A HYDRODYNAMIC FLOOD ROUTING MODEL FOR ONE DIMENSIONAL FLOW

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DECLARATION

I declare that this project report is my own, unaided work. It is being submitted for the Degree of Master of Science in Engineering in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

(Signature of candidate)

23rd day of December 1986

A B S T R A C T

A flood routing model based on the complete de St. Venant equations for unsteady one dimensional flow is described in this project report. The model enables the user to determine the degree of attenuation of the flood peak through a single river reach as well as the maximum water levels reached by the flood. The model does not require the estimation of routing constants, the physical characteristics of the river as defined for a normal backwater computation being given, and can therefore be applied to ungauged watercourses with more confidence than the more approximate flood routing models. The model is based on an implicit finite difference scheme, being Verwey's variant of the Preissman Scheme. It uses the double sweep algorithm for the solution of the difference equations and is therefore restricted to subcritical flows.

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A - cross-sectional area (L^2)
A1, A2 - coefficients used in discretized flow equations

Bs - channel width (L)
B1, B2 - coefficients used in discretized flow equations

c - propagation velocity (L/T)
C1, C2 - coefficients used in discretized flow equations

D - diffusion coefficient
D1, D2 - coefficients used in discretized flow equations

E1, E2 - coefficients used in discretized flow equations

F - coefficient used in double sweep algorithm
F_f - bed friction force acting on control volume (F)
F_g - gravitational force acting on control volume (F)
F_p - pressure force acting on control volume (F)
F_r - Froude number

g - acceleration due to gravity (L/T^2)
G - coefficient used in double sweep algorithm

h - water depth (L)
H - coefficient used in double sweep algorithm

i - computational point index
I - coefficient used in double sweep algorithm

J - coefficient used in double sweep algorithm

K - conveyance (L^3/T)
K - travel time parameter

m - routing coefficient

N - Manning coefficient

n - *time step index*

P - wetted perimeter (L)

Q - volumetric water discharge (L^3/T)

Q_o - uniform flow rate for given depth of flow (L^3/T)

q - continuous lateral inflow per unit length (L^2/T)

R - hydraulic radius (L)

S_f - energy line slope in x-direction (friction slope)

S_o - bed slope in x-direction

T wave period (T)

t - time (T)

Δt - time between two computational intervals (T)

u - water velocity in the x-di[illegible] [illegible])

u_o - uniform flow velocity (L/T)

u_q - component of lateral inflow velocity in the x-direction

x - longitudinal space co-ordinate (L)

Δx - d[illegible]ance between two computational poins in x[illegible]ection (L)

y - vertical space co-ordinate above datum (L)

- water surface elevation (L)

y_b - bed elevation (L)

y_w - weir crest level (L)

α - routing coeffient

- non-uniform kinetic energy [illegible]ction coefficient

β - non-uniform momentum correct[illegible] coefficient

η - distance of cross-section centroid from water surface (L)

θ	- weighting coefficient in finite difference approximations of functions and their space derivatives
ρ	- water mass density (M/L^3)
σ	- width of cross-section (L)
Φ	- combined resistance and bed slope terms
ψ	- weighting coefficient in finite difference approximations of functions and their time derivatives
τ	- dimension less wave period
γ	- weir discharge coefficient
ε	- coefficient used in double sweep algorithm

1. INTRODUCTION AND LITERATURE REVIEW

1.1 Introduction

The ravages of flooding through the ages have cost man dearly in terms of life and property. Villages and portions of towns situated on the banks of rivers have been washed away. Fertile lands on river flood plains have been destroyed by the deposition of silt by the sediment-laden flood-waters. Disruptions to communications and transport links and essential services have also resulted from severe flooding.

Knowledge of past flood events along a river provides the incentive for taking adequate flood protection measures and the curtailment of development on unprotected flood plains. Where cities have existed for centuries on the banks of rivers this knowledge has accumulated so as to provide a sound statistical basis for flood prediction. In a developing country such as South Africa flood records covering periods longer than sixty years at a particular river gauging station are rare. Little historical knowledge is therefore available for many rivers and watercourses regarding the probability distribution of the maximum flood levels reached during a flood event.

As development in a watershed occurs, the run-off characteristics of the watershed change. In general the same storm event will result in a run-off hydrograph from a developed watershed with a higher peak and shorter time to peak than that from the watershed in its undeveloped state. Furthermore, with the development in the catchment there is increasing utilization of flood plains of the watercourses for industrial, residential and agricultural purposes as the value of the land increases. Without judicious flood plain management this would result in drastic increases in damage as a result of flooding.

Statutory limitations on township development in flood plains do exist. For any watercourse traversing a proposed township development in South Africa it is a requirement of the Water Act (No. 54 of 1956 as amended) that the twenty year floodline for the watercourse be determined. In some provinces additional restrictions are applied; in the Transvaal the 50 year floodline is required by the Director of Local Government. Although these floodlines do not serve as limits of the flood plain in which no development whatsoever is permitted, special flood protection measures are required where the developer wishes to extend into this area.

Through the use of the urban drainage models such as WITWAT and ILLUDAS, better descriptions of the run-off process within developed catchments are being obtained, facilitating the design of the stormwater systems for these areas. However, the point at which the stormwater systems discharge into the natural watercourse is generally the limit of applicability of these models.

Once the characteristics of the flood hydrograph entering a particular reach of a natural watercourse have been estimated, the determination of the maximum levels reached by the flood is a hydraulic problem rather than a hydrological one. In some cases it is sufficient to carry out a backwater computation using the peak flow obtained from the hydrological analysis to determine these levels, thus assuming steady flow conditions. The effect of a volume of run-off entering a river channel in a relatively short period is to create a flood wave which progresses down the river. This unsteady flow condition can give rise to two effects ignored by the steady flow assumption. Firstly, if at a single point in the river reach observations of discharge and stage are made during the passage of the flood wave, the stage discharge or rating curve formed from a plot of the results will generally exhibit a looped form. Thus for any particular water level two flows are measured, the higher during the period that the flood is rising, the lower when the flood is falling. This is explained by the fact that the water surface slope is steeper before the peak of the floodwave than it is after the peak. The result of this is that often the peak flow does not coincide with the maximum flood level, the latter occurring some time after the peak flow.

The second effect ignored by the steady state assumption is that the shape of the flood wave changes as it progresses downstream.

The degree of change depends on the shape of the inflow hydrograph and the properties of the river channel. The most important of these properties are the amount of storage in the reach relative to the volume of the flood and the travel time through the reach relative to the duration of the flood. In the case of watercourses with wide flood plains which drain developed catchments characterized by sharp-peaked run-off hydrographs attenuation can be significant.

The process by which the characteristics such as the speed, shape and height of a flood wave in a channel are traced as these vary with time is termed flood routing. The use of flood routing techniques to determine the maximum flood levels or the peak flow along a reach can lead to narrower floodline zoning along a watercourse and more economical design of bridges and structures within the flood plain than would be obtained using simple time lag routing and assuming steady flow conditions. Indeed, in the conditions described above where attentuation is significant, the savings will also be significant.

A number flood routing techniques of varying complexity have been developed. At the one end of the scale are the simplified hydrological routing techniques with which the discharge hydrographs at intervals along the reach can be obtained. The Muskingum method is a well-known and commonly used example of this type of routing technique. At the other end of the scale are the so-called complete dynamic models based on the complete hydrodynamic equations for unsteady flow and with which the discharge and water level can be traced at all points along the reach for the duration of the flood. Obviously the data requirements for these latter techniques are greater than for the simpler techniques. However, correct reproduction of flood wave motion using the simplified techniques is dependent upon having historical flood data for the reach in question with which to determine the routing constants required. This pertains especially to those simplified techniques based on the kinematic wave equation, discussed futher in Section 1.2.1, where the attenuation obtained in the solution arises solely out of the numerical damping inherent in the technique.

While calibration of the complete dynamic models is also not possible without historical flood data, in the absence of flood records the complete dynamic models will provide a better simulation of the flood wave motion than the simplified techniques. Whereas the routing constants for the simplified techniques have to be "guesstimated" in these situations, the complete dynamic models utilise actual cross-section data in the routing procedure, although the channel and flood plain roughness coefficients have to be estimated, and a better description of the hydraulic characteristics of the reach is obtained.

The second disadvantage with the use of the simplified techniques is that in most cases a backwater computation is still required to determine the maximum flood levels along the reach. To carry out this backwater computation cross-section data and estimates of the channel and flood plain roughness coefficients are required. The data requirements for carrying out a simplified flood routing procedure and backwater computation are thus almost the same as those required for a run of a complete dynamic model.

A third disadvantage with the simplified models is that their range of applicability is limited in comparison with the complete dynamic models. The kinematic models are limited to relatively steep watercourses where backwater effects are negligible and to mildly sloped hydrographs. The simplified models based on the diffusion equation, which are discussed further in Section 1.2.2, have a far greater range of applicability than the kinematic models and are suitable for most flood routing exercises. However, the diffusion models do not offer any real advantage over the complete dynamic models in terms of computational speed or simplicity of input data when applied to natural channels of varying cross-section.

The objective of this project has thus been to develop a complete dynamic flood routing model for general application to single channel reaches which utilises the actual cross-sections of the watercourse in the routing procedure and from which maximum flood levels along the reach can be obtained. The model is based on an implicit finite difference scheme, being Verwey's variant of the

Preissmann scheme. As is discussed in Section 1.3.5, implicit finite difference schemes are far more suitable than explicit schemes for flood routing computations because the size of the time step used in the computation is restricted with the latter. Verwey's scheme is a relatively uncomplicated version of the tried and tested Preissmann's scheme.

The model was developed on a Hewlett Packard HP9845T desktop computer but has been converted to run on the Hewlett Packard Series 200 and Series 300 machines as well. Running time on the HP9845T machine is about 0,1 seconds per time step per cross-section; on the HP9816 machine it is about 0,18 seconds per time step per cross-section.

Data requirements for the model are the river cross-sections, the inflow hydrograph and the initial conditions in the river, the latter being in the form of the flow and water level at each cross-section.

The model is also suitable for inclusion in [illegible]ed or basin model as the channel routing component. [illegible] models are being increasingly used by local authorities for the purpose of determining floodlines for entire drainage systems and, in addition, provide them with the facility to study the effects of land use changes, canalisation and the introduction of dams on the system. There are obvious economies to be obtained where floodlines are determined for the entire watershed by the local authority concerned by means of a watershed model rather than on a piecemeal basis in individual townships.

Watershed models generally incorporate both the hydrological and hydraulic aspects of flood determination. The watershed is subdivided into subcatchments linked by channels. The hydrological characteristics of each sub-catchment are entered into the model such that for any given storm event an outlet hydrograph from the sub-catchment is obtained. Starting at the upstream end of the uppermost reach the inflow hydrograph to the reach is routed through the reach using the streamflow routing procedure. The outflow from the reach is then added to the outflow from the local sub-catchment to provide the inflow hydrograph to the next reach. The process is continued, adding any tributary inflows that may occur, until the total of all the routed hydrographs has been obtained at the watershed outlet. Reservoirs can be included in the model as reaches for which, if necessary, different routing procedures can be specified. In watersheds where there are a number of reservoirs in the drainage system, watershed modelling provides the *only* reliable solution technique for determining the response of the system to various storm inputs.

Heggen (1983) describes a watershed modelling technique which uses the Puls reservoir routing and the Muskingum channel routing technique. Other more sophisticated watershed models are mentioned in Section 1.4.

In Section 1.2 of this report the various approximate flood routing procedures and their applicability are discussed. The so-called complete dynamic models are presented in Section 1.3 and in Section 1.4 the reasons for the selection of Verwey's implicit finite difference scheme are discussed.

In Chapter 2 the finite difference scheme of Verwey is described. The de St. Venant equations governing gradually varied flow for use with natural channels are derived, the discretization of the equations is presented and the double sweep algorithm used in the solution and the treatment of the boundary conditions described.

In Chapter 3 the model is described with reference to the preparation of data and the selection of the time step and weighting coefficients. The effects of varying the time step weighting coefficients are presented and the various forms of output are described.

The application of the model to two systems is described in Chapter 4. In the application to the Swartvlei estuary and lake system a tidal condition is used as the downstream boundary condition and the results of the model runs are compared with measured data. The second application is to a reach of the Rietspruit watercourse in the Transvaal, under conditions of the 50 year flood. The maximum levels reached by the flood are compared with those obtained from a steady flow assumption.

A summary of the report is given in Chapter 5, together with the conclusion.

1.2 A Review of Approximate Flood Routing Methods

The motion of a flood wave in a river channel is a form of gradually varied unsteady flow; the two partial differential equations describing this flow as fist published by Barre de St. Venant in 1871 are as follows :

$$\frac{\delta A}{\delta t} + \frac{\delta Q}{\delta x} = 0 \qquad (1.1)$$

$$\frac{1}{g}\ \frac{\delta u}{\delta t} + \frac{u}{g}\ \frac{\delta u}{\delta x} + \frac{\delta h}{\delta x} + S_f - S_o = 0 \qquad (1.2)$$

Where A = cross-sectional area, u = mean velocity, h = depth of flow, g = gravitational acceleration, S_f = friction slope, S_o = bed slope, x = distance along the channel and t = time. In this form the equations do not include for lateral inflow to the channel.

Equation (1.1) is a statement of the conservation of mass in the system and is termed the continuity equation. Equation (1.2) is derived from considerations of the conservation of momentum and is termed the momentum equation or, more frequently, the dynamic equation, since it is seldom a true statement of momentum conservation. These equations are derived in the form used in the model developed in this project in Chapter 2.

The de St.Venant equations cannot be analytically integrated because of their non-linear nature and numerical integration techniques offer the only solution procedure for practical situations. While all flood routing models use the continuity equation in a form similar to that given by equation (1.1) different groups of models can be distinguished by the number of terms considered in the momentum equation. Following the nomenclature of Weinmann and Laurenson (1979), models which retain all terms in the momentum equation are called complete dynamic models. These are discussed in Section 1.3. While being the most complete, these models are also the most demanding on computer resources and approximate models which produce results at considerably less expense have been developed. On the other hand the approximate models are limited in their generality and accuracy and with large memory, high-speed desk top computers common in engineering offices the complete dynamic models are likely to become more popular. The group of models in which the inertia terms $^1/_g\ ^{\delta u}/_{\delta t}$ and $^u/_g\ ^{\delta u}/_{\delta x}$ are omitted from the dynamic equations are called approximate dynamic models, diffusion analogy models or kinematic models corrected for dynamic effects, depending on the form in which the equations are expressed. These models are discussed in Section 1.2.2. Further simplification of the dynamic equation results in the equation

$$S_f = S_o \tag{1.3}$$

which forms the basis of the kinematic wave models, also used extensively in overland flow modelling.

By using a general channel flow formula such as the Chezy or Manning formula of the form

$$Q = K\sqrt{S_f} \tag{1.4}$$

Where $Q = uA$ is the discharge and K is the conveyance, and by expressing the conveyance in terms of the uniform flow where Q_o

$$Q_o = K\sqrt{S_o} \tag{1.5}$$

an expression of the following form is obtained.

$$Q = Q_o\sqrt{\frac{S_f}{S_o}}$$

By substituting for S_f from the dynamic equation, Weinmann and Laurenson (1979) obtain an equation for a looped rating curve which is useful for illustrating the differe ces between the various flood routing models :

$$Q = Q_o \sqrt{1 - \frac{1}{S_o}\frac{\delta y}{\delta x} - \frac{u}{S_o g}\frac{\delta u}{\delta x} - \frac{1}{S_o g}\frac{\delta u}{\delta x}} \quad (1.6)$$

kinematic wave ________|
diffusion analogy ______________|
complete dynamic wave ________________________________|

A typical looped rating curve is shown in Figure 1.1.

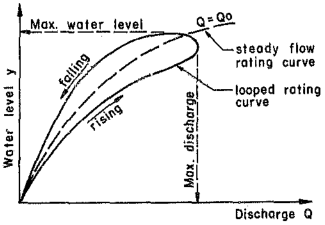

FIGURE 1.1 Typical looped rating curve

1.2.1 Kinematic models

The combination of the continuity equation and Equation (1.3) yields the kinematic wave equation

$$\frac{1}{c}\frac{\delta Q}{\delta t} + \frac{\delta Q}{\delta x} = 0 \quad (1.7)$$

in which the coefficient c, called *the kinematic wave speed*, can be determined at a particular cross-section at distance x along the reach from the Kleitz-Seddon Law

$$c = \frac{(dQ)}{(dA)x} = \frac{1}{Bs} \text{[illegible]} x \tag{1.8}$$

Where $B_s = {}^{dA}/_{dy}$ is the surface width at section x. The derivative $\frac{(dQ)}{(dy)}$ in this equation must be determined from the steady flow rating function in order for equation (1.7) to be equivalent to the combination of the continuity and uniform flow equations.

Cunge (1969) has developed an explicit finite difference scheme for Equation (1.7) which introduces a weighting coefficient in the finite difference approximation of the time derivative. The scheme is similar to the box schemes used in the implicit complete dynamic models, discussed further in Section 1.3.4. The kinematic wave equation is applied at a point within the four point grid as shown in Figure 1.2.

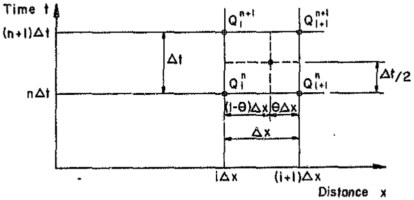

FIGURE 1.2 Computational grid for kinematic models

Using the superscripts n and n+1 to denote the value of the variables at time level $n\Delta t$ and $(n+1)\Delta t$ respectively and the subscripts i and i+1 to denote the value of the variable at distance $i\Delta x$ and $(i+1)\Delta x$ respectively, the discretized form of the equation is :

$$\frac{1}{<c>}\;\frac{\theta(Q_i^{n+1}-Q_i^n)+(1-\theta)(Q_{i+1}^{n+1}-Q_{i+1}^n)}{\Delta t}+\frac{Q_{i+1}^{n+1}-Q_i^{n+1}+Q_{i+1}^{n+1}-Q_i^n}{2\,\Delta x}$$

$$= 0 \qquad (1.9)$$

In which $\langle\frac{1}{c}\rangle = \frac{1}{<c>}$ where $<c>$ is the average value of c for the reach and is independent of time. By introducing the travel time parameter
$k = \Delta x/_{<c>}$ this equation can be reduced to the classical Muskingum equation by McCarthy :

$$Q_{i+1}^{n+1} = C_1\;Q_i^n + C_2 Q_i^{n+1} + C_3\;Q_{i+1}^n \qquad (1.10)$$

Where $C_1 = \dfrac{k\theta + \frac{\Delta t}{2}}{k(1-\theta) + \frac{\Delta t}{2}}$; $C_2 = \dfrac{-k\theta + \frac{\Delta t}{2}}{k(1-\theta) + \frac{\Delta t}{2}}$; $C_3 = \dfrac{k(1-\theta) - \frac{\Delta t}{2}}{k(1-\theta) + \frac{\Delta t}{2}}$

and $C_1 + C_2 + C_3 = 1$

The value of k can be determined from observed floods or by means of Equation (1.8). The value of θ can only be determined from observed floods. Since these values have not been determined from physical considerations, extrapolation is risky.

The kinematic wave equation, equation (1.7), does not enable attenuation of the flood wave to be represented; the attenuation obtained with the Muskingum method is numerical, due to its being a poor approximation of Equation (1.7). Cunge shows that the Muskingum equation is also an approximation of the diffusion equation

$$\frac{\delta Q}{\delta t} + c\,\frac{\delta Q}{\delta x} = D\;\frac{\delta^2 Q}{\delta x^2} \qquad (1.11)$$

Where D is the diffusion coefficient, if the parameter θ is evaluated from

$$\theta = \frac{1}{2}\left(1 - \frac{Q}{B_s S_o c \Delta x}\right) \; ; \quad 0 \leq \theta \leq 0.5 \tag{1.12}$$

The diffusion equation allows for flood wave attenuation and relates it to the physical characteristics of the river. It is discussed further in Section 1.2.2.

By retaining the time derivatives in the kinematic wave equation (1.7) and using finite difference approximations for the space derivatives, Koussis (1976) shows that equation (1.7) may be written as an ordinary differential equation with a Muskingum type solution given by Equation (1.10) but with coefficients

$$C_1 = \frac{\Delta x}{\langle c \rangle \Delta t}\,(1 - \gamma) - \gamma \; ; \quad C_2 = 1 - \frac{\Delta x}{\langle c \rangle \Delta t}\,(1 - \gamma) \; ; \quad C_3 = \gamma$$

$$\text{Where } \gamma = \exp\left(- \frac{\langle c \rangle \Delta t}{\Delta x (1 - \theta)}\right) \text{ and } C_1 + C_2 + C_3 = 1 \tag{1.13}$$

Koussis shows this to be a generalized kinematic routing model reducible to other well-known kinematic models, the Muskingum and Kalinin-Miljukov models, by appropriate selection of the routing parameters. In the flood routing procedure proposed by Koussis, dynamic effects are considered by introducing a looped rating curve of the form given by the Jones formula,

$$Q = Q_N \sqrt{1 + \frac{1}{cS_o} \frac{\delta y}{\delta t}} \tag{1.14}$$

to give improved estimates of the travel speed parameter $\langle c \rangle$ for the sub-reach. The procedure is an iterative one since the travel speed parameter $\langle c \rangle$ is first estimated for the subreach and the hydrograph is routed through the subreach. The stage hydrograph is then obtained for the downstream end of the subreach and used to compute the relevant segments of the looped rating curve at the downstream end of the subreach. The slopes of the looped rating curves then yield the improved value of <c> for the reach.

Weinmann and Laurenson (1979) describe this model as a kinematic model corrected for dynamic effects and show that it gives the closest approximation to the complete dynamic model of all the kinematic models. Weinmann has developed a general computer program which allows the user to select the kinematic model version and the input and output options that best suit his application.

Peterson and Verhoff (1982) have developed an equation of the Muskingum type which is still more general than that used in the Koussis model (Equation (1.13)). The generality results because the coefficients were not obtained from an approximation of a differential equation (Equation 1.7) but from mathematical approximations of the conservation of mass.

Huang (1978) presents an alternative approach to the solution of the kinematic wave equation to those which reduce to the Muskingum form. By expressing equation (1.3) in the form

$$Q = \alpha A^{m} \qquad (1.15)$$

Where α and m are routing coefficients whose values depend on the channel properties, the continuity equation (1.7) can be written as

$$\frac{\delta A}{\delta t} + \alpha m A^{m-1} \frac{\delta A}{\delta x} = 0 \qquad (1.16)$$

The reach is divided into a number of sub-reaches for which the coefficients and m are assumed to be constant. These coefficients are determined by application of the Manning formula to the typical cross-section for the sub-reach. A method is given for determining the values of α and m from topographic maps when data on discharge and flow area is not available. For natural channels, where the values of α and m vary with the flow area, an appropriate set of routing coefficients are selected at each time level based on the flow area at the previous time level.

Huang presents two finite difference schemes for the solution of equation (1.16), termed the linear and non-linear routing procedures. In the linear routing procedure the following discretizations are used :

$$\frac{\delta A}{\delta t} = (A_{i+1}^{n+1} - A_{i+1}^{n}) / \Delta t \tag{1.17}$$

$$\frac{\delta A}{\delta x} = (A_{i+1}^{n+1} - A_{i}^{n+1}) / \Delta x \tag{1.18}$$

$$A^{m-1} = [(A_{i}^{n+1} + A_{i+1}^{n})/2]^{m-1} \tag{1.19}$$

By substituting these equations into equation (1.16), an explicit equation relating the unknown A_{i+1}^{n+1} to the known values A_i^{n+1} , A_i^n and A_{i+1}^n is obtained. The solution thus proceeds without iteration.

In the non-linear procedure, first developed by Li, Simons and Stevens (1975), the continuity equation is first discretized using equation (1.17) and

$$\frac{\delta Q}{\delta x} = (Q_{i+1}^{n+1} - Q_{i}^{n+1})/\Delta x \tag{1.20}$$

before the following substitutions, obtained from equation (1.15) are made :

$$Q_{i+1}^{n+1} = \alpha \, (A_{i+1}^{n+1})^{m}$$

$$Q_{i}^{n+1} = \alpha \, (A_{i}^{n+1})^{m} \tag{1.21}$$

The resulting equation is non-linear in terms of the unknown A_{i+1}^{n+1} and must be solved iteratively. Using the linear solution as the first approximation, Huang presents an iterative procedure which converges rapidly, three iterations usually being sufficient.

In a comparison of the non-linear and linear routing procedures Huang found that for time steps of 15 minutes the solution obtained using the two procedures agreed quiet closely. When a longer time step of one hour was used a marked difference between the two solutions appeared, that from the non-linear procedure being closer to the solutions obtained using the 15 minute time step. Huang thus recommends the use of the linear routing procedure with short time steps. The method is simple and has been succesfully incorporated into a watershed model.

1.2.2 Approximate Dynamic and Diffusion Analogy Models

Where the continuity equation and the indicated part of Equation (1.6) are solved directly by the same numerical methods as used with complete models, the resulting models are called approximate dynamic models. However, since these models require almost the same computational effort as the complete models they do not hold any advantage over the complete models except that they have simpler equations for the coefficients in the solution procedure and are thus easier to programme.

Diffusion analogy models, on the other hand, are those where the continuity and simplified dynamic equations are combined into a single equation of the diffusion type, expressed either in terms of flow depth or discharge. For example, written in terms of the discharge the equation is

$$\frac{\delta Q}{\delta t} + \frac{\delta Q}{\delta x} = D \frac{\delta^2 Q}{\delta x^2} \qquad (1.22)$$

Where c is the convection speed and D the diffusion coefficient, both being constants. The diffusion method was derived by Hayami (1951) for application to flooding in irregular river channels. The convection speed c in this equation is not strictly a typical value of the wave speed averaged along the reach and over a certain range of discharge; it depends also on the degree of attenuation in the reach. The success of the method depends on a knowledge of c and D for the reach; a disadvantage is that these parameters can vary significantly with the magnitude of the flood.

In the variable parameter diffusion method developed by Price (1973) the parameters c and D are defined as functions of the discharge by correlating values of c and D calculated for a number of recorded floods with the average peak discharge along the reach in each case.

1.2.3 Applicability of the Approximate Flood Routing Methods

The relative importance of the various terms in the dynamic equation depends on the characteristics of the inflow flood hydrograph and the river system under consideration. In flood routing computations the inertia terms are generally small and can be ignored without significantly affecting the solution.

For example, a typical reach of the Rietspruit has a bed slope of about 0,4%. A flood with a 50 year recurrence interval has a peak of about 200 cumecs and results in values of the terms $\frac{1}{g}\frac{\delta u}{\delta t}$ and $\frac{u}{g}\frac{\delta u}{\delta x}$ of about 0,01% for a point midway on the rising limb of the hydrograph, not significant in comparison with the bed slope. Figures given by Miller and Cunge (1975) for situations where the inertia terms were expected to be significant show that in fact they could have been omitted. The approximate dynamic or diffusion analogy models would therefore seem to be sufficiently accurate for most flood routing purposes.

However, in situations where the maximum water levels reached by the flood are of interest, omission of the convective acceleration term $\frac{u}{g}\frac{\delta u}{\delta x}$ results in the incorrect representation of the backwater profile under steady flow conditions. The dynamic equation with the local acceleration term omitted can be written

$$\frac{\delta}{\delta x}\left(h + \frac{u^2}{2g}\right) + S_f - S_o = 0 \qquad (1.23)$$

which, under steady state conditions, is the energy equation for backwater profiles. Thus by omitting the $\frac{u}{g}\ \frac{\delta u}{\delta x}$ term the influence of the velocity head on the steady state water surface profile is ignored.

In a comparison of the Muskingum-Cunge, linear diffusion and variable parameter diffusion methods under conditions typical for flooding in British rivers, Price (1973) concluded that the Muskingum-Cunge method is generally as accurate as the linear diffusion method in all circumstances and because it is simpler conceptually preferred its use to the linear diffusion method. He found that the variable parameter diffusion method has some advantage in accuracy where there is inundation of a large flood plain and when a sequence of floods with a wide range of peak discharges is being routed. However, the advantages of the variable parameter diffusion method were outweighed in many cases by the more complicated computer programming required.

The distinguishing feature of the kinematic models such as the Muskingum-Cunge model is that the discharge is always equal to the normal discharge and is thus a single valued function of the depth. Backwater effects are thus completely ignored in these models. Weinmann and Laurenson (1979) show that for channels with well-developed loops in the rating curve, errors of up to 25% in the computed peak flow can arise with the use of a kinematic model. They conclude that :

1. The omission of the acceleration terms in approximate models will not significantly effect the accuracy of the flood routing results in normal circumstances.
2. For slowly rising hydrographs and moderately steep channels a model of the kinematic type can be expected to give results that differ only slightly from the ones obtained form more complete models.
3. In channels of flat grade or with steep hydrographs, or both, the pressure term $\frac{\delta y}{\delta x}$ is significant and a kinematic type model should not be used.

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By considering small sinusoidal perturbations to the equilibrium flow using linearised equations, Ponce, Li and Simons (1978) found the limit of applicability of the kinematic model, to be given by

$$T > \frac{\tau h_o}{S_o u_o} \qquad (1.24)$$

Where T = wave period of sinusoidal perturbation, equivalent in practical situations to the flood wave duration, h_o = uniform flow depth, u_o = uniform flow velocity, τ = dimensionless wave period of unsteady component of motion. They found that for at least 95% accuracy of the kinematic wave equation the dimensionless period τ has to be greater than 171. Thus for mild channel slopes the period has to be very long for the kinematic model to apply. Similarly the limit of applicability of the diffusion model was found to be

$$T \geq \frac{30}{S_o}\sqrt{\frac{h_o}{g}} \qquad (1.25)$$

They conclude that the diffusion model applies for a wider range of slopes and periods than the kinematic model with the added advantage that the diffusion model does allow for physical attenuation.

For the example of the Rietspruit mentioned above, and which is discussed further in Chapter 4, the flood wave duration would have to be more than eight hours for the kinematic model to apply and more than one hour for the diffusion model to apply.

1.3 Complete Dynamic Models

The two methods in general use for the numerical solution of the complete de St.Venant equations are the method of characteristics, based on the characteristic form of the equations, and the finite difference methods, which are based on the partial differential equations as derived.

1.3.1 The method of characteristics

Abbott (1975) describes the method of characteristics as a technique whereby the problem of solving *two simultaneous partial differential equations* can be replaced by the problem of solving four ordinary *differential equations*.

The characteristic form of the de St. Venant equations are :

$$\frac{dx}{dt} = u + c \tag{1.26}$$

$$\{\frac{\delta}{\delta t} + (u + c)\frac{\delta}{\delta x}\}(u + 2c) = -g(S_f - S_o) \tag{1.27}$$

$$\frac{dx}{dt} = u - c \tag{1.28}$$

$$\{\frac{\delta}{\delta t} + (u - c)\frac{\delta}{\delta x}\}(u - 2c) = -g(S_f - S_o) \tag{1.29}$$

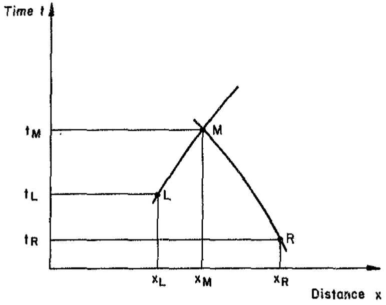

FIGURE 1.3 Basis of the characteristic method

Characteristics can be defined as lines in the (x,t) plane along which disturbances propagate. The differentiation operators in equations (1.27) and (1.29) are the *total* derivatives along the characteristics. Equations (1.26) and (1.27) define the so-called forward characteristics and equations (1.28) and (1.29) defining the backward characteristics. The solution of the characteristic equations is obtained at the intersection of the forward and backward characteristics by integrating along the characteristics. Thus for any point M in the (x,t) plane shown in Figure 1.3 at which the forward and backward characteristics from points L and R respectively intersect, the coordinates of point M and the values of the unknown flow variables u_M and y_M can be found from

$$x_M = x_L + \int_{t_L}^{t_M} (u + c)\, dt$$

$$u_M + 2c_M = u_L + 2c_L + \int_{t_L}^{t_M} g(S_o - S_f)\, dt$$

$$x_M = x_R + \int_{t_R}^{t_M} (u - c) dt$$

$$u_M - 2c_M = u_R - 2c_R + \int_{t_R}^{t_M} g(S_o - S_f) dt \qquad (1.30)$$

In taking integrals along the characteristics, no approximation is made compared to equations (1.26) to (1.29). The approximation arises in the evaluation of the integrals, usually by means of the trapezoidal rule. Four non-linear algebraic equations are obtained which are solved by iteration. Since the value of the co-ordinates (x_M, t_M) obtained in this way must be determined for each intersection point a variable grid of points is obtained in the (x, t) plane as is shown in Figure 1.4. This method is referred to as the variable grid method.

