and rainfall are modeled, the program models the losses, which are deducted from the rainfall to produce rainfall-excess, which is routed through the catchment storage model.

In FIT runs, the parameters of the adopted loss model are determined interactively as part of the fitting process. In DESIGN runs, the loss parameters are hypothetical and are input, interactively, by the user.

If the total storm is being modeled as more than one burst, separate loss parameters are determined for each burst. In FIT runs, this enables the rainfall-excess for each burst to be equal to the surface runoff in the corresponding hydrograph rise. In DESIGN runs, it allows the loss parameters to be varied with time.

If there is more than one gauging station in a FIT run, separate loss parameters are determined for each interstation area (interstation areas are defined in Section 2.3.4). This enables the rainfall-excess in each interstation area to be equal to the volume of surface runoff from that area as indicated by the recorded hydrographs.

For a catchment whose outlet is downstream of the last gauging station, the loss parameters for the area downstream of the last gauging station cannot be determined. Also, the runoff from that area cannot be used in fitting. Nevertheless, in case the user should wish to print out a hydrograph for some point downstream of the last gauging station, provision is made in the program for rainfall-excess to be calculated for this area. For this purpose, the area is treated as an additional interstation area and is assumed to have loss parameters equal to those of the interstation area immediately upstream.

### 3.4.2 Loss Models

Two alternative models of the loss processes are provided:

(i) Initial loss followed by a runoff coefficient (constant proportional rates of loss and of runoff).

(ii) Initial loss followed by a constant (continuing) loss rate.

The first of these is recommended for urban or partly urban catchments but either model may be suitable on rural catchments.

Initial loss is a threshold process defined by a depth of loss that must be satisfied by the storm rainfall before any rainfall-excess occurs. The runoff coefficient is a volumetric runoff coefficient, not to be confused with the runoff coefficient of the rational formula. The continuing loss rate is a capacity rate of loss that occurs only if rainfall is equal to or greater than that rate. For less intense rainfall periods, the loss is equal to the rainfall.

The initial loss must be selected and input by the user. In FIT runs for which a gauging station hydrograph has been input, the runoff coefficient or loss rate is then calculated by the program so as to produce the correct volume of rainfall-excess (for the relevant burst and interstation area) as indicated by the hydrograph data. In other runs, the user must input the runoff coefficient or loss rate. At the end of each run, the parameters specified may be changed for a new run without re-running the entire program. Similarly, the adopted loss model may be changed.

Areal variability of loss parameters from sub-area to sub-area is provided through a sub-area data item called the fraction imperviousness. This term is in common use and is self-explanatory for urban areas. For rural sub-areas, the same data item may be input but it represents a runoff capacity index, a value of zero signifying the least runoff capacity for the catchment and unity the greatest. A runoff capacity index of unity would indicate a completely impervious or completely saturated sub-area. It might be noted that assessment of the relative runoff capacities of different sub-areas of rural catchments is usually speculative. Users are accordingly cautioned against rash usage of this facility for rural catchments.

For rural catchments on which the losses are assumed uniform over the catchment, or over each interstation area if there are more than one, fractions imperviousness (or runoff capacity indices) are not input and are set to zero by the program. This condition, which is normal for rural catchments, is indicated to the program by an `impervious area flag' value of zero. A value for this flag of unity, which is normal for urban or part urban catchments, indicates that one or more sub-areas have some impervious area or that areal variability of loss parameters is desired. In this case, the initial loss, runoff coefficient, and loss rate input by the user or calculated by the program are those applicable to the pervious area for urban catchments or the most pervious area for rural ones. The parameters for each sub-area are then calculated as follows:

$$IL_i = (1 - F_i)\, IL_{perv}$$

However, if $IL_{perv}$ exceeds sub-area rainfall and $F_i > 0$, $IL_{perv}$ is set equal to the sub-area rainfall.

$$C_i = F_i\, C_{imp} + (1 - F_i)\, C_{perv} \qquad C_{perv} \leq C_{imp} \qquad \textit{Equation 3-5}$$

$$C_i = C_{imp} \qquad C_{perv} > C_{imp} \qquad \textit{Equation 3-6}$$

$$CL_i = (1 - F_i) CL_{perv} \qquad \textit{Equation 3-7}$$

where *IL*, *C* and *CL* are respectively the initial loss, runoff coefficient and continuing loss rate, *F* is the fraction imperviousness (or relative runoff capacity), and subscripts *i*, *imp*, and *perv* respectively indicate the $i^{th}$ sub-area, the impervious area and the pervious (or most pervious) area. The values subscripted *perv* are the ones input by the user to the program or output by the program to the user.

The impervious area runoff coefficient $C_{imp}$ is set by the program to 0.9, reflecting the fact that losses occur even on nominally impervious surfaces in urban areas.

## 3.5 Values of Loss Parameters

Initial loss values on Australian catchments can vary from zero to around 50 mm or even to 75 mm in extreme cases. Often the initial loss dominates the loss processes. In FIT, adjusting the initial loss mainly affects the commencement of rise of the hydrograph. However, it can also affect the hydrograph peak, sometimes dramatically in the case of floods with relatively small amounts of runoff.

In DESIGN runs, an initial loss of zero would be appropriate if the maximum possible or probable flood were being estimated and a value of about 10 mm if the flood of a given recurrence interval were being estimated from a storm of the same recurrence interval.

Volumetric runoff coefficients are quite variable and users must select values based upon knowledge of their own catchments or on data given in handbooks such as *Australian Rainfall and Runoff* (Ref. 24).

Pervious area continuing loss rates also are quite variable but for design purposes on Australian catchments, values of two to three mm/h are often appropriate for average design conditions and about one mm/h or even less for extreme design conditions. Reference 24 may also be helpful in this connection.

### 3.5.1 Derivation of Loss Parameters in FIT runs

When the initial loss has been nominated by the user (for the current burst and interstation area if there are more than one of either), the program calculates the pervious area runoff coefficient or continuing loss rate by iteratively achieving a volume balance of rainfall-excess with measured surface runoff. This satisfies continuity but it might be noted that volumetric errors in the rainfall and hydrograph data will affect the values of the derived loss parameters. In effect, the rainfall and hydrograph data are treated as if they were free of error.

In calculating the rainfall-excess for an interstation area, a rainfall hyetograph is calculated for each sub-area and the initial loss and continuing loss indicated by a trial loss parameter deducted from it.

Since the program provides for the existence of storage reservoirs that may be drawn down at the start of the event, for concentrated and distributed channel inflows from sources other than rainfall-excess, and for concentrated and distributed channel outflows other than at the outlet of the interstation area, conventional loss calculation procedures cannot be used. Rather, it is necessary to perform a `volume routing' exercise for the entire channel network, appropriately incrementing or decrementing the volume of runoff at each upstream gauging station, sub-area, special storage, concentrated channel inflow or outflow and each reach for which there is distributed channel inflow or outflow, if that feature is in the current interstation area but not otherwise. At the end of this process, the volume should equal the volume of the hydrograph at the downstream end of the interstation area. If the two volumes differ by more than one twentieth of a millimetre over the interstation area, the difference is used to calculate a better trial loss parameter and the `volume routing' traverse repeated.

A limit on the number of iterations has been programmed and if this limit is exceeded, a message to this effect will be printed and the user given the option of accepting the current value or of terminating execution.

Checks on the calculated loss parameters have been programmed to guard against negative loss rates or runoff coefficients and runoff coefficients greater than unity. Negative loss rates can occur if the input initial loss is too large, whereas negative runoff coefficients can arise if the initial loss is too small; in the latter case the runoff volume from the impervious area exceeds the observed runoff volume before pervious area runoff is added. On the other hand, pervious area runoff coefficients greater than unity stem from insufficient contribution from the impervious areas, perhaps caused by the initial loss being too large. If these problems arise, the user can usually overcome them by changing the initial loss, and the program provides for this.

If for a particular burst/rise, there is no runoff from a particular interstation area which contains a drawn-down storage, the loss parameter is determinate only within a range of values. In such circumstances, the program makes a reasonable assumption to calculate the parameter, prints a message to this effect, and allows the user to change the computed value to any other value within the valid range if desired.

Channel inflow or outflow, whether concentrated or distributed, within an interstation area for which a loss parameter is being calculated may not be a function of discharge, unless it is concentrated inflow/outflow exactly balanced by an outflow/inflow, since the discharge cannot be known until the loss parameter has been determined. Apart from inflows and outflows that exactly balance each other within an interstation area, this prohibits all outflows and all inflows except those defined by a user input hydrograph or a constant inflow. These circumstances are checked by the program at the data reading and checking stage. This limitation, of course, does not apply outside of the interstation areas or in DESIGN runs or FIT runs with no gauging station hydrographs.

# 4 FORMULATION OF CATCHMENT/STREAM MODEL

## 4.1 Delineation of Sub-Areas, Model Storages, and other Features

In the recommended model, the catchment area, if any, is divided into sub-areas centred on stream channels and bounded by drainage divides. The channel network also is sub-divided into reaches, each of which is associated with a model storage. Other features, including storage reservoirs, retarding basins, other special storages, gauging stations, points at which the hydrograph is wanted such as dam sites, and points at which concentrated channel inflows or outflows occur or distributed channel inflows or outflows commence, must be represented in the model in proper relation to the sub-areas and reach storages. General definitions of the above features were given in Chapter 2. In this chapter, the steps necessary to define a particular catchment/stream system are given.

Applications of RORB frequently involve the modeling of the same area in both pre-dam and post-dam conditions. The ability to model a stream reach either in its natural condition or as drowned by a storage simply by changing its reach type code (see Section 2.2.3) means that exactly the same sub-areas and channel reaches can be used for both pre-dam and post-dam models. The only changes to the data that are necessary are the changes to the relevant reach type codes and the insertion of a special model storage representing the storage constructed. To achieve this, the formulation of the pre-dam model must take account of the proposed dam site and the extent of the storage it will create.

Delineation of all relevant features can be achieved by the following procedure:

### 4.1.1 Special Storages

1. Decide which retarding basins, storage reservoirs, or natural flood storages on the catchment, if any, are significant enough to be modeled specifically, delineate their storage areas [A', Figure 4-1], and place nodes at the outlets [B', Figure 4-1]. [If a storage area is small, it may be represented by a point instead of a detailed outline.] Place model storages at the outlet nodes to represent the special storages [Storage 17, Figure 4-2].

### 4.1.2 Streams

2. Decide tentatively which streams are significant enough to be modeled separately (all of those shown on Figure 4-1) and place nodes:

   (a) at the points where they enter any storage areas delineated in 1 [C', Figure 4-1].

   (b) at their confluences [D', Figure 4-1] including any such confluences located in a storage area delineated in 1 [E', Figure 4-1].

   (c) at points where outflows from the streams occur (10, 20, 30, 40, and 50 and the concentrated outflow point in Figure 2-1).

   (d) at the downstream extremity(ies) of the modeled stream network (the catchment outlet in Figure 4-1, the channel terminations in Figure 2-1).

### 4.1.3 Gauging Stations and Other Points of Special Interest

3. If there are gauging stations, dam sites, points at which concentrated channel I/O occurs or distributed channel I/O commences, or other points of special interest on the stream that have not been marked in 1 or 2, place nodes at such points.

### 4.1.4 Sub-areas (If no sub-are is being modeled, proceed to Item 9.)

4. In drawing sub-area boundaries, consider only those streams being modeled separately and ignore all others, except insofar as they help to indicate the location of ridge lines. From each of the confluence nodes specified in 2(b) in turn, draw sub-area boundaries along the ridge lines between the streams that join at the confluence until the catchment boundary, another sub-area boundary, or the point of divergence of an effluent stream from a main stream is reached [see long dashed lines, Figure 4-1]. Multiple channel terminations are treated like the branches joining at a confluence; in this case the ridge lines between them start at the catchment boundary joining the channel terminations and end at a point of divergence of an effluent stream from a main stream.

5. From each of the nodes, confluence or otherwise, specified in 1, 2, or 3, draw sub-area boundaries up the valley side slopes on each side of the node, along the lines of steepest slope from the node to the adjoining ridges, then along such ridges in an upstream direction until another sub-area boundary or the catchment boundary is reached [see short dashed lines, Figure 4-1].

6. If any of the sub-areas formed in 4 and 5 has an area larger than about 25% of the total catchment area, or a length greater than about one third the length of the main stream, consideration should be given to subdividing it either by:

   (a) modeling one of its principal tributaries as a separate stream and proceeding as in 2 to 5, or

   (b) from a point or points (not to be considered nodes of the model) on the sub-area's main stream, approximately at its mid-point, third points, quarter points, etc. as the case may be, drawing sub-area boundaries as in 5. [See F', Figure 4-2]

7. Relatively small sub-areas frequently occur between nodes that are close together on the same stream. If any of the sub-areas formed in 4 to 6 is smaller than about 5% of the total catchment area, consideration should be given to eliminating it as follows:

   (a) if the sub-area has nodes at both its upstream and downstream ends, by eliminating the sub-area boundaries drawn from the upstream node under 5, and extending the sub-area boundary drawn from the upstream node under 4 along the line of the stream to the downstream node. For example, compare G' and H', Figure 4-2 with the same regions on Figure 4-1. The upstream node may be left or may be eliminated under Item 9 below.

   (b) if the sub-area has a node only at its downstream end, by eliminating the sub-area boundary drawn from that node under 4. For example, although it is not recommended, sub-area A, Figure 4-2, could be merged with the adjacent one by deleting Boundary I'.

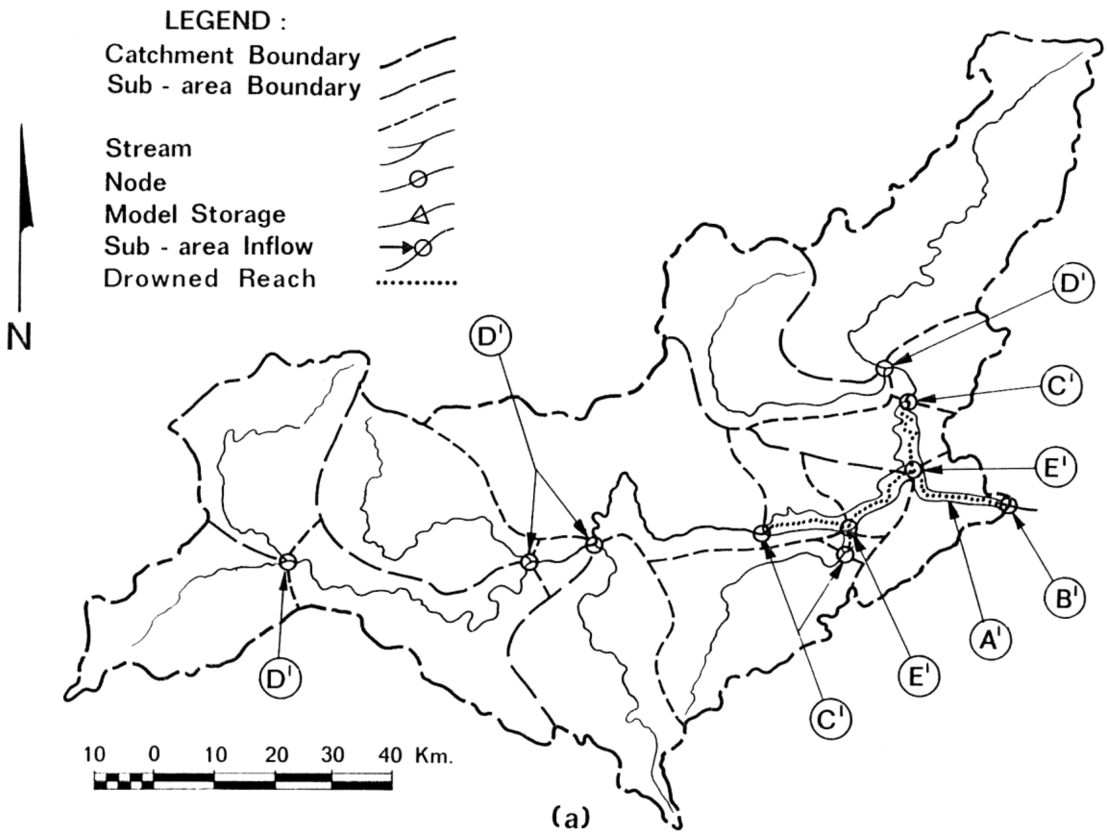


**Figure 4-1 Sub-division of catchment and placement of nodes (a) Intermediate**

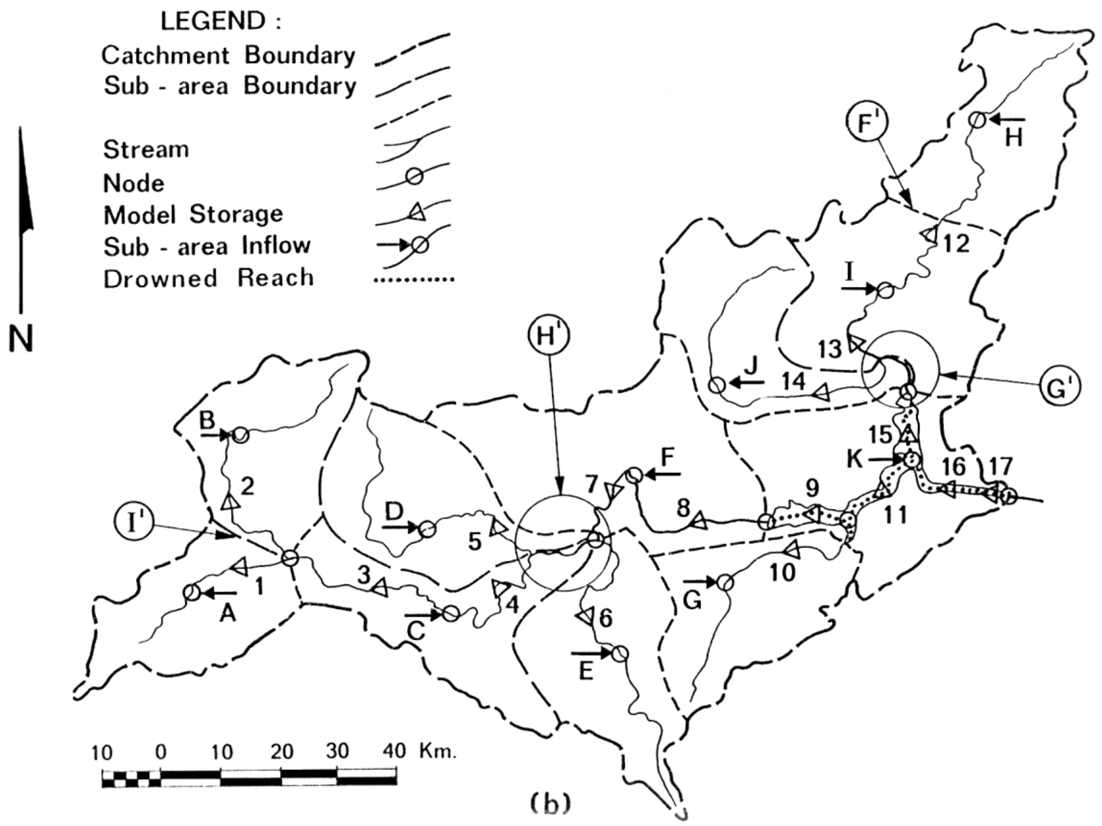


**Figure 4-2 Sub-division of catchment and placement of nodes (b) Final**

### 4.1.5 Sub-area Entry Points

8. Place a node in each sub-area at a point on its main stream adjacent to the centroid of the sub-area. [See circles marked with arrows, Figure 4-2.] These are the sub-area entry points, or points where the rainfall-excess enters the catchment storage model. The main stream of a sub-area is defined as the stream or stream segment within it that is being separately modeled. It is not uncommon for the main stream segment to be not centrally located in the sub-area and/or to be shorter than tributary streams originating in the sub-area but not being separately modeled.

### 4.1.6 Reach Storages

9. If any two or more adjacent nodes formed in the above procedure are sufficiently close together that the storage between them would have a negligible effect on the hydrograph, they may be merged and treated as a single node serving all the purposes of the original ones.
10. Place model reach storages on the stream segments between all pairs of adjacent nodes [see Figure 4-2].

## 4.2 Model Operations Available

In order that the program may simulate any given catchment/stream system, it performs a sequence of operations in an order controlled by the data file. Each available operation is identified by a number called its control code. Table 4-1 lists the available operations and their control codes. Details of the operations that occur when the various control codes are encountered are as follows:

**Code 0: End of Control Vector**. This is not a model operation; the code merely signals the end of the control vector. It must occur once, and only once, at the end, in every control vector.

**Code 1: Sub-area Inflow.** As for Code 2, but this code is used for each sub-area that does not have another sub-area upstream of it.

**Code 11**: As for Code 12.

**Code 2: Add in Sub-area Inflow and Route**. The rainfall-excess of the next sub-area, in the form of a hydrograph, is added to the running hydrograph and the combined hydrograph routed through the next reach storage of the model. This code is used for each sub-area that does have one or more sub-areas upstream of it.

**Table 4-1 Model Operations and Control Codes**

| Control Code | Operation |
|---|---|
| 0 | Nil. This code signifies end of control vector. |
| 1* | As for 2, except that running hydrograph is zero. |
| 2* | Add sub-area rainfall-excess hydrograph to running hydrograph and route through next reach storage. |
| 3 | Store running hydrograph. |
| 4* | Add stored hydrograph to running hydrograph. |
| 5* | Route running hydrograph through next reach storage. |
| 6* | Route running hydrograph through existing special storage. |
| 6.1* | Interactive design of special storage. |
| 7 | Print results for a point at which there is no actual hydrograph. |
| 7.1 | Print results at a gauging station, where there is an actual hydrograph for comparison. |
| 7.2 | Inset a 'dummy' gauging station, to allow for different model parameters if an interstation area would not otherwise be present |
| 8* | Translate hydrograph in time, forward or backward. |
| 9* | Channel inflow or outflow, concentrated or distributed. |

*Add 10 to given control code for relevant hydrograph information (see Section 9.1), including graph if desired, to be printed.

**Code 12**: As for Code 2 and, in addition, information on the sub-area rainfall-excess hydrograph and the running hydrographs upstream and downstream of the sub-area inflow point and at the downstream end of the reach is printed.

**Code 3: Store Hydrograph.** The running hydrograph is stored on the top of a stack of stored hydrographs. This code must be used upon any arrival except the last at a confluence or channel termination.

**Code 4: Add in Stored Hydrograph.** A hydrograph is removed from the top of the stack of stored hydrographs and added to the running hydrograph. This code must be used upon any arrival except the first at a confluence or channel termination

**Code 14**: As for Code 4 and, in addition, information on the two hydrographs being combined is printed.

**Code 5: Route Hydrograph.** The running hydrograph is routed through the next reach storage of the model. This code is used for reach storages into which there is no sub-area inflow.

**Code 15:** As for Code 5 and, in addition, information on the upstream and downstream running hydrographs is printed.

**Code 6: Existing Special Storage**. Relevant data are read and the running hydrograph is routed through the storage reservoir, retarding basin or other special storage, which may or may not be drawn down at the start of the event.

**Code 16**: As for Code 6 and, in addition, information on the inflow and outflow hydrographs is printed.

**Code 6.1: Special Storage to be Designed.** Relevant data are read from file and the user is required to input from the terminal the initial drawdown, if any, and the dimensions of the pipe outlet(s), if any, and spillway(s). The inflow hydrograph is then routed through the basin. Peak outflow, peak water level, and peak storage are printed and the user is then given the opportunity to change the basin characteristics and repeat the process. When this option is declined, the program proceeds to the next model operation.

**Code 16.1**: As for Code 6.1 and, in addition, information on the inflow and outflow hydrographs is printed.

**Code 7: Print Calculated Discharge.** Information on the calculated hydrograph is printed. The user may then exercise an option to print a graph of the hydrograph and, if the code occurs in an interstation area, the rainfall-excess hyetograph of the sub-catchment above the downstream end of that interstation area. This code may be used anywhere in the sequence of model operations.

**Code 7.1: Print Calculated and Actual Discharges.** Information on the calculated and actual hydrographs and various measures of the difference between them is printed. The user may then exercise an option to print a graph of the hydrographs and, if the gauging station is at the downstream end of an interstation area, the rainfall-excess hyetograph of the sub-catchment upstream of the gauging station. This code must be used wherever there is a gauging station (and actual hydrograph input).

**Code 7.2. Insert a 'dummy' gauging station**. This has the effect of treating the upstream area as an extra interstation area (Section 2.3.4), and provides the opportunity for the user to specify different loss and/or routing parameters for that area.

**Code 8: Translate Hydrograph**. The running hydrograph is translated without change in shape by $n$ time increments, where $n$ is a positive or negative integer input as data. If $n$ is positive (negative), the last (first) $n$ ordinates of the hydrograph are lost and $n$ zero ordinates are added to the start (end) of the hydrograph. This code is not required in the recommended form of catchment storage model but is required for some other forms.

**Code 18**: As for Code 8 and, in addition, information on the running hydrographs before and after translation is printed.

**Code 9: Channel Inflow and Outflow (I/O)**. Depending upon values of data items associated with this code, concentrated or distributed inflow or outflow may occur. If concentrated, a hydrograph is added to or subtracted from the running hydrograph at the current node, subject to the adjusted hydrograph being non-negative. If distributed, hydrographs proportional to reach length are added to or subtracted from the running hydrographs of a specified number of reaches, commencing at the current node, partly at the upstream end of the reach and partly at the downstream end. Again, the running hydrograph may at no stage become negative in this process.

At nodes where an effluent stream diverges from a main stream and both are to be modeled, this code must be used twice, firstly to define the outflow and, later, to recover the stored hydrograph for use as a concentrated inflow to the branch being modeled second. (See also Sections 2.1.2, 2.1.3, and 3.3.5).

*Depending on whether the user sets an outflow to be main stream or effluent stream to be modelled next can change the value of* $d_{av}$ *(and thus the relative delay time) of the catchment. Thus changing the order of the calculations undertaken in the catchment in this manner can change the results of the model, if the routing parameter* $k_c$ *is not also modified to allow for this.*

**Code 19**: As for Code 9 and, in addition, for concentrated I/O, information on the I/O hydrograph and the running hydrograph downstream of the I/O point is printed while, for distributed I/O, information on the I/O hydrograph and the running hydrograph at the downstream end of the reach is printed for each reach to which the distributed I/O applies. [Distributed I/O is sometimes called lateral I/O.]

## 4.3 Specifying Sequence Of Model Operations

The user must specify the sequence of operations required to model the catchment by including in the data file the corresponding sequence of control codes. This sequence of codes is called the *control vector*.

Most of the operations require a data input. The data associated with each code are included in the data file immediately after the code. Full details of the data items required by the control vector, referred to collectively as the storage data since they characterize the model storages, are given in Chapter 5.

To construct the control vector for a given catchment/stream system, the channel network is traversed in accordance with the rules set out below and at each model node, defined by the procedure of Section 4.1 above, the control code(s) appropriate to that node is(are) recorded.

**Definition**: The term `upstream end' in the following rules is defined as either:

(i) a node on a model stream that has no node upstream of it; or

(ii) at a point of channel outflow, where one model stream diverges from another, a point on one of the branches, adjacent to the point of divergence.

### 4.3.1 Rules for Channel Network Traverse

1. Start at an upstream end, conventionally one remote from the catchment outlet, and proceed downstream.

2. If a confluence or channel termination is encountered, jump to another upstream end whose stream has not yet been traversed and which drains (even if by way of a channel outflow and an effluent stream) to the confluence concerned or to another channel termination, whichever is relevant. Again proceed downstream along the relevant drainage path.

3. At each confluence, apply Rule 2 until all streams entering the confluence have been traversed, and then proceed downstream.

4. If a channel outflow point at which the outflow enters a model stream is encountered, identify the point uniquely by a non-zero integer and mark the stream branch not being traversed as an upstream end.

5. Proceed as above until the entire channel network has been traversed.

The control vector formed by the above procedure may be completed by inserting print codes (**7**) wherever required, translation codes (**8)** or (**18**) if and where required, and a Code **0** at the end.

As an example a control vector for the catchment of Figure 4-2, together with the sub-area, model storage, and gauging station labels, is shown in Table 4-2. It will be apparent that a variety of valid control vectors is possible for any given catchment. Other examples, including one with channel inflows and outflows and a looped stream network, are included in Chapter 9.

**Table 4-2 Control Vector and Model Labels for Catchment of Figure 4-2**

| Code | Sub-Area | Storage | Code | Sub-Area | Storage | Code | Sub-Area | Storage | Station |
|---|---|---|---|---|---|---|---|---|---|
| 1 | A | 1 | 4 | | | 3 | | | |
| 3 | | | 5 | | 7 | 1 | J | 14 | |
| 1 | B | 2 | 2 | F | 8 | 4 | | | |
| 4 | | | 5 | | 9 | 5 | | 15 | |
| 5 | | 3 | 3 | | | 4 | | | |
| 2 | C | 4 | 1 | G | 10 | 2 | K | 16 | |
| 3 | | | 4 | | | 6 | | 17 | |
| 1 | D | 5 | 5 | | 11 | 7.1 | | | 1 |
| 4 | | | 3 | | | 0 | | | |
| 3 | | | 1 | H | 12 | | | | |
| 1 | E | 6 | 2 | I | 13 | | | | |

## 4.4 Limits on Model Array Storage

Comments on the appropriate degree of subdivision of the catchment area, the stream system, and the storm/flood duration have been made in Section 2.1.4, 2.3.2 and 3.1.2. The degree of such subdivision, the size and complexity of the catchment, the density of its instrumentation, and the duration of the event combine to determine the amount of computer memory required for a particular run. While no specific upper limit exists on the value of any particular model parameter, there are three overall limits, on the total real array storage, the total integer array storage, and the total character array storage used by the program at any one time.

The model parameters that combine to determine the overall array storage requirements are the numbers of sub-areas, inter-station areas, normal (reach) storages, special storages (reservoirs and retarding basins), translations, channel inflow or outflow locations, result print-out locations, pluviographs, hyetograph ordinates, rainfall bursts, gauging stations, hydrograph ordinates for both actual and calculated hydrographs, control codes in the control vector, pairs of values in storage-discharge and storage-elevation tables of special storages, and in discharge-I/O tables, and the maximum stream order of a branched channel network. No specific upper limit exists on the value of any of those variables.

RORB has been configured to provide sufficient array storage for most problems envisaged for the program. If a message appears that indicates the inbuilt limits have been exceeded, the user should consider whether the task has been over-defined and reduce the level of detail. If this is not possible, he or she should contact the program authors (Section 1.8) and request a 'larger' version.

## 4.5 Alternative Models

### 4.5.1 Miscellaneous Examples

RORB can be used with catchment models other than the one defined in Section 4.1. For example, a model in which a lumped catchment rainfall-excess is routed through a cascade of storages (the *Nash* model) could be represented by the following control vector, assuming, for this example, a cascade of 5 storages:

1, 5, 5, 5, 5, 7, 0

A model in which the catchment was divided by isochrones into say five sub-areas, with rainfall-excess being translated to the outlet and then routed through a linear storage (the *Clark* model) would have the following control vector:

1, 8, 3, 1, 8, 4, 3, 1, 8, 4, 3, 1, 8, 4, 3, 1, 8, 4, 5(or 6), 7, 0

The model developed by *Laurenson* using isochrones to subdivide the catchment into say five sub-areas and with a nonlinear storage between each pair of sub-area nodes and between the last sub-area node and the outlet would have the following control vector:

1, 2, 2, 2, 2, 7, 0

*Dooge*'s proposed model with the storage in any channel reach represented by a translation and a linear concentrated storage could be represented in the manner detailed in Section 4.1 to 4.3 but adding a Code 8 (translation) after each sub-area or reach storage code (1, 2, or 5).

Two-layer models with one set of storages for surface runoff and another for sub-surface runoff could be handled by having two sets of storages in parallel. It would be necessary to use Codes 6 or Codes 1, 2 and 5 with some multiple of the real reach length for the sub-surface storages and a Code 9 to input the sub-surface flow to the system.

### 4.5.2 Separate Modeling of Sub-area Storage

It is possible with RORB to model the sub-area storage separately and differently from the channel storage. To do this, the sub-area is assumed to be separated from its entry point to the stream network by a storage or series of storages representing the sub-area storage. When, in the stream network traverse described in Section 4.3, a sub-area node is encountered, a Code 3 is used to store the running hydrograph. Routing of the sub-area rainfall-excess through sub-area storage is then performed in one of several possible ways after which the sub-area flow is added to the running hydrograph by using a Code 4 and the hydrograph is then routed through the next channel reach storage using a Code 5.

A simple way of routing the sub-area rainfall-excess through the sub-area storage is to use a Code 1 with an associated reach length representative of the overland flow distance of the sub-area. The reach type (see Section 2.2.3) for the overland flow storage could be different from those of the channel storages but the exponent $m$ and the coefficient $k_c$ applicable to the entire catchment would also apply to the overland flow storage.

To remove the latter restriction, the sub-area storage could be treated as an existing special storage (see Section 2.4), indicated by a Code 6 in the control vector. In this case, the user would have to determine appropriate $k_s$ and $m_s$ values (see Equation 2-6) for each sub-area based on an assumed model of the sub-area geometry and the Manning formula for sub-area discharge. It is usually difficult, however, to choose an appropriate value of Manning's $n$ and general advice on this cannot be given.

Both of the above procedures treat the sub-area rainfall-excess as lumped, all of it passing through the same sub-area storage. It is possible for the sub-area storage to be distributed into say N equal amounts, with 1/Nth of the sub-area rainfall-excess being added in to each of the N storages, which can be either reach storages or special storages. This is achieved, deviously, as follows:

1. Use a Code 1 with an associated reach length of zero to set the running hydrograph equal to the sub-area inflow hydrograph.

2. Divert the entire running hydrograph from the channel, using a Code 9 and values of a = b = 0.0 and c = d = 1.0 in Equation 3-1, so that it can then be used as a distributed channel inflow.

3. Use the hydrograph from (2) as a channel inflow hydrograph, distributed over N reaches, again using a Code 9 with appropriate data items.

4. Route the distributed inflow through N storages, equal amounts of inflow entering at each storage.
   (a) If these N storages are to be treated as normal reach storages with the same $m$ and $k_c$ values as the rest of the catchment, but possibly with different reach types, each will be represented by a Code 5 with an associated reach length of
   (b) L/N where L is the total length of the sub-area overland flow. If the N storages are to be treated as special storages with $k_s$ and $m_s$ values (for Equation 2-6) determined

by the user as discussed above for the lumped case, there will be N pairs of control codes 5 and 6 thus

- a Code 5 with a small associated reach length, say 1.0E-10 (this ensures that the distributed inflow is distributed equally but is small enough to have no noticeable effect on the hydrograph).
- a Code 6 with associated data including $k_S$ and $m_S$ values.

It might be observed that treating the sub-area storage separately and differently from the channel storage is achieved with some awkwardness and extension of the control vector and data file. However, it is likely to be necessary only in very unusual circumstances, when special attention is focussed on some proposed change to one or a few sub-areas. For the great majority of RORB applications, the model recommended in Section 4.1 is appropriate.

### 4.5.3 Watershed Bounded Network Model WBNM

The WBNM developed by Boyd et al. (References 16 p. 181; 19; 20) uses the same catchment subdivision as the RORB model. For each sub-area, rainfall-excess is routed through one model storage to the sub-area's outlet, while upstream flow, if any, is routed through a different model storage, also to the sub-area's outlet. The delay times of these two storages are proportional to functions of the sub-area's area, not to a reach length as in the RORB model.

RORB can be used to apply the WBNM as follows:

1. Catchment subdivision is performed as in Section 4.1, Items 4-7.
2. Two nodes are placed in each sub-area, one at its outlet and the other at any point not on the sub-area's main stream. The latter node is the sub-area entry point, and is joined to the node at the sub-area outlet by a notional stream. No other nodes are used.
3. Model storages are placed between all pairs of adjacent nodes, as in Section 4.1, Item 10. Relative delay time indicators (used in place of reach length in Item 1.3 of Table 5-2) are:
   (a) for model storages between a sub-area entry point and the sub-area outlet, $A^{0.57}$, where A is area in $km^2$ of the sub-area, and
   (b) for model storages between the upstream and downstream ends of a subarea, $0.6A^{0.57}$.
4. The control vector is formulated as specified in Section 4.3, noting that, for all sub-areas having an inflow at the upstream end as well as a rainfall-excess input, the downstream end is a confluence of the sub-area's main stream and the notional stream referred to in (ii) above. Control codes 3 and 4 are needed at those points.
5. To operate RORB in strict compliance with the WBNM model, the m parameter, which is entered interactively, should be 0.77; in practice, other values can be used to improve the fit.

If it is desired to calculate the WBNM c parameter, this can be done using the equation:

$$c = \frac{k_c}{d_{av}} \qquad \textit{Equation 4-1}$$

where $d_{av}$ is obtained from Equation 2-3. [RORB prints the value of $d_{av}$ at the foot of the sub-area data in its output text file.]

# 5 INPUT DATA

## 5.1 General

Most of the input required by the program must be stored in a disk file before program execution. This input is referred to as the datafile (or the file data) and described in detail in this chapter. Chapter 6 describes the Graphical Editor available to input data for the catchment file. The interactive input required during program execution is described in Chapter 7.

## 5.2 Units

The units etc. to be used for input data, and which are used for output, are shown in Table 5-1.

**Table 5-1 Units**

| Measures | Quantity | Units |
|---|---|---|
| DEPTHS | Rainfall | *mm* |
| | Initial Loss | *mm* |
| RATES | Continuing Loss Rate | *mm/h* |
| | Discharge | $m^3/s$ |
| TIMES | Time Increment | *h* |
| | All Other Times | Time increments (incs) |
| AREAS | Catchment, sub-catchment, inter-station, and sub-area areas | $km^2$ |
| VOLUMES | Storage | $m^3$ |
| | Initial Drawdown Volume | $m^3$ |
| | Volumes of Separate Hydrograph Rises | Arbitrary |
| ELEVATIONS | Elevations | *m* |
| LENGTHS | Channel Reach Lengths | *km* |
| | Pipe Outlet Length | *m* |
| | Spillway Effective Length | *m* |
| RATIOS | Slope of Reach | % |
| | Slope of Pipe | % |
| | Runoff Coefficient | Fraction |
| | Fraction Imperviousness | Fraction |

## 5.3 Data Items Required

The data items required in the file are detailed in Table 5-2. The actual items required for any particular run depend on the available data, the specific details of the catchment model, and whether the run is a FIT or DESIGN run.

For a catchment and storm for which there are little rainfall and/or streamflow data, many of the items listed in Table 5-2 are not required. Likewise for a small catchment or one with a simple model, only some of the data items will be necessary. Omissions are indicated in Table 5-2 by the directions in the column headed 'Skips', which direct the user to the next required data item if some listed items are to be omitted.

To prepare data for the file, the user should trace through Table 5-2 item by item following directions in the 'Skips' column and the footnotes, and recording the values of data items as their descriptions are encountered, until the `stop' instruction is reached.

The data file has the following features:

(i) All relatively constant data, relevant to the catchment, occur first, followed by the more ephemeral data, relevant to a particular storm event and to its usage as a FIT event or as a DESIGN storm. Note: these can be placed in separate '.cat' and '.stm' files for convenience if several storms are envisaged; for use of the generated design storm feature, only the '.cat' file is needed. (See instruction at top of Table 5-2).

(ii) Arbitrary descriptive comments may optionally be incorporated in the data file to facilitate its interpretation and editing. This can be done in two ways as detailed in the footnotes to Table 5-2. It is recommended that use be made of this feature.

(iii) Numerical data (but not character data) are in free format. Numerical data values must be separated by commas or blanks. Blanks adjacent to these separators are ignored. An end of record mark has the effect of a blank. Character data values should not be enclosed in single quotes.

(iv) Arrays and numerical data relating to each of the model storages are terminated by an integer constant -99 to facilitate checking of the data file.

(v) Character strings and the items immediately following character strings all start new lines of the data file.

## Table 5-2 File Data Items

NOTES:
1. ⇒ indicates 'skip to'.
2. Numerals in 'Skips' column are Item Nos. in this table.
3. -99 is an integer constant included for data checking purposes.
4. References to control codes < 10 imply also the given codes + 10 if they exist.
5. Use whole table for a full (.dat) data file. For preparation of a catchment (.cat) file, stop at Item 7.0. For preparation of a storm file (.stm), start at Item 7.1.

| Item No. | Data Item(s) | Skips | Ref. Sect. |
|---|---|---|---|
| | **0. CATCHMENT DATA** | | |
| 0.1 † | Catchment name (string < 69 char.) | | |
| 0.2 * | Reach type flag: 0 if more than one reach type (see 1.2 below) occurs; 1, 2, 3 or 4 if all reaches are of type 1, 2, 3 or 4 respectively.** | | 2.2.3 |
| 0.3 * | Control code ⇒ | If Code:<br>= 0** ⇒ 6.0<br>= 1* or 11* ⇒ 1.1<br>= 2* or 12* ⇒ 1.1<br>= 3** ⇒ 0.3<br>= 4** or 14** ⇒ 0.3<br>= 5* or 15* ⇒ 1.1<br>= 6** or 16** ⇒ 2.1<br>= 6.1** or 16.1** ⇒ 2.1<br>= 7** ⇒ 3.1<br>= 7.1** ⇒ 0.3<br>= 7.2** ⇒ 3.1<br>= 8* or 18* ⇒ 4.1<br>= 9* or 19* ⇒ 5.1 | 4.2 |
| | **1. CHANNEL STORAGE DATA** | | |
| 1.1 | ⇒ | If reach type flag ≠ 0 ⇒ 1.3 | |
| 1.2 | Reach type code: 1 if reach in natural condition, 2 if excavated but unlined, 3 if lined channel or pipe, 4 if drowned. | | 2.2.3 |
| 1.3 | Length, km, of reach represented by the model storage (or other function of reach properties related to travel time) ⇒ | If reach type code = 1 or 4, ⇒ 1.5 | 2.2.2 |

† Start a new line
* Start a new line, optionally preceded by any number of comment lines, each of which has a C in Column 1.
** This item may optionally be followed by a comma and a comment on the same line as the data item.

(Table 5.2 cont.)

| Item No. | Data Item(s) | Skips | Ref. Sect. |
|---|---|---|---|
| 1.4 | Slope of reach, % | | 2.2.3 |
| 1.5 | -99** ⇒ | ⇒ 0.3 | |
| | **2. RESERVOIR STORAGE DATA** | | |
| 2.1 † | Name of storage (string < 29 char.) | | |
| 2.2 * | Discharge relation flag:<br>0 for formula $S = 3600 k_s Q^{m_s}$, 1 for S-Q table,<br>2 for weir & pipe formulas, 3 for weir formula only ⇒ | If control code = 6.1,<br>⇒ 2.7 if Item 2.2 = 2 or 3<br>⇒ 2.17 if Item 2.2 = 0<br>(Item 2.2 = 1 illegal for 6.1) | 2.4.5 |
| 2.3 | Initial drawdown of storage:<br>For no drawdown i.e. initial water level at elevation of lowest outlet level, enter zero; for a volume below that elevation enter a negative volume ($m^3$); for a water level below that elevation enter a positive elevation not less than 1.0E-10 ($m$) ⇒ | If flag (Item 2.2):<br>= 0, ⇒ 2.13<br>= 1, ⇒ 2.14 | 2.4.4 |
| 2.4 | Number of separate spillways (≥ 1)** | | 2.4.5 |
| 2.5 † | Crest elevation, *m*, effective length, *m*, for lowest elevation spillway | | 2.4.5 |
| 2.6 | Repeat of data in item 2.5 for additional spillways | | |
| 2.7 | Weir coefficient $K_W$ in $Q = K_W L_s H^{3/2}$,<br>all spillways ⇒ | If flag (Item 2.2)<br>= 3, ⇒ 2.17 | 2.4.5 |
| 2.8 | Entrance loss coefficient, $k_e$, all pipes | | 2.4.5 |
| 2.9 | Bend loss coefficient, $k_b$, all pipes ⇒ | If control code<br>(Item 0.3) = 6.1, ⇒ 2.17 | 2.4.5 |

† Start a new line.
* Start a new line, optionally preceded by any number of comment lines, each of which has a C in Column 1
** This item may optionally be followed by a comma and a comment on the same line as the data item.

