

**DEVELOPING A REAL TIME HYDRAULIC MODEL
AND A DECISION SUPPORT TOOL
FOR THE OPERATION OF THE ORANGE RIVER**

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In fulfilment of the requirements of a
Master of Science in Engineering
as set out by the University of Natal

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Disclaimer

This thesis was submitted by Kerry Fair to fulfil the requirements of a Masters of Science in Engineering as set out by the University of Natal. The thesis was based on a project sponsored by the Water Research Commission of South Africa. All work described in this thesis was carried out by the author, except for the following:

- *The real time unit was designed in conjunction with Dr Chris Whitlow. Changes to the hydraulic model to include the real time data and the adaptive time stepping routine were made by Dr C Whitlow*
- *Data capturing required for the ISIS model of the Orange River was done by Kerry Fair and a team of BKS (Pty) Ltd staff members,*
- *Much of the theoretical information included is based on work published in **Applied Hydrology**, Chow Maidment and Mays, 1998.*

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ABSTRACT

This thesis describes the development of a decision support tool to be used in the operation of Vanderkloof Dam on the Orange River so that the supply of water to the lower Orange River can be optimised. The decision support tool is based on a hydrodynamic model that was customised to incorporate real time data recorded at several points on the river. By incorporating these data into the model the simulated flows are corrected to the actual flow conditions recorded on the river, thereby generating a best estimate of flow conditions at any given time. This information is then used as the initial conditions for forecast simulations to assess whether the discharge volumes and schedules from the dam satisfy the water demands of downstream users, some of which are 1400km or up to 8 weeks away.

The various components of the decision support system, their functionality and their interaction are described. The details regarding the development of these components include:

- The hydraulic model of the Orange River downstream of Vanderkloof Dam. The population and calibration of the model are described.
- The modification of the code of the hydrodynamic engine so that real time recorded stage and flow data can be incorporated into the model
- The development of a graphical user interface to facilitate the exchange of data between the real time network of flow gauging stations on the Orange River and the hydraulic model
- The investigation into the effect of including the real time data on the simulated flows
- Testing the effectiveness of the decision support system

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SUMMARY

Motivation for the research

The Orange River is one of South Africa's most important water resources and its catchment covers a significant portion of Southern Africa. Vanderkloof Dam, the last major storage structure on the river, is 1400km from the river mouth where the furthest downstream users obtain their water.

Upstream of Vanderkloof Dam the water in the Orange River supplies the demands in the Orange River basin. It is also used to augment the supply of water to other catchments through inter-basin transfer schemes. One of these schemes is the well known Lesotho Highlands Water Project, in which water is transferred from the upper reaches of the Orange River to the Vaal River catchment in order to augment the supply of water to Gauteng, the industrial and economic centre of South Africa. Water is also transferred from Gariep Dam on the Orange River through the Orange-Fish tunnel to the Eastern Cape for use in the Fish and Sundays River basins.

In the catchment downstream of Vanderkloof Dam there are agricultural, industrial, urban and mining developments that obtain their water from the Orange River. There are also important environmental requirements along the river and at the river mouth that must be satisfied. The catchment downstream of Vanderkloof Dam is extremely arid and the only reliable source of water in the lower reaches of the river are the releases made from Vanderkloof Dam.

As the water requirements on the Orange River catchment continue to grow, the water in the catchment is becoming ever more valuable. It is therefore becoming increasingly important to optimise the releases from Vanderkloof Dam in order to supply the downstream demands whilst minimising any excess releases, so as to conserve the valuable resource in Vanderkloof Dam.

The major difficulty in optimising the releases from Vanderkloof Dam is that it can take several weeks for water released from the dam to reach the river mouth where the domestic, industrial and environmental demands must be satisfied. The time taken for the releases to

reach the mouth depends on the prevailing conditions in the river in relation to the amount of water released. Releases during low flow conditions may take up to 6 weeks to reach the mouth. It is evident that there can be no quick relief from any shortages that might be experienced along the lower Orange River through additional releases from Vanderkloof Dam. Due to the nature of these demands any water shortages would have a major impact, both economic and environmental.

For this reason it was decided to develop a tool that could be used to help determine the required releases from Vanderkloof Dam. A hydraulic model of the Orange River was developed to be used by the decision makers to simulate and then evaluate various proposed release scenarios.

Motivation for the development of a real time hydraulic model

There are several potential challenges in modelling the Orange River downstream of Vanderkloof Dam:

- The length of the river is 1400km.
- Due to the remote terrain through which the river flows the abstractions from the river are not recorded in real time and it is not possible to include accurate abstraction data in the river model
- The return flows from irrigation along the river are also uncertain
- Losses from the river are also significant. Evaporation losses depend on the surface area of the water and therefore the local flow conditions.
- Inflows from tributaries are not recorded in real time.

In order to overcome these difficulties and improve the modelled results, it was decided to investigate the inclusion of real-time data in the hydraulic model. Real time conditions, either stage or flow, are recorded at several sites along the Orange River. By incorporating these data into the model at several points on the river the simulated flows would be corrected to the actual flow conditions. Not only would this improve the accuracy of the model but the difference between the modelled and recorded flows could be used to quantify the net inflow or abstractions from river reaches between the real time stations.

Objective of the research

The aim of the project was to develop a decision support tool for the operation of the Orange River downstream of Vanderkloof Dam. This tool would be based on a calibrated hydraulic model of the river. Flows from the Vaal River into the Orange River were to be included in the model. The model would be used to simulate various possible release scenarios so that

the resultant river conditions could be assessed before a decision is made regarding the release schedule.

Another aspect of the study was to investigate the possibility of linking the hydraulic model to the network of telemetry stations on the Orange River. The objective was to include the real time stage and discharge data recorded at these stations in the hydraulic model and to assess whether the forecast flow conditions on the river could be improved by including such data.

In order to satisfy the basic objective of the study the following tasks were conducted:

- Set up a hydraulic model of the Orange River downstream of Vanderkloof Dam using the ISIS hydro-dynamic model:*
- Calibrate the model for low flow conditions using historical data*
- Modify the code of the ISIS engine so that real time recorded stage and flow data can be incorporated into the model*
- Develop a graphical user interface (GUI) to facilitate the exchange of data between the real time network and the hydraulic model*
- Investigate the effect of including the real time data on the simulated flows*
- Test the effectiveness of the decision support system*

Technological Developments

In order to successfully model the Orange River several major and innovative developments were made to the ISIS software:

- The real time hydraulic unit. This development is a novelty in river modelling and was developed and tested as part of this project. This unit forces the simulated values in the model to match the actual stage and flow conditions recorded at several points on the river, thereby improving the accuracy of the model.*
- An adaptive timestepping routine. This routine allows a relatively long timestep to be used for the simulation. If instabilities occur the timestep is reduced until a solution is found, or the minimum timestep is reached. The longer timestep results in shorter computational times. This unit is now available in the commercial releases of ISIS.*
- A decision support tool was developed which will be used by the operators of Vanderkloof Dam to help determine the required releases from the dam in terms of quantity and timing. The tool will be used by the operators to simulate proposed release scenarios on a what if basis, given the current river conditions and expected downstream water requirements. The system would be made up of three components: A real time model of the Orange River, which will continuously update the simulations of the river using the real time data recorded on the river, so that a*

best estimate of actual flow and stage will be available at every computational node of the model. Once the real time model has been updated with the most recent data, the second component, the forecast model, is intended to simulate the scheduled releases from Vanderkloof Dam, giving a best estimate of future conditions on the river. The third component is to be used to simulate any proposed release scenarios, so that it can be determined whether they meet the required downstream flow requirements.

- *A Graphical User Interface. This development was made to facilitate the exchange of data from the bank of real time recorded data, which are available from the South African Department of Water Affairs and Forestry, and the hydraulic model.*

Conclusions and Recommendations

The strategy determined in this study will provide a rational basis for the operators of Vanderkloof Dam to determine a discharge release pattern to ensure that the various demands downstream of Vanderkloof Dam are satisfied. As the model is based on sound hydraulic principles rather than simplified routing methods the users can be confident in the simulated results, provided that the real time data are accurate and available. The use of real time data further improves the simulation because unmodelled events such as localised inflows can be taken into account.

One shortcoming of the project is that although the methodology has been developed and tested on historical data it has not been possible, to date, to test the process sufficiently on real time data. This is due to the lack of available accurate real time data. The following recommendations can therefore be made:

- *The acquisition of real time data from the Orange River be improved. The real time telemetry network must be well maintained so that accurate and up to date information is available.*
- *It is recommended that the real time data are made available on an FTP site, so that the data do not have to be e-mailed to interested parties. Alternatively the full set of information should be made available on the website. At present only the average readings for every hour are included.*
- *It is also strongly recommended that the releases from Vanderkloof Dam are made available in real time. This will require co-operation between Eskom and the Department of Water Affairs and Forestry.*
- *During the simulation procedure it was concluded that the data at Zeekoebaart were inaccurate. It is recommended that this inaccuracy be investigated further and corrected.*

Future Extensions to the Project

Various possible extensions to the project have been identified. They have not been investigated as part of this study but would be logical and useful developments:

- An automatic calibration routine could be developed as an extension to the real time hydraulic model. This routine would compare the actual conditions recorded on the river to the flows simulated by the model and automatically adjust the model parameters to improve the calibration of the model. This might be beneficially combined with a Kalman filter.*
- Extending the cross sectional data in the model to include detail for high flows so that the model could be used as a flood management tool.*
- Water quality modelling could also be included in the simulations*
- Modelling of the instream flow requirements of the river is a natural extension of the modelling process.*

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1 INTRODUCTION

1.1 MOTIVATION FOR THE RESEARCH

The Orange River is one of South Africa's most important water resources. It originates in the Maluti Mountains in Lesotho and flows 2300km westward across South Africa to the Atlantic Ocean (Figure 1.1).

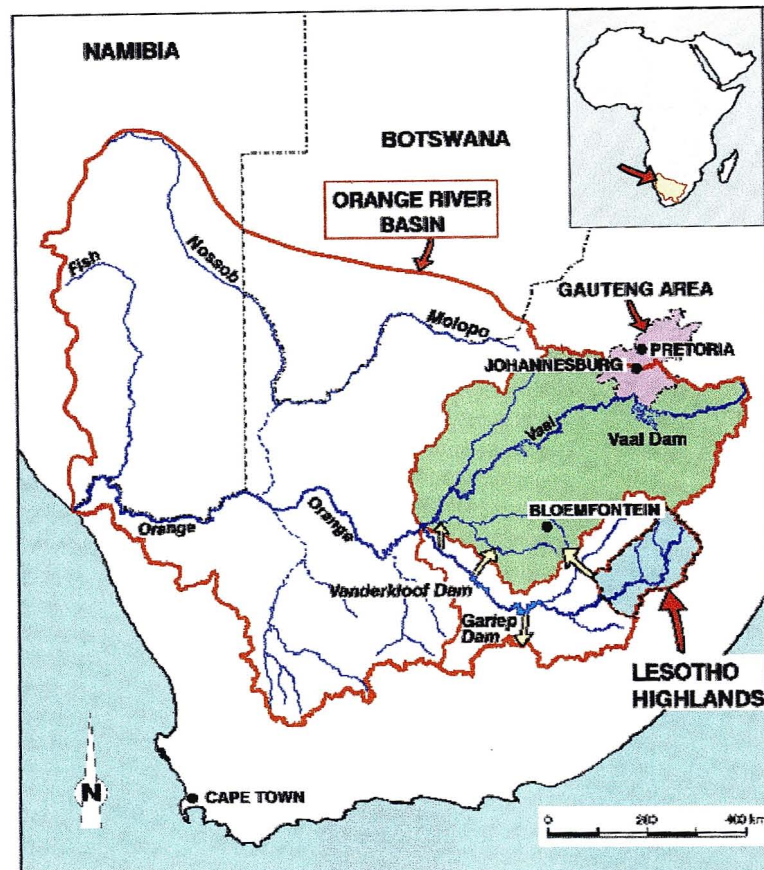


Figure 1.1: Location of the Orange River

As can be seen from Figure 1.1, the Orange River catchment covers a significant portion of Southern Africa. A logical division of this catchment (for the purpose of this study) occurs at Vanderkloof Dam as it is the last major storage structure on the river.

In the catchment downstream of Vanderkloof Dam there are agricultural, industrial, urban and mining developments which obtain their water from the Orange River. There are also important environmental requirements along the river and at the river mouth which must be satisfied. In fact, as it is of major importance to several migratory bird species, the Orange River Delta has recently been declared a RAMSAR site, which means it is an internationally acclaimed environmental site.

The catchment downstream of Vanderkloof Dam is extremely arid. The gross annual average evaporation of 2850mm far exceeds the annual average rainfall of 150mm with the result that the runoff from the catchment is sporadic and rarely significant (McKenzie and Craig, 1998). The only reliable source of water in the lower reaches of the river are the releases made from Vanderkloof Dam.

The Vaal River is a major tributary to the Orange with the confluence being downstream of Vanderkloof Dam. Gauteng (See Figure 1.1), the industrial and economic centre of South Africa, is supplied with water from the Vaal River catchment. The resources of the Vaal River catchment are insufficient to meet the demands in Gauteng and its supply is augmented through several inter-basin transfer schemes. One of these schemes is the well known, and recently implemented, Lesotho Highlands Water Project which transfers water from the upper reaches of the Orange River to the Vaal River catchment. Therefore, instead of the Vaal River contributing to the flow in the Orange River, the natural flow in the Orange River is in fact reduced due to the transfers from the upper Orange River to the Vaal River through the Lesotho Highlands Water Project. As a result the Vaal River is not operated to contribute to the flow in the Orange River.

Upstream of the Vanderkloof Dam the water in the Orange River supplies the demands in the Orange River basin. It is also used to augment the supply to other catchments through inter-basin transfer schemes as shown in Figure 1.1. In addition to the Lesotho Highlands Water Project, water is transferred from Gariep Dam through the Orange-Fish tunnel to the Eastern Cape for use in the Fish and Sundays River basins.

As demands for water in the industrialised Gauteng continue to increase, further transfers from the Orange River catchment upstream of Vanderkloof Dam have been planned to meet this need. These transfers will result in a decrease in the amount of water available for releases from Vanderkloof Dam. Because of these transfers from the Orange River, Vanderkloof Dam does not spill and there are no inflows from the Vaal River under normal hydrological conditions. The developments in the lower reaches of the Orange River are therefore dependent on the releases made from Vanderkloof Dam. However, it is evident that any releases from Vanderkloof Dam, in excess of those required by the downstream users, would result in a waste of an important resource.

It is therefore becoming increasingly important to optimise the releases from Vanderkloof Dam in order to supply the downstream demands whilst minimising any excess releases, so as to conserve the valuable resource in Vanderkloof Dam.

Releases from Vanderkloof Dam are also required for hydro-power generation by Eskom, the South African electricity supply utility. In the past, when the inter-basin transfers from the Orange River upstream of Vanderkloof Dam were smaller, the releases made for hydro-power generation at Vanderkloof Dam were in excess of the demands downstream of the dam. The situation is further complicated in that the releases are generally used for hydro-power generation during peak loads on the electricity network, particularly in the winter months, when the downstream irrigation demands are low. In contrast, when the irrigation demands from the river are high during the summer months, the hydro-power generation is lower due to lower electricity demands.

The major difficulty in optimising the releases from Vanderkloof Dam is that it can take several weeks for water released from the dam to reach the river mouth where the domestic, industrial and environmental demands must be satisfied. The time taken for the releases to reach the mouth depends on the prevailing conditions in the river in relation to the amount of water released. At low summer flows the releases of $100\text{m}^3/\text{s}$ from Vanderkloof Dam take approximately 4 weeks to reach the river mouth. In winter when both the demands and evaporation are lower, the releases of approximately $40\text{m}^3/\text{s}$ take over 6 weeks to travel the length of the river.

It is evident that there can be no quick relief from any shortages that might be experienced along the lower Orange River through additional releases from Vanderkloof Dam. Due to the nature of these demands any water shortages would have a major impact, both economic and environmental.

The primary purpose of this study was to develop a tool which could be used to help determine the required releases from Vanderkloof Dam. The aim was to develop a hydraulic model of the Orange River downstream of Vanderkloof Dam which could be used by the decision makers to simulate and then evaluate various proposed release scenarios from Vanderkloof Dam.

1.2 MOTIVATION FOR THE DEVELOPMENT OF A REAL TIME HYDRAULIC MODEL

There are several potential difficulties in modelling a river 1400km long. Firstly, due to the remote terrain through which the river flows the abstractions from the river are not recorded in real time and it is not possible to include accurate abstraction data in the river model. Irrigation abstractions in particular are difficult to model. Although there is a maximum annual quota which may be pumped from the river, it is not necessarily always used. The day to day abstractions differ from the annual quota because of variations in the requirements of the crops due to seasonal and daily climatic conditions.

Secondly, the return flows from irrigation along the river are also uncertain. Although several studies have been conducted which have improved these estimates (McKenzie and Roth, 1994; McKenzie and Craig, 1997b), there are still uncertainties with regard to both the quantity and timing of these flows. It is particularly difficult to model the return flows accurately when the amount of water being used for irrigation, from which the return flows are generated, is also not certain.

Thirdly losses from the river are also significant (McKenzie and Roth, 1994; McKenzie and Craig, 1999). Evaporation losses depend not only on the current weather conditions (which are more often than not extremely hot and dry) but also on the surface area of the water and therefore the local flow conditions. Inflows from tributaries are also not recorded in real time.

Another anomaly is that although the Vaal River is usually not operated to contribute to the flow in the Orange River during low flow conditions, it has recently been observed that inflow from the Vaal River does occur, particularly at the start of the winter period. These spills are thought to originate from return flows from irrigation along the Vaal River and could be used to reduce releases from Vanderkloof Dam if they can be quantified and included in the model.

In order to overcome these difficulties and improve the modelled results, it was decided to investigate the inclusion of real-time data in the hydraulic model. Real time conditions, either stage or flow, are recorded at several sites along the Orange River as part of the HYCOS system (Chapter 4). It was hoped that by incorporating these data into the model at several points on the river the simulated flows would be corrected to the actual flow conditions. Not only would this improve the accuracy of the model but the difference between the modelled and recorded flows could be used

to quantify the net inflow or abstractions from river reaches between the real time stations.

1.3 OBJECTIVES OF THE RESEARCH STUDY

The main objective of the project was to develop a decision support tool for the operation of the Orange River downstream of Vanderkloof Dam. This tool would be based on a calibrated hydraulic model of the river. Flows from the Vaal River into the Orange River were to be included in the model. The model would be used to simulate various possible release scenarios so that the resultant river conditions could be assessed before a decision is made regarding the release schedule.

Another aspect of the study was to investigate linking the hydraulic model to the network of telemetry stations on the Orange River. The objective was to include the real time stage and discharge data recorded at these stations in the hydraulic model and to assess whether the forecast flow conditions on the river could be improved by including such data. This idea is new to river modelling in South Africa and the hydraulic modelling package had to be modified before the real time data could be simulated. It was also necessary to develop a graphical user interface to facilitate the exchange of data between the real time telemetry network and the hydraulic model.

In order to satisfy the main objective of the study the following tasks were conducted. Unless otherwise stated the tasks were performed solely by the candidate.

- Set up a hydraulic model of the Orange River downstream of Vanderkloof Dam using the ISIS hydro-dynamic model. This involved obtaining information describing the 1300km of the river. The data capturing was performed by a team of BKS (Pty) Ltd staff including the candidate.
- Calibrate the model for low flow conditions using historical data. It was found that the calibration process was complicated by the inclusion of real time data in the simulations. During the calibration process it became evident that some of the real time stations were inaccurate and instead of improving the simulation, as was hoped, actually made the model unacceptably inaccurate.
- Modify the code of the ISIS engine so that real time recorded stage and flow data can be incorporated into the model. As the ISIS software package is a commercial package the required code changes could not be made by the author. Dr Chris Whitlow was responsible for the coding of the software. However the specifications for the real time unit were drawn up by the candidate and Dr Whitlow. As part of the research the candidate tested the functioning of the real time unit.