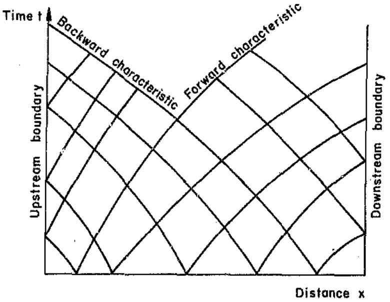

FIGURE 1.4 Characteristics on a variable grid

Liggett and Cunge (1975) provide an algorithm for the solution of the characteristic equations with rapid convergence and a good degree of accuracy.

The variable grid method requires two kinds of interpolation when applied to natural channels. Firstly the geometric and hydraulic characteristics of the channel are known or defined *only at a limited number* of sections but are needed at any point of the (x,t)-plane. Secondly, the computed results must be interpolated to obtain hydrographs at any point or free surface profiles at any time.

Abbott (1979) describes the three-point method of characteristics, with which values of the dependent variables can be obtained at fixed grid intervals, and the four-point method of *characteristics*, the latter being generally superior to the three-point method for machine computation. However, Abbott and Verwey (1970) recommend the use of implicit finite difference techniques rather than the method of *characteristics for* flow in natural watercourses.

According to Cunge, Holly and Verwey (1980) the original variable grid method of characteristics is not widely used for industrial modelling because of its complexity and the need to interpolate. Characteristics on fixed grid are employed somewhat more frequently but the method is complex and more costly than finite difference methods, without offering any improvement in accuracy.

However, the characteristic method does have two important fields of application. Firstly, it is used as a standard for other methods, since its solution may be brought as close to the true solution of the basic equations as is required. Secondly, it is used as a means of representing boundary conditions in methods which cannot compute all flow variables at exterior or interior boundary points of the model.

1.3.2 Finite Difference Methods

In finite difference methods the differential equations of flow are replaced by algebraic finite difference relationships which are solved at a finite number of grid points in the (x,t)-plane termed the computational grid.

There are two basic types of finite difference schemes. In the explicit schemes the equations are arranged to solve for one point at a time, generally making use of the values of the dependent variables determined at the previous time step at a few adjacent points. In implicit finite difference schemes a system of equations at the new time step involving all the points in the reach is solved simultaneously.

Liggett and Cunge (1975) describe in detail a number of different schemes of each type, namely the Diffusive, Leap-frog, Lax- Wendroff and Dronkers explicit Schemes and the Preissman, Vasiliev and Abbott-Ionescu implicit schemes.

1.3.3 *Explicit Finite Difference Schemes*

The explicit finite difference schemes are the simplest to program, *but are subject to* limitations on the size of the time step used in the computation, as is discussed further below.

In the *Diffusive or Lax Scheme* the discretization of the derivative is as follows

$$\frac{\delta f}{\delta t} = \frac{f_i^{n+1} - \left[\theta f_i^n + \frac{(1-\theta)}{2}(f_{i+1}^n + f_{i-1}^n)\right]}{\Delta t} \qquad (1.31)$$

and

$$\frac{\delta f}{\delta x} = \frac{(f_{i+1}^n - f_{i-1}^n)}{2\Delta x}$$

With $\theta = 1$ this scheme is unstable; with $\theta = 0$ the scheme represents the so-called diffusive scheme.

Using this discretization in the de St. Venant equations the solution at the point $(i, n+1)$ *for depth comes* immediately from the continuity equation and for velocity from the dynamic equation. It can be seen from equation 1.31 and Figure 1.5(a) that the solution at the point $(i, n+1)$ *is dependent* on that at $(i-1, n)$ and at $(i+1, n)$. The solution at the boundary points must therefore be determined using the characteristics method.

In the Leap-frog scheme centered difference approximations are used in both distance and time i.e.

$$\frac{\delta f}{\delta t} = \frac{(f_i^{n+1} - f_i^{n-1})}{2\Delta t} \qquad (1.32)$$

$$\frac{\delta f}{\delta x} = \frac{(f_{i+1}^n - f_{i-1}^n)}{2\Delta x}$$

The Leap-frog method is probably the earliest one ever used for one-dimensional flow modelling. It produces values of the unknowns at all points in the grid and is of second order accuracy. A disadvantage with the method is that the solution obtained is a saw-toothed line since the points where the dependent variables are computed are alternately odd and even, the solution at the odd points being independent of that at the even points. The discretization for two adjacent points is shown in Figure 1.5(b).

The Lax-Wendroff scheme was first applied to the open channel unsteady flow equations by Houghton and Karahora. It is a second order scheme which is not dissipative - it does not smooth out initial perturbations in the flow and the initial conditions provided must satisfy the flow equations as closely as possible. The scheme is described in detail by Liggett & Cunge (1975) for application to a trapezoidal channel.

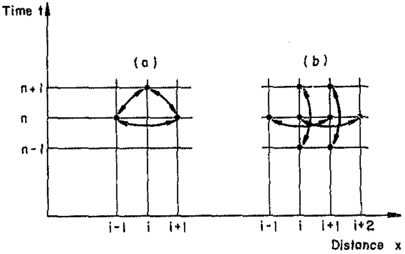

FIGURE 1.5 Explicit discretization schemes
(a) Diffusive (b) Leap-frog

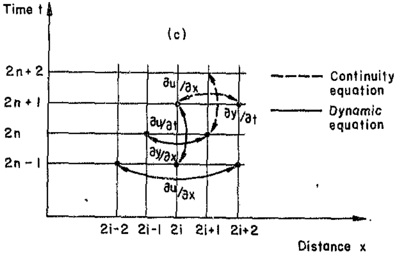

FIGURE 5 Explicit discretization schemes
(a) Diffusive (b) Leap-frog (c) Dronkers

In Dronker's explicit scheme, developed mainly for tidal flow computations, different discretizations are used in the dynamic and continuity equations.
For the dynamic equation

$$\frac{\delta u}{\delta x} = \frac{u_{2i+2}^{2n-1} - u_{2i-2}^{2n-1}}{4\Delta x} \; ; \; \frac{\delta y}{\delta x} = \frac{y_{2i+1}^{2n} - y_{2i-1}^{2n}}{2\Delta x} \tag{1.33}$$

and $\frac{\delta u}{\delta t} = \frac{u_{2i}^{2n+1} - u_{2i}^{2n-1}}{2\Delta t}$

while for the continuity *equation*

$$\frac{\delta u}{\delta x} = \frac{u_{2i+2}^{2n+1} - u_{2i}^{2n+1}}{2\Delta x} \; ; \; \frac{\delta y}{\delta t} = \frac{y_{2i+2}^{2n+2} - y_{2i+1}^{2n}}{2\Delta t} \tag{1.34}$$

It can be seen from the above discretizations, shown graphically in Figure 1.5(c) that the difference equations cover three time-steps, from level $(2n-1)\Delta t$ to $(2n+2)\Delta t$, and four distance steps, *from* $(2i-2)\Delta x$ to $(2i+2)\Delta x$. It is obvious that this scheme is valid only when flow variations and river geometry variations are slow and gentle in space and time.

The major disadvantage with explicit schemes is that the time step used in the computation is limited by the Courant-Friederichs-Lewy condition for stability :

$$\left[uo \pm \sqrt{gho}\right] \frac{\Delta t}{\Delta x} \leq 1 \qquad (1.35)$$

Huang and Song (1985) describe a second instability in the characteristic, diffusive and leap-frog schemes given by the Koren equation :

$$\Delta t \leq \frac{\sqrt{1+2\left|\frac{uo}{co}\right|}-1}{\left|\frac{uo}{co}\right|\left|\frac{gSo}{uo}\right|}$$

where u_o and c_o are the initial velocity and speed of the shallow water wave respectively. They found that this stability criterion can be increasingly relaxed the more the energy loss term is treated implicity. The Koren stability criterion becomes very restrictive when the Froude number, given by $\frac{u_o}{c_o}$ is small.

The combination of the two stability criteria given by equation 1.35 and 1.36 defines the zones of stability and instability in the selection of Δt and Δx shown in Figure 1.6.

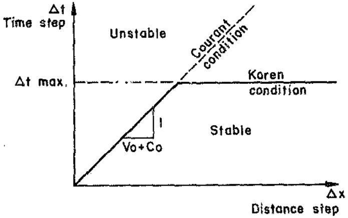

FIGURE 1.6 Zones of stability and instability for explicit schemes

In explicit finite difference schemes special attention must be given to determining the value of the unknown dependent variables at the boundaries. The method of characteristics is the only general technique for finding these boundary values. The general rule for the number of conditions specified on the boundary or initial line is that it must equal the number of characteristics originating on the boundary or initial line. Obviously two conditions will always be required along the initial line. In supercritical flow situations two characteristics originate at the upstream boundary and two upstream boundary conditions must be specified; no downstream boundary condition is required, the solution at a point on the boundary being determined from the characteristics intersecting at the point.

In subcritical flow situations the forward characteristic originates at the upstream boundary and the backward characteristic at the downstream boundary. One boundary condition must therefore be given at each boundary and the other determined using the equation for the backward characteristic at the upstream boundary and the forward characteristic at the downstream boundary.

1.3.4 Implicit finite difference methods

The implicit methods of finite differences were developed because of the limitations imposed on the time step using explicit schemes and have no limit on the time step, providing that a sufficient number of points on the hydrograph are obtained to define it adequately.

A number of implicit schemes have been developed; generally these differ in the discretization of the terms in the equations. A system of linear algebraic equations is obtained for all computational points which is solved at each time level, with the boundary conditions linearized in the dependent variables closing the system. Generally the matrix of coefficients is sparse, with non-zero elements banded on the diagonal and fast methods of solution such as the double sweep method described in Section 2.2 can be used.

The *Preissmann*, Amein and Verwey schemes are so-called four-point *or box schemes* with the flow equation applied at a point centred *within* the four point grid as shown in Figure 1.7.

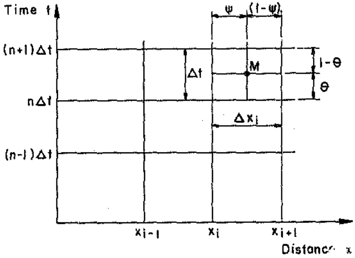

FIGURE 7 Four point grid for box schemes (1.7)

At the point M the average value and the partial derivatives of a function f are expressed by

$$f(M) = (1 - \psi)(1 - \theta)\ f_i^n + (1 - \psi)\theta\ f_i^{n+1} + \psi(1 - \theta)\ f_{i+1}^n + \psi\theta f_{i+1}^{n+1}$$

$$\frac{\delta f(M)}{\delta x} = (1 - \theta)\ \frac{f_{i+1}^n - f_i^n}{\Delta x} + \theta\ \frac{f_{i+1}^{n+1} - f_i^{n+1}}{\Delta x}$$

$$\frac{\delta f(M)}{\delta t} = (1 - \psi)\ \frac{f_i^{n+1} - f_i^n}{\Delta t} + \psi\ \frac{f_{i+1}^{n+1} - f_{i+1}^n}{\Delta t} \qquad (1.37)$$

A feature of the box scheme is that centred *difference approximations* are used for both time and space *derivatives* whereas other schemes use combinations of forward, centred *and* backward difference approximations. In the *scheme of* Amein *and* Fang (1970) $\theta = \psi = \frac{1}{2}$. Generally in the *Preissmann and* the Verwey schemes $\psi = \frac{1}{2}$ and $\frac{1}{2} \leq \theta \leq 1$

In the Preissmann scheme the discretized flow equations are linearized in terms of Δy and ΔQ by putting

$$y_i^{n+1} = y_i^n + \Delta y_i$$

$$Q_i^{n+1} = Q_i^n + \Delta Q_i \qquad (1.38)$$

and developing the terms of the *equations* by means of power series expansions with second and higher order terms neglected. The continuity and dynamic equations at each point reduce to the form

$$A_i \Delta Y_{i+1} + B_i \Delta Q_{i+1} = C_i \Delta Y_i + D_i \Delta Q_i + E_i$$

and (1.39)

$$F_i \Delta Y_{i+1} + G_i \Delta Q_{i+1} = H_i \Delta Q_{i+1} + I_i \Delta Q_i + J_i$$

With the boundary conditions linesrized in y and Q the system *of equations* is solved at each time step. Using the known values of the previous time step the coefficients A_i to J_i can be computed and the set of equations solved, providing the *second approximation* to the unknowns. The significant feature of the scheme is that in most cases the second approximation is sufficiently accurate and thus only one iteration is required at each time *step*.

In Verwey's scheme the equations are written in terms of Y_i^{n+1} , Q_i^{n+1} , Y_{i+1}^{n+1} and Q_{i+1}^{n+1} and are of the form

$$A_i Y_i^{n+1} + B_i Q_i^{n+1} + C_i Y_{i+1}^{n+1} + D_i Q_{i+1}^{n+1} = E_i$$

$$F_i Y_i^{n+1} + G_i Q_i^{n+1} + H_i Y_i^{n+1} + I_i Q_{i+1}^{n+1} = J_i \qquad (1.40)$$

In the first iteration the coefficients are evaluated by approximating the values of Y_i^{n+1} and Q_i^{n+1} with the known values Y_i^n and Q_i^n . *The coefficients are then* adjusted using the values of Y_i^{n+1} and Q_i^{n+1} obtained from the first iteration and the second iteration carried out. As a rule two iterations per time step are needed to give a sufficiently accurate simulation.

In the scheme of Amein the equations, in terms of y and u, are written

$$F_i(Y_i^{n+1}, U_i^{n+1}, Y_{i+1}^{n+1}, U_{i+1}^{n+1}) = 0$$

$$G_i(Y_i^{n+1}, U_i^{n+1}, Y_{i+1}^{n+1}, U_{i+1}^{n+1}) = 0 \qquad (1.41)$$

The solution proceeds by assigning trial values to the unknowns, usually the values from the previous time step. With these trial values the right hand sides of the equations will be non-zero, acquiring values known as the residuals. The residuals and the partial derivatives of Equation (1.41) are related according to the generalized Newton iteration method by

$$\frac{\delta F_i}{\delta Y_i} dy_i + \frac{\delta F_i}{\delta U_i} dU_i + \frac{\delta F_i}{\delta Y_{i+1}} dY_{i+1} + \frac{\delta F_i}{\delta U_{i+1}} dU_{i+1} = R_{1,i}$$

and (1.42)

$$\frac{\delta G_i}{\delta Y_i} dY_i + \frac{\delta G_i}{\delta U_i} + \frac{\delta G_i}{\delta Y_{i+1}} dY_{i+1} + \frac{\delta G_i}{\delta U_{i+1}} dU_{i+1} = R_{2,i}$$

Where $R_{1,i}$ and $R_{2,i}$ are the residuals associated with the functions Fi and Gi respectively and

$$dY_i = Y_{i,\,k+1} - Y_{i,k} \quad ; \quad dU_i = U_{i,\,k+1} - U_{i,k} \qquad (1.43)$$

Where $Y_{i,k}$ and $U_{i,k}$ are the values of the unknowns after the k^{th} iteration.

The system of equations (1.42) are solved and revised values for the unknowns are obtained and the procedure is repeated until the difference in values of any unknown in two consecutive iteration cycles falls below a tolerance limit. The number of iterations required to provide a reasonable solution is not stated.

Other implicit schemes which are not based on the box or four-point method are the Vasiliev scheme, the Abbott-Ionescu scheme, the Delft Hydraulics Laboratory Scheme and the Gunaratnam-Perkins Scheme.

The Vasiliev and Gunaratnam-Perkins Schemes are similar to the box schemes in that the values of the dependent variable are determined at each grid point. The discretization is different, however.

In the Vasiliev Scheme the discretization of the flow equations is based on

$$\frac{\delta f}{\delta t} \cong (f_i^{n+1} - f_i^n)/\Delta t \; ; \; \frac{\delta f}{\delta x} = (f_{i+1}^{n+1} - f_{i-1}^{n+1})/2\Delta x$$

The scheme is applied to the continuity equation in its usual form but with the dynamic equation in the following form

$$\frac{\delta Q}{\delta t} + 2u\frac{\delta Q}{\delta x} + (C^2 - U^2)B\,\frac{\delta y}{\delta x} = \phi$$

Where $c = \sqrt{g\frac{A}{B}}$ and Φ = resistance and slope terms. For N computational points and 2N unknown dependent variables a system of 2N - 4 equation is obtained. Two boundary conditions and two characteristic equations written for the limit points i = 1 and i = N close the system, which is then solved using the double sweep algorithm.

The Gunaratnam-Perkins scheme is a fully implicit scheme which links together three consecutive points i-1, i and i+1 using the discretization

$$\frac{\delta f}{\delta t} \cong \frac{1}{6}(f_{i-1}^{n+1} - f_{i-1}^{n})/\Delta t + \frac{2}{3}(f_i^{n+1} - f_i^n)/\Delta t + \frac{1}{6}(f_{i+1}^{n+1} - f_{i+1}^{n})/\Delta t$$

$$\frac{\delta f}{\delta x} \cong (f_{i+1}^{n+1} - f_{i-1}^{n+1})/2\Delta x$$

As with the Vasiliev scheme, two boundary conditions and two characteristic equations are required to close the system of equations.

The Delft Hydraulics Laboratory and Abbott-Ionescu *schemes are* similar in that the computational grid is divided alternately into "y-points" at which water stages are computed and "Q-points" at which the *flow* is *computed. The Delft* scheme is based on the concept of computational cells at the centre of which the water stages are computed and which are linked to adjacent *cells on the left and* right through discharge laws. In both schemes the space derivatives are the weighted mean of those at time level $n\Delta t$ and $(n+1)\Delta t$ using the weighting *coefficient* θ :

$$\frac{\delta f}{\delta x} = \theta(f_{i+1}^{n+1} - f_{i-1}^{n+1})/2\Delta x + (1-\theta)(f_{i+1}^{n} - f_{i-1}^{n})/2\Delta x$$

where $0{,}5 \leq \theta \leq 1{,}0$ in the Delft scheme and $\theta = 0{,}5$ in the Abbott-Ionescu scheme.

In the Abbott-Ionescu scheme the coefficients of the discretized equations are evaluated at time level $(n+\frac{1}{2})\Delta t$, requiring an *iterative solution procedure.* In the first iteration the valu of the coefficients are evaluated from the flow variables at time level $n\Delta t$. The set of equations is then solved *using* the double sweep algorithm. In the second iteration the coefficients are evaluated at time level $(n+\frac{1}{2})\Delta t$ using the values of the flow variables at $(n\Delta t)$ and *those at* $(n+1)\Delta t$ obtained from the first iteration. As for Verwey's scheme, two iterations per time step give satisfactory results.

1.3.5 Comparison of explicit and implicit schemes

Price (1974) compares the accuracy and efficiency of the Leap-frog explicit method, the two-step Lax-Wendroff *explicit method* the four point implicit scheme of Amein and the fixed mesh characteristic method, using a monoclinal wave for which the exact analytical solution of the *full equations is known.* The monoclinical wave bears a strong resemblance to a flood wave and in very long rivers the front of a flood wave may take the *form of a monoclinical wave.* Price found the explicit methods became unstable when the time step exceeded that given by the

Courant-Friederichs-Lewy condition and the characteristic method became unstable for time steps in excess of ten times this value. No instability was recorded using the implicit method. In the comparison Price adopted a 40 metre wide prismatic channel of bed slopes 0,001 and 0,00025, typical of British rivers, and of 100 km length. He used distance steps in the range 2 500 metres to 20 000 metres and time-steps in the range 180 to 14 400 seconds. In all methods used the smallest error was achieved for the smallest value of the distance step.

He found that the implicit method is markedly more efficient than any other method for a similar accuracy, the reason being that the least error was obtained with the implicit method for greater time steps.

Chaudhry (1979) lists the advantages and disadvantages of explicit and implicit finite difference schemes as follows :

1. Stability - Explicit schemes are conditionally stable; the Courant-Friederichs-Lewy condition must be satisfied. Implicit schemes are unconditionally stable.
2. Ease of programming - the explicit method is easier to programme.
3. Economy - with the larger time step allowed with implicit methods less computer time is required in the computations. However, seen in the broader context of the total cost of preparing the model and analysing the results, this aspect is not always important.
4. Computer memory requirements - usually more storage is required in an implicit method than in an explicit method.
5. Simulation of special cases - where conduits have closed tops, eg. in stormwater or sewer system and tailrace tunnels, the free surface width can become very small and the size of time step must be reduced accordingly - the implicit schemes should be used in these cases. On the other hand implicit schemes usually fail to represent supercritical flow so that when the Froude number becomes greater than one during computation the results may be unstable.

6. Simulation of sharp peaks - because of the smaller time step explicit methods are generally more suitable. With the same size of time step computational time with implicit schemes would be greater.
7. Formation of bores and shocks - the explicit schemes are more suitable for the analysis of transients in which a bore forms.

1.4. Selection of flood routing method

The selection of the flood routing method used in the model has been based on the requirement that the model provide both the stage and the flow at all points along the reach. A secondary requirement is that the model be suitable for inclusion in a watershed modelling system as the channel routing component.

The MOPSET watershed model described by Huang (1978) uses the kinematic method for routing floods down natural channels. The kinematic method is the simplest of the flood routing methods but its range of applicability is limited. Since it is based on a single-valued stage-discharge relationship at all points in the reach the kinematic method cannot reproduce the dynamic effect of flood wave propagation nor any backwater effects in the reach.

Akan and Yen (1981) provide a method for routing floods through channel networks using the diffusion analogy model. In a comparison with the complete dynamic and kinematic methods applied to the same channel network, they found the diffusion analogy model to be nearly as accurate as the complete dynamic method and faster and cheaper in computation than the kinematic method. However, although the diffusion analogy model will simulate the backwater effects at channel junctions etc., its exclusion of the convective acceleration term $\frac{u}{g}\frac{\delta u}{\delta x}$ can result in inaccuracies in the water surface profile obtained, especially in natural channels where differences in the velocity head at adjacent sections can be significant.

McMahon, Fitzgerald and McCarthy (1984) describe the BRASS (Basin runoff and streamflow simulation) model which incorporates the complete dynamic channel routing model developed by Fread (1978). The model is capable of simulating lateral inflows to streams and dendritic (branched) river systems. The disadvantage with the use of complete dynamic routing models in a watershed model is that the dynamic routing models are large programs requiring considerable core memory allocation for execution. Watershed models such as BRASS are therefore divided into segments with a root segment which resides in memory throughout a simulation. A second disadvantage is that the complete dynamic models are generally somewhat slower computationally than the approximate models. The models based on explicit finite difference schemes are slower because of the limitations on the time step imposed by the Courant-Lewy-Friedrichs condition, while those based on implicit finite difference schemes are slower because of their greater complexity. However, with the rapid advances being made in computer technology and high speed 32-bit computers becoming increasingly available in desk-top form, these disadvantages are not considered significant.

The model developed in this project has therefore been based on the complete dynamic form of the de St. Venant equations. An implicit finite difference scheme can be seen to have advantages over an explicit scheme for application to a general purpose flood routing model where the flow is generally subcritical in that relatively large time steps can be used in the computation. The Preissmann scheme has been extensively used in industrial models. The formulation of the coefficients used in the scheme is extremely complex, however, and Verwey's variant of the scheme has been used for this project.

Although the model developed in this project is for application to single channel reaches, the solution technique can be applied to dendritic channel networks after some modification.

Cunge, Holley and Verwey (1980) and Joliffe (1984) describe the application of implicit finite difference schemes to channel networks.

2. DESCRIPTION OF THE FINITE DIFFERENCE SCHEME OF VERWEY

The de St. Venant equations are derived for natural channels of irregular cross-section in Section 2.1 of this chapter. The discretization of the flow equations is presented in Section 2.2 and the double sweep algorithm used in the solution of the set of equations obtained from the discretization is presented in Section 2.3. Finally, the treatment of the boundary conditions is discussed in Section 2.4.

2.1 The de St. Venant Equations For One Dimensional Flow in Natural Channels

The de St Venant equations are based on the following hypotheses:

i) The flow is one-dimensional i.e. the velocity is uniform over the cross section and the water level across the section is horizontal.

ii) The streamline curvature is small and vertical accelerations are negligible, hence the pressure is hydrostatic.

iii) The effects of boundary friction and turbulence can be accounted for through resistance laws analogous to those used for steady state flows.

iv) The average channel bed slope is small so that the cosine of the angle it makes with the horizontal may be replaced by unity.

The co-ordinate system and definition of terms used in this report are shown in Figure 2.1. Two physical laws are used to derive the equations for unsteady flow in open channels, conservation of mass and conservation of momentum.

With respect to the control volume shown in Figure 2.1 the law of conservation of mass can be formulated as follows :

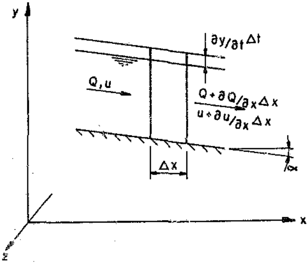

Figure 2 : Definition Sketch

Nett mass inflow into control volume = change in mass storage

$$\rho Q\Delta t - \rho(Q + \frac{\Delta Q}{\delta x}\Delta x)\Delta t = \rho(A + \frac{\delta A}{\delta t}\Delta t)\Delta x - \rho A\Delta x$$

or

$$\frac{\delta Q}{\delta x} + \frac{\delta A}{\delta t} = 0$$

Expressing this equation in terms of water level by using

$$\frac{\delta A}{\delta t} = \frac{\delta A}{\delta y}\frac{\delta y}{\delta t}$$

and putting $B_s = \frac{\delta A}{\delta y}$ the continuity equation is obtained.

$$\frac{\delta Q}{\delta x} + B_s \frac{\delta y}{\delta t} = 0 \qquad (2.1)$$

The momentum equation can be derived by equating the sum of the *nett rate of momentum* entering the control volume and the forces acting on it with the rate of change of momentum within the control volume.

Net rate of momentum entering the control volume

$$= Qu - (Qu + \frac{\delta(Qu)}{\delta x}\Delta x)$$

$$= -\frac{\delta(Qu)}{\delta x}\Delta x$$

Three types of forces are considered to act on the control volume, namely *gravity, pressure and friction.*

Gravity : $Fg = \rho gA\Delta x \sin x = - \rho gASo$

$$\text{Pressure : } Fp = F_p - (F_p + \frac{\delta F_p}{\delta x}\Delta x) = - \frac{\delta F_p}{\delta x}\Delta x$$

$$= \frac{\delta}{\delta x} \int^h \rho g(h(x)-\eta)\sigma(x,\eta)d\eta\Delta x$$

$$= -\rho g\Delta x \int^h \frac{\delta}{\delta x} (h(x)-\eta)\sigma(x,\eta)d\eta$$

$$= -\rho g\Delta x \left| \frac{\delta h}{\delta x} \int^h \sigma(x,\eta)d\eta + \int^h (h - \eta)\frac{\delta\sigma}{\delta x} (x,\eta) \, d\eta \right|$$

$$= -\rho g\Delta x \left| \frac{\delta h}{\delta x} A + \int^h (h - \eta) \frac{\delta\sigma}{\delta x} (x, \eta)d\eta \right|$$

The second term on the right hand side of this equation represents an increase in the pressure force as a result of a change in the width of the channel; in prismatic channels it is assumed to be zero. In non-prismatic channels as the channel widens or narrows the banks contribute an additional pressure force which exactly cancels this term.

Thus $F_p = -\rho gA \frac{\delta h}{\delta x}\Delta x$

Friction : $F_f = \rho gA \, S_f\Delta x$

where S_f is the friction slope.

The rate of change of momentum within the control volume can be written as

Rate of change of momentum $= \frac{\delta}{\delta t} (\rho Q)\Delta x$

Combining these elements gives the momentum equation

$$-\rho\frac{\delta Qu}{\delta x}\Delta x - \rho gASo\Delta x - \rho gA\frac{\delta h}{\delta x}\Delta x + \rho gAS_f\Delta x$$

$$= \frac{\delta}{\delta t} (\rho Q)\Delta x$$

Dividing through by ρ and Δx and rearranging gives

$$\frac{\delta Q}{\delta t} + \frac{\delta(Qu)}{\delta x} + gA\,\frac{\delta h}{\delta x} + gA(S_o - S_f) = 0$$

When working with natural channel cross sections it is convenient to choose *Q and y, the water* level, as the dependent variables.
Putting $h = y - y_b$ where y_b is the bed level,

$$\text{then } \frac{\delta h}{\delta x} = \frac{\delta y}{\delta x} - \frac{\delta yb}{\delta x} = \frac{\delta y}{\delta x} - S_o$$

Futhermore the velocity *u can be expressed as* Q/A.

In addition a momentum correction factor β, called the Boussinesq coefficient, should be *applied where* working with compound cross-sections with non-uniform velocity distribution:

$$\beta = \int_0^b u_z^{\,2} h_z dZ / (\bar{u}^2 A) \quad ,\beta = \beta(y) \qquad (2.2)$$

Where the subscripts Z denote local values of depth-averaged velocity and depth at position Z in the *cross-section.*

With these changes the momentum equation becomes

$$\frac{\delta Q}{\delta t} + \frac{\delta}{\delta x}\left(\frac{\beta Q^2}{A}\right) + gA\frac{\delta y}{\delta x} - gAS_f = 0 \qquad (2.3)$$

In this form the equation is generally referred to as the '*dynamic*' equation since it is seldom a true statement of momentum conservation.

The continuity and momentum equations presented above can be expanded to include for lateral inflow, which may be of the form of a distributed inflow to the channel from overland or groundwater sources, concentrated inflow at tributaries or distributed ouflows due to seepage losses.

Insofar as the continuity equation is concerned, for a distributed inflow of q per unit of channel length a volume of $q\Delta x\Delta t$ enters the control volume during the infinitesmal period Δt.

The continuity equation thus becomes :

$$\frac{\delta Q}{\delta x} + B_s \frac{\delta y}{\delta t} = q \qquad (2.4)$$

To the momentum equation must be added a term to account for the *additional momentum both entering and leaving the control volume.* The additional momentum entering the control volume is $\rho q u_q \Delta x$ where u_q is the downstream component of velocity of the lateral inflow. Upon leaving the control volume the velocity of the additional flow is the same as the channel velocity, so the momentum leaving is $\rho q u_q \Delta x$. Some authors (Liggett [1975]) and Cunge, Holly and Verwey (1980) include this latter term as *a separate term in the dynamic equation.* However, examination of the continuity equation including lateral inflow shows that the flow leaving the control volume includes the inflow to the control volume. The term representing the nett rate of momentum entering the control volume, $-\frac{\delta(Qu)}{\delta x}\Delta x$, will thus include for the momentum of the lateral inflow leaving the control volume and the momentum equation becomes :

$$\frac{\delta Q}{\delta t} + \frac{\delta}{\delta x} \left(\frac{\beta Q^2}{A}\right) + gA \frac{\delta y}{\delta x} - gAS_f - qu_q = 0$$

Other forces can be included in the momentum equation besides gravity, pressure and friction forces. Wind effects can be included where these are significant, *for example*, in the model of the Swartkops River and estuary in Port Elizabeth developed by the National Research Institute for Oceanology (Huizinga (1984))

The model developed in this project has been based on the Equations (2.1) and (2.3) derived above, without consideration *of lateral inflow or other forces.*

2.2 Discretization of the Flow Equations

Writing $f(x,t) = f(i\Delta x,\ n\Delta t) = f_i^n$ *for the general form* of the dependent variables at the point $(i\Delta x,\ n\Delta t)$ in the computational grid the *discretization of these* variables and their time and space derivatives follow the Preissmann scheme :

$$f(x,t) = (1-\theta)(1-\psi)f_i^n + \theta(1-\psi)f_i^{n+1} + (1-\theta)\psi\ f_{i+1}^n + \theta\psi f_{i+1}^{n+1}$$

$$\frac{\delta f}{\delta x} = \theta(f_{i+1}^{n+1} - f_i^{n+1})\Delta x + (1-\theta)(f_{i+1}^n - f_i^n)/\Delta x$$

$$\frac{\delta f}{\delta t} = \psi(f_{i+1}^{n+1} - f_{i+1}^n)/\Delta t + (1-\psi)(f_i^{n+1} - f_i^n)/\Delta t \qquad (2.4)$$

where θ and ψ are the time and space weighting coefficients respectively, as depicted in Figure 1.7.