(Table 5.2 cont.)

| Item No. | Data Item(s) | Skips | Ref. Sect. |
|---|---|---|---|
| 2.10 | Number of separate pipe outlet systems (≥ 0) i.e. pipe groups having different diameters or elevations** | | 2.4.5 |
| 2.11 † | For lowest elevation outlet pipe(s): Length, *m*; grade, %; entrance invert elevation, *m*; number of pipes; diameter of pipe(s), *m*. | | 2.4.5 |
| 2.12 | Repeat of data in item 2.11 for additional pipe outlet systems. ⇒ | ⇒ 2.17 | 2.4.5 |
| 2.13 | Coefficient, $k_S$, exponent $m_S$ in $S = 3600 k_s Q^{m_s}$ ⇒ | ⇒ 2.17 | 2.4.5 |
| 2.14 | No. of pairs of values in *S* - *Q* table ** | | 2.4.5 |
| 2.15 † | First (smallest) pair of storage (*S*), $m^3$, discharge (Q), $m^3/s$, values | | 2.4.5 |
| 2.16 | Succeeding pairs of storage, $m^3$, discharge, $m^3/s$, values | | |
| 2.17 | -99** | | |
| 2.18 * | Elevation (*H*) - storage (*S*) relation flag:<br>0 if there is none (this value illegal if discharge relation flag (Item 2.2) = 2 or 3)<br>1 for *H* - *S* table<br>2 for formula $S = a(H - H_o)b$ ⇒ | If this item = 0, ⇒ 2.23<br>If this item = 2, ⇒ 2.22 | 2.4.5 |
| 2.19 | No. of pairs of values in *H* - *S* table ** | | 2.4.5 |
| 2.20 † | First (smallest) pair of $H(m)$, $S(m^3)$ values | | 2.4.5 |
| 2.21 | Succeeding pairs of $H(m)$, $S(m^3)$ values ⇒ | ⇒ 2.23 | 2.4.5 |
| 2.22 | Parameters *a*, *b* & $H_O(m)$ in storage-elevation relation, $S = a(H - H_O)^b$ | | 2.4.5 |
| 2.23 | -99** ⇒ | ⇒ 0.3 | |

† Start a new line.
* Start a new line, optionally preceded by any number of comment lines, each of which has a C in Column 1.
**This item may optionally be followed by a comma and a comment on the same line as the data item.

(Table 5.2 cont.)

| Item No. | Data Item(s) | Skips | Ref. Sect. |
|---|---|---|---|
| | **3. PRINT-OUT LOCATION** | | |
| 3.1 † | Location of print-out (string < 29 characters) ⇒ | ⇒ 0.3 | |
| | **4. TRANSLATION DATA** | | |
| 4.1 | No. of time inc. hydrograph to be translated | | 2.5 |
| 4.2 | -99** ⇒ | ⇒ 0.3 | |
| | **5. CHANNEL INFLOW/OUTFLOW DATA** | | |
| 5.1 *** | Channel inflow/outflow definition flag:<br>0: User supplied hydrograph;<br>1: Formula ( Equation 3-1 & Equation 3-2);<br>2: Previously identified inflow or outflow<br>3: Table of I/O vs. channel discharge | | 3.3.5 |
| 5.2 | N, where I/O is distributed over next N reaches.<br>N = zero if I/O concentrated at current node.<br>N = -1 if I/O distributed over all remaining reaches. | | 3.3.5 |
| 5.3 | Inflow/outflow type flag:<br>1: Inflow<br>0: Outflow, main stream to be modeled next;<br>-1: Outflow, effluent (outflow) stream to be modeled next; | | 2.1.2 |
| 5.4 | Non-zero integer to identify I/O if required for later use or, if Item 5.1 = 2, to identify the previous I/O that defines this one, zero otherwise ⇒ | If Item 5.1 = 0 or 2, ⇒ 5.10 | 3.3.5 |
| 5.5 † | Location of inflow/outflow (string < 29 char.) ⇒ | If Item 5.1 = 3, ⇒ 5.7 | 3.3.5 |
| 5.6 * | Parameters $a$, $b$, $d$, $d$ of I/O ($D$) vs. discharge ($Q$) relation, $D = a + c(Q - b)^d$ ⇒ | ⇒ 5.10 | 3.3.5 |

† Start a new line.
* Start a new line optionally preceded by any number of comment lines, each of which has a C in Column 1.
** This item may optionally be followed by a comma and a comment on the same line as the data item.
*** On the same line as previous item.

(Table 5.2 cont.)

| Item No. | Data Item(s) | Skips | Ref. Sect. |
|---|---|---|---|
| 5.7 * | No. of pairs of values in discharge (Q) - I/O (D) table** | | 3.3.5 |
| 5.8 † | First (smallest) pair of $Q(m^3/s)$, $D(m^3/s)$ values (D = 0) | | |
| 5.9 | Succeeding pairs of $Q(m^3/s)$, $D(m^3/s)$ values | | 3.3.5 |
| 5.10 | -99** ⇒ | ⇒ 0.3 | |
| | **6. SUB-AREA DATA** | | |
| 6.0 | ⇒ | If no sub-areas, ⇒ 7.0 | |
| 6.1 * | Areas, $km^2$, of sub-A, B, ... | | 2.3.2 |
| 6.2 | -99** | | |
| 6.3 * | Impervious area flag: 0 if there is no impervious area (or runoff capacity indices = 0 for all sub-areas), 1 otherwise ⇒ | If this item = 0, ⇒ 6.5 | |
| 6.4 | Fractions imperviousness (or runoff capacity indices) of sub-areas A, B, ... | | 3.4.2 |
| 6.5 | - 99** | | |
| 7.0 | **STOP here if preparing a catchment file (.cat). Subsequent data (if required) to be a separate storm (.stm) file** | | |
| | **7. STORM DATA** | | |
| 7.1 † | Storm identification (string < 69 char.) (Initial time if FIT run) | | 3.1.2 |
| 7.2 † | Type of run (FIT or DESIGN anywhere in first 6 columns) | | 1.2 |
| 7.3 * | Time increment, $h$ | | 3.1.2 |

† Start a new line.
* Start a new line optionally preceded by any number of comment lines, each of which has a C in Column 1.
** This item may optionally be followed by a comma and a comment on the same line as the data item.

(Table 5.2 cont.)

| Item No. | Data Item(s) | Skips | Ref. Sect. |
|---|---|---|---|
| 7.4 | No of time incs. for which calcs req. ⇒ | If no sub-areas, ⇒ 7.8 | |
| 7.5 | No. of separate rainfall bursts | | 3.1.3 |
| 7.6 | No. of pluviographs (> 0) | | 3.2 |
| 7.7 | Areally uniform rainfall flag: 0 if the rainfall has the same depth & temporal pattern everywhere on the catchment, 1 otherwise | | 3.2 |
| 7.8 | -99** ⇒ | If no sub-areas, ⇒ 9.1 | |
| 7.9 * | No. of time incs. from initial time to (i) start and (ii) finish of first rainfall burst ⇒ | If only one burst**, ⇒ 8.1 | 3.2 |
| 7.10 | No of time incs. from initial time to (i) start and (ii) finish of subsequent bursts** | | 3.2 |
| | **8. RAINFALL DATA** | | |
| 8.1 † | Name of 1st pluvi. (string < 29 characters) | | 3.2 |
| 8.2 * | Rainfalls, *mm* 1st burst, 1st pluvi. ⇒ | If only one burst, ⇒ 8.4 | 3.2 |
| 8.3 | Rainfalls, *mm*, for subsequent bursts, 1st pluvi. | | 3.2 |
| 8.4 | -99** ⇒ | If only one pluvi., ⇒ 8.6 | |
| 8.5 | Repeat Items 8.1 to 8.4 for other pluvis. | | 3.2 |
| 8.6 | ⇒ | If rainfall areally uniform, ⇒ 9.1 | |
| 8.7 * | Rainfalls, *mm*, on sub-areas A, B, ..., 1st burst | | 3.2 |

† Start a new line.
* Start a new line optionally preceded by any number of comment lines, each of which has a C in Column 1.
** This item may optionally be followed by a comma and a comment on the same line as the data item.

(Table 5.2 cont.)

| Item No. | Data Item(s) | Skips | Ref. Sect. |
|---|---|---|---|
| 8.8 | -99** ⇒ | If only one burst, ⇒ 8.10; if only one pluvi., ⇒ 9.1 | |
| 8.9 | Repeat Items 8.7 & 8.8 for subsequent bursts ⇒ | If only one pluvi., ⇒ 9.1 | |
| 8.10 * | Reference Nos. of pluvis,. to be used for temporal patterns on sub-areas A, B, .., 1st burst | | 3.2 |
| 8.11 | -99** ⇒ | If only one burst, ⇒ 9.1 | |
| 8.12 | Repeat Item 8.10 & 8.11 for subsequent bursts. | | |
| | **9. HYDROGRAPH DATA** | | |
| 9.1 | ⇒ | If no hydrographs, stop; data file complete. | |
| 9.2 * | Starting and finishing times of all hydrographs (required by Control Codes 7.1 or 9) in order, measured in time incs. from initial time | | 3.3.1 |
| 9.3 | -99** | | |
| 9.4 † | Name of gauging station. or hydrograph location (string < 29 characters), 1st hydrograph | | 3.3.1 |
| 9.5 * | Discharges, $m^3/s$, 1st hydrograph ⇒ | If only one burst, ⇒ 9.8 | 3.3.1 |
| 9.6 | - 99** | | |
| 9.7 * | Volumes of runoff (in any volume unit) or proportions of total runoff that occur in the 1st, 2nd, ... rises, 1st hydrograph | | 3.1.3 |
| 9.8 | - 99** ⇒ | If only one hydrograph, stop; data file complete | |
| 9.9 | Repeat Items 9.4 to 9.8 for other hydrographs, then stop; data file complete. | | |

† Start a new line.
* Start a new line optionally preceded by any number of comment lines, each of which has a C in Column 1.
** This item may optionally be followed by a comma and a comment on the same line as the data item.

# 6 GRAPHICAL EDITOR†

## 6.1 Introduction

The Graphical Editor (GE) incorporated into RORB enables users to create catchment files based on a graphical network of nodes and reaches. The intent of the GE is to reduce the level of effort required to create a catchment file, and to improve user's ability to understand, review, and modify catchment details. It enables users to visually understand the catchment layout, and input appropriate data; when this is done, the GE will automatically write the entire catchment file.

This chapter describes the GE screen tools and their use. It also contains a simple example which illustrates many of the features of the GE, with the aim of assisting users to 'get started'. In particular:

- **Section 6.2** describes the different types of graphical elements used to represent a RORB catchment.
- **Section 6.3** describes how to create a new stream network.
- **Section 6.4** discusses the automatic checks done when saving files.
- **Section 6.5** describes how to import an existing RORB catchment file.

> ***Caution!*** *– The GE works out an 'optimum' order for a catchment model network, which may vary from the order of the file read in. This is of consequence if there are corresponding storm files (.stm) containing hyetograph and hydrograph data in the order of the original catchment file. A warning is displayed if such a change is to be made, and whether the user wishes to proceed. If so, it is the responsibility of the user to edit the corresponding storm files to 'match' the catchment file. Refer to Section 6.10 for more details.*

- **Sections 6.6 to 6.10** outline the information which can be entered in each of the dialog boxes for each model element, and the various ways in which the graphical workspace can be manipulated, including labelling and colours.
- **Section 6.9** provides a worked example to illustrate the use of the Graphical Editor for a new file, including matching the input to a background figure.

## 6.2 Types of elements

In RORB GE, catchments are represented using combinations of four basin elements as shown in Table 6.1.

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† The RORB Graphical Editor was produced by Robert Morden and David Stephens (SKM). They have also contributed the first draft of all but Section 6.11 of this chapter.

**Table 6.1 Element types as used in the RORB graphical editor**

| **Element** | **Types** | **Related control codes** | **Refer Section** |
|---|---|---|---|
| Nodes | End of model<br>Junction<br>Sub-area centroid | 0<br>3, 4, 7**, 8<br>1, 2 | 6.6.1 |
| Reaches | Natural, unlined, lined, drowned<br>Translation<br>Dummy | 1, 2, 5<br>8<br>n/a | 6.6.2<br>6.6.2<br>6.9 |
| Storages | | 6 | 6.6.3 |
| Inflow/Outflow Reaches | | 9 | 6.6.4 |

** All code 7 print statements must occur at a node.

## 6.3 Creating a catchment network

When creating a catchment network (or model) for the first time, *the nodes must be drawn first*. Once some nodes have been created, the other elements can be added:

- Reaches connect two nodes together
- Storages connect two nodes together
- Inflow / outflow reaches link to a single node either at the downstream end (inflow), or the upstream end (outflow).

When the model is completed, the most downstream node must be defined as the model outlet. This is done by opening the node dialog, and changing the “Node Type” to “End of model”.

There can be only one model outlet node in a catchment file. Also, all reaches must eventually drain to the model outlet. If there are any parts of the catchment which are not directly connected to the outlet (“islands” or “floating catchments”), the resulting catchment file may not work properly with RORB. An automatic check for floating catchments is undertaken whenever the catchment file is saved, and the user is alerted to any potential connectivity problems.

Once drawn, model elements can be moved on screen as required. The vertices of any element can be dragged on screen to ensure the visual representation of the catchment is similar to the true catchment shape, making the model easier to understand.

## 6.4 Saving Catchment Files

When a model is saved, the file created has the extension **.catg**. This indicates that the file is a RORB catchment file created with RORB GE. This **.catg** file can be used directly in RORB in place of a **.cat** file.

When a model is saved, a range of automatic checks are undertaken to ensure that the model arrangement is valid, and that the final control codes have included all elements within the model. The following checks are performed:

**Check for model outlet** – there should be exactly one model outlet node. If not, an error message will appear. In this case, no control codes are written, although the spatial data required to display the model and all model parameters will still be saved correctly.

**Check of inflows and outflows** – for each I/O reach based on a previously identified I/O reach, the integer identifier must be unique to that I/O reach pair. If not, an error message will appear. Also, for an I/O outflow defined so that the effluent stream is modelled first, then there must be a matching inflow. If not, an error message will appear. In both of these cases, no control codes are written, although the spatial data required to display the model and all model parameters will still be saved correctly.

**Check of connectivity** – every node in the model is tested to ensure that it will drain downstream to the model outlet. If any nodes fail this criterion, an error message will appear listing those nodes which do not connect properly. In this case, no control codes are written, although the spatial data required to display the model and all model parameters will still be saved correctly.

**Check of rainfall nodes** – any node which has an associated rainfall input must be followed by a valid reach. If a rainfall sub-area is followed by a dummy reach, an error message will appear. In this case, no control codes are written, although the spatial data required to display the model and all model parameters will still be saved correctly.

**Check of control codes** – as control codes are written, each element in the model is marked as having been used. If any elements remain unused at the end of the control vector, an error message appears. In this case, control codes are written as much as possible, and a listing of all unused model elements is written at the end of the file (after the impervious fractions).

## 6.5 Importing Existing Catchment Files

'Old' catchment files (ie type **.cat**) can be imported into the GE. When opening such a file, the user selects a .cat file (rather than the default **.catg** file type). If the selected file is a valid RORB catchment file, the program will read the control codes and interpret all data, including comments. The file will then open with all model elements displayed spatially.

The arrangement of an imported model will not usually match the true catchment shape, and many elements may need to be moved to better spatially represent the sub-areas and reaches.

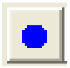
When an imported file is saved, the user will be prompted for a new file name with a **.catg** extension. Note the possibility that the new file will have a different ordering, and the consequences of that (see box in Section 6.1).

Some existing catchment files contain storm or hydrograph data following the catchment data. As this data cannot be saved using the GE, the user will be prompted to save this information in a separate file. If the data is not saved to another file it will be lost when the catchment file is saved as a **.catg** format.

## 6.6 Adding and Editing Model Elements

This section discusses the RORB GE graphical elements and how model parameters can be entered into the program to create a RORB catchment file. Where possible, the various parameter fields in the dialog boxes have been related back to their corresponding items in Table 5-2.

### 6.6.1 Nodes

To add a node, use the  button, or select *Add*, *Node*. The node can then be placed anywhere in the workspace with a single mouse click. As soon as the user clicks on the workspace, a dialog box will open with prompts to select the node parameters. The dialog is shown in Figure 6-1. Clicking on the *OK* button will add the node to the database and draw the appropriate symbol on the workspace in the selected location.

**Node Number -** Database number of the node. It cannot be changed by the user.

**Node Size -** Real number ranging from 0.001 to 10.000 indicating the size of the node as it appears in the workspace.

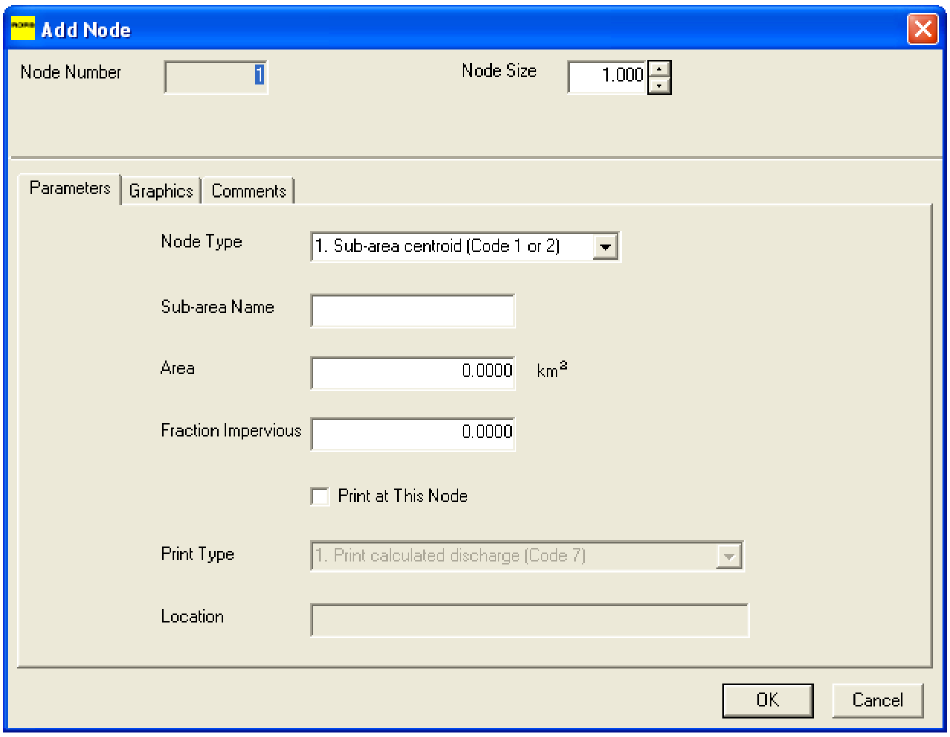



**Figure 6-1 Node Dialog Box**

**Parameters Tab –** has information on the parameters used to model the node within the RORB model simulation:

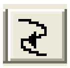
**Node Type** – Drop down menu to select the type of node. If the node is a rainfall input (i.e. Code 1 or Code 2), select Option 1. If the node is the end of the model (i.e. Code 0), select Option 3. All other nodes (i.e., junctions, print commands, etc) should be entered using Option 2.
**Sub-area Name -** Text string indicating the name of the sub-area. This field is not used by the RORB model simulation. This field is only active if the node type is set to 1.
**Area** - Real number greater than or equal to 0.000 indicating the area of the sub-area in $km^2$. This field is only active if the node type is set to 1.
**Fraction Impervious -** Real number ranging from 0.000 to 1.000 indicating the fraction impervious of the sub-area in %. This field is only active if the node type is set to 1.
**Print at This Node -** Checkbox indicating that a print command takes place at this node (Code 7).
**Print Type** – Drop down box indicating the type of print command (e.g. Code 7, Code 7.1 or Code 7.2) to be undertaken at this node. This menu is only active if the print at this node checkbox is checked.
**Location** – A text string less than 29 characters long indicating the location name of a Code 7 or Code 7.2 print command. This field is only active if the print at this node checkbox is checked and the print type is set to 1 or 3.

**Graphics Tab -** Information on the graphical representation of the node.

**X-Coordinate and Y-Coordinate -** The X-Y co-ordinates of the node. These can be changed using the *Move* function (refer Section 6.7).
**Arrow Location -** Drop down menu to select the location of the arrow indicating a sub-area centroid node. The default position is top right.
**Print Marker** - Drop down menu to select the location of the "P" marker indicating a print command at this node. The default position is top left.

**Comments Tab -** Comments on the node. A maximum of 50 comments is allowed per node. Each comment is a text string up to 256 characters long.

### 6.6.2 Reaches

Once two nodes have been added to the workspace, they may be connected with a reach. To add a reach, click the  button or select *Add, Reach*. The user is then prompted to select the upstream node of the reach by clicking once on any node in the workspace. The reach may then be drawn anywhere within the workspace by clicking on a blank point. A reach may have up to 200 mid points. When the reach has been drawn, select a downstream node by clicking on it once. Press *Escape* at any time to cancel drawing the reach. As soon as the user clicks on the downstream node, a dialog box will open with prompts to select the reach parameters. The dialog is shown in Figure 6-2. Clicking on the *OK* button will add the reach to the database and draw the appropriate line on the workspace.

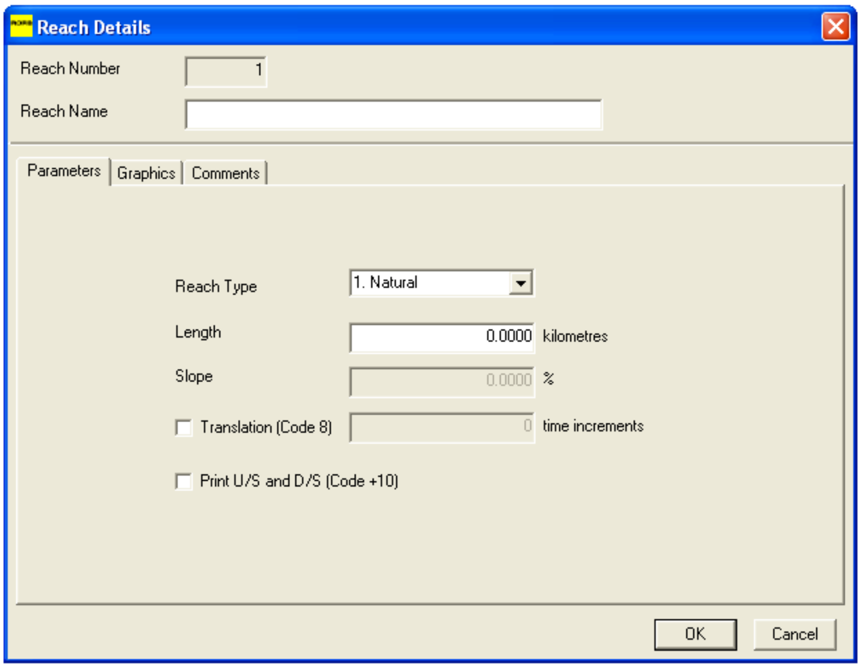



**Figure 6-2 Reach Dialog Box**

**Reach Number -** Database number of the reach. It cannot be changed by the user.

**Reach Name -** Text string less than 29 characters long. This field is not used by the RORB model simulation.

**Parameters Tab -** Information on the parameters used to model the reach within the RORB model simulation (Item numbers refer to usage in first column of Table 5-2):

**Reach Type -** Drop down box to select the reach type code (Item 1.2).
**Length -** Real number greater than or equal to 0.000 indicating the length of the reach in km (Item 1.3).
**Slope -** Real number greater than or equal to 0.000 indicating the slope of the reach in % (Item 1.4). This field is only activated if the reach type code is set to 2 or 3.
**Translation** - Checkbox indicating that the reach is a translation. The number of increments to translate the hydrograph is entered in the field to the right (Item 4.1). Checking this box de-activates the routing fields and activates the translation field.
**Print U/S and D/S -** Checkbox indicating whether hydrographs are outputted at the upstream and downstream ends of the reach. Checking this box is equivalent to adding 10 to the reach control code.

**Graphics Tab -** Information on the graphical representation of the reach.

**Upstream/Downstream Node -** The upstream/downstream node of the reach. It can be changed by clicking on *Change*. Hitting *Escape* at any time cancels the change.
**Number of Co-Ordinates -** Integer ranging from 1 to 200 to indicate the number of mid points in the reach. Individual mid points can be changed using the *Move* command (refer Section 6.7). Alternatively, the reach can be redrawn by clicking on *Redraw*. This allows the user to completely redefine the mid points and downstream node of the reach without losing the reach parameters. Hitting *Escape* at any time cancels the redraw.

**Comments Tab -** Comments on the reach. A maximum of 50 comments is allowed per reach. Each comment is a text string up to 256 characters long.

### 6.6.3 Storages

Once two nodes have been added the workspace, they may be connected via a "special storage". Storages are the graphical equivalent of a Code 6. To add a storage, click the  button or select *Add*, *Storage*. The user can then click the mouse on any location in the workspace – this represents the location of the storage. The user must then join the storage to an upstream node by clicking once on any node in the workspace. Finally, the user must join the storage to a downstream node clicking once on any other node in the workspace. A storage may not have the same upstream and downstream nodes. Press *Escape* at any time to cancel drawing the storage. As soon as the user clicks on the downstream node, a dialog box will open with prompts to select the storage parameters. The dialog is shown in Figure 6-3. Clicking on the OK button will add the storage to the database and draw the appropriate symbol and lines.

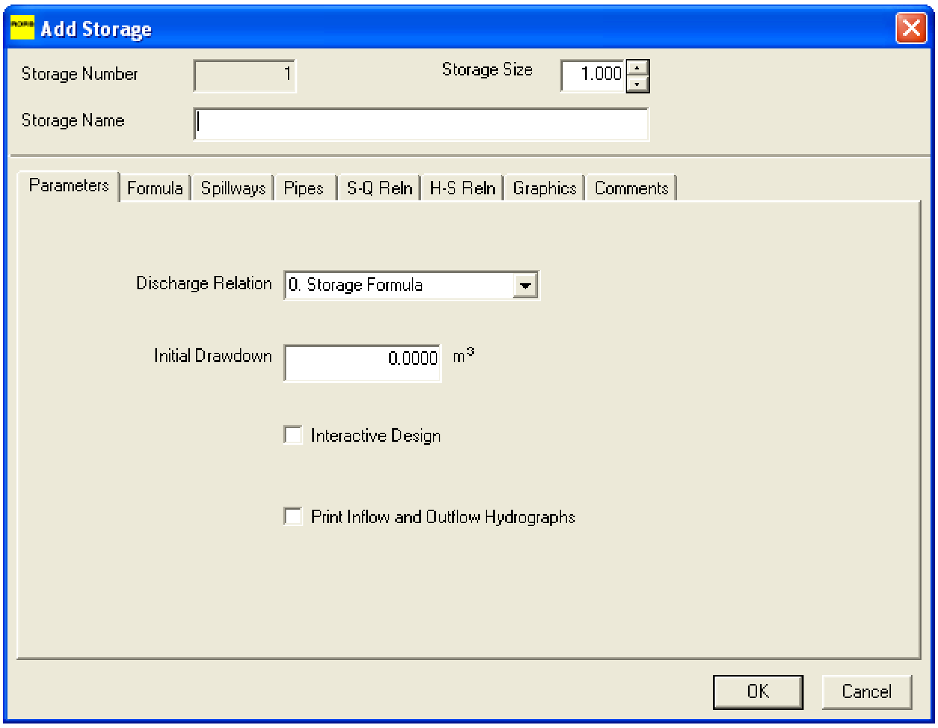



**Figure 6-3 Storage Dialog Box**

**Storage Number -** Database number of the storage. It cannot be changed by the user.

**Storage Size -** Real number ranging from 0.001 to 10.000 indicating the size of the storage as it appears in the workspace.

**Storage Name -** Text string less than 29 characters long (Item 2.1, Table 5-2).

**Parameters Tab -** Information on relationship used to calculate outflow from the storage and the initial drawdown. (Item numbers refer to usage in first column of Table 5-2):

**Discharge Relation -** Drop down box to select the discharge relation flag (Item 2.2).
**Initial Drawdown -** Real number to indicate the initial drawdown (Item 2.3).
**Interactive Design** – Checkbox to indicate whether the storage is to be modelled using interactive design. This is equivalent to a Code 6.1.
**Print Inflow and Outflow Hydrographs** – Checkbox to indicate whether inflow and outflow hydrographs should be printed (e.g. Code 16).

**Formula Tab** - information on the parameters of the storage equation:

**Coefficient ($k_s$) and Coefficient ($m_s$) -** Real numbers to indicate the parameter $k_s$ and the parameter $m_s$ (Item 2.13, Table 5-2). These fields are only activated if the discharge relation flag is set to 0.

**Spillways Tab -** Information on spillways. (Item numbers refer to usage in first column of Table 5-2):

**Number of Separate Spillways -** Integer ranging from 0 to 100 to indicate the number of separate spillways (Item 2.4). This field is only activated if the discharge relation flag is set to 2 or 3.
**Weir Coefficient ($K_w$) -** Real number greater than or equal to 0.000 to indicate the weir coefficient (Item 2.7). This field is only activated if the discharge relation flag is set to 2 or 3.
**Crest Elevation (m)** and **Effective Length (m) -** The spillway parameters are entered in this table (Item 2.5 and 2.6). The values are real numbers greater than or equal to 0.000. This field is only activated if the discharge relation flag is set to 2 or 3. The number of rows in the table is automatically changed when the number of separate spillways is changed. A maximum of 100 rows is allowed, and the data can be directly copied to or from an Excel spreadsheet. RORB GE will automatically sort the data into ascending order based on the crest elevation values.

**Pipes Tab -** Information on piped outlets:

**Entrance Loss Coefficient ($k_e$) -** Real number greater than or equal to 0.000 to indicate the entrance loss coefficient (Item 2.8). This field is only activated if the discharge relation flag is set to 2.
**Bend Loss Coefficient ($k_b$) -** Real number greater than or equal to 0.000 to indicate the bend loss coefficient (Item 2.9). This field is only activated if the discharge relation flag is set to 2.
**Number of Separate Pipe Outlets -** Integer ranging from 0 to 100 to indicate the number of separate pipe outlets (Item 2.10). This field is only activated if the discharge relation flag is set to 2.
**Length (m)**, **Slope (%)**, **Entrance Invert (m)**, **Number of Pipes** and **Diameter (m) -** The pipe outlet parameters are entered in this table (Item 2.11 and 2.12). The values are real numbers greater than or equal to 0.000, expect for the number of pipe which is an integer greater than or equal to 0. This field is only activated if the discharge relation flag is set to 2. The number of rows in the table is automatically changed when the number of separate pipe outlets is changed. A maximum of 100 rows is allowed, and the data can be directly copied to or from an Excel spreadsheet. RORB GE will automatically sort the data into ascending order based on the entrance invert elevation values.

**S-Q Relation Tab -** Information on the storage-discharge relationship:

> **Number of S-Q Pairs -** Integer ranging from 0 to 200 to indicate the number of pairs in the S-Q table (Item 2.14, Table 5-2). This field is only activated if the discharge relation flag is set to 1.
> **S ($m^3$) and Q ($m^3/s$) -** The values of the S-Q relationship are entered in this table (Item 2.15). The values are real numbers greater than or equal to 0.000. This field is only activated if the discharge relation flag is set to 1. The number of rows in the table is automatically changed when the number of S-Q pairs is changed. A maximum of 200 rows is allowed, and the data can be directly copied to or from an Excel spreadsheet. RORB GE will automatically sort the data into ascending order based on the S values.

**H-S Relation Tab -** Information on the elevation-storage relationship:

> **Elevation-Storage Relation -** Drop down box to select the elevation-storage relation flag (Item 2.17, Table 5-2). Note that if the discharge relation flag is set to 2 or 3, the elevation-storage flag cannot be 0.
> **Number of H-S Pairs -** Integer ranging from 0 to 200 to indicate the number of pairs in the H-S table (Item 2.18, Table 5-2). This field is only activated if the elevation-storage relation flag is set to 1.
> **H (m)** and **S ($m^3/s$) -** The values of the H-S relationship are entered in this table (Items 2.19 and 2.20, Table 5-2). The values are real numbers greater than or equal to 0.000. This field is only activated if the elevation-storage relation flag is set to 1. The number of rows in the table is automatically changed when the number of H-S pairs is changed. A maximum of 200 rows is allowed, and the data can be directly copied to or from an Excel spreadsheet. RORB GE sorts the data into ascending order based on the H values.
> **Parameters For Formula** $S = a(H - H_0)b$ - These three fields are real numbers with any range (Item 2.21). They are only activated if the elevation-storage relation flag is set to 2.

**Graphics Tab** - Information on the graphical representation of the storage:

> **Upstream/Downstream Node -** The upstream/downstream node of the storage. It can be changed by clicking on *Change*. Hitting *Escape* at any time cancels the change.
> **X-Coordinate and Y-Coordinate -** The X-Y co-ordinate of the storage. These can be changed using the *Move* function (refer Section 6.7).

**Comments Tab -** Comments on the storage. Comments are divided into three groups depending on their position in the vector file. A maximum of 50 comments is allowed in each group. Each comment is a text string up to 256 characters long.

### 6.6.4 Inflows and Outflows (I/O)

Once a node has been added the workspace, a channel inflow or outflow may be added to it. Inflow/outflows are the graphical equivalent of a Code 9. To add an inflow or outflow, click the  button or select *Add, Inflow/Outflow*. The graphical representation of inflows and outflows is an arrow – to add an inflow to a node, the user can click on any blank point in the workspace, followed by any other blank point, followed by the node where the inflow is to take place. To add an outflow, the user must first click on any node in the workspace, followed by two other blank points in the workspace. All I/O arrows have three points. Press *Escape* at any time to cancel drawing the I/O. As soon as the I/O arrow has been drawn, a dialog box will open with prompts to select the I/O parameters (Figure 6-4). Clicking on the OK button will add the I/O to the database and draw the appropriate symbol on the workspace.

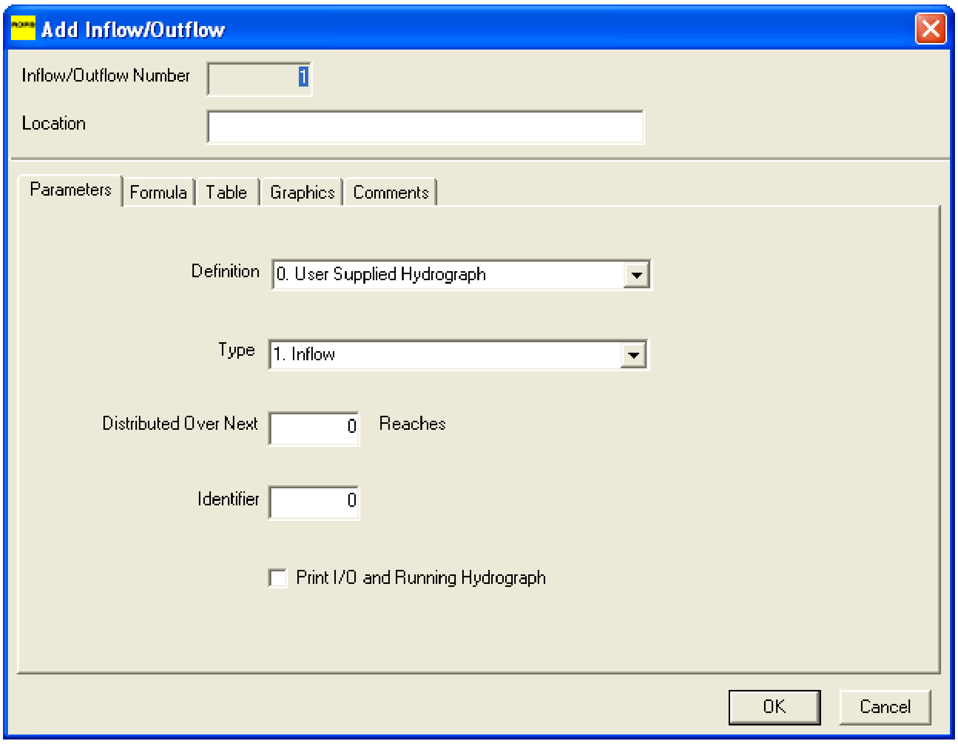



**Figure 6-4 Inflow/Outflow Dialog Box**

**Inflow/Outflow Number -** Database number of the I/O. It cannot be changed by the user.

**Location -** Text string less than 29 characters long (Item 5.5, Table 5-2).

**Parameters Tab -** Information on the parameters used to model the I/O within the RORB model simulation:

**Definition -** Drop down box to select I/O definition flag (Item 5.1, Table 5-2).
**Type -** Drop down box to select I/O type flag (Item 5.3).
**Distributed Over Next N Reaches -** Integer greater than or equal to -1 to indicate the number of reaches the I/O is distributed over (Item 5.2). *Note that RORB GE automatically assigns the calculation order when it creates the catchment file – as a result the effect any I/O distributed over all remaining reaches may be different from what is intended.*
**Identifier -** Integer greater than or equal to 0 to indicate the inflow/outflow identifier (Item 5.4).

**Formula Tab** – Information on the discharge formula.

**Parameters For Formula** $D = a + c(Q - b)^c$ **-** These four fields are real numbers with any range (Item 5.6, Table 5-2). They are only activated if the type flag is set to 1.

**Table Tab** – Information on the Q-D relationship:

**Number of Q-D Pairs -** Integer ranging from 0 to 200 to indicate the number of pairs in the Q-D table (Item 5.7). This field is only activated if the definition flag is set to 3.
**Q ($m^3/s$)** and **D ($m^3/s$) -** The values of the Q-D relationship are entered in this table (Items 5.8 and 5.9). The values are real numbers greater than or equal to 0.000. This field is only activated if the definition flag is set to 3. The number of rows in the table is automatically changed when the number of Q-D pairs is changed. A maximum of 200 rows is allowed, and the data can be directly copied to or from an Excel spreadsheet. RORB GE will automatically sort the data into ascending order based on the Q values.

**Graphics Tab -** Information on the graphical representation of the inflow/outflow.

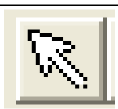
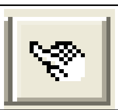
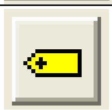
**Upstream Node -** The upstream node of the inflow/outflow. If the object is an inflow, this will be 0, otherwise it will be a non-zero integer. It can be changed by clicking on *Change*. This allows the user to select a new upstream node in the workspace. Hitting *Escape* at any time cancels the change.
**Downstream Node -** As for upstream node, however this field will be zero if the object is an outflow.
**Mid Point X Co-Ordinate and Mid Point Y Co-Ordinate -** The X-Y co-ordinate of the inflow/outflow mid point. This can be changed using the *Move* function (refer Section 6.7).

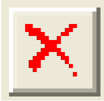
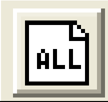
**Comments Tab** - Comments on the inflow/outflow. Comments are divided into two groups depending on their position in the vector file. A maximum of 50 comments is allowed in each group. Each comment is a text string up to 256 characters long.

## 6.7 Manipulating the Workspace

RORB GE includes various tools and functions that can be used to manipulate the workspace objects. These tools are interactive and simple to use, and are described in Table 6.2.

**Table 6-2:RORB GE Tools**

| Tool | Button | Command | Description |
|---|---|---|---|
| Select |  | *Edit, Select* | Selects any object in the workspace with one click; edits the object's properties with a double click. |
| Move |  | *Edit, Move* | Moves any object or object vertex in the workspace by a single click and drag. |
| Model Name |  | *Edit, Model Name* | Edits the name of the model and any general comments. This field must be filled in before the model can be saved. |
| Find | | *Edit, Find* | Finds a node or reach in the workspace with a particular name or attribute. |

| | | | |
|---|---|---|---|
| Delete |  | *Edit, Delete* | Deletes any object in the workspace with a single click. Alternatively, the object can be selected and then deleted with the delete key. Note that when any node with attached reaches, storages or inflow/outflows is deleted, all the attached objects will also be deleted. |
| Browse File |  | *Edit, Browse File* | A text editor to browse and edit any text file. |
| Data Tables | | *View, Data Tables* | Allows tables of attributes such as areas, fractions impervious and reach length to be viewed and edited globally. |
| Rescale | | *View, Rescale* | Rescales the graphical extent of the file to fit within the workspace. |
| Shrink Vector | | *View, Shrink Vector* | Shrinks the graphical extent of the file. |
| Enlarge Vector | | *View, Enlarge Vector* | Enlarges the graphical extent of the file. |
| Rotate | | *View, Rotate* | Rotates the graphical position of all objects in the workspace by the specified angle. |

## 6.8 Customising the Workspace

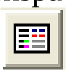
RORB GE has a number of customisation features that can be used to change the look and feel of the workspace. These include viewing background images and changing the sizes, colours and labelling of the objects.

### 6.8.1 Background Image

RORB GE has the ability to display an image file as the background to the workspace. Only Windows Metafiles **(*.wmf)** can currently be displayed in this way. To access this feature, select the menu option *View, Manage Background*.

The user is prompted to enter the file name of a ***.wmf** graphics file, and can select whether to display the image or not, and whether to save the name of the image with the catchment file or not. Once an image has been loaded, it can be turned on and off simply by checking or unchecking the show image checkbox.

### 6.8.2 Colours and Sizes

The colours and sizes of objects in the workspace can be customised by each user. This facility is accessed through the toolbar button  or the menu option *View, Customise*. This command opens the customise window, as shown in Figure 6-5.

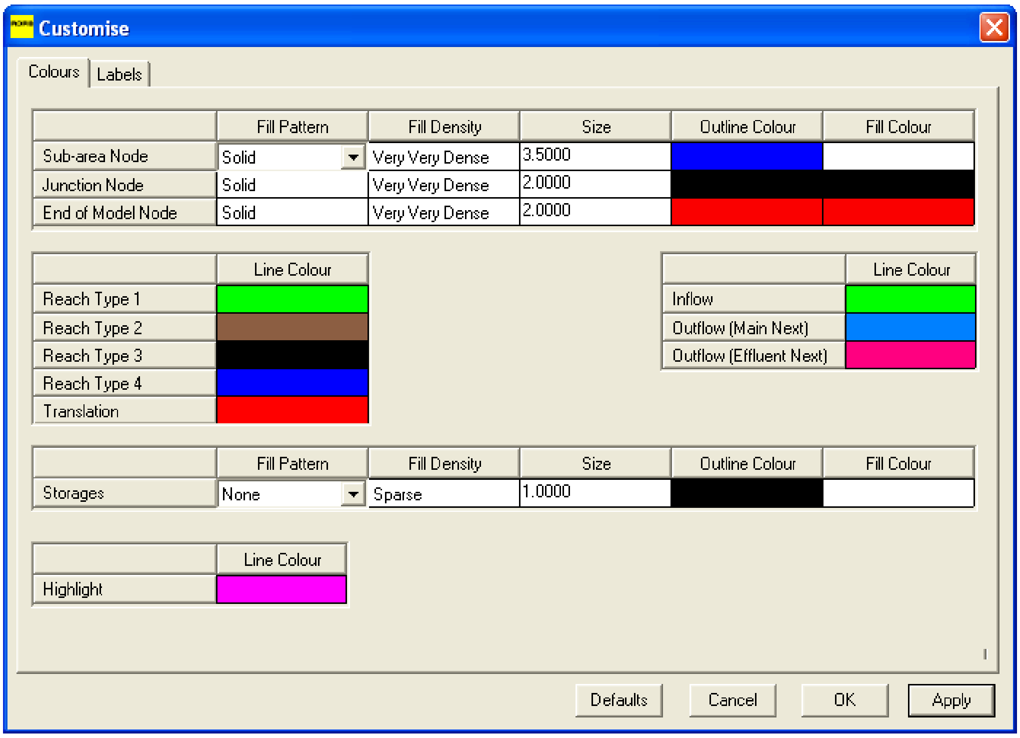



**Figure 6-5 Customise Colours Dialog Box**

Colours of the different types of nodes, reaches, inflow/outflows and highlighted objects can be changed simply by clicking on the colour the user wishes to change. This opens a new dialog box where a different colour can be chosen.

It is also possible to change the fill pattern and fill density of node and storage objects. This is done using the relevant dropdown menu. The global size of node and storage objects can also be changed. This global size provides a factor by which the sizes of the individual nodes and storages are multiplied.

Once the user has selected a new set of colours and sizes, these parameters will be saved for them every time RORB GE is opened. The default parameters supplied with the program can be reset at any time by clicking the “Default” button.

### 6.8.3 Labelling

RORB GE allows objects in the workspace to be labelled with a variety of different information and parameters. The labels can be easily turned on and off selectively without losing the customised format.

Customising the label format is accessed through the customise labels dialog box, which can be opened using the toolbar button  or the menu option *View, Customise*. The customise labels dialog box is shown in Figure 6-6.