- Develop a graphical user interface (GUI) to facilitate the exchange of data between the real time network and the hydraulic model. The candidate was responsible for the design, specifications, coding and testing of the user interface.
- Investigate the effect of including the real time data on the simulated flows. As part of the research the candidate ran numerous simulations of flow in the Orange River so that the effect of including the real time data in the simulations could be determined.
- Test the effectiveness of the decision support system. This task was performed by the candidate.

Various possible extensions to the project have been identified by the candidate. They have not been investigated as part of this study but would be logical and useful developments:

- An automatic calibration routine could be developed as an extension to the real time hydraulic model. This routine would compare the actual conditions recorded on the river to the flows simulated by the model and automatically adjust the model parameters to improve the calibration of the model. These parameters include the conveyance of a river section which changes with flow. This is generally adjusted by altering Mannings n for different flow regimes. Should this routine be developed its application on real river systems should be investigated fully as the calibration of the model will depend on the accuracy of the real time data. In the case of the Orange River the calibration process, which was done manually during an iterative process, was impeded by the poor quality of the real time data at several gauging stations. In fact the inaccuracies of the data were only determined in the calibration process (See Chapter 9.1.7)
- Extending the cross sectional data in the model to include detail for high flows so that the model could be used as a flood management tool. The possibility of flooding along the Orange River, particularly near Upington is a problem. The hydraulic model could be used to estimate potential flooding along the river and the effectiveness and impact of proposed prevention measures. In order to do this the model would also have to be calibrated for high flows.
- Water quality modelling could also be included in the simulations. Water quality and sedimentation are not yet a concern along the Orange River. However, it is envisaged that after the implementation of all the phases of the Lesotho Highlands Water Project the reduced flows in the river will result in a

higher concentration of irrigation return flows which may result in a corresponding deterioration in water quality.

- Modelling of the instream flow requirements of the river. Flow versus depth information is often required for environmental studies to determine the instream flow requirements of a river. The calibrated model could be used to determine this information for both existing and future conditions.

1.4 TECHNOLOGICAL DEVELOPMENTS

In order to successfully model the Orange River two major developments were made to the ISIS software. The first development was the real time hydraulic unit. This unit forces the actual stage and flow conditions recorded at several points on the river to be included in the hydraulic simulations. This unit was developed and tested in this study and the effect of the real time data on the simulated results was assessed. These modifications to the ISIS engine were made by Dr Chris Whitlow, an ISIS developer, in close co-operation with the candidate.

The second development was an adaptive timestepping routine. This routine allows a relatively long timestep to be used for the simulation. If instabilities occur the timestep is reduced until a solution is found, or the minimum timestep is reached. The longer timestep results in shorter computational times.

It was necessary to develop a user interface to facilitate the exchange of data from the real time recorded data, which are available from the South African Department of Water Affairs and Forestry, and the hydraulic model. This interface allows the user to easily manipulate the recorded data into the format required by the hydraulic model and to develop flow scenarios to be simulated by the model so that the resultant simulated flows can be evaluated by the user.

These developments are novel in hydraulic simulations. They were conceptualised, designed and tested by the candidate as part of this study.

1.5 BACKGROUND TO THE RESEARCH

Various detailed studies have been made of the Orange River. The Orange River System Analysis (ORSA) was completed in 1990 (BKS, 1991). This study analysed the Lesotho Highlands Water Project (LHWP) together with the remainder of the Orange River system. An important conclusion of the study was that the existing water demands downstream of Vanderkloof Dam would not be met if the proposed (as it was then) LHWP were to be fully developed. The study highlighted the importance of evaporation losses from the river as well as the difficulty of releasing

the correct amount of water from Vanderkloof Dam to supply the demands downstream of the dam without allowing too much water, over and above the environmental requirements, to spill into the Atlantic Ocean (BKS, 1990).

The findings of this study were further investigated during the Orange River Replanning Study (ORRS) (BKS, 1995) in which a framework was established for the future development of the river and the utilisation of its water.

The problem of determining the losses from the Orange River was addressed in the Orange River Losses Study Phase 1 (McKenzie et al , 1993, McKenzie and Roth, 1994, McKenzie and Stoffberg, 1995a). The aim of this study was to determine the losses from the Orange River downstream of Vanderkloof Dam. It was found that the net annual evaporation loss from the river was approximately 10% of the combined storage of the Vanderkloof and Gariep Dams and as such constituted a significant portion of the water resources of the Orange River.

Following the findings of the first phase of the Orange River Losses study, approval was given by the Water Research Commission for a further study on river losses. The purpose of the Evaporation Losses from South African Rivers (McKenzie and Craig, 1999) was to investigate the processes driving evaporation losses from rivers in order to refine previous estimates of losses from the Orange River between Vanderkloof Dam and the river mouth and also to determine a general methodology for estimating losses from other South African rivers.

In order to verify the methodology proposed by McKenzie and Craig (1999) a hydraulic model of the lower reaches of the Orange River, between Blouputs and Brandkaros, was prepared. In order to accurately model the evaporation from the river, which is dependent on the surface area of the river and therefore also on the flow, an evaporation module was developed by Dr Chris Whitlow for the hydraulic modelling package.

During the course of the evaporation losses study it became clear that a hydraulic model of the Orange River would be a useful decision support tool for the operation of the river and the determination of the releases from Vanderkloof Dam. The development of such a tool was a logical continuation of the model established for the evaporation losses study.

1.6 LITERATURE REVIEW

Apart from the studies conducted on the Orange River described in Section 1.5 there have been recent developments in river modelling technology that are of interest in the context of this study. The literature review on these developments is summarised in this section.

1.6.1 Real Time Hydraulic Modelling and River Management Decision Support Systems

The major developments in real time modelling and decision support systems in terms of river management have been made for hydropower optimisation and to ensure navigational safety for heavy shipping.

A real time river management system has been developed and implemented to increase the overall efficiency of Hydro-Quebec's hydropower capabilities. This system, operated under the name GASTEAU (Robitaille, 1995), was implemented to improve water management practices, optimise generating unit output and to minimise spillage. It was primarily designed to determine optimal unit loading conditions by minimising efficiency losses whilst enforcing all the relevant hydraulic constraints. To improve the accuracy of the results it interacts with a river routing model based on non-linear flow equations. Predictions of river water levels and flow rates can also be made.

It is interesting to note that river routing in GASTEAU is conducted through a combination of simple hydrologic modelling techniques and the more complex techniques based on the full St Venant equations.

The system was developed to benefit both the hydro electric plant performance and the operators of the system. Benefits to the operators include an improved environment in which to analyse the system, an integrated information retrieval system, support for decision system analysis, training of new staff and fast response times for decision making.

In his paper Allen (1996) also stresses the importance of the user friendly environment. Allen describes the decision support system used by Great Lakes Power, a private utility that generates, transmits and distributes power in the Algoma district of Northern Ontario, Canada. Great Lakes Power purchases power from Ontario when it cannot meet its own demand. It is estimated that the use of the

decision support system results in a saving of approximately \$1 million per annum in power purchases from Ontario.

Allen has found that operator satisfaction with computer software is a key aspect for successful implementation of decision support systems. Operators' aversion to computer software was mostly due to:

- a poor interface
- complicated manipulation of data and files
- various computer systems with different handling requirements
- lack of practical solutions
- programs designed to operate as 'Black Boxes'

Allen states that a successful implementation of a decision support system depends on its acceptance by the operators. This is facilitated by a user friendly environment and user-directed model evaluation techniques. In order for operators to gain acceptance of a DSS they must be able to review and test the results before implementation. Testing the solution is important for the operators' checking and understanding of the solution and its sensitivity. Another important aspect is the use of graphical displays to allow visual examination of the results.

The decision support system developed for the Orange River in this study differs from the Great Lakes Power system in that it does not (yet) make use of optimisation procedures. The user must define the release scenario and evaluate the resultant streamflow hydrograph. Nevertheless Allen's suggestions were implemented in that a user friendly environment was developed and graphical displays of input and results are available.

Another relevant application of decision support systems is for flood warning purposes. The system used for Sacramento, California (Ford, 2001) is made up of several components. The hub of the system is the emergency operations plan which includes a threat recognition and information dissemination plan. To apply the threat recognition rules in real time the system must be able to recognise an existing or potential threat in real time. To do so it determines actual and forecast rainfall depths and river stages and compares these to pre-defined conditions in the emergency operations plan.

Hydrometeorological data, including both rainfall and river stage are measured in real time throughout the catchment. These data are transmitted to and archived by the

system. The data are input to a series of commercially available applications (HEC-1F, PRECIP) to determine actual runoff hydrographs and to forecast future runoff from the system. These data are then used as boundary conditions in UNET which simulates one dimensional unsteady flow.

The Sacramento County Flood Warning Decision Support System therefore uses several commercially available software packages in a 'tightly integrated manner' to increase the warning time of flood events. To manage these components a graphical user interface (GUI) was developed which manages the execution of input to and visualisation of results from the program. This is the same concept as used in the GUI developed for the Orange River Decision Support System. The GUI reduces the effort required to learn the use of the application and reduces the likelihood of human errors. At the same time though human intervention and input is also allowed so that the system can be controlled when necessary.

Detering et al (1996) discuss the necessity of including hydraulic simulations in the closed loop control concept used in the regulation of water levels for navigable rivers and power generation. They state that the usual approach to learning about the dynamic behaviours of a controlled system is to measure the system reaction to erratically changing input values and to draw up a response function based on the recorded output. However, according to Detering, this system is not suitable to rivers because:

- Rivers are complex and non-linear systems because of the nature of unsteady open channel flow and their irregular geometry.
- It is seldom possible to generate the testing functions due to, for example, water availability, hampered navigation and bank protection
- The timeframe for the processes to take place are unlike that of other technological processes as the time water takes to flow from one control point to the next is a 'matter of hours', or in the case of the Orange River weeks. Using trial and error techniques would therefore take years to optimise the system.

For these reasons the authors suggest and explain the use of unsteady flow simulations in the operation of river systems.

In addition to discussing the use of unsteady modelling, closed loop control systems in the context of river management are also discussed. A common feature of all closed loop control principles is the requirement that the controlled system, eg

Vanderkloof Dam, be integrated with the controller, the decision support system, in a control loop. The controlled variable, the flow at the Orange River mouth enters the controller, which in turn influences the input variable of the controlled system, the releases from Vanderkloof Dam, which once again influences the controlled variable. Through this series of influences a change in the controlled variable triggers a change in the manipulated variable and thus a feedback into the process, closing the loop.

To set up the controller the system must be described mathematically, with the system linearised around a single operating point where the process takes place. This process is not applicable to rivers which are non-linear and have long delays between a change in the variable and the result of that change. The authors state that the standard procedures for adjusting the controller are therefore prevented and lead to an unfavourable and even unstable controller behaviour.

Feed forward controls were developed to remedy this. This involves an additional signal which is sent to the controller by the upstream reservoir and indicates the inflow into the reservoir to be controlled. This signal is transmitted to the controller with a delay and ideally causes a change in the discharge at the downstream control at the optimum time. If the feed forward control delay and the system reaction time are similar the control behaviour is acceptable, but if the two values are diverging the control becomes unstable. The time of flow depends on the amount of flow and therefore cannot be included in the control loop as individual values. The time of flow can only be determined by simulations which should therefore be included in the controller's configuration.

The authors also discuss fuzzy controllers based on fuzzy logic. This technique is not applicable to river systems because of their long reaction times.

Another method is model predictive control, which is usually used in the chemical industry. By means of an analytical model, using the present process status as initial conditions, different possible future process developments based on a set of variables are calculated faster than in real time. After various analyses have taken place a function chooses the variable development which resulted in the most favourable controlled variable development. Model predictive controls work in discrete time intervals with the first set of variables for each simulation being those recorded in reality. Model predictive controls have the advantage of not being limited by the number of variables in the system.

The decision support system developed for the Orange River was based on the model predictive control concept in that various simulations of future flows are conducted, using the actual conditions of the river as the initial conditions of the simulations, to determine the optimal result.

The operation of the reservoir Trier on the River Mosel in Germany is in several respects similar to the operation of the Orange River (Ackermann, 2000). The Trier Reservoir is the first storage structure on the Germany portion of the river after it crosses the French-German border. The releases from the river influence a series of 14 downstream reservoirs. This cascade of reservoirs is managed by an automatic control system that keeps the water elevation in each reservoir within a tolerance of 10cm for navigation purposes. The water elevations are highly sensitive to changing inflows and discharges.

Upstream of the reservoir Trier the river is controlled by French authorities and is used for hydroelectric power generation. The German control authorities have no control or input into the releases made by the French hydropower generation. The reservoir Trier is therefore used to attenuate the inflow waves to the German system of reservoirs. However the system is further complicated in that the tolerance of the level of the Trier reservoir itself is limited to 30cm, also for navigation purposes. Storage level regulation and flow variation attenuation are made more difficult because of the inflows from two rivers which flow directly into the reservoir Trier.

An optimisation program for river management (OPRiMa) has been implemented to operate the reservoir Trier within the specified tolerances without affecting the downstream automatic control.

Several aspects of this system discussed by Ackermann et al (2000) are of interest in terms of the operation of the Orange River and are described in the following paragraphs.

Firstly, the system dynamics are calculated using St Venants equations. However it was found that the solution of the equations were too slow for application in a real time optimisation exercise without some enhancements. A significant increase in simulation speed was obtained by linearising the (non-linear) dependency between the change of water elevation and the change of the release prior to each timestep. The hydrodynamics are then simulated using the St Venant equations.

Secondly, as in the Orange River, it was found that the simulation of the current conditions had to be updated before the forecasts could be made. The storage level at the current timestep did not always correspond to the actual storage level because of:

- inaccurate measurements
- unpredictable volumes flowing into and released from the reservoir when vessels pass through the locks.
- a delay from the time of calculation to implementation of a solution
- numerical inaccuracies inherent in the finite element model
- differences between the calculated releases and those actually made

Prior to calculating the future release scenario the simulated flow configuration throughout the system is updated by using the recorded flows for the previous 6 hours. The resultant simulated storage series is compared to the actual storage series, the inflow boundary conditions are modified and the flow is recalculated. The process is repeated until the simulated storage levels match the recorded levels.

This iterative correction to inflows is not necessary in the Orange River because the real time recorded data is input directly into the model, continuously correcting the simulation at several points on the river.

Thirdly the authors stress that the successful implementation of the operating system in the reservoir Trier was mainly due to the co-operation between the model developer, the operating personnel who would be using the system and the top management who could supply the data, equipment and permission for testing and implementation.

Summary

It was determined from the literature study that several basic concepts should be followed in order to implement a successful river management decision support system. These are:

- Due to the relatively slow reaction times and non-linear nature of open channel flow it is necessary to include flow simulation routines in the decision support system.
- The system variables should be regularly corrected to the actual conditions in the system to account for the differences in the modelled and actual flows.
- Good co-operation between all the involved parties is a requirement.

- A user friendly environment should be developed to facilitate the exchange of data and the seamless interfacing between the components of the system.
- Simple data structures should be developed.
- There should be transparency with regard to the system's functionality – it should not act as a black box.
- User intervention is a requirement. The user must be able to check the solutions and override the system if necessary.
- The opportunity for human error should be minimised.

In the next section the literature review on hydraulic modelling is discussed.

1.6.2 Hydraulic Modelling

The use of the implicit 4 point Priessman Scheme to model open channel flow is not a new development and the literature on it is extensive. The candidate chose to study the subject as described by Chow et al (1998) and the theory is covered in Chapters 2 and 3.

The literature review on hydraulic modelling was focussed on the interesting problems which were found when modelling the Orange River, including the stability of the four-point implicit scheme when modelling the very steep hydrographs released from Vanderkloof Dam.

Meselhe and Holly (1997) discuss the limitations of the Priessman implicit four point weighted finite element scheme used to solve the St Venant equations of flow. They show that the Priessman scheme results in a system of $2N-2$ equations and $2N$ unknowns. The Priessman scheme can therefore only be solved if there are 2 boundary conditions. In pure subcritical flow there are 2 boundary conditions. 1 at the upstream end and another at the downstream end. In pure supercritical flow there are 2 boundary conditions at the upstream end and none at the downstream end. Therefore the Priessman scheme can be applied to both these flow regimes. The problem comes in when there is a flow regime with both super- and subcritical flow, when the number of boundary conditions varies. For example consider supercritical flow that becomes subcritical through a hydraulic jump. Two boundary conditions must be established at the upstream end and one at the downstream end, giving a total of 3 boundary conditions. On the other hand consider upstream flow in a channel which becomes supercritical after the slope steepens. In this case there is only one boundary condition required at the upstream end. The Priessman scheme can therefore not be applied to either of these flow regimes.

FLDWAV is a generalised channel flood routing model developed by the United States National Weather Service. It is based on the four-point implicit finite difference solution of the St Venant equations. As described in the previous paragraph the four-point implicit scheme was found to have numerical stability problems when the flow changed from sub-critical to super-critical and visa versa, known as mixed flow regimes (Jin and Fread, 1997). A technique has been developed to model these mixed flow regimes in FLDWAV. This involves dividing the reach at each time step into a series of sub-critical and super critical sub-reaches and computing each subreach separately using the appropriate boundary conditions. The supercritical flows are solved in a downstream direction whereas the sub-critical flows are solved in a “double sweeping” process, first upstream to downstream followed by downstream to upstream. This process was successful where the control points are easy to determine, such as where the channel slope changes abruptly from subcritical to supercritical or supercritical to subcritical.

However, it was found that the four-point explicit scheme with the mixed flow routing method still had stability problems when modelling instantaneous or near instantaneous dam break induced flood waves and also for flow regimes bordering on critical flow (Jin and Fread, 1997).

A characteristic based, upwind, explicit scheme for the conservation form of the complete St Venant equations for nonprismatic channels was developed. It has the ability to model flows with strong shocks (dam breaks) or mixed super- and subcritical flows. It can also be used for modelling natural rivers with non-prismatic channels, off channel storage, abrupt cross sectional changes and various internal and external boundaries.

There are various trade-offs for both the explicit and implicit schemes. The implicit schemes are unconditionally stable for subcritical flow. They are also generally more accurate than the explicit schemes. However there are instability problems when modelling mixed flow regimes and dam break problems. On the other hand the stability of the explicit schemes limit the choice of timestep to that defined by the Courant condition. It has been found (Jin and Fread, 1997) that the explicit scheme requires a smaller computational timestep than the implicit scheme to obtain similar accuracy when modelling slowly rising flood waves, while it is equally fast or faster than the implicit scheme when modelling fast rising flood waves. Since fast rising waves tend to attenuate quickly the use of the explicit scheme for the whole reach

would be inefficient. In addition Jin and Fread have found that the explicit scheme is less accurate than the implicit scheme in a wide variety of conditions, especially when modelling flows in channels with very nonprismatic features or large changes in slope. A smaller distance step for the explicit scheme, with a resultant smaller timestep and longer computational time is often needed to produce the same accuracy as the implicit scheme.

To take advantage of both of these schemes the implicit-explicit multiple dynamic routing algorithm was incorporated into FLDWAV. The reach is divided into a number of subreaches and either the explicit or implicit scheme is applied to each reach, depending on the local flow conditions. The explicit subreaches will have several computational timesteps for the implicit subreaches' one timestep so that the Courant condition can be met in the explicit reach. The upstream subreach is computed first and a computed rating curve is used as the downstream boundary condition. The upstream reach is computed for one implicit timestep and the downstream subreach uses the computed hydrograph as its upstream boundary condition.