With the St.Venant equations as derived in Section 2.1

$$\frac{\delta Q}{\delta x} + B_s\ \frac{\delta y}{\delta t} = 0 \qquad (2.1)$$

and

$$\frac{\delta Q}{\delta t} + \frac{\delta}{\delta x}\ (\beta\frac{Q^2}{A}) + gA\ \frac{\delta y}{\delta x} - gAS_f = 0 \qquad (2.3)$$

the mass or continuity equation is simply discretized as *follows* :

$$\theta(Q_{i+1}^{n+1} - Q_i^{n+1})/\Delta x_i + (1-\theta)(Q_{i+1}^n - Q_i^n)/\Delta x_i + Bs_{i+1}^{n+\theta}$$

$$\psi(y_{i+1}^{n+1} - y_{i+1}^n)/\Delta t + Bs_i^{n+\theta}\ (1-\psi)(Y_i^{n+1} - Y_i^n)/\Delta t = 0$$

$$(1.5)$$

where

$$Bs_i^{n+\theta} = \theta Bs_i^{n+1} + (1-\theta)Bs_i^n$$

Grouping the dependent variables at time step n+1, i.e. the unknowns, on *the left hand side* of the equation, multiplying through by Δx_i and rearranging, an equation of the following form is obtained.

$$A1_i Q_i^{n+1} + B1_i y_i^{n+1} + C1_i Q_{i+1}^{n+1} + D1_i y_{i+1}^{n+1} = E1_i \tag{2.6}$$

where

$$A1_i = -\theta$$

$$B1_i = Bs_i^{n+\theta}(\psi - 1)\Delta x_i/\Delta t$$

$$C1_i = \theta \tag{2.7}$$

$$D1_i = Bs_{i+1}^{n+\theta}\,\psi\Delta x_i/\Delta t$$

$$E1_i = (1-\theta)(Q_i^n - Q_{i+1}^n) + B1_i y_i^n + D1_i y_{i+1}^n$$

Before discretizing the dynamic equation as given by Equation (2.3) it is necessary to convert the friction slope S_f to a relation between friction losses and discharge of the type

$$Q = K\sqrt{S_f}$$

Where $K = K(x,y)$ is the conveyance of the channel. The value of K can be determined from any of the well-known empirical resistance laws such as Manning or Chezy. *Expressed in terms* of the Manning equation

$$K = \frac{AR^{2/3}}{n} \tag{2.8}$$

The dynamic equation can now be written as

$$\frac{\delta Q}{\delta t} + \frac{\delta}{\delta x}\left(\beta\frac{Q^2}{A}\right) + gA\frac{\delta y}{\delta x} + gA\,\frac{Q|Q|}{K^2} = 0 \tag{2.9}$$

The discretization of the dynamic equation is complicated by the presence of the non-linear term $\frac{\delta}{\delta x}(\beta\frac{Q^2}{A})$ and $gA\frac{Q|Q|}{K^2}$
In Verwey's scheme, the discretization of these terms is similar to that used in the Abbott-Ionescu scheme, as follows :

$$\frac{\delta}{\delta x}(\beta\frac{Q^2}{A}) = (\beta_{i+1}^{n+\theta}\, Q_{i+1}^{n+1}\, Q_{i+1}^{n} \;/\; A_{i+1}^{n+\theta} - \beta_{i}^{n+\theta}\;\; Q_{i}^{n+1}\; Q_{i}^{n} \;/\; A_{i}^{n+\theta})/\Delta x_i$$

where (2.10)

$$\beta_{i}^{n+\theta} = \theta\beta_{i}^{n+1} + (1-\theta)\beta_{i}^{n}$$

$$A_{i}^{n+\theta} = \theta A_{i}^{n+1} + (1-\theta)A_{i}^{n}$$

and

$$gA\frac{Q|Q|}{K^2} = gA_{i+\psi}^{n+\theta}\,(\psi Q_{i+1}^{n+1}|Q_{i+1}^{n}|/(K_{i+1}^{n+\theta})^2 + (1-\psi)Q_{i}^{n+1}|Q_{i}^{n}|/(K_{i}^{n+\theta})^2)$$

Where

$$A_{i+\psi}^{n+\theta} = \psi A_{i+1}^{n+\theta} + (1-\psi)A_{i}^{n+\theta}$$

and Verwey uses

$$K_{i}^{n+\theta} = \theta K_{i}^{n+1} + (1-\theta)\,K_{i}^{n}$$

Since effectively $Q_{i}^{n+1}|Q_{i}^{n}|$ corresponds to the square of the *geometric mean* of the discharge at the two time levels, the value of $K_{i}^{n+\theta}$ used in the model has been taken to be the geometric mean of K_{i}^{n} and K_{i}^{n+1} rather than the arithmetic mean used by Verwey. Thus, in the model the following discretization has been used.

$$gA\frac{Q|Q|}{K^2} = gA_{i+\psi}^{n+\theta}\left[\psi Q_{i+1}^{n}|Q_{i+1}^{n}|/(K_{i+1}^{n+1}\,K_{i+1}^{n}) + (1-\psi)Q_{i}^{n+1}|Q_{i}^{n}|/(K_{i}^{n+1}\,K_{i}^{n})\right]$$

The different discretization is only significant if $\theta > \frac{1}{2}$ since the value of $K_{i}^{n+\theta}$ given by Verwey is depende.. on θ.

The discretized dynamic equation thus becomes

$$\psi(Q_{i+1}^{n+1} - Q_{i+1}^{n})/\Delta t + (1-\psi)(Q_i^{n+1} - Q_i^n)/\Delta t + \left[\beta_{i+1}^{n+\theta}Q_{i+1}^{n+1}Q_{i+1}^{n}/A_{i+1}^{n+\theta} - \beta_i^{n+\theta}Q_i^{n+1}Q_i^n/A_i^{n+\theta}\right]\Delta x_i + gA_{i+\psi}^{n+\theta}\left[\theta(y_{i+1}^{n+1} - y_i^{n+1}) + (1-\theta)(y_{i+1}^{n} - y_i^{n})\right]\Delta x_i + gA_{i+\psi}^{n+\theta}\left[\psi Q_{i+1}^{n+1}|Q_{i+1}^{n}|/(K_i^{n+1}\ K_{i+1}^{n}) + (1-\psi)Q_i^{n+1}|\ Q_i^n|/(K_i^{n+1}\ K_i^n)\right] = 0 \quad (2.13)$$

As for the mass equation, the dynamic equation can be written in form

$$A2_iQ_i^{n+1} + B2_iy_i^{n+1} + C2_iQ_{i+1}^{n+1} + D2_iy_{i+1}^{n+1} = E2_i \quad (2.14)$$

where

$$A2_i = (1-\psi)\Delta x_i/\Delta t - \beta_i^{n+\theta}Q_i^n/A_i^{n+\theta} + g\Delta x_i(1-\psi)\ A_{i+\psi}^{n+\theta}|Q_i^n|/(K_i^{n+1}K_i^n)$$

$$B2_i = -g\theta A_{i+\psi}^{n+\theta}$$

$$C2_i = \psi\Delta x_i/\Delta t - \beta_{i+1}^{n+\theta}Q_{i+1}^{n}/A_{i+1}^{n+\theta} + g\Delta x_i\ \psi A_{i+\psi}^{n+\theta}|Q_{i+1}^n|/(K_{i+1}^{n+1}K_{i+1}^{n})$$

$$D2_i = -B2_i \quad (2.15)$$

$$E2_i = \Delta x_i/\Delta t\ (\psi Q_{i+1}^n + (1-\psi)Q_i^n) + (1-\theta)gA_{i+\psi}^{n+\theta}\ (y_{i+1}^n - y_i^n)$$

The coefficients $A1_i$, $B1_i$, etc. and $A2_i$, $B2_i$, etc. are known functions of flow variables. Equations (2.6) and (2.14) are two linear algebraic equations in terms of Q_i^{n+1}, Y_i^{n+1}, Q_{i+1}^{n+1} and Y_{i+1}^{n+1} for every pair of points $(i, i+1)$. For N computational points there will be a system of 2N-2 equations for 2N unknowns.

With the addition of the boundary conditions the system can be solved using the double sweep algorithm.

It can be seen that the equations for the coefficients, Equation (2.7) and (2.15), include the values of the flow variables B_s, β, A and K at time level $(n+1)\Delta t$. The flow equations must therefore be solved iteratively at each time

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step. In the first iteration these flow variables are approximated with their values from the previous time level, n t; in the second iteration the solution is improved by using the results of the first iteration. Two iterations per time step have been found to give a sufficiently accurate simulation in most situations.

2.2 Double Sweep Algorithm

According to Strelkoff (1970) the double sweep algorithm has been in use as a means of solving the de St.Venant equations since the early 1960's. The method takes advantage of the fact that the only non-zero values in the matrix of coefficients lie in a band along the diagonal, as can be seen below.

With the boundary conditions linearized in y and Q the system of equations for a single channel reach has the form

$$
\begin{bmatrix}
L_1 & L_2 & & & & & & & & \\
A1_1 & B1_1 & C1_1 & D1_1 & & & & & & \\
A2_1 & B2_1 & C2_1 & D2_1 & & & & & & \\
 & & A1_2 & B1_2 & C1_2 & D1_2 & & & & \\
 & & A2_2 & B2_2 & C2_2 & D2_2 & & & & \\
 & & & & A1_3 & B1_3 & C1_3 & D1_3 & & \\
 & & & & A2_3 & B2_3 & C2_3 & D2_3 & & \\
 & & & & & & \ddots & & & \\
 & & & & & & A1_{N-1} & B1_{N-1} & C1_{N-1} & D1_{N-1} \\
 & & & & & & A2_{N-1} & B2_{N-1} & C2_{N-1} & D2_{N-1} \\
 & & & & & & & & R_1 & R_2
\end{bmatrix}
\begin{bmatrix}
Q_1^{n+1} \\ y_1^{n+1} \\ Q_2^{n+1} \\ y_2^{n+1} \\ Q_3^{n+1} \\ y_3^{n+1} \\ \vdots \\ Q_{N-1}^{n+1} \\ y_{N-1}^{n+1} \\ Q_N^{n+1} \\ y_N^{n+1}
\end{bmatrix}
=
\begin{bmatrix}
L_3 \\ E1_1 \\ E2_1 \\ E1_2 \\ E2_2 \\ E2_3 \\ \vdots \\ E2_{N-2} \\ E1_{N-1} \\ E2_{N-1} \\ R_3
\end{bmatrix}
\tag{2.16}
$$

where $L_1\, Q_1^{n+1} + L_2\, y_1^{n+1} = L_3$ (2.17)

and $R_1\, Q_N^{n+1} + R_2\, y_N^{n+1} = R_3$ (2.18)

are the left and right hand side boundary conditions respectively at the two ends of the reach. The double sweep algorithm uses the banded matrix structure of the linear system of equations to compute the solution, with the number of operations proportional to N.

Assume that there is a linear relationship of the type

$$Q_i^{n+1} = F_i y_i^{n+1} + G_i \tag{2.19}$$

which holds at point (i,n+1) in the computational grid

If this is true it can be shown that an analogous linear relationship also exists at the adjacent point, (i+1, n+1) so that:

$$Q_{i+1}^{n+1} = F_{i+1}\, y_{i+1}^{n+1} + G_{i+1} \tag{2.20}$$

This is done by subsituting equation (2.19) into equations (2.6) and (2.14).

$$(B1_i + A1_i F_i) y_i^{n+1} + C1_i Q_{i+1}^{n+1} + D1_i\, y_{i+1}^{n+1} = E1_i - A1_i G_i \tag{2.21}$$

$$(B2_i + A2_i F_i) y_i^{n+1} + C2_i Q_{i+1}^{n+1} + D2_i y_{i+1}^{n+1} = E2_i - A2_i G_i \tag{2.22}$$

From equation 2.21 a relationship between y_i^{n+1} and the dependent variables at point i + 1 is obtained :

$$y_i^{n+1} = -\frac{C1_i}{B1_i + A1_i F_i}\, Q_{i+1}^{n+1} - \frac{D1_i}{B1_i + A1_i F_i}\, y_{i+1}^{n+1} + \frac{E1_i - A1_i G_i}{B1_i + A1_i F_i} \tag{2.23}$$

Equation (2.23) can be written as

$$y_i^{n+1} = H_i Q_{i+1}^{n+1} + I_i y_{i+1}^{n+1} + J_i \tag{2.24}$$

where

$$H_i = -\frac{C1_i}{B1_i + A1_i F_i} \;;\; I_i = -\frac{D1_i}{B1_i + A1_i F_i} \;;\; J_i = \frac{E1_i - A1_i G_i}{B1_i + A1_i F_i} \tag{2.24}$$

Eliminating y_i^{n+1} from equations (2.21) and (2.22) gives

$$Q_{i+1}^{n+1}\, (C1_i\, (B2_i + A2_i F_i) - C2_i (B1_i + A1_i F_i)) + y_{i+1}^{n+1}$$

$$(D1_i (B2_i + A2_i F_i) - D2_i (B1_i + A1_i F_i))$$

$$= (E1_i - A1_i G_i)(B2_i + A2_i F_i) - (E2_i - A2_i G_i)(B1_i + A1_i F_i) \tag{2.26}$$

Dividing through by $(B1_i + A1_i F_i)$ and putting $\varepsilon_i = (B2_i + A2_i F_i)$ and expressing Q_i^{n+1} as a function of y_{i+1}^{n+1} one obtains

$$Q_{i+1}^{n+1} = -\frac{(\varepsilon_i I_i + D2_i)}{(\varepsilon_i H_i + C2_i)} y_{i+1}^{n+1} + \frac{E2_i - A2_i G_i - \varepsilon_i J_i}{\varepsilon_i H_i + C2_i} \qquad (2.27)$$

which is relationship of the form indicated by equation (2.20), where

$$F_{i+1} = -\frac{(\varepsilon_i I_i + D2_i)}{(\varepsilon_i H_i + C2_i)} \; ; \; G_{i+1} = \frac{E2_i - A2_i G_i - \varepsilon_i J_i}{\varepsilon_i H_i + C2_i} \qquad (2.28)$$

Thus it is shown that, if the relationship Equation (2.19) exists at any point in the model, similar equations can be written for all the following points. By linearizing the boundary conditions in this form a set of equations for all points in the model can be obtained.

In the forward sweep of the double-sweep algorithm the values of the coefficients are determined at all points in the model as follows. The sweep is intialised by determining the coefficients F_l and G_l at the first point from the boundary conditions. By combining Equations (2.17) and (2.19) one obtains

$$F_1 = \frac{L_2}{L_1} \; ; \; G_1 = \frac{L_3}{L_1} \qquad (2.29)$$

The values of H_1, I_1 and J_1 are then computed from Equation (2.25) and F_2 and G_2 from Equation (2.28). The algorithm proceeds in this manner from gridpoints 2 to N as is shown in Figure 2.2(a). To initialise the return sweep at the right hand boundary, use is made of the relation combining Equations (2.18) and (2.19), namely

$$y_N^{n+1} = \frac{R_3 - R_1 G_N}{R_2 + R_1 F_N} \qquad (2.30)$$

The values of Q_N^{n+1} and y_{N-1}^{n+1} can then be determined from Equations (2.19) and (2.24) respectively and the return sweep continues with the values of Q_i and y_i being determined at each successive gridpoint as is shown in Figure 2.2(b).

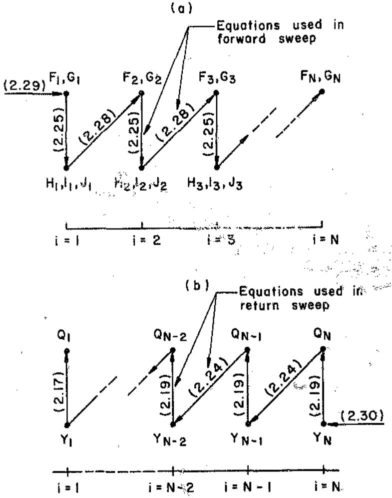

FIGURE 2.2 Schematization of the double sweep algorithm
(a) Forward Sweep (b) Return Sweep

The double-sweep algorithm is only applicable to subcritical flows. Supercritical flows can be handled with a single-sweep algorithm, with two boundary conditions being given at the upstream boundary. Where the flow passes from subcritical to supercritical through a continuous nearly horizontal flow, the two algorithmic structures have to be used in a single computation; the method of computation is described by Abbott (1979).

2.4 Initial and Boundary Conditions

In order to solve the system of simultaneous equations in y and Q at time level $(n + 1)\Delta t$ it is necessary to know the values of y and Q at all points at the previous time level $(n\Delta t)$, together with the inflow to reach, Q_1^{n+1}, and the stage at the downstream end of the reach y_N^{n+1}.

To start a flood routing computation it is thus necessary to define the inital conditions in the reach in terms of Q and y at all points in the reach.

The boundary conditions are generally specified in the form of an inflow hydrograph at the upstream end of the reach and a stage-discharge or stage-time relationship at the downstream end.

The values of L_1, L_2 and L_3 in the equation for the upstream boundary conditions, Equation (2.17), can then be determined from the ordinate of the inflow hydrograph at the applicable time level as follows :

$$L_1 = 1 \; ; \quad L_2 = 0 \; ; \quad L_3 = Q_1^{n+1} \qquad (2.31)$$

The values of F_1 and G_1 required to initiate the forward sweep can be obtained from equations (2.29).

Where for the downstream boundary condition the stage is given as a function of time, the values of R_1, R_2 and R_3 in equations (2.18) can be obtained directly as follows :

$$R_1 = 0 \; ; \quad R_2 = 1 \; ; \; R_3 = y_N^{n+1} \qquad (2.32)$$

The values of F_N and G_N are known, having been determined in the forward sweep, and Q_N^{n+1} can be determined directly from equation (2.19).

Where the stage is given as a function of the flow it is necessary to obtain a linear approximation of the *function.* For example, a weir-type formulation

$$Q = \gamma(y - y_w)^{3/2} \qquad (2.33)$$

where y_w is the level of the weir crest, can be linearised by using *a first order* Taylor series development :

$$Q_N^{n+1} = Q_N^n + \left(\frac{\delta Q}{\delta y}\right)^n (y_N^{n+1} - y_N^n) \qquad (2.34)$$

where $\left(\frac{\delta Q}{\delta y}\right)^n = {}^3/_2\ \gamma(y_N^n - Y_w)^{1/2}$

Comparing this equation with Equation (2.18) gives

$$R_1 = 1 \ ; \quad R_2 = -\frac{3}{2}\gamma(y_N^n - y_w)^{\frac{1}{2}} \ ; \quad R_3 = Q_N^n - \frac{3}{2}\gamma(y_N^n - y_w)^{\frac{1}{2}} y_N^n$$

By specifying critical flow conditions at the downstream end of the reach the equation *for the Froude* Number is set equal to unity.

$$F_R = \frac{\alpha Q^2 B}{g A^3} = 1$$

and solved iteratively with the equation

$$Q_N^{n+1} = F_N y_N^{n+1} + G_N$$

This has the disadvantage that the speed of the computation is slowed while these equations are solved; the alternative is to provide *the information* as discrete points in a rating table. The linear approximation to the rating curve in the region of interest will then provide the values of the coefficients.

3. DESCRIPTION OF COMPUTER PROGRAM

The computer program comprises a data entry section, the main computational section and an output section and is described under these headings in Sections 3.1, 3.2 and 3.3. The user instructions and a listing of the program are given in Appendixes A1 and B1 respectively. In the form presented in the Appendix B1 the program has a capacity of 100 cross-sections of 20 points each and 5 000 time steps. The programme capacity is purely dependent on the memory size of the hardware and can easily be altered to suit.

3.1 Data Input

The input data required for the model are the cross-section data describing the reach, the boundary conditions and the initial conditions in the reach. The boundary conditions usually are of the form of an inflow hydrograph at the upstream end of the reach and some description of the stage at the downstream end.

3.1.1. Cross-section data

Cross-section data for natural channels is arranged in tabular form with the distance along the cross-section from some arbitrary starting point, the ground level and the Manning roughness coefficient given for each point in the cross-section. The data file containing cross-section data is set up using a separate program. The user instructions for this program and a listing thereof are given in Appendices A2 and B2 respectively. The data file structure is the same as that used for a conventional backwater program, the principle difference being that the order of cross-sections required for the backwater program is the reverse of that required for the model.

The advantage of having the structure of the data file the same as used in a backwater program is that steady state initial conditions for the reach can be determined and stored using the backwater program. This aspect is discussed further in Section 3.1.3. A sample from a cross-section data file listing is given in *Table 3.1*.

REACH NUMBER 2

CROSS SECTION	REACH DISTANCE	POINT NUMBER	SECTION DISTANCE	GROUND LEVEL	MANNING N
1	0.00	1	0.0	5.00	.020
		2	10.0	0.00	.020
		3	1000.0	0.00	.020
		4	1010.0	5.00	.020
2	100.00	1	0.0	5.00	.040
		2	70.0	3.45	.040
		3	95.0	1.80	.040
		4	100.0	1.80	.040
		5	120.0	2.85	.040
		6	131.0	3.40	.040
		7	203.0	5.00	.040
3	160.00	1	0.0	5.00	.025
		2	75.0	3.50	.025
		3	82.0	2.20	.025
		4	92.0	2.20	.025
		5	137.0	5.45	.025
4	220.00	1	0.0	5.30	.040
		2	11.0	3.85	.040
		3	13.0	1.27	.040
		4	22.0	1.60	.040
		5	43.0	4.00	.040
		6	63.0	4.85	.040
		7	77.0	5.00	.040

TABLE 3.1 Sample listing of cross-section data file

It is not necessary for the cross-sections to be spaced at regular intervals along the reach; any spacing which adequately describes the physical configuration of the watercourse is acceptable. Generally for floodline determination the distance step should be of the order of 50 m to 200 m, depending on the flooded width.

Reservoirs can be included in a reach without special treatment provided that the spillway is given as the most downstream section *if it operates under free discharge conditions;* its discharge characteristic is then given as the downstream boundary condition. Cunge, Holly and Verwey (1980) describe methods of treating weirs as internal boundary conditions in a reach. This involves special treatment of the coefficients at the section and the facility cannot be included in a model for general application. Where a reservoir or weir occurs in a reach the preferred method is to divide the reach into two with the reservoir spillway or weir forming the boundary between the two reaches. The outflow from the upstream reach then becomes the inflow to the downstream reach.

The most important characteristic of a reservoir besides its outflow characteristic is its surface area to water level relationship. Since the water level slope in most smaller reservoirs is negligible it is not necessary to accurately represent the ground profile beneath the spillway crest. The cross-sections selected to define the basin should provide a reasonable representation of the surface area to water level relationship above the spillway crest.

The most convenient source of cross-section data is large scale topographic mapping (1:1000 to 1:2500) with contours at 0,5 or 1 m intervals. This type of mapping is frequently available in municipal areas. Field surveys should be carried out to define hydraulic controls such as weirs and drainage structures in the watercourse as well as to determine the channel depth in perennial rivers.

For larger rivers, where the objective of the flood routing exercise is not to determine the maximum flood levels with any accuracy but to obtain information on the passage of a flood, 1:10000 orthophoto mapping with 5 m contour intervals will often provide sufficient data for the exercise.

3.1.2 Inflow hydrograph data

The inflow hydrograph to the reach is usually defined by a number of *discrete points on the hydrograph.* With the size of the time step given, the values of the inflow are interpolated at corresponding intervals. The hydrograph can be stored in a file for use in other runs. A number of inflow hydrographs can be added in the model either directly or with a given time lag. This feature is useful where the outflow hydrograph from the upstream reach must be added to the run-off from the local catchment.

It is intended that the model be able to use hydrographs generated by hydrological models such as WITWAT or ILLUDAS when the facility for storing outflow hydrographs from these models is available.

Since the hydrograph is imposed upon a reach which is in an initial state defined by the initial conditions, it is necessary that the initial flow of the inflow hydrograph does not differ substantially from the flow in the reach in this initial state.

3.1.3 Downstream boundary condition

As was stated in Section 2.4 the downstream boundary condition can be specified by giving the stage as a function of the discharge or as a function of time.

A *stage-discharge* relationship can be given in one of three ways. The first is merely to specify that critical conditions occur at the downstream end of the reach. The second is to give the coefficients of a weir-tv [illegible] a such as Equation (2.33). In the third method [illegible] oints on the rating curve are given.

The most common form of stage-time relationship used in one-dimensional modelling is the tidal equation. The tidal equation used in the model includes only the semi-diurnal lunar and solar components of the tide.

$$y = \bar{y} + A_{M2}\sin\frac{(2\pi T)}{(T_{M2})} + A_{S2}\sin((2\pi(T + \phi)/T_{S2}) \qquad (3.1)$$

where

$\bar{y}$ = datum for mean sea level

A_{M2} = amplitude of the semi-diurnal lunar component (M2)

A_{S2} = amplitude of the semi-diurnal solar component (S2)

T_{M2} = period of the M2 component (= 44 714 s)

T_{S2} = period of the S2 component (= 43 200 s)

ϕ = phase difference between M2 and S2 components (= 0 at new moon)

A constant downstream water level can be handled by putting the amplitudes of the tidal components equal to zero.

3.1.4 Initial conditions

In order to initialise a run of the model the initial conditions in the reach must be given in the form of the water level and discharge at each point in the reach. Obviously this initial state should be as consistent as possible with the flow equations in the model to minimise the amount of computer time spent in stabilising the system prior to imposing the flood hydrograph on it. The most direct way of achieving this is to furnish an initial condition as a backwater curve for a steady state condition with the flow equivalent to the base flow in the river or some fraction of the peak flow.

It is still necessary to run the model under steady state conditions by holding the boundary condition constant for a number of time steps to allow initial perturbations to dissipate or as given by equation 2.13 reduces to the discrete form of the energy equation by putting

$$Q_i^{n+1} = Q_{i+1}^{n+1} = Q_i^n = Q_{i+1}^n = Q$$

and

$$y_i^{n+1} = y_i^n = y_i$$

then

$$(\beta_{i+1}Q^2/A_{i+1} - \beta_i Q^2/A_i)/\Delta x_i + gA(y_{i+1} - y_i)/\Delta x_i$$

$$+ g\frac{A}{2}\left(\frac{Q^2}{K_{i+1}^2} + \frac{Q^2}{K_i^2}\right) = 0$$

Dividing through by $\frac{gA}{\Delta xi}$ and putting $(Q/K_i)^2$ S_{fi}

$$\frac{Q}{gA}(\beta_{i+1} u_{i+1} - \beta_i u_i) + (y_{i+1} - y_i)$$

$$+ \tfrac{1}{2}(S_{fi+1} + S_{fi})\Delta x_i = 0$$

Writing $\frac{Q}{A} = \left(\frac{u_{i+1} + u_i}{2}\right)$ and assuming that $\beta_{i+1} \cong \beta_i = \beta$, the desired fo[illegible] the energy equation is obtained :

$$\frac{1}{2g}(\beta u_{i+1}^2 - \beta u_i^2) + (y_{i+1} - y_i) + \tfrac{1}{2}(S_{fi+1} + S_{fi})\Delta x = 0 \qquad (3.2)$$

The values of y_i and Q_i at each point in the model obtained from this run can be stored in a file for use as initial conditions with flood hydrograph runs, provided that the initial flow of the flood hydrograph corresponds to the steady state flow.

3.1.5 Selection of the weighting coefficients and time step

With the weighing coefficients θ and ψ set at 0,5 the discretization used in the finite difference approximation of the flow equation corresponds to a centred-difference approximation and is thus of second order accuracy. However, for values of the Courant Number greater than unity parasitic oscillations appear in the solution with θ = 0,5 which disappear with the numerical damping in the scheme for $\theta > 0{,}66$. Liggett and Cunge 1975 recommend values of θ given by $0{,}6 \leq \theta \leq 1{,}0$

A disadvantage with setting the value of θ close to unity when using relatively large time steps in the computation is that the errors introduced in the conservation of mass in the system become significant. With $\theta = 1$ and $\psi = 0{,}5$ the discretized continuity equation can be written :

$$(Q^{n+1}_{i+1} - Q^{n+1}_{i})\Delta t + [Bs^{n+1}_{i+1} \; (y^{n+1}_{i+1} - y^{n}_{i+1}) + Bs^{n+1}_{i} \; (y^{n+1}_{i} - y^{n}_{i})] \; \Delta x/_{2} = 0 \qquad (3.3)$$

The primary source of the error is that the net inflow to the control volume is expressed only in terms of the flow at the end of the time step, rather than the average inflow during the time step. Similarly, the volume change is expressed only in terms of the surface width at the end of the time step, rather than the average surface width.

The effect of varying θ is demonstrated further in Chapter 4 with the application of the model to two natural watercourses and a prismatic channel. The effect of varying the value of ψ was not investigated.

The selection of the size of time step to be used in the computation should be based on obtaining a reasonable description of the inflow hydrograph at the upstream boundary or a tidal condition at the downstream boundary if such exists.

The only definitive way to see whether a time step is too large or not is to simulate the same event on the model using successively smaller time steps. If the size of the time step significantly affects the solution, it is too large. Generally values of the time step between 0,01 and 0,25 can be used, the lower limit in situations with steeply sloping hydrographs, the upper limit for low-flow tidal simulations. It is shown in Chapter 4 that the degree of non-linearity in the relationship between flow depth and area can affect the stability of the computation. Where the flow depth to area relationship is highly non-linear, as occurs often with natural channels, the size of the time step must be reduced to eliminate this instability.

3.2 *Main Computation*

The flow diagram for the main computation is shown in Figure 3.1. The computation proceeds with the determination of the section properties, namely the flow area, surface width, conveyance, and momentum correction factor at all cross-sections with the initial conditions imposed. The dependent variables Q and y at each point at the end of the first time step are set equal to the initial values for the first iteration and the coefficients A1 to E1 and A2 to E2 calculated from Equations (2.7) and (2.15) and the double sweep algorithm is then applied, from which the first approximations to the dependent variables at the end of the first time step are obtained. The section properties at the end of the first time step are then calculated using the first approximations to the dependent variables and the coefficients A1 to E1 and A2 to E2 recalculated. The double sweep routine yields the accepted values for the dependent variables at the new time step for which the section properties are then calculated. The program has the facility to print these values after every time step, or if specified by the user, only after selected time periods. The computation then proceeds to the next time step.

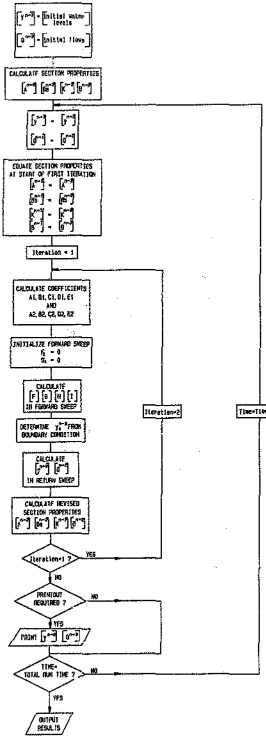

Figure 3.1 : Flow Chart for Main Computation

In order to limit the memory required in the computation the values of the dependent variables and the associated section properties at the two time levels required in the computation, i.e. at time level $n\Delta t$ and $(n+1)\Delta t$, are retained in memory, all information at previous time levels being overwritten. The first step at the new time level is to transfer the values of the dependent variables Q and y and the associated section properties from time level $(n+1)\Delta t$ to time level $n\Delta t$ variables. The values of the time level $(n+1)\Delta t$ variables are retained as initial values for the first iteration at the new time step. The procedure then repeats itself, two iterations being carried out at the new time level.

It can be seen that at each time level the section properties at each cross-section are calculated twice. Initially the program was set up such that any cross-section of Np points would be divided into Np-1 sub-sections and the section properties calculated for each sub-section and then summated for the cross-section. Indeed the kinetic energy and momentum correction factors, α and β can only be calculated in this way.

Obviously where the cross-section shape is complex, with a large number of sub-sections, the calculation of the section properties in this manner becomes the most time-consuming part of the computation cycle.

An alternative method of obtaining the section properties is to interpolate them from tabulated values, the table of values being set up at the start of the run. It is only in the estimation of the non-linear functions, namely the area, conveyance and the momentum and kinetic energy correction factors, that inaccuracies are introduced by the interpolation. These inaccuracies can be eliminated in the case of the area and reduced in the case of the conveyance by expressing these quantities in terms of those which can be directly interpolated, namely the surface width and wetted perimeter, and, in the case of the conveyance, interpolating a composite Manning roughness value for the cross-section. For example, for a water level y^{n+1}

falling between two values y_J and y_{J+1} in the table of section properties, the area and conveyance can be calculated from

$$A_i^{n+1} = A_J + \tfrac{1}{2}\,(Bs_i^{n+1} + Bs_J)(y_i^{n+1} - y_J)$$

and

$$K_i^{n+1} = \frac{1}{N_i^{n+1}}\, A_i^{n+1}(A_i^{n+1}/P_i^{n+1})^{2/3}$$

Where Bs_i^{n+1} , N_i^{n+1} and P_i^{n+1} are the interpolated values of the surface width, Manning roughness coefficient and wetted perimeter respectively, and A_J and B_{sJ} are the tabulated values of the area and surface width corresponding to the water level y_J.

Since the section properties can be directly calculated in this manner the speed of computation is greatly enhanced.

3.3 Presentation of Results

The output from the model can take a number of forms. During the computation the values of Q^{n+1} and y^{n+1} can be printed after each time step or after a preselected number of time steps.