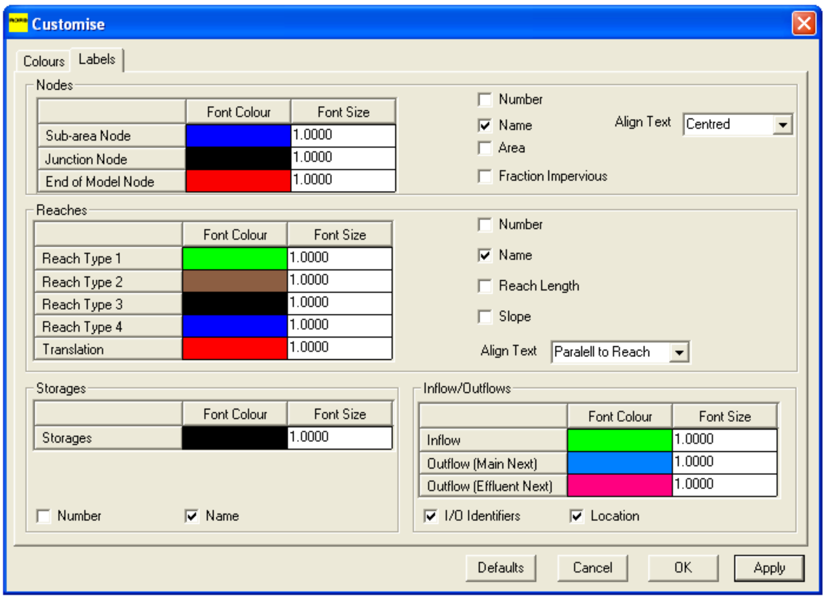


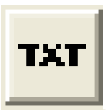

**Figure 6-6 Customise Labels Dialog Box**

The user has the option to change the text colour and font of the labels of each different object type. As with the actual object colours, the colour is changed simply by clicking once on any colour in the dialog box.

The label that will be printed for each object is also selected by the user in this dialog box. For example, Figure 6-6 shows that there are four parameters that can be displayed as a node label. These are the node database number, the node name, the area of the sub-area attached to the node, and the fraction impervious of the sub-area attached to the node. The last three options will only display labels for nodes which are sub-area centroids. Any combination of these four parameters can be selected. Additionally, the user can also select the alignment of the text using the dropdown box.

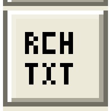
As with the customise colours dialog box, the user's preferred settings are automatically saved and will be re-loaded each time RORB GE is opened. The default settings can be selected at any time by clicking the "Default" button.

To display the text labels in the workspace, RORB GE has three toggle buttons that turn text on and off. These are shown below.




Text toggle button. This button turns all text labels in the workspace on and off.


Node text toggle button. This button turns node text labels on and off.




Reach text toggle button. This button turns reach text labels on and off.

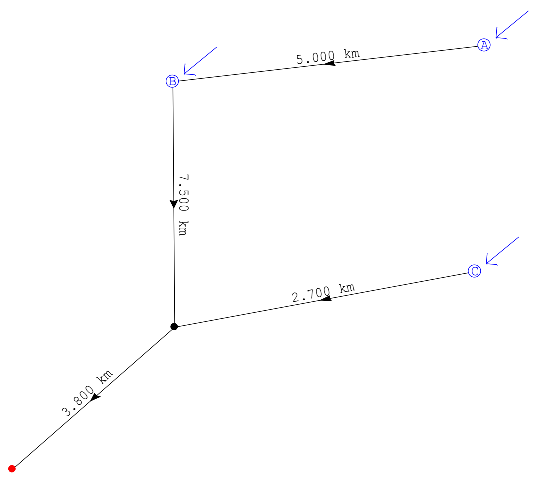
## 6.9 Understanding Dummy Reaches

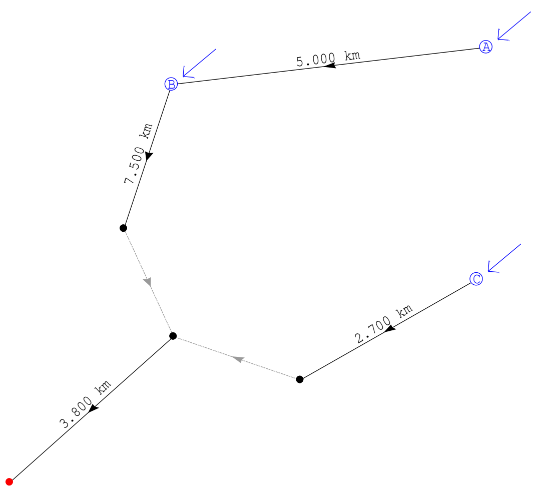
A dummy reach is drawn the same as a normal reach, except the "Type" property is set to "Dummy".

Dummy reaches have been introduced in RORB GE as a way of ensuring that the control codes in the final catchment file are written in a specific order. For example, consider the following control codes:

```
1, 3, 5.0, -99              , Sub area A
2, 3, 7.5, -99              , Sub area B
7                           , Print
Tributary A
3                           , Save hydrograph
1, 3, 2.7, -99              , Sub area C
7                           , Print
Tributary B
4                           , Add to previously saved hydrograph
7                           , Print
Tribs A and B combined
5, 3, 3.8, -99              , Route
```

This catchment is shown below, as it would appear in RORB GE if it were drawn both with and without dummy reaches.




The representation without dummy reaches makes it very difficult to specify the individual print statements (Code 7) for Tributary A and Tributary B. The print statements must appear at the node at the downstream end of each reach, but in this case all three print statements would appear on the same node. Therefore, the print statements will not produce the expected results.

However, when dummy reaches are added, the print statements can be placed correctly. The dummy reach forces the print statement to occur before the junction, which is the original intent.

Dummy reaches can be used in the same way to customise print statements and inflow/outflow reach locations. In addition, a dummy reach can force a dummy gauging station (refer Section 4.2) to appear in the correct location.

There is no control code associated with a dummy reach. They have no data such as length or slope, they simply ensure that the elements around them are written within the catchment file in the correct sequence.

## 6.10 How the program writes control codes

When preparing a catchment network, it can be useful to understand how the program writes control codes, and in what order the codes will be written. [This material is included for reference purposes, and not necessary for uncomplicated RORB applications.]

The most important issue is the overall order of output for the catchment file as a whole. Each time a file is saved, the program automatically assesses the network, and determines the best order to write control codes. The automatic nature of the program means that the whole catchment file could be re-written in a quite different order if only a few changes are introduced to the network. [A message is displayed to give users the option of accepting this, or rejecting the changed file. See text box in Sect. 1.1].

The program will always start calculations from a branch end, and will select the most appropriate branch end based on the following three rules (in order of preference):

- Select the branch end with the highest priority based on Code 9 (I/O) requirements (for matching pairs of inflows and outflows, the upstream outflow must be written first);
- If inflow/outflow priorities are equal, select the branch end with the most number of downstream reaches to the outlet;
- If the number of downstream reaches is equal, select the branch end with the lowest node number.

At each node in the network, there is a standard sequence of checks to be performed to ensure that all possible network features are dealt with in the correct order. At each node, the sequence of checks and calculations is as follows:

*Check if current node is the same as the last node saved in the stack (ie. a Code 3 stored hydrograph)*

If yes, then write a Code 4, clear the top entry in the stack, then go to step 1).

*Check if there are any unused reaches or storages flowing into the current node*

If yes, then write a Code 3, add the current node to the stack, move to the next branch, then return to step 1).

*Check if there are any unused inflow/outflow reaches flowing* ***into*** *the current node*

If yes, then write a Code 9 to retrieve the inflow hydrograph, then go to step 1).

*Check if the current node is actually retrieving the main stream from where an inflow/outflow reach was previously written and the effluent stream was modelled first (Code 9, Type 1).*

If yes, then write a Code 9 to retrieve the main stream hydrograph, then go to step 1).

*Check for print statements at the current node. Write a Code 7 if required.*

*Check if there are any unused inflow/outflow reaches flowing* ***out*** *of the current node.*

If yes, then write a Code 9 (if multiple Code 9's, select the one with the lowest I/O number) to create the outflow hydrograph.

If the Code 9 is of type -1 (effluent stream modelled next), then move to the effluent stream and go to step 1).

*Check if there are any storages immediately downstream of the current node*

If yes, then write a Code 6, move downstream to the next node, then go to step 1).

*Check if there are any reaches immediately downstream of the current node*

If yes, and there is currently no running hydrograph, write a Code 1, move downstream to the next node, then go to step 1).

If yes, and there is currently a running hydrograph, write a Code 2, move downstream to the next node, then go to step 1).

If yes, and there is currently a running hydrograph, and there is no sub-area data, write a Code 5, move downstream to the next node, then go to step 1).

If yes, and it is a dummy reach, then move downstream without writing any data, then go to step 1).

If yes, and the number of translation time steps is not 0, write a Code 8, move downstream to the next node, then go to step 1).

*If the current node is the model outlet, then write a Code 0 including all sub-area and impervious area information.*

The order of these operations is important, for example if there is an outflow reach and a print statement located at a node, the print statement will be written first. If this is not the desired result, then a dummy reach should be added downstream, and the print statement placed at the downstream node.

Similarly, if there are multiple inflow reaches at a single node, they will be processed in order of their “I/O number” (not their “identifier”). If this order is inappropriate, then they must be separated using dummy reaches to force the correct order.

Care must also be taken when using outflow Code 9s that both the effluent (outflow) stream and the main stream are connected to the model outlet, where applicable. A model could be setup such that an outflow (with the effluent stream to be modelled next) is applied to a particular node, and there is no further interest in the portion of the hydrograph remaining at that node. Regardless of this, the node still needs to be connected to the outlet of the model, most usefully with a dummy reach.

## 6.11 An introductory example

The Graphical Editor (GE) is part of the RORB software, with a Windows ‘look and feel’ and written to be self explanatory for the user. Context-sensitive help is included for each window screen. It is expected that, once the user gets a start with a simple example, progress to more complicated cases will readily follow.

The application presented here is for the hypothetical catchment shown in Figure A-3, reproduced below with sub-area areas and reach lengths added in.

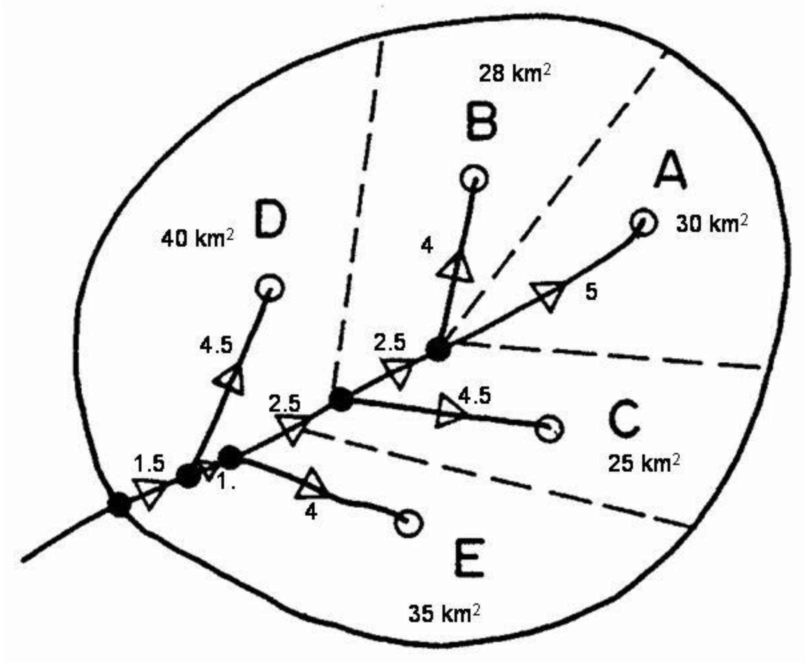



**Figure 6.7 Hypothetical rural catchment, showing sub-area areas. Reach lengths (in km) are shown next to the respective reach storages**.

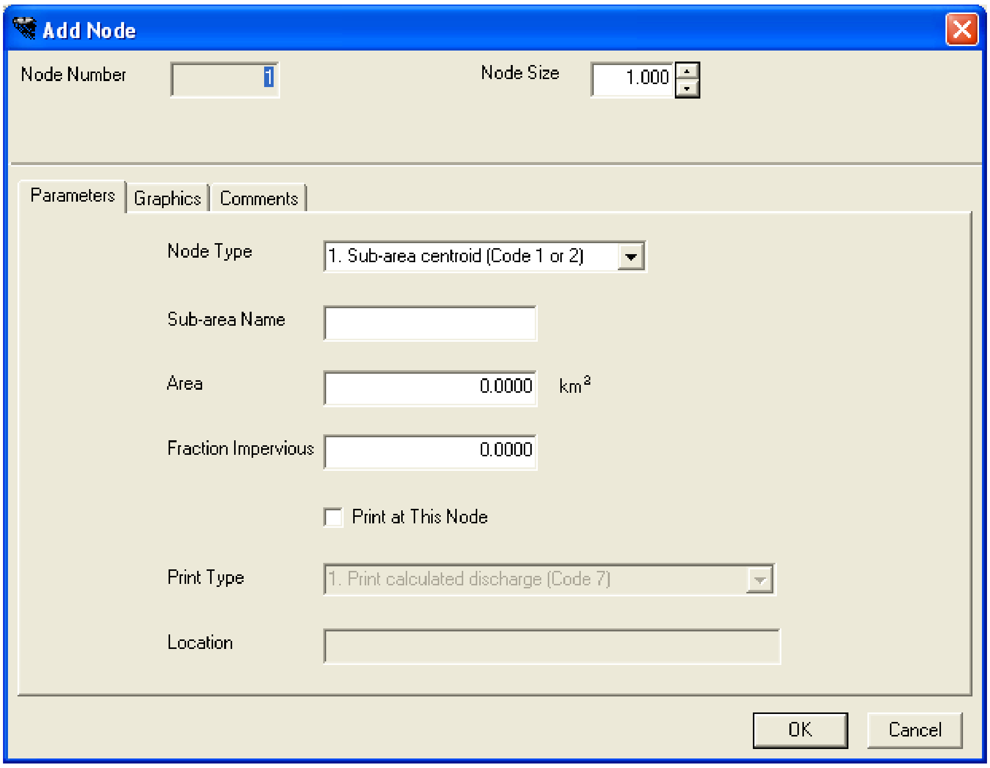
The GE is used to create a RORB catchment file (**.catg**) for this catchment, as explained in the following steps.

1. Click on the RORBWin icon to start the RORB program. Wait for the blue centre logo to disappear (or left mouse-click somewhere outside it if you are in a hurry).

2. Click on the GE button, or click ‘Edit’ and ‘Catchment File (GUI)’, to start the GE.

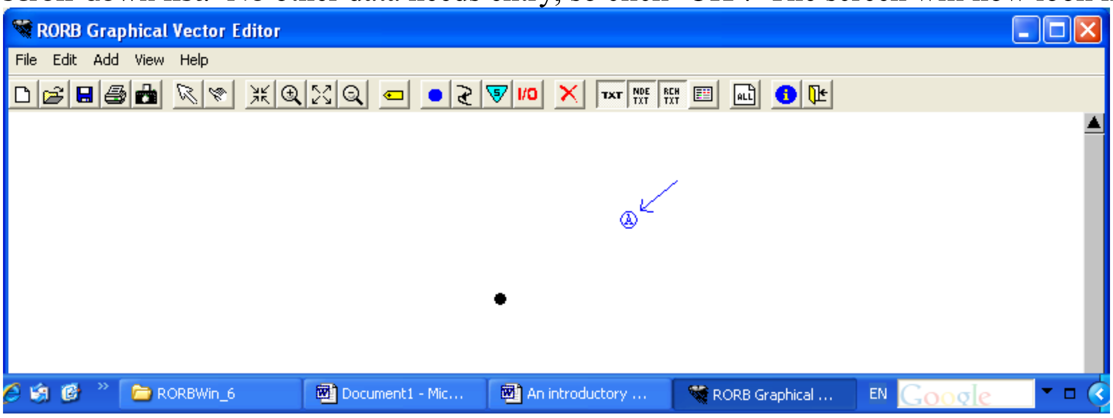
3. Click ‘File’ then ‘New’ to create a new datafile. [If an existing whole or part RORB datafile exited for this catchment, you would click ‘Open’ rather than ‘New’]. All of the toolbar buttons are now activated. Move the cursor slowly along the row so as to display the purpose of each button.

4. Click on the ‘add a node’ button (near the middle) showing the blue dot. Move the cursor to the upper right quarter of the screen to a point where the input node for sub-area A will be placed, then left click to locate it. An entry screen appears for the user to enter appropriate data for sub-area A, as follows

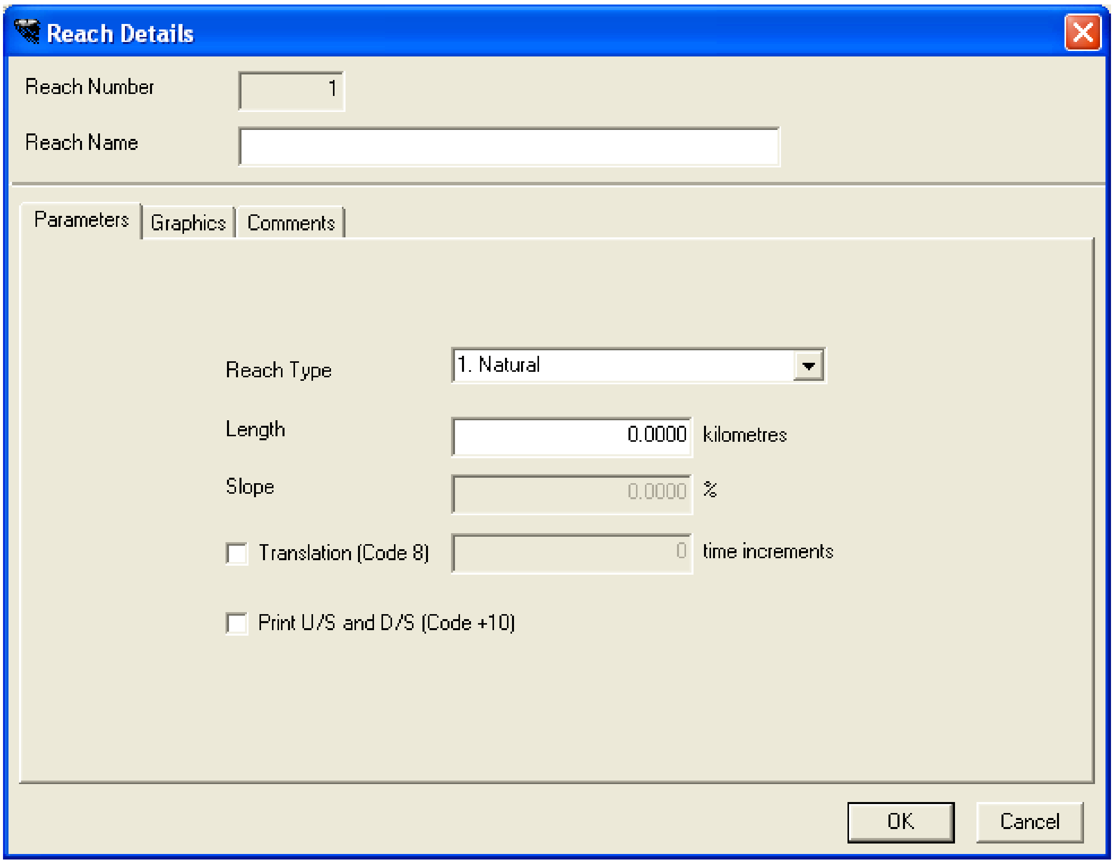



This is a sub-area centroid (Code 1) so the default Node Type is correct. Enter the sub-area name ('A') in the box below, and its area (30 km$^2$) in the box below that. We are dealing with a rural catchment so the default fraction impervious (0%) is correct. Click 'OK', and a circle labelled 'A' and with an 'input' arrow showing it as a location of rainfall-excess input appears.

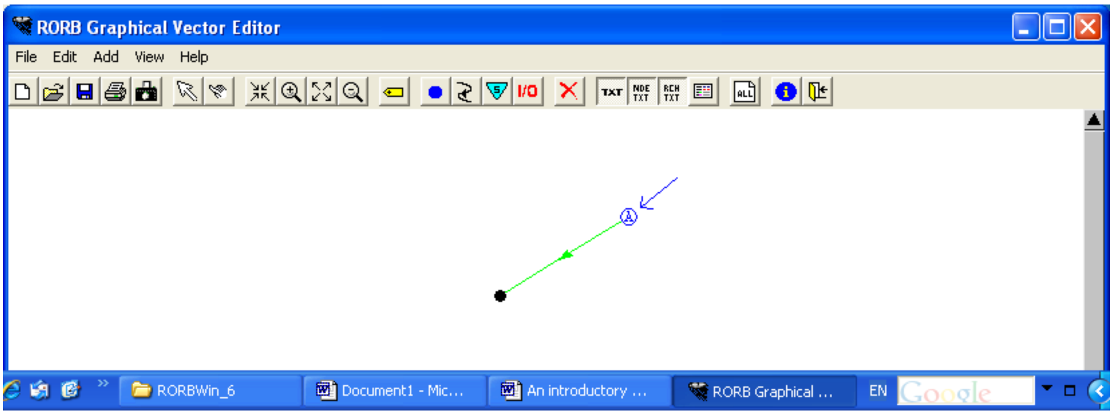
5. Since reaches join nodes, it is necessary to place the next node on the screen before the reach can be drawn. Hence, repeat the process to place a node corresponding to the junction of the tributaries from sub-areas A&B. Flows from the tributaries will be added at this point, hence select Node Type 'Junction (Code 3 .. Code 4)' from the options in the scroll-down list. No other data needs entry, so click 'OK'. The screen will now look like:




6. We can now join these two nodes by a reach. To the immediate right of the 'Add a node' button, is the 'Add a reach' button. Click on that, and move the cursor onto the page; note the cursor symbol. Click on the node labelled 'A', and drag the cursor to the other node to draw the reach. Click when on the second node to complete the depiction, and an input screen appears.



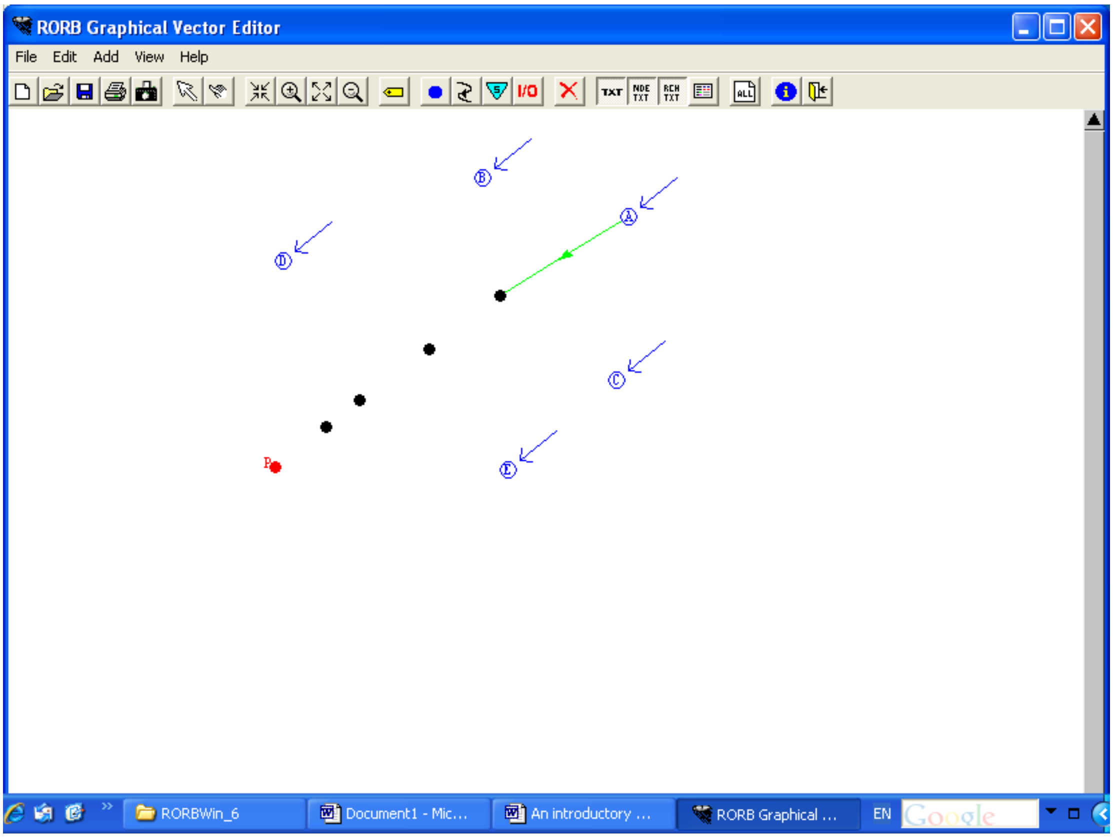

The reach name is optional (but could be used to label a stream), but the reach length is necessary (5 km). [We assume the reach type is 'natural' for a rural catchment.] Press 'OK'. Your screen will show:



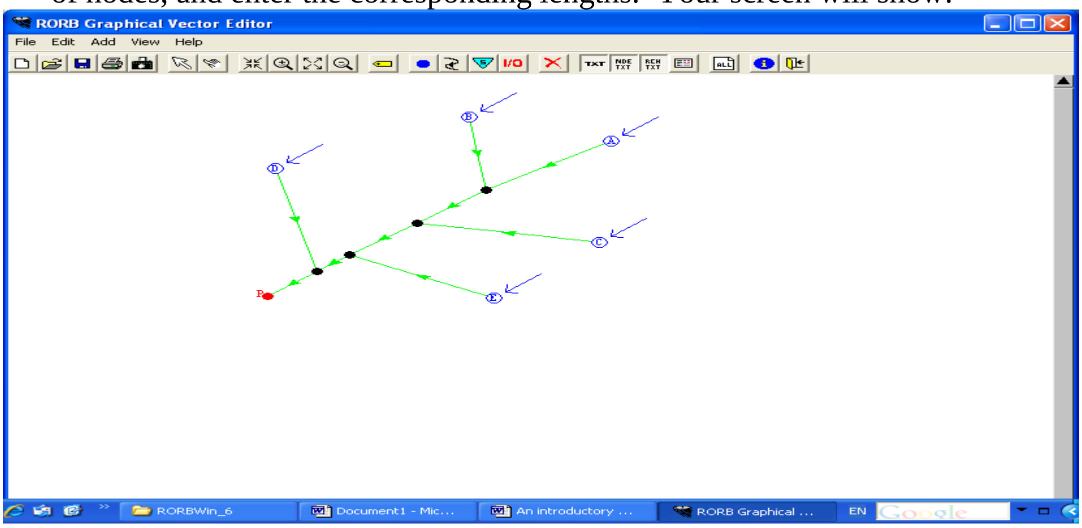

7. At this stage, we can look at the editing capability of the GE. Click on the button showing the arrow (6th from left end), then double click on the node marked 'A'. A window appears which shows the data for that node. Similarly click on the second node, and look at its window. Double click on the arrowhead in the middle of the reach to look at the reach data. [Note that all such data can be edited or adjusted this way – say for urbanisation].

8. Further nodes and reaches can be added one at a time or, easier, by drawing all the nodes first. Using the 'Add a node' button, place all of the remaining nodes on the

screen. (The order is not important, but it is usual to proceed ‘down’ the catchment, picking up sub-areas along the way.) The last node (the one at the catchment outlet) is the end of your modelled area, and is designated Type ‘End of Model’. It would be usual to require a hydrograph printout at this point, so click the ‘Print at This Node’ box. The print Type box is activated. Here the “Print calculated discharge (Code 7) is correct, but you will need a location. Enter ‘Catchment outlet” and click the OK button. Your screen should now look something like:




9. Now click on the ‘Add a reach’ button, draw reaches to connect the appropriate pairs of nodes, and enter the corresponding lengths. Your screen will show:




10. At this point it (or at perhaps several earlier ones for large and/or complex catchments) it is wise to save the datafile. Click the save button (or use “File” and

'Save as"). A message that the catchment needs a name will appear, and a window to input the name, along with any comments the user may want to add to document the datafile. [Note you could also have entered this data at any time using the 'Model name' button.] Name your file 'Catchment of Fig 6.7' and click 'OK'. You will then be prompted for a filename for the saved file. Call it say 'Fig 6-7'. Your '**catg**' file is completed, and ready for use by the RORB program.

11. Here, you could check out the operation of some other buttons:
    - the 'pointed finger' - 'Select and move a node or reach' - can be used to shift nodes. Select a node and drag it. You will note that the stream reach 'kinks' in the middle. Click on the arrowhead in the middle of the reach and drag it to straighten it up. [This feature can be used to fine tune a match to the actual stream network.]
    - The 'zoom in' and 'zoom out buttons are self explanatory
    - the 'TXT' button toggle the screen text on and off. The buttons to the right of it do this selectively for the node text and (if it exits) the reach text. Try these buttons.

12. The basic example is finished. Exit the editor by clicking on the exit button (at the far right of the button toolbar).

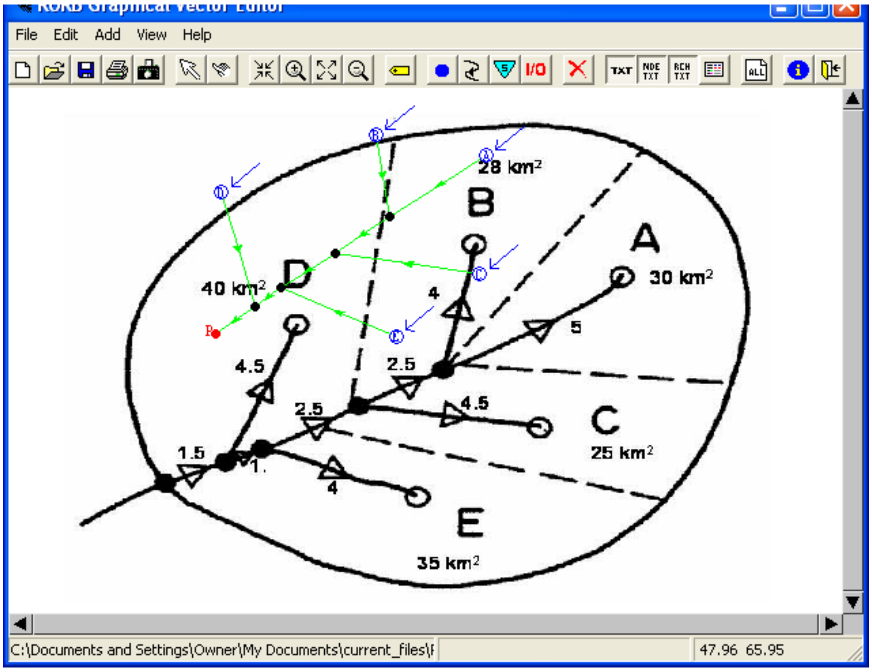
---

**Additional Exercise 1 (file creation with a background picture)**

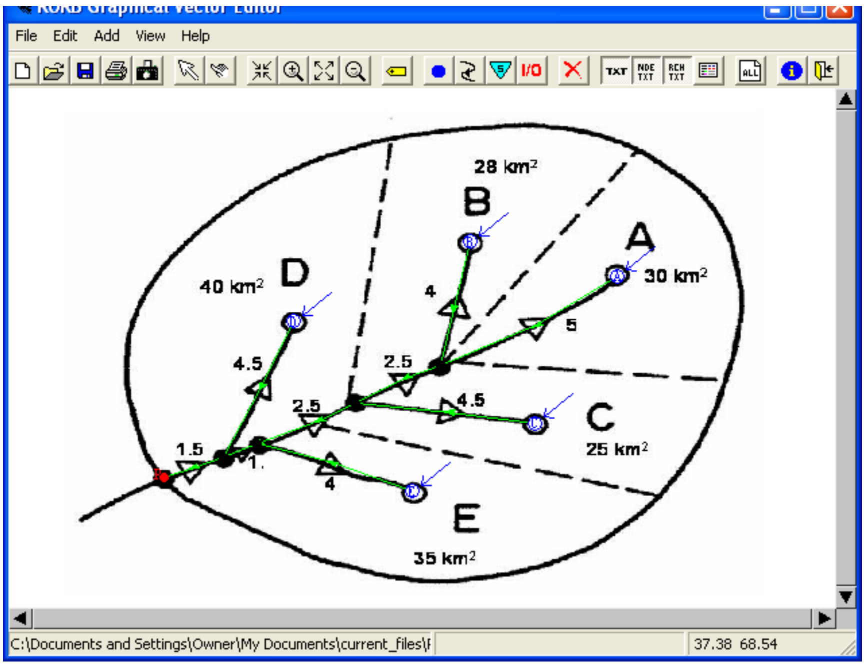
Before Step 4 in the above sequence, click on 'View' then 'Manage Background'. On the 'Manage Background' window which appears, either enter the directory and filename for the file 'Fig 6-7.wmf', or click on the button marked '…' to show your directories to help you locate and select it. Make sure that 'Show image' is checked, and click on the 'OK' button. You will then see the picture (Figure 6.7) displayed on the GUI screen. Continue with Steps 4-10 in the example as above, but selct your node locations to match the background picture. [You can turn the picture on and off any time with the 'Manage Background command.]

**Additional Exercise 2 (matching an existing file to a background)**

Complete the first two steps above, then click on 'Open' to load the file you saved in Step 10 [ie. 'Fig6-7.catg']. Then click on 'View' and 'Manage Backgound' and load picture 'Fig6-7.wmf' supplied with the program [as per Additional Exercise 1 above]. You will then have something on your screen like"




The aim of this exercise is to move the nodes and reaches such that they match the background picture. Hence, select the ‘Select and move a node or reach’ button [the pointed finger], and move the cursor to the node labelled ‘A’. Drag it to the corresponding node for Sub-area A on the background picture. Then click and drag the reach (using the green arrowhead at its midpoint) to a position over the triangle depicting the reach storage on the background picture. Continue with other nodes and reaches, until your screen looks like:




Save this file as say ‘Fig6-7 with background’ to complete the exercise. [Clearly, you can use much more complicated background files to assist interpretation of your RORB datafiles.]

# 7 OPERATION OF PROGRAM

## 7.1 Normal Operation

To use the program on a particular catchment/stream system, it is first necessary to formulate the specific model of that system as detailed in Chapter 4 and then to prepare a data file or files (of type .dat, .cat, and/or .stm) as detailed in Chapter 5. A general description of how then to use the program is given in Section 1.2. More detailed matters related to interactive inputs to the program when running are described below.

The various prompts and messages output by the program are designed to be self-explanatory but the user can be better prepared by studying this chapter.

Table 7-1 below lists the information that can be required in various circumstances in normal program operation. Abnormal conditions are dealt with in Section 7.6 below.

**Table 7-1 User Inputs (Normal Operation)**

| Information required | Where Required | Remarks |
|---|---|---|
| Type of datafiles | Run Specification Screen | Necessary to inform program of the nature of the data being input |
| Name of datafile(s) | Run Specification Screen | User to specify directory. 'Browse' will prompt for '.dat', '.cat', and/or '.stm' files as specified in previous item above, but user can override this by specifying other file types on the 'Select RORB File' screen. |
| Parameter configuration. | Run Specification Screen | User can choose to have catchment-wide parameters or to vary parameters with interstation area |
| Loss model to be used. | Run Specification Screen | Choice of either initial loss/continuing loss, or initial loss/runoff coefficient, models |
| Run options | Run Specification Screen | Can override what is in the input datafile |
| Level of detail in text output file | Run Specification Screen | User can choose three levels of detail [See Section 9.1] |
| Output filename root | Run Specification Screen | User can choose generated names linked to various file input |
| Location of output file | Run Specification Screen | Users can choose directory linked to various input files |
| $m$ and $k_c$ values. | Parameter Specification Screen | If some normal reach storage is being modeled (i.e. nearly always). Pressing the '??' button for $k_c$ brings up choices for a starting value |

(Table 6.1 cont.)

| Information required | Where Required | Remarks |
|---|---|---|
| Initial loss | Parameter Specification Screen | Values are required for all bursts and interstation areas |
| Pervious area runoff coefficient or continuing loss rate, depending on Loss Model selected. | Parameter Specification Screen | Values are required for all bursts and interstation areas for DESIGN runs, or FIT runs with no gauging station. (Model calculates values for other runs) |
| Location details | Design Rainfall Specification Screen | User can select a site for which (ARR) design storm parameters are available, or edit in the parameters for a new location |
| Storm details | Design Rainfall Specification Screen | Option here for a single storm burst (of specified ARI and duration), multiple storm bursts (Section 7.5), or a Monte Carlo simulation (Chapter 8). Duration of calculations needs to be long enough for specified purpose (Section 2.2.5). |
| Temporal pattern details | Design Rainfall Specification Screen | Option to use ARR patterns directly, or to smooth them to eliminate rarer sub-events within the burst |
| Areal reduction factors | Design Rainfall Specification Screen | Option to use ARR87 values (US data), or values derived from Australian data (ARR99). Users can change the area if required. [An area of zero would imply no reduction factor.] |
| Loss factor details | Design Rainfall Specification Screen | If it is desired to have losses varying with ARI, this can be specified. The user has to supply the factors which apply. |
| Output directory | Design Rainfall Specification Screen | User can specify the computer directory for output files |
| Initial drawdown specification | Interactive design of storage screen. | Three options provided. See Section 2.4.4 for more details. |
| Spillway data | Interactive design of storage screen. | User to specify elevation and effective length for spillways in order of increasing elevation (See Section 2.4.3) The discharge coeff. (for all spillways) can be changed interactively. |

(Table 6.1 cont.)

| Information required | When required | Remarks |
|---|---|---|
| Pipe data | Interactive design of storage screen. | User to specify inlet elevation, number of pipes, length, gradient and diameter (in order of increasing elevation (see Section 2.4.3) The entrance loss and bend loss coeffs. (for all pipes) can be changed interactively. |
| Storage-elevation parameters. | Special storage design, storage-elevation relation = 2. Interactive design of storage screen. | Can be changed interactively if required (See Equation 2-15). |

## 7.2 Fitting Model to Recorded Data

1. Variation of model parameters to obtain a good fit of the calculated to the actual hydrograph is controlled entirely by the user; automatic fitting or optimization is not performed.

2. Decreasing $k_c$ increases the hydrograph peak and decreases the lag, while increasing $k_c$ does the opposite; $k_c$ is the principal parameter of the model and is the main means of achieving a fit; values of $k_c$ determined for different runoff events on the same catchment sometimes vary by a factor of as much as 2. (The variation is due to data errors such as recording errors, baseflow separation errors and rainfall variability, and also to the model not fully representing the hydrologic processes involved in runoff generation.)

3. Varying initial loss is also an important means of achieving a fit; it affects directly the start of hydrograph rise but, especially in long, variable storms, changing the initial loss can have unexpected results on the time distribution of rainfall-excess and thereby cause significant changes in hydrograph peak; selection of Type 2 output is useful in showing the areal as well as the temporal distribution of rainfall-excess.

4. Decreasing the *m* value, as long as the consequent change in $k_c$ is made (see Section 2.2.4), tends slightly to delay the start of rise and also the tail of the recession but to advance the peak. Increasing *m* has the opposite effect. These effects are sometimes useful in improving a fit, but are less important than those of $k_c$ and initial loss. Low values of m (eg less than 0.7) should be viewed with suspicion.

5. A useful procedure for fitting the model to a catchment, due to Weeks (Ref. 9), makes use of several events, preferably covering a wide range of peak discharges. For each event, a selected range of *m* values is used. For each *m* value, $k_c$ is varied until the best fit with that *m* value is achieved. A graph of $k_c$ vs. *m* for that event is then plotted. When these

graphs for the several events are superimposed, they usually indicate a unique pair of $k_c$ and $m$ values that provide a good fit for all events.

Such graphs also indicate that if, for a constant $m$ value, the fitted $k_c$ values vary from flood to flood, then the $m$ value is likely to be in error and should be increased if $k_c$ increases with increasing size of flood, or decreased if $k_c$ decreases with increasing size of flood.

6. Sometimes, especially on catchments whose lower reaches are very flat, the shape of a hydrograph can be reproduced but not its timing; if this seems to be happening, it may be necessary to insert a translation into the model by using a Code 8 in the control vector. Another option for clearly non-homogeneous catchments (in terms of slope) is to use L/√Sc rather than just L for the reach parameter (Section 2.2.2).

7. Small events, containing less than about 10 *mm* of runoff, are often more difficult to fit than large ones due to extreme areal variability of runoff, partial area runoff, and large differences in the time distribution of rainfall-excess caused by small errors in the adopted loss model; they should be avoided if sufficient large rises are available.

8. In fitting the model to a hydrograph of surface runoff, the calculated hydrograph is always asymptotic to the base line, while the actual hydrograph intersects the base line; lack of fit at the end of the hydrograph and a small negative volume error are normal as a result.

9. Multi-peaked hydrographs can usually be fitted better if the event is treated as a multi-burst, multi-rise event (see Section 3.1.3); the additional data preparation required for this is not great, and this facility should be used if a reasonable estimate of the proportions of total runoff in the different rises can be made.

10. The user must decide on the relative importance to be assigned to the various error measures printed out; error in peak discharge is generally considered most important, but overall fit should also be considered important.

11. The time increment and areal sub-division should not be excessively fine, though they must be small enough that temporal and areal variability can be adequately represented. The user must decide these matters subjectively. It is recommended that at least 5 sub-areas be placed above any hydrograph printout point to allow sufficient smoothing and attenuation of the rainfall excess hyetographs.

12. A limited form of fitting is possible without a gauging station hydrograph if an estimated peak or discharge-time value is available and rainfall is known. In such cases, the initial loss, loss parameter and m value must be estimated and the $k_c$ value fitted.

13. Note that if significant natural or artificial storages are present, then these must be specifically modelled. The "internal" hydrographs will not be calculated correctly if the effect of a large storage is "spread" over the catchment during fitting to a gauged hydrograph.

## 7.3 Testing Model

1. The model used in this program does not purport to predict the loss parameters for a given event, a limitation common to most design flood estimation procedures and which, it is hoped, might be removed by future research. In testing the model, therefore, it is legitimate to vary initial loss within reasonable limits to improve the fit, unless the user is, at the same time, testing a procedure, external to the program, for predicting the losses.

2. Having assessed the model through testing on events not used in fitting, the user may then legitimately use the test data for fitting and so improve the estimates of $k_c$ and m.

3. Events used for testing should, if possible, sample events considerably different from those used for fitting. Since use of the model in design is likely to be for larger floods than those used in fitting and testing, the greater the range of conditions for which the model has been verified the more confidence can be had in its use under the more extreme design conditions.

4. Users should be realistic is their expectations of accuracy. “Good” streamflow data are considered accurate to ± 15%; peak flows (where rating curves have been extrapolated) are less reliable. Rainfall sampling errors can be very significant. Note that timing errors between rainfall and runoff can have a big influence on some fit parameters.

## 7.4 Use of Program in Design

1. In catchment studies, the ***same*** overall catchment area and channel network ***must*** be used for design as for fitting and testing (see Section 2.2.4); they are determined by the most downstream point of interest, whether it be a design site (e.g. dam site) or a gauging station used in fitting.

2. Some features of the catchment will normally be different under design conditions than previously due to construction of a storage, channel works, etc. Consequently, the model and the control vector and/or the data must be modified to represent the changed conditions. However, only features directly affected by the construction should be changed in the model. In the case of a dam this is achieved by changing the reach type codes (Section 2.2.3) of reaches drowned by the storage and inserting a special model storage to represent the one constructed. The sub-areas and channel network can be left unchanged (see Section 4.1).

3. It is suggested that a reasonable variety of assumed design conditions (loss rates, storm durations, frequencies, etc.) be studied as part of a general sensitivity analysis.

4. The facility for time variation of loss parameters in DESIGN runs by defining the design storm as a multi-burst storm should not be overlooked; it may be useful for long design storms.

5. For design on ungauged catchments, the values of $k_c$ and $m$ should, if possible, be determined on the basis of values derived by fitting for catchments in the same region

rather than by using 0.8 for $m$ and Equation 2-4 for $k_c$. [Note: This procedure is less reliable if there is a great differences in catchment shape.]

If only one gauged catchment is available for such fitting, its results may be used to re-derive the coefficient (2.2) in Equation 2-4, while the exponent of A (0.5) may be retained. [Note that for $m$ = 0.8 in Equation 2-4, the $Q_p$ term becomes unity.]