This mixed routing method could have been useful in modelling the steep hydrographs released from Vanderkloof Dam. However, the use of the adaptive timestepping routine largely overcame the stability versus computational time problems (See Section 5). Mixed flow regimes must also occur at in the river, but the level of accuracy of the simulations at these positions did not warrant any further investigation.

Venutelli (2002) also investigated the stability and accuracy of weighted four point implicit finite difference schemes for open channel flow. The main purpose of his paper was to present a complete analysis of Priessman's general schemes which have been applied to St Venant's flow equations. He has proposed some general relations using Fourier's linear theory which can be used to evaluate the characteristics of stability, dissipation and dispersion of various schemes in different flow conditions. In particular instabilities were analysed that are due to an increasing frictional resistance term which causes progressive accumulation of dispersion errors, seen as oscillations in the results. Venutelli's analyses determined the most suitable values for the time weighting coefficients which are able to produce the necessary dissipation for stabilizing the numerical solution.

Summary

The literature review on hydraulic modelling was focussed on the limitations of the Priessman four point scheme as a solution to the St Venant equations of flow. The scheme is not applicable to mixed flow regimes and other models should then be applied.

The stability and accuracy of the implicit and explicit schemes was also explored and the advantages and disadvantages of the schemes were determined.

1.6.3 River Model Calibration

Two interesting papers on river model calibration are worth mentioning. The first highlights the risk of the dependence of accuracy on the number of gauged events used in the calibration of the model (Khatibi, 2001). This highlights the fact that the limited recorded data, which could be used for the calibration of the model in this study, is a serious problem.

The second deals with the optimal estimation of channel roughness, an important aspect in any study of open channel flow. Many empirical procedures have been suggested in the past for estimating the value of Manning's n . It is possible to estimate Manning's n from field measurements of discharge and depths using optimisation techniques. In most proposed procedures it is necessary to solve the governing equations of flow in a separate solver outside of the optimisation model. This means that the solver must be run a number of times in order to determine the objective function and the influence co-efficient matrix for the optimisation model.

In the study conducted by Ramesh et al (Ramesh, 2000) a new approach was proposed to estimate channel roughness co-efficients by directly embedding the finite difference approximations of the governing equations of flow into a non-linear optimisation model as equality constraints. An algorithm is then used to minimise the non-linear objective function based on least square error criterion. This approach would be worth considering if an automatic calibration routine were to be developed.

Summary

From the literature review on river model calibration it was established that some work has been performed on automatic calibration routines. It was also confirmed that the level of confidence of the simulated results is dependent on the extent to which the model was calibrated against recorded data.

1.7 LAYOUT OF DISSERTATION

This dissertation documents the methods used to develop the Orange River decision support system and to record the results of the research conducted during the study. The layout is described below.

Before attempting to make the required changes to a hydraulic modelling package the theoretical concepts of open channel flow had to be understood. In Chapter 2 the St Venant equations of open channel flow are derived from the Reynolds Transport Theorem.

In Chapter 3 various flow routing methods are summarised and discussed in terms of their applicability to the Orange River. As it was decided that a full dynamic model should be used it was necessary to understand how the St Venant equations are solved. This is summarised in Chapter 4.

The development of the real time and adaptive timestepping units are described in Chapter 5.

The design of the decision support system is explained in Chapter 6 and more detail on the Graphical User Interface is give in Chapter 7.

The telemetry network from which the real time data is obtained is described in Chapter 8.

The preparation and calibration of the hydraulic model are described in Chapter 9. Finally the decision support tool was assessed in Chapter 10. The conclusions made and recommendation arising from this study are summarised in Chapter 11.

2 THEORETICAL BACKGROUND

Before attempting to develop the real time unit for the hydraulic modelling software, the theoretical background to open channel flow was researched. In this chapter the St Venant equations of flow are derived using the Reynolds' transport theorem. This material is adapted from Chow et al (1998).

2.1 REYNOLDS TRANSPORT THEOREM

The Reynolds transport theorem states:

The total rate of change of an extensive property of fluid is equal to the rate of change of the extensive property stored in the control volume plus the net outflow of the extensive property through the control surface.

In the Reynolds transport theorem, the physical laws which are usually applied to discrete masses are adapted to apply to fluid flowing continuously through a control volume. Two types of fluid properties are defined for the Reynolds transport theorem:

- Intensive properties, which are independent of the mass present in the control volume
- Extensive properties, which are dependent on the mass present in the control volume

For any extensive property B there is a corresponding intensive property β which is defined as the quantity of B per unit mass of fluid:

$$\beta = \frac{dB}{dm} \quad (2.1)$$

The extensive and intensive properties can be scalar or vector properties, depending on the definition of the property under consideration.

The Reynolds transport theorem is based on the time rate of change of a property and the net amount of that property flowing into and out of the control volume. Consider Figure 2.1. The control volume is depicted by the shaded area and is bounded by the solid lines. The fluid flows through this fixed control volume from left to right. After a small time interval Δt the fluid occupies the space indicated by the dashed lines. We can therefore identify three regions:

- Region I, which is occupied at time t but not at time $t+\Delta t$
- Region II, which is occupied at time t and $t+\Delta t$
- Region III which is occupied only at time $t+\Delta t$

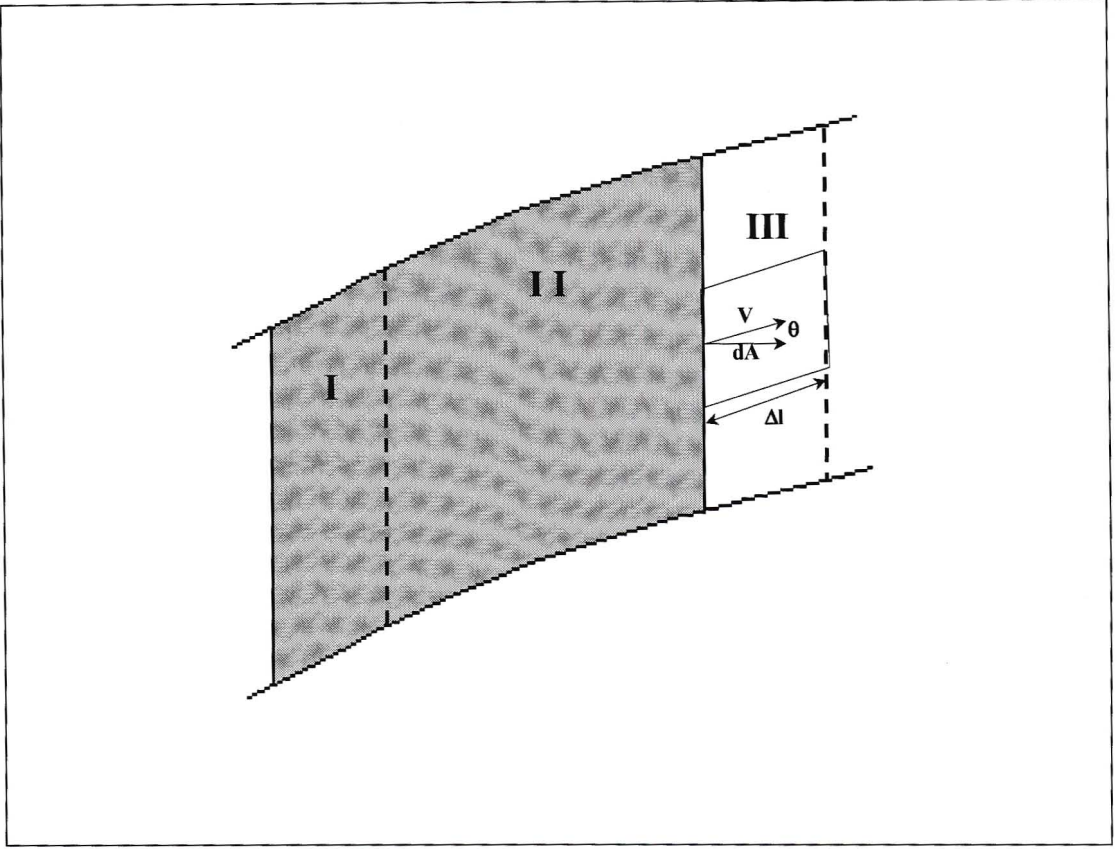


Figure 2.1: Control Volume of fluid

Within the control volume the volume of fluid is dV . If the density of the fluid is ρ , then the mass of the element is $dm = \rho dV$. The amount of extensive property B in the fluid element is $dB = \beta dm = \beta \rho dV$. The total amount of any extensive property is the integral of the elements in the total volume:

$$B = \iiint \beta \rho dV \quad (2.2)$$

where \iiint is the integral over the volume.

For the shaded control volume originally in the control volume the time rate of change of the extensive property is:

$$\begin{aligned} \frac{dB}{dt} &= \lim_{\Delta t \rightarrow 0} \frac{t}{\Delta t} \left[(B_{II} + B_{III})_{t+\Delta t} - (B_I + B_{II})_t \right] \\ \frac{dB}{dt} &= \lim_{\Delta t \rightarrow 0} \frac{t}{\Delta t} \left[(B_{II})_{t+\Delta t} - (B_{II})_t + (B_{III})_{t+\Delta t} - (B_I)_t \right] \end{aligned} \quad (2.3)$$

As Δt approaches 0 region II becomes the control volume, therefore

$$\lim_{\Delta t \rightarrow 0} \frac{t}{\Delta t} \left[(B_{II})_{t+\Delta t} - (B_{II})_t \right] = \frac{d}{dt} \iiint \beta \rho dV \quad (2.4)$$

$\lim_{\Delta t \rightarrow 0} \frac{1}{\Delta t} \left[(B_{III})_{t+\Delta t} - (B_I)_t \right]$ represents the flow of the extensive property carried by the fluid across the control surface.

Figure 2.1 shows a close up of the outflow from the control volume into region III. An element in the area through which the fluid flows is represented by dA . The volume of water which flows through dA in time Δt is $d\forall$. The length of this tube is represented by Δl , with $\Delta l = V\Delta t$, where V is the velocity of the fluid. The volume of the fluid is therefore $d\forall = \Delta l \cos\theta dA$, where θ is the angle between the direction perpendicular to the outflow surface and the velocity vector \mathbf{V} . The total amount of extensive property in Region III can be found by integrating over the area. Therefore:

$$\lim_{\Delta t \rightarrow 0} \frac{1}{\Delta t} \left[(B_{III})_{t+\Delta t} \right] = \lim_{\Delta t \rightarrow 0} \frac{1}{\Delta t} \iint_{III} \beta \rho \Delta l \cos\theta dA \quad (2.5)$$

where the double integral represents the integral over the surface dA .

As Δt approaches 0, $\Delta l/\Delta t$ approaches the magnitude of the velocity V . Let the vector \mathbf{dA} be the vector with magnitude dA and direction normal to the area dA , pointing out of the control volume. Then the term $V \cos\theta dA$ is the vector dot product $\mathbf{V} \cdot \mathbf{dA}$. Equation 2.5 can then be re-written as:

$$\lim_{\Delta t \rightarrow 0} \frac{1}{\Delta t} \left[(B_{III})_{t+\Delta t} \right] = \iint_{III} \beta \rho \mathbf{V} \cdot \mathbf{dA} \quad (2.6)$$

Similarly it may be determined that

$$\begin{aligned} \lim_{\Delta t \rightarrow 0} \frac{1}{\Delta t} \left[(B_I)_t \right] &= \lim_{\Delta t \rightarrow 0} \frac{1}{\Delta t} \iint \beta \rho \Delta l \cos(180 - \theta) dA \\ &= - \iint \beta \rho \mathbf{V} \cdot \mathbf{dA} \end{aligned} \quad (2.7)$$

Substituting equations 2.5 to 2.7 into 2.3 gives

$$\frac{dB}{dt} = \frac{d}{dt} \iiint_{cv} \beta \rho d\forall + \iint_{III} \beta \rho \mathbf{V} \cdot \mathbf{dA} + \iint_I \beta \rho \mathbf{V} \cdot \mathbf{dA} \quad (2.8)$$

It should be noted that for fluid entering the control volume, the angle between the velocity vector pointing into the control volume and the area vector pointing out of the control volume, is in the range $90^\circ \leq \theta \leq 270^\circ$ for which $\cos\theta$ is negative. Therefore $\mathbf{V} \cdot \mathbf{dA}$ is always negative for inflow. Similarly $\mathbf{V} \cdot \mathbf{dA}$ is always positive for outflow. At the bounding streamlines \mathbf{V} and \mathbf{dA} are perpendicular which gives $\mathbf{V} \cdot \mathbf{dA} = 0$. The integrals over Region I and Region II can therefore be replaced by a single integral over the entire control surface, representing the net outflow from the control volume:

$$\frac{dB}{dt} = \frac{d}{dt} \iiint_{CV} \beta \rho dV + \iint_{CS} \beta \rho \mathbf{V} \cdot d\mathbf{A} \quad (2.9)$$

Rate of change Rate of change within the control volume net outflow through the control surface

This is the governing equation of the Reynolds' transport theorem.

2.2 THE ST VENANT EQUATIONS

The St Venant equations describe one dimensional unsteady open channel flow. In order to derive these equations the following assumptions must be made:

- The flow is one dimensional, with depth and velocity being constant across the channel cross section
- It is assumed that the hydrostatic pressure governs the flow, with vertical accelerations being negligible
- The longitudinal axis of the channel is approximately a horizontal straight line
- The channel cross sections are not altered by the effects of scour and deposition
- Resistance coefficients for steady uniform flow are applicable (i.e. Manning's and Chezy's equations)
- The fluid has a constant density throughout.

The St Venant equations are derived by considering the conservation of mass (also known as continuity) and the conservation of momentum.

The continuity equation

Consider the Reynolds equation with mass being the extensive property under consideration:

$$B = m$$

$$\beta = \frac{dB}{dm} = 1 \quad (2.10)$$

By the law of conservation of mass (mass cannot be created or destroyed):

$$\frac{dB}{dt} = \frac{dm}{dt} = 0 \quad (2.11)$$

Substituting these equations into the Reynolds' equation (2.9) gives:

$$\frac{dB}{dt} = \frac{d}{dt} \iiint_{CV} \rho dV + \iint_{CS} \rho \mathbf{V} \cdot d\mathbf{A} = 0 \quad (2.12)$$

The first integral term represents the time rate of change of the storage within the control volume. The second integral term, the net outflow of mass, can be represented by the difference between the outflow and inflow of mass. Assuming that the density, ρ , is constant, it can be eliminated:

$$\begin{aligned}
\frac{dB}{dt} &= 0 \\
\frac{d}{dt} \iiint_{CS} dV + \iint_{CV} \mathbf{V} \cdot d\mathbf{A} &= 0 \\
\frac{dS}{dt} + \iint_{outlet} \mathbf{V} \cdot d\mathbf{A} + \iint_{inlet} \mathbf{V} \cdot d\mathbf{A} &= 0
\end{aligned} \tag{2.13}$$

This is the conventional form of the integral equation of continuity for an unsteady flow with constant density.

Consider an elemental control volume of length dx . The inflow to the control volume is a quantity ρQ entering the upstream end and the lateral inflow per unit length, ρq , entering as distributed inflow along the length of the channel. The total mass inflow to the control volume is therefore:

$$\iint_{inlet} \rho \mathbf{V} \cdot d\mathbf{A} = -\rho Q - \rho q dx \tag{2.14}$$

The outflow from the control volume is the inflow plus the rate of change of the flow over the channel length using the first two terms of a Taylor series expansion:

$$\iint_{outlet} \rho \mathbf{V} \cdot d\mathbf{A} = \rho Q + \rho \frac{\partial Q}{\partial x} dx \tag{2.15}$$

inf low change of flow

The volume of the channel element is $A dx$, where A is the average cross sectional area. The rate of change of mass stored in the control volume is:

$$\frac{d}{dt} \iiint_{CV} \rho dV = \frac{\partial(\rho A dx)}{\partial t} \tag{2.16}$$

The partial derivative is used because the control volume is defined to be fixed in size.

Substituting 2.14 to 2.16 into 2.12 gives:

$$\frac{\partial(\rho A dx)}{\partial t} - \rho(Q + q dx) + \rho(Q + \frac{\partial Q}{\partial x} dx) = 0$$

which after cancellation and division by ρdx becomes

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q = 0 \tag{2.17}$$

assuming that the density is constant.

The momentum equation

Consider the Reynolds transport theorem applied to momentum. The extensive property is the momentum in the control volume:

$$\begin{aligned}\mathbf{B} &= m\mathbf{V} \\ \text{so that} \\ \frac{d\mathbf{B}}{dm} &= \mathbf{V}\end{aligned}\tag{2.18}$$

By Newton's second law the time rate of change of momentum is equal to the net forces applied to the object in the direction of the change in momentum:

$$\frac{d\mathbf{B}}{dt} = \frac{d(m\mathbf{V})}{dt} = \Sigma \mathbf{F}\tag{2.19}$$

Substituting this into the Reynolds' transport theorem:

$$\frac{d\mathbf{B}}{dt} = \frac{d}{dt} \iiint_{cv} \mathbf{V} \rho dV + \iint_{cs} \mathbf{V} \rho \mathbf{V} \cdot d\mathbf{A} = \Sigma \mathbf{F}\tag{2.20}$$

This states that the rate of change of momentum stored within the control volume plus the net outflow of momentum from the control volume is equal to the sum of the forces applied to the control volume. We will consider each of these terms in turn.

Momentum

The mass inflow rate to the control volume is the sum of the inflow at the upstream end plus the distributed inflow along the length of the channel. The corresponding momentum is therefore:

$$\iint_{inlet} \mathbf{V} \rho \mathbf{V} \cdot d\mathbf{A} = -\rho(\beta VQ + \beta v_x q dx)\tag{2.21}$$

where β is the momentum correction factor or Boussinesq factor and v_x is the component of the velocity of the lateral inflow in the direction x .

The momentum leaving the control volume is:

$$\iint_{outlet} \mathbf{V} \rho \mathbf{V} \cdot d\mathbf{A} = \rho(\beta VQ + \frac{\partial(\beta VQ)}{\partial x} dx)\tag{2.22}$$

The volume of the control volume is $A dx$. Its momentum is therefore $\rho A dx V$. Therefore the time rate of change of the stored momentum is:

$$\begin{aligned}\frac{d}{dt} \iiint_{cv} \mathbf{V} \rho dV &= \rho \frac{\partial AV}{\partial t} dx \\ &= \rho \frac{\partial Q}{\partial t} dx\end{aligned}\tag{2.23}$$

Substituting 2.21 to 2.23 in 2.20:

$$\begin{aligned}\Sigma \mathbf{F} &= \rho \frac{\partial Q}{\partial t} dx + \rho(\beta VQ + \frac{\partial(\beta VQ)}{\partial x} dx) - \rho(\beta VQ + \beta v_x q dx) \\ &= \rho \frac{\partial Q}{\partial t} dx + \frac{\partial(\beta VQ)}{\partial x} dx - \beta v_x q dx\end{aligned}\quad (2.24)$$

Forces

There are five forces acting on the control volume in the x direction. These are defined below.

- The gravity force. The volume of fluid in the control volume is Adx . Its weight is $\rho g Adx$. The component of gravity force is therefore:

$$F_g = \rho g Adx \sin \theta = \rho g A S_o dx \quad (2.25)$$

where θ is the angle of the channel slope to the horizontal.

- The friction force. The shear stress on the wetted perimeter, P , of the channel is $-\tau_0 P dx$. But $\tau_0 = \gamma R S_f = \rho g A / P S_f$ under the assumption that the friction slope can be computed from the uniform flow equation. The friction force is therefore:

$$F_f = -\rho g A S_f dx \quad (2.26)$$

where S_f is derived from resistance equations such as the Manning or Chezy equations.