A plot of the inflow and outflow hydrographs can be obtained, useful in that it presents the user with an immediate impression of the degree of attenuation and the time lag in the reach. For tidal computations a plot of the water levels at both ends of the reach is also made.

Since the maximum water levels reached during the flood and the maximum flows at each point in the reach are usually also of interest these are printed together with the times at which these maxima occur. Since flow reversals can occur during tidal computations, the minimum water levels and flows are tabulated if the flow is tidal.

The results of a mass balance calculation are provided to allow a check on the integrity of the computation. The error in the mass balance calculation is generally larger for $\theta = 1$ than for $\theta = 0,5$, as has been explained in Section 3.1.4.

4. *APPLICATION OF THE MODEL*

4.1 Introduction

The application of the model to two different flow situations is presented in this chapter. In the first application, described in Section 4.2, the Swartvlei Estuary is modelled under tidal conditions. The Swartvlei Estuary is on the south-eastern Cape Coast in the vicinity of the coastal resort of Wilderness and comprises a lake connected to the sea by the estuary. Tidal propagation in the estuary extends to the lake, where the maximum tidal range is of the order of 20 millimetres with a variation in the mean level between spring and neap tides of 100 millimetres. Cross-sections of the estuary have been obtained and tidal measurements made at a number points in the estuary by the National Research Institute for Oceanology (NRIO) of the CSIR. This data provides an useful check on the model.

In the second application of the model, described in Section 4.3, a reach of Rietspruit is modelled under conditions of the fifty year flood. As this is an ungauged watercourse it is not possible to obtain flood flow records for comparison with the model. Ideally a reach of watercourse with two relatively close gauging stations is required to test the model under flood conditions. Cross-sections of the watercourse between the gauging stations are also required. However, it is of value to examine the behaviour of the model with inflow hydrographs of different shapes and steepness. The effect of the steepness of the wave front on the stability of the solution is demonstrated and the limiting effect of the non-linear stage-area relationship common to watercourses such as the Rietspruit on the size of the time step is shown by introducing the same hydrographs to a wide prismatic channel.

4.2 The Swartvlei Estuary

The Swartvlei Estuary and lake system is shown in Figure 4.1. The lake has a surface area of nine square kilometres at its average water level of +0,7 m above Land Levelling Datum (LLD). It is largely flat bottomed, with a bed level in the deepest section of -11 m LLD. Four rivers drain into the lake, the Diep, the Klein Wolwe, the Hoëkraal and the Karatara rivers which have a combined catchment area of about 350 square kilometres. The estuary itself is some seven kilometres in length, with a surface area of approximately one square kilometre and comprises three sections. The upper estuary between tide gauges 3 and 4 is about 2,5 kilometres long with a main channel about 50 m wide in a 700 m wide flood plain. The middle estuary, between tide gauges 4 and 5, is 3,5 kilometres long, but with a more confined flood plain. The lower estuary is about 1,2 kilometres long and has a dynamic character as a result of the strong tidal flows.

Cross-section data for the estuary was obtained from the NRIO report on the hydrographic survey of the estuary (NRIO 1975) and that on the hydraulic study of the estuary (NRIO 1978). The 47 cross-sections used are given in Appendix C1. To prevent the occurrence of negative water levels during the computation a constant of 3 metres has been added to all levels. Land Levelling Datum (LLD) thus corresponds to +3,0 metres in the model.

Manning roughness values have been selected to minimise the contribution of the areas outside the main flow channels to the conveyance of the section. This obviates the necessity for defining separate widths for the flow channel and for the flood plain, the former being used in the dynamic equation, the latter in the continuity equation. For the main flow channels the Manning roughness was varied between 0,04 and 0,06, while for the flood plain the values were varied between 0,1 and 0,3.

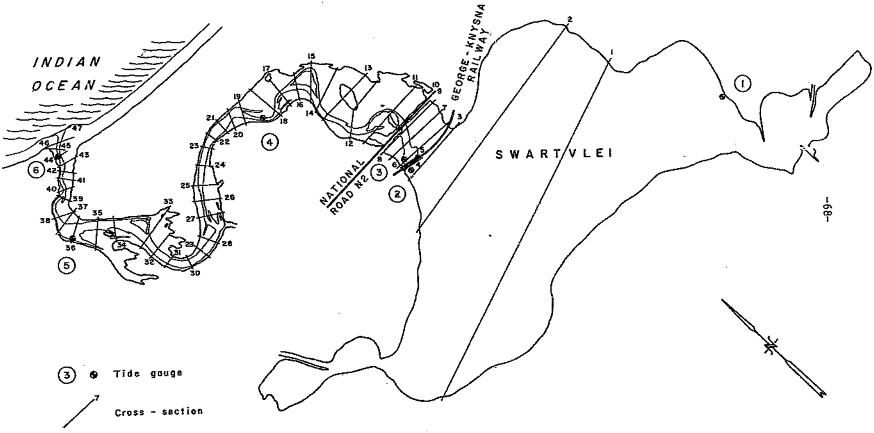

Figure 4.1 : Swartvlei Estuary

It is a requirement of the cross-sections used in the model that they provide an accurate representation of the water surface area of the estuary, and hence the volume in the estuary, at all depths of flow encountered while modelling. It can be seen from Figure 4.1 that the cross-sections are generally at right angles to the flood plain and not to the main flow channel. Consequently the length of the main channel between two cross-sections is in some cases longer than the distance between the cross-sections. To take account of this the roughness coefficients used in the main channel have been increased accordingly. Once the correct surface area representation is obtained from the cross-sections, the selection of the roughness coefficients provides the chief calibration tool in the model.

For the upstream boundary condition the inflow was given as $Q = 2m^3/_s$. In the application of the NRIO model to the estuary zero inflow was found to result in a net outflow from the lake while an inflow of $2\ m^3/_s$, corresponding to the combined average annual inflow from the four rivers to the lake, was found to result in a constant mean level in the lake.

For the downstream boundary condition in the model the ocean tide levels were used, reproduced by the simplified tidal equation given in Section 3.1.3 by Equation (3.1). The amplitude used for the semi-diurnal lunar (M2) component was 0,68 metres and for the solar component (S2) was 0,35 metres. These amplitudes correspond to those for the nearest ocean tide recording station at Mossel Bay, some 40 kilometres to the west. Although the recording station at Knysna is closer, this is situated on the Knysna Lagoon and therefore does not reflect the ocean tide. Mean sea level at the mouth of the Swartvlei estuary is 0,16 metres above LLD. To this must be added an allowance for the wave set-up at the estuary mouth. In the calibration of the NR10 model a wave set-up of 0,13 metres was assumed, making the local mean sea level +0,29 m above LLD or, with the constant added, +3,29 m above the model datum.

For the initial conditions a constant water level of +3,75 metres and a flow of $2m^3/_s$ was used at all cross-sections. To *obtain a corresponding* level of the tide from the downstream boundary condition the time of the tide was set to -19,33 hours. The model was than subjected to a number of stabilizing runs of 360 hours, approximately 30 tidal cycles. In these runs the values of the roughness coefficients were varied for calibration purposes.

The NRIO model was calibrated using measurements obtained during the spring tide of 15th May 1976. Since these measurements are presented in detail in the NRIO report (NRIO 1978) they have been used to calibrate the model developed in this project.

It was found that time steps of up to 0,5 hours could be used in the calibration runs with $\theta = 1$ although oscillations appear in the solution. With $\theta = 1$ the errors in the mass balance calculation for the run, in which the change in volume in the reach was compared with the net inflow to the reach over the period of the run, were found to be as much as 20%. The most *satisfactory* results in terms of smooth outflow hydrographs were obtained with a time step of 0,1 hours and $\theta = 0{,}6$. Computational time for a 360 hour run on the HP 9845T machine was about four hours. With the section properties of the cross-section calculated individually for each sub-section and summated, as described *in Section 3.2, the stabilizing* runs were taking 13 hours to compute, making calibration extremely time-consuming. The graphical output from a 360 hour run is given in Figure 4.2. The spring tide cycle and its effect on the lake water levels are evident in this figure.

The period of the spring tide cycle as determined by the simpli*fied tidal equation* is 354,4 hours. After the stabilizing run of 360 hours the tide state in the model corresponds to 340,67 hours, since the initial tide state was -19,33 hours. Imposing a further two tidal cycles on the model after the stabilizing run carries it through the spring tide condition. The water

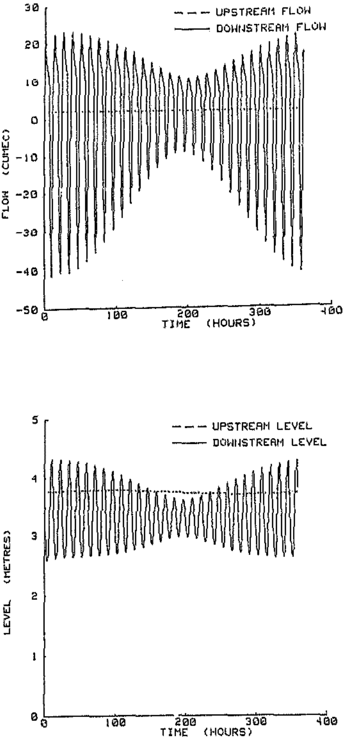

Figure 4.2 : Stage and Discharge Hydrographs for 360 Hour Run

levels and flows in the estuary after the 360 hour run were therefore stored as initial conditions and a 25 hour run carried out, starting with a tide state of 340,67 hours. The results of this latter run were then compared with the measured data for calibration purposes. The output from the model for this run are given in Appendix C2.

The tidal envelope for the estuary is given in Figure 4.3. Also shown are the maximum and minimum levels measured at the various tide gauges during the calibration tide. It can be seen that the tidal wave is increasingly damped as it moves up the estuary from the sea. The damping is strongest in the lower estuary and is only slight in the middle estuary with little decrease in the tidal range. The increased flood plain width of the upper estuary causes further damping.

The water levels as measured at the various gauges during one complete tidal cycle are compared with those predicted by the model in Figure 4.4. The agreement between the predicted and measured data can be regarded as satisfactory.

Finally in Figure 4.5 the flows through the estuary mouth as measured and predicted by the model are depicted. Again the agreement is close. It was found that the flow through the estuary mouth was more sensitive to variations in the roughness coefficients than was the tide levels at the various gauges. The maximum inflow to the estuary therefore provided the first indication as the whether the roughness values selected were generally too high or too low and these were then factored in the next run. The results shown in Figures 4.2 to 4.5 and presented in Appendix C2 were obtained in this way, with factors of 1,5 and 3 respectively applied to the main channel and flood plain roughness coefficients given in Appendix C1. Adjustment of individual cross-section roughnesses was then carried out to a very limited extent to obtain a reasonable fit between measured and predicted tidal levels.

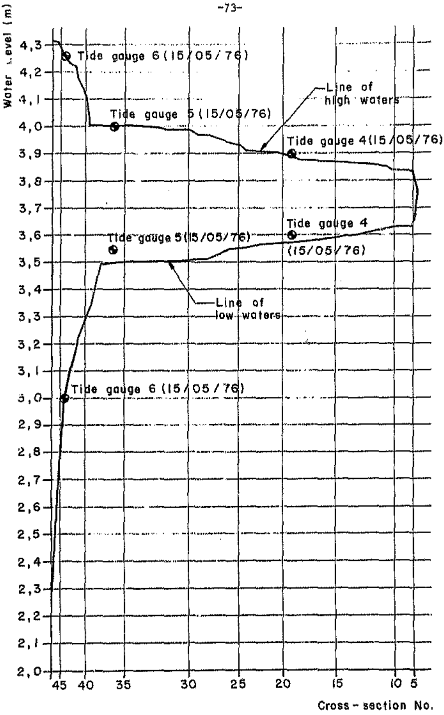

Figure 4.3 : *Tidal Envelope for Swartvlei Estuary*

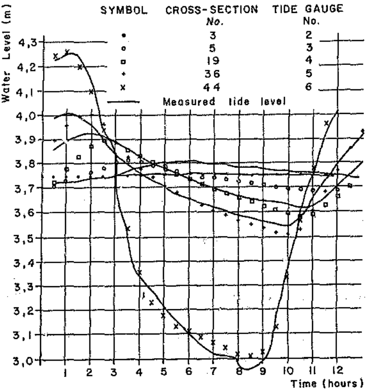

Figure 4.4 : Comparison of Measured and Predicted Water Levels

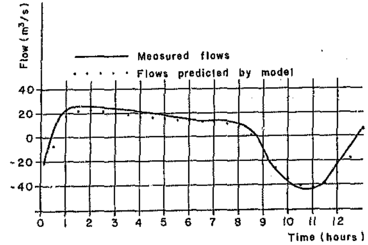

Figure 4.5 : Comparison of Measured and Predicted Water Flow

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4.3 The Rietspruit Watercourse

The reach of the Rietspruit to which the model was applied is located in the Brakpan Municipal area, upstream of the confluence with the Withokspruit. The catchment area is shown in Figure 4.6.

The area of the catchment at the head of the reach is 37 square kilometres. With two tributaries discharging into the reach, one with a catchment area of 19 square kilometres, the other with a catchment area of 6,5 square kilometres, the catchment area at the downstream end is significantly greater, being 73 square kilometres. Because of the size of the tributary catchment relative to the catchment at the head of the reach the reach should be divided into subreaches and the model applied to the channel network. However, for the purposes of this project the reach has been left undivided since the model can only be applied to single channel reaches. Furthermore a reasonable length of channel is obtained thereby which has only gradual variations in cross-section shape and which has a bed slope flat enough to result in subcritical flow in the reach for the range of flows applied.

Cross-sections along the reach were obtained from 1:2000 topographic mapping with contours at 0,5 m intervals. The watercourse consists typically of a wide flood plain with no defined channel; the definition obtained from the topographic mapping is thus acceptable. The cross-sections used are listed in Appendix D.

The Manning roughness coefficients were selected in the range 0,04 to 0,05 for the deepest sections of the cross-sections, and between 0,06 and 0,08 on the flanks. The vegetation in the flood plain varies from extensive reed growth to a thick veld grass. In some areas maize is cultivated on the flood plain. The denseness of the vegetal cover to the flood plain varies seasonally; veld fires during the winter months can

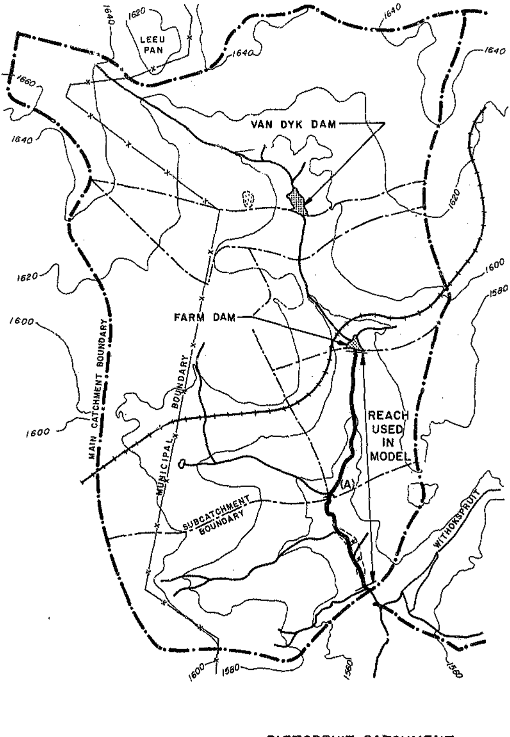


Figure 4.6 : RIETSPRUIT CATCHMENT

7.

denude the flood plain, the vegetation only reaching maximum density at the end of the wet season. The values of the roughness coefficient used are considered representative of the watercourse in the middle of the wet season.

Three inflow hydrographs were used in the model runs, each generated by the watershed model set up for the determination of flood lines along the Rietspruit. The representative catchment for the reach was assumed to be that at point A in Figure 4.6 where the catchment area is 61 square kilometres. In the watershed model this catchment was divided into 14 subcatchments for which time-area relationships were developed. The three hydrographs were generated using the fifty year, four hour storm over the catchment, the difference between them being in the form of the hyetograph used and the degree of attenuation introduced in the channels and two dams upstream of the reach. Hydrograph Nos. 1 and 2 were generated using a Chicago type hyetograph from which losses in the form of initial abstractions, surface detention and infiltration were deducted. Hydrograph No.3 was generated using a rectangular hyetograph with a uniform loss rate given as a fraction of the rainfall depth, equivalent to the run-off coefficient used with the Rational Method. The three hydrographs are shown in Figure 4.7. The use of the Chicago type hyetographs results in hydrographs from the individual subcatchments which have a higher peak flow and are of shorter duration than those obtained using the rectangular hyetograph. For Hydrograph No. 1 routing constants were selected so as to give what was expected to be the normal attenuation through the water course upstream of the reach. The influence of the two dams upstream is marked, with the run-off from the tributary catchment downstream of the dams causing an isolated peak in the hydrograph. With hydrograph No. 2 minimal attenuation was introduced in the watershed, with the dams left out of the model. The resulting hydrograph has three distinct peaks. For Hydrograph No. 3 limited attenuation in the channels and in the reservoirs was introduced, resulting in a hydrograph with a single peak and almost triangular shape.

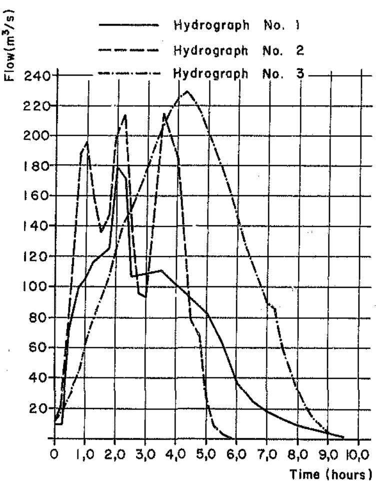


Figure 4.7 : Inflow Hydrographs Used in Model Runs

For the downstream boundary condition an *artificial control* section was introduced. This cross-section was selected so that with critical conditions occurring at the section the water levels at the adjacent section are close to the uniform flow levels for the range of flows encountered. This is equivalent to providing a single-valued stage discharge relationship at the most downstream section based on the uniform flow rating curve at the section.

For initial conditions a backwater computation with a steady flow of 10 $m^3/_s$ was carried out. Prior to introducing a flood hydrograph into the model it is necessary to run the model under steady state conditions for a period to allow initial perturbations to propagate out of the system. Using the results of the backwater computation as initial conditions a run was carried out using a constant inflow of 10 $m^3/_s$. With a time step of 0,01 hours and a value of θ of 0,5, the flow stabilizes after a period of 200 time steps. The maximum deviation in the flow during the run was less than 1,4 $m^3/_s$. Using a value of θ of 1,0 the initial perturbations were not more rapidly damped out, the flow stabilizing after the same number of time steps and with the same maximum deviation in the flow.

The first run of the model was made using Hydrograph No.1, with a time step of 0,01 hours and a value of θ of 0,5. Parasitic oscillations appeared in the solution during the initial steeply rising phase of the hydrograph. A comparison of the inflow and outflow hydrographs is shown in Figure 4.8. The front of the wave reaches the downstream end of the reach after about 1,5 hours. At this stage the flow decreases to almost zero and then oscillates wildly, reaching a maximum of more than 360 $m^3/_s$ before converging to a smooth curve.

The outflow hydrograph from the reach obtained after the second run, in which the value of θ was set at 1.0, is given in Figure 4.9. The dip in the outflow hydrograph is again evident after about 1,5 hours, but the oscillations apparent in the solution with $\theta = 0,5$ do not occur.

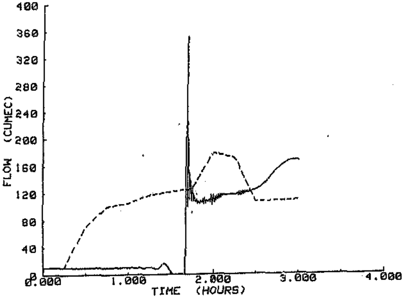


Figure 4.8 : Outflow Hydrograph with Hydrograph No.1 and θ = 0,5

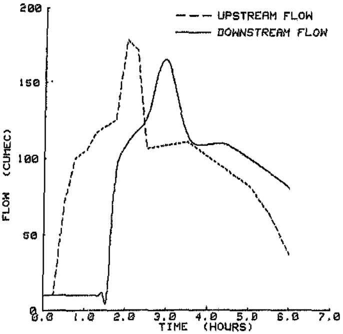


Figure 4.9 : *Outflow Hydrograph* with Hydrograph No.1 and θ = 1,0

According to Joliffe (1984), instability in four point schemes can occur when the time step is large relative to the wave period, when the wave front approaches an abrupt form rather than a gradually varied one, or when the cross-section has a highly non-linear relationship between the depth and cross-sectional area.

To check the effect of the size of time step on the solution time steps in the range 0,005 hours to 0,05 hours were used in a number of runs with Hydrograph No.1 as the inflow hydrograph. With a time step of 0,005 hours and $\theta = 1$ the computation stopped as a result of small depths occurring at the downstream end of the reach during the dip in the outflow hydrograph. The upper limit for the time step was found to be 0,02 hours; longer time steps resulted in unstable behaviour in the model.

Since the wave front resulting from Hydrograph No. 2 is steeper than that from Hydrograph No. 1, while that from Hydrograph No. 3 is the flattest, the effect of the steepness of the wave front on the solution can be examined. The outflow hydrographs obtained when using Hydrograph No.2 and 3 are shown in Figures 4.10 and 4.11. The dip in the outflow hydrograph is evident with Hydrograph No. 2 but it does not occur when Hydrograph No. 3 is used. It can be concluded that the dip is caused by the steepness of the wave front resulting from Hydrograph Nos. 1 and 2.

By introducing the same three hydrographs to a wide trapezoidal channel the limiting effect of the non-linear depth-area relationship typical of natural channels on the time step used in the computation can be examined. The trapezoidal channel used is 3,41 kilometres long with a base width of 100 metres, side slopes of 1:1, a longitudinal slope of 0,1% and a Manning roughness coefficient of 0,03. A distance step of 200 metres was used in the computation.

For each hydrograph time steps in the range 0,01 hours to 0,25 hours were used and the minimum values of θ which resulted in a

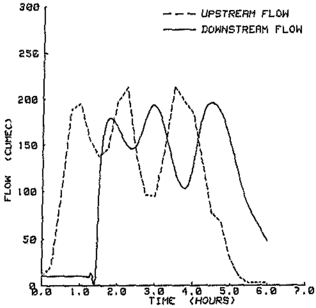


Figure 4.10 : Outflow Hydrograph with Hydrograph No.2 and θ = 1,0

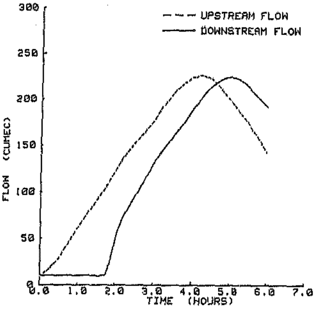


Figure 4.11 : Outflow Hydrograph with Hydrograph No.3 and θ = 1,0

smooth outflow hydrograph were determined. These values of θ are given in Tables 4.1 to 4.3 below. Where acceptably minor oscillations were evident in the solution the value of θ is marked with an asterisk.

Time step (h)	Min value of θ	Peak Outflow (cumec)	Peak Outflow for θ = 1 (cumec)
0,01	0,5	157,5	156,2
0,05	0,6	156,4	152,0
0,10	0,6	155,7	149,25
0,25	1,0	140,0	140,0

TABLE 4.1 Minimum values of θ - Hydrograph No.1

Time step (h)	Min value of θ	Peak Outflow (cumec)	Peak Outflow for θ = 1 (cumec)
0,01	0,5	189,1	187,7
0,05	0,6	188,0	184,1
0,10	0,6*	188,8	181,7
0,25	1,0	178,6	176,6

TABLE 4.2 Minimum values of θ - Hydrograph No.2

Time step (h)	Min value of θ	Peak Outflow (cumec)	Peak Outflow for θ = 1 (cumec)
0,01	0,5	223,1	222,9
0,05	0,5	223,2	222,0
0,10	0,5*	224,0	220,9
0,25	0,67	222,9	218,2

TABLE 4.3 Minimum values of θ - Hydrograph No.3

It is evident that much larger time steps can be used with this channel than with the natural channel. On the assumption that the solution with a ime step of 0,01 hours and θ = 1 is the closest to the exact solution, the solutions obtained with time steps of 0,1 hours are within about 1% of the exact solution. Solutions were obtained using time steps of 0,25 hours with θ = 1 for Hydrograph Nos. 1 and 2 and with θ = 0,67 for Hydrograph No. 3, although these solutions are somewhat damped. The longest time step that could be used with the natural channel was 0,02 hours. This limitation is therefore caused by the non-linearity of the depth-area relationship of the natural channel cross-sections.

The effect of the steepness of the wave front on the stability of the solution is again evident in Tables 4.1 to 4.3. Generally the value of θ required to obtain a smooth outflow hydrograph is higher with Hydrograph Nos. 1 and 2 than it is with Hydrograph No. 3.

The selection of the size of the time step and the value of θ is far more important in flood routing computations than it is in tidal computations. Where the time step is small relative to the period of the flood wave the solution is insensitive to the value of θ. Unless the stage-area relationship of the natural channel is almost linear, implying a well-defined channel with no flood plain flow, the use of short time steps and values of θ between 0,75 and 1,0 is recommended for the initial runs. Lower values of θ can be tried should subsequent runs be carried out.

5. SUMMARY AND CONCLUSION

5.1 Summary

Flood routing computations are required to trace the changes in a flood wave as it progresses down a river channel. These changes in the shape of the flood wave can be significant where the flood wave originates in urban catchments characterized by sharp-peaked run-off hydrographs and where the amount of storage in the river channel is significant relative to the volume of the flood.

The passage of a flood wave in a river channel is a form of gradually varied flow described by the de St. Venant equations. Generally flood routing computations are based on simplified forms of these equations, the complete equations requiring complex numerical techniques for their solution which are somewhat demanding on computer resources.

The simplified methods can be classified according to which terms are left out of the de St. Venant equations. In their most simplified form the equations form the basis of the kinematic models of which the Muskingum method is a well-known and commonly used example. These models incorporate artificial damping of the flood wave and the degree of attenuation is dependent on the routing parameters c and k. These parameters must be estimated from historical flood data although in some methods they can be derived from the channel properties. Because the kinematic models are based on a single valued rating curve, errors can occur where the rating curve exhibits a loop. In the Koussis model dynamic effects can be included to a limited extent but, as for all kinematic models, backwater effects are ignored.

The approximate dynamic and diffusion analogy models include the resistance term in the dynamic equation and can therefore reproduce backwater effects. However, because the inertia terms are not included the effect of the variation in velocity head on the flow profile is not reproduced. These models have greater applicability than the kinematic models and can be used for most flood routing computations.

The complete dynamic models, those based on the complete de St. Venant equations, can be classified according to the type of numerical scheme used in their solution. The characteristics based methods are not widely used for industrial models. They do serve as a check on other methods since their solution can be brought as close to the exact solution as is desired and are also used to determine boundary conditions within other models. Explicit finite difference schemes are relatively simple to program. However, restrictions on the time step used in the computation brought about by the Courant-Lewy-Friedrichs conditions for stability severely limits their applicability. The implicit finite difference schemes have no restriction on the time step providing a reasonable description of time-dependent boundary conditions is obtained. These schemes are used extensively in industrial modelling.

The finite difference scheme used in the model developed in this project is based on Verwey's variant of the Preissman scheme. The Preissman scheme is well documented and its stability has been thoroughly investigated by various researchers. Although its formulation is extremely complex it has the advantage that only one iteration is required at each time step. Verwey's scheme is simpler but requires two iterations per time step to obtain a satisfactory solution. The discretized equations are written in terms of the variables at the four adjacent points (i,n), (i+1,n), (i,n+1) and (i+1,n+1). The scheme uses the double sweep algorithm for the simultaneous solution of the unknown dependent variables at all points in the reach at the new time level. One boundary condition is required at each end of the reach to close the set of equations. Usually an inflow hydrograph is provided at the upstream end and a stage-time (tidal) or stage-discharge equation at the downstream end. To initiate the run the flow and water level must be defined at all points in the reach. Generally this is given by a steady state flow throughout the reach with the associated backwater curve defining the water levels.

In the model cross-section data is arranged in the form of a series of distance, ground level and Manning roughness values for each point in the cross-section. Cross-section data is stored on a file using a separate data handling program. The data is stored in a form compatible with that used in a backwater program developed by the writer. This has the advantage that the backwater program can be used to compute the initial water surface profile in the reach with a steady state flow that is some fraction of the peak flow of the inflow hydrograph. Reservoirs and lakes can be included in the reach, usually at the downstream end with the outlet characteristic defining the downstream boundary condition.

Inflow hydrograph data is provided in the form of a series of points on the hydrograph. Allowance has been made in the model for conversion routines to convert hydrographs generated by a hydrological model to the form used in the model.

A number of options are available in the model at present for defining the right hand boundary condition. Either a stage-discharge relationship, usually in the form of critical conditions, or a stage-time relationship, usually a tidal condition, can be selected.

Both the weighting coefficient θ and the size of the time step affect the solution. For values of θ less than 0,5 the scheme is always unstable. With θ between 0,5 and 0,6 parasitic oscillations appear in the solution for values of the Courant Number greater than unity. Values of θ between 0,67 and 1,0 generally yield stable results. The value of the time step must be selected to adequately describe the time-dependent boundary conditions. Instability in the scheme can also arise when the time step is long relative to the flood wave period, when the wave front approaches an abrupt form and when the stage-area relationship of the channel cross-section is highly non-linear. Shortening the time step in these circumstances will generally eliminate the instability.

In the main computation the section properties have to be calculated at all points in the reach for each iteration, i.e. twice per time step. Initially the model was set up such that these values were calculated for each sub-section in a cross-section and then summated for the cross-section. This was found to be a very time-consuming part of the calculation and a routine was devised in which a table of values of the section properties at discrete water levels is set up for each cross-section. The required values at intermediate water levels can then be interpolated. This routine was found to reduce the computation time by a factor of 3.

The model is set up such that the dependent variables are continuously overwritten, only the current estimates of the unknown dependent variables at the new time level and the known values from the previous time level being retained in memory. A printout of the solution after each time step can therefore be obtained to provide a detailed description of the simulation. At the end of the run a plot of the inflow and outflow hydrographs can be obtained and the maximum flows and water levels at each section are tabulated. The results of an independent mass balance calculation are also printed. This is based on a comparison between the difference in the volume in the reach at the start and end of the run and the net inflow to the reach over the period of the simulation.

The model was applied to two different flow situations. In the first application the tidal motion in the Swartvlei Estuary and lake system was modelled. The estuary has been modelled by NRIO and tide gauge readings at a number of points in the estuary are available. A hydrographic survey of the estuary was carried out by NRIO and with this data, together with the tide gauge readings, a check on the model could be made. Using a constant inflow of 2 $m^3/_s$ to the lake and a tidal water level defined by the simplified tidal equation as the boundary condition at the sea the model was able to reproduce the measured water levels to within about 50 mm.

In this application a time step of 0,1 hour was used, with a value of the weighting coefficient θ of 0,6. Errors in the mass balance of about 11% were found to occur with this value of θ.

In the second application a reach of the Rietspruit was modelled using the fifty year flood hydrograph. The Rietspruit catchment falls within the Brakpan municipal area and is generally zoned for residential development. The water course comprises a wide flood plain with no defined channel for which cross-sections were obtained from 1:2000 topographic mapping. Three hydrographs of different shapes and steepness were generated using the watershed model set up to determine the fifty year flood lines along the reach. The steepness of the wave front occurring in the channel was found to affect the stability of the computation. For the steeper hydrographs an oscillation occurs at the front of the wave which can destroy the solution. This effect limits the size of the time step to about 0,02 hours. The stability of the computation was also found to be affected by the degree of non-linearity in the depth-area relationship of the cross-sections used. By introducing the same hydrographs to a wide prismatic channel, time steps up to 0,25 hours could be used in the computation without any evidence of the oscillation at the wave front. In a comparison of the peak outflow from the reach obtained for time steps in the range of 0,01 hours to 0,25 hours with various values of θ it was found that using a short time step with θ = 1 generally yields results which are within 1% of those obtained with the minimum value of θ for which a smooth solution is obtained for the same time step. Using a longer time step the error in solution with $\theta = 1$ increases, values of θ between 0,67 and 0,80 being necessary to improve the solution.

5.2 Conclusion

A flood routing model has been developed which is suitable for application to single channel reaches where subcritical conditions prevail. It has the advantage over more simplified techniques that routing constants do not have to be estimated, the cross-sections of the water course with estimated Manning roughness coefficients being entered in lieu thereof. Furthermore, since the model outputs the maximum water level reached by the flood along the reach it is not necessary to carry out backwater computations to determine these levels, as is often required with simplified models, and no additional data than would be used for a flood line calculation is required.