If two or more nearby gauged catchments are available for fitting, selection of $k_c$ for the ungauged catchment will be facilitated by plotting their fitted $k_c$'s against catchment area on log log paper and plotting Equation 2-4 with $m$ = 0.8 on the same graph. [Note that if the catchments have marked differences in shape

*6.* $k_c$ values determined by fitting with different $m$ values are not comparable with each other (see Section 2.2.4). $k_c$ values being averaged to provide a design value should all relate to the same $m$ value, the one to be used in design.

7. A common requirement in flood estimation is to generate hydrographs (or peak flows) which correspond to a range of recurrence intervals, storm durations, and (perhaps) runoff coefficients. To achieve this required a lot of repetitive runs with previous versions of the program.

   RORB allows users to specify ‘Multiple Design Storms’ on its ‘Design Rainfall Specification’ screen. It gives them the option to vary storm recurrence interval, storm duration, and/or runoff coefficient over a series of individual runs in a single ‘batch’ job. The areal reduction factor can also be chosen to vary with storm severity and duration.

   A summary file of the main results of the individual runs is displayed at the end of the batch job, with more details stored in separate text files for each of the combinations selected.

## 7.5 Generation of Design Rainfalls

RORB can generate design storms (or rather bursts) for catchments in accordance with either the procedures recommended in Australian Rainfall and Runoff (Ref. 24) or using user-defined rainfall intensity-frequency-duration (IFD) data.

When modelling Australian catchments, it is recommended to apply the Australian Rainfall and Runoff procedures by selecting “ARR IFD” in the Design Rainfall Specification window; these apply for a range of durations (up to 72 hours), average recurrence intervals (up to 100 years), in conjunction with appropriate temporal patterns. The methodology for a given site requires the specification of 9 values obtained from maps in ARR Volume 2, and the corresponding rainfall zone.

The files supplied with the RORB program give values for a number of selected locations (eg capital cities), but provision is made for users to input the parameters for any location of interest and to save these for subsequent use.

In addition, the user has the option to ‘filter’ the patterns (ie. adjust them to smooth out any ‘rare’ sub-durations that may be embedded in the design bursts), or not.

Another option is to apply the updated areal reduction factors (Ref. 24, Book VI) now available for several states in Australia. Alternatively, the user may elect to apply the US-based areal reduction factors that were used in ARR 1987 (Ref 16), or to apply none at all. The latter option would facilitate sensitivity studies on the effect of areal reduction factors on calculated hydrographs, for instance. The rainfall can be distributed uniformly over the catchment, or else if desired a specific spatial pattern could be used.

If the catchment to be modelled is outside Australia or otherwise requires special IFD data, this can be input by the user by selecting “User defined IFD”. The user is required to nominate the average recurrence intervals (in years) and durations (in hours) to be modelled, and the rainfall intensity (in mm/hr) for each. A temporal pattern is also required for each duration (or set of patterns if using Monte-Carlo simulation). Selecting user defined IFD data necessarily disables some of the other options in the Design Rainfall Specifications window. For example, the option to apply temporal pattern filtering and areal reduction factors will not be applicable for user defined data.

The program automatically creates the required rainfall data files when the “Separate catchment and generated design storm(s)” option is selected in the Run Specification dialogue. Either single or multiple design storms can be generated and then run – where both input storm and output files are created as individual files according to the traditional design event approach – or else the Monte-Carlo simulation approach can be used. The difference between these two approaches and further information on the latter are provided in Chapter 8.

## 7.6 Abnormal Conditions

Where possible, abnormal conditions, in both the data checking and the computation phases, are handled by the program. When certain abnormal conditions arise during execution, the user will be informed by an error message and (generally) given the opportunity of selecting a course of action. The conditions under which such messages occur, the forms of the messages, and the actions that follow user responses are detailed in Table 7-2 below.

Most other abnormal conditions that arise due to the input of an invalid value or one that leads to an invalid condition result in the printing of an error message and the opportunity to input a different value. However, it is not practicable to guard against every conceivable invalid input and some such inputs could result in erroneous results or in system programmed failures. Because of the former possibility, the user must exercise great care and responsibility in the input of values and in the interpretation and checking of results as in normal good engineering practice. If a system programmed failure occurs, the user should check the validity of the data that has been input interactively and also that in the data file. If such a check reveals no error, there may be a program bug; in such instance, please contact the program authors with details (see Section 1.8).

Another possible cause of a system programmed failure is an invalid data file attribute or structure. Such matters may be software-dependent and cannot be guarded against in the program. If a system programmed failure occurs during the data reading and checking phase, the user should carefully check both the data and the data file attributes.

**Table 7-2 Error conditions and actions (Abnormal Conditions)**

| No. | Condition & [Message] | Action on user 'yes' response | Action on user 'no' response |
|---|---|---|---|
| 1 | Runoff coeff. negative. [Note negative coeff. Want to increase initial loss (yes or no)?] | Opportunity provided to input new loss parameter(s). | Execution proceeds normally using abnormal coefficient value. |
| 2 | Runoff coefficient greater than unity. [Runoff coeff. > 1.0. Want lower initial loss (yes or no?)] | Opportunity provided to input new loss parameter(s). | Execution proceeds normally using abnormal coefficient value |
| 3 | Zero discharge at gauging station & non-zero initial drawdown of storage.<br><br>[No flow passed gauging station. Runoff coeff./loss rate & storage contents indeterminate. Calculated runoff coeff./loss rate minimal and storage contents minimal/ maximal. Want to change current runoff coeff./loss rate (yes or no)?] | Opportunity provided to input new value of runoff coeff. or loss rate.<br>(The value input might be based on values derived for other bursts or other interstation areas, on knowledge of what the drawdown of storage should be at end of burst, or on comparison of actual and calculated hydro-graphs.)<br><br>New final drawdown(s) is (are) computed (and also a new runoff coeff./loss rate if the input value was invalid) and the final query of the adjacent message is repeated. | Execution proceeds using current values of runoff. Coeff./loss rate and storage contents/drawdown. |
| 4 | Excessive initial loss or excessive draw-down of storage adopted with Loss Model 1, (See 2 above re Loss Model 0). [Note negative loss rate. Lower initial loss or drawdown required. Want lower initial loss (yes or no)?] | Opportunity provided to input new initial loss. | Execution terminated. *Note*: Excessive initial drawdown might be due to a wrong value in the data file or, if Loss Model 0 is used, to a wrong final draw-down value calculated in a previous burst for which there was no flow at the gauging station - see 3 above. The latter error can be corrected in a new run. |

(contd.)

Table 6.2 (cont.)

| No. | Condition & [Message] | Action on user yes response | Action on user no response |
| --- | --- | --- | --- |
| 5 | Calculation of runoff coeff./loss rate, an iterative process, not converging.<br><br>[Loss parameter, calculation not con-verging, burst *n* station *m*.  Adopt current value & proceed (yes or no?)] | Execution proceeds using current (erroneous) value of runoff coeff. or loss rate for the relevant burst and interstation area. | Execution terminated. |
| 6 | Routing calculation, an iterative process, not converging.<br><br>[Reach storage no. *n*, or name of special storage.<br><br>Routing calculations not converging, time inc. *m*. Adopt current outflow value & proceed (yes or no)?] | Execution proceeds using mean of current and previous outflow estimates and, for retarding basins, water surface elevation. | Execution terminated. |

# 8 MONTE-CARLO SIMULATION

RORB includes the facility for estimation of design floods using Monte-Carlo simulation. The overall concepts behind this approach are discussed below, however it should be noted that use of this technique represents emerging, rather than established, design practice, and accordingly it should only be used by practioners with a sound grasp of flood estimation theory. Readers interested in gaining more background to this than is presented below may care to refer to Refs. 25 and 26.

## 8.1 Overall Concepts

Current practice for estimation of design floods is typically based on the "design event" approach, in which all parameters other than rainfall are input as fixed, single values. Considerable effort is made to ensure that the single values of the adopted parameters are "AEP-neutral", that is, they are selected with the objective of ensuring that the resulting flood has the same annual exceedance probability as its causative rainfall.

While this approach represents current "best practice" in Australia (and overseas), it does suffer from the limitations that:

- the AEP-neutrality of some inputs can only be tested on frequent events for which independent estimates are available;
- for more extreme events, the adopted values of AEP-neutral inputs must be conditioned by physical and theoretical reasoning; and,
- the treatment of more complex interactions (such as the seasonal variation of inputs) becomes rapidly more complex and less easy to defend.

Joint probability techniques offer an alternative to the design event method. These techniques recognise that any design flood characteristics (e.g. peak flow) could result from a variety of combinations of flood producing factors, rather than from a single combination. For example, the same peak flood could result from a moderate storm on a saturated basin, or a large storm on a dry basin; in probabilistic terms, a 1 in 100 AEP flood could be the result of a 1 in 50 AEP rainfall on a very wet catchment, or a 1 in 200 AEP rainfall on a dry catchment. Joint probability approaches attempt to mimic "mother nature" in that the influence of the most important flood producing factors are treated as variables, thereby providing a more realistic representation of the flood generation processes.

In the current implementation of RORB the two factors can be treated in a stochastic manner, namely initial loss and the rainfall temporal pattern (Figure 8-1). While it is possible to consider continuing loss as a variable, its value is dependent on initial loss, and added complexity would be required to deal with this correlation; furthermore, the likelihood distribution of proportional loss has not been studied to date and the required information on its distributional characteristics is not available. However, for most routine flood estimation studies, particularly those focused on estimating peak flows, the stochastic treatment of initial loss and the temporal distribution of rainfalls should be sufficient to capture the influence of variability in the main flood producing factors.

The following sections outline the overall framework adopted, and the nature of the evidence that can be used to characterise the distribution of the inputs.

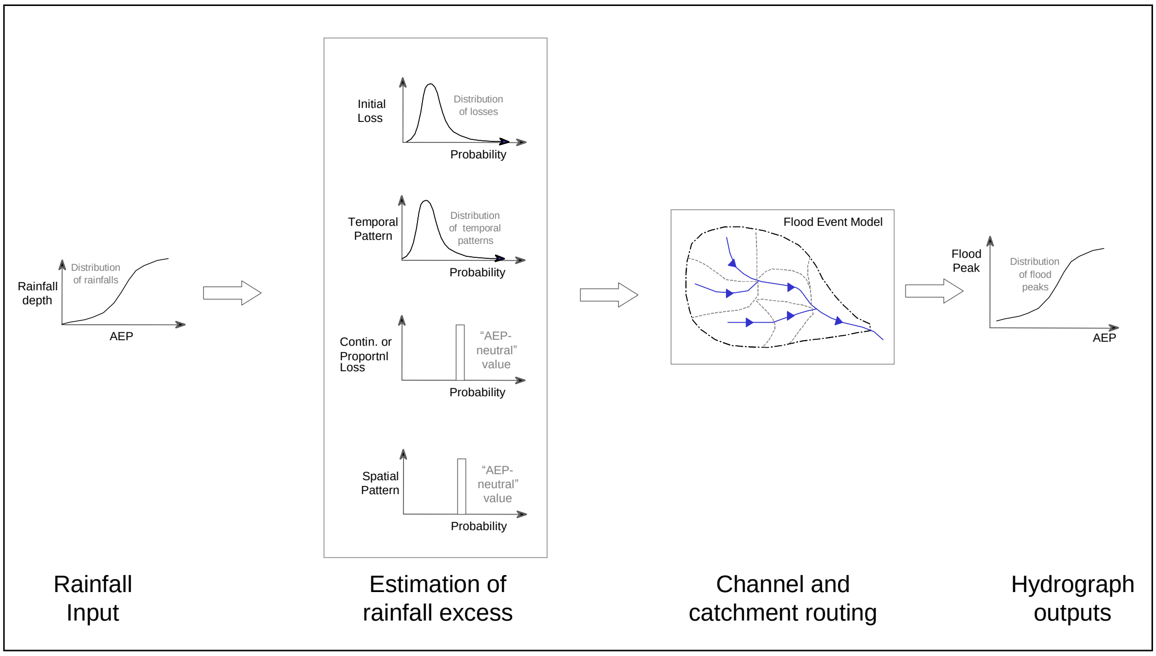



**Figure 8-1 Schematic illustrating the stochastic and fixed nature of flood producing factors available in RORB.**

## 8.2 Simulation Framework

An overview of the joint probability framework adopted is illustrated in Figure 8-2. In essence the approach involves the undertaking of numerous model simulations where selected model inputs are varied in accordance with that observed in nature. The inputs are sampled from non-parametric distributions that are either based on readily available design information or else on the results of recent research.

In developing the joint probability framework particular attention was given to ensuring that the nature of the inputs and the manner in which they are incorporated are consistent with general "design event" concepts; this is done not only to facilitate the transition between design event and Monte-Carlo approaches, but also to ensure that the design information used is as consistent as possible with the design information used in traditional flood estimation practice (as provided in such guidelines as Ref 24). The following briefly describes the main elements of the approach, and the manner in which they relate to established design information.

*Select rainfall depth* (see Figure 8-2). Rainfall depths are stochastically sampled from the cumulative distribution of rainfall depths. The relationship between burst depth and annual exceedance probability is based either directly on the design rainfall information in Book II of Australian Rainfall and Runoff (24) (the user need only specify the location of interest as discussed in Section 7.5) or user defined intensity-frequency-duration data. The program samples rainfalls over the range from 1 ARI to in excess of 500 ARI (or the range of user defined ARIs), though results are only provided for a narrower range of likelihoods that are found to be reliably derived through Monte-Carlo simulation. The rainfall distribution is divided into a number of discrete sampling intervals to reduce the number of simulations required to define the rarer events of interest (Section 8.5).

*Select storm losses:* Storm initial losses are stochastically sampled from a non-parametric distribution that is specified by the user. Information on the shape of the distribution is available from the analysis of concurrent rainfall and streamflow data (Section 8.3), and in general the median of this distribution should equate to the typical loss value used in the design event approach.

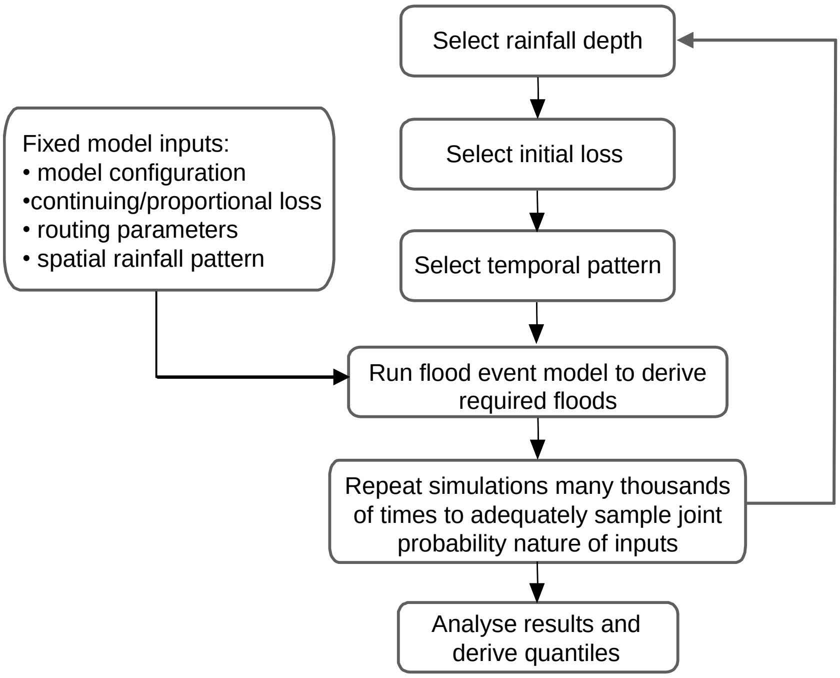



**Figure 8-2 Overview of joint probability framework.**

*Select temporal pattern.* Temporal patterns are randomly selected from a sample relevant to the area and duration of the selected storm for the region of interest. The temporal patterns may be derived from large historic storms that have been observed in the region, and a utility to extract the necessary temporal patterns is provided with the program (Section 8.4). In concept, a sample of such temporal patterns is the same as that used to derive the design temporal patterns in Volume II of Australian Rainfall and Runoff.

*Other inputs*: All other inputs, namely model configuration, continuing or proportional loss values, rainfall spatial pattern, and routing parameters, are input as fixed values as applied in the traditional design event approach.

*Run simulation*: Simulation involves running the RORB model numerous times within each rainfall interval (Section 8.5). Due to the large number of simulations being undertaken the inputs and outputs of each individual run are not written to disc file, but a summary of each individual run can be saved is desired.

*Analyse results*: The individual run results are analysed in a way that provides direct (expected probability) estimates of the flood frequency curve, that is the program provides the relationship between flood peaks and their annual probabilities of exceedance (Section 8.5). The program allows several different durations to be run sequentially, and automatically calculates the “critical duration” for each standard exceedance probability. Both graphical and tabulated output is provided.

## 8.3 Derivation of Initial Loss Distribution

The stochastic nature of the seasonal loss distributions is best obtained from regional information such as provided by Hill et al. (27) and Ilahee (28). The results of the Hill et al. study were based on a study of 22 rural catchments located across Victoria. For each catchment loss values were obtained for all flood events on record, which yielded information for an average of 17 events per catchment (a total of 384 individual events across all the catchments). The results of Ilahee were based on 48 small to medium (less than 1000 $km^2$) rural, unregulated catchments in Queensland. These catchments are mostly located in coastal areas and have streamflow and rainfall records of between 11 and 48 years.

When the results provided by Hill et al. and Ilahee are standardised by representing each value as a proportion of the median loss, the loss distributions exhibit a high degree of consistency (Figure 8-3); the results clearly support the assumption that while the magnitude of losses may vary between different catchments, the shape of the distribution does not. In other words, while it may be expected that typical loss rates vary from one catchment to another, the likelihood of a catchment being in a relatively dry or wet state is similar for all catchments. The concept of how the location of the loss distribution changes but not its shape is schematically illustrated in Figure 8-4. A tabulation of the standardised distributions, and the average of the two, is provided in Table 8-1.

The average distribution (the last row in Table 8-1) is provided as a program default, though the values can be altered by the user if desired. During simulation the loss factors are stochastically sampled, and the initial loss used in the loss modelling is simply the product of the loss factor and the initial loss value provided by the user in the “Parameter Specification” dialogue box; this value is additionally multiplied by the ARI-dependent loss factor if the “Losses vary with ARI” option is selected from the “Design Rainfall Specification” dialogue box.

**Table 8-1 Distribution of initial loss factors derived from empirical data**

| Source | Initial loss (as proportion of the median) exceeded a given proportion of time | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0% | 10% | 20% | 30% | 40% | 50% | 60% | 70% | 80% | 90% | 100% |
| Hill et al. (27) | 3.03 | 1.90 | 1.60 | 1.38 | 1.16 | 1.00 | 0.85 | 0.71 | 0.52 | 0.35 | 0.10 |
| Ilahee (28) | 3.51 | 2.48 | 1.88 | 1.51 | 1.17 | 1.00 | 0.78 | 0.59 | 0.38 | 0.19 | 0.04 |
| Average | 3.27 | 2.19 | 1.74 | 1.45 | 1.17 | 1.00 | 0.82 | 0.65 | 0.45 | 0.27 | 0.07 |

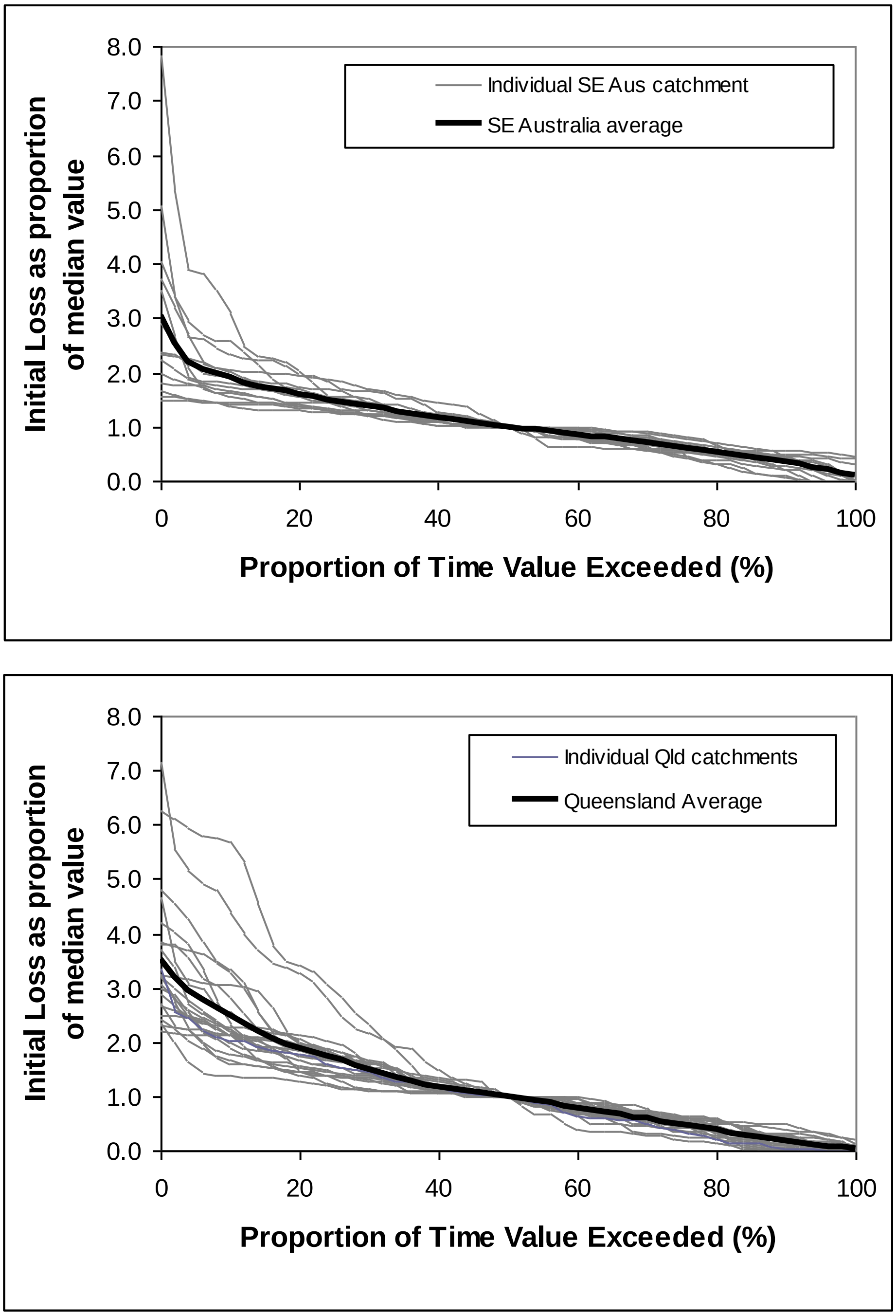



**Figure 8-3 Initial loss distributions for (a) south eastern Australia (27) and (b) Queensland (28) (distributions have been standardised by dividing by the median, where thin lines represent individual catchment results and thick lines represent the regional average).**

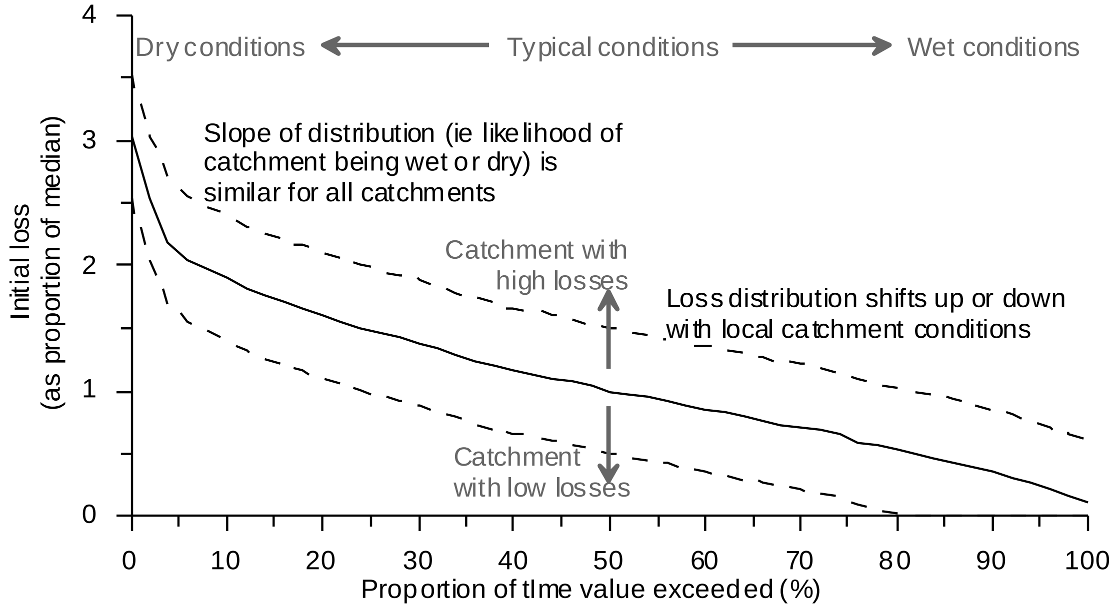



**Figure 8-4 Schematic illustration of variation in location but not shape of initial loss distribution.**

## 8.4 Specification of Temporal Patterns

Temporal patterns obtained from large observed storms in the region of interest are ideally suited for use in the Monte-Carlo simulation. A useful guide to applicability is to select temporal patterns from a pluviograph station located same rainfall zone as the catchment of interest (see Figure 2.2, Book 1, Vol 1 of Australian Rainfall and Runoff, Ref 24).

It has been observed (e.g. 29, 30) that temporal patterns tend towards a more uniform distribution with increasing ARI of the rainfall event. Accordingly, if desired the temporal patterns used in the Monte-Carlo simulation can be randomly selected from a restricted range of storms with similar ARI to that of the rainfall event being simulated. For example, if "Censor pattern selection using window of 10 nearest ARIs" is selected from the "Monte-Carlo Simulation Details" dialogue, then a temporal pattern is selected from a moving window of 10 temporal patterns corresponding to the ARI of the rainfall depth being generated. This censored sampling approach is illustrated in Figure 8-5, where the difference in temporal pattern sample is shown for two different rainfall depths corresponding to ARIs of 10 and 28 years. If the sample of available temporal patterns is limited, or else there is no evident dependence between temporal pattern shape and ARI , then unconditional sampling should be used (that is, for the example shown in Figure 8-5, temporal patterns would be selected from all 30 available patterns without regard to their associated ARI).

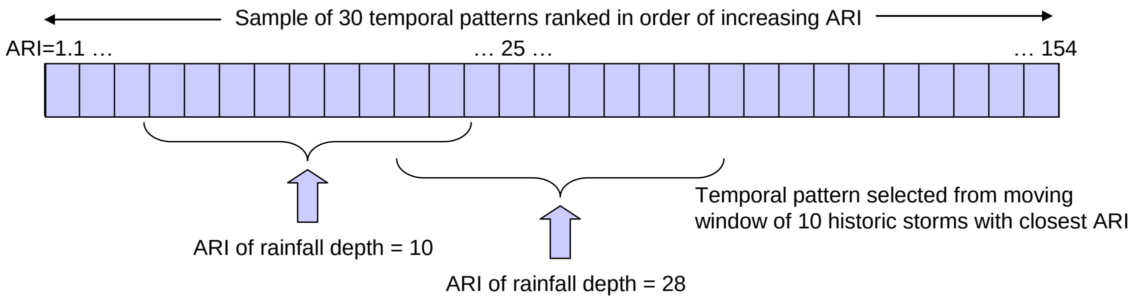



**Figure 8-5 Schematic illustration of optional censoring of temporal pattern according to the ARI of the rainfall depth.**

An example set of appropriate temporal patterns for Zone 1 is supplied with the RORB installation files. These patterns were extracted from six minute rainfalls for the Melbourne Regional Office (Gauge Ref 086071), for the period 30 April 1873 to 31 Dec 2001 (where 33.5% of the data was missing and ignored). A plot of the cumulative patterns for the 30 largest 24 hour storms is shown in Figure 8-6. It is worth noting that for the range of events contained in this example (the ARI of the storms range from around 3 to 80 years) that there is only a weak indication that temporal patterns tend towards uniformity, and thus in this case it would be justified to sample the patterns unconditionally. For comparison purposes, the corresponding design temporal pattern recommended in ARR (24) for Zone 1 is also plotted in Figure 8-6. It is seen that the ARR storm is more “rear-loaded” than most of the storms in the historical sample, with rainfall intensities for the first third of the storm being amongst the lowest in the sample, and those for the last third amongst the highest. Thus, in this particular example, the ARR temporal pattern does not appear to be representative of the majority of the storms recorded at this station.

The RORB program provides a utility for the extraction of temporal patterns from pluviograph data (to be found under the “Tools” menu). This utility reads in pluviograph data in a format typically provided by the meteorological agencies (where all rainfall readings for the day are provided on a single line, with daily records arranged in chronological sequence, and missing data is denoted by a negative number – the precise spacing and format of the data can be specified by the user). The program extracts temporal patterns from the data file for a standard range of burst durations equal to or longer than five times the time resolution of the data. Thus, if 6 minute data is input then the minimum storm duration provided is 30 minutes. The temporal patterns can be aggregated to any whole number of durations specified by the user. The utility outputs a set of raw and filtered temporal patterns; the former set can be used for auditing purposes, and the latter set is used as an input to the Monte-Carlo simulation. If desired, temporal patterns can be manually added or deleted from the Monte-Carlo file using a simple text editor, though if this is done the data record containing the number of temporal patterns for the selected duration must be altered to reflect the number of patterns remaining.

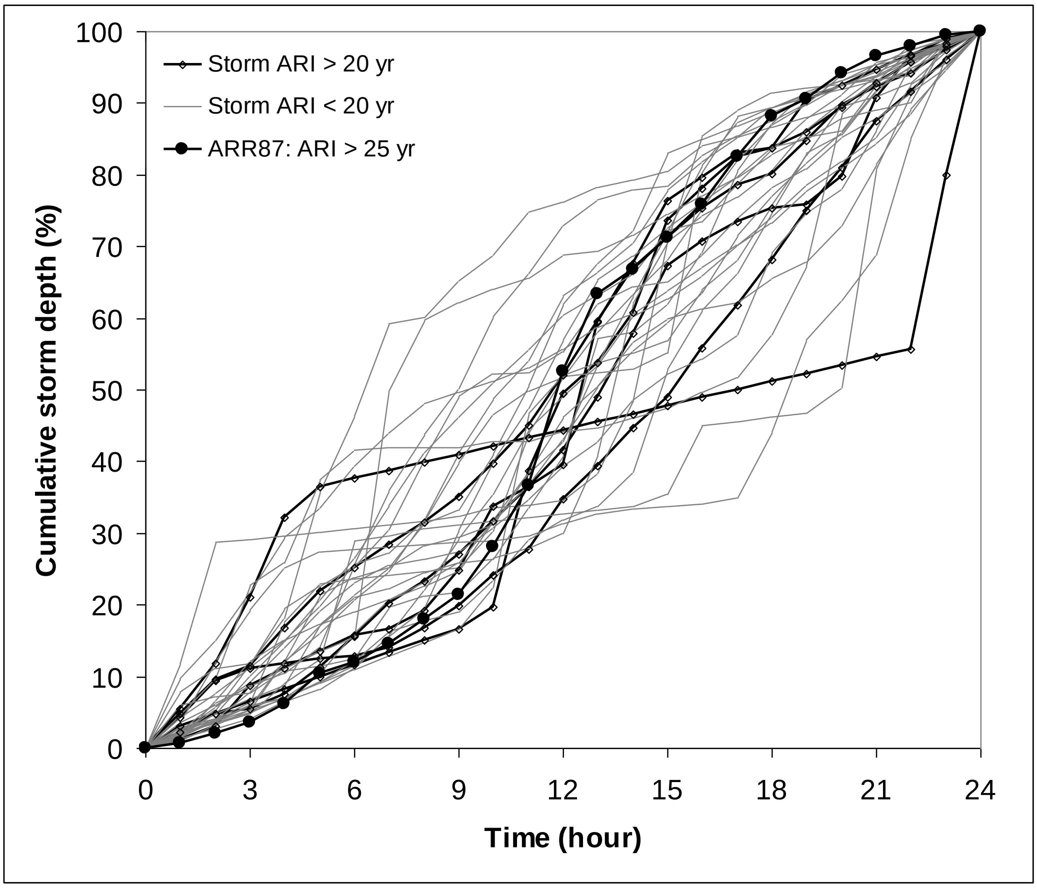



**Figure 8-6 Cumulative plot of 24 hour temporal patterns from 30 largest storms extracted from Melbourne Regional Office pluviograph station (divided into patterns from storms with an ARI less than and greater than 20 years), and corresponding Zone 1 pattern for ARI > 25 year from ARR (Ref 24).**

## 8.5 Derivation of Frequency Curves

The Monte-Carlo simulations are undertaken using a stratified sampling approach in which the sampling procedure focuses selectively on the probabilistic range of interest. Thus, rather than undertake many thousands of simulations in order to precisely estimate an event with, say, a 1 in 100 probability of exceedance, a reduced number of simulations are undertaken over a specified number of probability intervals. As illustrated in Figure 8-7, the rainfall frequency curve is divided into a specified number of intervals (in this example 50) uniformly spaced over the standardised normal probability domain (Detail A in Figure 8-7), and a specified number of stochastic samples (in this example 20) of rainfall depths are taken within each division (Detail B). For each rainfall depth, a simulation is undertaken using appropriately sampled inputs (as described above), thus providing 20 estimates of flow peak for each division (Detail C). These flow peaks are then analysed using the Total Probability Theorem to yield expected probability estimates of the flood frequency curve. In all, for this example a total of 1000 simulations are undertaken to derive the frequency curve corresponding to each storm duration considered.

It is normally sufficient to select between 30 and 50 discrete rainfall intervals and around 20 combinations of inputs per interval. The objective here is to select a sufficient number of intervals and combinations such that a sufficient number of temporal patterns and losses are trialled to span all possible combinations of interest. If in any doubt as to the number of simulations required an increasing number of runs can be undertaken until there is negligible change in results.

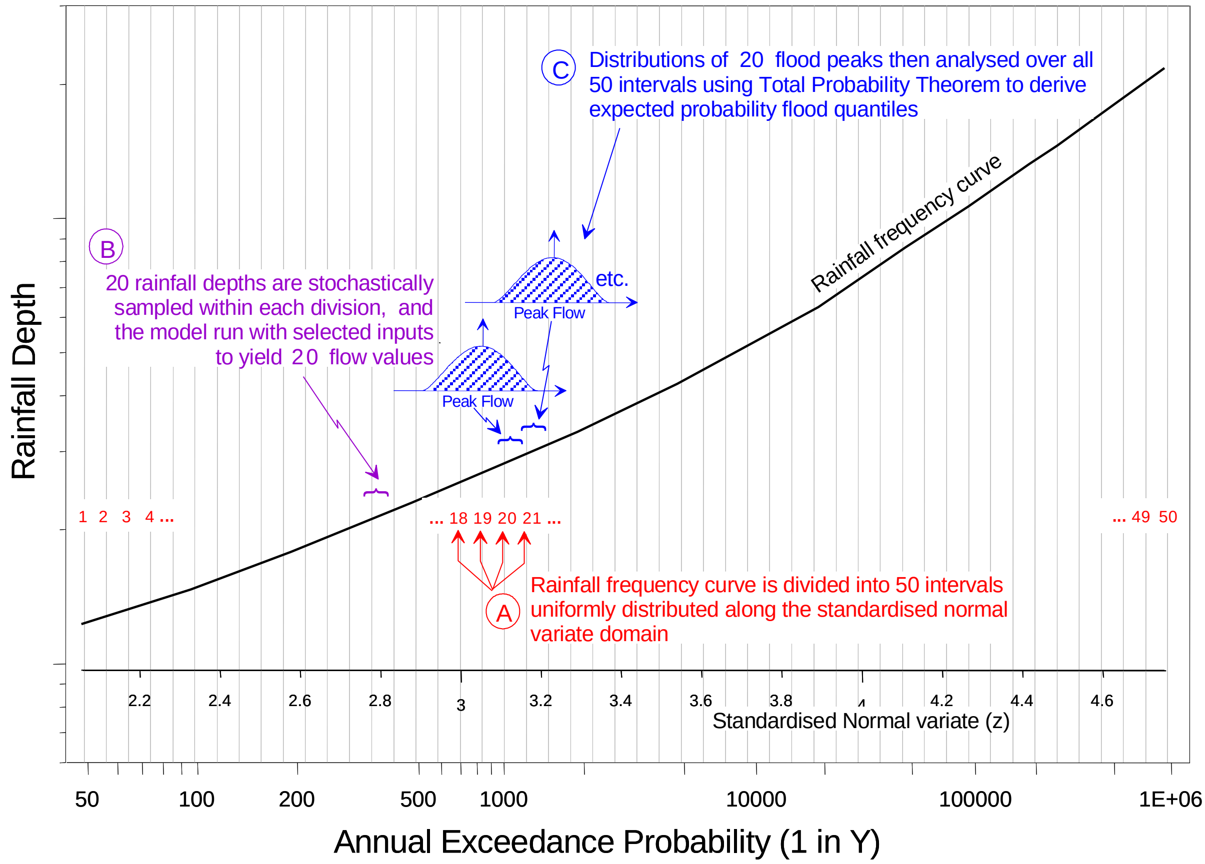



**Figure 8-7 Illustration of manner in which stratified sampling is applied to rainfall frequency curve**.

# 9 OUTPUT

Both graphical and text output can be viewed on the screen, and saved as required to a file to provide a record of the run. A separate parameter file can be saved to facilitate reruns.

This chapter deals principally with the output text file, as the graphical plots generated by the program are regarded as generally self-explanatory.

## 9.1 Control of Text Output

The principal text output of the RORB program is calculated hydrograph data (always output), and parameter values, fit summaries, and error messages (which are always output if present). Other possible output includes hyetographs of rainfall and rainfall-excess, formatted listings of the data, indicators of progress in reading the data file, and storage-elevation-discharge tables for special storages. Furthermore, hydrograph data may be output for any node on the catchment model.

The units used for output variables are printed with the values, and are listed in Table 5-1.

The extent of output is controlled both interactively (level of detail selected on the Run Specification Screen) and through the data file (as discussed in the next paragraph below). The selection options on the Run Specification Screen are given in Table 9-1; the default 'Flows and input data' produces the most output, and the fullest documentation of the run.

**Table 9-1 Information Detail (Run Specification Screen)**

| Option | Output Produced |
|---|---|
| 'Flows only' | Hydrograph information<br>Parameter values<br>Fit summaries<br>Error messages |
| 'Flows & rainfall only' | As for 1, *plus* Hyetographs |
| 'Flows and input data' | As for 2, *plus* Data listing and data file reading indicators |

Use of control codes 7, 7.1, 11, 12, 14, 16, 16.1, 18 and 19 (see Section 4.2) causes hydrograph information to be output as indicated in Table 9-2. When this information is printed, the user must indicate whether or not the complete hydrograph(s), and, when appropriate, a hyetograph in the case of Codes 7, 7.1, 11, and 12, are also to be printed/plotted.

**Table 9-2 Information Output by Various Control Codes**

| Code | Information Printed on: | Hydrograph Label | Heading |
|---|---|---|---|
| 7 | Running hydrograph | Calc. | Calculated by hydrograph at:Location |
| | If code occurs in an interstation area, rainfall-excess hyetograph of sub-catchment above outlet of that interstation area | | |
| 7.1 | Running hydrograph | Calc. | Gauging station at: Name of gauging station |
| | Gauging station hydrograph | Actual | |
| | If gauging station at outlet of sub-catchment, rainfall-excess hyetograph of that sub-catchment | | |
| 11 or 12 | Running hydrograph before addition of sub-area input | Upstream | Sub-area inflow point, sub-area *a* |
| | Rainfall-excess hydrograph of sub-area | Sub-area | |
| | Rainfall-excess hyetograph of current sub-catchment | | |
| | Information as for Code 15 | | |
| 14 | Running hydrograph before confluence | This branch | Confluence at d/s end of reach *nn* |
| | Stored hydrograph of other branch | Prev. branch | |
| 15 | Running hydrograph, downstream end of reach | Downstream | Normal storage Reach no. *nn* |
| | Running hydrograph, upstream end of reach | Upstream | |
| 16, or 16.1 | Outflow hydrograph | Outflow | Special Storage: Name of special storage |
| | Inflow hydrograph | Inflow | |
| 18 | Running hydrograph after translation | After | Hydrograph translation no. *nn* |
| | Running hydrograph before translation | Before | |
| 19 | Running hydrograph after inflow/outflow | Downstream | Concentrated channel Inflow* at location |
| | Concentrated inflow/outflow hydrograph | Inflow* | |
| | Running hydrograph, downstream end of reach | Downstream | Reach nn Lateral inflow*: Form of I/O |
| | Lateral inflow/outflow hydrograph for reach | Inflow* | |

*or Outflow as the case may be.

## 9.2 Description of Output

### 9.2.1 Option 'Flows only'

#### *9.2.1.1 Hydrograph Information*

The following items are printed, when relevant: *Peak discharge* (the maximum hydrograph ordinate); *Time to peak* (from the initial time); *Volume* (volume of hydrograph over duration of calculations); *Time to centroid* (initial time to time of hydrograph centroid); *Lag (c.m. to c.m.)* (time from centroid of concentrated inputs to channels upstream of the point concerned, including both sub-area inflows and concentrated channel inflow hydrographs, to time of centroid of hydrograph); *Lag to peak* (as above, but to peak instead of centroid of hydrograph).

Where a calculated hydrograph is being compared with an actual one (i.e. Control Code = 7.1), the errors in the above quantities are also printed, both as absolute values and as percentages. In addition, a useful overall measure of fit, the average absolute error in the calculated hydrograph ordinates over the duration of calculations, is printed. These error measures are for the purpose of judging the goodness of fit of the calculated to the actual hydrograph.

#### *9.2.1.2 Parameter Values*

These comprise the values of the coefficient $k_c$ and exponent $m$ for the catchment as a whole, if present, and also the loss parameters, if any. The loss parameters are the initial loss and either the pervious area runoff coefficient or the continuing loss rate according as Loss Model 0 or 1 has been selected. If there is more than one burst, loss parameters are printed for each. Also, if there is more than one interstation area, loss parameters are printed for each.

When present, as they usually are, parameter values are printed immediately above the first set of hydrograph information printed for each run. If the user elects, at the conclusion of a run, to run again with new parameters, the parameters are again printed to facilitate selection of the new values.

#### *9.2.1.3 Error Messages*

These include messages about erroneous or abnormal data items, warnings about abnormal parameter values that have been calculated or input from the terminal, and messages about non-converging iterative computations. For details of the latter two types of message, see Table 7-2.

#### *9.2.1.4 Hydrograph Tabulations*

All of the calculated hydrographs for which printouts have been requested are listed in tabular form at the end of the text file. This is to facilitate cut-and-paste of the information into other applications such as WORD and spreadsheet files.

### 9.2.2 Option ‘Flows & rainfall only’

This option comprises ‘Flows only’ output (see Section 9.2.1 above) plus the hyetographs used in the RORB run. These are tabulations of the rainfall and rainfall-excess hyetographs and totals for the catchment as a whole, for each sub-catchment, each interstation area, and for each sub-area.

If there is more than one interstation area (in a FIT run), separate tabulations are printed for each. In such cases, the sub-area hyetographs printed are for those sub-areas in the interstation area. For the purposes of this paragraph, the catchment outlet is treated as a gauging station if it is located downstream of the last gauging station. For rainfall but not rainfall-excess hyetographs, the reference number of the pluviograph used to define the temporal pattern for each sub-area is also given.

Sub-area rainfall hyetographs are not printed if the rainfall is uniform over the whole catchment. Similarly, sub-area rainfall-excess hyetographs are not printed if both rainfall and losses are uniform over the entire catchment. In such cases the sub-area hyetographs would be identical to the interstation area one.

If there is more than one burst of rainfall, separate tabulations are provided for each.

The hyetographs provided in this type of output can be a valuable aid in fitting the model to a catchment and in selecting a value of initial loss.

After an initial run has been completed and a re-run with new parameters is being done, the only hyetographs printed will be rainfall-excess hyetographs changed by the new loss parameters. Rainfalls and unchanged rainfall-excesses will not be printed again.

### 9.2.3 Option 'Hydrographs & all input data'

The output for this option comprises output as for "Flows & rainfall only' (Section 9.2.2 above) plus the data plus indicators of progress in reading and checking the data file.