- The contraction or expansion force. Abrupt changes in the channel cross section cause energy losses due to the creation of turbulent eddies in the flow. These eddies are related to the changes in velocity head, $V^2/2g$.

$$F_e = -\rho g A S_e dx \quad (2.27)$$

where $S_e = \frac{K_e}{2g} \frac{\partial \left(\frac{Q}{A} \right)^2}{\partial x}$, with K_e , the expansion/contraction co-efficient,

negative for channel expansion and positive for contraction.

- Wind Shear force. This is caused by the frictional force of wind against the surface of the fluid. It is defined as:

$$F_w = -W_f B \rho dx \quad (2.28)$$

where τ_w is the wind shear stress, defined as

$$W_f = \frac{C_f |V_r| V_r}{2} \quad (2.29)$$

and V_r is the velocity of the fluid relative to the air. τ_w is defined as acting opposite to the direction of flow:

$$V_r = Q/A - V_w \cos \omega \quad (2.30)$$

with V_w being the velocity of the air at an angle ω to the flow direction.

- **Pressure Force.** The unbalanced pressure force in the direction of flow is the result of the hydrostatic force on the upstream end of the control volume, the hydrostatic force on the downstream side and the component of the pressure force exerted by the banks of the channel:

$$F_p = F_{u/s} - F_{d/s} + F_{bank} \quad (2.31)$$

Consider an element of thickness dw at a depth from the surface of $y-w$ with a width b . The hydrostatic pressure on the element is $\rho g(y-w)b dw$. The total hydrostatic force on the upstream end of the control volume:

$$F_{u/s} = \int_0^y \rho g(y-w)b dw \quad (2.32)$$

The hydrostatic force on the downstream end:

$$F_{d/s} = F_{u/s} + \frac{\partial F_{u/s}}{\partial x} dx$$

and using the Liebnitz rule for differentiation of an integral where the limits are functions of the variable of integration:

$$\begin{aligned} \frac{\partial F_{u/s}}{\partial x} &= \int_0^y \rho g \frac{\partial y}{\partial x} b dw + \int_0^y \rho g(y-w) \frac{\partial b}{\partial x} dw \\ \text{because } A &= \int_0^y b dw, \\ \frac{\partial F_{u/s}}{\partial x} &= \rho g A \frac{\partial y}{\partial x} + \int_0^y \rho g(y-w) \frac{\partial b}{\partial x} dw \end{aligned} \quad (2.33)$$

The compound force due to the changing width of the banks is related to the rate of change of the width of the channel:

$$F_{banks} = \left[\int_0^y \rho g(y-w) \frac{\partial b}{\partial x} dw \right] dx \quad (2.34)$$

Therefore:

$$\begin{aligned} F_p &= F_{u/s} - (F_{u/s} + \frac{\partial F_{u/s}}{\partial x} dx) + F_{banks} \\ &= - \frac{\partial F_{u/s}}{\partial x} dx + F_{banks} \\ &= - \left[\rho g A \frac{\partial y}{\partial x} + \int_0^y \rho g(y-w) \frac{\partial b}{\partial x} dw \right] dx + F_{banks} \\ &= - \rho g A \frac{\partial y}{\partial x} dx \end{aligned} \quad (2.35)$$

The momentum equation (2.24) in its full form after division by dx is therefore:

$$\rho g A S_0 - \rho g A S_f - \rho g A S_e - W_f B \rho - \rho g A \frac{\partial y}{\partial x} = -\rho \left[\beta v_x q - \frac{\partial(\beta V Q)}{\partial x} \right] + \rho \frac{\partial Q}{\partial t} \quad (2.36)$$

which simplifies to:

$$\frac{\partial Q}{\partial t} + \frac{\partial(\beta V Q)}{\partial x} + g A \left(\frac{\partial y}{\partial x} - S_0 + S_f + S_e \right) - \beta v_x q + W_f B = 0$$

The pair of St Venant equations are therefore the continuity and momentum equations:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q = 0 \quad (2.37)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial(\beta V Q)}{\partial x} + g A \left(\frac{\partial y}{\partial x} - S_0 + S_f + S_e \right) - \beta v_x q + W_f B = 0 \quad (2.38)$$

2.3 SUMMARY

In this chapter the St Venant equations of flow were derived from the Reynolds Transport Theorem. The following chapter discusses various distributed flow routing methods, which are based on the St Venant equations and compares them to lumped flow routing methods in terms of their applicability to the Orange River before the choice of model used to simulate the Orange River is justified.

3 HYDRAULIC MODELS

The unsteady flow of water in an open channel can be described by various mathematical models. These models can be either lumped flow routing models, which are based solely on the principle of continuity of mass, or distributed flow routing models which are described by the St Venant equations of flow which were derived in Chapter 2. The theoretical principles of these models are briefly described below (The material is adapted from Chow et al (1998)) and their applicability to the Orange River is discussed.

3.1 LUMPED FLOW ROUTING

Lumped flow routing is based on the continuity equation in which the change in time of the volume of water stored within a river section, $\frac{dS}{dt}$, is the difference between the inflow to and outflow from the section. (See Equation 2.13)

$$\frac{dS}{dt} = I(t) - Q(t) \quad (3.1)$$

or by discretising

$$S_2 - S_1 = \frac{I_2 + I_1}{2} \Delta t + \frac{Q_2 + Q_1}{2} \Delta t \quad (3.1a)$$

where the subscripts 1 and 2 refer to the variable at time = t and time = $t + \Delta t$ and S is the storage, I the inflow to the system and Q the outflow from the system.

If the inflow at time = $t + \Delta t$ is known, the unknown variables in equation 3.1a are S_2 and Q_2 . In order to solve these unknown variables, a second function relating storage to inflow and outflow is required. In general the storage is a function of the inflow and outflow from the system and their rate of change with regard to time. The storage function which is actually applied is dependent on the nature of the river which is to be modelled. Three methods of lumped flow routing are described in the following sections.

3.1.1 Level Pool Routing

In level pool routing it is assumed that the water surface level remains horizontal. Storage is assumed to be a function of the water surface level and the outflow is controlled by a structure with the flow being a function of water surface level. The two equations, both functions of water level, can be combined so that the storage is expressed as a function of outflow, Q . The equations used in this method are generally expressed as:

$$\frac{dS}{dt} = I(t) - Q(t) \quad (3.2)$$

$$S = Q(t) \quad (3.3)$$

This method is applicable only where the water surface remains horizontal such as in large pools or reservoirs which are comparatively wider and deeper than they are long in the direction of flow. This method is therefore not applicable to the Orange River and it will not be discussed further in this report.

3.1.2 The Muskingum Method

This method is commonly used to describe a variable discharge-storage relationship and is included in the ISIS Flow software package. The storage in the river section can be conceived to comprise of two parts, the prism storage beneath the line parallel to the river bed and the wedge storage which is the volume of water above the parallel line and the water surface. The storage in the prism is equal to KQ , where K is a storage constant. The storage in the wedge is equal to $KX(I-Q)$, where the magnitude of X determines the importance of the difference between the inflow and outflow and is in the range $0 \leq X \leq 0.5$. The storage can therefore related to inflow and outflow by:

$$S = KQ + KX(I - Q) \quad (3.4)$$

Or rearranged to:

$$S = K [XI + (1-X) Q] \quad (3.5)$$

Substituting 3.5 in 3.1a and rearranging gives:

$$Q_2 = C_1 I_1 + C_2 I_2 + C_3 Q_1$$

where

$$\begin{aligned} C_1 &= \frac{\Delta t - 2KX}{2K(1-X) + \Delta t} \\ C_2 &= \frac{\Delta t + 2KX}{2K(1-X) + \Delta t} \\ C_3 &= \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t} \end{aligned} \quad (3.6)$$

It should be noted the $C_1 + C_2 + C_3 = 1$ and C_1, C_2, C_3 should be greater than 0, which places restrictions on Δt . K is the time of travel of the hydrograph through the reach and X controls the shape of the hydrograph from translation to diffusion.

3.1.3 The Linear Reservoir Model

The linear reservoir (or tank) model is one where the storage is linearly related to its outflow:

$$S = kQ \quad (3.7)$$

where k is the storage constant with the dimension of time and is closely related to the Muskingum K . A single linear reservoir is therefore a special case of the Muskingum model where $X = 0$.

Substituting this equation into equation 3.1a and re-arranging gives

$$Q_2 = C_1 I_1 + C_2 I_2 + C_3 Q_1$$

where

$$\begin{aligned} C_1 &= 1 - \frac{k(1 - C_3)}{\Delta t} \\ C_2 &= \frac{k(1 - C_3)}{\Delta t} - C_3 \\ C_3 &= e^{-\frac{\Delta t}{k}} \end{aligned} \quad (3.8)$$

The impulse response function describes the unique response of a system to a unit impulse. If a system receives an instantaneous unit input at time τ the response at some later time t is $u(t - \tau)$. Consider that the continuous input to a system is a sequence of infinitesimal impulses, then $I(t)d\tau$ is the volume input to the system, during $d\tau$ where $I(t)$ is the rate of input. The response to the complete input to the system is therefore:

$$Q(t) = \int_0^t I(\tau)u(t - \tau)d\tau \quad (3.9)$$

This expression is known as the convolution integral.

It can be shown that for a linear reservoir the impulse response function is:

$$u(t) = \frac{1}{k} e^{-\frac{t}{k}} \quad (3.10)$$

A river reach may be represented by a series of linear reservoirs with the outflow from one reservoir being the inflow to the next with each of these reservoirs having the same storage constant k . Consider a system of two reservoirs. The outflow from the first reservoir $q_1 = u(t) = \frac{1}{k} e^{-\frac{t}{k}}$ and is the inflow $I(t)$ to the next reservoir.

Therefore the outflow from the second reservoir is then

$$\begin{aligned} q_2(t) &= \int_0^t I(\tau)u(t - \tau)d\tau \\ &= \int_0^t \left(\frac{1}{k}\right) e^{-\frac{\tau}{k}} \frac{1}{k} e^{-\frac{(t-\tau)}{k}} d\tau \\ &= \frac{t}{k^2} e^{-\frac{t}{k}} \end{aligned} \quad (3.11)$$

Developing this concept further it can be shown that the outflow from the n th reservoir is a gamma function:

$$q_n(t) = \frac{1}{k(n-1)!} \left(\frac{t}{k} \right)^{n-1} e^{-\frac{t}{k}} \quad (3.12)$$

Nash proved that the values of k and n can be determined using the first and second moments, M_1 and M_2 , of the inflow hydrograph about the origin $t=0$:

$$\begin{aligned} M_1 &= nk \\ M_2 &= n(n+1)k^2 \end{aligned} \quad (3.13)$$

The two equations can be combined to solve for n and k

$$\begin{aligned} M_{Q_1} - M_{I_1} &= nk \\ M_{Q_2} - M_{I_2} &= n(n+k)k^2 + 2nkM_{I_1} \end{aligned} \quad (3.14)$$

where M refers to the moment, Q refers to the outflow, I to the inflow and the subscript numbers to the first or second moment about the origin. Note that k (for the single tank) or M_1 for n tanks is the time between the centroids of the inflow and outflow hydrographs in the reach, so is in fact the time of travel through the reach, the analogue of K in the Muskingham equation. Note these times of travel are independent of flow rate.

3.2 DISTRIBUTED FLOW ROUTING

The St Venant equations describe one dimensional unsteady open channel flow. These equations are applicable only under certain conditions which are listed in Chapter 2.2

Whereas the lumped flow routing methods are based on the continuity equation and an equation relating storage and flow (which replaces the momentum equation), the distributed flow routing methods are based on the St Venant equations, which are based on the principles of conservation of mass and conservation of momentum. These equations are derived in Chapter 2 using Reynolds Transport theorem. Neglecting wind shear and eddy losses these equations (2.37 and 2.38) become:

The Continuity Equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \quad (3.15)$$

The Momentum equation

$$\underbrace{\frac{1}{A} \frac{\partial Q}{\partial t}}_{\text{Local acceleration term}} + \underbrace{\frac{1}{A} \frac{\partial}{\partial x} \left(\frac{\beta Q^2}{A} \right)}_{\text{Convective acceleration term}} + \underbrace{g \frac{\partial y}{\partial x}}_{\text{Pressure force term}} - \underbrace{g(S_0 - S_f)}_{\substack{\text{Gravity force term} \\ \text{Friction force term}}} = 0 \quad (3.16)$$

The *local acceleration* term describes the change in velocity with time. The *convective acceleration* term describes the change in momentum due to the change in velocity over the channel length. The *pressure force* term describes the gradient in the hydrostatic force along the length of the channel. The *gravity force* is proportional to the change in bedslope S_0 . The *friction force* is proportional to the friction slope S_f and is assumed to be inversely proportional to the square of the channel conveyance, K . Note that K in equation 3.17 refers to the conveyance whereas in Section 3.2 it refers to the storage constants in the lumped flow routing methods.

$$S_f = \frac{Q|Q|}{K^2}, \text{ where} \quad (3.17)$$

$$K^2 = \frac{A^2 R^{\frac{2}{3}}}{n^2} \text{ and } R = \frac{A}{P}$$

R is the hydraulic radius, P is the length of the wetted perimeter and n is Manning's roughness coefficient.

There are several variations of the distributed flow routing methods. These models are based on the St Venant equations. They are referred to as distributed models because the flow is described as a function of both space and time. These distributed models differ with respect to the terms of the St Venant's momentum equations which are neglected. They include:

- the kinematic wave model, which neglects the local acceleration, convective acceleration and the pressure force term. This model is only applicable if it can be assumed that $S_0 = S_f$
- the diffusion wave model neglects the local acceleration and convective acceleration terms
- the dynamic wave model does not neglect any terms

A wave is defined as a variation in depth or flow which propagates through the channel. Changes in flow rate or flow depth are waves. The velocity with which a wave propagates relative to the local flow velocity is known as the wave's celerity.

3.2.1 The Kinematic Wave Model

The kinematic model is described by the following equations:

Continuity

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \quad (3.18)$$

Momentum

$$S_0 = S_f \quad (3.19)$$

Equation 3.19 can also be written as

$$A = \alpha Q^\beta \quad (3.20)$$

An example is the Manning's equation where

$$\begin{aligned} Q &= \frac{1}{n} A R^{\frac{2}{3}} S_0^{\frac{1}{2}} \\ &= \frac{1}{n P^{\frac{2}{3}}} S_0^{\frac{1}{2}} A^{\frac{5}{3}} \end{aligned}$$

Therefore

$$\begin{aligned} A &= \left(\frac{n P^{\frac{2}{3}}}{S_0^{\frac{1}{2}}} \right)^{\frac{3}{5}} Q^{\frac{3}{5}} \\ &= \alpha Q^\beta \end{aligned}$$

Differentiating A in 3.20 gives

$$\frac{\partial A}{\partial t} = \alpha \beta Q^{\beta-1} \frac{\partial Q}{\partial t} \quad (3.21)$$

Substituting 3.21 into 3.18 gives

$$\frac{\partial Q}{\partial x} + \alpha \beta Q^{\beta-1} \frac{\partial Q}{\partial t} = q \quad (3.22)$$

It can therefore be seen that kinematic waves are dependent on flow only. If the lateral inflow

$$q = \frac{dQ}{dx} \quad (3.23)$$

and

$$\alpha \beta Q^{\beta-1} = \frac{dt}{dx} \quad (3.24)$$

then substituting 3.24 into 3.22 gives

$$\frac{\partial Q}{\partial x} + \frac{\partial Q}{\partial t} \frac{dt}{dx} = \frac{dQ}{dx} \quad (3.25)$$

Differentiating 3.20 with respect to A gives

$$1 = \alpha \beta Q^{\beta-1} \frac{dQ}{dA} \quad (3.26)$$

$$\frac{dQ}{dA} = \frac{1}{\alpha \beta Q^{\beta-1}}$$

Comparing 3.24 and 3.26 it can be seen that

$$c_k = \frac{dQ}{dA} = \frac{dx}{dt} \quad (3.27)$$

If Mannings equation is used to describe the frictional slope

$$c_k = \frac{5Q}{3A} = \frac{5v}{3}$$

The kinematic celerity of the element of a wave at flow rate Q is described as c_k and has a constant velocity of $\frac{dx}{dt}$. The flow changes with distance only if there is lateral inflow, i.e. $q > 0$.

3.2.2 The Diffusion Wave Model

The Muskingum method (Equation 3.6) is an approximation of the kinematic wave. If K and X are defined as certain functions of flow, Q , and its kinematic celerity, c_k , the Muskingum equation is a simplified diffusion model. This model is referred to as the Muskingum-Cunge method and is defined as follows:

$$Q_2 = C_1 I_1 + C_2 I_2 + C_3 Q_1, \text{ where} \quad (3.28)$$

$$C_1 = \frac{\Delta t - 2KX}{2K(1-X) + \Delta t}$$

$$C_2 = \frac{\Delta t + 2KX}{2K(1-X) + \Delta t}$$

$$C_3 = \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t}$$

with:

$$K = \frac{\Delta x}{c_k} = \frac{\Delta x}{\frac{dQ}{dA}} \text{ and} \quad (3.29)$$

$$X = \frac{1}{2} \left(1 - \frac{Q}{B c_k S_0 \Delta x} \right)$$

where c_k is the kinematic celerity of the wave corresponding to the (varying) flow rate Q and the width of the water surface, B . For stability it is required that $0 \leq X \leq 0.5$.

3.2.3 The Dynamic Flow Model

The dynamic flow model is based on the full St Venant equations, incorporating all of the terms. The solution to the St Venant equations is described in Chapter 4. The St Venant equations are derived in Chapter 2.

A full discussion in the merits of the various flow models is now discussed in Section 3.3.

3.3 SELECTION OF A MODEL FOR THE ORANGE RIVER

The selection of a routing model for a particular application is influenced by the relative importance one places on (Fread, 1985):

- Model accuracy
- The accuracy required for the application
- The type and availability of the required data
- The available computational facilities
- The computational costs
- The extent of the flood wave information desired
- One's familiarity with the model
- The extent of documentation, range of applicability, and availability of a routing model
- The complexity of the mathematical formulation if the model has to be coded.
- One's capability and time available to develop a particular type of routing model

Kinematic wave routing can be used when the inertial and pressure forces can be ignored, when channel slopes are steep and the backwater effect is not important. A diffusion wave model can be used when the pressure forces become important but inertial forces are not important.

The releases from Vanderkloof Dam are of particular importance when choosing a model for the Orange River. The maximum releases from Vanderkloof Dam are in the order of $170\text{m}^3/\text{s}$ to $200\text{m}^3/\text{s}$ depending on the level in the dam. The valves through which the releases are controlled can be opened within 3 minutes. Consequently when the valves are opened a rapid change in flow and stage results, which propagates rapidly downstream. The model which is chosen must be able to handle these rapid changes. Discussions relating to the various types of models follow.

3.3.1 Lumped Flow Routing Models

The advantage of lumped flow routing models is that they are mathematically simple and are therefore computationally faster than the more complex dynamic models. A further advantage is that they require no data describing the modelled river and are therefore cheaper to set up than the dynamic models.

The first disadvantage of the lumped flow models, specifically the Muskingum and tank models, is that they have fixed travel times, as indicated at the end of Section 3.1.3. Records of flow times show that the travel time in the Orange River is strongly dependent on the flow rate, so for this reason lumped models are not appropriate here. With regard to a system of linear reservoirs Papastylianios (1997) showed that although Nash's method of determining n and k worked for a hypothetical channel section it was not applicable to a reach of the Orange River (Papastylianios, 1997). However, Papastylianios did show that the reach could be successfully modelled for a system of reservoirs where n and k were determined by calibration against recorded data. However, this method was only tested for two input hydrographs and the range of flows for which these values were applicable was not investigated.