In its present form the model could be included in a multiple channel watershed model where backwater influences between channels are negligible. Further development would permit the modelling of dendritic channel networks where inter-channel backwater effects are taken into account. In specific applications the model could be expanded to include internal boundary conditions such as weirs, bridges etc.

APPENDIX A

USER INSTRUCTIONS

APPENDIX A1 :
USER INSTRUCTIONS - FLOOD ROUTING PROGRAM : FLOW MOD

The model was written on an HP9845T with 1,2 megabytes of random access memory and the Structural Programming and Advanced Programming ROMs (Read only memory). The system is connected to a Shared Resource Manager (SRM) with 64 megabyte disc storage. Data files can be read either from the SRM or from a local disc drive.

1. Setting the Default Mass Storage Device

After the program is loaded and the RUN key pressed the following prompt appears on the screen :

Local (L) or Remote (R)

The user enters (L) for a local mass storage unit such as one of the built-in tape drives on the HP9845 or a floppy disc drive. The prompt

Enter the local device address (e.g. :H7,0,1)
then appears on the screen, to which the user can respond according to the type of storage device.

For an SRM based system the user enters (R). The prompt

Enter the directory path
then appears and the user enters the path to the directory in which the data files are stored or are to be stored.

When the program is re-run the prompt
CURRENT MASS STORAGE IS (CONT if OKAY - LEAVE BLANK TO RE-ENTER)
together with the last entered storage device address is displayed. If the address is still applicable the user simply

presses the CONT key. To change the device address to user first clears the line and then presses the CONT key; the program then resumes operation as described above after switch-on.

2. Cross-section data entry

The cross-section data file is read next. The user enters the name of the cross-section data file when the prompt :

Enter name of data file containing cross-section data?

appears on the screen. The data file is created using a separate program "BACKDATA" for which the user instruction are given in Appendix A2. The program assumes that the cross-sections are ordered from the downstream end of the reach to the upstream end, as is required for a backwater computation.

3. *Setting of the time-step, total run time and weighting* coefficient

These are entered next with the appearance of the prompt :

ENTER TIME STEP, TOTAL RUN TIME (HOURS) AND WEIGHTING COEFFICIENT. The time step and total run time are given in hours. The time step should be between 0,01 and 0,25 hours, depending on the steepness of the inflow hydrograph and the irregularity of the channel. For normal flood routing in natural channels a time step of 0,01 hours is recommended. For prismatic channels the time step can be increased to 0,1 hours. For tidal computations a time step of 0,1 hours can be used with natural channels.

The only limitation on the total run time is that it should not exceed 5 000 time steps. The inflow hydrograph data should span the total run time.

The weighting coefficient must be between the limits of 0,5 and 1,0. It is desirable to have it as low as possible so as to reduce numerical damping introduced by having it greater 0,5. However, if a small time-step is used, the numerical damping becomes insignificant and a weighting coefficient of 1,0 can be used. A practical lower limit for the weighting coefficient is about 0,67; values lower than this often result in parasitic oscillations appearing in the solution.

4. Inflow Hydrograph Data Entry

The following options are printed on the screen for the inflow hydrograph data entry :

INFLOW HYDROGRAPH DATA
READ DATA FROM FILE :
1. Illudas generated
2. WITWAT generated
3. Generated by option 4 below
4. ENTER DATA MANUALLY
Select option (1, 2, 3, or 4)

Options 1 and 2 :
The first two options are not as yet operative.
Option 3 :
Selecting option 3 causes the following prompt :

NAME OF HYDROGRAPH FILE?
After the name of the file is entered the opportunity is given to factor the ordinates of the inflow hydrograph with the prompt
DO YOU WISH TO MULTIPLY ALL INFLOW ORDINATES BY A CONSTANT? (Y/N)
To factor the ordinates of the hydrograph as read from the data file the user enters Y which causes the prompt :
ENTER THE CONSTANT

If no factoring is required the user enters N to the above question. This facility is useful where the hydrograph is generated using unit hydrograph techniques. The ordinates of the unitgraph are then stored in the data file and hydrographs of various recurrence intervals can be obtained by entering a factor equal to the excess rain for the associated recurrence interval and storm duration.

Option 4 : Enter Data Manually
Where the inflow hydrograph data has not been entered previously and stored in a file it must be entered manually. This is done by selecting option 4 from the above list of options. The data entered in the form of time and flow co-ordinate pairs with the following prompt appearing before each pair is entered.

ENTER TIME (HRS) & INFLOW (M3/S) - (CONT WHEN FINISHED)
As each pair is entered it is printed in tabular form on the screen. Once all data has been entered the user presses the CONT key leaving the entry line blank. The data can then be edited, the prompt
DO YOU WISH TO MAKE ANY CHANGES TO THE ABOVE DATA? (Y/N)
appearing on the screen.
To correct the data entered the user enters Y and the prompt
ENTER NO. OF POINT TO BE CHANGED - (CONT WHEN FINISHED)
appears. After the number of the point to be changed (say point 5) is entered the current values of the time and flow co-ordinates for point 5 are displayed in the entry line together with the prompt
TIME, INLFOW for point 5 displayed below, change as required.
The user can re-enter the co-ordinate pair entirely or, by moving the cursor to the appropriate position, change selected digits. The next point can then be edited. To exit the data editing mode the user leaves the entry line blank and presses the CONT key when prompted for the number of the point to be changed.

The inflow hydrograph can then be stored in a data file. The user is asked for the name of the file.

ENTER THE NAME OF THE FILE IN WHICH YOU WISH TO STORE HYDRO-GRAPH DATA.
The length of the file name must obviously comply with the requirements of the storage device, being a maximum of 6 characters long for local devices and 15 characters for the SRM.

5. Specifying the Downstream Boundary Conditions

Five options are available for specifying the downstream boundary conditions whcih are printed on the screen as follows :

RHS BOUNDARY CONDITIONS
Stage-Time relationship given
1. *Tidal Equation (M2 and S2 components only)*
2. Water level at discrete time intervals
Stage-Discharge relationship given
3. Critical conditions
4. Weir-type formula
5. Rating curve co-ordinates.
Select option (1, 2, 3, 4 or 5)

Only options 1 and 3 have been developed so far, these being the most common.

Option 1 - Tidal equation :
The form of the equation and an explanation of the terms is printed on the screen as follows :

TIDAL EQUATION

LEVEL = MSL + Am2 * SIN (2 * PI * T/Tm2 + As2 * SIN
(2 * PI *(T + Phi)/Ts2)

Where MSL = DATUM FOR MEAN LEVEL

Am2 = AMPLITUDE OF M2 COMPONENT

As2 = AMPLITUDE OF S2 COMPONENT

Tm2 = PERIOD OF M2 COMPONENT (= 44714s)

Ts2 = PERIOD OF S2 COMPONENT (= 43200s)

[illegible] = PHASE DIFFERENCE BETWEEN M2 and S2 COMPONENTS

ENTER Msl, Am2, As2 (all in metres) and the Phase Difference (seconds)

The four values are entered, separated by commas. To achieve a constant water level as the downstream boundary condition, the amplitudes can be entered as zeros. The hour of the tide is then entered with the prompt :

HOUR OF TIDE (0 TO 3,125 and 9,375 TO 12,5 RISING, 3,125 TO 9,375 FALLING)

By means of the values of Phi and the hour of the tide the state of the tide in the spring-neap-spring cycle at the start of the computation can be fixed. With both values set to zero the compuation starts with a spring tide; with Phi set at Ts2/2 and the hour of the tide set to zero, it starts with a neap tide. Similarly with Phi set to zero and the hour of the tide set to half the spring-neap-spring cycle period of 354 hours, the computation starts with a neap tide.

The values of Msl, Am2, Phi and the hour of the tide are printed on the screen. The user has the chance to correct any errors made on entering the data when the prompt

DO YOU WISH TO MAKE ANY CHANGES TO THE ABOVE DATA? (Y/N).

If Y is entered the user can re-enter the data for the tidal equation in the manner described above.

Option 3 - Critical conditions :
If option 3 is selected, i.e. critical conditions specified at the downstream boundary, no further information is required to describe the boundary condition and the initial conditions can then be specified as described below.

6. Initial Conditions

There are thus two options for entering the initial conditions which are displayed on the screen as follows :

INITIAL FLOWS AND LEVELS GIVEN AT EACH SECTION

1. Enter data

2. Read data from disc

Enter option (1 or 2)

Option 1 - Enter data
The data is entered in the form of the flow and level at each cross-section. The program prompts the user with
AT SECTION ...... ENTER FLOW, LEVEL
The value for the flow and level for the particular section are entered, seperted by a comma. As the data is entered it is tabulated on the screen. After the initial data for all cross-sections has been entered the data can be edited. The editing routine is entered by entering (Y) to the prompt :
DO YOU WISH TO MAKE ANY CHANGES TO THE ABOVE DATA? (Y/N)
The data for any section can be changed by entering the number of the section when prompted.
ENTER THE SECTION AT WHICH EDITING REQUIRED (CONT IF FINISHED)
The current values of initial flow level at the particular section are then displayed together with the prompt
EDIT INITIAL FLOW, LEVEL AT SECTION ...
and the user can either re-enter the data line completely or change selected digits in the displayed data line using the cursor.

The editing mode can be terminated by leaving the entry line blank when prompted for the number of the section at which editing is required.

The initial data is then stored in a data file for possible use in other runs. The name of the file is entered when the following prompt appears on the screen :

ENTER NAME OF FILE FOR STORING INITIAL DATA.

Program operation then proceeds with the selection of the form of the output for the run.

Option 2 - Read data from disc

The name of the data file containing the initial data is requested with the prompt :

ENTER NAME OF FILE IN WHICH INITIAL DATA IS STORED

The file containing the initial data can either be created as described under option 1 above or it can be created using the results of a backwater run. This latter method is the most convenient. The backwater program uses the cross-section data file and requires only the steady state flow in the reach to determine the steady state backwater profile. The results obtained using the backwater program are fairly consistent with those obtained from a steady state analysis by the model and thus the period required to stabilize the system prior to imposing an unsteady condition on the model is usually not significant.

7. Selection of Type of Output

There are three possible forms of printed output from the model. Since the values of the flow and level at all points in the computational grid are not retained in memory the printed output provides the only detailed record of the run. The options are displayed on the screen as follows :

OUTPUT TYPE

1. Flow and level at all time steps
2. Flow and level at selected time intervals
3. Flow and level at selected cross- sections.

Enter option (1, 2 or 3)

Option 1 : Flow and Level at all time steps
With option 1 the results of the computation at all cross-sections are printed after ach time step. Where the time step is small relative to the total run time the amount of the print-out is excessive and it is preferable to select option 2 or 3.

Option 2 : Flow and Level at selected time intervals
The results of the computation at all cross-sections are printed after a selected time interval. The time interval is entered after the following prompt :

ENTER THE TIME INTERVAL (in hours) FOR OUTPUT
The time interval entered should obviously be a whole number of time steps.

Option 3 : Flow and level at selected cross sections
This option gives the most compact form of output. The data for up to five cross-sections is output at selected time intervals. The cross-sections at which the output is required are entered with the prompt.

Enter the cross-sections at which output required
(Max. 5 - eg. 1,5,8,23,44)

The time interval for output is then entered as for Option 2 above.

8. Changing Data for a New Run

After a run is completed the user is given the facility to change part of the data and re-run the model. The options for changing the data are displayed on the screen as *follows :*

CHANGE DATA FOR NEW RUN

1. Cross-sections
2. LHS Boundary conditions
3. RHS boundary conditions
4. Initial conditions
5. Theta
6. NO CHANGES - END OFF.

SELECT OPTION (1, 2, 3, 4, 5 or 6)

The program loops back to the list of options after each data change, allowing more than one set of data to be changed, until the entry line is left blank and the CONT button depressed, in which case the new run is started, or option 6 is selected. If option 6 is selected program execution is terminated. If the entry line is left blank and the CONT button is depressed the *type of output required is requested as is described* in Section 7 above. The remaining options are described below.

Option 1 : Cross-section data
The new cross-section data is entered as described in Section 2 above.

Option 2 : LHS Boundary Conditions
The new time step and total run time are first entered. These are requested by prompt
ENTER TIME STEP, TOTAL RUN TIME (HOURS)
Both quantities should be given in hours. The inflow hydrograph data is then requested as is described in Section 4.

Option 3 : RHS Boundary Conditions
The new downstream boundary condition is entered as described in Section 5.

Option 4 : Initial Conditions
The user has the facility to use the state of the reach at the end of the previous run as initial conditions for the new run:
DO YOU WANT TO USE DATA FROM PREVIOUS RUN AS INITIAL CONDITIONS? (Y/N)
If Y is input the program requests the name of the file in which the initial conditions are to be stored. If N is input the new initial conditions are input as described in Section 6.

Option 5 : Theta
The new value of the weighting coefficient Theta is requested with the prompt :
ENTER VALUE FOR THETA BETWEEN 0,5 and 1,0

APPENDIX A2

USER INSTRUCTIONS - CROSS-SECTION DATA INPUT PROGRAM : BACKDATA

This program creates a data file containing the cross-sections of a river reach required for use in the one-demensional flow model FLOW_MOD or in the backwater program *BACKWATER. The file* name is made up of the prefix "Reac" and the number of the reach, which can be between 0 and 99 inclusive.

The program begins with the setting of the default mass storage device.

After the program is loaded and the RUN key pressed the following prompt appears on the screen :

Local (L) or Remote (R)

The user enters **(L)** for a local mass storage unit such as one of the built-in tape drives on the HP9845 or a floppy disc drive. The prompt

Enter the local device address (e.g. :H7,0,1)
then appears on the screen, to which the user can respond according to the type of storage device.

For an SRM based system the user enters **(R)**. The prompt

Enter the directory path
then appears and the user enters the path to the directory in which the data files are stored or are to be stored.

When the program is re-run the prompt
CURRENT MASS STORAGE IS (CONT if OKAY - LEAVE BLANK TO RE-ENTER)
together with the last entered storage device address is displayed. If the address is still applicable the user simply presses the CONT key. To change the device address to user

2

first clears the line and then presses the CONT key; the program then resumes operation as described above *after switch-on.*

The number of the reach is entered when the following prompt is displayed on the screen :

Enter the Reach number - (CONT IF FINISHED)

The program checks whether a data file for the reach number entered already exists at the default mass storage address and reads the file into memory if it does. The main menu for the program is then displayed on the screen as follows :

DATA FOR REACH ..
1. ENTER CROSS SECTION DATA
2. PRINT HARD COPY OF CROSS SECTION DATA
3. EDIT CROSS SECTION DATA
4. RUN BACKWATER PROGRAM WITH DATA FOR REACH ...
5. ENTER A NEW REACH

ENTER OPTION (1, 2, 3, 4 or 5)

Option 1 : Enter cross section data
The data for each point in the cross-section is entered with the prompt :
ENTER DISTANCE, LEVEL, MANNING N - (CONT WHEN FINISHED WITH THIS SECTION)

The distance given is that from some arbitrary origin in the cross-section. The three quantities entered must be separated by commas. As the data for each point is entered, it is displayed in tabular form on the screen. Once all the points in the cross-section have been entered, the user leaves the entry line blank and depresses the CONT key. The user then has the facility to edit the data just entered :

DO YOU WISH TO MAKE ANY CHANGES TO THE ABOVE DATA? (Y/N)

Entering Y to this prompt allows the user to change the data for the cross-section just entered. It is only possible to change data for other cross-sections once all data has been entered by selecting Option 3 from the main menu. The editing of data is discussed further under this option.

Once the changes to cross-section have been made the next cross-section can be entered. The distance between the last entered cross-section and the cross-section about to be entered is requested with the prompt

CHANNEL LENGTH BETWEEN SECTION ... AND ... (CONT IF ... IS LAST SECTION)

If the *last cross-section entered is the final cross-section* in the reach the user leaves the entry line blank and presses the CONT key. The data for the reach is then stored by the program *and the main menu is displayed.* If a distance is entered in response to the above prompt the data at each point in the cross-section is then entered as described above.

Option 2 : Print hard copy of cross-section data

A print out of the cross-section data for the reach on the thermal printer is obtained if this option is selected.

Option 3 : Edit cross-section data

Three options are available for editing the cross-section data for a reach which are displayed as follows :

EDITING OF DATA FOR REACH ...

1. INSERT A NEW CROSS SECTION

2. *CHANGE EXISTING DATA*

3. DELETE A CROSS SECTION

ENTER OPTION (1, 2 or 3) - CONT WHEN FINISHED

Edit option 1 : Insert a new cross-section

The number of the cross-section immediately below the one to be inserted is first entered. The prompt is :

ENTER NO. OF CROSS SECTION BELOW NEW SECTION

The distance from the adjacent lower-numbered section and the data for the cross-section are then entered as described under Option 1 above.
It should be noted that the distance between the section inserted and the adjacent higher-numbered section retains the value of the distance interval into which the section was inserted. The length of the reach thus increases by the distance between the inserted section and its adjacent lower-numbered section.

Edit Option 2 : Change existing data
The user first enters the number of the cross-sectin to be changed:
WHICH CROSS-SECTION DO YOU WISH TO CHANGE? (*CONT WHEN FINISHED*)
After a cross-section has been changed this prompt is again displayed to allow any number of cross-sections to be altered. *Once all changes have been made the user* leaves the entry line blank and presses the CONT key. The program then reverts to the Editing Menu.

When the user has entered the number of the cross-section to be changed the following prompt is displayed :
DO YOU WANT TO CHANGE THE WHOLE SECTION? (Y/N)

Entering Y in response to this prompt allows the user to completely redefine the cross-section. Data entry for the cross-section then follows in the manner described under Option 1 above. Entering N in response to the above prompt allows the user to change *individual points in the cross-section.* The distance between the section and the adjacent lower-numbered section can be changed when the following prompt is displayed :
***CHANGE THE DISTANCE BETWEEN SECTION ...* and SECTION ...**
The current value of the distance is displayed in the data entry line. If no change is required the user merely presses the CONT button; otherwise the new distance is entered. The next prompt is :
WHICH POINT DO YOU WISH TO CHANGE IN CROSS-SECTION ... (CONT WHEN FINISHED)

When the number of the point to be changed is entered the current values of the distance, level and Manning toughness coefficient for the point are displayed in the data entry line together with the prompt

CHANGE DISTANCE, LEVEL OR MANNING N AT POINT ...

The user can then change the values as required. *The data* for the cross-section is then printed on the screen and the next point to be changed can then be entered. If the entry line is left blank and the CONT button depressed when the prompt for the next point to be changed in the cross-section is displayed the next cross-section can be changed.

Edit Option 3 : Delete a cross-section

The number of the cross-section to be deleted is requested with the prompt :

WHICH CROSS-SECTION DO YOU WISH TO DELETE? (CONT WHEN FINISHED)

Leaving the entry line blank causes program operation to resume with the menu for the editing options.

It should be noted that deleting a cross-section results in the renumbering of all higher numbered cross-sections. If a number of cross-sections are to be deleted they should be deleted in order of decreasing magnitude.

When *no further editing* is required the user exits edit mode by leaving the entry line for the editing menu blank. The program then stores the data. If a data file under the reach number already exists the user is given the option to change the reach number or purge the existing file :

ENTER A NEW REACH NO. OR *PURGE FILE Reac* ...

The file must be purged manually by typing in

PURGE "Reac X"

Where X is the reach number, and pressing the EXECUTE key. Pressing the CONT key thereafter will result in the new data file being stored under this name.

Option 4 : Run backwater program with data for Reach ...

Selecting this option results the backwater program "BACKWATER" being loaded.

Option 5 : Enter a new reach

This option allows data for a new reach to be entered. Program operation resumes with the number of the new reach being entered.