The formatted listings of the file data include some calculated values as noted below, and are printed under various headings as follows:

Time data
Pluviograph data
Hydrograph data
Control Vector
  Each control code is accompanied by a Step Number and a verbal description of the function performed by that code.
Sub-area data
  In addition to the data input, total catchment area and average flow distance $d_{av}$ (see Section 2.2.2) are printed. Also, if there is more than one interstation area, the identifying numbers of the interstation area and the sub-catchment(s) of which each sub-area forms a part are printed.
Reach Storage data
  In addition to the data input, the relative delay time (see Section 2.2.2) is printed for each reach.
Special Storage data
Translation data
Channel Input and Output data

Data File Messages comprise information on the next data items to be read from file and checked and messages either that the data check has been completed, in which case no error has been found, or about the nature of any error found, with instructions for correcting it. These outputs facilitate checking and correction of the data file, especially if an error arises from some data file attribute that cannot be checked by the program. Hence this option is recommended with the running of all new data files.

## 9.3 Length of Output Line

All output fits on a line of seventy-two characters with the possible exceptions of the pluviograph and hydrograph data in the formatted listing of the data (Type 3 Output) and the sub-area rainfall and rainfall-excess hyetographs (Type 2 Output). The maximum output line length for these three items depends on the particular implementation; in the program as issued, it is 75. If the number of pluviographs, hydrographs, or sub-areas is too great for the relevant output to fit on a line of 75 characters, the data are printed in blocks that do not exceed that line length.

# 10 WORKED EXAMPLES

## 10.1 Introduction

This chapter presents four worked examples - a flood routing (with bank overflow) problem, a dam design problem, an urbanization application, and an illustration of Monte Carlo analysis - to show some of the capabilities of the program, the format of the data input, the screen windows, and the printed output.

It is recommended that users study each of the data files in conjunction with Table 5-2 to assist in the understanding of the required format; comments have been included in the data files to make them easier to follow. Further, the example runs of the program are interspersed with explanatory notes to help explain the strategy used, and copies of screen images. A printout of the output data file is included in each case; the hydrographs calculated during the program run is located at the end of this (for easy transfer to a word processing program or spreadsheet).

[It should be noted that many other examples of RORB applications can be found in the Australian technical literature, especially in the Proceedings of the Engineers Australia's Hydrology and Water Resources Symposia. It should also be noted that, although the worked examples in this chapter are relatively straightforward, much more complicated applications are possible (eg. see Ref. 22).]

## 10.2 Werribee River - Flood Routing Example

In this example it is required to compute the hydrograph of overbank flow and river return flow for a design flood (see Figure 10-1). The upstream and downstream hydrographs are known for a recorded flood (February 1973) and these hydrographs are used to calculate $k_c$ in a FIT run. In the second part of the example, the fitted $k_c$ is used to route the estimated 1 in 100 year design flood to a known overflow point in the stream. The split between river and overbank flow is calculated using a formula based on backwater calculations. Each hydrograph is printed and then routed to the downstream location (Werribee Weir), where one half of the overbank flow is assumed to re-enter the stream.

The example shows:

(i) the use of the model for flood routing (no sub-areas);
(ii) input of hydrographs;
(iii) calculation of a river over-bank flow using a formula;
(iv) routing of both river and overbank flows; and,
(v) return of flow to a river, i.e. rejoining branches.

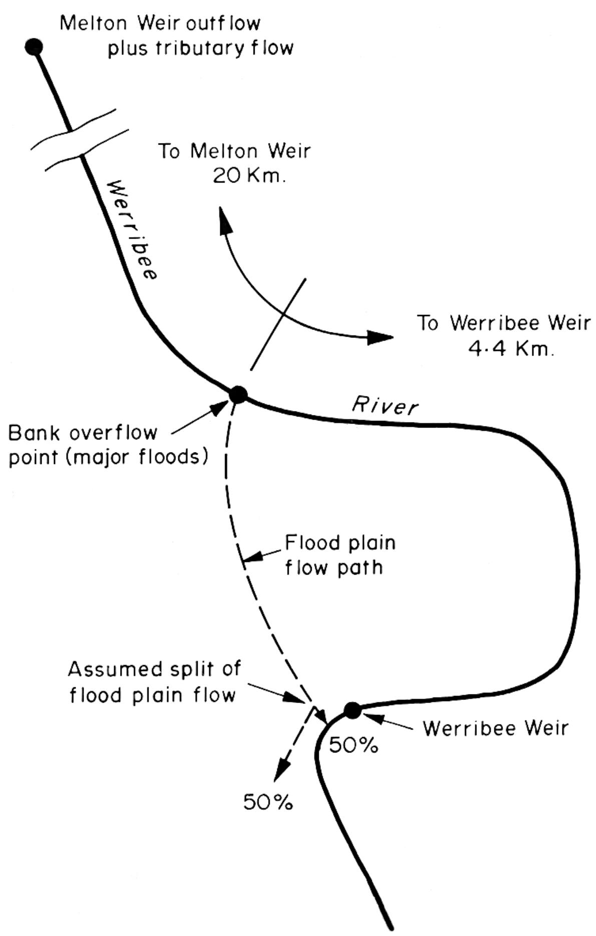



**Figure 10-1 Werribee River and Overbank Flow Network**

### 10.2.1 Werribee River Example, FIT Run

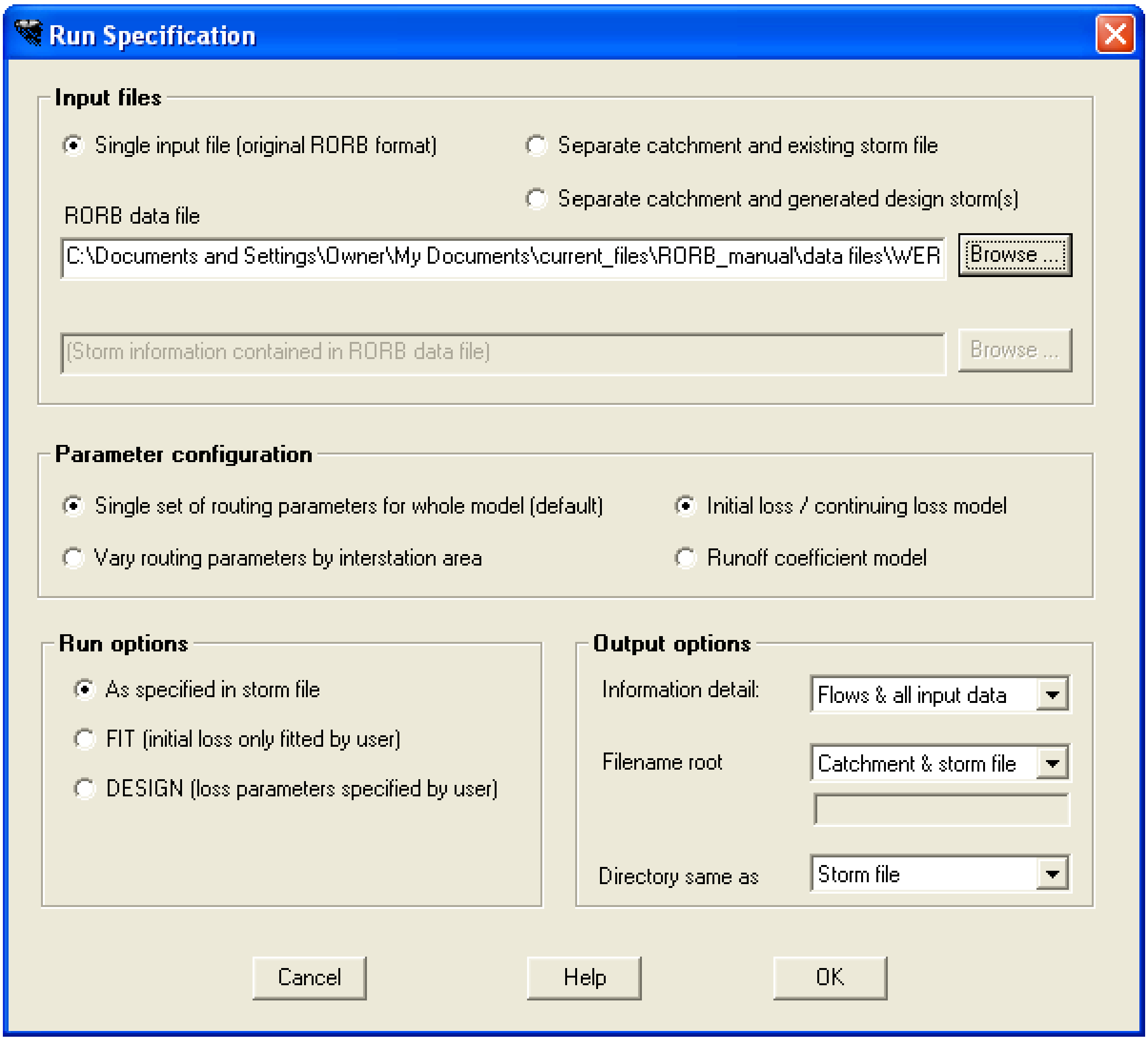
The data file required for the RORB model is prepared separately, according to Table 5-2. In this case it includes the lengths of the two reaches involved, and the observed upstream and downstream hydrographs.

#### *10.2.1.1 Werribee River FIT Run Data File (WERFIT.dat)*

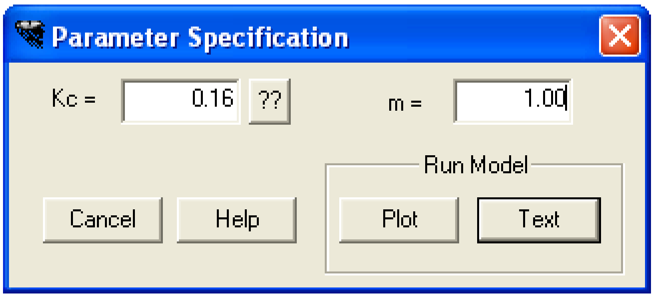
```
WERRIBEE RIVER Fit Run: Melton Reservoir to Werribee Weir
1,     Reaches in natural condition
C   Total inflow below Melton Res.
9,0,0,1,0,-99,      input h/g (data later in this file)
5,20,-99,           route to node at o'flow point (20 km)
5,4.4,-99,          route to Werribee Weir (4.4 km)
7.1,                compare with obs. h/g
0,                  end of control vector
1200 hrs 15 may 1974
FIT
2,28,-99,           time inc=2h, calcs for 28 incs
0,28,0,28,-99,      start & finish times for h/g's
Melton Res. Outflow (+ trib)
0,0,66,150,253,325,391,420,309,247,211,166,139,88
86,82,63,55,54,52,50,49,48,47,37,36,36,36,36,-99
Werribee Weir
0,0,8,34,64,147,245,310,356,330,290,245,216,185,150
122,104,96,90,76,68,62,59,57,55,53,50,42,36,-99
```

### *10.2.1.2 Werribee River FIT Run: Output*

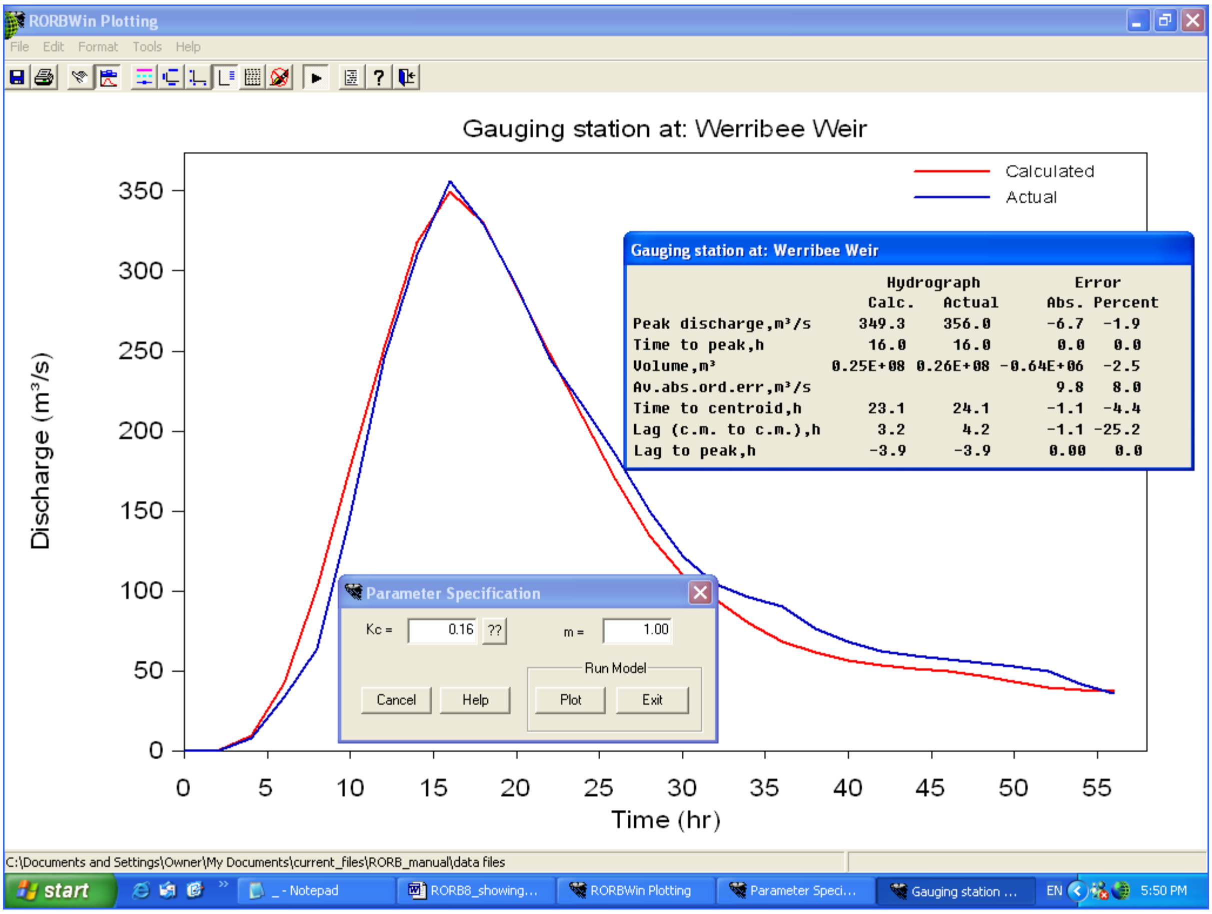
Click on the RORBWin icon (on the Windows desktop) to start the program, and then on ‘File’, ‘New’, and ‘Run specification’ to bring up the Run Specification screen. For this example, we have a single data file, so the program looks for a ‘.dat’ file when it prompts for the filename (Note: the location will be dependent on the user’s file structure). Other parameters on the screen (shown) are left as the default options.



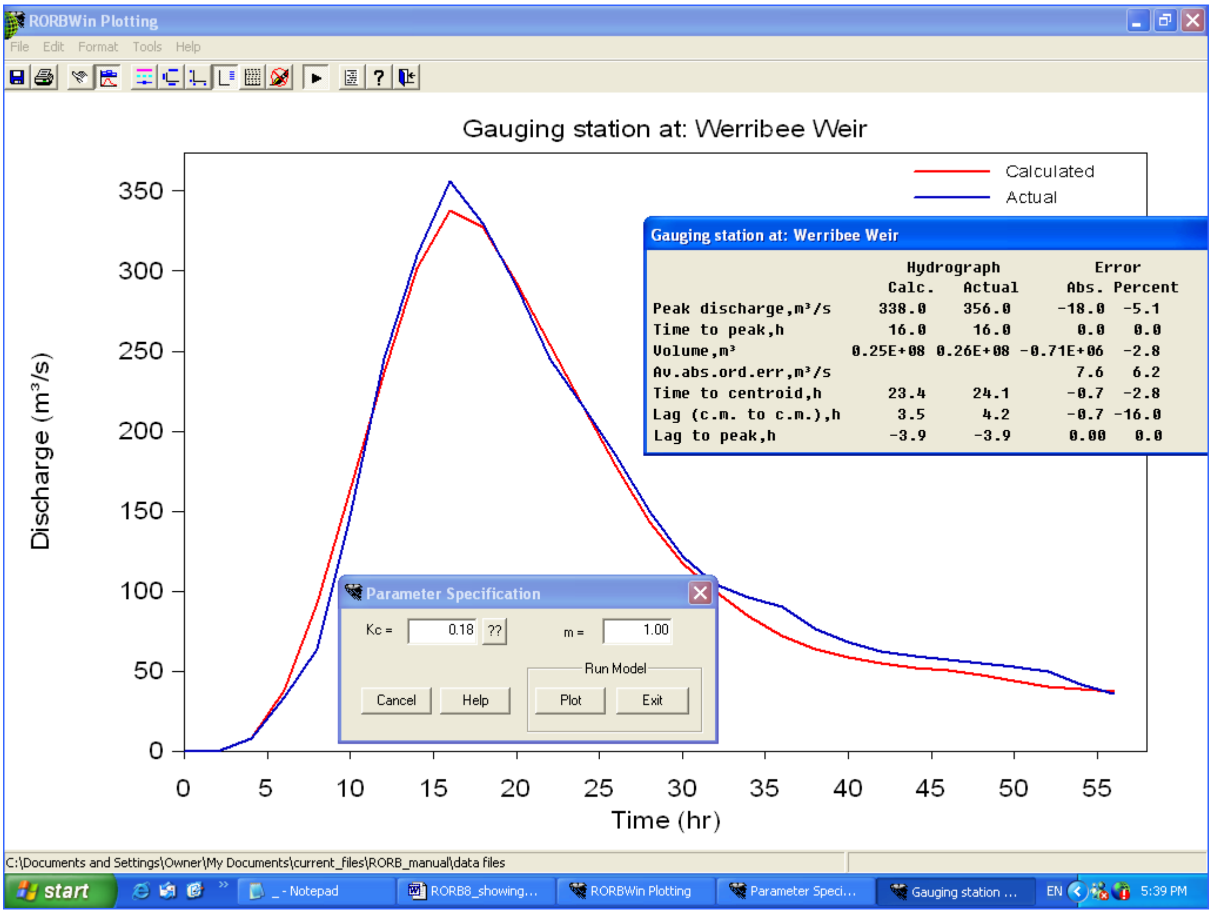

Selection of ‘OK’ moves the program on to the Parameter Specification window. Normally, pressing the ‘??’ button will give helpful values on starting values. However, these are for rainfall-runoff routing (rather than flood routing with no catchment involved) and so not applicable. A better suggestion will be found in the ‘Help’ screen. It enables use of an independent estimate of reach travel time of 4 hours. With a total reach length of 24.4 km, and a linear model (ie $m$ = 1), the product of $k_c$ and reach length will equal 4. Hence, starting values of $k_c$ of 4/24.4 (0.16) and $m$ = 1 are selected.




Selection of ‘Plot’ runs the model and displays the comparison of the observed and calculated hydrographs; the fit with these initial parameters is good. A right click of the mouse and selection of ‘Statistics’ will bring up quantitative measures of the goodness of fit, as shown below. (Also shown is the parameter screen, obtained by using the solid right arrow button on the toolbar).




The effect of changes in parameters can be examined by changing the parameter value and re-clicking the ‘Plot’ button. A (slightly) better overall fit – shown overleaf - is obtained with a $k_c$ value of 0.18, and this is the value chosen for the design run (next section).




A formatted text listing of the RORB run is obtained by selecting the appropriate button on the toolbar.

```
RORBWin Output File
 ******************

 Program version 6.00 (last updated 1st September 2007)
 Copyright Monash University and Sinclair Knight Merz

 Date run: 08 Oct 2007 10:12

 Data file          : C:\...\RORB_manual\data files\WERFIT.DAT
 Output information: Flows & all input data


 Data checks:
 ************
 Next data to be read & checked:

 Catchment name & reach type flag
 Control vector & storage data
 Initial storm data
 Hydrograph times
   2 hydrograph(s) expected in data
 Hydrograph  1
 Hydrograph  2

 Data check completed
```

```
Data:
****

WERRIBEE RIVER Fit Run: Melton Reservoir to Werribee Weir


Time data, in increments from initial time
Initial time:
1200 hrs 15 may 1974
Time increment (hours)=  2.00

                  Start   Finish
Hydrograph times:
  Hydrograph  1     0       28    Melton Res. Outflow (+ trib)
  Hydrograph  2     0       28    Werribee Weir

End of hyeto/hydrographs:   28
Duration of calculations:   28


Hydrograph data (time in incs, discharges in m³/s)

     1:Melton Res. Outflow (+ trib)
         I    2:Werribee Weir
Time     1        2
  0      0.0      0.0
  1      0.0      0.0
  2     66.0      8.0
  3    150.0     34.0
  4    253.0     64.0
  5    325.0    147.0
  6    391.0    245.0
  7    420.0    310.0
  8    309.0    356.0
  9    247.0    330.0
 10    211.0    290.0
 11    166.0    245.0
 12    139.0    216.0
 13     88.0    185.0
 14     86.0    150.0
 15     82.0    122.0
 16     63.0    104.0
 17     55.0     96.0
 18     54.0     90.0
 19     52.0     76.0
 20     50.0     68.0
 21     49.0     62.0
 22     48.0     59.0
 23     47.0     57.0
 24     37.0     55.0
 25     36.0     53.0
 26     36.0     50.0
 27     36.0     42.0
 28     36.0     36.0

Total volumes
 mm      0.0      0.0
 m³ 2.54E+07 2.56E+07


FIT    run control vector

Step  Code              Description
  1    9     Add concentrated inflow, Melton Res. Outflow (+ trib)
  2    5     Route hydrograph thru normal storage  1
  3    5     Route hydrograph thru normal storage  2
  4    7.1   Print results, Werribee Weir
  5    0     **********End of control vector**********
```

```
Normal storage data

Storage   Length   Rel. delay      Type         Slope
  no.       km*       time                      percent
  1        20.0      20.000       Natural
  2         4.4       4.400       Natural

* or other function of reach properties related to travel time


Channel inflow/outflow data

      Location or            I/O No. of    Inflow or outflow, m^3/s,=
      description                reaches

Melton Res. Outflow (+ trib)  I     0     see hydrograph data above


Input of parameters:
********************

WERRIBEE RIVER Fit Run: Melton Reservoir to Werribee Weir
FIT    Run
Initial time:
1200 hrs 15 may 1974
Time increment = 2.00 hours


Routing results:
****************
WERRIBEE RIVER Fit Run: Melton Reservoir to Werribee Weir
Initial time:
1200 hrs 15 may 1974
FIT    run no.  1


Parameters:  kc =     0.18    m = 1.00


*** Gauging station at: Werribee Weir

                          Hydrograph          Error
                         Calc.   Actual    Abs. Percent
Peak discharge,m^3/s     338.0    356.0    -18.0  -5.1
Time to peak,h            16.0     16.0      0.0   0.0
Volume,m^3            2.48E+07 2.56E+07-7.12E+05  -2.8
Av.abs.ord.err,m^3/s                         7.6   6.2 over dur. of calcs
Time to centroid,h        23.4     24.1     -0.7  -2.8
Lag (c.m. to c.m.),h      3.54     4.21    -0.67 -16.0
Lag to peak,h            -3.91    -3.91     0.00   0.0


Hydrograph summary
******************

Site  Description
  01  Gauging station at: Werribee Weir - Calculated
  02  Gauging station at: Werribee Weir - Actual

Inc    Time    Hyd001    Hyd002
  1    2.00     0.000     0.000
  2    4.00     0.000     0.000
  3    6.00     8.007     8.000
  4    8.00    37.806    34.000
  5   10.00    92.598    64.000
  6   12.00   163.077   147.000
  7   14.00   236.298   245.000
  8   16.00   302.075   310.000
  9   18.00   337.999   356.000
 10   20.00   327.515   330.000
 11   22.00   292.287   290.000
 12   24.00   254.062   245.000
```

```
  13   26.00   216.021   216.000
  14   28.00   178.231   185.000
  15   30.00   142.870   150.000
  16   32.00   117.351   122.000
  17   34.00   100.052   104.000
  18   36.00    84.542    96.000
  19   38.00    72.073    90.000
  20   40.00    64.000    76.000
  21   42.00    58.707    68.000
  22   44.00    54.950    62.000
  23   46.00    52.309    59.000
  24   48.00    50.392    57.000
  25   50.00    47.780    55.000
  26   52.00    43.812    53.000
  27   54.00    40.403    50.000
  28   56.00    38.490    42.000
  29   58.00    37.407    36.000
```

### 10.2.2 Werribee River Example DESIGN Run

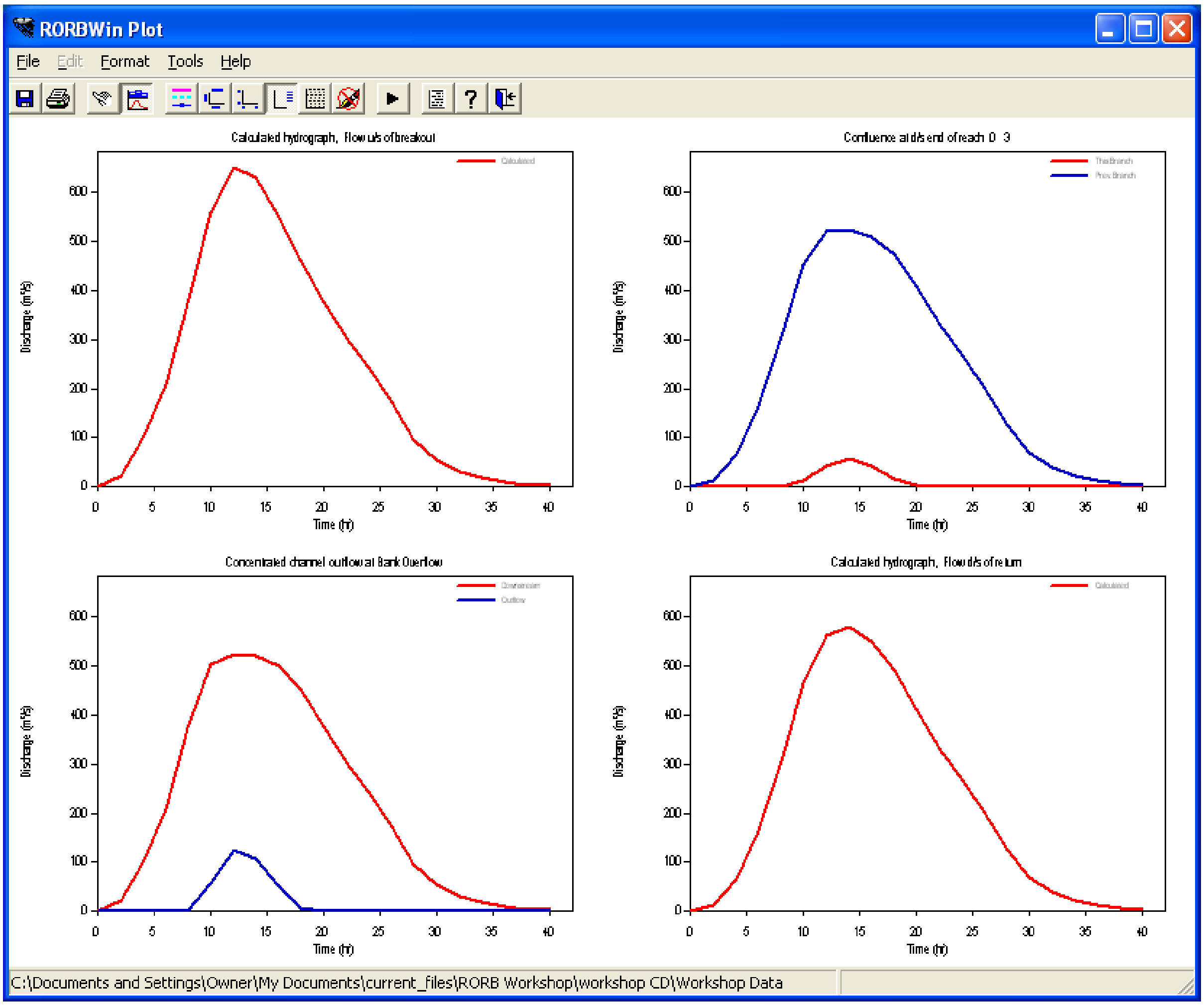
#### *10.2.2.1 Werribee River DESIGN Run Data File (WERDES.dat)*

```
WERRIBEE RIVER Design Run: Flood Overflow Calculation
1,     all reaches natural
9,0,0,1,0,-99,     input design flood
5,20,-99,     route to overflow point
7
Flow u/s of breakout
19,1,0,0,1,     calc overflow and print
Bank Overflow
0,425,0.03,1.54,-99,     parameters for o'bank flow
5,4.4,-99,     route river h/g to werribee weir
3,     store river h/g
C   Now route overbank flow
9,2,0,1,1,-99,     recover stored overflow h/g
5,6.6,-99,     route overland to werribee river
C   split overland flow: 50% back to river, 50% elsewhere
9,1,0,0,0
Non-return flow to river
0,0,0.5,1,-99,     parameters in formula for 50% split
14,     add to stored river flow and print
7
Flow d/s of return
0,     end of control vector
1 in 100 year design flood - Werribee River
DESIGN
2,20,-99
0,12,-99,     start and end times for inflow h/g
Melton Res. (+ trib) h/g
0,107,280,420,750,850,690,510,390,300,230,180,155,-99
```

#### *10.2.2.2 Werribee River, DESIGN Run: Output*

Proceed as for the FIT Run, but this time select the data file WERDES.dat, and input the parameters obtained in the FIT run (ie $k_C$ = 0.18, $m$ = 1.00). A click of the 'Plot' button on the Parameter Specification Window produces the screen below.

These four hydrograph plots show (i) the river upstream of the breakout, (ii) the flow that remains in the river (the larger h/g) and that which overflows, (ii) the return flow to the river (the smaller h/g) and the river flow just upstream of that point, and (iv) the river flow




downstream of the return flow. Note that right clicking on any graph gives the options to zoom in (ie show just that graph), zoom out (return to the screen showing both), and for the statistics of the fit. A left click on a graph at any point shows the numerical discharge and time values.

A click on the View Text button on the toolbar displays the formatted text output, including tabulated hydrograph values, thus:

```
RORBWin Output File
 ******************

 Program version 6.00 (last updated 1st September 2007)
 Copyright Monash University and Sinclair Knight Merz

 Date run: 04 Sep 2007 08:45

 Data file          : C:\Documents and Settings\Owner\My
Documents\current_files\RORB Workshop\workshop CD\Workshop Data\werdes.daT
 Output information: Flows & all input data

 Data checks:
 ************
 Next data to be read & checked:

 Catchment name & reach type flag
 Control vector & storage data
 Code no.    3      7.0 Location read as Flow u/s of breakout
 Code no.    4     19.0 Location read as Bank Overflow
```

```
Code no.   9     9.0 Location read as Non-return flow to river
Code no.  11     7.0 Location read as Flow d/s of return
Initial storm data
Hydrograph times
  1 hydrograph(s) expected in data
Hydrograph  1

Data check completed

Data:
****

WERRIBEE RIVER Design Run: Flood Overflow Calculation

Time data, in increments from initial time
1 in 100 year design flood - Werribee River
Time increment (hours)=  2.00

                  Start   Finish
Hydrograph times:   0       12

End of hyeto/hydrographs:   12
Duration of calculations:   20

Hydrograph data (time in incs, discharges in m³/s)

     1:Melton Res. (+ trib) h/g
Time     1
  0      0.0
  1    107.0
  2    280.0
  3    420.0
  4    750.0
  5    850.0
  6    690.0
  7    510.0
  8    390.0
  9    300.0
 10    230.0
 11    180.0
 12    155.0

Total volumes
 mm      0.0
 m³ 3.50E+07

DESIGN run control vector

Step  Code              Description
  1    9     Add concentrated inflow, Melton Res. (+ trib) h/g
  2    5     Route hydrograph thru normal storage  1
  3    7.0   Print hydrograph, Flow u/s of breakout
  4   19     Subtract concentrated outflow Bank Overflow
  5    5     Route hydrograph thru normal storage  2
  6    3     Store hydrograph from step   5; reset hydrograph to zero
  7    9     Add concentrated inflow, Bank Overflow
  8    5     Route hydrograph thru normal storage  3
  9    9     Subtract concentrated outflow Non-return flow to river
 10   14     Add h-graph ex step   6 to h-graph ex step   9
 11    7.0   Print hydrograph, Flow d/s of return
 12    0     **********End of control vector**********

Normal storage data

Storage   Length   Rel. delay      Type         Slope
  no.       km*       time                     percent
  1        20.0      20.000       Natural
  2         4.4       4.400       Natural
  3         6.6       6.600       Natural
* or other function of reach properties related to travel time
```

```
Channel inflow/outflow data

      Location or            I/O No. of    Inflow or outflow, m³/s,=
      description                reaches

Melton Res. (+ trib) h/g      I     0     see hydrograph data above
Bank Overflow                 O     0      0.00 + 0.03*(Q- 425.00)**1.54
Bank Overflow                 I     0     I/O for same location above
Non-return flow to river      O     0      0.00 + 0.50*(Q-   0.00)**1.00

Input of parameters:
********************

WERRIBEE RIVER Design Run: Flood Overflow Calculation
DESIGN Run
1 in 100 year design flood - Werribee River
Time increment = 2.00 hours

Routing results:
****************
WERRIBEE RIVER Design Run: Flood Overflow Calculation
1 in 100 year design flood - Werribee River
DESIGN run no.  1

Parameters:  kc =     0.18    m = 1.00

*** Calculated hydrograph,  Flow u/s of breakout

                          Hydrograph
                         Calc.
Peak discharge,m³/s     650.0
Time to peak,h           12.0
Volume,m³            3.50E+07
Time to centroid,h       15.4
Lag (c.m. to c.m.),h     3.58
Lag to peak,h           0.134

*** Concentrated channel outflow at Bank Overflow

                          Hydrograph
                       Downstr. Outflow
Peak discharge,m³/s     524.3    125.8
Time to peak,h           12.0     12.0
Volume,m³            3.25E+07 2.51E+06
Time to centroid,h       15.6     13.0
Lag (c.m. to c.m.),h     3.76     1.15
Lag to peak,h           0.134    0.134

*** Confluence at d/s end of reach  0   3

                          Hydrograph
                       This Br. Prev.br.
Peak discharge,m³/s      57.0    522.8
Time to peak,h           14.0     14.0
Volume,m³            1.25E+06 3.25E+07
Time to centroid,h       14.2     16.4
Lag (c.m. to c.m.),h     2.26     4.47
Lag to peak,h            2.06     2.06


*** Calculated hydrograph,  Flow d/s of return

                          Hydrograph
                         Calc.
Peak discharge,m³/s     579.7
Time to peak,h           14.0
Volume,m³            3.37E+07
Time to centroid,h       16.3
Lag (c.m. to c.m.),h     4.39
Lag to peak,h            2.06
```

```
Hydrograph summary
******************

Site  Description
  01  Calculated hydrograph,  Flow u/s of breakout
  02  Concentrated channel outflow at Bank Overflow - Downstream
  03  Concentrated channel outflow at Bank Overflow - Outflow
  04  Confluence at d/s end of reach  0   3 - This Branch
  05  Confluence at d/s end of reach  0   3 - Prev. Branch
  06  Calculated hydrograph,  Flow d/s of return

Inc    Time     Hyd001    Hyd002    Hyd003    Hyd004    Hyd005    Hyd006
  1    2.00      0.000     0.000     0.000    0.0000     0.000     0.000
  2    4.00     23.261    23.261     0.000    0.0000    12.980    12.980
  3    6.00     97.278    97.278     0.000    0.0000    65.758    65.758
  4    8.00    207.157   207.157     0.000    0.0000   162.253   162.253
  5   10.00    371.437   371.437     0.000    0.0000   304.043   304.043
  6   12.00    557.768   501.962    55.807   12.7529   452.097   464.850
  7   14.00    650.043   524.263   125.780   42.5918   520.195   562.786
  8   16.00    628.285   520.737   107.548   56.9796   522.768   579.747
  9   18.00    550.770   499.428    51.342   41.2053   508.610   549.815
 10   20.00    461.305   453.728     7.576   17.0044   472.860   489.865
 11   22.00    375.955   375.955     0.000    3.1924   408.107   411.300
 12   24.00    301.627   301.627     0.000    0.2743   330.745   331.019
 13   26.00    243.311   243.311     0.000    0.0236   265.704   265.728
 14   28.00    171.219   171.219     0.000    0.0020   200.482   200.484
 15   30.00     96.776    96.776     0.000    0.0002   126.281   126.281
 16   32.00     54.699    54.699     0.000    0.0000    69.871    69.871
 17   34.00     30.917    30.917     0.000    0.0000    39.667    39.667
 18   36.00     17.475    17.475     0.000    0.0000    22.400    22.400
 19   38.00      9.877     9.877     0.000    0.0000    12.663    12.663
 20   40.00      5.583     5.583     0.000    0.0000     7.157     7.157
 21   42.00      3.155     3.155     0.000    0.0000     4.045     4.045
```

## 10.3 Thomson River - Dam Design Example

This example illustrates the application of the model to assist in the determination of the spillway design flood for a major dam.

In the first part, the parameters $k_c$ and $m$ are determined for the catchment in a FIT run. Subdivision is as shown in Figure 10-2. Note the provision for subsequent drowning of reaches by the dam (when full). Hydrograph data are included for two gauging stations and three pluviographs for the November 1971 event. Baseflow is included.

For a design inflow to the dam corresponding to a given storm duration, a design storm is used in conjunction with conservative loss values. $k_c$ and $m$ are as determined in a FIT run as explained above (and confirmed by test runs on independent storms not illustrated here). The reaches drowned by the dam are given a reach type code = 4. The resulting inflow hydrograph is printed and then routed through the reservoir using a hypothetical spillway configuration. [An alternative use of the program would be to interactively design the spillway length and crest height, but that has not been done here.]

The example shows:

(i) the interstation areas resulting from more than one gauging station - hence different loss rates;
(ii) printout of the input from a sub-area;
(iii) how baseflow can be included in the model;
(iv) how to meet the requirement that the channel network and catchment area are not changed from the FIT run to the DESIGN run. This is necessary if the same $k_c$ is to be

used on the smaller catchment (i.e. to the dam site rather than further downstream to the The Narrows). It is achieved by using 'drowned' reaches and including the sub-area downstream of the dam. [Note that the very conservative loss values are sufficient to model the rainfall on the "impervious" surface of the reservoir in this case.]

(v) routing a design flood through a reservoir.

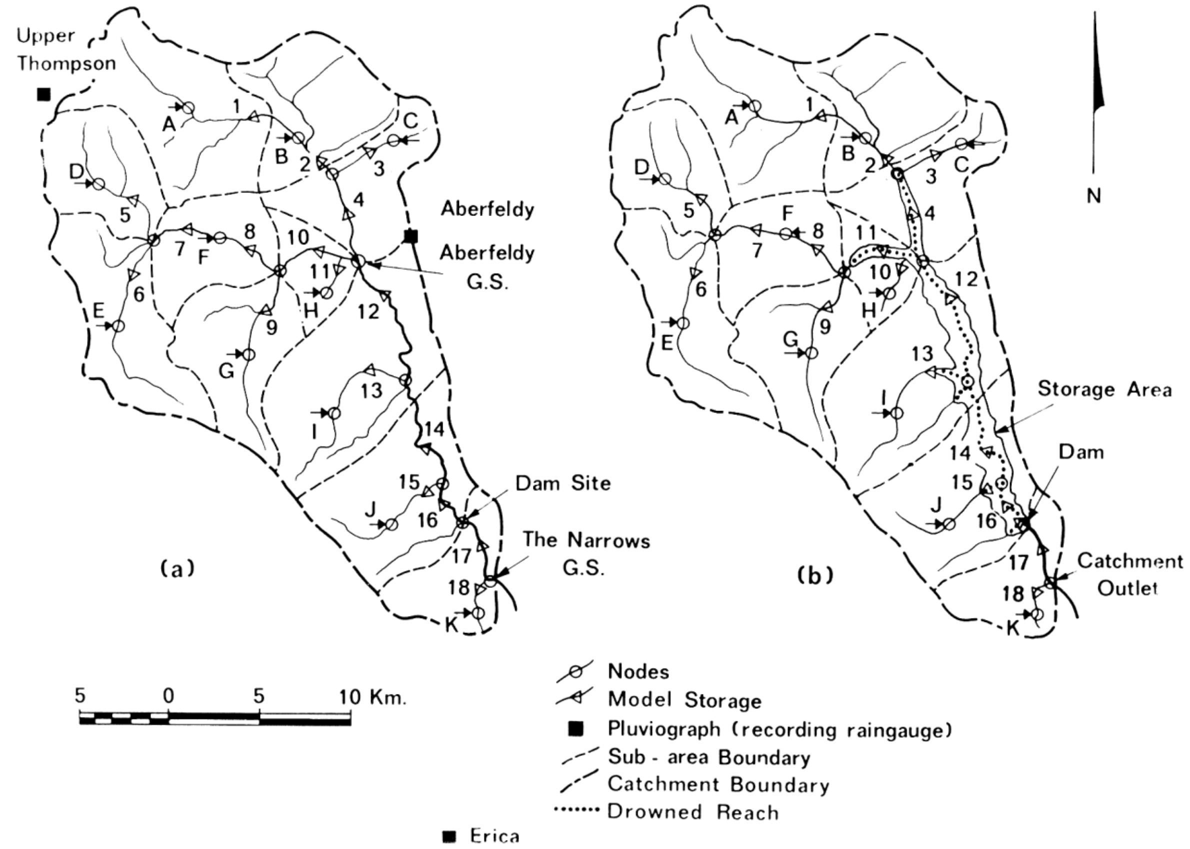



**Figure 10-2 Thomson River Catchment (a) Pre-dam, (b) Post-dam**

### 10.3.1 Thomson River Example, FIT Run

This example shows the use of two data files; the catchment data (TOMFIT.cat) and the storm data (in this case TOMNOV71.stm). When several storms are being used for fitting and testing, this arrangement avoids the repetition of the catchments data. Note the arbitrary printout specified at sub-area F to show the use of a Code 12 (no special significance is attached to Sub-area F).