Secondly, the backwater effects cannot be modelled in lumped flow routing models as the term for momentum is discarded. When there is a structure or a control point in a reach with sub-critical flow, the effect of the control on the flow of water propagates upstream. This effect is known as the backwater effect and is due to the change in the momentum of the flow caused by the control. Where this effect is significant, the lumped flow routing models may not be sufficiently accurate.

Thirdly, level pool routing is only applicable where the water surface remains horizontal such as in large pools which are comparatively wider and deeper than they are long in the direction of flow. This method is therefore not applicable to the Orange River.

Finally, the evaporation module which was developed for the ISIS software during the Evaporation Losses from South African Rivers (McKenzie and Craig, 1999) models evaporation from the river surface area. As the surface area is dependent on water surface level this module was developed for the full hydrodynamic model which includes the water surface level and backwater effects in its solution, rather than for the Muskingum model which is an "input-output" model.

3.3.2 Kinematic Wave Model

Using Manning's equation ($\frac{Q}{A} = \frac{1}{n} \left(\frac{A}{P} \right)^{\frac{2}{3}} S_f^{\frac{1}{2}}$) equation 3.27 shows that the celerity of a kinematic wave increases as Q increases, which would result in the wave moving down the river with its rising front getting steeper, but without diffusion or attenuation. It will be shown in Chapter 9 (and in particular in Figures 9.20 to 9.22) that there is a large diffusion effect on the flow in the Orange River. Because kinematic modelling essentially translates without diffusion (neglecting backwater effects) and should usually only be used on slopes in excess of 1:500 (the Orange River has a flat slope of +/-1:3000), it was decided that the kinematic wave model was inappropriate for modelling the Orange River.

Another check on the applicability of the kinematic wave model is given by Ponce and Symons (1977) and Ponce et al (1978) who suggested approximate criteria for the applicability of kinematic type models, including the Muskingum model. If the following criteria are met the routing errors will be less than 5 percent:

$$\tau_r S_o^{1.6} (q_o^{0.2} n^{1.2}) \geq 0.014$$

where τ_r is the time of rise (in hours) of the rising arm of the hydrograph, S_o is the channel slope and q_o is the unit width reference discharge.

For the Orange River downstream of Vanderkloof Dam this equates to:

$$0.05 \times 0.0003^{1.6} \times (200/150)^{0.2} \times 0.027^{1.2} = 1.6 \times 10^{-9} \ll 0.014$$

A similar criterion was suggested for diffusion type models including Muskingum-Cunge:

$$\tau_r S_o^{1.5} (q_o n)^{0.3} \geq 0.003$$

Applied to the Orange River just downstream of Vanderkloof Dam this yields:

$$0.05 \times 0.0003^{1.5} \times (200/150 \times 0.027)^{0.3} = 9.6 \times 10^{-8} \ll 0.003$$

A further drawback of lumped flow routing including the Muskingum method and the kinematic and diffusion methods, including Muskingum-Cunge, is that the water level is not modelled as part of the calculations. The water levels must be determined as a function of a stage discharge relationship at the points where it is required.

However, it should be noted that the rate of rise of the hydrograph released from Vanderkloof Dam is extreme and the simplified models may be applicable to sections further down the river, where the hydrographs have dissipated and the slope is steeper.

The dynamic model was chosen for this study in order to handle the conditions at Vanderkloof Dam and to include the evaporation module.

The disadvantages of the dynamic model are worth mentioning. The first, and main, disadvantage is that the dynamic model's demand for data is intensive, requiring information regarding cross sections of the river and estimates of the roughness at relatively frequent intervals. Providing this data for the full length of the Orange River was a time consuming procedure, with cross sectional data being taken from 1:50000 maps and aerial photographs, and the roughness and sub-surface cross sections being calibrated against recorded flow (See Chapter 9). Although the resolution of this data has limited accuracy, reasonable results were obtained with the information which was modelled

A further disadvantage of the dynamic model is that it is mathematically more complex than the simplified models and is therefore computationally more expensive. However, with the improvements of computer hardware and processing speed, this is no longer seen as a reason to reject the dynamic model. The hydraulic models developed in this study are based on the full dynamic modelling technique.

It should also be noted that there are various dynamic modelling packages available commercially and freely from the internet (HEC-2, HEC RAS etc). ISIS Flow was chosen because of the project team's association with Halcrows and because the code was made available in order to make the necessary changes for the purposes of this study.

3.4 SUMMARY

Having compared the various flow routing models and discussed their applicability to the Orange River it was decided to use the fully hydrodynamic model, based on the full St Venant equations, to simulate the Orange River downstream of Vanderkloof Dam. In the following chapter the solution of the St Venant equations is described so that the hydraulic modelling developments discussed in Chapter 5 can be understood.

4 SOLUTION OF THE ST VENANT EQUATIONS

The St Venant equations (2.37 and 2.38) are a pair of 1st order non-linear partial differential equations. Other than in certain restricted conditions the St Venant equations cannot be solved using analytical methods. If the general equations are to be solved, numerical methods must be used. These numerical methods can be classified as direct methods or characteristic methods. In the direct methods the St Venant equations of continuity and momentum are written as finite difference equations. In other words the partial derivatives of flow and stage are determined as the incremental differences between points in time and points in space. The method of characteristics involves resolving the St Venant equations into their characteristic form and then solving the characteristic equations either numerically or by finite difference approximations. The characteristic method will not be used here, but knowledge of the characteristics affects the way the boundary conditions are established. What follows is a paraphrase of Chow et al. (1998) and is included for completeness.

4.1 FINITE DIFFERENCE SCHEMES

Consider a function $u(x)$. A Taylor series expansion of $u(x)$ at $x + \Delta x$ and $x - \Delta x$ gives:

$$u(x + \Delta x) = u(x) + \Delta x \frac{\partial u(x)}{\partial x} + \frac{1}{2} \Delta x^2 \frac{\partial^2 u(x)}{\partial x^2} + \frac{1}{6} \Delta x^3 \frac{\partial^3 u(x)}{\partial x^3} + \dots + \frac{1}{n!} \Delta x^n \frac{\partial^n u(x)}{\partial x^n} \quad (4.1)$$

$$u(x - \Delta x) = u(x) - \Delta x \frac{\partial u(x)}{\partial x} + \frac{1}{2} \Delta x^2 \frac{\partial^2 u(x)}{\partial x^2} - \frac{1}{6} \Delta x^3 \frac{\partial^3 u(x)}{\partial x^3} + \dots + (-1)^n \frac{1}{n!} \Delta x^n \frac{\partial^n u(x)}{\partial x^n} \quad (4.2)$$

A *central difference scheme* is based on subtracting equation 4.2 from equation 4.1:

$$u(x + \Delta x) - u(x - \Delta x) = 2\Delta x \frac{\partial u}{\partial x} + R$$

where R is the residual representing the third and higher order terms and is assumed to be negligible if Δx is small enough. This gives:

$$\frac{\partial u}{\partial x} \approx \frac{u(x + \Delta x) - u(x - \Delta x)}{2\Delta x} \quad (4.3)$$

A *forward difference scheme* is obtained by subtracting $u(x)$ from equation 4.1:

$$u(x + \Delta x) - u(x) = \Delta x \frac{\partial u}{\partial x} + R$$

assuming that R , representing second and higher order terms, is negligible gives:

$$\frac{\partial u}{\partial x} \approx \frac{u(x + \Delta x) - u(x)}{\Delta x} \quad (4.4)$$

A *backward difference scheme* is obtained by subtracting equation 4.2 from $u(x)$:

$$u(x) - u(x + \Delta x) = \Delta x \frac{\partial u}{\partial x} - R$$

$$\frac{\partial u}{\partial x} \approx \frac{u(x) - u(x + \Delta x)}{\Delta x} \quad (4.5)$$

4.1.1 Solution of finite difference schemes

The independent variables of the St Venant equations, given in equations 2.37 and 2.38, are x and t . The unknown hydraulic quantities in the equations are flow, Q , and stage, h , which is used instead of the cross-sectional area A . Note that $h = z + y$ (level of bed + depth of flow) as shown in Figure 4.1. x is taken as the horizontal distance along the water course bed and the slope S_0 is assumed to be small (less than 1:20).

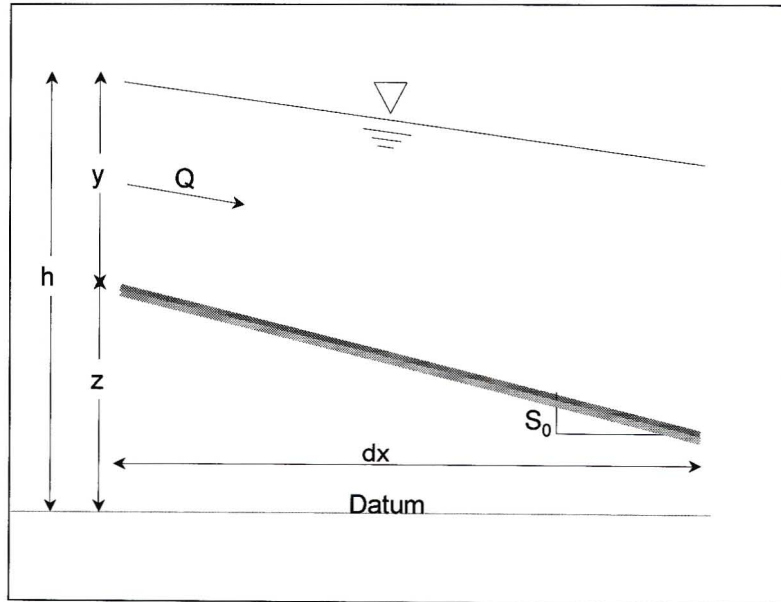


Figure 4.1: Definition of $h=y+z$

The unknown variables Q and h can be found by discretising the x - t plane into a grid of finite intervals as shown in Figure 4.2. Consider a river reach as being defined as a having an upstream and downstream control section, where boundary conditions apply. The river reach is divided into discrete intervals of length Δx . In ISIS these intervals are defined by the RIVER and INTERPOLATE sections. The time of the simulation is divided into discrete time intervals of length Δt . The finite difference

equations are determined and solved at the discrete points on this x-t plane. The point $(i\Delta x, j\Delta t)$ is designated as (i, j) in Figure 4.2.

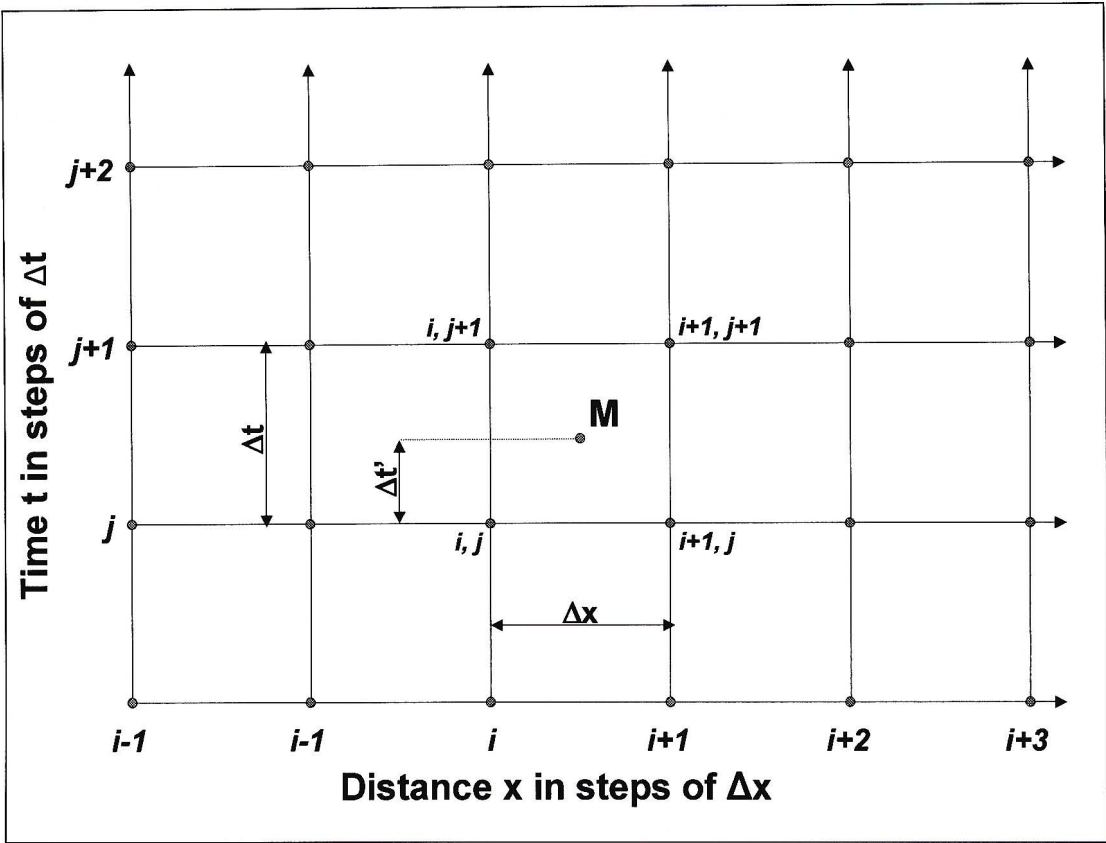


Figure 4.2: The x-t plane used for the solution of finite difference equations

The points on the time line perpendicular to the X-axis which pass through point $(i\Delta x, j\Delta t)$ represent that point at sequential time intervals. The points on the distance line perpendicular to the time line which passes through point $(i\Delta x, j\Delta t)$ represent the sequential points along the river $i, i+1, i+2, \dots$ at time $j\Delta t$.

The solution of finite difference equations can be obtained using either an explicit scheme or an implicit scheme. In the explicit schemes the solutions for the unknown values are determined sequentially along a time line (row) from one distance point to the next. In the implicit scheme the unknowns are determined simultaneously along a time line (row).

4.1.2 The Explicit Scheme

Assume that either of the hydraulic properties (Q or h) are represented by u and that these properties are known along the time row j . The hydraulic properties must be determined at the point $(i\Delta x, (j+1)\Delta t)$. The simplest stable scheme is established using a forward difference scheme for the time derivative and a central difference

scheme for the distance derivative. The spatial derivative is written using known terms on time row j :

$$\begin{aligned}\frac{\partial u_i^j}{\partial t} &= \frac{u_i^{j+1} - u_i^j}{\Delta t}, \text{ and} \\ \frac{\partial u_i^j}{\partial x} &= \frac{u_{i+1}^j - u_{i-1}^j}{2\Delta x}\end{aligned}\quad (4.6)$$

When a finite difference scheme is applied numerical errors are unavoidably introduced into the computation which affects the accuracy. By contrast the numeric stability of a scheme is dependent on the size of Δx and Δt . The Courant condition is a guideline for stability:

$$\Delta t \leq \min_i \left[\frac{\Delta x_i}{|v| + |c|} \right] \quad (4.7)$$

where v is the velocity at x and $c = \pm \sqrt{gD}$ is the celerity of the dynamic wave. The Courant condition must be met for stability but it does not guarantee accuracy.

4.1.3 The Implicit Scheme

In the implicit scheme the unknown variables are solved simultaneously row by row, from one time step to the next. The St Venant equations are applied to all the values on a time row simultaneously before the solutions are determined. The space derivative in the interval $(i, i+1)$ at time $j\Delta t$ is written as:

$$\frac{\partial u}{\partial x} = \frac{u_{i+1}^j - u_i^j}{\Delta x} \quad (4.8)$$

Equation 4.8 will, on average, most accurately represent the value of the derivative at the mid-point of the interval. This idea is exploited in the Priessman Four Point implicit scheme (Halcrow and HR Wallingford (1997), Meselhe and Holly (1997), Venutelli (2002)) (which is used in the ISIS software) in the following way as a weighted four point scheme. Consider the point M in Figure 4.2 half way between the columns i and $i+1$ and at $\Delta t'$ from row j . The time derivative of the hydraulic property u at M , which can stand for either Q or h is the average of the time derivatives at spatial nodes i and $i+1$:

$$\begin{aligned}\frac{\partial u}{\partial t} &\approx \frac{1}{2} \left[\frac{u_{i+1}^{j+1} - u_{i+1}^j}{\Delta t} + \frac{u_i^{j+1} - u_i^j}{\Delta t} \right] \\ &= \frac{u_{i+1}^{j+1} + u_i^{j+1} - u_{i+1}^j - u_i^j}{2\Delta t}\end{aligned}\quad (4.9)$$

The spatial derivative is determined by finding the derivative at times $j\Delta t$ and $(j+1)\Delta t$ and then combining them using a weighting factor, $\theta = \frac{\Delta t'}{\Delta t}$, depending on the distance of M from row j :

The spatial derivative at j :

$$\frac{\partial u_i^j}{\partial x} = \frac{u_{i+1}^j - u_i^j}{\Delta x}$$

The spatial derivative at $j+1$:

$$\frac{\partial u_i^{j+1}}{\partial x} = \frac{u_{i+1}^{j+1} - u_i^{j+1}}{\Delta x}$$

The spatial derivative at M

$$\frac{\partial u}{\partial x} = \theta \frac{u_{i+1}^{j+1} - u_i^{j+1}}{\Delta x} + (1 - \theta) \frac{u_{i+1}^j - u_i^j}{\Delta x} \quad (4.10)$$

If θ is equal to 0 the scheme is fully explicit as point M will be at time row $j\Delta t$, whereas if $\theta = 1$ the scheme is fully implicit with the M being at time $(j+1)\Delta t$. For practical implicit schemes $0.5 \leq \theta \leq 1$.

The major difference between the implicit and explicit schemes is that explicit schemes are only stable if the Courant condition is met ($\Delta t \leq \min_i \left[\frac{\Delta x_i}{|v| + |c|} \right]$), whereas this implicit scheme is unconditionally stable for all time steps, although accuracy is another issue.

Using the implicit scheme the unknown variables are solved simultaneously at a particular time before progressing on to the following time step. In the ISIS software an under-relaxation factor α (where $\alpha < 1$) is used to smooth the convergence between the timesteps.