APPENDIX B

PROGRAM LISTINGS

APPENDIX B1 : PROGRAM LISTING - FLOW_MOD

```
10  ! "FLOW_MOD"     ! ONE DIM. FINITE DIFFERENCE
20  !                  OPEN CHANNEL UNSTEADY FLOW MODEL
30  !                  BASED ON de St Venant EQUATIONS AND SOLVED USING
40  !                  THE PREISSMAN SCHEME AS MODIFIED BY VERWEY
50  !
60   ! DIMENSION ARRAYS !
70  !
80      OPTION BASE 1
90      COM Distance(100,20),Np(100),Level1(20),Mann(20),Path$[50],Device$[20],Ru
n
100     DIM Reach(100,20,3),Points(100),Lx(100)
110 ! Section Properties:
120     DIM Area1(100),Area2(100),Area(100),Bsurf1(100),Bsurf2(100),Alpha2(100),B
surf(100)
130     DIM Beta1(100),Beta2(100),Dx(100),X(100),Ybot(100),Gg(100,100)
140     DIM Level(100,20),B(100,20),Ar(100,20),P(100,20),Alpha(100,20),Beta(100,2
0),Pointer(20),Mn(100,20),Counter(100)
150     DIM Np1(100)
160 ! Boundary and Initial Conditions:
170     DIM Q_lhs(5000),Q_rhs(5000),Q_discrete(5000),Stage(100),Stage_out(100)
180     DIM Qstart(100),Ystart(100),Y_lhs(5000),Y_rhs(5000)
190 ! Coefficients:
200     DIM A2(100),B1(100),B2(100)
210     DIM C2(100),D1(100),D2(100),E1(100),E2(100),F(100),G(100),H(100),I(100)
220     DIM J(100),K(100),K1(100),K2(100),Para1(100),Para2(100),Para3(100)
230 ! Main Variables:
240     DIM Q1(100),Q2(100),Y1(100),Y2(100),Qmax(100,2),Ymax(100,2),Qmin(100),Ymi
n(100)
250 ! Duput_control:
260     DIM Cross_print(20)
270 !
280 ! DIMENSION STRINGS
290 !
300     DIM Edit$[80]
310 !
320 ! SET CONSTANTS AND DEFFULTS :
330     RAD
340     PRINTER IS 16
350 !
360     Theta=.5
370     Psi=.5
380     N_factor=1
390     Complete=1
400 !
410 ! ************************************************************************
420 !                          MAIN CONTROL ROUTINE
430 ! ************************************************************************
440     GOSUB Storage_device        ! SETS MASS STORAGE DEVICE
450     GOSUB Data_input            ! DATA INPUT MASTER ROUTINE
460     LOOP
470        GOSUB Main_calc          ! CALCULATION MASTER ROUTINE
480        GOSUB Data_change        ! ALLOWS CHANGES TO DATA FOR NEXT RUN
490     EXIT IF Op_ch$="5"          ! EXITS IF NO FURTHER RUNS REQUIRED
500        GOSUB Output_type        ! SETS PRINT INTERVAL FOR NEW RUN
510     END LOOP
520     STOP
530 ! ************************************************************************
```

```
530 ! ***********************************************************************
540 !                              DATA INPUT                           Level1
550 ! ***********************************************************************
560 Data_input:                      !
570     GOSUB Cross_sect
580     GOSUB Redim                   ! REDIMENSIONS ARRAYS
590     INPUT "ENTER TIME STEP , TOTAL RUN TIME (HOURS),Weighting coefficient THE
TA",Dt1,Tmax,Theta
600     Dt=Dt1*3600
610     GOSUB Lh_bound               ! SETS LHS BOUND. CONDITIONS
620     GOSUB Rh_bound               ! SETS RHS BOUND. CONDITIONS
630     GOSUB Init_cond              ! SETS INITIAL CONDITIONS
640     GOSUB Output_type            ! SETS INTERVAL FOR PRINT OUT
650     RETURN
660 !
670 ! ***********************************************************************
680 !                          MAIN CALCULATION                         Level 1
690 ! ***********************************************************************
700 Main_calc:                       !
710     Run=Run+1
720     MAT Q2=Qstart
730     MAT Y2=Ystart
740     MAT Ymax=ZER
750     MAT Qmax=ZER
760     MAT Ymin=(999)
770     MAT Qmin=(999)
780     N=0
790     GOSUB Printhead              ! PRINTS HEADING FOR RUN
800     GOSUB Print                  ! PRINTS INITIAL CONDITIONS
810     N=1
820     FOR I=1 TO Nsections
830        GOSUB Section_prop
840     NEXT I
850     N_iterations=2               ! SETS No. OF ITERATIONS PER TIME STEP
860     Qsum_in=Qstart(1)/2          ! INITIALIZES VALUES FOR MASS BALANCE
870     Qsum_out=Qstart(Ii)/2
880     GOSUB Storage                ! CALCS INITIAL STORAGE FOR MASS BALANCE
890     Vol_start=Volume
900     LOOP
910     EXIT IF N>Nn-1
920        MAT Y1=Y2                 ! INITIALIZES VALUES USING PREVIOUS TIME STEP
930        MAT Q1=Q2
940        MAT Area1=Area2
950        MAT K1=K2
960        MAT Beta1=Beta2
970        MAT Bsurf1=Bsurf2
980        FOR Iter=1 TO N_iterations
990           GOSUB Coefficients
1000          GOSUB Sweep
1010          FOR I=1 TO Nsections
1020             GOSUB Section_prop
1030          NEXT I
1040       NEXT Iter
1050       IF FRACT(N*Dt/Print_int)=0 THEN GOSUB Print
1060       IF N>1 THEN
1070          Qsum_in=Qsum_in+Q_lhs(N)
1080          Qsum_out=Qsum_out+Q_rhs(N)
1090       END IF
1100       N=N+1
1110    END LOOP
1120    Qsum_in=(Qsum_in+Q_lhs(Nn)/2)*Dt
1130    Qsum_out=(Qsum_out+Q_rhs(Nn)/2)*Dt
1140    GOSUB Print_max
1150    CALL Plot_hydro(Nn,Dt1,Q_lhs(*),Q_rhs(*),"FLOW  (CUMEC)")
1160    IF Rh_bc=2 THEN CALL Plot_hydro(Nn,Dt1,Y_lhs(*),Y_rhs(*),"LEVEL  (METRES)
")
1170    GOSUB Storage                ! CALCS FINAL STORAGE IN REACH
```

```
1180   Vol_end=Volume
1190   GOSUB Print_balance
1200   RETURN
1210 ! ****************************************************************
1220 !                          CHANGE DATA FOR NEW RUN          Level 1
1230 ! ******************************************************************
1240 Data_change:                 !
1250   LOOP
1260      PRINT PAGE,"  CHANGE DATA FOR NEW RUN",LIN(1)
1270      PRINT "  1. Cross_sections",LIN(1)
1280      PRINT "  2. LHS Boundary Conditions",LIN(1)
1290      PRINT "  3. RHS Boundary Conditions",LIN(1)
1300      PRINT "  4. Initial Conditions",LIN(1)
1310      PRINT "  5. Theta",LIN(1)
1320      PRINT "  6. NO CHANGES - END OFF"
1330      LINPUT "SELECT OPTION (1,2,3,4,5 or 6) - CONT TO START RUN",Op_ch$
1340   EXIT IF (Op_ch$="") OR (Op_ch$="6")
1350      Op_ch=VAL(Op_ch$)
1360      SELECT Op_ch
1370      CASE 1
1380         GOSUB Cross_sect
1390      CASE 2
1400         INPUT "ENTER TIME STEP , TOTAL RUN TIME (HOURS)",Dt1,Tmax
1410         Dt=Dt1*3600
1420         GOSUB Lh_bound
1430      CASE 3
1440         GOSUB Rh_bound
1450      CASE 4
1460         LOOP
1470            INPUT " DO YOU WANT TO USE DATA FOR LAST RUN AS INITIAL CONDITIO
NS ? (Y/N)",Edit$
1480         EXIT IF (Edit$="N") OR (Edit$="Y")
1490            BEEP
1500            DISP "ENTER Y OR N - ";
1510         END LOOP
1520         IF Edit$="Y" THEN
1530            MAT Ystart=Y2
1540            MAT Qstart=Q2
1550            GOSUB Init_store
1560         ELSE
1570            GOSUB Init_cond
1580         END IF
1590      CASE 5
1600         LOOP
1610            INPUT "ENTER VALUE FOR THETA BETWEEN 0,5 AND 1,0 ",Theta
1620         EXIT IF (Theta>=.5) AND (Theta<=1)
1630            BEEP
1640            DISP "INCORRECT ENTRY - ";
1650         END LOOP
1660      CASE ELSE
1670         BEEP
1680         GOTO 1330
1690      END SELECT
1700   END LOOP
1710   RETURN
1720 ! ****************************************************************
1730 !                          CROSS SECTION DATA               Level 2
1740 ! ******************************************************************
1750 Cross_sect:                  !
1760   PRINT PAGE,"  CROSS SECTION DATA",LIN(3)
1770   INPUT "Enter name of data file containing cross_section data ?",File$
1780   ON ERROR GOTO 1770
1790   ASSIGN #1 TO File$
1800   READ #1;Cum_np,Nxs
1810   Nsections=Nxs
1820   REDIM Reach(Nxs,20,3),Distance(Nxs,20),Np(Nxs),Np1(Nxs),Dx(0:Nxs-1),Count
er(Nxs)
1830   READ #1;Np(*),Dx(*),Reach(*)
1840   ASSIGN #1 TO *
1850   OFF ERROR
1860   MAT SEARCH Np,MAX;Mp
```

```
1870    REDIM Level(Nxs,Mp),Mn(Nxs,Mp),Ar(Nxs,Mp),B(Nxs,Mp),P(Nxs,Mp),Alpha(Nxs,M
p),Beta(Nxs,Mp)
1880    FOR Nx=1 TO Nxs
1890       I=Nxs-Nx+1
1900       Lx(I)=Dx(Nx-1)
1910       REDIM Level1(Np(Nx)),Pointer(Np(Nx))
1920       FOR J=1 TO Np(Nx)
1930          Distance(I,J)=Reach(Nx,J,1)
1940          Level1(J)=Reach(Nx,J,2)
1950          Mann(J)=Reach(Nx,J,3)
1960       NEXT J
1970       MAT SORT Level1 TO Pointer
1980       Jj=1
1990       FOR Ii=1 TO Np(Nx)
2000          J=Pointer(Ii)
2010          Wsi=Level1(J)
2020          IF Ii=1 THEN Wsi=Level1(J)+.001
2030          E=1
2040          LOOP
2050          EXIT IF Ii+E>Np(Nx)
2060          EXIT IF Level1(J)<>Level1(Pointer(Ii+E))
2070             E=E+1
2080          END LOOP
2090          Ii=Ii+E-1
2100          CALL Section_table(I,Nx,Wsi,B(I,Jj),Ar(I,Jj),K,P(I,Jj),Alpha(I,Jj),
Beta(I,Jj))
2110          Level(I,Jj)=Level1(J)
2120          Mn(I,Jj)=Ar(I,Jj)*(Ar(I,Jj)/P(I,Jj))^(2/3)/K
2130          Jj=Jj+1
2140       NEXT Ii
2150       Np1(I)=Jj-1
2160    NEXT Nx
2170    MAT Np=Np1
2180    MAT Dx=Lx
2190    Ii=Nsections
2200    Xx=Nsections
2210    RETURN
2220 ! ************************************************************************
2230 !                          REDIMENSION ARRAYS                    Level 2
2240 ! ************************************************************************
2250 Redim:
2260    REDIM Area1(Nxs),Area2(Nxs),Area(Nxs),Bsurf1(Nxs),Bsurf2(Nxs),Alpha2(Nxs)
,Bsurf(Nxs)
2270    REDIM Beta1(Nxs),Beta2(Nxs),Dx(Nxs),X(Nxs),Ybot(Nxs)
2280    REDIM A2(Nxs),B1(Nxs),B2(Nxs)
2290    REDIM C2(Nxs),D1(Nxs),D2(Nxs),E1(Nxs),E2(Nxs),F(Nxs),G(Nxs),H(Nxs),I(Nxs)
2300    REDIM J(Nxs),K(Nxs),K1(Nxs),K2(Nxs),Para1(Nxs),Para2(Nxs),Para3(Nxs)
2310    REDIM Qmax(Nxs,2),Ymax(Nxs,2)
2320    RETURN
2330 ! ************************************************************************
2340 !                        LHS BOUNDARY CONDITIONS                  Level 2
2350 ! ************************************************************************
2360 Lh_bound:
2370    PRINT PAGE,"  INFLOW HYDROGRAPH DATA:",LIN(2)
2380    PRINT "  READ DATA FROM FILE :",LIN(1)
2390    PRINT "  1.     Illudas generated"
2400    PRINT "  2.     WITWAT generated"
2410    PRINT "  3.     Generated by option 4 below",LIN(1)
2420    PRINT "  4. ENTER DATA MANUALLY",LIN(1)
2430    INPUT " Select option ( 1,2,3 or 4)",Op_inflo
2440    SELECT Op_inflo
2450    CASE 1,2
2460    CASE 3
2470       GOSUB Hydro_read
2480    CASE 4
2490       GOSUB Hydro_input
2500       GOSUB Hydro_store
2510    CASE ELSE
2520       BEEP
2530       INPUT "Enter 1,2,3 OR 4 TO INDICATE OPTION !!",Op_inflo
2540       GOTO 2440
```

```
2550   END SELECT
2560   GOSUB Discrete_hydro
2570   MAT Q_lhs=Q_discrete
2580   REDIM Q_rhs(Nn)
2590   Lh_bc=1
2600   RETURN
2610 ! ***********************************************************************
2620 !                          RHS BOUNDARY CONDITIONS                  Level 2
2630 ! ***********************************************************************
2640 Rh_bound:
2650   PRINT PAGE,"  RHS BOUNDARY CONDITIONS ",LIN(2)
2660   PRINT "  Stage-Time relationship given",LIN(1)
2670   PRINT "  1. Tidal Equation (M2 AND S2 components only)",LIN(1)
2680   PRINT "  2. Water level at discrete time intervals",LIN(1)
2690   PRINT "  Stage-Discharge relationship given",LIN(1)
2700   PRINT "  3. Critical conditions",LIN(1)
2710   PRINT "  4. Weir-type formula",LIN(1)
2720   PRINT "  5. Rating curve co-ordinates",LIN(1)
2730   INPUT " Select option (1,2,3,4 or 5)",Rh_bc
2740   SELECT Rh_bc
2750   CASE 1
2760      PRINT PAGE;" TIDAL EQUATION :",LIN(2)
2770      PRINT "LEVEL= MSL+ Am2*SIN(2*PI*T/TM2+As2*SIN((2*PI*T+Phi)/Ts2)",LIN(2
)
2780      PRINT "            where:  MSL=DATUM FOR MEAN LEVEL"
2790      PRINT "                    Am2=AMPLITUDE OF M2 COMPONENT"
2800      PRINT "                    As2=AMPLITUDE OF S2 COMPONENT"
2810      PRINT "                    Tm2=PERIOD OF M2 COMPONENT (=44714 s)"
2820      PRINT "                    Ts2=PERIOD OF S2 COMPONENT (=43200 s)"
2830      PRINT "                    Phi=PHASE DIFFERENCE BETWEEN M2 AND S2 COMPON
ENTS",LIN(1)
2840      INPUT "ENTER MSL (m),Am2 (metres),As2 (metres), and the Phase Differen
ce (seconds)",Msl,Am2,As2,Phi
2850      PRINT "                    MSL= ";Msl;" m"
2860      PRINT "                    Am2= ";Am2;" m"
2870      PRINT "                    As2= ";As2;" m"
2880      PRINT "                    Phi= ";Phi;" s"
2890      INPUT "HOUR OF THE TIDE (0 TO 3.125 and 9.375 TO 12.5 RISING ,3.125 TO
 9.375 FALLING)",Tide_state
2900      PRINT LIN(1),"         Time of Tide= ";Tide_state;" h"
2910      GOSUB Edit
2920      IF Edit$[1,1]="Y" THEN 2840
2930      Tm2=44714
2940      Ts2=43200
2950      Tide_state=Tide_state*3600
2960   CASE ELSE
2970      BEEP
2980      DISP " INCORRECT ENTRY - TRY AGAIN - ";
2990      GOTO 2740
3000   END SELECT
3010   RETURN
3020 !
3030 ! ***********************************************************************
3040 !                          INITIAL CONDITIONS                       Level 2
3050 ! ***********************************************************************
3060 !
3070 Init_cond:
3080   PRINT PAGE,"  INITIAL CONDITIONS :",LIN(2)
3090   PRINT PAGE,"  INITIAL FLOWS AND LEVELS GIVEN AT EACH SECTION",LIN(2)
3100   PRINT "  1. Enter data",LIN(1)
3110   PRINT "  2. Read data from disc",LIN(1)
3120   INPUT "Enter option (1 or 2)",Op_init2
3130   SELECT Op_init2
3140   CASE 1
3150      PRINT PAGE," INPUT OF INITIAL FLOWS AND LEVELS",LIN(2)
3160      FOR I=1 TO Nsections
3170         DISP "AT SECTION ";I;
3180         INPUT " ENTER FLOW,LEVEL",Qstart(I),Ystart(I)
3190         PRINT USING " 3X,4D,5X,4D.DD,5X,4D.3D";I,Qstart(I),Ystart(I)
3200      NEXT I
3210      GOSUB Edit
```

```
3220       IF Edit$[1,1]="Y" THEN
3230          LOOP
3240             LINPUT "ENTER THE SECTION AT WHICH EDITTING REQUIRED ( CONT IF
 FINISHED )",Edit$
3250            EXIT IF Edit$=""
3260               I=VAL(Edit$)
3270               Edit$="     "&VAL$(Qstart(I))&Comma$&VAL$(Ystart(I))
3280               DISP "EDIT INITIAL FLOW ,LEVEL AT SECTION ";I;
3290               EDIT Edit$
3300               GOSUB Strings
3310               Qstart(I)=VAL(Str$(1))
3320               Ystart(I)=VAL(Str$(2))
3330               PRINT USING " 3X,4D,5X,4D.DD,5X,4D.3D";I,Qstart(I),Ystart(I)
3340            END LOOP
3350       END IF
3360       GOSUB Init_store
3370    CASE 2
3380       GOSUB Init_read
3390    CASE ELSE
3400       BEEP
3410       INPUT "ENTER 1 OR 2 TO INDICATE OPTION !!",Op_init2
3420       GOTO 3130
3430    END SELECT
3440    Q_lhs(1)=Qstart(1)
3450    Q_rhs(1)=Qstart(Xx)
3460    Y_lhs(1)=Ystart(1)
3470    Y_rhs(1)=Ystart(Xx)
3480    RETURN
3490 !
3500 ! *******************************************************************
3510 !                      OUTPUT TYPE                          Level 2
3520 ! *******************************************************************
3530 Output_type:
3540    PRINT PAGE,"  OUTPUT TYPE : ",LIN(2)
3550    PRINT "  1. Flow and level at all Time Steps",LIN(1)
3560    PRINT "  2. Flow and level at Selected Time Interval",LIN(1)
3570    PRINT "  3. Flow and level at Selected Cross_sections",LIN(1)
3580    INPUT "Enter option (1, 2 or 3)",Op_out
3590    SELECT Op_out
3600    CASE 1
3610       Print_int=1
3620    CASE 2
3630       INPUT "ENTER THE TIME INTERVAL (in hours) FOR OUTPUT ",Print_int
3640       Print_int=Print_int*3600
3650    CASE 3
3660       LINPUT "ENTER THE CROSS SECTIONS AT WHICH OUTPUT REQUIRED (MAX 5 - eg.
 1,5,8,23,44",Edit$
3670       GOSUB Strings
3680       FOR I=1 TO All
3690          Cross_print(I)=VAL(Str$(I))
3700       NEXT I
3710       INPUT "ENTER THE TIME INTERVAL (in hours) FOR OUTPUT ",Print_int
3720       Print_int=Print_int*3600
3730    CASE ELSE
3740       BEEP
3750       DISP " ERROR ON DATA ENTRY - ";
3760       GOTO 3580
3770    END SELECT
3780    RETURN
3790 ! *******************************************************************
3800 !                CALCULATION OF SECTION PROPERTIES          Level 2
3810 ! *******************************************************************
3820 Section_prop:
3830    Step=1
3840    IF Y2(I)<Level(I,1) THEN Y2(I)=Y1(I)
3850    Wd=Y2(I)-Level(I,1)
3860    IF N=1 THEN
3870       Counter(I)=1
3880    ELSE
3890       IF Y2(I)<Y1(I) THEN Step=-1
3900    END IF
```

```
3910   LOOP
3920   EXIT IF Y2(I)>=Level(I,Np(I))
3930   EXIT IF (Y2(I)>=Level(I,Counter(I))) AND (Y2(I)<Level(I,Counter(I)+1))
3940      IF Counter(I)>1 THEN
3950         IF Y2(I)<Level(I,Counter(I)) THEN Step=-1
3960      ELSE
3970         IF Step=-1 THEN Step=1
3980      END IF
3990      Counter(I)=Counter(I)+Step
4000   END LOOP
4010   IF Y2(I)<Level(I,Np(I)) THEN
4020      J=Counter(I)
4030      Dy=Y2(I)-Level(I,J)
4040      F=Dy/(Level(I,J+1)-Level(I,J))
4050      Db=(B(I,J+1)-B(I,J))*F
4060      Bsurf2(I)=B(I,J)+Db
4070      Area2(I)=Ar(I,J)+(Bsurf2(I)-Db/2)*Dy
4080      Perim=P(I,J)+(P(I,J+1)-P(I,J))*F
4090      Man=Mn(I,J)+(Mn(I,J+1)-Mn(I,J))*F
4100      K2(I)=Area2(I)*(Area2(I)/Perim)^(2/3)/Man
4110      Alpha2(I)=Alpha(I,J)+(Alpha(I,J+1)-Alpha(I,J))*F
4120      Beta2(I)=Beta(I,J)+(Beta(I,J+1)-Beta(I,J))*F
4130   ELSE
4140      J=Np(I)
4150      Dy=Y2(I)-Level(I,J)
4160      F=Dy/(Level(I,J)-Level(I,J-1))
4170      Bsurf2(I)=B(I,J)
4180      Area2(I)=Ar(I,J)+Bsurf2(I)*Dy
4190      Perim=P(I,J)
4200      Man=Mn(I,J)
4210      K2(I)=Area2(I)*(Area2(I)/Perim)^(2/3)/Man
4220      Alpha2(I)=Alpha(I,J)
4230      Beta2(I)=Beta(I,J)
4240      Counter(I)=Np(I)
4250   END IF
4260   RETURN
4270 ! ******************************************************************
4280 !                 CALCULATION OF VOLUME OF STORAGE IN REACH       Level 2
4290 ! ******************************************************************
4300 Storage:
4310   Volume=0
4320   FOR I=1 TO Nsections-1
4330      Volume=Volume+(Area2(I)+Area2(I+1))/2*Dx(I)
4340   NEXT I
4350   RETURN
4360 ! ******************************************************************
4370 !                          CALCULATION OF COEFFICIENTS            Level 2
4380 ! ******************************************************************
4390 Coefficients:
4400 !
4410   A1=-Theta
4420   C1=Theta
4430   FOR I=1 TO Nsections
4440      K(I)=K1(I)*(1-Theta)+K2(I)*Theta
4450      Bsurf(I)=Bsurf1(I)*(1-Theta)+Bsurf2(I)*Theta
4460      Area(I)=(1-Theta)*Area1(I)+Theta*Area2(I)
4470      Para1(I)=((1-Theta)*Beta1(I)+Theta*Beta2(I))*Q1(I)/Area(I)
4480      Para3(I)=9.81*ABS(Q1(I))/(K1(I)*K2(I))
4490   NEXT I
4500 !
4510 !
4520   FOR I=1 TO Nsections-1
4530      Dx=Dx(I)
4540      Area=Psi*Area(I+1)+(1-Psi)*Area(I)
4550      Para2(I)=9.81*(Psi*Area(I+1)+(1-Psi)*Area(I))
4560 !
4570 !
4580      B1(I)=Bsurf(I)*Dx/Dt*(1-Psi)
4590      D1(I)=Bsurf(I+1)*Dx/Dt*Psi
4600      E1(I)=(1-Theta)*Q1(I)+B1(I)*Y1(I)-(1-Theta)*Q1(I+1)+D1(I)*Y1(I+1)
4610 !
```

```
4620 !
4630      IF Complete=0 THEN                   ! INERTIA TERMS OMITTED
4640         A2(I)=(1-Psi)*Dx*(Para3(I)*Area)
4650         B2(I)=-Para2(I)*Theta
4660         C2(I)=Psi*Dx*(Para3(I+1)*Area)
4670         D2(I)=Para2(I)*Theta
4680         E2(I)=Para2(I)*(1-Theta)*(Y1(I)-Y1(I+1))
4690      ELSE                                 ! COMPLETE DYNAMIC EQUATION
4700         A2(I)=(1-Psi)*Dx*(1/Dt+Para3(I)*Area)-Para1(I)
4710         B2(I)=-Para2(I)*Theta
4720         C2(I)=Psi*Dx*(1/Dt+Para3(I+1)*Area)+Para1(I+1)
4730         D2(I)=Para2(I)*Theta
4740         E2(I)=Dx/Dt*((1-Psi)*Q1(I)+Psi*Q1(I+1))+Para2(I)*(1-Theta)*(Y1(I)-Y
1(I+1))
4750      END IF
4760   NEXT I
4770   RETURN
4780 !
4790 ! ******************************************************************************
4800 !                         FORWARD AND BACKWARD SWEEPS                  Level 2
4810 ! ******************************************************************************
4820 Sweep:                |
4830   SELECT Lh_bc                    ! Left Hand Boundary Condition
4840   CASE 1                          ! Q given
4850      F(1)=0
4860      Q2(1)=Q_lhs(N+1)
4870      G(1)=Q2(1)
4880   CASE 2                          ! Stage given
4890      F(1)=1E6
4900      Y2(1)=Stage(N)
4910      G(1)=-F(1)*Y2(1)
4920   END SELECT
4930   FOR I=1 TO Ii-1
4940      Gamma=B1(I)+A1*F(I)
4950      Alfa=B2(I)+A2(I)*F(I)
4960      H(I)=-C1/Gamma
4970      I(I)=-D1(I)/Gamma
4980      J(I)=(E1(I)-A1*G(I))/Gamma
4990      Kappa=Alfa*H(I)+C2(I)
5000      F(I+1)=-(Alfa*I(I)+D2(I))/Kappa
5010      G(I+1)=(E2(I)-A2(I)*G(I)-Alfa*J(I))/Kappa
5020   NEXT I
5030   SELECT Rh_bc                    ! Right Hand Boundary Condition
5040   CASE 1                          ! Tidal equation
5050      T=N*Dt
5060      T1=T+Tide_state
5070      Y2(Ii)=Msl+Am2*SIN(2*PI*T1/Tm2)+As2*SIN(2*PI*(T1+Phi)/Ts2)
5080      Q2(Ii)=F(Ii)*Y2(Ii)+G(Ii)
5090   CASE 2
5100      Y2(Ii)=Stage_out(N)
5110      Q2(Ii)=F(Ii)*Y2(Ii)+G(Ii)
5120   CASE 3
5130      LOOP
5140         Q2(Ii)=F(Ii)*Y2(Ii)+G(Ii)
5150         LOOP
5160            Froude=Alpha2(Ii)*Q2(Ii)^2*Bsurf2(Ii)/(9.81*Area2(Ii)^3)
5170            IF Froude<.01 THEN Froude=.01
5180            IF Froude>100 THEN Froude=100
5190         EXIT IF ABS(1-Froude)<.01
5200            Dy=(1-Froude)^2*Area2(Ii)/Bsurf2(Ii)*SGN(1-Froude)
5210            IF ABS(Dy)>Wd/2 THEN Dy=Wd/2*[illegible])
5220            Y2(Ii)=Y2(Ii)-Dy
5230            GOSUB Section_prop1
5240         END LOOP
5250         Q=F(Ii)*Y2(Ii)+G(Ii)
5260      EXIT IF ABS(Q2(Ii)-Q)/Q<.01
5270      END LOOP
5280      PRINT " Q= ";Q2(Ii),"Y= ";Y2(Ii),"Fr= ";Froude,LIN(1)
5290   END SELECT
5300   FOR I=Ii TO 1 STEP -1
5310      IF I<Ii THEN
```

```
5320        Y2(I)=H(I)*Q2(I+1)+I(I)*Y2(I+1)+J(I)
5330        Q2(I)=F(I)*Y2(I)+G(I)
5340     END IF
5350     IF Iter>1 THEN
5360        IF Y2(I)>Ymax(I,2) THEN
5370           Ymax(I,1)=N*Dt1
5380           Ymax(I,2)=Y2(I)
5390        END IF
5400        IF Q2(I)>Qmax(I,2) THEN
5410           Qmax(I,1)=N*Dt1
5420           Qmax(I,2)=Q2(I)
5430        END IF
5440        IF Y2(I`<Ymin(I) THEN Ymin(I)=Y2(I)
5450        IF Q2(I)<Qmin(I) THEN Qmin(I)=Q2(I)
5460     END IF
5470   NEXT I
5480   IF Iter>1 THEN
5490      Q_rhs(N+1)=Q2(Xx)
5500      Y_lhs(N+1)=Y2(1)
5510      Y_rhs(N+1)=Y2(Xx)
5520   END IF
5530   RETURN
5540 ! ***********************************************************************
5550 !                          PRINT HEADING                          Level 2
5560 ! ***********************************************************************
5570 Printhead:                !
5580   PRINTER IS 0
5590   PRINT PAGE
5600   Lines=0
5610   PRINT "________________________________________________________________________
________"
5620   PRINT LIN(1),"                   FINITE DIFFERENCE FLOW ANALYSIS"
5630   IF Complete=0 THEN
5640      PRINT "              using the APPROXIMATE DYNAMIC EQUATIONS "
5650   ELSE
5660      PRINT "               using the COMPLETE DYNAMIC EQUATIONS "
5670   END IF
5680   PRINT "________________________________________________________________________
________"
5690   PRINT "  DATA FILES :"
5700   PRINT "  Cross sections          : ";File$
5710   PRINT "  LHS Boundary Conditions: ";
5720   SELECT Lh_bc
5730   CASE 1
5740      PRINT "Inflow hydrograph: ";Inflo$
5750   END SELECT
5760   PRINT "  RHS Boundary Conditions: ";
5770   SELECT Rh_bc
5780   CASE 1
5790      PRINT "Outflow hydrograph: ";Outflo$
5800   CASE 2
5810      IF (Am2=0) AND (As2=0) THEN
5820         PRINT "Constant water level: ";Msl
5830      ELSE
5840         PRINT "Tidal variation: "
5850         PRINT USING "27X,18A,DD.2D,2A";"Mean sea level  : ";Msl;" m"
5860         PRINT USING "27X,18A,DD.3D,2A";"M2 amplitude    : ";Am2;" m"
5870         PRINT USING "27X,18A,DD.DD,2A";"S2 amplitude    : ";As2;" m"
5880         PRINT USING "27X,18A,5D,5A";"Phase Difference: ";Phi;" s"
5890         PRINT USING "27X,17A,3D.2D,2A,/";"Time of Tide    : ";Tide_state/36
00," h"
5900      END IF
5910   CASE 3
5920      PRINT "Critical Flow Conditions"
5930   END SELECT
5940   PRINT "  Initial Conditions      : ";Init$
5950   PRINT LIN(1);"  THETA : ";Theta;"     PSI : ";Psi
5960   PRINT USING "15A,Z.DD,X,5A";"  TIME STEP : ";Dt/3600;"hours"
5970   PRINT "________________________________________________________________________
________"
```

```
5980 Print_page:                           ! HEADING AT TOP OF PAGE FOR SELECTED
5990                                       ! CROSS SECTIONS
6000   IF Op_out=3 THEN
6010      PRINT USING "/,6A,DD,A,/ ";"RUN ";Run;""
6020      PRINT USING "#,5X"
6030      FOR I=1 TO All
6040         PRINT USING "#,3X,9A,DD";"SECTION",Cross_print(I)
6050      NEXT I
6060      PRINT
6070      PRINT USING "#,8A";"TIME"
6080      FOR I=1 TO All
6090         PRINT USING "#,2X,7A,2X,6A,X";"STAGE","FLOW"
6100      NEXT I
6110      PRINT USING "/"
6120   END IF
6130   RETURN
6140 ! ******************************************************************
6150 !                            PRINT OUTPUT                        Level 2
6160 ! ******************************************************************
6170 Print:
6180   PRINTER IS 0
6190   SELECT Op_out
6200   CASE 1,2
6210      PRINT USING "/,10A,DD,A,25A,DD.3D,9A,/ ";"     RUN ";Run;"";"
   TIME ";N*Dt/3600,"  hours"
6220      PRINT "  SECTION       LEVEL       FLOW          SECTION      LE
VEL       FLOW "
6230      Ij=INT(Nsections/2)
6240      FOR I=1 TO Ij
6250         PRINT USING "2(2X,5D,5X,5D.3D,X,M6D.2D,4X)";I,Y2(I),Q2(I),I+Ij,Y2(I
+Ij),Q2(I+Ij)
6260      NEXT I
6270      IF FRACT(Nsections/2)<>0 THEN
6280         I=Nsections
6290         PRINT USING "38X,5D,5X,5D.3D,X,M. . .";I,Y2(I),Q2(I)
6300      END IF
6310   CASE 3
6320      PRINT USING "#,3D.2D";N*Dt1
6330      FOR Ij=1 TO All
6340         I=Cross_print(Ij)
6350         PRINT USING "#,X,DD.3D,4D.2D";Y2(I),Q2(I)
6360      NEXT Ij
6370      PRINT
6380      IF FRACT(  ines-40)/62)=0 THEN
6390         PRINT
6400         GOSUB P  nt_page
6410      END IF
6420      Lines=Lines+1
6430   END SELECT
6440   PRINTER IS 16
6450   RETURN
6460 ! ******************************************************************
6470 !                 PRINT MAXIMUM STAGE / FL W VALUES          Level 3
6480 !                     MINIMA FOR TIDAL FLOWS ONLY
6490 ! ******************************************************************
6500 Print_max:
6510   PRINTER IS 0
6520   PRINT USING "@,K,//";"_____________________________________________
_____________________"
6530   PRINT USING "K,//";"  MAXIMUM STAGE AND FLOW VALUES "
6540   PRINT USING "#,K";"                 MAXIMUM                    MAXIMUM"
6550   IF Rh_bc=2 THEN
6560      PRINT USING "K";"                           MINIMUM"
6570   ELSE
6580      PRINT
6590   END IF
6600   PRINT USING "#,K";"  SECTION     STAGE     TIME          FLOW       T
IME"
6610   IF Rh_bc=2 THEN
6620      PRINT USING "K";"           STAGE     FLOW"
```

```
6630   ELSE
6640      PRINT
6650   END IF
6660   PRINT USING "#,K";"                 (m)    (hours)      (cumec)  (hours)"
6670   IF Rh_bc=2 THEN
6680      PRINT USING ",K,/";"             (m)    (cumec)"
6690   ELSE
6700      PRINT USING "//"
6710   END IF
6720   FOR I=1 TO Il
6730      PRINT USING "#,3X,3D,2(6X,3D.3D,2X,3D.3D)";I,Ymax(I,2),Ymax(I,1),Qmax(
I,2),Qmax(I,1)
6740      IF Rh_bc=2 THEN
6750         PRINT USING "7X,3D.3D,2X,3D.3D";Ymin(I),Qmin(I)
6760      ELSE
6770         PRINT
6780      END IF
6790   NEXT I
6800   PRINTER IS 16
6810   RETURN
6820   ! ***************************************************************
6830   !                     PRINT MASS BALANCE RESULTS          Level 3
6840   ! ***************************************************************
6850 Print_balance:                            !
6860   PRINTER IS 0
6870   PRINT USING "//,K,//";"__________________________________________
______________"
6880   PRINT USING "K,//";" MASS BALANCE RESULTS: "
6890   PRINT USING "#,31A,7DZ.3D";"  Initial Volume     (1000m3) :",Vol_start/10
00
6900   PRINT USING "3X,21A,7DZ.3D";" Inflow     (1000m3) :",Qsum_in/1000
6910   PRINT USING "#,31A,7DZ.3D";"  Final Volume       (1000m3) :",Vol_end/1000
6920   PRINT USING "3X,21A,7DZ.3D,";" Outflow    (1000m3) :",Qsum_out/1000
6930   Vol_change=Vol_end-Vol_start
6940   PRINT USING "31X,12A,24X,12A,/";"____________","____________"
6950   PRINT USING "#,31A,7DZ.3D";"  Increase in Volume (1000m3) :",Vol_change/1
000
6960   PRINT USING "3X,21A,7DZ.3D,/";" Net Inflow (1000m3) :",(Qsum_in-Qsum_out)
/1000
6970   PRINT USING " 31A,7DZ.3DX,A";"  Error              :",(Vol_change-(Qsum_in
-Qsum_out))/Vol_change*100,"%"
6980   PRINT USING "//,K,//";"__________________________________________
______________"
6990   PRINTER IS 16
7000   RETURN
7010   ! ***************************************************************
7020   !                     HYDROGRAPH DATA INPUT               Level 3
7030   ! ***************************************************************
7040   !
7050 Hydro_input:                        ! INPUT OF A  H'GRAPH
7060   DIM A(100,2)
7070   PRINT PAGE
7080   PRINT " HYDROGRAPH INPUT : ",LIN(2)
7090   PRINT "POINT                 TIME                     INFLOW"
7100   PRINT " No.                   (h)                     (cumec)",LIN(1)
7110   I=1
7120   LOOP
7130      LINPUT "ENTER TIME(HRS) & INFLOW(M3/S)- (CONT WHEN FINISHED)",Edit$
7140   EXIT IF Edit$=""
7150      GOSUB Strings
7160      A(I,1)=VAL(Str$(1))
7170      A(I,2)=VAL(Str$(2))
7180      PRINT USING 7190;I,A(I,1),A(I,2)
7190      IMAGE   2X,DD,16X,DD.DD,16X,5D.DDD
7200      I=I+1
7210   END LOOP
7220   Set3=I-1
7230   GOSUB Edit
7240   IF Edit$[1,1]="Y" THEN GOSUB Hydro_edit
7250   RETURN
7260   !
```

```
7270  ! ***********************************************************************
7280  !
7290 Hydro_add:                        ! INPUT OF MULTIPLE INFLOW H'GRAPHS
7300   PRINT PAGE
7310   PRINT "NOTES TO ADDING OF H'GRAPHS:"
7320   PRINT "----------------------------"
7330   PRINT LIN(1);"1. ALL H'GRAPHS TO BE ADDED MUST HAVE DATA GIVEN WITH THE S
AME TIME INCREMENTS."
7340   PRINT "2. ALL LAG TIMES TO BE A WHOLE NO. OF TIME INCREMENTS."
7350   DIM Hydro(100,2),Lag(10),Time(10)
7360   MAT Hydro=ZER
7370   Set3_max=0
7380   INPUT "HOW MANY H'GRAPHS DO YOU WISH TO ADD ?",Nhydro
7390   FOR Z=1 TO Nhydro
7400      DISP "IS  H'GRAPH NO. ";Z;"ON A FILE?";
7410      INPUT J4$
7420      IF J4$="Y" THEN 7520
7430      IF J4$="N" THEN 7470
7440      BEEP
7450      INPUT "ENTER Y IF H'GRAPH ON FILE OR N IF NOT !! ",J4$
7460      GOTO 7420
7470      GOSUB Hydro_input
7480      GOSUB Hydro_store
7490      DISP "ENTER LAG TIME FOR H'GRAPH NO. ";Z;
7500      INPUT Lag(Z)
7510      GOTO 7560
7520      DISP "ENTER FILE NAME FOR H'GRAPH NO. ";Z;" AND LAG TIME";
7530      INPUT Inflo$,Lag(Z)
7540      ASSIGN #3 TO *
7550      GOSUB Sum_filed
7560      IF Lag(Z)=0 THEN Jj=0
7570      IF Lag(Z)>0 THEN Jj=INT(Lag(Z)/T)
7580      Time(Z)=(A(Set3,1)-A(1,1))/(Set3-1)
7590      IF Z<2 THEN 7610
7600      IF Time(Z)<>Time(Z-1) THEN 9640
7610      T=Time(Z)
7620      FOR J=1 TO Set3
7630         A(J,1)=A(J,1)+Lag(Z)
7640         IF Set3_max<Set3 THEN Hydro(J,1)=(J-1)*T
7650         Hydro(J+Jj,2)=Hydro(J+Jj,2)+A(J,2)
7660      NEXT J
7670      IF Set3_max<Set3+Jj THEN Set3_max=Set3+Jj
7680   NEXT Z
7690   Set3=Set3_max
7700   MAT A=Hydro
7710   PRINT PAGE
7720   GOSUB Hydro_print
7730   RETURN
7740  !
7750  ! ***********************************************************************
7760  !
7770 Hydro_edit:                    !    EDITS INFLOW H'GRAPH
7780   LOOP
7790      LINPUT "ENTER NO. OF POINT TO BE CHANGED - ( CONT WHEN FINISHED )",I
$
7800   EXIT IF I$=""
7810      I=VAL(I$)
7820      Edit$="   "&VAL$(A(I,1))&" ,    "&VAL$(A(I,2))
7830      DISP "TIME ,INFLOW for Point ";I;" displayed below ,Change as required
";
7840      EDIT Edit$
7850      GOSUB Strings
7860      A(I,1)=VAL(Str$(1))
7870      A(I,2)=VAL(Str$(2))
7880      GOSUB Hydro_print
7890   END LOOP
7900   RETURN
7910  !
7920  ! ***********************************************************************
*
7930  !
```

```
7940 Hydro_store:          ! SUBROUTINE TO STORE INFLOW H'GRAPH ON DISC
7950   INPUT "- ENTER NAME OF FILE IN WHICH YOU WISH TO STORE H'GRAPH DATA",Infl
o$
7960   REDIM A(Set3,2)
7970   Records=INT(8*2*Set3/256)+1
7980   CREATE Inflo$,Records
7990   ASSIGN #6 TO Inflo$
8000   PRINT #6;A(*)
8010   ASSIGN #6 TO *
8020   RETURN
8030   !
8040   ! ********************************************************************
8050   !
8060 Hydro_print:                          ! PRINTS INFLOW HYDROGRAPH
8070   PRINTER IS 16
8080   PRINT PAGE
8090   PRINT "_HYDROGRAPH :_",LIN(2)
8100   PRINT "POINT                  TIME                        FLOW"
8110   PRINT "_No._                  _(h)_                       _(cumec)_",LIN(1)
8120   FOR I=1 TO Set3
8130      PRINT USING 8150;I,A(I,1),A(I,2)
8140   NEXT I
8150   IMAGE      2X,DD,16X,DD.DD,16X,5D.DDD
8160   RETURN
8170   !
8180   ! *******************************************************************
8190   !
8200 Hydro_read:                 ! READS  H'GRAPH FROM DISC
8210   INPUT "NAME OF H'GRAPH FILE?",Inflo$
8220 Sum_filed:                  ! SUBROUTINE TO READ INFLOW H'GRAPH FROM DISC
8230   REDIM A(100,2)
8240   ON ERROR GOTO 8210
8250   ASSIGN #3 TO Inflo$
8260   OFF ERROR
8270   Set3=1
8280   LOOP
8290      READ #3;A(Set3,1),A(Set3,2)
8300      ON END #3 GOTO 8330
8310      Set3=Set3+1
8320   END LOOP
8330   ASSIGN #3 TO *
8340   Set3=Set3-1
8350   REDIM A(Set3,2)
8360   INPUT "DO YOU WISH TO MULTIPLY ALL INFLOW ORDINATES BY A CONSTANT ? (Y/N)
",Edit$
8370   LOOP
8380   EXIT IF (Edit$[1,1]="Y") OR (Edit$[1,1]="N")
8390      BEEP
8400      INPUT "ENTER Y IF YOU WISH TO FACTOR THE INFLOW H'GRAPH OR N IF NOT !"
,J4$
8410   END LOOP
8420   IF Edit$[1,1]="Y" THEN
8430      INPUT "ENTER THE CONSTANT",Const
8440      FOR I=1 TO Set3
8450         A(I,2)=A(I,2)*Const
8460      NEXT I
8470   END IF
8480   RETURN
8490   !
8500   ! ********************************************************************
8510 Discrete_hydro:             ! DISCRETIZES HYDROGRAPH
8520   MAT Q_discrete=ZER
8530   REDIM Q_discrete(5000)
8540   Tstep=INT(A(Set3,1)/Dt1)+1
8550   Nn=INT(Tmax/Dt1)+1
8560   FOR I=1 TO Set3-1
8570      Step1=INT((A(I+1,1)-A(I,1))/Dt1)
8580      Qstep=A(I,1)/Dt1+1
8590      FOR K=Qstep TO Step1+Qstep
8600         Dq=(A(I+1,2)-A(I,2))/(A(I+1,1)-A(I,1))
8610         Q_discrete(K)=(K-Qstep)*Dq*Dt1+A(I,2)
```

```
8620       NEXT K
8630    NEXT I
8640    Set3_old=Set3
8650    Set3=Tstep
8660    Q_discrete(Set3)=A(Set3_old,2)
8670    REDIM Q_discrete(Nn)
8680    RETURN
8690    ! ********************************************************************
8700    !
8710    ! ******************************************************************
8720    !
8730 Strings:                         ! BREAKS DOWN A COMMA-SEPARATED STRING
8740                                  ! INTO COMPONENT STRINGS
8750    Comma=POS(Edit$,",")
8760    IF Comma<>0 THEN Str$(1)=Edit$[1,Comma]
8770    IF Comma=0 THEN Str$(1)=Edit$
8780    Ij=2
8790    LOOP
8800    EXIT IF Comma=0
8810       Edit$=Edit$[Comma+1]
8820       Comma=POS(Edit$,",")
8830       IF Comma<>0 THEN
8840          Str$(Ij)=Edit$[1,Comma-1]
8850       ELSE
8860          Str$(Ij)=Edit$
8870       END IF
8880       Ij=Ij+1
8890    END LOOP
8900    All=Ij-1
8910    RETURN
8920 Edit:                            ! CHECKS IF ANY EDITTING TO DISPLAYED
8930                                  ! DATA IS REQUIRED
8940    Edit$=""
8950    LOOP
8960       INPUT "DO YOU WISH TO MAKE ANY CHANGES TO THE ABOVE DATA ? (Y/N)",Edit
$
8970    EXIT IF (Edit$[1,1]="Y") OR (Edit$[1,1]="N")
8980       BEEP
8990       DISP "ENTER YES OR NO !!!"
9000       WAIT 2000
9010    END LOOP
9020    RETURN
9030    ! ******************************************************************
9040    !                    STORING AND READING INITIAL CONDITIONS FROM DISC
9050    ! ******************************************************************
9060 Init_store:                      !
9070    INPUT "ENTER NAME OF FILE FOR STORING INITIAL DATA",Init$
9080    Records=Nsections*2*8/256+1
9090    CREATE Init$,Records
9100    REDIM Qstart(Nsections),Ystart(Nsections)
9110    ASSIGN #5 TO Init$
9120    PRINT #5;Qstart(*),Ystart(*)
9130    ASSIGN #5 TO *
9140    RETURN
9150    ! ******************************************************************
9160    !
9170    ! ******************************************************************
9180 Init_read:                       !
9190    REDIM Qstart(Nsections),Ystart(Nsections)
9200    INPUT "ENTER NAME OF FILE IN WHICH INITIAL DATA IS STORED",Init$
9210    ASSIGN #5 TO Init$
9220    READ #5;Qstart(*),Ystart(*)
9230    ASSIGN #5 TO *
9240    RETURN
9250    ! ******************************************************************
9260    !                          MASS STORAGE DEVICE               Level 3
9270    ! ******************************************************************
**
9280 Storage_device:                  !
9290    Edit$=Path$&Device$
9300    IF Edit$<>"" THEN
```

```
9310      EDIT "CURRENT MASS STORAGE IS  (CONT if OKAY - LEAVE BLANK TO RE-ENT
ER)",Edit$
9320      IF Edit$="" THEN 9370
9330      Colon=POS(Edit$,":")
9340      Device$=Edit$[Colon]
9350      IF Colon>1 THEN Path$=Edit$[1,Colon-1]
9360    ELSE
9370      LOOP
9380         INPUT "Local (L) or Remote (R)",Device$
9390      EXIT IF (Device$="L") OR (Device$="R")
9400         BEEP
9410         DISP " L OR R EXPECTED FOR ENTRY : ";
9420         WAIT 1000
9430      END LOOP
9440      IF Device$="R" THEN
9450         Device$=":REMOTE"
9460         INPUT "Enter the DIRECTORY PATH ",Path$
9470      ELSE
9480         LINPUT "Enter the Local device Address ( eg. :H7,0,1 )",Device$
9490         Path$=""
9500      END IF
9510    END IF
9520    MASS STORAGE IS Path$&Device$
9530    RETURN
9540    ! ***********************************************************************
9550    !                      PLOTS INFLOW AND OUTFLOW HYDROGRAPHS
9560    ! ***********************************************************************
9570 Plot_hydro:SUB Plot_hydro(Nn,Dt1,Q_lhs(*),Q_rhs(*),Ytitle$)
9580      POINTER IS 0
9590      PRINT PAGE
9600      FIXED 1
9610      PLOTTER IS "GRAPHICS"
9620      GRAPHICS
9630      LIMIT 20,160,0,140
9640      LOCATE 13,95,10,99
9650      Qmax=0
9660      MAT SEARCH Q_lhs,MAX;Qmax
9670      MAT SEARCH Q_lhs,MIN;Qmin
9680      MAT SEARCH Q_rhs,MAX;Qmax1
9690      MAT SEARCH Q_rhs,MIN;Qmin1
9700      IF Qmax<Qmax1 THEN Qmax=Qmax1
9710      IF Qmin>Qmin1 THEN Qmin=Qmin1
9720      Tmax=(Nn-1)*Dt1
9730      Xscale=INT(LGT(Tmax))
9740      IF Xscale<1 THEN
9750         IF Tmax<=4 THEN Xstep=.5
9760         IF Tmax>4 THEN Xstep=1
9770         Xscale=PROUND(Tmax+.5,0)
9780      ELSE
9790         Xstep=10^Xscale
9800         Xscale=10^Xscale*INT(Tmax/10^Xscale+1)
9810      END IF
9820      Yscale=INT(LGT(Qmax))
9830      Ymax=10^Yscale*INT(Qmax/10^Yscale+1)
9840      IF Qmin>=0 THEN
9850         Ymin=0
9860         Ystep=10^Yscale
9870      ELSE
9880         Yscale_min=INT(LGT(-Qmin))
9890         Ymin=-10^Yscale_min*INT(-Qmin/10^Yscale_min+1)
9900         Ystep=10^Yscale_min
9910      END IF
9920      SCALE 0,Xscale,Ymin,Ymax
9930      Yrange=Ymax-Ymin
9940      IF Yrange/Ystep<4 THEN Ystep=Ystep/2
9950      IF Xscale/Xstep<4 THEN Xstep=Xstep/2
9960      AXES Xstep,Ystep,0,Ymin,4,5,2
9970      MOVE 0,Q_lhs(1)
9980      FOR I=2 TO Nn
9990         LINE TYPE 5,1
10000        DRAW (I-1)*Dt1,Q_lhs(I)
```

```
10010      NEXT I
10020      MOVE 0,Q_rhs(1)
10030      LINE TYPE 1
10040      FOR I=2 TO Nn
10050         DRAW (I-1)*Dt1,Q_rhs(I)
10060      NEXT I
10070 ! Y-AXIS LABELS
10080      CSIZE 3.5,.6
10090      LORG 8
10100      FOR Y=Ymin TO Ymax STEP Ystep
10110         MOVE 0,Y
10120         LABEL USING "K,X";Y
10130      NEXT Y
10140 !
10150 ! X-AXIS LABELS
10160      LORG 6
10170      FOR X=0 TO Xscale STEP Xstep
10180         MOVE X,Ymin-Yrange/100
10190         LABEL X;
10200      NEXT X
10210 !
10220 ! Y-AXIS TITLE
10230      SETGU
10240      DEG
10250      LDIR 90
10260      LORG 5
10270      MOVE 2,50
10280      LABEL USING "#,K";Ytitle$
10290 !
10300 ! X-AXIS TITLE
10310      LDIR 0
10320      MOVE 60,5
10330      LABEL USING "#,K";"TIME  (HOURS)"
10340 !
10350 ! LEGEND
10360      MOVE 40,95
10370      LORG 5
10380 !      LABEL "Run ";Run
10390      MOVE 50,97
10400      LINE TYPE 5,1
10410      DRAW 60,97
10420      LINE TYPE 1
10430      LORG 2
10440      CSIZE 3.5,.6
10450      LABEL " UPSTREAM "&Ytitle$[1;5]
10460      MOVE 50,92
10470      DRAW 60,92
10480      LORG 2
10490      LABEL " DOWNSTREAM "&Ytitle$[1;5]
10500      DUMP GRAPHICS
10510      EXIT GRAPHICS
10520      PRINTER IS 16
10530      STANDARD
10540  SUBEND
10550 ! !!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
10560 Section_table:SUB Section_table(I,Nx,Wsi,B,Area,Ksum,Psum,Alpha,Beta)
10570      OPTION BASE 1
10580      COM Distance(*),Np(*),Level1(*),Mann(*)
10590      DIM A(20),K(20),P(20),R(20),Xs(20),Ys(20),Startx(10),Endx(10),Xx(20)
10600      IF FRACT(Wsi)<.0001 THEN Wsi=INT(Wsi)
10610      Area=Ksum=Psum=0
10620      As=Ks=Ps=Wd=Beta=B=0
10630      Alpha_sum=Beta_sum=0
10640      Start=End=0
10650      MAT A=(0)
10660      MAT K=(0)
10670      MAT Startx=ZER
10680      MAT Endx=ZER
10690      FOR J=1 TO Np(Nx)-1
10700         X1=Distance(I,J)
```

```
10710         X2=Distance(I,J+1)
10720         Y1=Level1(J)
10730         Y2=Level1(J+1)
10740         N=(Mann(J)+Mann(J+1))/2       ! MANNING N GIVEN AT A POINT
10750         IF (Wsi>Y1) OR (Wsi>Y2) THEN
10760            IF Wsi<Y1 THEN
10770               Dy=Wsi-Y2
10780               Dx=Dy*(X1-X2)/(Y1-Y2)
10790               Xs(J)=X2+Dx
10800               Ys(J)=Y2+Dy
10810            ELSE
10820               Xs(J)=X1
10830               Ys(J)=Y1
10840            END IF
10850            IF Wsi<Y2 THEN
10860               Dy=Wsi-Y1
10870               Dx=Dy*(X2-X1)/(Y2-Y1)
10880               Xs(J+1)=X1+Dx
10890               Ys(J+1)=Y1+Dy
10900            ELSE
10910               Xs(J+1)=X2
10920               Ys(J+1)=Y2
10930            END IF
10940            Lx=Xs(J+1)-Xs(J)
10950            B=B+Lx
10960            Ly=Ys(J+1)-Ys(J)
10970            P(J)=SQR(Lx*Lx+Ly*Ly)
10980            A(J)=Lx*(Wsi-(Ys(J)+Ys(J+1))/2)
10990            R(J)=ABS(A(J)/P(J))
11000            K(J)=A(J)*R(J)^(2/3)/N
11010            Area=Area+A(J)
11020            Ksum=Ksum+K(J)
11030            Beta_sum=Beta_sum+K(J)^2/A(J)
11040            Alpha_sum=Alpha_sum+K(J)^3/A(J)^2
11050            Psum=Psum+P(J)
11060         END IF
11070      NEXT J
11080      IF Area<>0 THEN
11090         Beta=Beta_sum/(Ksum^2/Area)
11100         Alpha=Alpha_sum/(Ksum^3/Area^2)
11110      ELSE
11120         Beta=Alpha=1
11130      END IF
11140   SUBEND
```