#### *10.3.1.1 Thomson River FIT Run: Catchment Data File (TOMFIT.cat)*

```
Thomson River at the Narrows
1,     channel type flag - all reaches natural
9,0,11,1,0,-99,     distributed baseflow input
1,6.5,-99,     sub-area A (6.5km)
2,2.5,-99,     sub-area B
3,     store h/g.
1,2.5,-99,     sub-area C
4,     add to A&B
5,5.5,-99,     route to Aberfeldy
```

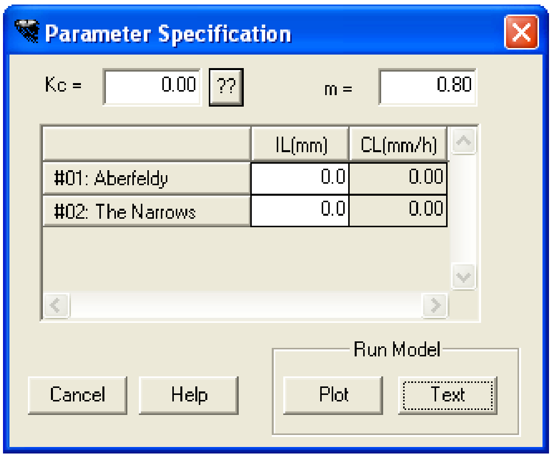
```
3,     store h/g (Aberfeldy)
1,5.0,-99,     sub-area D
3,     store
1,5.0,-99,     sub-area E
4,     add h/g's from D&E
5,4.0,-99,     route to F
12,4.0,-99,     sub-area F  (print input & running h/g's)
3
1,5.0,-99,     sub-area G
4
5,5.0,-99,     route to Aberfeldy
4,     add to h/g at Aberfeldy
3,     store sum
1,2.5,-99,     sub-area H
4
7.1,     now at Aberfeldy gauging station
9,1,-1,1,0,     baseflow for interstation area d/s of Aberfeldy
Baseflow d/s of Aberfeldy
2.5,0,0,0,-99
5,7.0,-99
3
1,4.0,-99,     sub-area I
4
5,7.5,-99
3
1,3.0,-99,     sub-area J
4
5,3.0,-99,     route to dam site
5,5.0,-99,     to Narrows
3
1,2.5,-99,     sub-area K
4,     add to h/g at Narrows
7.1,     now at The Narrows
0,     end of control vector
C   sub-area areas
62,42,44,59,41,31,44,21,80,67,28,-99,     sq.km.
0,-99,     all areas pervious
```

### 10.3.1.2 Thomson River FIT Run: Storm Data File (TOMNOV71.stm)

```
0900 hours on 6 November 1971
Fit
C   time inc=6h, calcs for 24 time incs, 1 burst,
C   3 pluvis, non uniform pattern
6.,24,1,3,1,-99
0,8,     rain from 0-8 time incs
Upper Thomson
30,7,4,3,7,20,15,3,-99
Aberfeldy
23,4,3,4,11,12,10,4,-99
Erica
17,3,4,18,10,26,17,9,-99
C   sub area rainfall totals (from isohyetal map)
70,50,60,125,100,70,90,65,80,90,100,-99
C   pluvi ref numbers
1,2,2,1,1,2,2,2,2,2,3,-99
C   hydrograph data
0,24,0,24,0,24,-99,     start and end times
Baseflow to Aberfeldy
10.5,10.5,10.5,10.5,10.5,10.5,10.5,10.5,11,12,13,14,15,16,17,18,19,20,21
22,22,21,20.5,20,20,-99
Aberfeldy
10.5,18,19,19,20,38,70,86,74,63,50,42,36,34,31,30,27,26,24,23,
22,21,20.5,20,20,-99
The Narrows
13,14,18,21,24,30,60,103,103,97,86,76,67,57,52,48,44,42,39,36,33,30,
28,26,24,-99
```

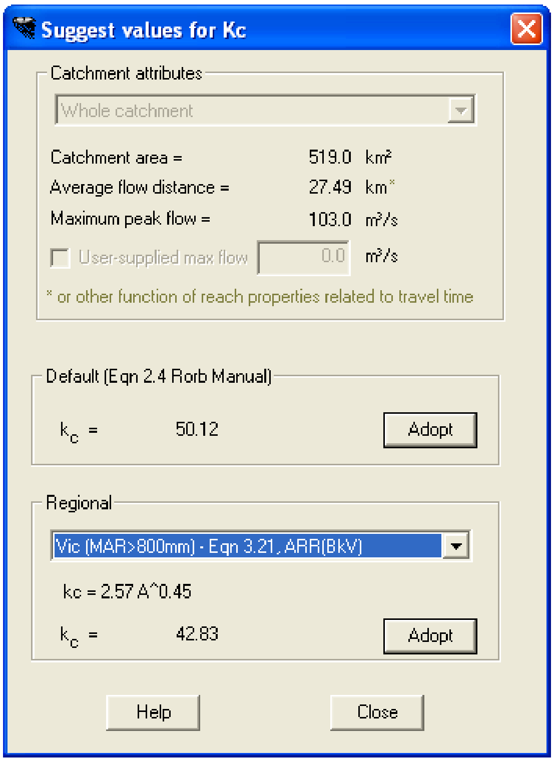
### *10.3.1.3 Thomson River FIT Run: Output*

Click on the RORBWin icon to start the program, and then on ‘File’, ‘New’, and ‘Run specification’ to bring up the Run Specification screen. For this example, there are two data files; when this option is specified, the program looks for a ‘.cat’ file when it prompts for the filename of catchment data, and for a ‘.stm’ file for the storm data (Note: these are only defaults and the user can specify other filetypes as required). Other parameters (ie loss model etc) on the screen are left as the default options for this example. Pressing the ‘OK’ button brings up the Parameter Specification window:



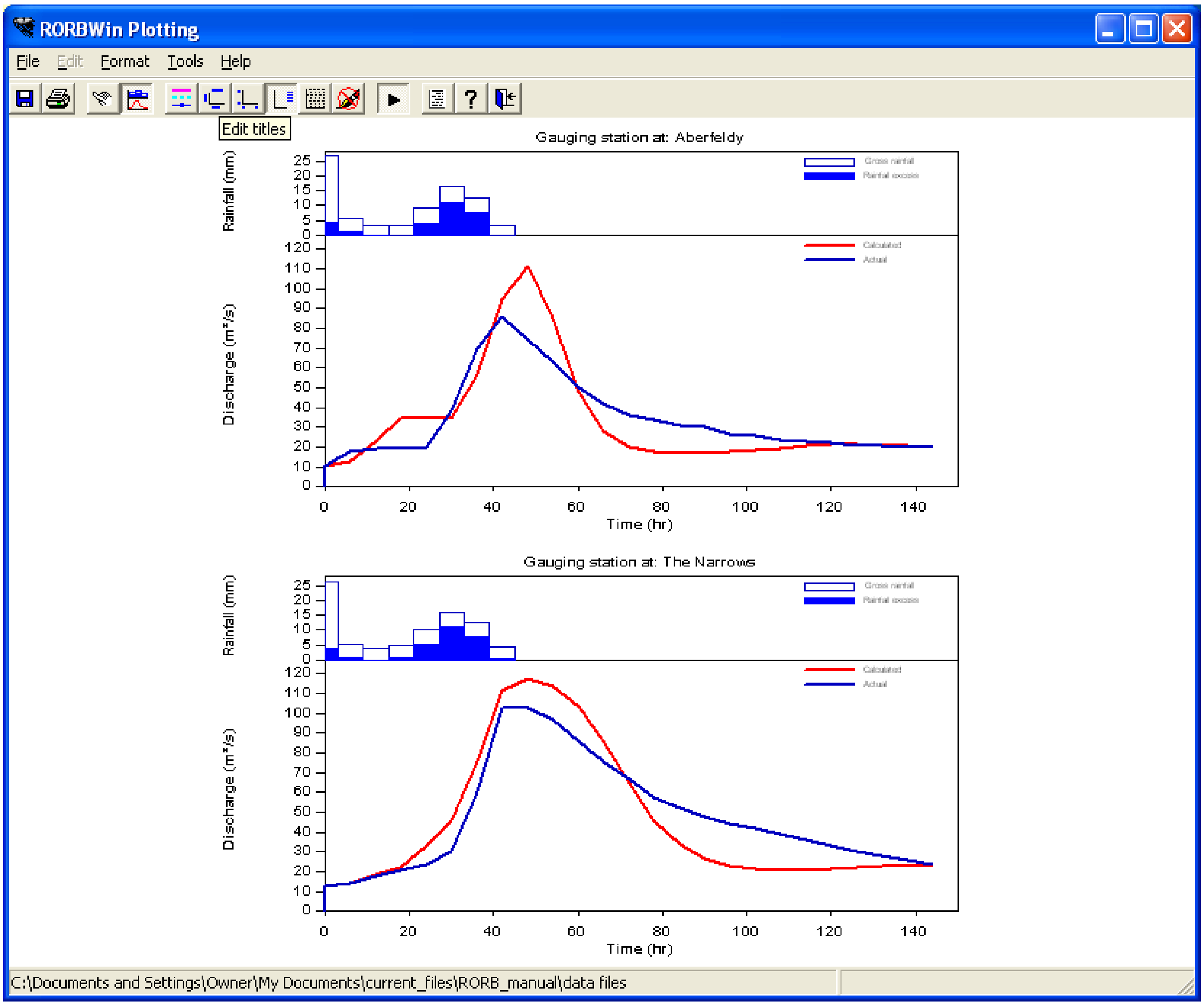

This dialogue shows the requirement to specify $k_C$ and $m$, and two values of initial loss (one for each interstation area. (Note the continuing loss values are calculated by the program for FIT runs, to match the observed hydrograph volumes).

In this case, the user can obtain a starting $k_C$ value (leaving $m$ at the default 0.8) by pressing the ‘??’ button.




Here, we have used the default option from the RORB manual ($k_C$ = 50.12) by pressing the appropriate ‘Adopt’ button. Initial loss values of 20 mm for each interstation area were selected (the program then computes continuing loss values of 0.85 and 0.77 mm/h respectively).

Selection of ‘Plot’ brings up the four hydrographs specified in the data file. Two of these are spurious as regards fitting (they were included to show the capability of the program), and can be removed by clicking on “File” and ‘Select Plots’. This leaves the plot overleaf showing on the screen. Clicking on the solid arrow button (fourth from the right on the toolbar) displays the Parameter Specification window, allowing real time fitting to take place. Various combinations of parameters are possible, depending on the emphasis the user puts on particular measures. It might be noted that the Thomson Baseflow hydrograph was an assumed one (for this example) and masks the fit. We chose a $k_C$ value of 65 (m = 0.8), with initial losses of 20 mm each as being best for this example.




The text output file (click on the View Text button on the toolbar) is reproduced below. Note the opportunity to examine rainfall and rainfall-excess patterns on individual sub-areas.

```
RORBWin Output File
 ******************

 Program version 6.00 (last updated 1st September 2007)
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 Date run: 08 Oct 2007 10:25

 Vector file        : C:\...\RORB_manual\data files\TOMFIT.CAT
 Storm file         : C:\...\RORB_manual\data files\TOMNOV71.STM
 Output information: Flows & all input data


 Data checks:
 ************
 Next data to be read & checked:

 Catchment name & reach type flag
 Control vector & storage data
 Code no.   24    9.0 Location read as Baseflow d/s of Aberfeldy
 Sub-area areas
 Impervious flag
 Initial storm data
 Rainfall burst times
 Pluviograph 1
```

```
Pluviograph 2
Pluviograph 3
Sub-area rainfalls
Pluviograph reference nos.
Hydrograph times
  3 hydrograph(s) expected in data
Hydrograph  1
Hydrograph  2
Hydrograph  3

Data check completed


Data:
****

Thomson River at the Narrows

Time data, in increments from initial time
Initial time:
0900 hours on 6 November 1971
Time increment (hours)=  6.00

                  Start   Finish
Rainfall times:     0        8

Hydrograph times:
  Hydrograph  1     0       24    Baseflow to Aberfeldy
  Hydrograph  2     0       24    Aberfeldy
  Hydrograph  3     0       24    The Narrows

End of hyeto/hydrographs:   24
Duration of calculations:   24


Pluviograph data (time in incs, rainfall in mm, in
                  increment following time shown)

        1:Upper Thomson
           I  2:Aberfeldy
           I     I  3:Erica
  Time     1     2     3
    0    30.0  23.0  17.0
    1     7.0   4.0   3.0
    2     4.0   3.0   4.0
    3     3.0   4.0  18.0
    4     7.0  11.0  10.0
    5    20.0  12.0  26.0
    6    15.0  10.0  17.0
    7     3.0   4.0   9.0

  Total  89.0  71.0 104.0


Hydrograph data (time in incs, discharges in m³/s)

     1:Baseflow to Aberfeldy
         I    2:Aberfeldy
         I        I    3:The Narrows
Time     1        2        3
  0     10.5     10.5     13.0
  1     10.5     18.0     14.0
  2     10.5     19.0     18.0
  3     10.5     19.0     21.0
  4     10.5     20.0     24.0
  5     10.5     38.0     30.0
  6     10.5     70.0     60.0
  7     10.5     86.0    103.0
  8     11.0     74.0    103.0
  9     12.0     63.0     97.0
 10     13.0     50.0     86.0
 11     14.0     42.0     76.0
```

```
 12     15.0     36.0     67.0
 13     16.0     34.0     57.0
 14     17.0     31.0     52.0
 15     18.0     30.0     48.0
 16     19.0     27.0     44.0
 17     20.0     26.0     42.0
 18     21.0     24.0     39.0
 19     22.0     23.0     36.0
 20     22.0     22.0     33.0
 21     21.0     21.0     30.0
 22     20.5     20.5     28.0
 23     20.0     20.0     26.0
 24     20.0     20.0     24.0

Total volumes
 mm      0.0     53.0     48.7
 m³ 8.33E+06 1.82E+07 2.53E+07


Fit    run control vector

Step  Code              Description
  1    9     Start distributed inflow (Baseflow to Aberfeldy       )
  2    1     Add sub-area 'A' inflow & route thru normal storage   1
  3    2     Add sub-area 'B' inflow & route thru normal storage   2
  4    3     Store hydrograph from step   3; reset hydrograph to zero
  5    1     Add sub-area 'C' inflow & route thru normal storage   3
  6    4     Add h-graph ex step   4 to h-graph ex step   5
  7    5     Route hydrograph thru normal storage  4
  8    3     Store hydrograph from step   7; reset hydrograph to zero
  9    1     Add sub-area 'D' inflow & route thru normal storage   5
 10    3     Store hydrograph from step   9; reset hydrograph to zero
 11    1     Add sub-area 'E' inflow & route thru normal storage   6
 12    4     Add h-graph ex step  10 to h-graph ex step  11
 13    5     Route hydrograph thru normal storage  7
 14   12     Add sub-area 'F' inflow & route thru normal storage   8
 15    3     Store hydrograph from step  14; reset hydrograph to zero
 16    1     Add sub-area 'G' inflow & route thru normal storage   9
 17    4     Add h-graph ex step  15 to h-graph ex step  16
 18    5     Route hydrograph thru normal storage 10
 19    4     Add h-graph ex step   8 to h-graph ex step  18
 20    3     Store hydrograph from step  19; reset hydrograph to zero
 21    1     Add sub-area 'H' inflow & route thru normal storage  11
             End distributed inflow
 22    4     Add h-graph ex step  20 to h-graph ex step  21
 23    7.1   Print results, Aberfeldy
 24    9     Start distributed inflow (Baseflow d/s of Aberfeldy    )
 25    5     Route hydrograph thru normal storage 12
 26    3     Store hydrograph from step  25; reset hydrograph to zero
 27    1     Add sub-area 'I' inflow & route thru normal storage  13
 28    4     Add h-graph ex step  26 to h-graph ex step  27
 29    5     Route hydrograph thru normal storage 14
 30    3     Store hydrograph from step  29; reset hydrograph to zero
 31    1     Add sub-area 'J' inflow & route thru normal storage  15
 32    4     Add h-graph ex step  30 to h-graph ex step  31
 33    5     Route hydrograph thru normal storage 16
 34    5     Route hydrograph thru normal storage 17
 35    3     Store hydrograph from step  34; reset hydrograph to zero
 36    1     Add sub-area 'K' inflow & route thru normal storage  18
             End distributed inflow
 37    4     Add h-graph ex step  35 to h-graph ex step  36
 38    7.1   Print results, The Narrows
 39    0     **********End of control vector**********


Sub-area data

Sub-     Area      Dist.     Intersta-  Part of sub-
area      km²       km*      tion area  catchment(s)

  A    6.20E+01  3.70E+01        1      1 2
  B    4.20E+01  3.05E+01        1      1 2
```

```
   C    4.40E+01  3.05E+01        1      1 2
   D    5.90E+01  4.05E+01        1      1 2
   E    4.10E+01  4.05E+01        1      1 2
   F    3.10E+01  3.15E+01        1      1 2
   G    4.40E+01  3.25E+01        1      1 2
   H    2.10E+01  2.50E+01        1      1 2
   I    8.00E+01  1.95E+01        2      2
   J    6.70E+01  1.10E+01        2      2
   K    2.80E+01  2.50E+00        2      2

 Total 5.190E+02

 For whole catchment     ; Av. Dist., km* =      27.49
 For interstation area  1; Av. Dist., km* =      12.09; ISA Factor =   2.274
 For interstation area  2; Av. Dist., km* =      13.53; ISA Factor =   2.032

 * or other function of reach properties related to travel time


 Normal storage data

 Storage   Length   Rel. delay      Type         Slope
   no.       km*       time                     percent
   1         6.5       0.236       Natural
   2         2.5       0.091       Natural
   3         2.5       0.091       Natural
   4         5.5       0.200       Natural
   5         5.0       0.182       Natural
   6         5.0       0.182       Natural
   7         4.0       0.146       Natural
   8         4.0       0.146       Natural
   9         5.0       0.182       Natural
  10         5.0       0.182       Natural
  11         2.5       0.091       Natural
  12         7.0       0.255       Natural
  13         4.0       0.146       Natural
  14         7.5       0.273       Natural
  15         3.0       0.109       Natural
  16         3.0       0.109       Natural
  17         5.0       0.182       Natural
  18         2.5       0.091       Natural

 * or other function of reach properties related to travel time


 Channel inflow/outflow data

       Location or             I/O No. of    Inflow or outflow, m³/s,=
       description                 reaches

 Baseflow to Aberfeldy          I    11     see hydrograph data above
 Baseflow d/s of Aberfeldy      I     7      2.50 + 0.00*(Q-   0.00)**0.00


 Input of parameters:
 ********************

 Thomson River at the Narrows
 Fit    Run
 Initial time:
 0900 hours on 6 November 1971
 Time increment = 6.00 hours

 Constant loss model selected


 Rainfall, mm, in time inc. following time shown
 interstation area  1, above Aberfeldy
 Time Sub-  Inter   Sub-
      catch stn.    area
 Incs ment  area    A   B   C   D   E   F   G   H
```

```
   0  27.0  27.0   24  16  19  42  34  23  29  21
   1   5.6   5.6    6   3   3  10   8   4   5   4
   2   3.6   3.6    3   2   3   6   4   3   4   3
   3   3.6   3.6    2   3   3   4   3   4   5   4
   4   9.1   9.1    6   8   9  10   8  11  14  10
   5  16.3  16.3   16   8  10  28  22  12  15  11
   6  12.8  12.8   12   7   8  21  17  10  13   9
   7   3.6   3.6    2   3   3   4   3   4   5   4

Tot. 81.5  81.5   70  50  60 125 100  70  90  65
Pluvi. ref. no.    1   2   2   1   1   2   2   2

Continuing loss rate =   0.85 mm/h


Rainfall-excess, mm, in time inc. following time shown
interstation area  1, above Aberfeldy
Time Sub-  Inter   Sub-
     catch stn.    area
Incs ment  area     A   B   C   D   E   F   G   H

   0   4.5   4.5    0   0   0  17   9   0   4   0
   1   1.2   1.2    0   0   0   5   3   0   0   0
   2   0.1   0.1    0   0   0   1   0   0   0   0
   3   0.0   0.0    0   0   0   0   0   0   0   0
   4   4.0   4.0    0   3   4   5   3   6   9   5
   5  11.3  11.3   11   3   5  23  17   7  10   6
   6   7.7   7.7    7   2   3  16  12   5   8   4
   7   0.0   0.0    0   0   0   0   0   0   0   0

Tot. 28.8  28.8   18   8  13  66  43  17  31  15


Rainfall, mm, in time inc. following time shown
interstation area  2, above The Narrows
Time Sub-  Inter   Sub-
     catch stn.    area
Incs ment  area     I   J   K

   0  26.5  25.6   26  29  16
   1   5.2   4.5    5   5   3
   2   3.6   3.6    3   4   4
   3   4.6   6.8    5   5  17
   4  10.3  12.5   12  14  10
   5  16.2  16.0   14  15  25
   6  12.7  12.6   11  13  16
   7   4.2   5.4    5   5   9

Tot. 83.4  87.0   80  90 100
Pluvi. ref. no.    2   2   3

Continuing loss rate =   0.77 mm/h


Rainfall-excess, mm, in time inc. following time shown
interstation area  2, above The Narrows
Time Sub-  Inter   Sub-
     catch stn.    area
Incs ment  area     I   J   K

   0   3.8   2.3    1   5   0
   1   0.9   0.2    0   0   0
   2   0.1   0.0    0   0   0
   3   0.7   2.2    0   0  13
   4   5.4   7.9    8   9   5
   5  11.3  11.4    9  11  20
   6   7.8   8.0    7   8  12
   7   0.3   0.8    0   0   4

Tot. 30.2  33.0   25  34  54
```

```
Routing results:
****************
Thomson River at the Narrows
Initial time:
0900 hours on 6 November 1971
Fit    run no.  1


Parameters:  kc =    65.00    m = 0.80

Loss parameters     Initial loss (mm)   Cont. loss (mm/h)
  Interstation area above:
  Aberfeldy              20.00              0.85
  The Narrows            20.00              0.77


*** Sub-area inflow point, sub-area F

                          Hydrograph
                       Upstr'm  Subarea
Peak discharge,m³/s      59.85     9.69
Time to peak,h            48.0     36.0
Volume,m³             8.04E+06 5.36E+05
Time to centroid,h        52.2     35.7
Lag (c.m. to c.m.),h      26.4      9.9
Lag to peak,h             22.2     10.2


*** Normal storage, reach no.   0   8

                          Hydrograph
                       Downstr. Upstream
Peak discharge,m³/s      61.66    63.54
Time to peak,h            48.0     42.0
Volume,m³             9.21E+06 8.58E+06
Time to centroid,h        56.4     51.2
Lag (c.m. to c.m.),h      30.6     25.4
Lag to peak,h             22.2     16.2


*** Gauging station at: Aberfeldy

                          Hydrograph          Error
                         Calc.   Actual    Abs. Percent
Peak discharge,m³/s      96.51    86.00    10.51  12.2
Time to peak,h            48.0     42.0      6.0  14.3
Volume,m³             1.78E+07 1.82E+07-4.42E+05  -2.4
Av.abs.ord.err,m³/s                          7.2  21.4 over dur. of calcs
Time to centroid,h        64.0     66.1     -2.1  -3.2
Lag (c.m. to c.m.),h      36.0     38.1     -2.1  -5.6
Lag to peak,h             20.0     14.0      6.0  42.9


*** Gauging station at: The Narrows

                          Hydrograph          Error
                         Calc.   Actual    Abs. Percent
Peak discharge,m³/s       98.8    103.0     -4.2  -4.1
Time to peak,h            48.0     42.0      6.0  14.3
Volume,m³             2.40E+07 2.53E+07-1.29E+06  -5.1
Av.abs.ord.err,m³/s                          6.2  13.2 over dur. of calcs
Time to centroid,h        67.3     70.2     -2.9  -4.2
Lag (c.m. to c.m.),h      38.5     41.4     -2.9  -7.1
Lag to peak,h             19.2     13.2      6.0  45.6


Hydrograph summary
******************

Site  Description
  01  Sub-area inflow point, sub-area F - Upstream
  02  Sub-area inflow point, sub-area F - Subarea
```

```
   03  Normal storage, reach no.   0   8 - Downstream
   04  Normal storage, reach no.   0   8 - Upstream
   05  Gauging station at: Aberfeldy - Calculated
   06  Gauging station at: Aberfeldy - Actual
   07  Gauging station at: The Narrows - Calculated
   08  Gauging station at: The Narrows - Actual
```

| Inc | Time | Hyd001 | Hyd002 | Hyd003 | Hyd004 | Hyd005 | Hyd006 | Hyd007 | Hyd008 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 6.00 | 3.0947 | 0.0000 | 3.9789 | 3.0947 | 10.5000 | 10.5000 | 13.0000 | 13.000 |
| 2 | 12.00 | 9.9074 | 0.0000 | 6.3956 | 9.9074 | 11.9361 | 18.0000 | 13.9463 | 14.000 |
| 3 | 18.00 | 26.1826 | 0.0000 | 16.1392 | 26.1826 | 17.7076 | 19.0000 | 16.8434 | 18.000 |
| 4 | 24.00 | 28.3793 | 0.0000 | 26.2513 | 28.3793 | 26.6612 | 19.0000 | 19.5211 | 21.000 |
| 5 | 30.00 | 16.9986 | 0.0000 | 23.9618 | 16.9986 | 31.3257 | 20.0000 | 27.5737 | 24.000 |
| 6 | 36.00 | 11.7546 | 8.2791 | 20.0801 | 20.0338 | 35.2750 | 38.0000 | 36.4624 | 30.000 |
| 7 | 42.00 | 25.2677 | 9.6941 | 27.1692 | 34.9618 | 51.6576 | 70.0000 | 58.4134 | 60.000 |
| 8 | 48.00 | 56.6722 | 6.8642 | 47.7191 | 63.5364 | 78.7037 | 86.0000 | 89.8124 | 103.000 |
| 9 | 54.00 | 59.8455 | 0.0000 | 61.6630 | 59.8455 | 96.5072 | 74.0000 | 98.8143 | 103.000 |
| 10 | 60.00 | 32.9716 | 0.0000 | 48.3060 | 32.9716 | 87.4270 | 63.0000 | 97.6265 | 97.000 |
| 11 | 66.00 | 15.4142 | 0.0000 | 27.6101 | 15.4142 | 62.3782 | 50.0000 | 93.3844 | 86.000 |
| 12 | 72.00 | 9.1674 | 0.0000 | 15.6930 | 9.1674 | 41.3087 | 42.0000 | 87.8557 | 76.000 |
| 13 | 78.00 | 6.6681 | 0.0000 | 10.4917 | 6.6681 | 29.0769 | 36.0000 | 77.6562 | 67.000 |
| 14 | 84.00 | 5.6640 | 0.0000 | 8.2181 | 5.6640 | 22.8784 | 34.0000 | 64.0733 | 57.000 |
| 15 | 90.00 | 5.3098 | 0.0000 | 7.2513 | 5.3098 | 19.9669 | 31.0000 | 51.0648 | 52.000 |
| 16 | 96.00 | 5.2690 | 0.0000 | 6.9216 | 5.2690 | 18.7898 | 30.0000 | 40.7516 | 48.000 |
| 17 | 102.00 | 5.3886 | 0.0000 | 6.9231 | 5.3886 | 18.5649 | 27.0000 | 33.3802 | 44.000 |
| 18 | 108.00 | 5.5925 | 0.0000 | 7.1022 | 5.5925 | 18.8784 | 26.0000 | 28.4466 | 42.000 |
| 19 | 114.00 | 5.8411 | 0.0000 | 7.3777 | 5.8411 | 19.4997 | 24.0000 | 25.3430 | 39.000 |
| 20 | 120.00 | 6.1136 | 0.0000 | 7.7051 | 6.1136 | 20.2968 | 23.0000 | 23.5615 | 36.000 |
| 21 | 126.00 | 6.2945 | 0.0000 | 7.9641 | 6.2945 | 20.9242 | 22.0000 | 22.7145 | 33.000 |
| 22 | 132.00 | 6.2835 | 0.0000 | 8.0376 | 6.2835 | 21.0925 | 21.0000 | 22.4947 | 30.000 |
| 23 | 138.00 | 6.1856 | 0.0000 | 7.9832 | 6.1856 | 20.9907 | 20.5000 | 22.6326 | 28.000 |
| 24 | 144.00 | 6.0638 | 0.0000 | 7.8567 | 6.0638 | 20.7103 | 20.0000 | 22.8919 | 26.000 |
| 25 | 150.00 | 5.9815 | 0.0000 | 7.7461 | 5.9815 | 20.4627 | 20.0000 | 23.1045 | 24.000 |

### 10.3.2 Thomson River Example, DESIGN Run

The design run storm is a hypothetical extreme storm, here included in the catchment data file. [Other options for prescribing storm data are possible, but note that the storm generation software in RORB is limited to average recurrence intervals of 100 years or less.]

#### *10.3.2.1 Thomson River DESIGN Run Data File (TOMDES.dat)*

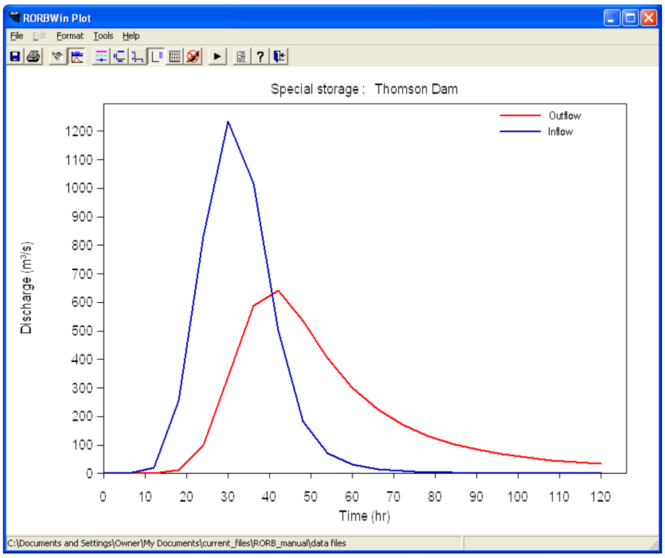
```
Thomson River at Thomson Dam
0,     Mixture of reach types
1,1,6.5,-99,     Sub-Area A
2,1,2.5,-99,     Sub-Area B
3
1,1,2.5,-99,     Sub-Area C
4
5,4,5.5,-99,     reach drowned by reservoir
3,               store h/g (input to reservoir)
1,1,5.0,-99,     Sub-Area D
3
1,1,5.0,-99,     Sub-Area E
4
5,1,4.0,-99
2,1,4.0,-99,     Sub-Area F
3
1,1,5.0,-99,     Sub-Area G
4
5,4,5.0,-99,     reach drowned by reservoir
4,               add to reservoir input h/g
3,               store sum
1,1,2.5,-99,     Sub-Area H
4,     add to reservoir input h/g
5,4,7.0,-99,     reach drowned by reservoir
3
1,1,4.0,-99,     Sub-Area I
4,               add to reservoir input h/g
```

```
5,4,7.5,-99,      reach drowned by reservoir
3
1,1,3.0,-99,      Sub-Area J
4,                add to reservoir input h/g
5,4,3.0,-99,      reach drowned by reservoir
16
Thomson Dam
C dimensions hypothetical
3,0,1
200,100,2,-99,           crest elev.=200 m, length 100 m, coeff.=2
2,2.23e7,1,200,-99,      storage= surf.area*depth
C   it is necessary to route to the narrows to ensure consistency in kc
C   value
5,1,5.0,-99,      this route to The Narrows
3
C   sub-catchment k is necessary
1,1,2.5,-99,      Sub-Area K
4,      sum at the narrows
0,      end of control vector
C  sub-area areas
62,42,44,59,41,31,44,21,80,67,28,-99
0,-99,      no impervious areas
A Design Storm
DESIGN
6,20,1,1,0,-99,     time inc=6h, calcs for 20 incs, 1 burst, 1 pluvi,uni r/f
0,8
Design pattern
7,16,41,71,71,41,16,7,-99
```

### *10.3.2.2 Thomson River DESIGN Run: Output*

Running RORBWin proceeds as for the FIT run, except that only one data file is required and conservatively small values of the loss parameters are used (IL = 0 mm, CL = 2 mm/h). The values of $k_c$ and *m* from the fit run are used.

The inflow and spillway hydrographs calculated for Thomson Dam for this storm are shown below.



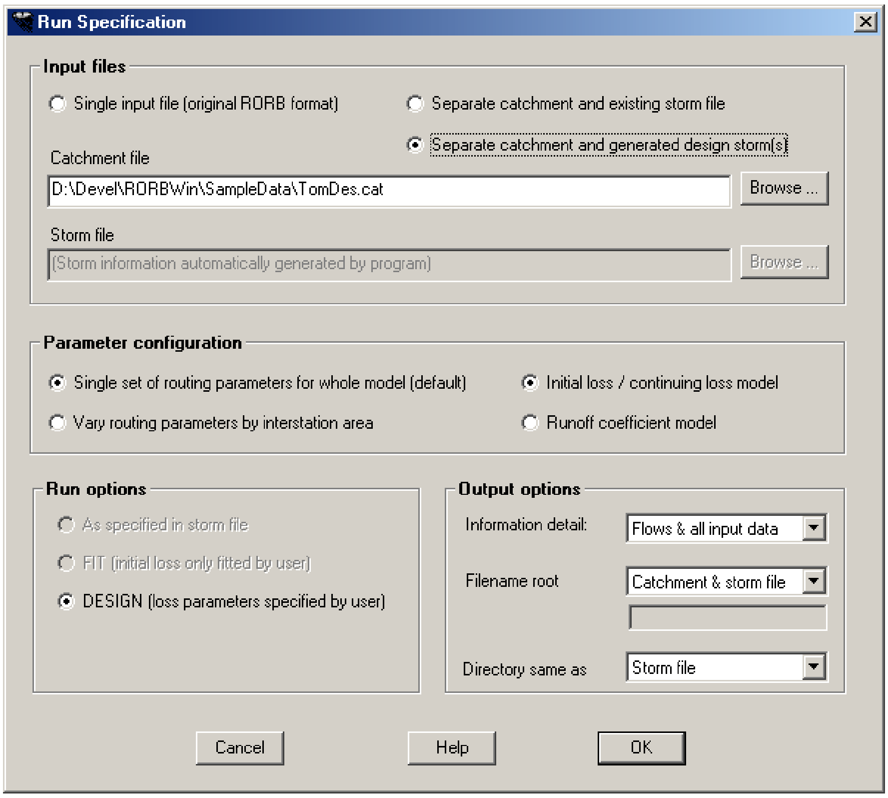

The formatted text file (listed below) also contains the maximum water level and spillway surcharge storage for the passage of this flood. (Note: this is the 'minimum' output, obtained by specifying 'Flows only' on the Run Specification Window (Output Options, Information Detail)

```
RORBWin Output File
 *******************

 Program version 6.00 (last updated 1st September 2007)
 Copyright Monash University and Sinclair Knight Merz

 Date run: 08 Oct 2007 10:35

 Data file         : C:\...\RORB_manual\data files\TOMDES.dat
 Output information: Flows only

 Data check completed


 Input of parameters:
 ********************

 Thomson River at Thomson Dam
 DESIGN Run
 A Design Storm
 Time increment = 6.00 hours

 Constant loss model selected


 Routing results:
 ****************
 Thomson River at Thomson Dam
 A Design Storm
 DESIGN run no.  1


 Parameters:  kc =    65.00    m = 0.80

 Loss parameters     Initial loss (mm)   Cont. loss (mm/h)
                            0.00              2.00

 Results of routing through special storage Thomson Dam
  Peak elevation=  202.17 m
  Peak outflow =   641.83 m³/s (spillway flow)
  Peak storage =     4.85E+07 m³


 *** Special storage :   Thomson Dam

                           Hydrograph
                        Outflow   Inflow
 Peak discharge,m³/s      642.    1233.
 Time to peak,h           42.0     30.0
 Volume,m³            8.38E+07 9.03E+07
 Time to centroid,h       53.1     32.7
 Lag (c.m. to c.m.),h     29.1      8.7
 Lag to peak,h            18.0      6.0


 Hydrograph summary
 ******************

 Site  Description
   01  Special storage :   Thomson Dam - Outflow
   02  Special storage :   Thomson Dam - Inflow

 Inc    Time    Hyd001    Hyd002
   1    6.00     0.000      0.00
   2   12.00     0.000      0.00
   3   18.00     0.636     21.39
   4   24.00    10.854    256.48
   5   30.00    98.715    830.90
   6   36.00   340.004   1233.13
   7   42.00   588.155   1016.06
   8   48.00   641.829    501.86
```

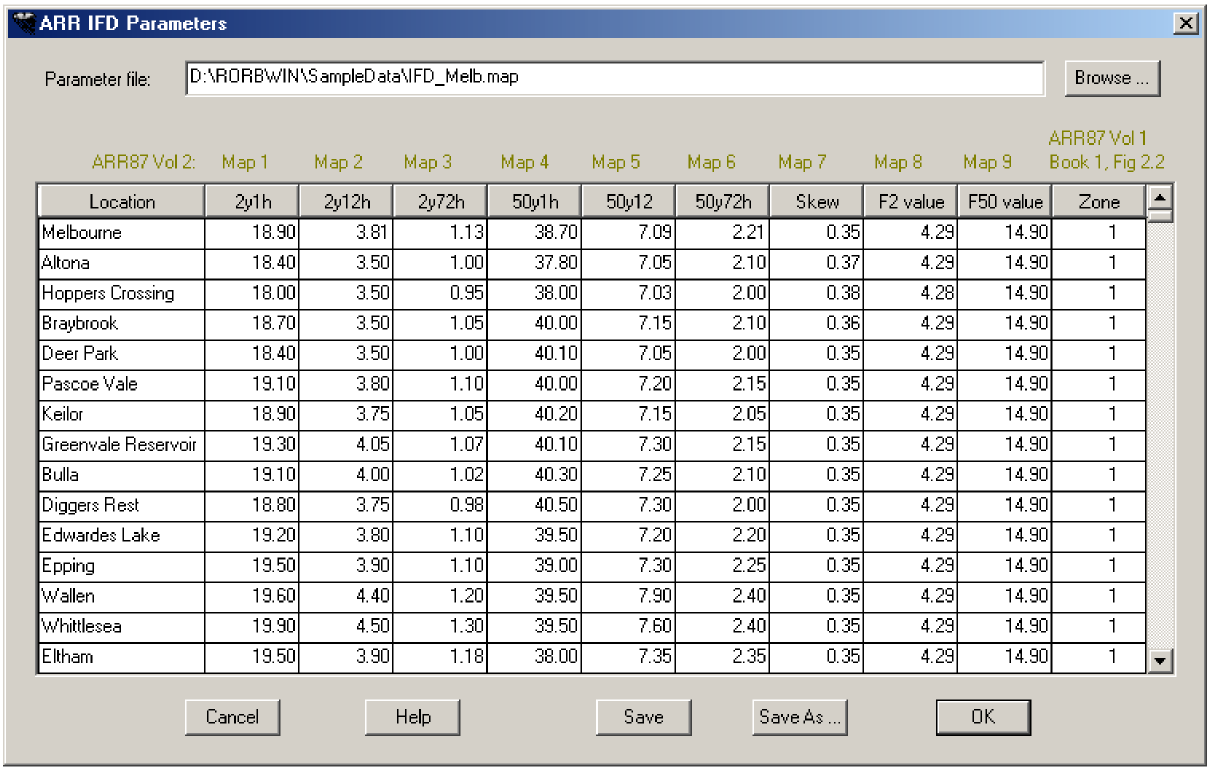
| | | | |
|---|---|---|---|
| 9 | 54.00 | 536.355 | 181.54 |
| 10 | 60.00 | 404.274 | 70.72 |
| 11 | 66.00 | 299.461 | 30.92 |
| 12 | 72.00 | 224.431 | 15.12 |
| 13 | 78.00 | 170.248 | 8.22 |
| 14 | 84.00 | 132.269 | 4.82 |
| 15 | 90.00 | 104.470 | 2.99 |
| 16 | 96.00 | 83.813 | 1.94 |
| 17 | 102.00 | 68.066 | 1.31 |
| 18 | 108.00 | 56.173 | 0.91 |
| 19 | 114.00 | 46.800 | 0.65 |
| 20 | 120.00 | 39.482 | 0.47 |
| 21 | 126.00 | 33.295 | 0.35 |

### 10.3.3 Thomson River Example, DESIGN BATCH Run

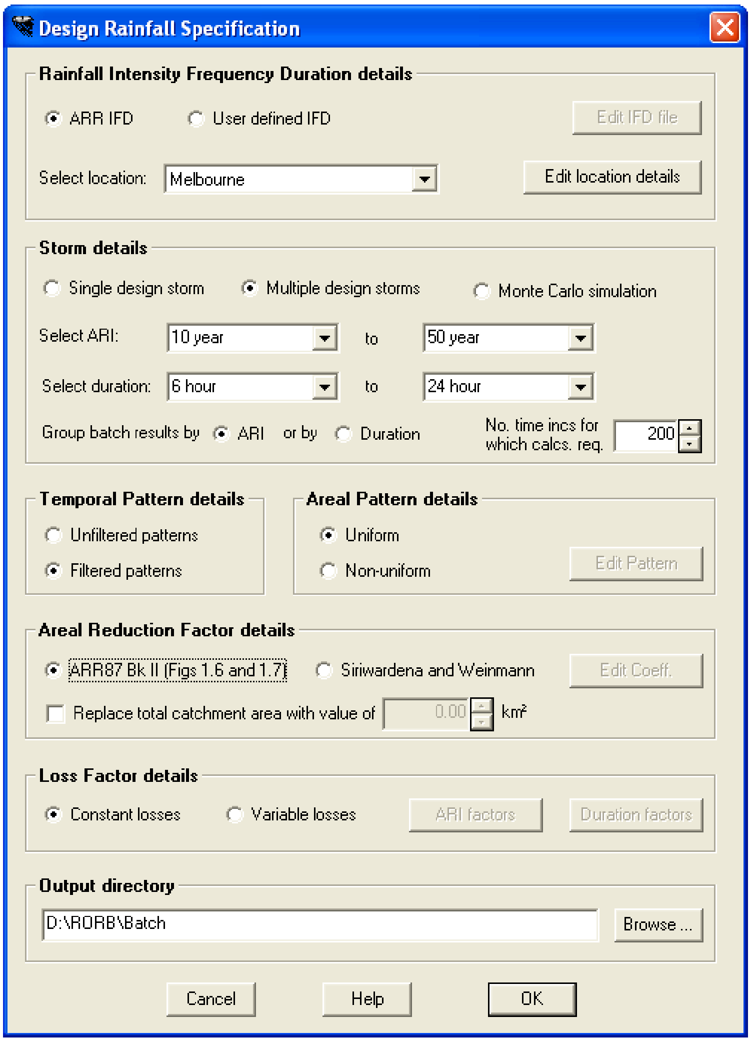
Rather than derive a single design storm, the batch run facilities of the program could be used to generate a number of design storms at the same time. Using the file “TomDes.cat” (this is the same file as “TomDes.dat” though the last six lines defining the design storm have been deleted), the input file in the run specification dialogue should be set to “separate catchment and generated design storm(s)”, as in the following:




After pressing the ‘OK’ button, the next dialogue that appears is the “Design Rainfall Specification”. This dialogue provides the means to generate design rainfalls for any location specified in the design rainfall parameter file (accessed via the “Edit Location Details” button). The design rainfall parameter file contains the nine map parameters required for specification of design rainfalls using the methodology contained in Australian Rainfall and Runoff (24). A sample screen from the design rainfall parameter dialogue is shown below using one of the data files (IFD_Melb.map) provided with the program.