4.2 SOLVING THE ST VENANT EQUATIONS

Recall the St Venant equations (equations 2.37 and 2.38), slightly modified:

The Continuity Equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q$$

or

$$\frac{\partial Q}{\partial x} + \frac{\partial (A + A_0)}{\partial t} - q = 0 \quad (4.11)$$

The Momentum equation

$$\frac{1}{A} \frac{\partial Q}{\partial t} + \frac{1}{A} \frac{\partial}{\partial x} \left(\frac{\beta Q^2}{A} \right) + g \frac{\partial h}{\partial x} + S_f + S_e - \frac{1}{A} \beta q v_x + \frac{1}{A} W_f B = 0$$

or

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left(\frac{\beta Q^2}{A} \right) + gA \left(\frac{\partial h}{\partial x} + S_f + S_e \right) - \beta q v_x + W_f B = 0 \quad (4.12)$$

where

x = the distance along the river channel

t = time

A = the cross sectional area of the channel at x

A_0 = cross sectional area of off channel storage

q = lateral inflow per unit length of the channel

β = momentum correction factor

g = gravity acceleration

h = water surface elevation

S_f = friction slope

S_e = eddy loss slope

v_x = velocity of lateral flow

W_f = wind shear force

B = Breadth of the channel at B

Now using equation 4.9 to determine the partial differentials with respect to time in equations 4.11 and 4.12 (Note that the bed slope has been included in the stage and velocity has been transformed to volumetric flow rate). Δt_j represents the timestep at the point in time $j\Delta t$.

$$\frac{\partial Q}{\partial t} = \frac{Q_{i+1}^{j+1} + Q_i^{j+1} - Q_{i+1}^j - Q_i^j}{2\Delta t_j} \quad (4.13)$$

$$\frac{\partial (A + A_0)}{\partial t} = \frac{(A + A_0)_{i+1}^{j+1} + (A + A_0)_i^{j+1} - (A + A_0)_{i+1}^j - (A + A_0)_i^j}{2\Delta t_j} \quad (4.14)$$

and using equation 4.10 to determine the partial differentials with respect to distance in equations 4.11 and 4.12:

$$\frac{\partial Q}{\partial x} = \theta \frac{Q_{i+1}^{j+1} - Q_i^{j+1}}{\Delta x_i} + (1 - \theta) \frac{Q_{i+1}^j - Q_i^j}{\Delta x_i} \quad (4.15)$$

$$\frac{\partial h}{\partial x} = \theta \frac{h_{i+1}^{j+1} - h_i^{j+1}}{\Delta x_i} + (1 - \theta) \frac{h_{i+1}^j - h_i^j}{\Delta x_i} \quad (4.16)$$

The q and A also depend on the position x :

$$q = \theta \frac{q_{i+1}^{j+1} + q_i^{j+1}}{2} + (1 - \theta) \frac{q_{i+1}^j + q_i^j}{2} \quad (4.17)$$

$$q = \theta \bar{q}_i^{j+1} + (1 - \theta) \bar{q}_i^j$$

$$A = \theta \frac{A_{i+1}^{j+1} + A_i^{j+1}}{2} + (1 - \theta) \frac{A_{i+1}^j + A_i^j}{2} \quad (4.18)$$

$$A = \theta \bar{A}_i^{j+1} + (1 - \theta) \bar{A}_i^j$$

where \bar{A} and \bar{q} are the average values between the cross sections Δx apart.

Substituting equations 4.13 to 4.18 into 4.11 and 4.12 and multiplying by Δx_i we get:

The continuity equation

$$\begin{aligned} & \theta(Q_{i+1}^{j+1} - Q_i^{j+1}) + (1 - \theta)(Q_{i+1}^j - Q_i^j) \\ & + \frac{\Delta x}{2\Delta t} [(A + A_0)_{i+1}^{j+1} + (A + A_0)_i^{j+1} - (A + A_0)_{i+1}^j - (A + A_0)_i^j] = 0 \end{aligned} \quad (4.19)$$

The momentum equation

$$\begin{aligned} & \frac{\Delta x}{2\Delta t_j} [Q_{i+1}^{j+1} + Q_i^{j+1} - Q_{i+1}^j - Q_i^j] + \\ & \theta \left\{ \left(\frac{\beta Q^2}{A} \right)_{i+1}^{j+1} - \left(\frac{\beta Q^2}{A} \right)_i^{j+1} + g \bar{A}_i^{j+1} [h_{i+1}^{j+1} - h_i^{j+1} + \bar{S}_{f_i}^{j+1} \Delta x_i + \bar{S}_{e_i}^{j+1} \Delta x_i] \right. \\ & \left. - (\beta q v_x)_i^{j+1} \Delta x_i + (\bar{W}_f \bar{B})_i^{j+1} \Delta x_i \right\} \\ & + (1 - \theta) \left\{ \left(\frac{\beta Q^2}{A} \right)_{i+1}^j - \left(\frac{\beta Q^2}{A} \right)_i^j + g \bar{A}_i^j [h_{i+1}^j - h_i^j + \bar{S}_{f_i}^j \Delta x_i + \bar{S}_{e_i}^j \Delta x_i] \right. \\ & \left. - (\beta q v_x)_i^j \Delta x_i + (\bar{W}_f \bar{B})_i^j \Delta x_i \right\} = 0 \end{aligned} \quad (4.20)$$

where the average values are defined as $\bar{Z} = \frac{Z_i + Z_{i+1}}{2}$.

The frictional losses may be defined as

$$\bar{S}_{f_i} = \frac{|Q_i| Q_i}{\bar{K}^2} \quad (4.21)$$

with \bar{K} being the average channel conveyance, which, if defined according to Manning's equation is:

$$\bar{K} = \frac{\bar{A}^2 \bar{R}^{\frac{4}{3}}}{\bar{n}^2} \quad (4.22)$$

with R being the average hydraulic radius, $\bar{R} = \frac{\bar{A}}{\bar{P}}$

Similarly the losses resulting from extraction and expansion are

$$\bar{S}_{ei} = \frac{K_{ei}}{2g\Delta x_i} \left[\left(\frac{Q}{A} \right)_{i+1}^2 - \left(\frac{Q}{A} \right)_i^2 \right] \quad (4.23)$$

The velocity of the wind relative to the water surface is

$$\bar{v}_{ri} = \frac{\bar{Q}_i}{A_i} - v_w \cos \omega \quad (4.24)$$

with ω being the angle between the wind and the water velocity vectors. The wind shear factor is defined as:

$$\bar{W}_{fi} = C_{wi} |\bar{v}_{ri}| \bar{v}_{ri} \quad (4.25)$$

where C_w is a frictional drag coefficient.

The chosen river reach is divided into N discrete intervals between the upstream boundary at $i=0$ and the downstream boundary at $i=N$. The St Venant equations of momentum and continuity (2 equations) are applied to each of the intervals between the upstream and downstream boundaries, which results in $2N-2$ equations.

In equations 4.19 and 4.20 all of the variables with the subscript j , i.e. all of the variables at time j are known, either from the initial conditions (if $j=0$) or from the solution to the previous time step. In addition the following terms are also either known or can be calculated: g , Δx_i , Δt_j , βx_i , K_e , C_w , V_w . The remaining unknown terms are:

$Q_i^{j+1}, Q_{i+1}^{j+1}, h_i^{j+1}, h_{i+1}^{j+1}, A_i^{j+1}, A_{i+1}^{j+1}, B_i^{j+1}, B_{i+1}^{j+1}$. However as the area, A , and breadth, B , can be expressed in terms of h the only unknowns that remain are $Q_i^{j+1}, Q_{i+1}^{j+1}, h_i^{j+1}, h_{i+1}^{j+1}$.

Therefore if there are N cross sectional intervals there are $2N$ (Q and h) unknowns and $2N-2$ equations. In order to solve these equations 2 additional equations are

required and these are obtained from the boundary conditions which will now be elaborated upon.

4.2.1 Boundary Conditions

The upstream and downstream boundary conditions of a reach of the river determine the uniqueness of the solution of the St Venant equations. Boundary conditions can either be external or internal. The river is divided into reaches by internal boundary conditions such as control structures, junctions or reservoirs. In ISIS the reaches are further divided into intervals by the RIVER and INTERPOLATE units. The RIVER units define the geometry (cross-sectional and longitudinal slope) of the sections of the reaches. INTERPOLATE units are required when the distance or change in geometry between RIVER units is too large for an accurate solution be determined for the model.

External Boundary Conditions

External boundary conditions describing the upstream and downstream ends of the river must be specified in order to solve the St Venant equations. In fact the disturbances which result in unsteady flow are usually introduced at the upstream or downstream boundaries. These boundary conditions determine the uniqueness of the solution.

For subcritical flow, disturbances in flow propagate both upstream and downstream, whereas for supercritical flow the disturbances only propagate downstream. Therefore for subcritical flow two independent boundary conditions are required, one at the upstream and the other at the downstream ends of the river.

For sub-critical flow the boundary conditions which may be applied are:

- A discharge hydrograph upstream and a stage hydrograph downstream
- A discharge hydrograph upstream and a rating curve (stage vs. flow depth) downstream

The Orange River has a discharge hydrograph (the releases from Vanderkloof Dam) at the upstream end of the model and a stage-discharge relationship at the downstream end of the model.

Internal Boundary Conditions

At the internal boundaries the flow varies rapidly and the St Venant equations cannot be applied across them. This break may be caused by control structures such as

weirs, reservoirs, waterfalls or confluences. Empirical stage-discharge relationships are used to model the river at these points. The mathematical model is separated into different reaches by these internal boundaries where the stage-discharge relationship is defined by empirical formulae.

Cross sections must be specified immediately upstream and downstream of the internal boundaries. As with any computational node two equations are required to solve the unknown variables. These are:

- The continuity equations, with negligible time dependent storage:

$$Q_i^{j+1} - Q_{i+1}^{j+1} = 0$$

- The empirical relationship between stage and discharge. The definition of these relationships change as the mode of the flow changes. The mode of the flow describes the flow conditions at the internal boundary, for example if the weir is drowned or flowing freely.

The Real Time Unit as described in Section 5 imposes a set of modified boundary conditions.

4.2.2 Initial Conditions

The initial conditions of the system must be specified in order to solve the first timestep when $t=0$. The initial conditions are the values for flow and stage, Q and h , at every computational node.

In the context of the operational model of the Orange River the results of the most recent real time simulation are used as the initial conditions for the following real time simulations and for the forecast simulations. Both the real time simulations and the forecast simulation are therefore started at an accurate estimate of the actual, unsteady conditions of the river. The accuracy is essential otherwise computational instabilities result. This practice is at variance with the usual start-up which uses a steady state gradually varied flow computation to determine the initial stage from constant flow. The Orange River is so long that it can never be assumed to be steady.

4.2.3 The Newton-Raphson Method as a solution to the St Venant equations

In order to simplify the notation, the momentum and continuity equations for the remainder of this discussion will be referred to as follows:

$UB(h_0, Q_0) = 0$	the upstream boundary condition at node 0	
$C_1(h_1, Q_1, h_2, Q_2) = 0$	the continuity equation and	
$M_1(h_1, Q_1, h_2, Q_2) = 0$	the momentum equation between nodes 1 and 2	
$C_i(h_i, Q_i, h_{i+1}, Q_{i+1}) = 0$	the continuity equation and	
$M_i(h_i, Q_i, h_{i+1}, Q_{i+1}) = 0$	the momentum equation between nodes i and i+1	
$C_{N-1}(h_{N-1}, Q_{N-1}, h_N, Q_N) = 0$	the continuity equation and	
$M_{N-1}(h_{N-1}, Q_{N-1}, h_N, Q_N) = 0$	the momentum equation between nodes N and N-1	
$DB(h_N, Q_N) = 0$	the downstream boundary condition at N	(4.26)

This forms a system a $2N$ linear equations with $2N$ unknowns and is solved, for each time step, on an iterative basis using the Newton-Raphson method. At the start of time step $j+1$, the first estimate of the unknown variables (Q and h) at $j+1$ are assigned at the $N-1$ points in the model. These first estimates are related, by means of the weighting factor θ (Section 4.1.3), to the known values calculated at timestep j . Computing the relationships in equations 4.26 results in a set of $2N$ residuals, because they will not balance to zero. The estimated values are corrected iteratively until the residuals are acceptably small. An under-relaxation factor, α , is used to smooth or accelerate the convergence between these iterations.

Consider, initially, a system of vector equations $f(\mathbf{x}) = 0$. where \mathbf{x} is the vector of unknown quantities and \mathbf{x}^k is the vector for iteration k . For the St Venant equation the non-linear system is linearised to

$$f(\mathbf{x}^{k+1}) \approx f(\mathbf{x}^k) + J(\mathbf{x}^k)(\mathbf{x}^{k+1} - \mathbf{x}^k) \approx 0 \quad (4.27)$$

where $J(\mathbf{x}^k)$ is made up of the first partial derivatives of $f(\mathbf{x})$ at \mathbf{x}^k and is known as the Jacobian matrix. The residual error is represented by $f(\mathbf{x}^{k+1})$ and the aim is to force it to zero. Rearranging equation 4.27 we have:

$$J(\mathbf{x}^k)(\mathbf{x}^{k+1} - \mathbf{x}^k) = -f(\mathbf{x}^k) \quad (4.28)$$

The system is solved for $(\mathbf{x}^{k+1} - \mathbf{x}^k) = \Delta \mathbf{x}^k$ and the next estimate is determined knowing $\Delta \mathbf{x}^k$. The process is repeated until $(\mathbf{x}^{k+1} - \mathbf{x}^k)$ is less than a specified tolerance. Applying equation 4.28 to the system of equations in 4.26 we get:

$$\frac{\partial UB}{\partial h_0} dh_0 + \frac{\partial UB}{\partial Q_0} dQ_0 = -RUB^k$$

for $i = 1..N - 1$:

$$\frac{\partial C_i}{\partial h_i} dh_i + \frac{\partial C_i}{\partial Q_i} dQ_i + \frac{\partial C_i}{\partial h_{i+1}} dh_{i+1} + \frac{\partial C_i}{\partial Q_{i+1}} dQ_{i+1} = -RC_i^k \quad (4.29)$$

$$\frac{\partial M_i}{\partial h_i} dh_i + \frac{\partial M_i}{\partial Q_i} dQ_i + \frac{\partial M_i}{\partial h_{i+1}} dh_{i+1} + \frac{\partial M_i}{\partial Q_{i+1}} dQ_{i+1} = -RM_i^k$$

$$\frac{\partial DB}{\partial h_N} dh_N + \frac{\partial DB}{\partial Q_N} dQ_N = -RDB^k$$

These equations can then be written as a sparse matrix, with four elements along the diagonal as shown below (for $N=5$).

$$\begin{bmatrix} \frac{\partial UB}{\partial h_1} & \frac{\partial UB}{\partial Q_1} & & & & & & & & & \\ \frac{\partial C_1}{\partial h_1} & \frac{\partial C_1}{\partial Q_1} & \frac{\partial C_1}{\partial h_2} & \frac{\partial C_1}{\partial Q_2} & & & & & & & \\ \frac{\partial M_1}{\partial h_1} & \frac{\partial M_1}{\partial Q_1} & \frac{\partial M_1}{\partial h_2} & \frac{\partial M_1}{\partial Q_2} & & & & & & & \\ \frac{\partial h_1}{\partial h_1} & \frac{\partial h_1}{\partial Q_1} & \frac{\partial h_2}{\partial h_2} & \frac{\partial h_2}{\partial Q_2} & & & & & & & \\ & & \frac{\partial C_2}{\partial h_2} & \frac{\partial C_2}{\partial Q_2} & \frac{\partial C_2}{\partial h_3} & \frac{\partial C_2}{\partial Q_3} & & & & & \\ & & \frac{\partial M_2}{\partial h_2} & \frac{\partial M_2}{\partial Q_2} & \frac{\partial M_2}{\partial h_3} & \frac{\partial M_2}{\partial Q_3} & & & & & \\ & & & & \frac{\partial C_3}{\partial h_3} & \frac{\partial C_3}{\partial Q_3} & \frac{\partial C_3}{\partial h_4} & \frac{\partial C_3}{\partial Q_4} & & & \\ & & & & \frac{\partial M_3}{\partial h_3} & \frac{\partial M_3}{\partial Q_3} & \frac{\partial M_3}{\partial h_4} & \frac{\partial M_3}{\partial Q_4} & & & \\ & & & & & & \frac{\partial C_4}{\partial h_4} & \frac{\partial C_4}{\partial Q_4} & \frac{\partial C_4}{\partial h_5} & \frac{\partial C_4}{\partial Q_5} & \\ & & & & & & \frac{\partial M_4}{\partial h_4} & \frac{\partial M_4}{\partial Q_4} & \frac{\partial M_4}{\partial h_5} & \frac{\partial M_4}{\partial Q_5} & \\ & & & & & & & & \frac{\partial DB}{\partial h_5} & \frac{\partial DB}{\partial Q_5} & \end{bmatrix} \begin{bmatrix} dh_1 \\ dQ_1 \\ dh_2 \\ dQ_2 \\ dh_3 \\ dQ_3 \\ dh_4 \\ dQ_4 \\ dh_5 \\ dQ_5 \end{bmatrix} = \begin{bmatrix} -RUB \\ -RC_1 \\ -RM_1 \\ -RC_2 \\ -RM_2 \\ -RC_3 \\ -RM_3 \\ -RC_4 \\ -RM_4 \\ -RDB \end{bmatrix} \quad (4.30)$$

Solving this set of equation gives values for dh_i and dQ_i . The values for the unknowns in the next iteration ($k+1$) are then

$$\begin{aligned} h_i^{k+1} &= h_i^k + dh_i \\ Q_i^{k+1} &= Q_i^k + dQ_i \end{aligned} \quad (4.31)$$

The process is repeated until dh_i and dQ_i are acceptably small.

Finally, to give more detail, we need to define the derivatives in the Jacobian matrix (4.30).

Differentiating 4.19 with respect to h_i^{j+1} and h_{i+1}^{j+1} :

$$\begin{aligned}\frac{\partial C}{\partial h_i} &= \frac{\partial C}{\partial A_i} \frac{\partial A_i}{\partial h_i} \\ &= \frac{\Delta x_i}{2\Delta t_j} \frac{\partial (A + A_0)_i^{j+1}}{\partial h} \\ &= \frac{\Delta x_i}{2\Delta t_j} (B + B_0)_i^{j+1}\end{aligned}\quad (4.32)$$

because $\Delta A = B\Delta h$

$$\begin{aligned}\frac{\partial C}{\partial h_{i+1}} &= \frac{\partial C}{\partial A_{i+1}} \frac{\partial A_{i+1}}{\partial h_{i+1}} \\ &= \frac{\Delta x_{i+1}}{2\Delta t_j} \frac{\partial (A + A_0)_{i+1}^{j+1}}{\partial h} \\ &= \frac{\Delta x_{i+1}}{2\Delta t_j} (B + B_0)_{i+1}^{j+1}\end{aligned}\quad (4.33)$$

where B_0 is the breadth of the off-channel storage

Differentiating equation 4.19 with respect to Q_i^{j+1} and Q_{i+1}^{j+1} :

$$\frac{\partial C}{\partial Q_i} = -\theta \quad (4.34)$$

$$\frac{\partial C}{\partial Q_{i+1}} = -\theta \quad (4.35)$$

Rewrite the momentum equation 4.20 as:

$$\begin{aligned}M_i &= \frac{\Delta x}{2\Delta t_j} [Q_{i+1}^{j+1} + Q_i^{j+1} - Q_{i+1}^j - Q_i^j] + \\ &\theta \left\{ \left(\frac{\beta Q^2}{A} \right)_{i+1}^{j+1} - \left(\frac{\beta Q^2}{A} \right)_i^{j+1} + g \frac{A_i^{j+1} + A_{i+1}^{j+1}}{2} \left[h_{i+1}^{j+1} - h_i^{j+1} + \frac{S_{f_i}^{j+1} + S_{f_{i+1}}^{j+1}}{2} \Delta x_i + \frac{S_{e_i}^{j+1} + S_{e_{i+1}}^{j+1}}{2} \Delta x_i \right] \right. \\ &\quad \left. - (\overline{\beta q v_x})_i^{j+1} \Delta x_i + \left(\overline{W_f} \frac{B_{f_i}^{j+1} + B_{f_{i+1}}^{j+1}}{2} \right)_i^{j+1} \Delta x_i \right\} \\ &+ (1 - \theta) \left\{ \left(\frac{\beta Q^2}{A} \right)_{i+1}^j - \left(\frac{\beta Q^2}{A} \right)_i^j + g \overline{A}_i^j \left[h_{i+1}^j - h_i^j + \overline{S}_{f_i}^j \Delta x_i + \overline{S}_{e_i}^j \Delta x_i \right] \right. \\ &\quad \left. - (\overline{\beta q v_x})_i^j \Delta x_i + (\overline{W_f B})_i^j \Delta x_i \right\} = 0\end{aligned}$$

when the unknown averages at the timestep $j+1$ have been expanded.