APPENDIX B2 : PROGRAM LISTING - BACKDATA

```
20 !      "HYDRO/BACKDATA:REMOTE"
40 !       UPDATE 1985-08-01  FJGG
60    OPTION BASE 1
80    COM Path$[50],Device$[50]
100   COM Reach(60,20,3),Np(60),L(60),Data
120   DIM Cp(60),Bp(60),Beach(60,20,3)
140   DIM Edit$[80]
160   Edit_value=0
180   Com$="   ,   "
200   PRINTER IS 16
220   PRINT PAGE;"  BACKWATER PROGRAM - CROSS SECTION DATA INPUT",LIN(2)
240   PRINT CHR$(27)&"]"
260   GOSUB Storage_device
280   LOOP
300      LINPUT "Enter the REACH Number -( CONT if FINISHED )",Nr$
320   EXIT IF Nr$=""
340      Nr=VAL(Nr$)
360      GOSUB Retrieve
380      GOSUB Options
400   END LOOP
420   STOP
440   !
460   ! ******************************************************************
480   !
500 Options:                        ! AVAILABLE OPTIONS IN THIS PROG
520   LOOP
540      PRINT PAGE,"  DATA FOR REACH ";Nr;"",LIN(1)
560      PRINT "      1. ENTER CROSS SECTION DATA ",LIN(1)
580      PRINT "      2. PRINT HARD COPY OF CROSS SECTION DATA",LIN(1)
600      PRINT "      3. EDIT CROSS SECTION DATA",LIN(1)
620      PRINT "      4. RUN BACKWATER PROGRAM WITH DATA FOR REACH ";Nr,LIN(1)
640      PRINT "      5. ENTER A NEW REACH "
660      INPUT "ENTER OPTION ( 1,2,3,4 or 5)",Op
680      SELECT Op
700      CASE 1
720         GOSUB Data
740         GOSUB Store
760      CASE 2
780         GOSUB Printout1
800      CASE 3
820         GOSUB Data_edit
840         GOSUB Store
860      CASE 4
861         Data=1
880         LOAD "HYDRO/BACKWATER:REMOTE",1
900      CASE 5
920   EXIT IF Op=5
940      CASE ELSE
960         BEEP
980         INPUT " ENTER AN INTEGER BETWEEN 1 AND 5 TO INDICATE OPTION !! ",Op
1000        GOTO 680
1020     END SELECT
1040  END LOOP
1060  RETURN
1080  !
1100  ! ******************************************************************
1120  !
1140 Data:                          ! INPUT OF CROSS-SECTION DATA IN A REACH
1160  PRINT PAGE
1180  PRINT "  POINT           DISTANCE          LEVEL          MANNING "
1200  PRINT "   No.              (m)             (m)              N     "
1220  PRINT CHR$(27)&"l"
1240  Np_max=0
1260  I=1
1280  LOOP
1300 Data1:                         ! START OF BRANCH FOR RE-ENTERING A CROSS SECT
```

```
1320      PRINT LIN(1)," Cross Section No. ";I;"",LIN(1)
1340      IF I>1 THEN
1360         DISP "CHANNEL LENGTH BETWEEN SECTIONS";I-1;"AND";I;" (CONT IF ";I-
1;"IS LAST SECTION )";
1380         LINPUT " ",Edit$
1400   EXIT IF Edit$=""
1420         L(I)=VAL(Edit$)
1440      END IF
1460      J=1
1480      LOOP
1500         LINPUT "ENTER DISTANCE,LEVEL ,MANNING N - ( CONT WHEN FINISHED WIT
H THIS SECTION )",Edit$
1520      EXIT IF Edit$=""
1540         GOSUB Strings
1560         Reach(I,J,1)=VAL(Str$(1))
1580         Reach(I,J,2)=VAL(Str$(2))
1600         Reach(I,J,3)=VAL(Str$(3))
1620         IMAGE 3X,DD,11X,4D.D, 8X,3D.DD, 9XD.DDD
1640         PRINT USING 1620;J,Reach(I,J,1),Reach(I,J,2),Reach(I,J,3)
1660         J=J+1
1680      END LOOP
1700      Np(I)=J-1
1720      IF Np(I)>Np_max THEN Np_max=Np(I)
1740      IF I>1 THEN
1760         PRINT "DISTANCE BETWEEN SECTIONS ";I-1;" AND";I;" IS";L(I)
1780      END IF
1800      PRINT LIN(2)
1820      GOSUB Edit
1840      IF Edit$[1,1]="Y" THEN
1860         GOSUB Data_edit1
1880      END IF
1900      IF Edit_value=1 THEN 2000
1920      Cum_np=Cum_np+Np(I)
1940      I=I+1
1960   END LOOP
1980   Nx=I-1
2000   RETURN
2020   !
2040   ! *****************************************************************
2060   !
2080 Data_edit1:                      ! EDITTING OF DATA
2100   IF I>1 THEN
2120      Edit$=VAL$(L(I))
2140      DISP " CHANGE THE DISTANCE BETWEEN SECTION";I;" AND SECTION";I-1;
2160      EDIT Edit$
2180      L(I)=VAL(Edit$)
2200   END IF
2220   LOOP
2240      DISP "WHICH POINT DO YOU WISH TO CHANGE IN CROSS-SECTION";I;" (CONT W
HEN FINISHED )";
2260      LINPUT Edit$
2280   EXIT IF Edit$=""
2300      Cp=VAL(Edit$)
2320      Edit$="  "&VAL$(Reach(I,Cp,1))&Com$&VAL$(Reach(I,Cp,2))&Com$&VAL$(Reach
(I,Cp,3))
2340      DISP " CHANGE DISTANCE,LEVEL OR MANNING N AT POINT ";Cp;
2360      EDIT "  ",Edit$
2380      GOSUB Strings
2400      Reach(I,Cp,1)=VAL(Str$(1))
2420      Reach(I,Cp,2)=VAL(Str$(2))
2440      Reach(I,Cp,3)=VAL(Str$(3))
2460      GOSUB Printout
2480   END LOOP
2500   RETURN
2520   !
2540   ! ******************************************************************
*
2560   !
2580 Store:                           ! STORES DATA ON DISC
2600   REDIM Reach(Nx,20,3),Np(Nx),L(Nx)
2620   Records=INT((8*60*Nx+16*Nx)/256)+1
```

```
2640  ON ERROR GOTO 2780
2660  Name$="Reac"&VAL$(Nr)
2680  CREATE Name$,Records
2700  ASSIGN #1 TO Name$
2720  PRINT #1;Cum_np,Nx,Np(*),L(*),Reach(*)
2740  ASSIGN * TO #1
2760  GOTO 2920
2780  BEEP
2800  PRINT ERRM$
2820  IF ERRN=54 THEN
2840     DISP "ENTER A NEW REACH NO. OR PURGE FILE ";Name$;
2860     INPUT Nr
2880     GOTO 2660
2900  END IF
2920  OFF ERROR
2940  RETURN
2960  !
2980  ! *****************************************************************
3000  !
3020 Printout:PRINTER IS 16    !PRINTS OUT CROSS SECTION DATA
3040  PRINT TAB(5);"REACH NUMBER";Nr
3060  PRINT TAB(5);"================"
3080  PRINT TAB(5);"CROSS";TAB(20);"POINT";TAB(35);"DISTANCE";TAB(50);"LEVEL";TA
B(65);"MANNING"
3100  PRINT TAB(4);"SECTION";TAB(20);"NUMBER";TAB(69);"N"
3120  PRINT TAB(4);"=======";TAB(20);"======";TAB(35);"========";TAB(50);"=====";
TAB(65);"======="
3140  PRINT LIN(2)
3160  PRINT USING 3180;I,1,Reach(I,1,1),Reach(I,1,2),Reach(I,1,3)
3180  IMAGE      5X,3D,13X,DD,10X,5D.D, 8X,3D.DD, 9XD.DDD
3200  FOR J=2 TO Np(I)
3220     PRINT USING 3240;J,Reach(I,J,1),Reach(I,J,2),Reach(I,J,3)
3240     IMAGE    21X,DD,10X,5D.D, 8X,3D.DD, 9XD.DDD
3260  NEXT J
3260  IF I=1 THEN 3320
3300  PRINT "DISTANCE BETWEEN SECTIONS ";I-1;" AND";I;" IS";L(I)
3323  PRINT LIN(2)
3340  RETURN
3360  STOP
3380  !
3400  ! *****************************************************************
3420  !
3440 Retrieve:                         ! READS DATA FROM DISC
3460  Name_reach$="Reac"&VAL$(Nr)
3480  ON ERROR GOTO 3600
3500  ASSIGN #3 TO Name_reach$
3520  READ #3;Cum_np,Nx
3540  REDIM Reach(Nx,20,3),Np(Nx),L(Nx)
3560  READ #3;Np(*),L(*),Reach(*)
3580  ASSIGN #3 TO *
3600  OFF ERROR
3620  RETURN
3640  !
3660  ! *****************************************************************
3680  !
3700 Data_edit:                        ! DATA EDIT OPTIONS
3720  Edit_value=1
3740  LOOP
3760     PRINT PAGE,"  EDITTING OF DATA FOR REACH ";Nr;"",LIN(2)
3780     PRINT "  1. INSERT A NEW CROSS SECTION ",LIN(1)
3800     PRINT "  2. CHANGE EXISTING DATA",LIN(2)
3820     PRINT "  3. DELETE A CROSS SECTION ",LIN(2)
3840     LINPUT "ENTER OPTION ( 1or 2) - (CONT WHEN FINISHED )",Edit$
3860  EXIT IF Edit$=""
3880     Op_edit=VAL(Edit$)
3900     SELECT Op_edit
3920     CASE 1
3940        INPUT "ENTER No. OF CROSS-SECTION BELOW NEW SECTION ",Jk
3960        Jj=Jk+1
3980        GOSUB Insert
4000        I=Jj
```

```
4020          GOSUB Data1
4040       CASE 2
4060          LOOP
4080             LINPUT "WHICH CROSS-SECTION DO YOU WISH TO CHANGE ? ( CONT WHEN
 FINISHED )",Edit$
4100          EXIT IF Edit$=""
4120             I=VAL(Edit$)
4140             INPUT "DO YOU WANT TO CHANGE THE WHOLE SECTION ? (Y/N)",Edit$
4160             LOOP
4180             EXIT IF (Edit$[1,1]="Y") OR (Edit$[1,1]="N")
4200                BEEP
4220                INPUT "ENTER  Y IF YOU WANT TO CHANGE THE WHOLE SECTION OR
N IF NOT ",Edit$
4240             END LOOP
4260             IF Edit$="Y" THEN
4280                GOSUB Data1
4300             ELSE
4320                GOSUB Data_edit1
4340             END IF
4360          END LOOP
4380       CASE 3
4400          LOOP
4420             LINPUT "WHICH CROSS-SECTION DO YOU WISH TO DELETE ? ( CONT WHEN
 FINISHED )",Edit$
4440          EXIT IF Edit$=""
4460             Ii=VAL(Edit$)
4480             L(Ii+1)=L(Ii)+L(Ii+1)
4500             FOR I=Ii TO Nx-1
4520                FOR J=1 TO Np(I+1)
4540                   Reach(I,J,1)=Reach(I+1,J,1)
4560                   Reach(I,J,2)=Reach(I+1,J,2)
4580                   Reach(I,J,3)=Reach(I+1,J,3)
4600                NEXT J
4620                Np(I)=Np(I+1)
4640                L(I)=L(I+1)
4660             NEXT I
4680             Nx=Nx-1
4700             REDIM Reach(Nx,20,3),Np(Nx),L(Nx)
4720          END LOOP
4740       CASE ELSE
4760          BEEP
4780       END SELECT
4800    END LOOP
4820    RETURN
4840    !
4860    ! ******************************************************************
4880    !
4900 Printout1:PRINTER IS 0     !PRINTS OUT CROSS SECTION DATA
4920    PRINT TAB(10);"REACH NUMBER";Nr
4940    PRINT TAB(10);"================"
4960    PRINT TAB(10);"CROSS";TAB(20);"  REACH";TAB(32);"POINT";TAB(42);"SECTION";
TAB(55);"GROUND";TAB(65);"MANNING"
4980    PRINT TAB(9);"SECTION";TAB(20);"DISTANCE";TAB(32);"NUMBER";TAB(42);"DISTAN
CE";TAB(55);"LEVEL";TAB(69);"N"
5000    PRINT TAB(9);"=======";TAB(20);"========";TAB(32);"======";TAB(42);"======
==";TAB(55);"======";TAB(65);"======="
5020    PRINT LIN(1)
5021    Dist=0
5040    FOR I=1 TO Nx
5041    Dist=Dist+L(I)
5060       PRINT USING 5080;I,Dist,1,Reach(I,1,1),Reach(I,1,2),Reach(I,1,3)
5080       IMAGE   10X,3D,7X,4D.2D,7X,DD,6X,4D.D, 6X,3D.DD, 4XD.DDD
5100       FOR J=2 TO Np(I)
5120          PRINT USING 5140;J,Reach(I,J,1),Reach(I,J,2),Reach(I,J,3)
5140          IMAGE 34X,DD, 6X,4D.D, 6X,3D.DD, 4XD.DDD
5160       NEXT J
5220       PRINT LIN(1)
5240    NEXT I
5260    PRINTER IS 16
5280    RETURN
5300    !
```

```
5320  ! **********************************************************************
5340  !
5360 Insert:MAT Cp=Np
5380  MAT Bp=L
5400  MAT Beach=Reach
5420  REDIM Reach(N  1,20,3),Np(Nx+1),L(Nx+1),Cp(Nx+1),Bp(Nx+1),Beach(Nx+1,20,3)
5440  FOR I=Jj+1 TO Nx+1
5460     FOR J=1 TO Np(I-1)
5480        Beach(I,J,1)=Reach(I-1,J,1)
5500        Beach(I,J,2)=Reach(I-1,J,2)
5520        Beach(I,J,3)=Reach(I-1,J,3)
5540     NEXT J
5560     Cp(I)=Np(I-1)
5580     Bp(I)=L(I-1)
5600  NEXT I
5620  MAT Reach=Beach
5640  MAT Np=Cp
5660  MAT L=Bp
5680  Nx=Nx+1
5700  RETURN
5720  STOP
5740  ! **********************************************************************
5760  !
5780 Strings:                          ! BREAKS DOWN A COMMA-SEPARATED STRING
5800                                   ! INTO COMPONENT STRINGS
5820  Comma=POS(Edit$,",")
5840  IF Comma<>0 THEN Str$(1)=Edit$[1,Comma]
5860  IF Comma=0 THEN Str$(1)=Edit$
5880  Ii=2
5900  LOOP
5920  EXIT IF Comma=0
5940     Edit$=Edit$[Comma+1]
5960     Comma=POS(Edit$,",")
5980     IF Comma<>0 THEN
6000        Str$(Ii)=Edit$[1,Comma-1]
6020     ELSE
6040        Str$(Ii)=Edit$
6060     END IF
6080     Ii=Ii+1
6100  END LOOP
6120  RETURN
6140 Edit:                             ! CHECKS IF ANY EDITTING TO DISPLAYED
6160                                   ! DATA IS REQUIRED
6180  Edit$=""
6200  LOOP
6220     INPUT "DO YOU WISH TO MAKE ANY CHANGES TO THE ABOVE DATA ? (Y/N)",Edit$
6240  EXIT IF (Edit$[1,1]="Y") OR (Edit$[1,1]="N")
6260     BEEP
6280     DISP "ENTER YES OR NO !!!"
6300     WAIT 2000
6320  END LOOP
6340  RETURN
6360  ! **********************************************************************
6380  !                          MASS STORAGE DEVICE                  Level 3
6400  ! **********************************************************************
**
6420 Storage_device:
6440  Edit$=Path$&Device$
6460  IF Edit$<>"" THEN
6480     EDIT "CURRENT MASS STORAGE IS  (CONT if OKAY)",Edit$
6500     IF Edit$="" THEN 6600
6520     Colon=POS(Edit$,":")
6540     Device$=Edit$[Colon]
6560     IF Colon>1 THEN Path$=Edit$[1,Colon-1]
6580  ELSE
6600     LOOP
6620        INPUT "Local (L) or Remote (R)",L vice$
6640     EXIT IF (Device$="L") OR (Device$="R")
6660        BEEP
6680        DISP " L OR R EXPECTED FOR ENTRY : ";
```

```
6700        WAIT 1000
6720     END LOOP
6740     IF Device$="R" THEN
6760        Device$=":REMOTE"
6780        INPUT "Enter the DIRECTORY PATH ",Path$
6800     ELSE
6820        LINPUT "Enter the Local device Address ( eg. :H7,0,1 )",Device$
6840        Path$=""
6860     END IF
6880  END IF
6900  MASS STORAGE IS Path$&Device$
6920  RETURN
6940  ! ***********************************************************************
6960  END
```

APPENDIX C

SWARTVLEI ESTUARY

APPENDIX C1 : SWARTVLEI ESTUARY - CROSS SECTION DATA

CROSS SECTION	REACH DISTANCE	POINT NUMBER	SECTION DISTANCE	GROUND LEVEL	MANNING N
1	0.00	1	0.0	5.00	.020
		2	10.0	0.00	.020
		3	1000.0	0.00	.020
		4	1010.0	5.00	.020
2	100.00	1	0.0	5.00	.040
		2	70.0	3.45	.040
		3	95.0	1.80	.040
		4	100.0	1.80	.040
		5	120.0	2.85	.040
		6	131.0	3.40	.040
		7	203.0	5.00	.040
3	160.00	1	0.0	5.00	.025
		2	75.0	3.50	.025
		3	82.0	2.20	.020
		4	92.0	2.20	.025
		5	137.0	5.45	.025
4	220.00	1	0.0	5.30	.040
		2	11.0	3.85	.040
		3	13.0	1.27	.040
		4	22.0	1.60	.040
		5	43.0	4.00	.040
		6	63.0	4.85	.040
		7	77.0	5.00	.040
5	320.00	1	0.0	5.00	.100
		2	12.0	2.75	.040
		3	14.0	1.67	.040
		4	15.0	1.67	.040
		5	18.0	2.38	.040
		6	23.0	2.60	.040
		7	65.0	2.70	.040
		8	81.0	3.65	.100
		9	111.0	4.25	.100
		10	132.0	4.05	.100
		11	136.0	3.16	.100
		12	137.0	3.15	.100
		13	150.0	4.10	.100
		14	166.0	5.00	.100
6	410.00	1	0.0	5.00	.040
		2	5.0	1.70	.040
		3	20.5	2.05	.040
		4	28.5	2.90	.040
		5	67.0	3.25	.040
		6	125.0	4.40	.100
		7	126.0	5.00	.100
7	490.00	1	0.0	5.00	.040
		2	4.0	3.15	.040
		3	10.0	1.90	.040
		4	14.0	1.80	.040
		5	22.0	1.90	.040
		6	41.0	3.30	.040
		7	78.0	3.40	.100

		8	115.0	3.90	.100
		9	127.0	4.60	.100
		10	163.0	5.00	.100
8	610.00	1	0.0	5.00	.100
		2	8.0	3.00	.100
		3	33.0	3.15	.100
		4	50.0	3.00	.040
		5	60.0	2.67	.040
		6	68.0	2.05	.040
		7	74.0	2.15	.040
		8	83.0	3.65	.040
		9	102.0	4.20	.040
		10	141.0	5.00	.100
9	700.00	1	0.0	5.00	.040
		2	8.0	3.50	.040
		3	14.0	2.15	.040
		4	16.0	2.12	.040
		5	18.0	2.75	.040
		6	30.0	3.15	.040
		7	62.0	2.02	.040
		8	74.0	2.80	.100
		9	76.0	2.75	.100
		10	94.0	3.55	.100
		11	105.0	3.02	.100
		12	107.0	3.00	.100
		13	113.0	3.20	.100
		14	114.0	3.20	.100
		15	126.0	5.00	.100
10	900.00	1	0.0	5.00	.100
		2	15.0	3.75	.040
		3	28.0	1.00	.040
		4	34.0	.20	.040
		5	48.0	1.30	.040
		6	54.0	1.80	.040
		7	61.0	3.50	.040
		8	215.0	4.10	.100
		9	216.0	5.00	.100
11	980.00	1	0.0	5.00	.100
		2	20.0	2.75	.040
		3	27.0	1.50	.040
		4	36.0	1.50	.040
		5	43.0	2.30	.040
		6	54.0	2.35	.040
		7	65.0	3.10	.040
		8	72.0	3.10	.040
		9	78.0	1.30	.040
		10	87.0	1.30	.040
		11	112.0	2.50	.040
		12	128.0	2.35	.040
		13	137.0	3.65	.040
		14	260.0	4.00	.100
		15	293.0	4.42	.100
		16	317.0	5.00	.100
12	1110.00	1	0.0	6.56	.100
		2	.0	3.85	.040
		3	10.0	3.00	.040
		4	27.0	2.75	.040
		5	70.0	3.05	.040
		6	93.0	2.77	.040
		7	104.0	1.45	.040

		8	106.0	1.45	.040
		9	118.0	2.65	.040
		10	128.0	2.90	.040
		11	152.0	2.15	.040
		12	165.0	2.10	.040
		13	177.0	1.50	.040
		14	183.0	1.50	.040
		15	196.0	2.15	.040
		16	203.0	3.80	.040
		17	209.0	4.15	.100
		18	212.0	4.15	.100
		19	218.0	4.05	.100
		20	305.0	5.00	.100
13	1340.00	1	0.0	5.00	.100
		2	25.0	3.80	.100
		3	40.0	2.20	.100
		4	49.0	3.40	.100
		5	83.0	3.65	.100
		6	170.0	3.72	.100
		7	195.0	3.15	.100
		8	223.0	3.40	.040
		9	237.0	2.40	.040
		10	247.0	2.60	.040
		11	265.0	1.42	.040
		12	295.0	1.35	.040
		13	316.0	2.40	.040
		14	339.0	5.00	.040
14	1520.00	1	0.0	5.00	.100
		2	.5	4.32	.100
		3	24.5	3.62	.040
		4	36.5	2.65	.040
		5	37.5	2.65	.040
		6	52.5	3.50	.040
		7	85.5	3.40	.100
		8	111.5	3.50	.040
		9	174.5	2.00	.040
		10	193.5	.45	.040
		11	200.5	.45	.040
		12	209.5	.85	.040
		13	220.5	3.82	.040
		14	237.5	5.00	.100
15	1920.00	1	0.0	5.25	.100
		2	15.0	3.90	.040
		3	78.0	1.60	.040
		4	101.0	1.60	.040
		5	121.0	2.65	.040
		6	220.0	3.55	.040
		7	432.0	3.85	.100
		8	452.0	3.80	.100
		9	482.0	4.05	.100
		10	482.5	5.00	.100
16	2080.00	1	0.0	5.00	.040
		2	43.0	3.30	.040
		3	57.0	1.40	.040
		4	94.0	2.05	.040
		5	151.0	3.60	.040
		6	188.0	3.90	.040
		7	291.0	3.90	.040
		8	390.0	5.00	.100
17	2250.00	1	0.0	5.00	.040