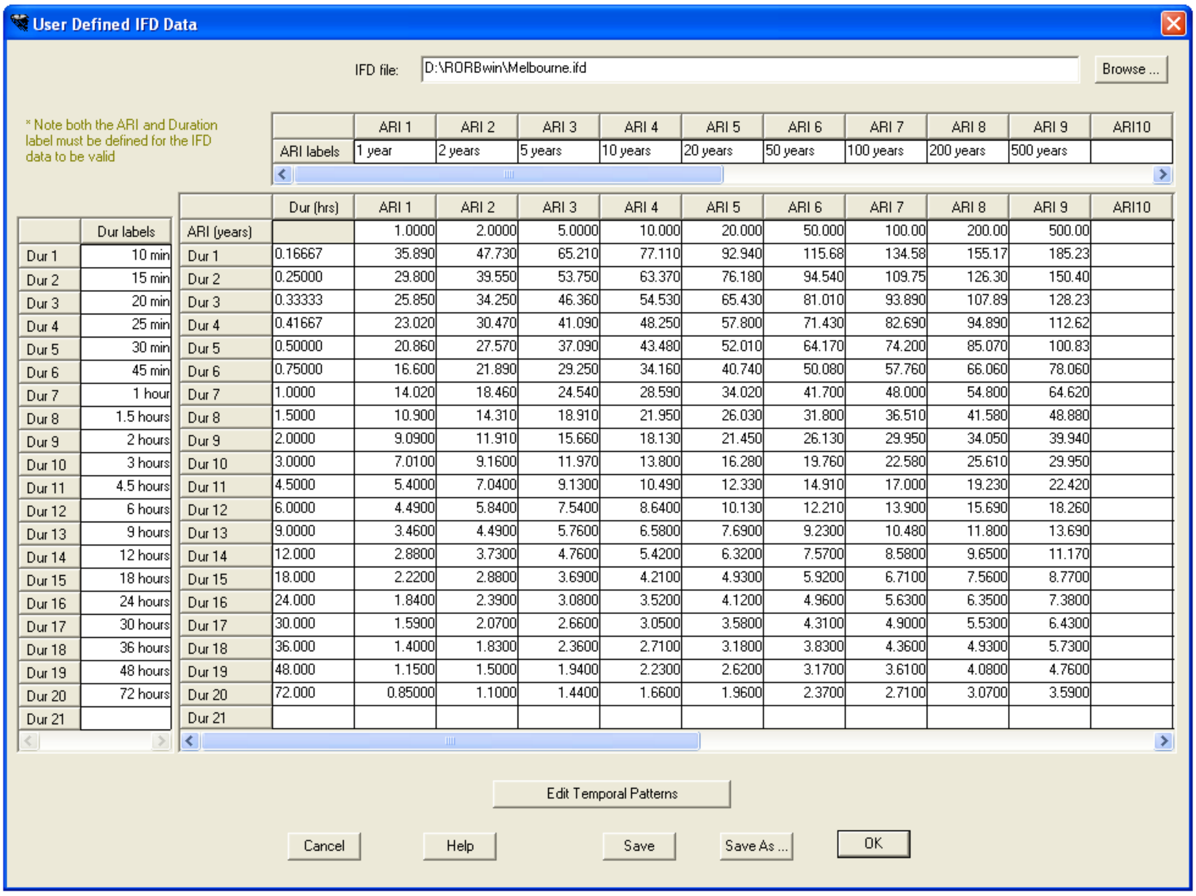



| Location | 2y1h | 2y12h | 2y72h | 50y1h | 50y12 | 50y72h | Skew | F2 value | F50 value | Zone |
|---|---|---|---|---|---|---|---|---|---|---|
| Melbourne | 18.90 | 3.81 | 1.13 | 38.70 | 7.09 | 2.21 | 0.35 | 4.29 | 14.90 | 1 |
| Altona | 18.40 | 3.50 | 1.00 | 37.80 | 7.05 | 2.10 | 0.37 | 4.29 | 14.90 | 1 |
| Hoppers Crossing | 18.00 | 3.50 | 0.95 | 38.00 | 7.03 | 2.00 | 0.38 | 4.28 | 14.90 | 1 |
| Braybrook | 18.70 | 3.50 | 1.05 | 40.00 | 7.15 | 2.10 | 0.36 | 4.29 | 14.90 | 1 |
| Deer Park | 18.40 | 3.50 | 1.00 | 40.10 | 7.05 | 2.00 | 0.35 | 4.29 | 14.90 | 1 |
| Pascoe Vale | 19.10 | 3.80 | 1.10 | 40.00 | 7.20 | 2.15 | 0.35 | 4.29 | 14.90 | 1 |
| Keilor | 18.90 | 3.75 | 1.05 | 40.20 | 7.15 | 2.05 | 0.35 | 4.29 | 14.90 | 1 |
| Greenvale Reservoir | 19.30 | 4.05 | 1.07 | 40.10 | 7.30 | 2.15 | 0.35 | 4.29 | 14.90 | 1 |
| Bulla | 19.10 | 4.00 | 1.02 | 40.30 | 7.25 | 2.10 | 0.35 | 4.29 | 14.90 | 1 |
| Diggers Rest | 18.80 | 3.75 | 0.98 | 40.50 | 7.30 | 2.00 | 0.35 | 4.29 | 14.90 | 1 |
| Edwardes Lake | 19.20 | 3.80 | 1.10 | 39.50 | 7.20 | 2.20 | 0.35 | 4.29 | 14.90 | 1 |
| Epping | 19.50 | 3.90 | 1.10 | 39.00 | 7.30 | 2.25 | 0.35 | 4.29 | 14.90 | 1 |
| Wallen | 19.60 | 4.40 | 1.20 | 39.50 | 7.90 | 2.40 | 0.35 | 4.29 | 14.90 | 1 |
| Whittlesea | 19.90 | 4.50 | 1.30 | 39.50 | 7.60 | 2.40 | 0.35 | 4.29 | 14.90 | 1 |
| Eltham | 19.50 | 3.90 | 1.18 | 38.00 | 7.35 | 2.35 | 0.35 | 4.29 | 14.90 | 1 |




The Design Rainfall Specification dialogue presents all the inputs necessary to run single or batch runs for derivation of design floods. In the example shown here, it is assumed (purely for illustration purposes) that the location of the catchment, and hence the parameters for the design rainfalls, are obtained for Melbourne. Multiple design runs are selected, and for this illustration a number of design events are selected, ranging between 10 to 50 year ARI, and 6 to 24 hours duration. A uniform areal pattern is selected, and the temporal patterns are filtered to remove embedded intensities of higher ARI. The areal reduction factors as specified in Book II of ARR (24), and losses are assumed to be constant with ARI. This routine generates an input and output file for each individual design run, thus it is worth creating/selecting an output directory specific to the design runs of interest.

The Design Rainfall Specification dialogue box also allows selection of user-defined IFD data as an alternative to ARR IFD. When the "Edit IFD File" button is pressed, another window is opened allowing these user-defined IFD files to be opened, edited and saved. This is shown below, with the example file provided with the program (melbourne.ifd) loaded for editing. The information is entered by providing labels for each ARI and duration to be modelled. It is also necessary to enter numerical values in years (for ARIs) and hours (for durations) for each value. Then the intensity in mm/hr for each duration and ARI should be entered. The program allows specification of up to 21 durations and 20 ARIs. The files can be saved and opened similarly to the ARR IFD files. The button "Edit Temporal Pattern" allows the user to define a temporal pattern for each duration. The patterns are entered as a percentage of total rainfall depth per time increment. The time increment is then defined by the duration divided by the number of values in the pattern. Note that RORB assumes the pattern ends at the first zero value. A check is also made to ensure that the temporal pattern sums to 100% across all increments.




| | ARI 1 | ARI 2 | ARI 3 | ARI 4 | ARI 5 | ARI 6 | ARI 7 | ARI 8 | ARI 9 | ARI10 |
|---|---|---|---|---|---|---|---|---|---|---|
| ARI labels | 1 year | 2 years | 5 years | 10 years | 20 years | 50 years | 100 years | 200 years | 500 years | |

| | Dur labels |
|---|---|
| Dur 1 | 10 min |
| Dur 2 | 15 min |
| Dur 3 | 20 min |
| Dur 4 | 25 min |
| Dur 5 | 30 min |
| Dur 6 | 45 min |
| Dur 7 | 1 hour |
| Dur 8 | 1.5 hours |
| Dur 9 | 2 hours |
| Dur 10 | 3 hours |
| Dur 11 | 4.5 hours |
| Dur 12 | 6 hours |
| Dur 13 | 9 hours |
| Dur 14 | 12 hours |
| Dur 15 | 18 hours |
| Dur 16 | 24 hours |
| Dur 17 | 30 hours |
| Dur 18 | 36 hours |
| Dur 19 | 48 hours |
| Dur 20 | 72 hours |
| Dur 21 | |

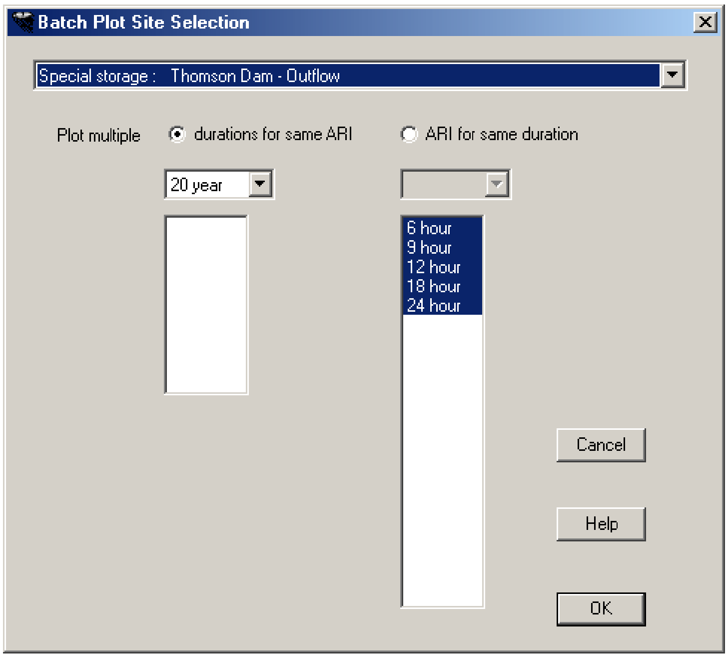
| | Dur (hrs) | ARI 1 | ARI 2 | ARI 3 | ARI 4 | ARI 5 | ARI 6 | ARI 7 | ARI 8 | ARI 9 | ARI10 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ARI (years) | | 1.0000 | 2.0000 | 5.0000 | 10.000 | 20.000 | 50.000 | 100.00 | 200.00 | 500.00 | |
| Dur 1 | 0.16667 | 35.890 | 47.730 | 65.210 | 77.110 | 92.940 | 115.68 | 134.58 | 155.17 | 185.23 | |
| Dur 2 | 0.25000 | 29.800 | 39.550 | 53.750 | 63.370 | 76.180 | 94.540 | 109.75 | 126.30 | 150.40 | |
| Dur 3 | 0.33333 | 25.850 | 34.250 | 46.360 | 54.530 | 65.430 | 81.010 | 93.890 | 107.89 | 128.23 | |
| Dur 4 | 0.41667 | 23.020 | 30.470 | 41.090 | 48.250 | 57.800 | 71.430 | 82.690 | 94.890 | 112.62 | |
| Dur 5 | 0.50000 | 20.860 | 27.570 | 37.090 | 43.480 | 52.010 | 64.170 | 74.200 | 85.070 | 100.83 | |
| Dur 6 | 0.75000 | 16.600 | 21.890 | 29.250 | 34.160 | 40.740 | 50.080 | 57.760 | 66.060 | 78.060 | |
| Dur 7 | 1.0000 | 14.020 | 18.460 | 24.540 | 28.590 | 34.020 | 41.700 | 48.000 | 54.800 | 64.620 | |
| Dur 8 | 1.5000 | 10.900 | 14.310 | 18.910 | 21.950 | 26.030 | 31.800 | 36.510 | 41.580 | 48.880 | |
| Dur 9 | 2.0000 | 9.0900 | 11.910 | 15.660 | 18.130 | 21.450 | 26.130 | 29.950 | 34.050 | 39.940 | |
| Dur 10 | 3.0000 | 7.0100 | 9.1600 | 11.970 | 13.800 | 16.280 | 19.760 | 22.580 | 25.610 | 29.950 | |
| Dur 11 | 4.5000 | 5.4000 | 7.0400 | 9.1300 | 10.490 | 12.330 | 14.910 | 17.000 | 19.230 | 22.420 | |
| Dur 12 | 6.0000 | 4.4900 | 5.8400 | 7.5400 | 8.6400 | 10.130 | 12.210 | 13.900 | 15.690 | 18.260 | |
| Dur 13 | 9.0000 | 3.4600 | 4.4900 | 5.7600 | 6.5800 | 7.6900 | 9.2300 | 10.480 | 11.800 | 13.690 | |
| Dur 14 | 12.000 | 2.8800 | 3.7300 | 4.7600 | 5.4200 | 6.3200 | 7.5700 | 8.5800 | 9.6500 | 11.170 | |
| Dur 15 | 18.000 | 2.2200 | 2.8800 | 3.6900 | 4.2100 | 4.9300 | 5.9200 | 6.7100 | 7.5600 | 8.7700 | |
| Dur 16 | 24.000 | 1.8400 | 2.3900 | 3.0800 | 3.5200 | 4.1200 | 4.9600 | 5.6300 | 6.3500 | 7.3800 | |
| Dur 17 | 30.000 | 1.5900 | 2.0700 | 2.6600 | 3.0500 | 3.5800 | 4.3100 | 4.9000 | 5.5300 | 6.4300 | |
| Dur 18 | 36.000 | 1.4000 | 1.8300 | 2.3600 | 2.7100 | 3.1800 | 3.8300 | 4.3600 | 4.9300 | 5.7300 | |
| Dur 19 | 48.000 | 1.1500 | 1.5000 | 1.9400 | 2.2300 | 2.6200 | 3.1700 | 3.6100 | 4.0800 | 4.7600 | |
| Dur 20 | 72.000 | 0.85000 | 1.1000 | 1.4400 | 1.6600 | 1.9600 | 2.3700 | 2.7100 | 3.0700 | 3.5900 | |
| Dur 21 | | | | | | | | | | | |

After pressing the "OK" button in the Design Rainfall Specification dialogue, the dialogue for specifying the model parameters is presented, and for this example the same parameters as used previously are adopted (IL = 0 mm, CL = 2 mm/hr, $k_c$ = 65, $m$ = 0.8). If the "Text" button is selected then the following output summary is provided:

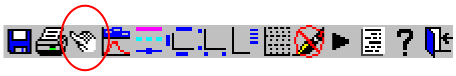
```
RORBWin Batch Run Summary
 ************************

 Program version 6.00 (last updated 1st September 2007)
 Copyright Monash University and Sinclair Knight Merz

 Date run: 08 Oct 2007 10:42

 Catchment file   : C:\...\RORB_manual\data files\TOMDES.CAT
 Rainfall location: Melbourne
 Temporal pattern : AR&R87 Volume 2 for zone 1 (filtered)
 Spatial pattern  : Uniform
 Areal Red. Fact. : Based on ARR87 Bk II, Figs 1.6 and 1.7
 Loss factors     : Constant with ARI


 Parameters:  kc =    65.00    m = 0.80

 Loss parameters     Initial loss (mm)   Cont. loss (mm/h)
                            0.00               2.00

 Peak  Description
   01  Special storage :   Thomson Dam - Outflow
   02  Special storage :   Thomson Dam - Inflow

 Run   Dur   ARI  Rain(mm)     ARF    Peak01    Peak02
   1    6h   10y     51.85    0.89      74.5     555.5
   2    9h   10y     59.20    0.90      80.4     612.3
   3   12h   10y     65.06    0.91      84.7     530.7
   4   18h   10y     75.89    0.92      81.5     423.6
   5   24h   10y     84.44    0.93      76.8     447.2
   6    6h   20y     60.81    0.89     100.5     695.1
   7    9h   20y     69.21    0.90     109.8     771.7
   8   12h   20y     75.89    0.91     116.1     668.0
   9   18h   20y     88.71    0.92     116.9     542.3
  10   24h   20y     98.86    0.93     113.2     574.6
  11    6h   50y     73.31    0.89     140.1     865.8
  12    9h   50y     83.12    0.90     154.6     974.1
  13   12h   50y     90.90    0.91     167.3     846.8
  14   18h   50y    106.52    0.92     172.3     704.5
  15   24h   50y    118.94    0.93     175.2     723.0

Elapsed Run Time (hh:mm:ss) = 00:00:00
```

It is seen that the above provides a summary of peak flows for each location for each combination of ARI and storm duration selected. Details of each individual design run can be found in the individual storm and output files that are saved to the selected output directory. For this example, the generated storm files are all “TomDes_XhYy.stm” and the corresponding output files have the suffix “.out”. (Different options for naming the output files can be selected from the “Filename Root” menu on the first Run Specification dialogue.

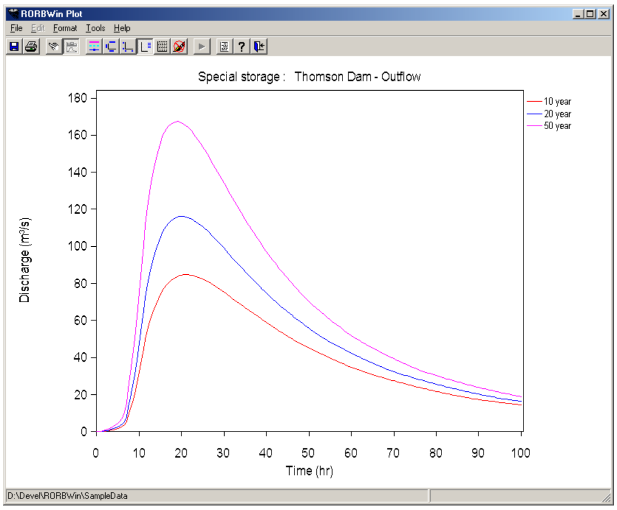
Plots of the batch runs can be viewed by either selecting the “Plot” menu in the text editor, or by selecting the “Plot” button in the Parameter Specification dialogue. The results can be viewed for a selected location as multiple durations for a single ARI, or multiple ARIs for a single duration:

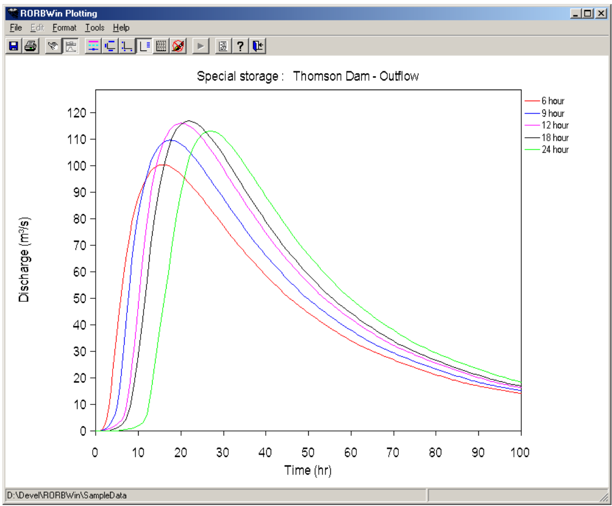



In the example shown on the left, all durations between 6 hours and 18 hours will be plotted for an ARI of 20 years, for the Thomson Dam outflows. If desired, selected durations can be selected/omitted by simply clicking on the menu list. This dialogue can be displayed at any time by selecting the "hand" icon on the plot menu, as highlighted below:



Two different examples of the batch plots are shown below:




On the left hand plot above the increase in hydrograph peak with increasing ARI is clearly seen; on the right-hand plot it is seen that the 18 hour storm yields the highest peak outflow.

An alternative approach to generating multiple design runs – as a flood frequency plot – is described in Section 10.5.

## 10.4 South Creek -- Urbanization Example

This example uses the storm of 1 March 1956 on South Creek to determine the parameters $k_c$ and $m$. Subdivision of the catchment is shown in Figure 10-3. The storm used is treated as a two burst storm, i.e. different loss rates are used for each burst. [Note: it is more common for loss rates to be smaller for successive bursts; however in this example the second burst is short and the interval between the bursts is quite long. There is also some uncertainty in the rainfall due to suspected malfunction of the Narellan Pluviograph in the first burst.]

In the second part of the example, a hypothetical development of the lower catchment has been assumed. This is shown by input of an impervious fraction, and the forming and/or lining of major channels. Three storages are present; a retarding basin to be designed (with a water level limit of 70 *m* RL in a 10 year ARI event), an existing retarding basin, and an existing storage (which is not full). Generated design storms are used as inputs in the DESIGN run.

The example shows, in the first part (FIT):

(i) use of two bursts for specifying the rainfall;
(ii) fitting of the parameters with observed data;

and in the second part (DESIGN):

(i) use of the inbuilt storm generation software
(ii) use of the multiple design storm option (to determine the critical duration for this basin)
(iii) design of a retarding basin;
(iv) routing through existing storages and basins;
(v) how catchment development is modeled.

### 10.4.1 South Creek Example, FIT Run

Two datafiles are used to specify the (natural) South Creek catchment, and the storm used for fitting.

#### *10.4.1.1 South Creek FIT Run: Catchment Data File (SCKFIT.cat)*

```
South Creek at Mulgoa Road, mult. burst
1,      all reaches in natural condition
1,4.7,-99,      sub-area A (reach length 4.7 km)
3,      store h/g
1,4.1,-99,      sub-area B
4,      add h/g's from A & B
3,      store sum
1,3.2,-99,      sub-area C
4,      add h/g from C to A+B
5,2.1,-99,      route to D input
2,2.6,-99,      sub-area D
3,      store
1,3.2,-99,      sub-area E
4,      add in stored h/g
5,2.4,-99,      route to F input
2,4.2,-99,      sub-area F
7.1,      compare obs. and calc. h/g's
0,      end of control vector
C areas
21.8,16.9,13.7,12.7,9.8,14.8,-99
0,-99,         all pervious
```

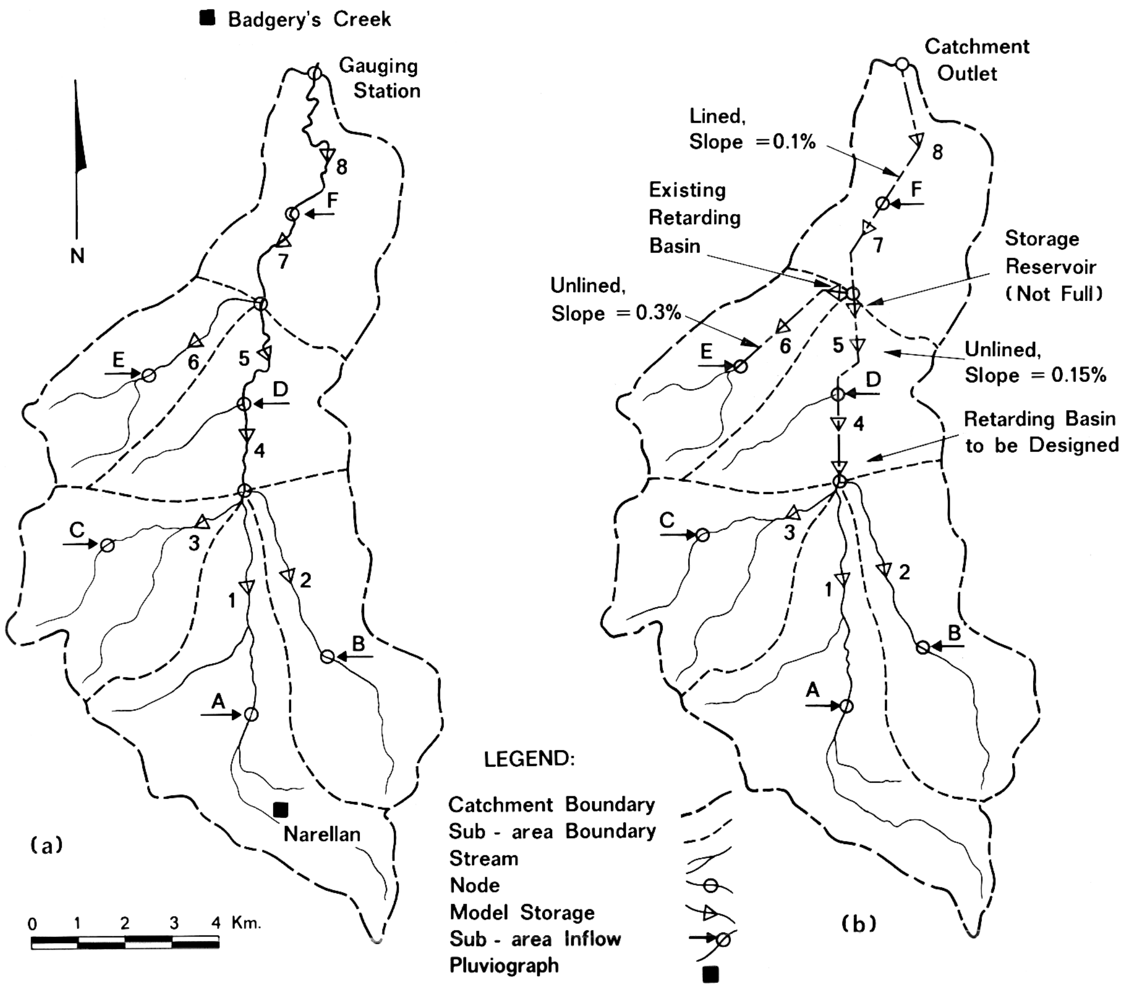



**Figure 10-3 South Creek Catchment, (a) Existing, (b) with Hypothetical Development**

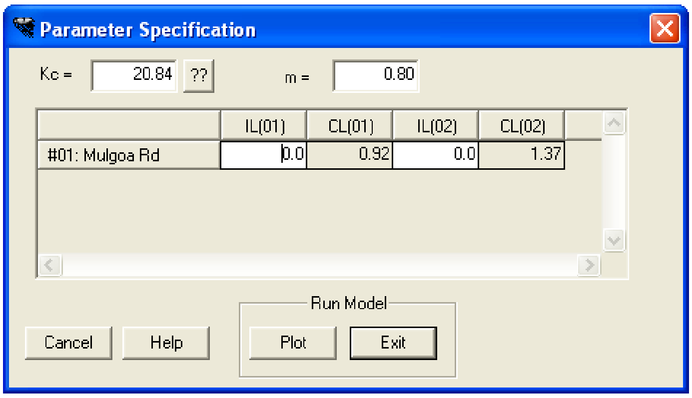
### *10.4.1.2 South Creek FIT Run: Storm Data File (SCKMAR56.stm)*

```
200 hours, 1 March 1956
  Fit
C   time inc =2h, calcs for 36 incs, 2 bursts, 2 pluvis, r/f not uniform
2,36,2,2,1,-99
C   burst 1 from 0-11 incs, burst 2 from 19-21 incs
0,11,19,21
Narellan
.9,2.5,8.7,.2,1.6,5.6,6.5,3.6,9.4,19.2,0,0.4,21.5,-99, mm/time inc
Badgery's Creek
6.5,7.6,8.5,1.6,2.1,5.5,8.7,5.3,23.6,20.6,24.4,0,28.6,-99,mm/time inc
74,79,77,86,83,99,-99,      r/f's on sub-areas, burst 1
26,28,33,33,31,23,-99,       r/f's on sub-areas, burst 2
2,2,2,2,2,2,-99,   pluvi ref no's, burst 1 (pluvi 1 unreliable burst 1)
1,1,1,1,2,2,-99,   pluvi ref no's, burst 2
C hydrograph data
0,35,-99,     h/g from 0-35 time incs
Mulgoa Rd
0,4.57,9.15,13.7,17.2,19.9,23.9,28,35,42.8,53.3,77.2,111
114,83.1,49.5,32.8,22.6,16.9,11.8,8.61,14.8,39.5,54.9
64.3,53,34.2,22.6,15.3,10.2,7,4.57,2.96,1.61,0.8,0,-99,   ords in m^3/s
C proportional vols. in each burst
785,316,-99,     any units
```

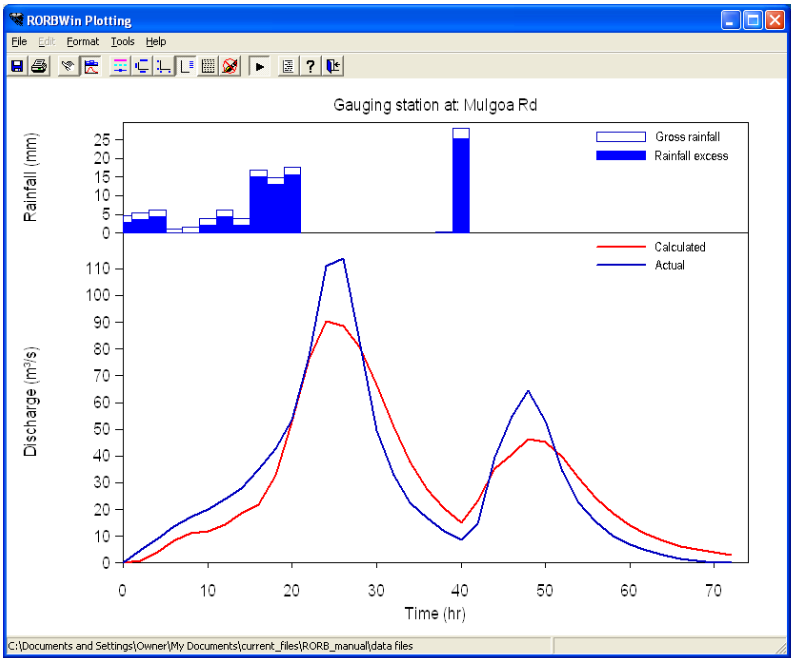
### *10.4.1.3 South Creek FIT Run: Output*

Click on the RORBWin icon to start the program, and then on ‘File’, ‘New’, and ‘Run specification’ to bring up the Run Specification screen. For this example, we have two data files; when this option is specified, the the program looks for a ‘.cat’ file when it prompts for the filename of catchment data, and for a ‘.stm’ file for the storm data. [Note: these are only defaults and the user can specify other filetypes as required.] Other choices (ie loss model etc) on the screen are left at the default options for this example. Pressing the ‘OK’ button brings up the Parameter Specification window.

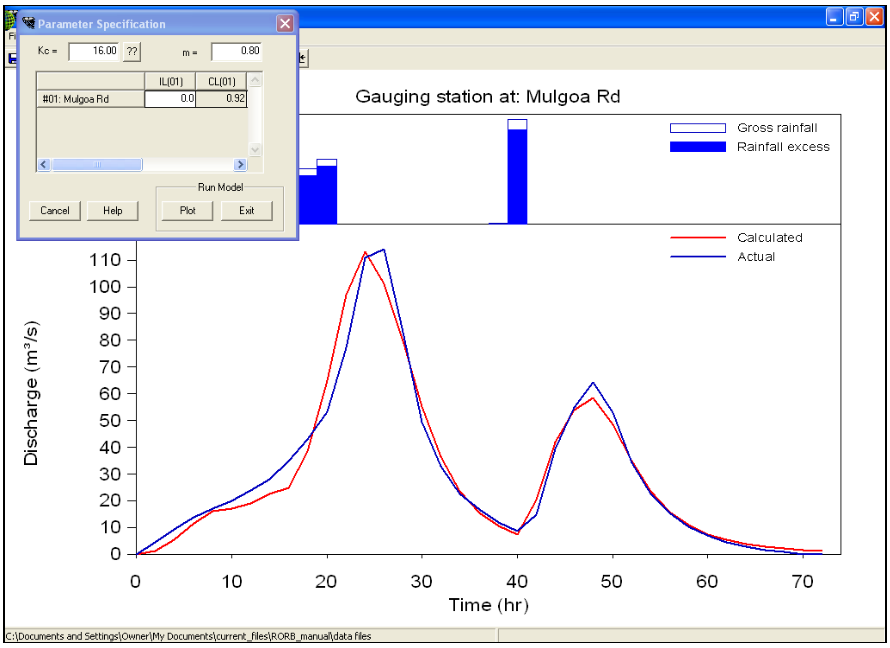
For this Window, the initial $k_C$ value suggest by the program is chosen (ie. 20.8) with $m$ = 0.8. Examination of the timing of the two bursts suggests initial loss values of 0. These are input on the screen (extended to the right) as shown; the program calculates the respective continuing losses, as shown here from a leter screen. (Note: the higher loss rate for a second burst is unusual, but not significant here because of its short duration).




Pressing the ‘Plot’ button produces:



Trail and error fitting produces a value of $k_C$ of 16 as shown.




The formatted text file is obtained by pressing the 'View Text Output' button on the toolbar.

```
RORBWin Output File
 *******************

 Program version 6.00 (last updated 1st September 2007)
 Copyright Monash University and Sinclair Knight Merz

 Date run: 08 Oct 2007 11:16

 Vector file        : C:\...\RORB_manual\data files\SCKFIT.CAT
 Storm file         : C:\...\RORB_manual\data files\SCKMAR56.STM
 Output information: Flows & rainfall only

 Data check completed


 Input of parameters:
 ********************

 South Creek at Mulgoa Road, mult. burst
   Fit  Run
 Initial time:
 200 hours, 1 March 1956
 Time increment = 2.00 hours

 Constant loss model selected


 Rainfall, mm, in time inc. following time shown, burst  1
 Time               Sub-
      Catch         Area
 Incs ment          A   B   C   D   E   F

   0   4.7          4   4   4   5   5   6
   1   5.5          5   5   5   6   6   7
   2   6.1          5   6   6   6   6   7
   3   1.1          1   1   1   1   1   1
   4   1.5          1   1   1   2   2   2
   5   4.0          4   4   4   4   4   5
   6   6.3          6   6   6   7   6   8
   7   3.8          3   4   4   4   4   5
   8  17.0         15  16  16  18  17  20
```

```
   9  14.8          13  14  14  15  15  18
  10  17.5          16  17  16  18  18  21

 Tot. 82.2          74  79  77  86  83  99
 Pluvi. ref. no.     2   2   2   2   2   2

 Burst no.  1
 Continuing loss rate =   0.92 mm/h


 Rainfall-excess, mm, in time inc. following time shown, burst  1
 Time                Sub-
      Catch          Area
 Incs ment            A   B   C   D   E   F

   0   2.8            2   3   3   3   3   4
   1   3.6            3   3   3   4   4   5
   2   4.3            4   4   4   5   4   6
   3   0.0            0   0   0   0   0   0
   4   0.0            0   0   0   0   0   0
   5   2.1            2   2   2   2   2   3
   6   4.4            4   4   4   5   4   6
   7   2.0            2   2   2   2   2   3
   8  15.1           13  14  14  16  15  19
   9  13.0           11  12  12  14  13  16
  10  15.7           14  15  15  17  16  19

 Tot. 63.0           55  60  58  67  64  79


 Rainfall, mm, in time inc. following time shown, burst  2
 Time                Sub-
      Catch          Area
 Incs ment            A   B   C   D   E   F

  19   0.4            0   1   1   1   0   0
  20  28.1           26  27  32  32  31  23

 Tot. 28.5           26  28  33  33  31  23
 Pluvi. ref. no.      1   1   1   1   2   2

 Burst no.  2
 Continuing loss rate =   1.37 mm/h


 Rainfall-excess, mm, in time inc. following time shown, burst  2
 Time                Sub-
      Catch          Area
 Incs ment            A   B   C   D   E   F

  19   0.0            0   0   0   0   0   0
  20  25.4           23  25  30  30  28  20

 Tot. 25.4           23  25  30  30  28  20


 Routing results:
 ****************
 South Creek at Mulgoa Road, mult. burst
 Initial time:
 200 hours, 1 March 1956
   Fit  run no.  1


 Parameters:  kc =    16.00    m = 0.80

 Loss parameters     Initial loss (mm)   Cont. loss (mm/h)
                              0.00              0.92  (Burst  1)
                              0.00              1.37  (Burst  2)
```

```
*** Gauging station at: Mulgoa Rd

                          Hydrograph          Error
                         Calc.   Actual    Abs. Percent
Peak discharge,m³/s      113.1    114.0      -0.9  -0.8
Time to peak,h            24.0     26.0      -2.0  -7.7
Volume,m³             7.88E+06 7.93E+06-4.60E+04  -0.6
Av.abs.ord.err,m³/s                           3.6  11.6 over dur. of calcs
Time to centroid,h        31.5     30.9       0.6   2.1
Lag (c.m. to c.m.),h      8.71     8.07      0.64   7.9
Lag to peak,h             1.16     3.16     -2.00 -63.2


Hydrograph summary
******************

Site  Description
  01  Gauging station at: Mulgoa Rd - Calculated
  02  Gauging station at: Mulgoa Rd - Actual

Inc    Time    Hyd001    Hyd002
  1    2.00     0.000     0.000
  2    4.00     1.302     4.570
  3    6.00     5.437     9.150
  4    8.00    11.612    13.700
  5   10.00    15.957    17.200
  6   12.00    17.042    19.900
  7   14.00    18.948    23.900
  8   16.00    22.635    28.000
  9   18.00    24.987    35.000
 10   20.00    38.290    42.800
 11   22.00    65.021    53.300
 12   24.00    97.005    77.200
 13   26.00   113.127   111.000
 14   28.00   101.304   114.000
 15   30.00    79.842    83.100
 16   32.00    55.387    49.500
 17   34.00    36.354    32.800
 18   36.00    23.620    22.600
 19   38.00    15.612    16.900
 20   40.00    10.594    11.800
 21   42.00     7.383     8.610
 22   44.00    20.168    14.800
 23   46.00    41.848    39.500
 24   48.00    54.058    54.900
 25   50.00    58.334    64.300
 26   52.00    48.568    53.000
 27   54.00    34.925    34.200
 28   56.00    23.533    22.600
 29   58.00    15.784    15.300
 30   60.00    10.773    10.200
 31   62.00     7.522     7.000
 32   64.00     5.372     4.570
 33   66.00     3.917     2.960
 34   68.00     2.911     1.610
 35   70.00     2.201     0.800
 36   72.00     1.689     0.000
 37   74.00     1.315     0.000
```

### 10.4.2 South Creek Example, DESIGN Run

The data file for the design run provides for hypothetical development in the catchment by specifying the degrees of imperviousness for individual sub-areas, the reach codes for the hydraulic improvement of the main channels, and three types of retarding basin (for display of RORB's capabilities). Since generated design storms are being used (without, in this case, areal reduction factors), the catchment files does not include storm data.

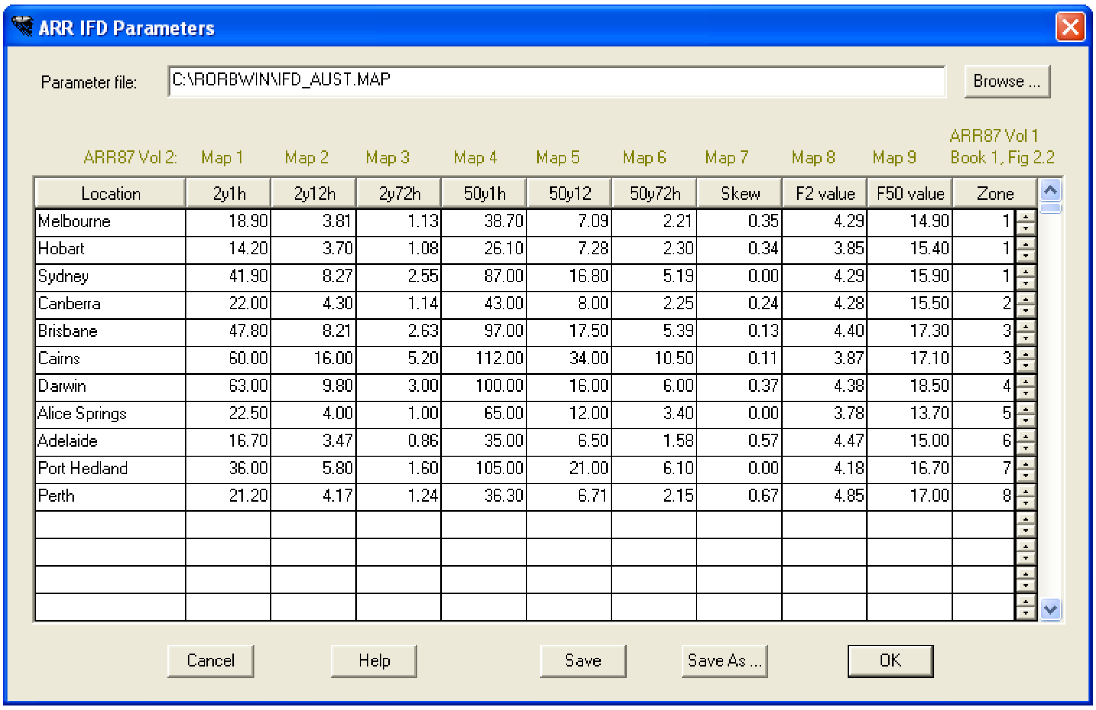
#### *10.4.2.1 South Creek DESIGN Run: Catchment Data File (SCKDES.cat)*

```
South Creek at Mulgoa Road (with hypothetical development)
0,     mixture of reach types
1,1,4.7,-99,     sub-area A (reach type 1, length 4.7 km)
3
1,1,4.1,-99,              B
4
3
1,1,3.2,-99,              C
4
16.1,     basin to be designed (print h/g's)
Basin to be designed
C   pipe+weir flag, s'way coeff, pipe ent loss coeff, bend loss coeff
2,1.9,.5,0,-99
C   flag for stor-elev formula, formula parameters
2,3780,3,65.0,-99
5,2,2.1,0.15,-99,     unlined reach (2.1 km, slope 0.15%)
2,2,2.6,0.15,-99,        D
C    existing drawn down storage
16,     (print h/g's)
Existing storage (not full)
0,-2.0E5,4,1,-99
0,-99,            no h-s data
3
1,2,3.2,0.3,-99,          E
C    existing basin with s-q table
16,     (print h/g's)
Existing basin
1,0,14
0,0,100,2,800,4,2000,6,4000,8,8000,10
1.6E4,12.5,3.5E4,15,9.0E4,17.5,2.0E5,20,3.0E5,30,
4.0E5,40.,5.0E5,50.,6.0E5,60.,-99
0,-99,            no h-s data
4
5,3,2.4,0.1,-99,      lined reach (2.4 km, slope 0.1%)
2,3,4.2,0.1,-99,          F
7
Catchment outflow
0
C    sub-area data
21.8,16.9,13.7,12.7,9.8,14.8,-99,     areas(sq. km)
1,0,0,0,0.2,0.2,0.35,-99,     flag & fracts. impervious
```

#### *10.4.2.2 South Creek DESIGN Run: Screen Output*

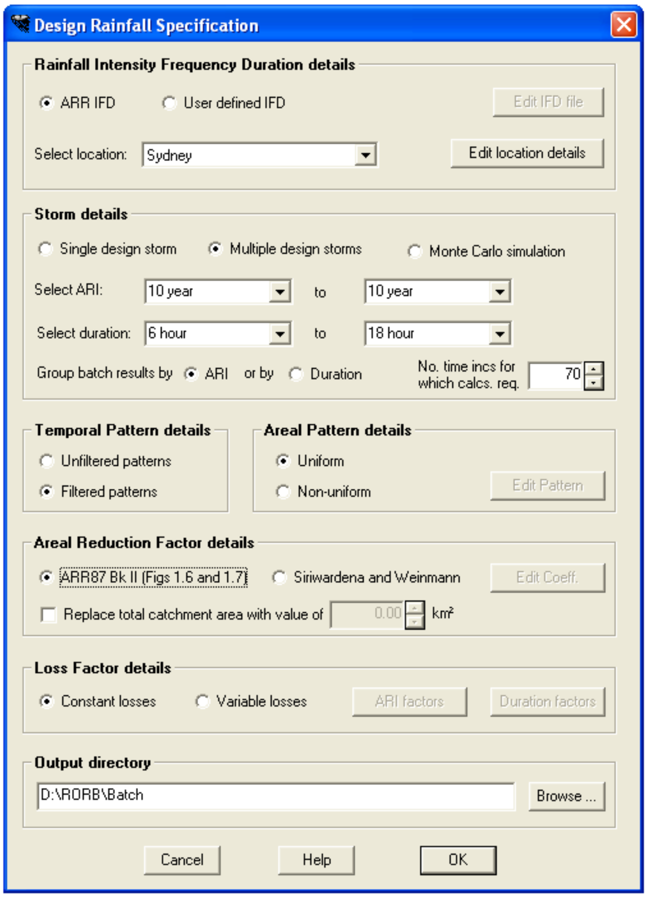
For this case the 'Separate Catchment and Generated Design Storm(s)' option is chosen on the Run Specification window, together with the 'Runoff Coefficient Model', before moving on by pressing 'OK'.

The program uses stored IFD information (using ARR, 1999) as shown in the following table – more locations can be input as required.

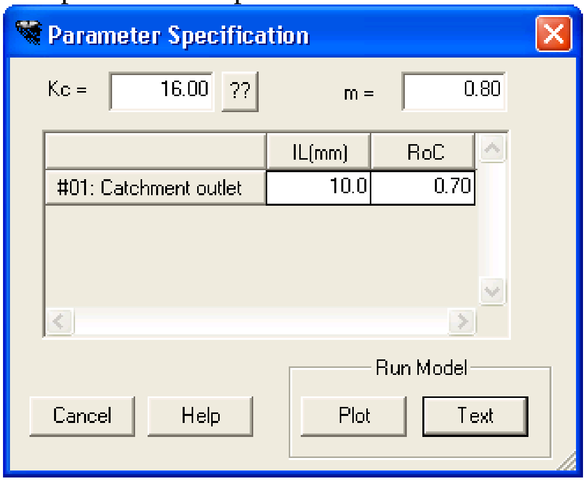



| Location | 2y1h | 2y12h | 2y72h | 50y1h | 50y12 | 50y72h | Skew | F2 value | F50 value | Zone |
|---|---|---|---|---|---|---|---|---|---|---|
| | Map 1 | Map 2 | Map 3 | Map 4 | Map 5 | Map 6 | Map 7 | Map 8 | Map 9 | |
| Melbourne | 18.90 | 3.81 | 1.13 | 38.70 | 7.09 | 2.21 | 0.35 | 4.29 | 14.90 | 1 |
| Hobart | 14.20 | 3.70 | 1.08 | 26.10 | 7.28 | 2.30 | 0.34 | 3.85 | 15.40 | 1 |
| Sydney | 41.90 | 8.27 | 2.55 | 87.00 | 16.80 | 5.19 | 0.00 | 4.29 | 15.90 | 1 |
| Canberra | 22.00 | 4.30 | 1.14 | 43.00 | 8.00 | 2.25 | 0.24 | 4.28 | 15.50 | 2 |
| Brisbane | 47.80 | 8.21 | 2.63 | 97.00 | 17.50 | 5.39 | 0.13 | 4.40 | 17.30 | 3 |
| Cairns | 60.00 | 16.00 | 5.20 | 112.00 | 34.00 | 10.50 | 0.11 | 3.87 | 17.10 | 3 |
| Darwin | 63.00 | 9.80 | 3.00 | 100.00 | 16.00 | 6.00 | 0.37 | 4.38 | 18.50 | 4 |
| Alice Springs | 22.50 | 4.00 | 1.00 | 65.00 | 12.00 | 3.40 | 0.00 | 3.78 | 13.70 | 5 |
| Adelaide | 16.70 | 3.47 | 0.86 | 35.00 | 6.50 | 1.58 | 0.57 | 4.47 | 15.00 | 6 |
| Port Hedland | 36.00 | 5.80 | 1.60 | 105.00 | 21.00 | 6.10 | 0.00 | 4.18 | 16.70 | 7 |
| Perth | 21.20 | 4.17 | 1.24 | 36.30 | 6.71 | 2.15 | 0.67 | 4.85 | 17.00 | 8 |

For this example, the following screen shows the storm parameters selected for the multi-storm run (Sydney, ARI = 10 years, durations 6 to 18 hours, filtered temporal patterns, no areal reduction factor). [Users should make the most appropriate choices for their particular applications].