The derivatives of the momentum equation with respect to h_i^{j+1} , h_{i+1}^{j+1} , Q_i^{j+1} , Q_{i+1}^{j+1} are:

$$\begin{aligned}
\frac{\partial M}{\partial h_i} &= \theta \left\{ \left(\frac{\beta Q^2}{A^2} \frac{\partial A}{\partial h_i} \right)_i^{j+1} + g \frac{1}{2} \frac{\partial A_i^{j+1}}{\partial h_i} [h_{i+1}^{j+1} - h_i^{j+1} + \bar{S}_{f_i}^{j+1} \Delta x_i + \bar{S}_{e_i}^{j+1} \Delta x_i] + \right. \\
&\quad \left. g \bar{A}_i^{j+1} \left[-1 + \left(\frac{\partial S_f}{\partial h} \right)_i^{j+1} \Delta x_i + \left(\frac{\partial S_e}{\partial h} \right)_i^{j+1} \Delta x_i \right] \right. \\
&\quad \left. + \frac{1}{2} \left(\bar{W}_f \frac{dB}{dh} \right)_i^{j+1} \Delta x_i \right\} \\
&= \theta \left\{ \left(\frac{\beta Q^2 B}{A^2} \right)_i^{j+1} + \frac{g B_i^{j+1}}{2} [h_{i+1}^{j+1} - h_i^{j+1} + \bar{S}_{f_i}^{j+1} \Delta x_i + \bar{S}_{e_i}^{j+1} \Delta x_i] + \right. \\
&\quad \left. g \bar{A}_i^{j+1} \left[-1 + \left(\frac{\partial S_f}{\partial h} \right)_i^{j+1} \Delta x_i + \left(\frac{\partial S_e}{\partial h} \right)_i^{j+1} \Delta x_i \right] \right. \\
&\quad \left. + \frac{1}{2} \left(\bar{W}_f \frac{dB}{dh} \right)_i^{j+1} \Delta x_i \right\}
\end{aligned} \tag{4.36}$$

after substituting B for $\frac{\partial A}{\partial h}$ and similarly:

$$\begin{aligned}
\frac{\partial M}{\partial h_{i+1}} &= \theta \left\{ \left(-\frac{\beta Q^2 B}{A^2} \right)_{i+1}^{j+1} + \frac{1}{2} g B_{i+1}^{j+1} [h_{i+1}^{j+1} - h_i^{j+1} + \bar{S}_{f_i}^{j+1} \Delta x_i + \bar{S}_{e_i}^{j+1} \Delta x_i] \right. \\
&\quad \left. + g \bar{A}_i^{j+1} \left[1 + \left(\frac{\partial S_f}{\partial h} \right)_{i+1}^{j+1} \Delta x_i + \left(\frac{\partial S_e}{\partial h} \right)_{i+1}^{j+1} \Delta x_i \right] \right. \\
&\quad \left. + \frac{1}{2} \left(\bar{W}_f \frac{dB}{dh} \right)_{i+1}^{j+1} \Delta x_i \right\}
\end{aligned} \tag{4.37}$$

$$\frac{\partial M}{\partial Q_i} = \frac{\Delta x}{2 \Delta t_i} + \theta \left\{ -2 \left(\frac{\beta Q}{A} \right)_i^{j+1} + g \bar{A}_i^{j+1} \left[\left(\frac{\partial S_f}{\partial Q} \right)_i^{j+1} \Delta x_i + \left(\frac{\partial S_e}{\partial Q} \right)_i^{j+1} \Delta x_i \right] \right\} \tag{4.38}$$

$$\frac{\partial M}{\partial Q_{i+1}} = \frac{\Delta x_i}{2 \Delta t_i} + \theta \left\{ 2 \left(\frac{\beta Q}{A} \right)_{i+1}^{j+1} + g \bar{A}_i^{j+1} \left[\left(\frac{\partial S_f}{\partial Q} \right)_{i+1}^{j+1} \Delta x_i + \left(\frac{\partial S_e}{\partial Q} \right)_{i+1}^{j+1} \Delta x_i \right] \right\} \tag{4.39}$$

The derivative for the friction loss is found by differentiating equation 4.21

$$\begin{aligned}
\bar{S}_{f_i} &= \frac{|Q_i| Q_i}{K^2} = \frac{|Q_i| Q_i n_i^2}{\bar{A}^2 \bar{R}^3} = \frac{|Q_i| Q_i n_i^2}{\bar{A}^2 \left(\frac{\bar{A}}{\bar{P}} \right)^3} = \frac{|Q_i| Q_i n_i^2 \bar{P}^3}{\bar{A}^{10}} \\
\frac{\partial \bar{S}_f}{\partial h_i} &= 2 \bar{S}_{f_i} \left(\frac{1}{n_i} \frac{dn_i}{dh_i} - \frac{5 B_i}{6 \bar{A}_i} + \frac{1}{3 \bar{P}_i} \frac{dP_i}{dh_i} \right)
\end{aligned} \tag{4.40}$$

$$\frac{\partial \bar{S}_f}{\partial h_{i+1}} = 2\bar{S}_{fi} \left(\frac{1}{n_i} \frac{dn_i}{dh_{i+1}} - \frac{5B_{i+1}}{6A_i} + \frac{1}{3P_{i+1}} \frac{dP_{i+1}}{dh_{i+1}} \right) \quad (4.41)$$

$$\frac{\partial \bar{S}_f}{\partial Q_i} = \bar{S}_{fi} \left(\frac{1}{n_i} \frac{dn_i}{dQ_i} + \frac{1}{\bar{Q}_i} \right) \quad (4.42)$$

$$\frac{\partial \bar{S}_f}{\partial Q_{i+1}} = \bar{S}_{fi} \left(\frac{1}{n_i} \frac{dn_i}{dQ_{i+1}} + \frac{1}{\bar{Q}_{i+1}} \right) \quad (4.43)$$

The derivatives of S_e are found by differentiating equation 4.23

$$\begin{aligned} \bar{S}_{ei} &= \frac{K_{ei}}{2g\Delta x_i} \left[\left(\frac{Q}{A} \right)_{i+1}^2 - \left(\frac{Q}{A} \right)_i^2 \right] \\ \frac{\partial S_e}{\partial h_i} &= -\frac{K_{ei}}{2g\Delta x_i} \left[-2 \left(\frac{Q}{A} \right)_i \left(\frac{Q}{A^2} \right) \frac{dA}{dh_i} \right] \\ &= \left(\frac{K_e Q^2 B}{g\Delta x A^3} \right)_i \end{aligned} \quad (4.44)$$

$$\frac{\partial S_e}{\partial h_{i+1}} = \left(-\frac{K_e Q^2 B}{g\Delta x A^3} \right)_{i+1} \quad (4.45)$$

$$\frac{\partial S_e}{\partial Q_i} = \frac{K_{ei}}{2g\Delta x_i} \left[-2 \left(\frac{Q}{A} \frac{1}{A^2} \right)_i \right] = -\left(\frac{K_e Q}{g\Delta x A^2} \right)_i \quad (4.46)$$

$$\frac{\partial S_e}{\partial Q_{i+1}} = \left(\frac{K_e Q}{g\Delta x A^2} \right)_{i+1} \quad (4.47)$$

Finally the boundary conditions are considered. The partial derivatives at the upstream boundary are derived as follows:

If the upstream boundary condition is a discharge hydrograph

$$\frac{\partial UB}{\partial h_1} = 0 \text{ and } \frac{\partial UB}{\partial Q_1} = 1 \quad (4.48)$$

If the downstream boundary condition is discharge hydrograph

$$\frac{\partial DB}{\partial h_N} = 0 \text{ and } \frac{\partial DB}{\partial Q_N} = 1 \quad (4.49)$$

where N represents the last node (or the boundary) of the system.

If the downstream boundary condition is stage hydrograph

$$\frac{\partial DB}{\partial h_N} = 1 \text{ and } \frac{\partial DB}{\partial Q_N} = 0 \quad (4.50)$$

If the downstream boundary is a stage-discharge rating curve then

$$\frac{\partial DB}{\partial h_N} = \frac{Q_{k+1} - Q_k}{h_{k+1} - h_k} \text{ and } \frac{\partial DB}{\partial Q_N} = 1 \quad (4.51)$$

where k is the iteration number.

Once all the equations have been determined they are input to the matrices and the solution is found for the iteration within the timestep. Iterations are discontinued once convergence has been achieved within tolerance. The computation is then advanced to the next time step, which is decided using the Courant condition and if necessary adjusted by the adaptive time stepping routine (Section 5.2)

4.3 SUMMARY

This chapter describes the solution of the St Venant equations of flow using an implicit finite difference scheme. Once the finite difference approximation has been set up the solution to the system of equations is found using the Newton-Raphson method.

The inclusion of real time data in the application of the model to the Orange River meant updating the computed values with observed values recorded at the ends of certain reaches. In order to do this specialised units describing modified boundary conditions were developed and are explained in Chapter 5.

5 HYDRAULIC MODELLING DEVELOPMENTS

The purpose of this chapter is to describe the developments made to the ISIS hydraulic model in this study so that it could be applied to the Orange River. The first development was the inclusion of a real time control unit, which would enabled computational values to be replaced by recorded data at the real time controls. The second development was the adaptive timestepping routine which significantly reduced the computational times of the simulations.

5.1 THE REAL TIME UNIT

The ISIS model uses the Priessman 4 point implicit solution method to solve the St Venant equations of flow. Internal boundaries are control sections which divide the river into separate reaches. The internal boundary conditions are defined by equations relating flow and stage in the general form $Q=ah^b$, where h is the water depth, a is a coefficient depending on the structure and b is a value greater than or equal to 1.5 for weirs and 0.5 for free flow under a sluice gate. These internal boundaries are modelled using empirical or semi empirical equations and include different types of weirs, sluices, bridges, junctions and bifurcations. Those used in the model of the Orange River are described in more detail in Chapter 9.

External boundary conditions are required at the extreme ends of the model. Under sub-critical flow conditions an independent boundary condition is required at the upstream and downstream end of the model. For supercritical flow two independent boundary conditions are required at the upstream end of the model. These boundary conditions are defined by stage-discharge (rating curves), stage-time or discharge time relationships.

The real time unit which is described in this section is designed to force the simulated stage and flow at a control section in the model to match that recorded on the river. By doing so the simulated results are corrected to actual values. The accuracy of the simulation downstream of the real time unit (for sub-critical flow) is therefore improved.

In order to make this unit independent of the format in which the real time data are recorded, the real time data must be re-written to an ASCII *.RTM (real time) file, which is then read by the ISIS model. The Graphical User Interface (GUI) developed in this study performs this task.

In technical terms the real time unit is a modified boundary condition. (See Section 4.2.1) It can either be a real flow-time control (QTBREAL) at the upstream boundary, or a flow-stage control (QHREAL) at an internal boundary.

5.1.1 The Flow-Time Real Time Control (QTBREAL)

This control defines an external boundary relationship and is used to model inflows and abstractions from the river in real time. The real time flows are recorded as a set of data pairs comprising flow and time. These flows are input into the model at the recorded times. The time intervals do not have to match the modelled timesteps because the model interpolates between these data pairs, either linearly or using a cubic spline (to be selected by the user).

The user must also specify a default flow-time relationship. If no real time data are available the simulations will occur using these default values.

The QTBREAL unit was developed to model the released flows from Vanderkloof Dam and abstractions from the river in real time, although the latter are not recorded in real time on the Orange River. Using the GUI the user can alter the abstraction scenarios before a simulation run is started. The abstraction information is then written to the RTM file and used by the QTBREAL unit in the simulation. This allows the user to easily analyse various scenarios without changing the ISIS data file, which includes the river geometry and flow controls which are not operated in real time.

A smoothing function was written for the transition between the default Q-t relationship (which is modelled when no real time data are available) and the real time Q-t relationship. The user must specify the number of time-steps over which this transition is to take place and the percentage change which must take place between each time step. The purpose of this smoothing is to allow the model to damp out (numerically diffuse) the differences in the data sets which, because they are transients, would cause instabilities in the modelled flows.

5.1.2 The Flow-Stage Real Time Control (QHREAL)

This unit is an internal control based on recorded stage at weirs or gauges and is used to model recorded stage and its associated flow. The real time stage at a gauging station is recorded as a set of data pairs comprising stage and time. The unit acts as a flow-stage control and as such the user must define the flow-stage (Q-h) relationship that exists at the control. The model forces the stage immediately upstream of the node to equal that which was recorded. The related real time flow is determined, using the Q-h relationship and propagated downstream of the control

structure. This is in accordance with sub-critical flow conditions. The flow upstream of the weir is not adjusted to the real time flow but is left at the simulated flow which has propagated down the reach upstream of the control. Comparing the simulated flow immediately upstream of the weir to the real time flow immediately downstream of the weir gives an indication of the accuracy of the calibration of the upstream reach.

It is necessary to define river cross sections immediately upstream and downstream of a real time control unit. The three units defining a real time gauge or weir are therefore the upstream cross section, the real time unit and the downstream cross sections as shown in Figure 5.1. Although these units are shown some distance apart in Figure 5.1 (for clarity), the defined distance between the sections in the model is zero. Two sets of results are available at the real time unit, the results at the upstream cross section and the results at the downstream cross section.

Consider for example Figure 5.2 and Figure 5.3 (Note the change in scale for the charts). The positions of the 2 sets of charts are indicated in Figure 5.1.

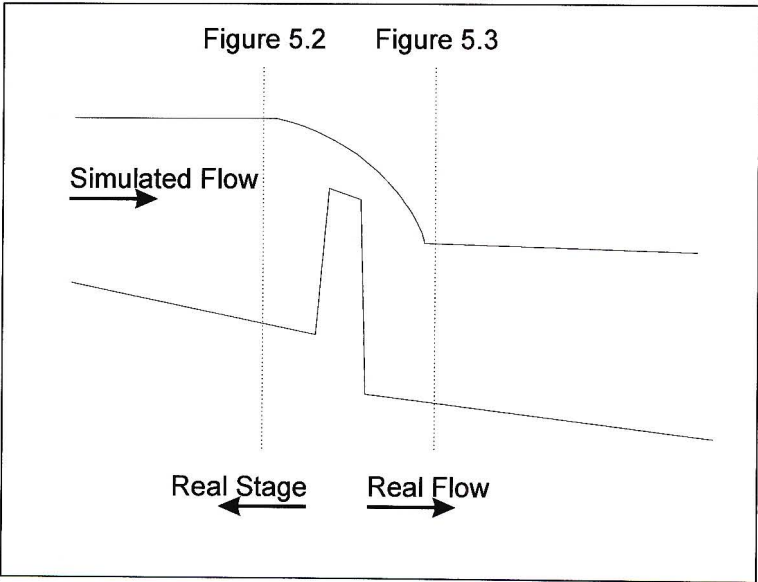


Figure 5.1: Longitudinal section of a real time flow-stage unit

Figure 5.2 shows the simulated flow and stage immediately upstream of the real time unit, compared to the recorded stage and its associated flow. It can be seen that the stage upstream of the control is equal to that of the recorded stage (because the unit has forced the stage to the recorded stage). The flow upstream of the unit does not equal that of the “recorded flow”. Instead it is simulated flow which has propagated downstream and reached the control. Differences between the recorded and simulated results give an indication of whether the model is correctly calibrated and the net effect of inflows or abstractions from the upstream reach.

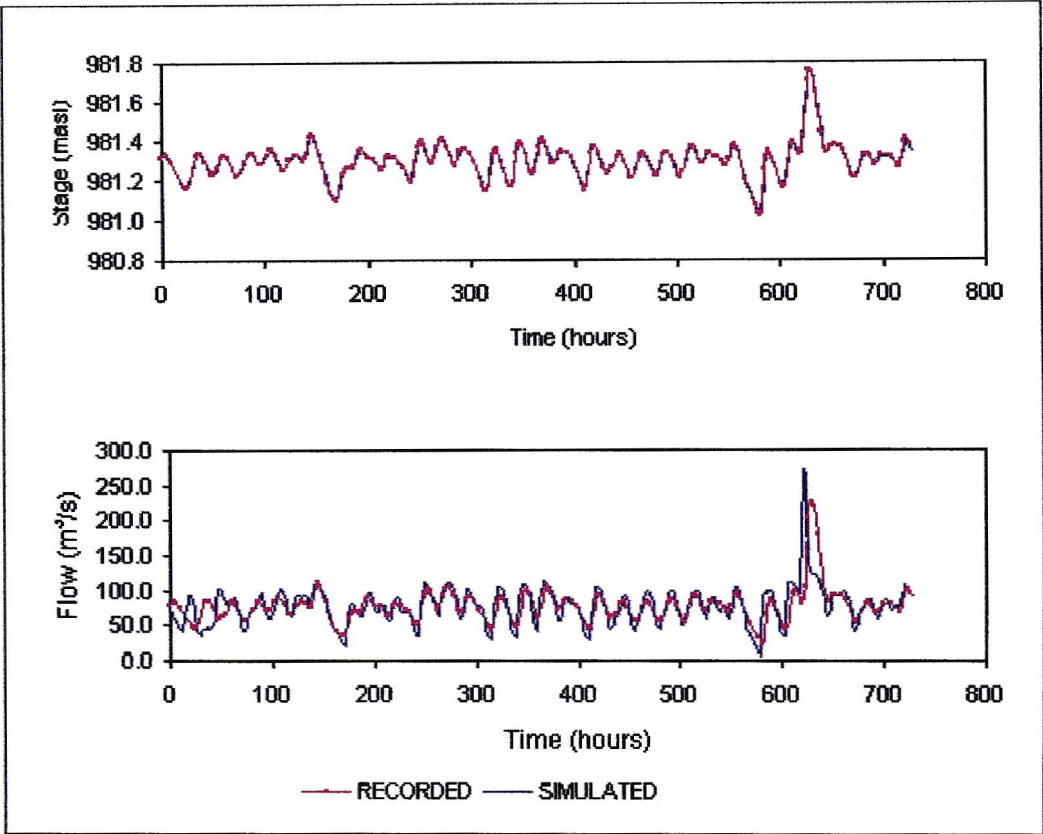


Figure 5.2: Conditions upstream of the real time flow-stage unit

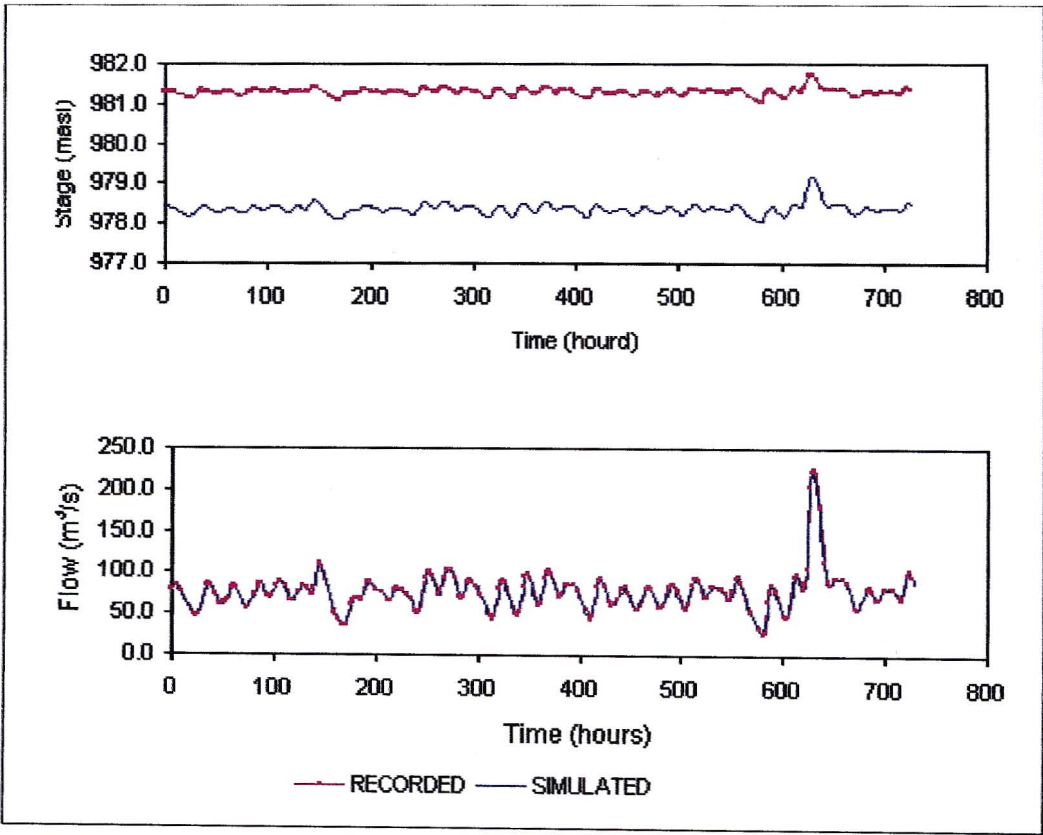


Figure 5.3: Conditions downstream of the real time flow-stage unit

Figure 5.3 shows the flow and stage immediately downstream of the weir. It can be seen that the simulated flow downstream of the unit matches the “recorded flow”. The simulated stage, however, no longer matches the recorded stage. The differences between the recorded and simulated stage are due to the backwater effect downstream of the weir or control which this unit represents (for sub-critical flow). The stage immediately downstream of the unit is dependent on the backwater effect of the sections further down the river (for sub-critical flow).