		2	30.0	3.25	.040
		3	40.0	2.20	.040
		4	58.0	1.50	.040
		5	86.0	1.85	.040
		6	133.0	3.95	.040
		7	157.0	5.00	.040
18	2500.00	1	0.0	5.00	.040
		2	15.0	4.25	.040
		3	41.0	1.45	.040
		4	55.0	1.45	.040
		5	98.0	2.60	.040
		6	111.0	4.00	.040
		7	135.0	4.35	.040
		8	138.0	5.00	.040
19	2680.00	1	0.0	5.00	.040
		2	30.0	3.20	.040
		3	72.0	2.00	.040
		4	78.0	2.60	.040
		5	86.0	2.00	.040
		6	94.0	2.00	.040
		7	116.0	3.50	.040
		8	136.0	3.15	.040
		9	175.0	5.00	.040
20	2830.00	1	0.0	5.00	.040
		2	32.0	1.90	.040
		3	48.0	2.80	.040
		4	67.0	3.00	.040
		5	82.0	1.62	.040
		6	113.0	3.00	.040
		7	193.0	3.25	.100
		8	214.0	3.70	.100
		9	218.0	4.20	.100
		10	229.0	5.00	.100
21	3090.00	1	0.0	5.00	.100
		2	32.0	2.70	.100
		3	41.0	3.00	.100
		4	50.0	2.50	.100
		5	69.0	3.35	.100
		6	82.0	3.50	.100
		7	126.0	3.50	.040
		8	138.0	2.20	.040
		9	155.0	3.32	.040
		10	172.0	3.50	.040
		11	183.0	3.32	.040
		12	206.0	2.15	.040
		13	209.0	2.15	.040
		14	213.0	3.50	.040
		15	242.0	3.65	.040
		16	266.0	5.40	.040
22	3260.00	1	0.0	5.00	.100
		2	38.0	3.65	.100
		3	84.0	3.70	.100
		4	129.0	3.75	.100
		5	165.0	3.50	.040
		6	187.0	3.00	.040
		7	200.0	2.00	.040
		8	220.0	2.00	.040
		9	225.0	3.10	.040
		10	287.0	3.90	.100
		11	304.0	5.00	.100

23	3420.00	1	0.0	5.00	.100
		2	22.0	3.85	.100
		3	156.0	3.60	.040
		4	168.0	2.70	.040
		5	194.0	1.70	.040
		6	204.0	2.80	.040
		7	208.0	3.00	.040
		8	240.0	3.50	.040
		9	276.0	5.00	.040
24	3630.00	1	0.0	5.00	.040
		2	21.0	4.00	.040
		3	46.0	3.25	.040
		4	69.0	2.60	.040
		5	93.0	2.55	.040
		6	121.0	2.55	.040
		7	127.0	2.90	.040
		8	164.0	5.00	.040
25	3820.00	1	0.0	5.00	.040
		2	17.0	3.60	.040
		3	26.0	2.00	.040
		4	35.0	1.30	.040
		5	48.0	.70	.040
		6	65.0	.70	.040
		7	77.0	1.30	.040
		8	95.0	3.50	.040
		9	100.0	4.00	.040
		10	109.0	5.00	.040
26	3940.00	1	0.0	5.00	.040
		2	48.0	2.15	.040
		3	78.0	1.25	.040
		4	89.0	1.25	.040
		5	102.0	1.85	.040
		6	126.0	3.95	.040
		7	145.0	5.00	.040
27	4090.00	1	0.0	5.00	.040
		2	53.0	3.50	.040
		3	75.0	2.55	.040
		4	107.0	2.07	.040
		5	115.0	2.07	.040
		6	174.0	3.50	.040
		7	203.0	5.00	.040
28	4210.00	1	0.0	5.00	.040
		2	29.0	3.50	.040
		3	132.0	2.65	.040
		4	180.0	2.65	.040
		5	188.0	2.75	.040
		6	197.0	3.50	.040
		7	206.0	5.00	.040
29	4350.00	1	0.0	5.00	.040
		2	47.0	2.65	.040
		3	75.0	2.90	.040
		4	151.0	2.85	.040
		5	166.0	2.50	.040
		6	169.0	2.50	.040
		7	201.0	3.30	.040
		8	226.0	3.60	.040
		9	293.0	5.00	.040

23	3420.00	1	0.0	5.00	.100
		2	22.0	3.85	.100
		3	156.0	3.60	.040
		4	168.0	2.70	.040
		5	194.0	1.70	.040
		6	204.0	2.80	.040
		7	208.0	3.80	.040
		8	240.0	3.50	.040
		9	276.0	5.00	.040
24	3630.00	1	0.0	5.00	.040
		2	21.0	4.00	.040
		3	46.0	3.25	.040
		4	69.0	2.80	.040
		5	93.0	2.55	.040
		6	121.0	2.55	.040
		7	127.0	2.90	.040
		8	164.0	5.00	.040
25	3820.00	1	0.0	5.00	.040
		2	17.0	3.60	.040
		3	26.0	2.00	.040
		4	35.0	1.30	.040
		5	48.0	.70	.040
		6	65.0	.70	.040
		7	77.0	1.30	.040
		8	95.0	3.50	.040
		9	100.0	4.00	.040
		10	109.0	5.00	.040
26	3940.00	1	0.0	5.00	.040
		2	48.0	2.15	.040
		3	78.0	1.25	.040
		4	89.0	1.25	.040
		5	102.0	1.85	.040
		6	126.0	3.95	.040
		7	145.0	5.00	.040
27	4090.00	1	0.0	5.00	.040
		2	53.0	3.50	.040
		3	75.0	2.55	.040
		4	107.0	2.07	.040
		5	115.0	2.07	.040
		6	174.0	3.50	.040
		7	203.0	5.00	.040
28	4210.00	1	0.0	5.00	.040
		2	29.0	3.50	.040
		3	132.0	2.65	.040
		4	180.0	2.65	.040
		5	188.0	2.75	.040
		6	197.0	3.50	.040
		7	206.0	5.00	.040
29	4350.00	1	0.0	5.00	.040
		2	47.0	2.65	.040
		3	75.0	2.90	.040
		4	151.0	2.85	.040
		5	166.0	2.50	.040
		6	169.0	2.50	.040
		7	201.0	3.30	.040
		8	226.0	3.60	.040
		9	293.0	5.00	.040

30	4630.00	1	0.0	5.25	.040
		2	27.0	2.75	.040
		3	36.0	2.40	.040
		4	46.0	2.40	.040
		5	105.0	3.25	.040
		6	190.0	3.50	.040
		7	391.0	3.70	.100
		8	404.0	3.80	.100
		9	430.0	3.85	.100
		10	480.0	5.00	.100
31	4830.00	1	0.0	5.40	.100
		2	30.0	2.90	.100
		3	86.0	3.40	.100
		4	106.0	3.30	.040
		5	120.0	2.95	.040
		6	130.0	3.10	.040
		7	161.0	2.20	.040
		8	178.0	3.25	.040
		9	203.0	3.50	.100
		10	331.0	3.80	.100
		11	342.0	5.00	.100
32	5030.00	1	0.0	5.20	.040
		2	30.0	3.90	.040
		3	146.0	2.70	.040
		4	248.0	3.15	.040
		5	281.0	3.65	.040
		6	301.0	5.20	.040
33	5190.00	1	0.0	5.20	.040
		2	8.0	4.15	.040
		3	53.0	3.15	.040
		4	133.0	2.80	.040
		5	182.0	3.50	.040
		6	309.0	3.30	.100
		7	332.0	2.80	.100
		8	346.0	2.80	.100
		9	480.0	5.00	.100
34	5350.00	1	0.0	5.15	.040
		2	25.0	3.00	.040
		3	37.0	2.50	.040
		4	118.0	3.30	.040
		5	178.0	3.40	.100
		6	196.0	3.50	.100
		7	421.0	3.50	.100
		8	405.0	3.30	.100
		9	507.0	3.80	.100
		10	516.0	5.00	.100
35	5550.00	1	0.0	5.50	.040
		2	35.0	3.35	.040
		3	42.0	2.55	.040
		4	103.0	3.00	.040
		5	132.0	3.50	.040
		6	315.0	3.55	.100
		7	329.0	3.80	.100
		8	334.0	4.95	.100
		9	458.0	4.95	.100
		10	470.0	3.75	.100
		11	656.0	3.70	.100
		12	683.0	4.00	.100
		13	712.0	4.72	.100
		14	732.0	5.20	.100

36	5750.00	1	0.0	5.00	.100
		2	26.0	4.10	.100
		3	89.0	3.75	.100
		4	170.0	4.10	.100
		5	200.0	3.70	.100
		6	473.0	3.75	.100
		7	515.0	5.00	.100
		8	535.0	3.70	.060
		9	656.0	3.50	.040
		10	760.0	2.85	.040
		11	792.0	3.20	.040
		12	840.0	5.00	.040
37	6100.00	1	0.0	5.00	.150
		2	40.0	3.55	.150
		3	170.0	3.75	.150
		4	222.0	2.65	.150
		5	428.0	4.10	.050
		6	465.0	3.50	.050
		7	497.0	2.55	.050
		8	508.0	3.50	.050
		9	835.0	4.00	.150
		10	960.0	4.10	.150
		11	972.0	5.00	.150
38	6250.00	1	0.0	6.70	.100
		2	.1	4.30	.100
		3	18.0	4.20	.100
		4	20.0	3.80	.050
		5	36.0	1.70	.050
		6	50.0	1.00	.050
		7	64.0	1.80	.050
		8	102.0	2.90	.100
		9	119.0	3.70	.100
		10	119.1	6.70	.100
39	6260.00	1	0.0	6.70	.100
		2	.1	4.30	.100
		3	18.0	4.20	.100
		4	20.0	3.80	.060
		5	36.0	1.70	.060
		6	50.0	1.00	.060
		7	64.0	1.80	.060
		8	102.0	2.90	.060
		9	119.0	3.70	.100
		10	119.1	6.70	.100
40	6290.00	1	1.0	5.00	.200
		2	10.0	4.50	.200
		3	20.0	4.00	.200
		4	40.0	3.50	.200
		5	115.0	3.00	.060
		6	125.0	2.80	.040
		7	139.0	2.50	.040
		8	150.0	2.00	.040
		9	160.0	2.00	.040
		10	180.0	2.50	.040
		11	185.0	2.80	.040
		12	195.0	3.00	.060
		13	302.4	3.50	.200
		14	426.2	4.00	.200
		15	535.2	4.50	.200
		16	627.5	5.00	.200

41	6440.00	1	0.0	5.00	.200
		2	10.0	4.50	.200
		3	20.0	4.00	.200
		4	40.0	3.50	.200
		5	115.0	3.00	.060
		6	125.0	2.80	.040
		7	130.0	2.50	.040
		8	150.0	2.00	.040
		9	160.0	2.00	.040
		10	180.0	2.50	.040
		11	185.0	2.80	.040
		12	195.0	3.00	.060
		13	302.4	3.50	.200
		14	426.2	4.00	.200
		15	535.2	4.50	.200
		16	627.5	5.00	.200
42	6580.00	1	0.0	5.00	.200
		2	10.0	4.50	.200
		3	20.0	4.00	.200
		4	40.0	3.50	.200
		5	115.0	3.00	.060
		6	125.0	2.80	.040
		7	130.0	2.50	.040
		8	180.0	2.50	.040
		9	185.0	2.80	.040
		10	195.0	3.00	.060
		11	302.4	3.50	.200
		12	426.2	4.00	.200
		13	535.2	4.50	.200
		14	627.5	5.00	.200
43	6610.00	1	0.0	5.00	.100
		2	7.5	4.30	.070
		3	20.0	4.10	.060
		4	23.5	3.40	.040
		5	24.0	3.00	.040
		6	30.0	3.00	.040
		7	30.5	3.40	.040
		8	35.0	4.10	.060
		9	57.5	4.30	.070
		10	65.0	5.00	.100
44	6620.00	1	0.0	5.00	.100
		2	7.5	4.30	.070
		3	20.0	4.10	.060
		4	23.5	3.40	.040
		5	24.0	3.00	.040
		6	30.0	3.00	.040
		7	30.5	3.40	.040
		8	35.0	4.10	.060
		9	57.5	4.30	.070
		10	65.0	5.00	.100
45	6650.00	1	0.0	5.00	.060
		2	10.0	4.50	.060
		3	20.0	4.00	.060
		4	100.0	3.50	.060
		5	200.0	3.00	.040
		6	404.0	3.00	.040
		7	1120.0	3.50	.060
		8	1186.0	4.00	.060
		9	1212.0	4.50	.060
		10	1239.0	5.00	.060
46	7350.00	1	0.0	5.00	.100

		2	10.0	4.50	.100
		3	20.0	4.00	.060
		4	40.0	3.50	.060
		5	100.0	3.00	.040
		6	304.0	3.00	.040
		7	1060.0	3.50	.060
		8	1186.0	4.00	.060
		9	1212.0	4.50	.100
		10	1239.0	5.00	.100
47	9350.00	1	0.0	5.00	.060
		2	100.0	4.50	.060
		3	200.0	4.00	.060
		4	300.0	3.50	.060
		5	1000.0	3.00	.060
		6	1800.0	-4.00	.060
		7	1900.0	-7.00	.060
		8	3400.0	-7.00	.060
		9	3600.0	-4.00	.060
		10	5346.0	3.00	.060
		11	7680.0	3.50	.060
		12	7834.0	4.00	.060
		13	7898.0	4.50	.060
		14	7961.0	5.00	.060

APPENDIX C2 : SWARTVLEI ESTUARY - RESULTS OF TIDAL RUN

FINITE DIFFERENCE FLOW ANALYSIS
using the COMPLETE DYNAMIC EQUATIONS

DATA FILES :
Cross sections : *Reac2*
LHS Boundary Conditions: Inflow hydrograph: QTWO
RHS Boundary Conditions: Tidal variation:
Mean sea level : 3.29 m
M2 amplitude : .680 m
S2 amplitude : .35 m
Phase Difference: 0 s
Time of Tide :340.67 h

Initial Conditions : INIT2_363

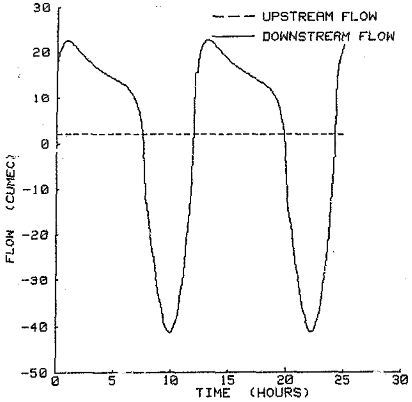
THETA : .6 PSI : .5
TIME STEP : 0.10 hours

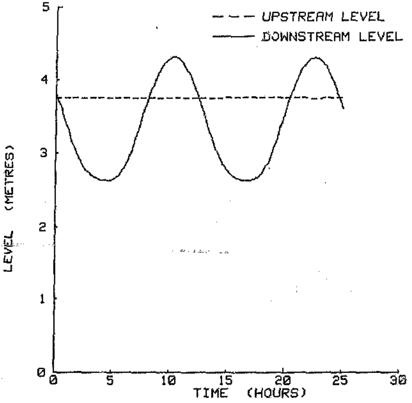
RUN 2

	SECTION 3		SECTION 5		SECTION 19		SECTION 36		SECTION 44	
TIME	STAGE	FLOW	STAGE	FLOW	STAGE	FLOW	STAGE	FLOW	STAGE	FLOW
0.00	3.738	-3.87	3.784	-3.96	3.686	-14.48	3.913	9.13	3.832	16.62
1.00	3.741	-4.22	3.803	-4.28	3.833	6.16	3.792	18.10	3.427	21.99
2.00	3.742	-2.88	3.777	-2.91	3.785	8.17	3.718	17.94	3.191	19.82
3.00	3.744	-1.36	3.753	-1.37	3.738	8.47	3.659	16.16	3.116	17.37
4.00	3.744	1.60	3.735	1.62	3.695	8.43	3.611	14.56	3.069	15.60
5.00	3.744	2.27	3.718	2.29	3.657	8.20	3.569	13.34	3.023	14.15
6.00	3.744	2.66	3.703	2.67	3.622	7.90	3.534	12.25	3.003	12.76
7.00	3.744	2.93	3.689	2.94	3.589	7.49	3.504	10.96	3.223	9.16
8.00	3.743	3.13	3.677	3.14	3.566	5.95	3.560	-8.44	3.679	-14.68
9.00	3.743	3.17	3.673	3.17	3.633	-5.77	3.723	-26.72	4.052	-32.94
10.00	3.743	2.47	3.705	2.47	3.722	-12.49	3.894	-36.96	4.244	-41.12
11.00	3.743	-.08	3.745	-.09	3.826	-18.11	3.999	-31.53	4.204	-30.93
12.00	3.745	-1.97	3.779	-1.86	3.891	-17.70	3.968	-3.60	3.941	3.91
13.00	3.747	-3.27	3.811	-3.19	3.850	3.69	3.822	17.23	3.534	22.09
14.00	3.749	-2.74	3.790	-2.70	3.804	8.01	3.741	18.50	3.234	20.79
15.00	3.751	-1.66	3.765	-1.65	3.756	8.53	3.679	16.85	3.137	10.17
16.00	3.751	1.81	3.746	1.88	3.71`	8.57	3.629	15.16	3.086	16.21
17.00	3.751	2.34	3.729	2.30	3.674	8.43	3.585	13.84	3.040	14.70
18.00	3.751	2.73	3.714	2.75	3.638	8.13	3.548	12.71	3.006	10.36
19.00	3.751	2.99	3.699	3.01	3.604	7.75	3.517	11.62	3.123	10.87
20.00	3.750	3.19	3.687	3.20	3.575	7.11	3.523	.73	3.565	-5.92
21.00	3.750	3.31	3.677	3.32	3.618	-3.95	3.579	-22.01	3.964	-20.32
22.00	3.750	2.83	3.700	2.83	3.699	-10.63	3.853	-35.26	4.212	-40.32
23.00	3.750	1.33	3.741	1.33	3.802	-16.86	3.982	-34.56	4.237	-35.40
24.00	3.752	-3.84	3.775	-4.00	3.883	-18.89	3.995	-15.52	4.033	-10.88
25.00	3.754	-4.92	3.813	-5.05	3.871	-3.83	3.852	15.17	3.648	20.96

MAXIMUM STAGE AND FLOW VALUES

SECTION	MAXIMUM STAGE (m)	TIME (hours)	MAXIMUM FLOW (cumec)	TIME (hours)	MINIMUM STAGE (m)	MINIMUM FLOW (cumec)
1	3.753	25.000	2.000	.100	3.738	2.000
2	3.753	25.000	3.196	21.100	3.738	-4.155
3	3.754	25.000	3.310	21.000	3.738	-4.917
4	3.789	24.900	3.318	21.000	3.702	-5.049
5	3.813	25.000	3.318	21.000	3.671	-5.048
6	3.849	13.000	3.320	20.800	3.626	-4.945
7	3.849	13.000	3.455	20.600	3.626	-4.655
8	3.849	13.000	3.676	20.500	3.625	-5.351
9	3.849	13.000	3.706	20.500	3.625	-5.453
10	3.849	13.000	3.711	20.500	3.625	-5.468
11	3.851	13.000	3.900	20.300	3.620	-6.321
12	3.860	25.000	4.556	19.500	3.601	-9.834
13	3.863	25.000	4.963	18.700	3.595	-11.691
14	3.866	24.900	5.538	17.200	3.591	-13.426
15	3.869	24.800	6.127	16.600	3.587	-14.614
16	3.872	24.800	6.588	16.600	3.585	-15.515
17	3.878	24.600	7.083	16.500	3.579	-16.453
18	3.888	24.500	7.739	16.400	3.572	-17.732
19	3.895	24.500	8.595	16.300	3.566	-19.225
20	3.899	24.400	8.910	15.500	3.564	-19.708
21	3.902	24.400	9.143	15.400	3.562	-20.065
22	3.905	24.400	9.369	15.200	3.561	-20.439
23	3.906	24.300	9.513	15.200	3.561	-20.672
24	3.912	24.300	9.755	15.100	3.557	-21.090
25	3.927	24.200	10.150	14.900	3.549	-21.850
26	3.939	24.100	10.628	14.600	3.542	-22.666
27	3.955	24.000	11.241	14.500	3.530	-23.571
28	3.971	23.900	12.146	14.400	3.515	-24.917
29	3.976	23.900	12.565	14.300	3.512	-25.541
30	3.983	23.800	12.906	14.300	3.509	-26.072
31	3.989	23.800	13.339	14.200	3.507	-26.744
32	3.994	23.700	13.714	14.200	3.505	-27.417
33	3.998	23.700	14.419	14.000	3.504	-28.885
34	4.002	23.700	16.579	13.800	3.501	-33.355
35	4.005	23.600	17.511	13.700	3.501	-35.050
36	4.008	23.600	18.725	13.600	3.500	-37.268
37	4.009	23.600	19.239	13.600	3.499	-38.322
38	4.010	23.600	19.474	13.600	3.498	-38.897
39	4.057	10.900	20.083	13.[illegible]	3.390	-39.841
40	4.109	10.700	20.379	13.500	3.299	-40.107
41	4.176	10.500	20.841	13.400	3.224	-40.407
42	4.202	10.400	21.235	13.400	3.182	-40.618
43	4.225	10.400	21.774	13.200	3.103	-40.899
44	4.257	10.300	22.243	13.200	3.000	-41.115
45	4.290	10.300	22.451	13.100	2.839	-41.198
46	4.310	10.300	22.847	13.100	2.627	-41.352
47	4.319	10.300	30.586	13.100	2.260	-43.199


UPSTREAM FLOW
DOWNSTREAM FLOW
30
20
10
0
-10
-20
-30
-40
-50
FLOW (CUMEC)
0
5
10
15
20
25
30
TIME (HOURS)



MASS BALANCE RESULTS:

Initial Volume	(1000m3) :	33480.700	Inflow	(1000m3)	180.000
Final Volume	(1000m3) :	33589.078	Outflow	(1000m3)	58.876
Increase in Volume	(1000m3) :	108.378	Net Inflow	(1000m3)	121.124
Error	:	-11.760 %			

APPENDIX D

RIETSPRUIT WATERCOURSE CROSS-SECTION DATA

APPENDIX D : RIETSPRUIT WATERCOURSE CROSS SECTION DATA

CROSS SECTION	REACH DISTANCE	POINT NUMBER	SECTION DISTANCE	GROUND LEVEL	MANNING N
1	0.00	1	0.0	49.00	.060
		2	5.0	47.20	.030
		3	105.0	47.20	.030
		4	110.0	49.00	.060
2	10.00	1	0.0	49.00	.060
		2	102.0	48.50	.055
		3	200.0	48.00	.050
		4	320.0	47.50	.045
		5	416.0	47.07	.040
		6	460.0	47.50	.045
		7	490.0	48.00	.050
		8	512.0	48.50	.055
		9	536.0	49.00	.060
3	210.00	1	0.0	49.00	.055
		2	50.0	48.50	.050
		3	222.0	48.00	.045
		4	272.0	47.90	.040
		5	304.0	48.00	.045
		6	346.0	48.50	.050
		7	367.0	49.00	.055
4	420.00	1	0.0	50.00	.060
		2	18.0	49.50	.050
		3	56.0	49.00	.045
		4	244.0	48.75	.040
		5	293.0	49.00	.045
		6	306.0	49.50	.050
		7	318.0	50.00	.055
		8	330.0	50.50	.060
		9	343.0	51.00	.065
		10	356.0	51.50	.070
5	620.00	1	0.0	51.00	.060
		2	16.0	50.50	.055
		3	32.0	50.00	.050
		4	107.0	49.50	.045
		5	250.0	49.37	.040
		6	300.0	49.50	.045
		7	325.0	50.00	.050
		8	346.0	50.50	.055
		9	366.0	51.00	.060
		10	378.0	51.50	.065
6	820.00	1	0.0	52.50	.065
		2	16.0	52.00	.060
		3	27.0	51.50	.055
		4	39.0	51.00	.050
		5	68.0	50.50	.045
		6	173.0	50.04	.040
		7	322.0	50.50	.045
		8	332.0	51.00	.050
		9	350.0	51.50	.055
		10	367.0	52.00	.060
		11	380.0	52.50	.065
7	1020.00	1	0.0	52.50	.060

		2	16.0	52.00	.055
		3	34.0	51.50	.050
		4	60.0	51.00	.045
		5	158.0	50.74	.040
		6	264.0	51.00	.045
		7	310.0	51.50	.050
		8	345.0	52.00	.055
		9	380.0	52.50	.060
8	1220.00	1	0.0	53.90	.070
		2	35.0	53.16	.065
		3	68.0	52.32	.050
		4	95.0	52.05	.055
		5	145.0	51.75	.050
		6	178.0	51.70	.050
		7	192.0	51.97	.045
		8	235.0	52.00	.045
		9	308.0	51.53	.040
		10	373.0	52.50	.050
		11	385.0	53.00	.060
9	1420.00	1	0.0	53.82	.060
		2	47.0	53.51	.055
		3	96.0	53.27	.050
		4	142.0	52.88	.045
		5	158.0	52.25	.040
		6	167.0	52.86	.045
		7	212.0	52.43	.050
		8	260.0	52.20	.055
		9	282.0	52.00	.060
		10	358.0	53.00	.065
10	1520.00	1	0.0	54.14	.060
		2	40.0	53.53	.055
		3	83.0	53.21	.050
		4	103.0	52.86	.045
		5	150.0	52.92	.045
		6	196.0	52.58	.040
		7	238.0	52.72	.045
		8	307.0	53.00	.050
		9	340.0	54.00	.060
11	1620.00	1	0.0	54.20	.060
		2	48.0	59.87	.055
		3	68.0	53.76	.050
		4	92.0	53.29	.045
		5	108.0	52.49	.040
		6	115.0	53.34	.045
		7	127.0	52.67	.050
		8	152.0	53.27	.055
		9	195.0	53.03	.060
		10	234.0	53.09	.065
		11	246.0	53.36	.065
		12	380.0	54.00	.070
12	1720.00	1	0.0	55.09	.060
		2	44.0	54.74	.055
		3	87.0	54.21	.050
		4	133.0	53.79	.045
		5	152.0	52.90	.040
		6	168.0	53.52	.045
		7	212.0	53.24	.050
		8	230.0	53.50	.055
		9	278.0	55.00	.060

13	1820.00	1	0.0	56.28	.060
		2	45.0	55.57	.055
		3	95.0	54.95	.050
		4	144.0	53.91	.045
		5	156.0	53.30	.040
		6	164.0	53.71	.045
		7	206.0	53.59	.050
		8	253.0	55.36	.055
		9	288.0	56.00	.060
14	1870.00	1	0.0	56.40	.060
		2	40.0	56.10	.055
		3	87.0	54.99	.050
		4	130.0	54.18	.045
		5	143.0	53.44	.040
		6	159.0	53.64	.045
		7	280.0	54.27	.050
		8	247.0	55.71	.055
		9	270.0	56.00	.060
15	1920.00	1	0.0	56.71	.065
		2	56.0	56.11	.060
		3	94.0	56.35	.055
		4	140.0	54.59	.050
		5	157.0	54.00	.045
		6	167.0	53.55	.040
		7	190.0	54.00	.045
		8	234.0	55.00	.050
		9	254.0	55.71	.055
		10	272.0	56.00	.060
16	1970.00	1	0.0	56.49	.065
		2	45.0	56.65	.060
		3	90.0	64.95	.055
		4	128.0	54.54	.050
		5	132.0	54.50	.045
		6	155.0	54.12	.040
		7	168.0	54.50	.045
		8	175.0	54.69	.050
		9	216.0	56.83	.060
17	2070.00	1	0.0	56.62	.065
		2	50.0	56.54	.060
		3	100.0	56.56	.055
		4	147.0	56.43	.055
		5	192.0	56.72	.055
		6	249.0	55.11	.050
		7	2[illegible].0	54.40	.045
		8	282.0	54.94	.050
		9	339.0	56.30	.055
		10	378.0	56.09	.060
		11	418.0	56.50	.065
18	2270.00	1	0.0	59.12	.060
		2	20.0	58.00	.055
		3	76.0	66.95	.050
		4	117.0	56.36	.045
		5	149.0	55.72	.040
		6	155.0	56.68	.045
		7	263.0	56.50	.050
		8	289.0	57.50	.060
19	2370.00	1	0.0	58.76	.055
		2	27.0	58.00	.050

		3	53.0	57.00	.045
		4	150.0	56.50	.040
		5	195.0	57.00	.050
		6	250.0	57.50	.060
		7	304.0	58.00	.070
20	2475.00	1	0.0	60.00	.065
		2	18.0	59.00	.060
		3	40.0	58.00	.055
		4	52.0	57.50	.050
		5	76.0	57.00	.045
		6	114.0	56.87	.040
		7	172.0	57.00	.045
		8	235.0	57.50	.050
		9	547.0	58.00	.055
		10	640.0	59.00	.060
21	2675.00	1	0.0	61.00	.065
		2	27.0	60.00	.060
		3	55.0	59.00	.055
		4	80.0	58.50	.050
		5	98.0	58.00	.045
		6	153.0	57.89	.040
		7	260.0	58.00	.045
		8	290.0	58.50	.050
		9	320.0	59.00	.055
		10	380.0	60.00	.060
22	2875.00	1	45.0	61.00	.060
		2	95.0	60.00	.055
		3	112.0	59.50	.050
		4	131.0	59.00	.045
		5	188.0	58.77	.040
		6	258.0	59.00	.045
		7	306.0	59.50	.050
		8	347.0	60.00	.055
		9	384.0	61.00	.060
23	3075.00	1	59.0	61.00	.060
		2	76.0	60.50	.055
		3	98.0	60.00	.050
		4	115.0	59.50	.045
		5	136.0	59.32	.040
		6	176.0	59.50	.045
		7	252.0	60.00	.050
		8	318.0	60.50	.055
		9	342.0	61.00	.060
24	3275.00	1	37.0	62.00	.060
		2	69.0	61.00	.055
		3	96.0	60.50	.050
		4	120.0	60.00	.045
		5	138.0	59.98	.040
		6	152.0	60.00	.045
		7	225.0	60.50	.050
		8	280.0	61.00	.055
		9	335.0	62.00	.060
25	3405.00	1	0.0	62.50	.070
		2	28.0	62.00	.065
		3	48.0	61.50	.060
		4	122.0	61.00	.055
		5	232.0	60.50	.050
		6	245.0	60.43	.045

		7	257.0	60.50	.050
		8	268.0	61.00	.055
		9	285.0	61.50	.060
		10	299.0	62.00	.065
		11	313.0	62.50	.070
26	3605.00	1	0.0	64.00	.060
		2	42.0	63.00	.055
		3	65.0	62.50	.050
		4	90.0	62.00	.045
		5	200.0	61.54	.040
		6	275.0	62.00	.045
		7	298.0	63.00	.050
		8	323.0	64.00	.060
27	3805.00	1	0.0	65.00	.065
		2	30.0	64.00	.060
		3	50.0	63.50	.055
		4	72.0	63.00	.050
		5	128.0	62.58	.045
		6	152.0	63.00	.050
		7	176.0	63.50	.055
		8	244.0	64.00	.060
		9	264.0	65.00	.065
28	4005.00	1	0.0	65.00	.060
		2	28.0	64.50	.055
		3	58.0	64.00	.050
		4	114.0	63.56	.045
		5	173.0	64.00	.050
		6	192.0	64.50	.055
		7	208.0	65.00	.060
29	4205.00	1	0.0	68.00	.070
		2	24.0	67.00	.065
		3	50.0	66.00	.060
		4	65.0	65.50	.055
		5	140.0	65.00	.050
		6	192.0	64.74	.045
		7	208.0	65.00	.050
		8	229.0	66.00	.055
		9	250.0	66.50	.060
		10	267.0	67.00	.065
		11	294.0	68.00	.070
30	4405.00	1	16.0	69.00	.070
		2	30.0	68.00	.065
		3	50.0	67.00	.060
		4	60.0	66.50	.055
		5	78.0	66.00	.050
		6	154.0	65.75	.045
		7	178.0	66.60	.050
		8	235.0	67.00	.060
		9	272.0	68.00	.065
		10	306.0	69.00	.070
31	4605.00	1	0.0	71.20	.070
		2	38.0	68.22	.065
		3	80.0	67.13	.060
		4	114.0	66.83	.055
		5	150.0	66.83	.050
		6	190.0	66.60	.045
		7	225.0	66.85	.050
		8	268.0	67.76	.055

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