The parameters input are those from the FIT run, with appropriate losses.




Pressing 'Text' ('Plot' is also an option) takes the program to the screen for interactive storage design. Recall that the objective is to design a basin such that the peak elevation during passage of the design flood(s) is 70 m AHD. Hence, the overflow spillway level is set at 70 m (with an arbitrary effective length of 25 m), and the number and/or diameter of pipes varied as required.

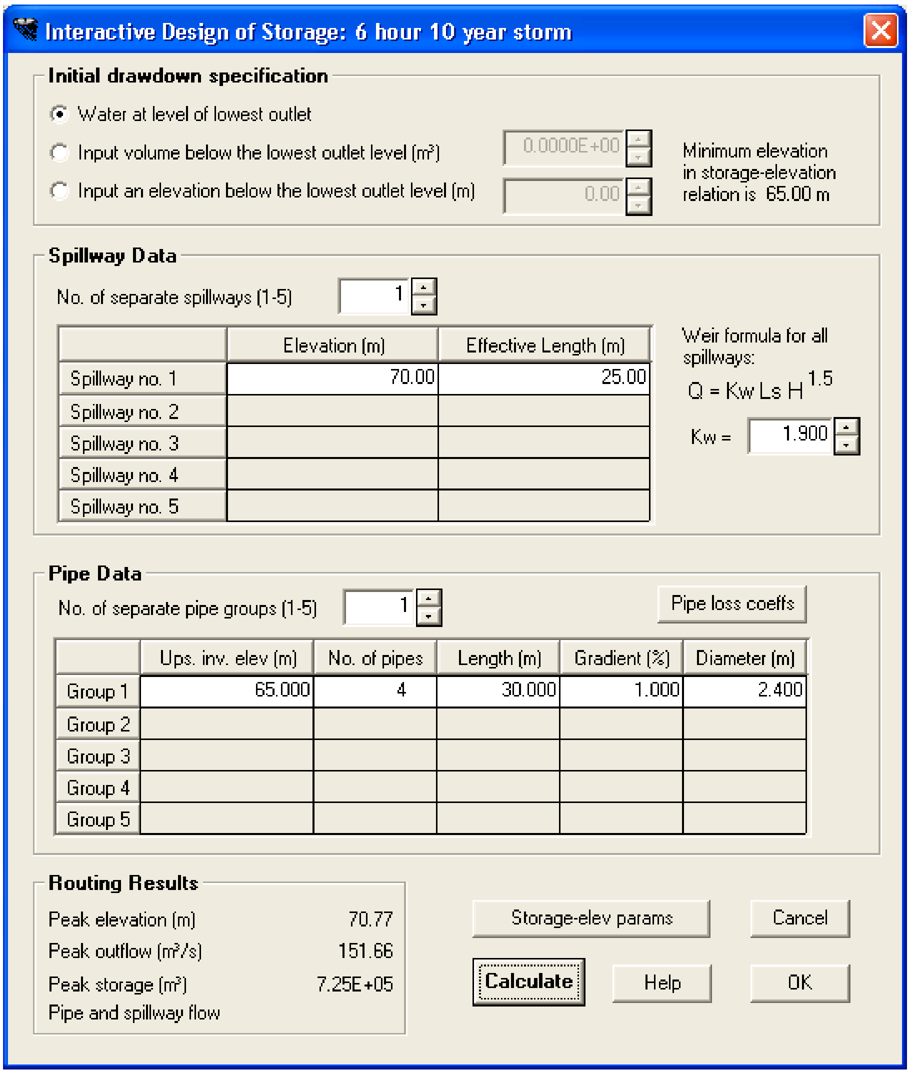
The screen pictured overleaf shows the input data, and the routing results for the first run. Clearly, *more pipes* are needed to reduce the peak level to 70 m AHD.

Six 2.4 m dia pipes produce a level of 69.97 m and a peak flow of 159 $m^3/s$ for a 6 hour storm.

Pressing 'OK' brings up the same screen for the next duration – 9 hours. Pressing 'Calculate' shows a peak of 70.31 m (flow 176 $m^3/s$). Using *seven pipes* (level 69.92, peak 184 $m^3/s$) meets the design criterion.

The 12 hour storm reaches 69.5 m (169 $m^3/s$), and the 18 hour storm 69.1 m (154 $m^3/s$). Clearly, 9 hours is the critical duration storm for the given design criteria.

The program produces a summary of the batch run results (with individual run files stored on disk for retrieval as needed). Plots are available for the storms used for the batch runs.

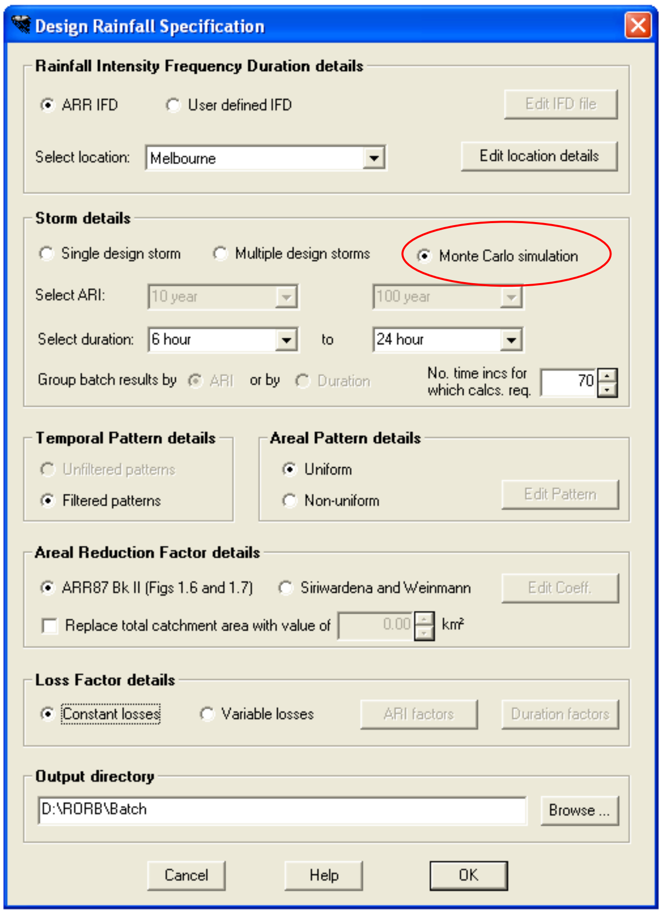



```
RORBWin Batch Run Summary
 ************************

 Program version 6.00 (last updated 1st September 2007)
 Copyright Monash University and Sinclair Knight Merz

 Date run: 08 Oct 2007 11:59

 Catchment file    : C:\...\RORB_manual\data files\SCKDES.CAT
 Rainfall location: Sydney
 Temporal pattern  : AR&R87 Volume 2 for zone 1 (filtered)
 Spatial pattern   : Uniform
 Areal Red. Fact.  : Based on ARR87 Bk II, Figs 1.6 and 1.7
 Loss factors      : Constant with ARI


 Parameters:  kc =    16.00     m = 0.80

 Loss parameters      Initial loss (mm)      Runoff coeff.
                            10.00                 0.70

 Peak  Description
   01  Special storage :   Basin to be designed - Outflow
   02  Special storage :   Basin to be designed - Inflow
   03  Special storage :   Existing storage (not full) - Outflow
   04  Special storage :   Existing storage (not full) - Inflow
   05  Special storage :   Existing basin - Outflow
   06  Special storage :   Existing basin - Inflow
   07  Calculated hydrograph,  Catchment outflow
```

```
 Run    Dur    ARI   Rain(mm)     ARF     Peak01     Peak02     Peak03     Peak04
Peak05     Peak06     Peak07
   1     6h    10y    116.68     0.96      159.0      199.0      120.7      183.1
29.3       70.5      138.6
   2     9h    10y    133.80     0.96      183.6      228.2      140.0      200.2
31.6       67.3      162.0
   3    12h    10y    147.50     0.97      169.4      200.0      146.4      215.7
29.7       56.5      177.0
   4    18h    10y    172.34     0.97      153.8      159.8      144.2      187.8
26.7       39.3      182.6

Elapsed Run Time (hh:mm:ss) = 00:06:05
```

## 10.5 Monte-Carlo Simulation Example

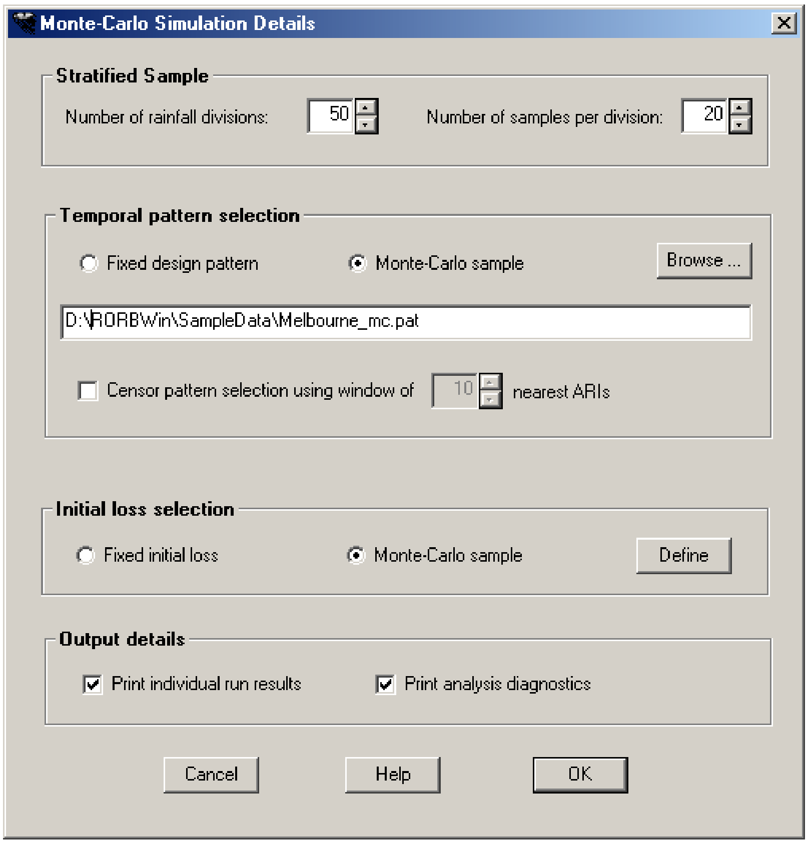
This example continues the illustration of the model to assist in the determination of the spillway design flood for a major dam (as discussed in Section 10.3). However, with this example, the Monte-Carlo facility of the program is used.




Adopting the inputs selected for the example presented in Section 10.3.3, the Monte-Carlo option is selected instead of the multiple batch runs:

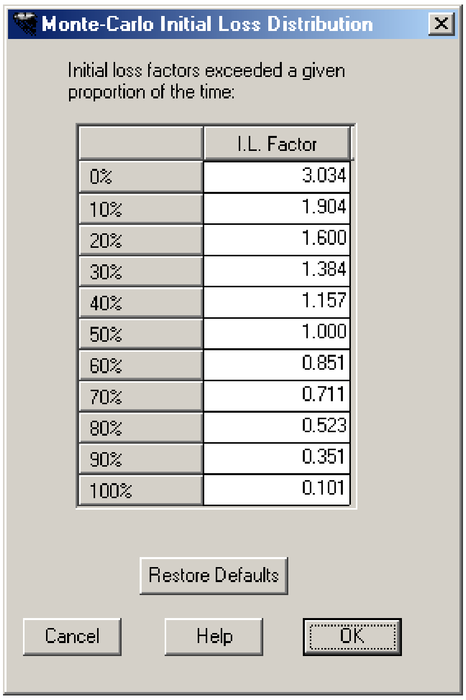
Note when the Monte-Carlo option is selected it is only necessary to select the storm duration of interest, as the program automatically generates the design rainfalls required to derive design floods with ARIs ranging between 2 and 200 years.

All other inputs are as specified in the example presented in Section 10.3.3. After clicking the "OK" button, the standard Parameter Specification dialogue is presented, and again for this example the same parameters as used previously are adopted (CL = 2 mm/hr, $k_c$ = 65, $m$ = 0.8, though note to illustrate the effect on initial loss an IL of 10 mm is adopted).



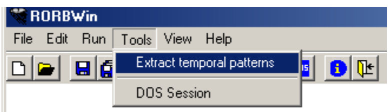

The next dialogue to be displayed is the Monte-Carlo Simulation Details, as shown in the adjacent graphic. As discussed in Section 8.2 it is usually sufficient to divide the rainfall distribution into 50 intervals, and to run the model 20 times with different combinations of inputs within each rainfall interval. If more or less than these divisions are desired then the required number can be specified in the top two integer fields.

The simulation can be undertaken with either or both temporal patterns and initial losses selected stochastically, or else either can be treated as fixed inputs. The distribution of initial losses can be specified in the Initial Loss Distribution dialogue which is accessed via the “Define” button shown above.



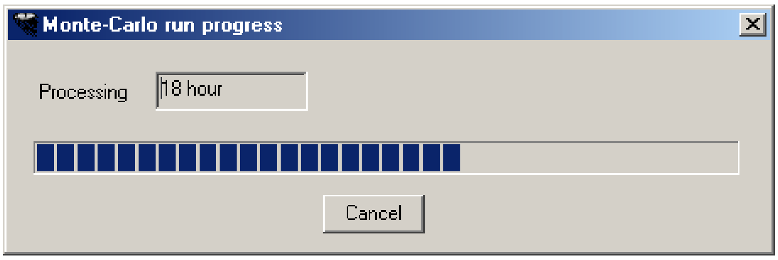

The program provides a default distribution that should be suitable for most applications, however if desired an alternative distribution may be entered as cumulative exceedance values.

A file of temporal patterns to be used in the Monte-Carlo simulation must be specified. The file must be in a specific format as generated by the “Extract Temporal Pattern” routine provided under the “Tools” menu:




and an example file called “Melbourne_mc.pat” is provided with the program’s installation files. If using user-defined IFD data, it is important to ensure that the duration lables in the Monte Carlo temporal pattern file match those in the IFD file.

When the “OK” button is selected on the “Monte-Carlo Simulation Details” dialogue, a progress bar is displayed that monitors the proportion of the simulation completed:




Selection of the “Cancel” button halts the simulation and intermediate results are provided.

If the “Text” button in the “Parameter Specification” dialogue is selected, then the program first displays the text output. To save space only selected sections of the output is presented here, though it should be noted that output is provided for each storm duration considered, and the final set of results are presented at the very end of the file.

Information on which temporal patterns are available, and the minimum and maximum ARIs of each event, are obtained from the selected temporal pattern file.

```
 RORBWin Monte-Carlo Run Summary
 *******************************

Program version 6.00 (last updated 1st September 2007)
 Copyright Monash University and Sinclair Knight Merz

 Date run: 08 Oct 2007 12:24

 Catchment file    : D:\Devel\RORBWin\SampleData\TomDes.cat
 Rainfall location: Melbourne
 Spatial pattern   : Uniform
 Areal Red. Fact. : Based on ARR87 Bk II, Figs 1.6 and 1.7
 Loss factors      : Constant with ARI

 Temporal pattern : Monte-Carlo (with filtering) sampled from D:\Melbrne_mc.pat
                    Duration    6m 10m 15m 20m 25m 30m 45m  1h1.5h  2h  3h4.5h
                    Num avail    0   0   0   0   0  30   0  30  30  30  30  30
                    Max ARI      0   0   0   0   0 547   0 818 880 781 525 292
                    Min ARI      0   0   0   0   0   5   0   4   3   3   4   4

 Parameters:  kc =    65.00    m = 0.80

 Loss parameters      Initial loss (mm)   Cont. loss (mm/h)
                            10.00               2.00


Results for Storm Duration of 6 hour
~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~

Rorb model check run completed OK:
   No. of time increments in run     = 201
   Simulation time increment (hrs)  =   0.500
   Number of incs in rainfall burst =  12
   Number of sub- & intrstn-areas    =  11  1
   Number of hydrographs to track    =   2

Monte-Carlo sampling undertaken using  20 runs over each rainfall interval,
where the rainfall distribution is discretised into  50 intervals.

Label   Description

-----   -----------
Div     Number of discretised rainfall interval
Run     Number of run within given rainfall interval
ARI     ARI of stochastic rainfall depth
Depth   Depth (mm) of stochastic design rainfall burst
TPat    Rank of temporal pattern selected from *_mc.pat file for given duration
I.L.    Stochastic factor applied to initial loss parameter
IL_F    ARI-dependent deterministic factors applied to initial loss parameter
IL_F    ARI-dependent deterministic factors applied to loss rate parameter
Peak01  Special storage :   Thomson Dam - Outflow
Peak02  Special storage :   Thomson Dam - Inflow
```

| Div | Run | ARI | Depth | TPat | I.L. | IL_F | LR_F | Peak01 | Peak02 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 1 | 1.0 | 24.9 | 21 | 1.10 | 1.00 | 1.00 | 6.24 | 78.90 |
| 1 | 2 | 1.0 | 24.9 | 4 | 1.67 | 1.00 | 1.00 | 4.22 | 62.37 |
| 1 | 3 | 1.0 | 24.9 | 2 | 2.15 | 1.00 | 1.00 | 1.00 | 13.80 |
| 1 | 4 | 1.0 | 24.9 | 30 | 0.50 | 1.00 | 1.00 | 20.15 | 270.45 |
| 1 | 5 | 1.0 | 24.9 | 24 | 2.50 | 1.00 | 1.00 | 0.00 | 0.00 |
| 1 | 6 | 1.0 | 24.9 | 8 | 1.08 | 1.00 | 1.00 | 9.87 | 133.11 |
| . | . | . | . | . | . | . | . | . | . |
| . | . | . | . | . | . | . | . | . | . |
| . | . | . | . | . | . | . | . | . | . |
| 50 | 15 | 1189.2 | 110.3 | 24 | 1.66 | 1.00 | 1.00 | 276.90 | 1950.39 |
| 50 | 16 | 1043.7 | 108.4 | 4 | 0.48 | 1.00 | 1.00 | 306.63 | 1742.78 |
| 50 | 17 | 1180.8 | 110.2 | 11 | 1.51 | 1.00 | 1.00 | 278.27 | 1854.26 |
| 50 | 18 | 1082.0 | 108.9 | 30 | 1.64 | 1.00 | 1.00 | 271.71 | 2102.99 |
| 50 | 19 | 1039.6 | 108.3 | 21 | 1.81 | 1.00 | 1.00 | 252.54 | 1569.52 |
| 50 | 20 | 1189.2 | 110.3 | 20 | 1.26 | 1.00 | 1.00 | 295.86 | 1959.94 |

If "Print individual runs" results is selected, then the peak flows and summary of inputs is provided for each run undertaken. The "Div" column denotes the rainfall division, and "Run" column denotes the run number within each division. "ARI" indicates the approximate ARI of the rainfall depth ("Depth"), and "TPat" indicates the reference number of the temporal pattern randomly selected from the sample provided. "I.L." denotes the stochastically generated initial loss factor, which is multiplied by the ARI-dependent factor ("IL_F", if selected) and the initial loss value to obtain the initial loss (in mm) that is used in the run. "LR_F" denotes the ARI-dependent loss rate factor (if selected). The subsequent columns summarise the peak flows obtained for each run.

>>> Distributional check on temporal patterns

| Pattern#: | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
|---|---|---|---|---|---|---|---|---|---|---|
| Freq% : | 3.90 | 3.50 | 2.70 | 3.10 | 2.80 | 4.30 | 2.60 | 2.80 | 4.00 | 3.20 |

| Pattern#: | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |
|---|---|---|---|---|---|---|---|---|---|---|
| Freq% : | 3.90 | 3.00 | 3.20 | 3.80 | 3.10 | 3.60 | 2.60 | 2.80 | 2.30 | 3.60 |

| Pattern#: | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 |
|---|---|---|---|---|---|---|---|---|---|---|
| Freq% : | 2.40 | 4.10 | 3.00 | 2.50 | 3.50 | 3.90 | 3.90 | 3.50 | 3.70 | 4.70 |

>>> Exceedance frequency percentiles check on losses (prior to factoring)

| Percentile | Initial Loss |
|---|---|
| 0 | 3.01 |
| 5 | 2.42 |
| 10 | 1.89 |
| 15 | 1.71 |
| 20 | 1.60 |
| 25 | 1.50 |
| 30 | 1.38 |
| 35 | 1.25 |
| 40 | 1.16 |
| 45 | 1.08 |
| 50 | 1.00 |
| 55 | 0.93 |
| 60 | 0.88 |
| 65 | 0.81 |
| 70 | 0.73 |
| 75 | 0.62 |
| 80 | 0.54 |
| 85 | 0.46 |
| 90 | 0.35 |
| 95 | 0.21 |
| 100 | 0.10 |

If "Print analysis diagnostics" is selected, then distributional checks on the temporal patterns sampled and the cumulative exceedance distribution of losses are presented. If sufficient runs are undertaken these should be the same as the input distributions.

```
Peak  Description
  01  Special storage :   Thomson Dam - Outflow
  02  Special storage :   Thomson Dam - Inflow

Peak flows for selected AEPs (* indicates value unreliable, ie bounded by
interval contributions to AEP > 20% total)

  No.      AEP(1inY)        Peak01         Peak02
    1              2         22.64         240.13
    2              5         40.23         387.09
    3             10         54.16         484.70
    4             20         77.45         633.96
    5             50        115.72         855.33
    6            100        150.76        1037.18
    7            200        181.70*       1241.42*
```

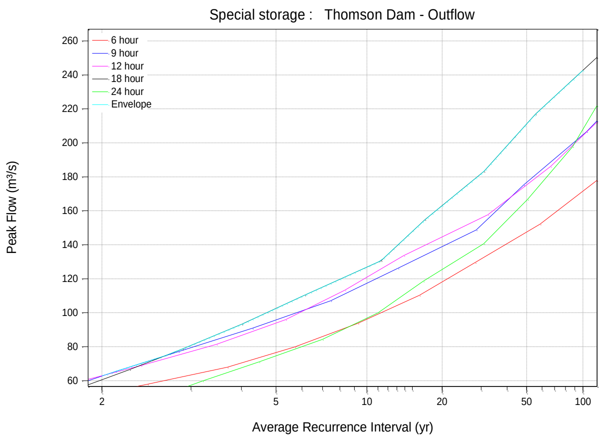
These are the expected probability estimates of the design floods peaks at the selected locations. Note that the 200 ARI event is flagged as being an unreliable estimate and should not be relied upon.

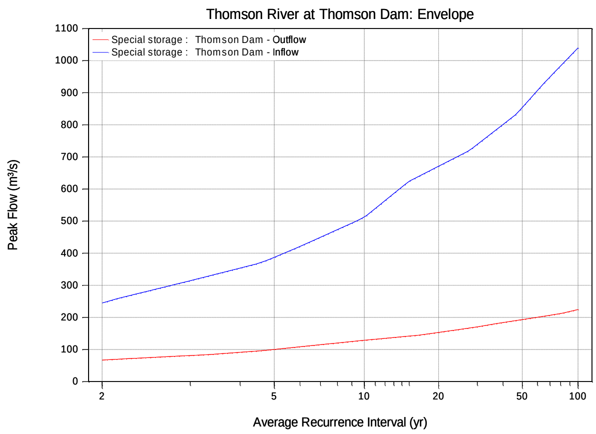
Results for other durations follow, but are not reproduced here to save space.

```
Summary of Runs for Critical Duration Analysis
~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~

Peak  01: Special storage :   Thomson Dam - Outflow
  Num AEP (1:Y)        6h        9h       12h       18h       24h      Peak   Tcrit
    1         2     22.64     24.92     26.13     25.30     27.14     27.14    24h
    2         5     40.23     46.18     50.04     51.99     54.85     54.85    24h
    3        10     54.16     67.13     68.40     75.80     83.62     83.62    24h
    4        20     77.45     90.44     99.93    105.46    114.49    114.49    24h
    5        50    115.72    128.16    140.52    158.74    177.64    177.64    24h
    6       100    150.76    167.30    177.35    206.24    226.04    226.04    24h
    7       200    181.70    211.03    223.87    260.79    282.32    282.32    24h

Peak  02: Special storage :   Thomson Dam - Inflow
  Num AEP (1:Y)        6h        9h       12h       18h       24h      Peak   Tcrit
    1         2    240.13    243.21    226.57    196.31    198.30    243.21     9h
    2         5    387.09    391.43    391.77    341.16    341.87    391.77    12h
    3        10    484.70    511.50    510.45    458.54    478.91    511.50     9h
    4        20    633.96    633.35    649.86    561.76    582.16    649.86    12h
    5        50    855.33    843.18    841.63    725.27    759.13    855.33     6h
    6       100   1037.18   1016.93    992.49    915.11    917.67   1037.18     6h
    7       200   1241.42   1218.14   1217.11   1072.54   1065.15   1241.42     6h

Elapsed Run Time (hh:mm:ss) = 00:00:10
```

The above table provides a summary of all the different durations (but note the warnings about unreliable estimates are not reproduced here) and the envelope of critical durations that yield the highest peaks is also presented.

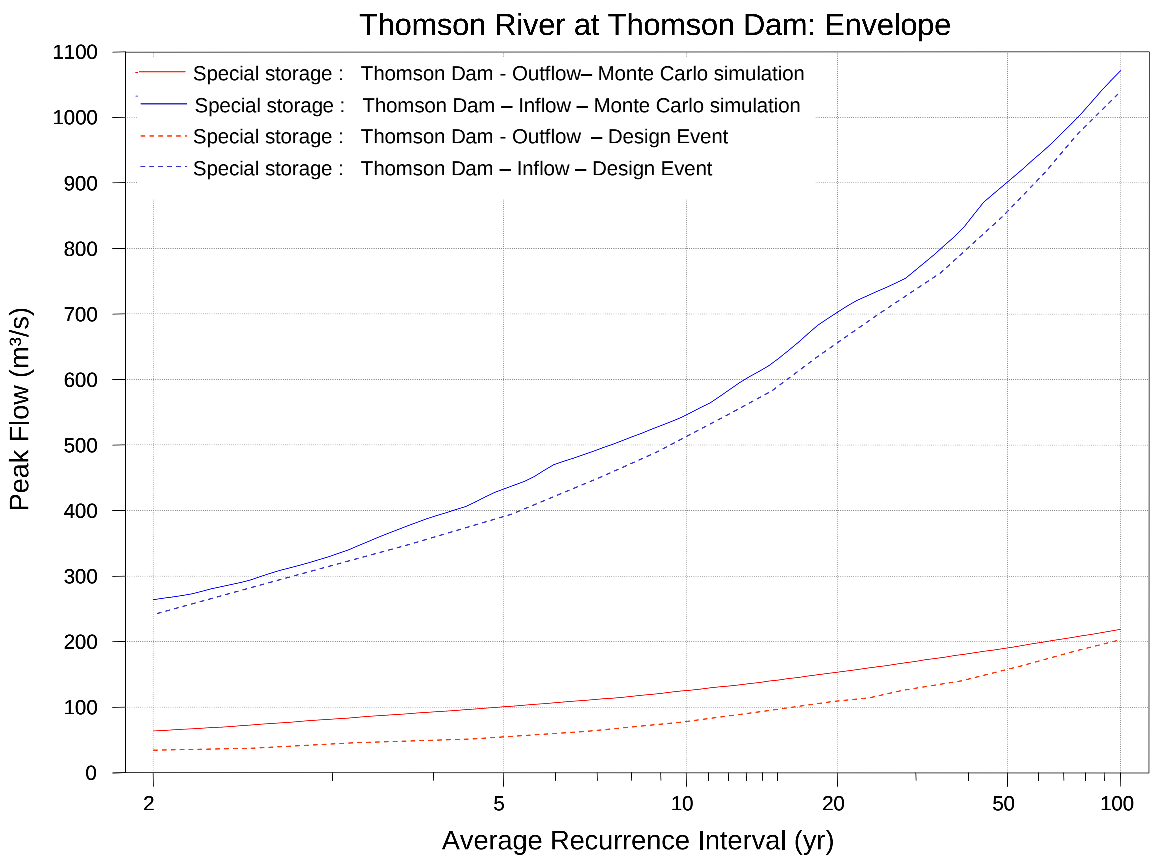
If the plot option is selected (either from the Parameter Specification dialogue or the Text Editor) then flood frequency curves are generated for either multiple durations at a single site, or else a single duration (including the envelope curve) at multiple sites. Two sample plots are provided below:




It the left-hand plot above it is seen that the envelope curve of critical durations changes between the 12 and then 24 hour events, and in the right-hand plot the difference in flood frequency curves between inflows and outflows is markedly evident.

It is worth noting that both initial loss and temporal patterns area treated as fixed values, then the above process yields flood frequency curves that are equivalent to the traditional design event approach. The plot below provides a comparison between the two approaches, and in this instance it is seen that the Monte Carlo approach yields slightly higher results.



# 11 ACKNOWLEDGEMENTS

The main change in this version of RORB is the addition of the graphical editor. It is a pleasure to acknowledge here the work of Robert Morden and David Stephens (SKM), who created the editor and its documentation.

The development of both Versions 5 & 6 of RORB were greatly facilitated by the financial support of Melbourne Water. Their willingness to making the program freely available is acknowledged with great thanks.

Special thanks are also due to Erwin Weinmann (Monash University) and George Kuczera (University of Newcastle) who helped with the development of the Monte-Carlo simulation scheme used in Version 5.

The Bureau of Meteorology's assistance in providing pluviograph data for the Monte Carlo example is also much appreciated.

Version 4.7 featured the Windows Interface developed by Tony Jones, in conjuction with Tony Wong; that version was the ‘flagship’ for nearly ten years, and a tremendous contribution to the continued standing of the program. It is a pleasure to acknowledge their role, and Tony Wong’s continuing contribution as a member of the RORB ‘team’.

We thank the many users over the years those who drew our attention to programming ‘bugs’ and invite them and others to do so again if bugs are found in the current version.

The authors acknowledge with thanks the important initial role played by ACADS in facilitating the use of RORB by a wide range of Australian authorities and consultants; the program was distributed through ACADS until September 1990.

Many people have helped with RORB over its 30 year life and it is not possible to mention more than a few by name. Assistance in programming was rendered by Gerard Garlick, Jacki Turney and Peter Boland (Version 3) and by Mark Besley and Andrew Haines (Version 4). Andrew has been the Monash manager of RORB for nearly 20 years. The late Joy Helm typed the Version 4 User Manual; much of her work prevails in the current revision. We acknowledge their contributions with appreciation.

# 12 REFERENCES


1 MEIN, R.G., E.M. LAURENSON and T.A. McMAHON, Simple Nonlinear Model for Flood Estimation, *Journal of Hydraulics Division*, American Society of Civil Engineers, Vol. 100, 1974, No. HY11, pp. 1507-1518.

2 LAURENSON, E.M., A Catchment Storage Model for Runoff Routing, *Journal of Hydrology*, Vol. 2, 1964,, pp. 141-163.

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# APPENDIX A: RUNOFF ROUTING CONCEPTS

The purpose of this appendix is to introduce readers to some of the concepts of runoff-routing as they apply to RORB, i.e. to help explain why the model was formulated the way it was. It begins (Section A.1) by showing how a series of concentrated storages can represent the attenuation and translation effects of a catchment on the rainfall-excess hyetograph. Section A.2 provides theoretical and empirical justification of the power function equation $S = kQ^m$ used to simulate reach storage-discharge behaviour and indicates the factors affecting $k$ values. Some justification for splitting the parameter $k$ into catchment ($k_c$) and reach ($k_r$) components is given in Section A.3, along with the ideas behind the use of $d_{av}$ as an intermediate parameter in the model. To conclude, Section A.4 summarizes the main advantages of the approach adopted in the RORB program.

## A.1 Simulation of Catchment Behaviour

We begin by considering a storm large enough to produce runoff at the outlet of a catchment. If the hyetograph of rainfall-excess (i.e. the rainfall hyetograph less the losses due to infiltration, etc.) is plotted *on the same scale as* the observed hydrograph at the catchment outlet, the resulting figure shows the relationship of one to the other. Figure A-1 shows typical `input' and `output' curves in hydrograph units.

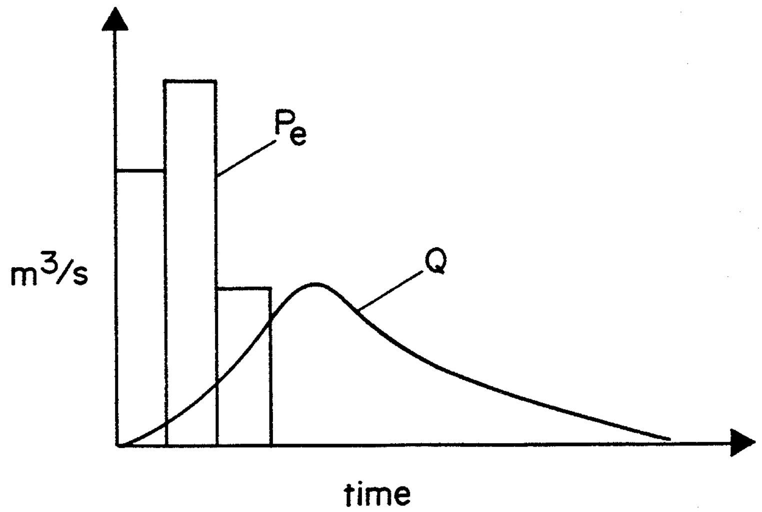


**Figure A-1 The Effect of Catchment Storage**

In Figure A-1, the volumes beneath the rainfall-excess hyetograph and the runoff `hydrograph' are equal. However, the latter is much flatter than the former, and its peak occurs later in time. That is, the effect of a catchment on the input hydrograph is like that of a storage in that it causes:

- *attenuation* of the input pattern; and
- *translation* of peak flows in time.

As shown in Figure A-2, the attenuation and translation effects of a catchment on the rainfall-excess hyetograph can be *simulated* by routing the input through a series of concentrated storages. The first routing through such a storage produces a hydrograph which peaks on the falling limb of the input hyetograph. The routing through the second storage produces further attenuation, and a peak more remote from the original input. Additional routings continue to flatten the hydrograph and to shift its peak further to the right. The routing of the rainfall-excess hyetograph through a string of concentrated storages thus can be seen to simulate the effect of a catchment on its rainfall-excess input.

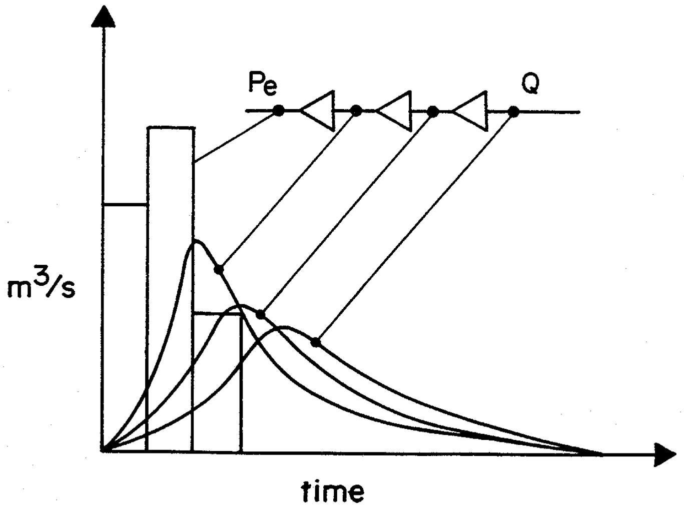


**Figure A-2 Simulation of Catchment Effect with a Series of Concentrated Storages**

Consider now Figure A-3, in which the concentrated storages have been placed on the major stream reaches of the catchment, which itself has been divided into a number of sub-areas draining to the stream system. The figure shows that the rainfall-excess for sub-area A, entered at the node on A, is routed through a series of storages on its way to the catchment outlet; runoff from sub-area B is acted on by the storages which lie on the streams between it and the outlet. Similarly for C, D, and E. With this formulation, the rainfall-excess hyetographs from different parts of the catchment are routed through an amount of concentrated storage which depends on the remoteness of each area from the outlet.

As will be explained in Section A.4, such subdivision of the catchment provides the opportunity to model spatial variability of rainfall and losses, different reach characteristics, and separate allowance for major storages which may exist (or are proposed) on the catchment.

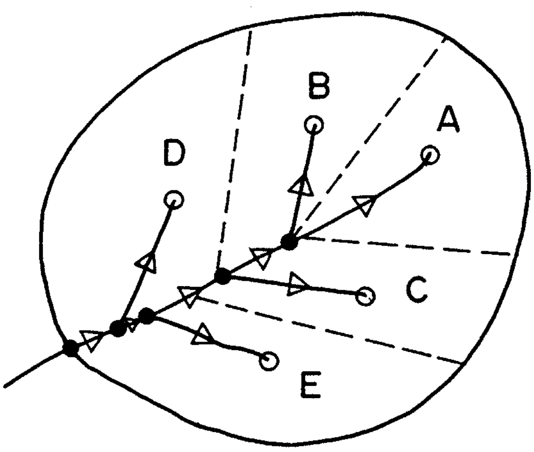


**Figure A-3 Distributed Storage Network to Represent a Catchment**

## A.2 Representation of Reach Storage ($S = kQ^m$)

The form of the storage equation $S = kQ^m$ is justifiable by both theoretical and empirical means. Theoretically, application of a uniform flow equation in a prismatic channel will yield an equation of this form as shown below.

Consider Figure A-4 which shows a reach (length, $L$) of a channel with a triangular cross-section. The depth of flow in the channel is $y$, the Manning roughness is $n$, the bed slope is $S_b$ and the channel side slopes are $z$ horizontal to 1 vertical. The water 'stored' in the channel is given by the product of the cross-section (area $A$) and the length, i.e.

$$S = AL = zy^2L \qquad \textit{Equation A-1}$$

Using Manning's equation, the discharge $Q$ can be expressed as

$$Q = vA = \frac{1}{n}\left(\frac{zy^2}{2y\sqrt{1+z^2}}\right)^{2/3} S_b^{1/2} zy^2 \qquad \textit{Equation A-2}$$

*Equation A-1* and Equation A-2 can be combined to eliminate $y$. The result is an equation in the form $S = kQ^m$ where $m = 0.75$ and

$$k = \frac{n^{0.75}(2\sqrt{1+z^2})^{0.5} L}{z^{0.25} S_b^{0.375}} \qquad \textit{Equation A-3}$$

Similar analyses can be made for other cross-section shapes (e.g. Ref. 1)

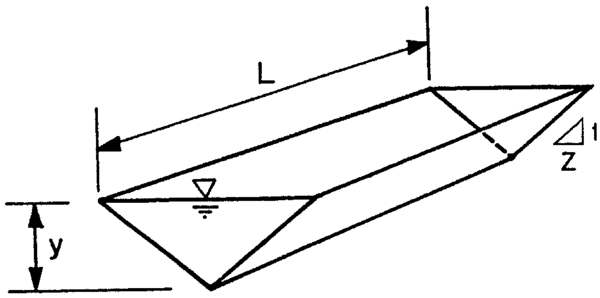


**Figure A-4 Uniform Flow in a Prismatic Channel of Triangular Cross-section**

### A.2.1 Value of exponent *m*

Analyses like the one above, can be performed for other cross-section shapes to produce the following exponent values:

| | |
|---|---|
| Triangular cross-section | m = 0.75 |
| Trapezoidal cross-section | m = 0.74 |
| Parabolic cross-section | m = 0.69 |
| Wide rectangular channel | m = 0.60 |

The results are for *uniform flow* in open channels. However, similar *m* values, ranging from 0.68 to 0.8, have been reported in *field studies* using natural catchments (Ref. 1); these studies included two events with overbank flow. More recent experience with runoff-routing models has led the authors to adopt an 'average' *m* value of 0.8, although other values in the range 0.6-1.0 are frequently encountered. [It might be noted here that an *m* value of 1.0 implies a *linear* model of catchment response.]

### A.2.2 Value of *k*

The relation for *k* given by Equation A-3 shows the nature of its dependence on channel roughness, cross-section shape, bed-slope, and length. Such a relationship could be of use for routing in *prismatic* channels where these parameters can be estimated. In *natural* river channels the effects of slope and roughness often tend to be compensatory; in such cases, the *k* value can be taken as proportional to *L*. In *urban* catchments with lined channels, this compensating effect is missing so the effect of slope on reach storage must be specifically included.

## A.3 Splitting *k* into $k_c$ and $k_r$

The time between the occurrence of a particular element of rainfall-excess at a particular point on the catchment and its effect at the outlet is called the storage delay time for that point. On a linear catchment, it is constant for any point but on a nonlinear one it varies with discharge.

It was proposed by Laurenson (Ref. 2) that the *lag* of a catchment (i.e. the time between the centroid of rainfall-excess and the centroid of the resulting surface runoff) was equal to the *average storage delay time* for all elements of rainfall-excess throughout the storm and over the entire catchment. Assuming areally uniform rainfall-excess, this means that lag is equal to the storage delay time of points on the catchment corresponding to the centroid of the time-area diagram. On nonlinear catchments, both the lag and the storage delay times of all points were assumed to vary with discharge. It was also assumed that the storage-delay time value of the centroid of the time-area diagram varied with outlet discharge in the same way that lag varied with mean outlet discharge of the flood from which it was derived. This latter relation was found empirically to have the form

$$t = k_c\, Q^{\,p} \qquad \textit{Equation A-4}$$

where $t$ = lag($h$) and $Q$ = mean outlet discharge $m^3/s$ and $k_c$ and $p$ are constants, $p$ having a value of the order of -0.25. Adopting a catchment storage model of the form shown in Figure A-3, the storage delay time (in hours) of a storage i is thus assumed to have the form

$$k_i = k_c\, k_r\, Q^{\,p} \qquad \textit{Equation A-5}$$

implying a storage function

$$S_i = 3600\, k_c\, k_r\, Q^{p+1} \qquad \textit{Equation A-6}$$

where $k_r$ is the *relative* delay time of storage i, i.e. the ratio of its delay time to that of the centroid of the time-area diagram and $S_i$ is in $m^3$.

The RORB program computes the catchment mean travel distance (the distance from the outlet to the centroid of the time-area diagram) from the reach lengths input in the catchment data file. If there are $n$ sub-catchments and the **i**th one has area $a_i$ (km$^2$) and a travel distance to the *catchment* outlet $d_i$(km), then the mean travel distance $d$ is given by:

$$d_{av} = \sum_{i=1}^{n} (a_i d_i)/A \qquad \textit{Equation A-7}$$

where A is the total catchment area (km$^2$). Considering the catchment storage between two adjacent nodes of the model, **i** upstream and **j** downstream, its $k_r$ value is given in dimensionless form by

$$k_r = (d_i - d_j)/d_{av} \qquad \textit{Equation A-8}$$

Such relative delay times will have values of the order of 0.1.

With $k_r$ expressed in its dimensionless form, $k_c$ becomes equal to the coefficient of the formula for delay time of the centroid of the time-area diagram and of the lag-mean discharge formula (*Equation A-4*). The advantage of this is that $k_c$ can be estimated directly from lag formulae.

## A.4 Advantages of the Runoff Routing Approach

The main advantage of the runoff-routing procedure over methods which are based on unitgraphs is its **flexibility**; the catchment formulation adopted in RORB can easily account for:

- variability of rainfall depth over the catchment;
- variability of rainfall pattern over the catchment; variability of losses over a catchment;
- nonlinearity of catchment response;

- the modelling of existing or proposed storages in the catchment, using equations appropriate to each specific storage;
- the modelling of the effects of reaches `drowned out' by reservoir inundation;
- the modelling of flows in branched networks;
- the effects of urbanization of all, or part of, the catchment; and,
- the ability to represent other forms of runoff-routing model.

This flexibility comes from the subdivision of the catchment into sub-areas (assumed homogenous), the routing of the runoff from each part of the catchment through the appropriate amount of reach storage, and the separate modelling of significant artificial or natural storages . The formulation allows for the output of calculated hydrographs at any point in the catchment, and thus for the user to compute design flows for any region of interest.

*Authors' Note*: The power function form of the storage-discharge relation (Section A.2), while valid for most applications, may not always be the most suitable for a stream reach. The user should decide – using FIT runs and judgement - whether the formulation is suited for the task at hand.

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