5.1.3 Potential Future Developments to the Real Time Control

Simulated flows which have propagated from upstream are available at the real time control. In calibrating the model these flows are compared with the recorded data at the real time control. Differences in volumes between these data are generally due to unmodelled events, such as local inflow or abstractions from the reach upstream of the control. Differences in the timing of peaks or troughs, with associated differences in the flow at these peaks and troughs indicate that the model is not properly calibrated in terms of the attenuation of flow. The parameter in the ISIS model which is used to calibrate the attenuation of flows is Manning’s n , which is defined in the ISIS RIVER units (not between the computational Δx ’s). There can be a number of river units, each defining a different n , within a reach upstream of a real time unit.

A potential and useful development to the ISIS model would be to develop a self-calibration routine which would be based on the continual comparison of the simulated and recorded data at the real time units. This routine would then adjust the Manning’s n values in each RIVER unit according to the required percentage change.

This would be particularly useful considering that the conveyance of the river, K changes with different flow regimes. This is evident in the following definitions:

$$K^2 = \frac{A^2 R^{\frac{2}{3}}}{n^2} \text{ and } R = \frac{A}{P}$$

where R is the hydraulic radius, P is the length of the wetted perimeter and n is Manning’s roughness coefficient and A and P are dependent on flow. Currently n is assumed constant for all modelled flow regimes as suggested by the Manning formula, unless specifically adjusted by the user. This means that the model should be re-calibrated for normal, flood and low flow conditions. The model was only calibrated for low flow conditions in this study.

5.2 ADAPTIVE TIMESTEPPING

There are 2 turbines at Vanderkloof Dam with a combined maximum discharge of between 170 m³/s and 200 m³/s depending on the water level of the dam. The valves which control the flow of water through these turbines can be opened or closed within approximately three minutes. The result is that the discharge from Vanderkloof Dam varies rapidly.

This situation causes instability and non-convergence in the solution of the St Venant equations, unless the timestep is sufficiently short. These very short timesteps resulted in long run times for the simulations, which was wasteful, because once the wave diffuses downstream, a longer time step could be used. There was thus a mismatch in the required precision depending on the distance of the reach from Vanderkloof Dam. It was decided to develop an adaptive timestepping routine as these short time steps were only required during the periods when the releases were changing rapidly, which only occurs approximately four times in 24 hours.

The adaptive timestepping routine enables a relatively long default timestep to be used for the simulation. If, after an increase in model time (timestep), computational instabilities occur, the timestep is reduced and the calculations are repeated until a solution for that point in time is found, where after the timestep is restored to the default and the next timestep is simulated. The timesteps are only shortened as required.

In ISIS the user can set the convergence criteria by adjusting the flow and head tolerances. The maximum number of iterations for determining a solution must also be set.

After an iteration the convergence ratios are calculated for flow

$$QRatio = \frac{Max(abs(Q(I)_{new} - Q(I)_{old}))}{max(abs(Q(I)_{new}))}$$

and for stage

$$HRatio = \frac{Max(abs(H(I)_{new} - H(I)_{old}))}{max(abs(H(I)_{new}))}$$

where H(I) and Q(I) refer to the set of stage and flow results for the iteration, I, in the solution at a certain modelled time. The subscript new and old refer to the solution at iteration I and I-1. If the convergence ratios violate the convergence criteria another iteration must be performed. If the maximum number of iterations is exceeded,

without the convergence criteria being met the timestep is reduced and the calculations are repeated.

Note that this methodology relies on detecting instabilities due to fast rise times (computationally the generation of surge waves) which occur in spite of the time step satisfying the Courant condition. The instabilities manifest themselves as waves which grow until negative depths and flows are computed.

At present the solutions for the whole model are re-calculated when the timestep is reduced. Theoretically it would be possible to implement adaptive timestepping only in the reach (between 2 hydraulic controls) where the instability occurred. The solutions of the flow equations for that reach would then be calculated on a shorter timestep until the instability is “overcome” and the simulated time for the reach is the same as for the rest of the model.

Even without this additional improvement, it was found that when using the adaptive timestepping routine the times for the simulations of the Orange River were reduced by up to a factor of 5 compared to when the routine was not used.

5.3 SUMMARY

This chapter described the modelling developments made to the ISIS engine so that it could be used to model the Orange River using real time data. The real time stage-time and flow-time units were described and the adaptive time stepping unit was explained.

To this point the theoretical aspects of the dissertation have been discussed. The St Venant equations of flow were derived from the transport theorem (Chapter 2) before the different flow routing models were described and evaluated in terms of their applicability to the Orange River (Chapter 3). Once the selection of the full hydrodynamic model was justified, the solution of the St Venant equations using a Priessman 4 point implicit finite difference scheme and the Newton-Raphson method was described (Chapter 4). Finally the developments made to the ISIS modelling engine were discussed (Chapter 5).

In the following chapters the graphical user interface (GUI) which was developed in this dissertation is described (Chapters 6 and 7). The function of the GUI is to control the flow of data and results between the real time telemetry system, the ISIS hydraulic engine and the users, uniting the various components into the decision

support tool. After the GUI is described, more detail is given on the real time telemetry network (Chapter 8) and the data input to and calibration of the hydraulic model (Chapter 9). Finally, the decision support tool is assessed (Chapter 10) before final conclusions and recommendations are made (Chapter 11).

6 BASIC CONCEPTS OF THE DECISION SUPPORT SYSTEM

Because the time taken for a change in flow to travel from Vanderkloof Dam to the river mouth at Alexander Bay is in the order of a few weeks, it is important to understand the consequence of any releases (power and other) on the river. This can only be done by simulating the possible future scenarios based on the current condition of the river. To decide how to optimise the release scenario a decision support tool was devised.

The basic concept of the decision support system is to use a hydraulic model of the Orange River to simulate the releases from Vanderkloof Dam. The ISIS Flow software (Halcrows/IH Wallingford, 1997) was chosen as the modelling software because the project team had close connections to the software developers and the source code could be customised for the project. At this stage (November 2001), the software which has been developed (and is described in this chapter) is yet to be applied in real time, due to the lack of suitable real time data recorded on the river (Chapter 8).

The hydraulic model has been adapted to simulate the forecast effect of the possible future release hydrographs (and the actual releases made) from Vanderkloof Dam, on the stage and flow downstream of the dam. Before these forecast simulations are conducted the initial conditions for the simulation must be known. These initial conditions describe the unsteady flow and stage at the start of the simulation at every computational node in the model. The initial conditions are a mathematical requirement of the model (Section 4.2.2) but they also represent the physical state of the river at the time when the forecast simulation is to be conducted. It follows that the closer the model's initial conditions are to the actual conditions in the river the more accurate the forecast simulation. Because of the length of the river (1400km) and its slow response time (3-4 weeks) the conventional method of gradually changing the steady flow starting conditions to the current conditions is impractical.

In order to improve the estimate of the initial conditions for any forecast simulation it was decided to develop a separate real time hydraulic model of the Orange River to run in parallel with the forecast model used for simulating proposed release scenarios. Real time recorded data are available at several gauging stations on the Orange River as described in Chapter 8. In the real time model the stage at the nodes corresponding to the telemetry stations is "forced" to the recorded values at the real time controls, as described in Section 5.1. The flows at these nodes are then

calculated according to the relevant stage-discharge relationship and propagated downstream. This has the effect of improving the simulated flows downstream of the real time station.

The real time model is run at regular intervals to update the simulated results. The frequency and run length of the updates are determined by the availability of the data and the time since the previous updates. If the data are available in real time the model can be run in real time, alternatively the model is updated at the intervals when the data become available. After the model has been updated with the latest real time data, the results represent a best estimate of the actual conditions of the river. These results can then be used as the real time initial conditions for the forecasting runs. Each forecasting run therefore starts from the most recent estimate of the current river conditions.

The real time model and the forecasting models are identical with respect to the physical characteristics of the river. They differ only in their use of either real time or forecast data.

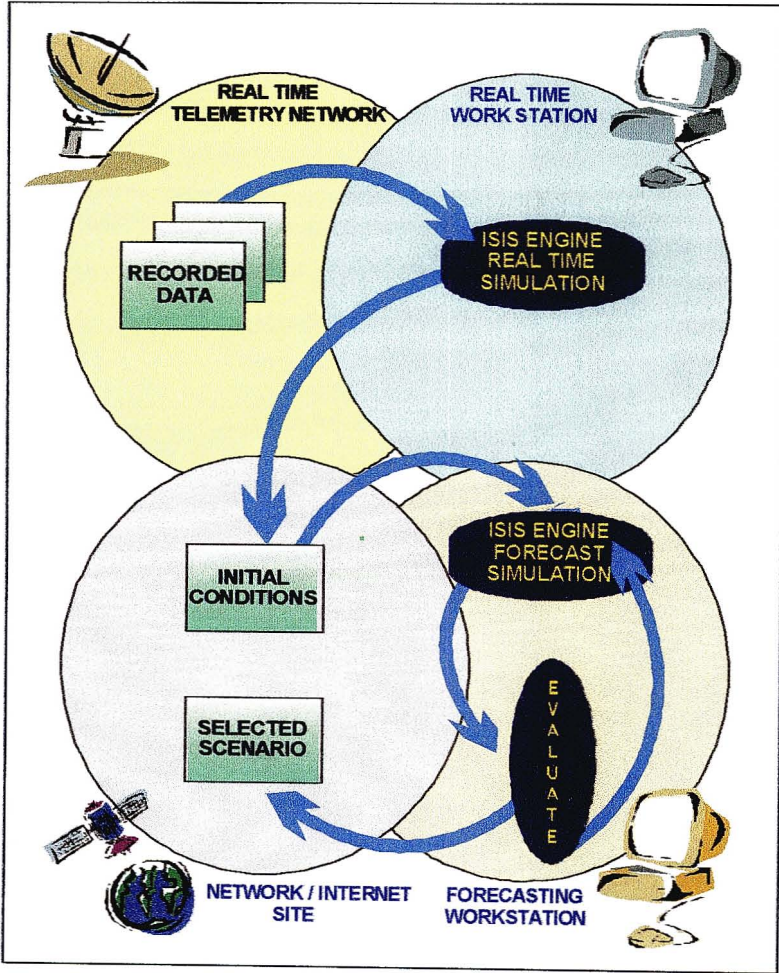


Figure 6.1: Conceptualisation of the Decision Support System

The decision support tool developed in this study consists of several components. The inter-relationship between these components is depicted in Figure 6.1 where each circle represents a component:

- The first component is the Telemetry Network of gauging stations to be described in Chapter 8. The stage and flow data are recorded at these stations and transmitted to the Department of Water Affairs and Forestry (DWAF) where they are archived.
- At the **Real Time Workstation** the ISIS hydraulic model operates in real time. The recorded data are downloaded from DWAF and the real time simulation is updated as the recorded data are incorporated into the simulation. The up to date conditions of the river are then stored on the project network site so that they are available to the forecasting workstation or any other interested party with access to the site.
- At the **Forecast Work Station** the proposed release hydrographs are evaluated. The current river conditions available on the project network site are downloaded and used as the initial conditions of the forecast simulation. Proposed release scenarios from Vanderkloof Dam and/or changes to the downstream demands are evaluated on a “what if” basis.
- The **Graphical User Interface (GUI)** to be described in Chapter 7 facilitates the exchange of data between the telemetry network and the real time and forecasting models and the user. The GUI is not represented by a “bubble” in Figure 6.1 but rather controls the processes and the transfer of data.

There is no requirement for the real time and forecasting models to run on the same machine. The only requirement is that both models have access to the project network site where the current conditions of the river are stored.

The purpose of the GUI is to collect and collate the remote real time data, manipulating it into the format required by the hydraulic modelling component. The GUI also facilitates the exchange of data between the real time and forecasting models and the FTP server or LAN. This information includes the results of the simulations, the pre-determined release scenarios and information regarding downstream demands that have to be modelled. This GUI was developed by the candidate for this project and is described in detail in Chapter 7.

The decision support tool will be used in an iterative manner. When a release scenario is to be evaluated the procedure follows the steps below:

1. Download the latest recorded data (which are stored at DWAF) to the real time work station
2. Run the real time model to update the initial (current) conditions on the real time work stations
3. Load the current conditions to the project network site
4. Suggest a proposed release pattern
5. Download the current conditions of the river from the project network site to the forecast work station. These are used as the initial conditions for the forecast simulation.
6. Simulate (forecast) the release pattern using the hydraulic model on the forecast work station.
7. Evaluate the results of the simulated forecast to determine whether the magnitude, duration and timing of the releases are such that the downstream water requirements are satisfied, the losses from the river are minimised and the hydro-electric requirements from Eskom are met.
8. If necessary the suggested release pattern can be altered in Step 4 and Steps 4 to 7 can be repeated until a satisfactory result is obtained.

6.1 REAL TIME HYDRAULIC MODELLING

The real-time telemetry data are only available for those nodes in the model that correspond to the real-time flow gauging stations. The model of the Orange River consists of 1726 nodes of which 14 are control structures (Table 9.1). These nodes describe the physical geometry of the river and the simulated results are stored for these nodes. These nodes divide the river into the 14 reaches defined by the 14 modelled control structures (weirs, gauging stations, rapids and water falls), other than the first node which is at Vanderkloof Dam. Of these 14 control structures only 5 are real stage-time controls. There are also 3 real flow-time controls at Vanderkloof Dam and the inflows from the Vaal and Fish rivers.

The purpose of the real-time modelling is to incorporate the incoming recorded data into the hydraulic model so that the modelled results, at all the nodes, other than and including the nodes corresponding to the real time gauging stations, are a best estimate of the flow conditions in the river. The results of the simulation are “forced” to the recorded conditions at the real time controls and the hydrodynamic model then performs the necessary hydraulic routing calculations to determine the subsequent

conditions at all the other nodes where no real-time data exist. These procedures were explained in Chapter 5.

There are various unforeseen, and therefore unmodelled, factors that can alter the flow conditions, such as localised rainfall events, incremental inflows and unexpected losses and abstractions from the river. The effect of these occurrences is minimised in the real-time simulations as the modelled results are corrected to the recorded values at the nodes corresponding to the gauging weirs with telemetry stations.

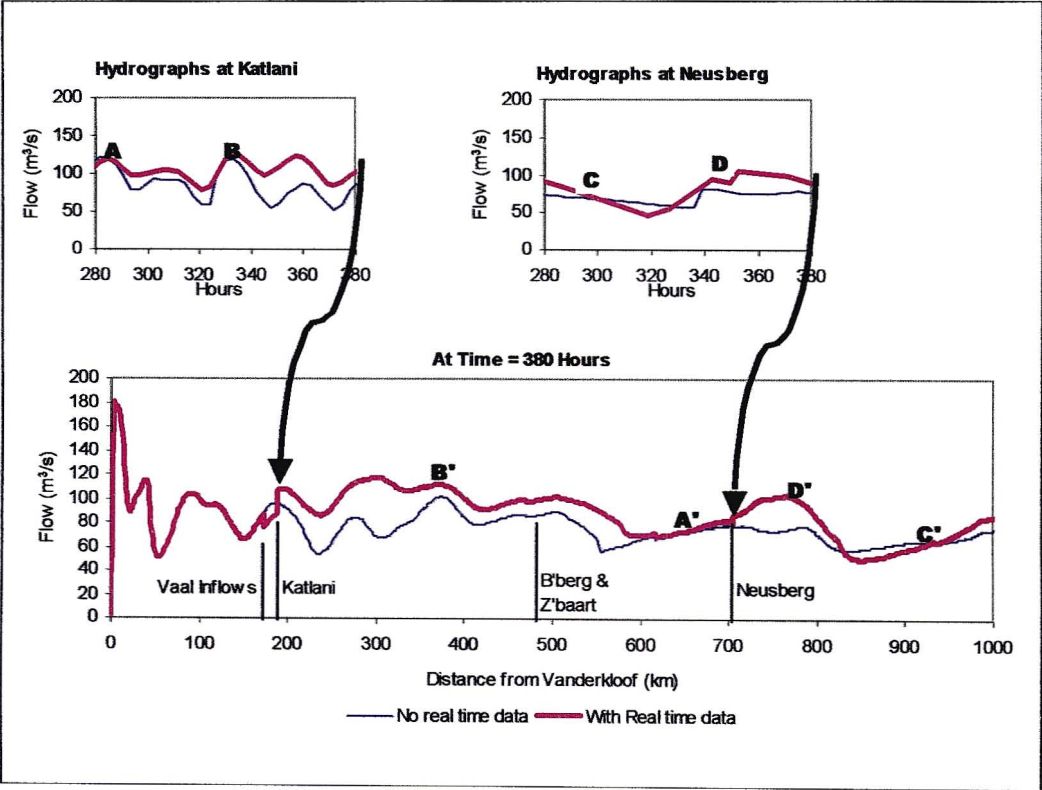


Figure 6.2: Real Time modelling - Flow trajectory output with and without real time telemetry data

Consider for example Figure 6.2 in which a long section of the Orange River is depicted at a certain point in time (380 hours after the start of the run) for two simulations. The thick pink line is the flow profile for the real time simulation with updates from the telemetry data. The thin blue line represents the flow when no recorded data were used, i.e. when the telemetry data were “switched off”. Recorded data at Katlani, 190 km from Vanderkloof Dam, just downstream of the Orange-Vaal confluence, and at Neusberg Weir at 703 km from Vanderkloof Dam were included in the real time simulation. There are other real time stations but only these two were used for this explanation. The positions of these stations on the river are indicated by the arrows. The recorded (thick pink) and simulated (thin blue) hydrographs at these

stations are shown in the smaller graphs. Note that the main chart shows flow versus distance at a particular point in time, whereas the two smaller charts show flow versus time at two locations on the river. The (negative) vertical discontinuity at the Vaal River confluence is due to the discrete river losses and demands taken off at this point. The Vaal River inflows were not modelled.

A and B are crests on waves at Katlani. C and D are “points” on the hydrograph at Neusberg Weir. “Crest” A travels from Katlani to position A’ (450 km) in 96 hours ($380 - 284 = 96$ hours). “Crest” B travels from Katlani to position B’ (180 km) in 50 hours ($380 - 330 = 50$ hours). “Cross-over” C travels from Neusberg Weir to position C’ (220km) in 80 hours and “crest” D travels from Neusberg to position D’ (80 km) in 24 hours.

At the nodes corresponding to Neusberg and Katlani the simulated data were continuously adjusted (up to the latest time at 380 hours) to the recorded values resulting in vertical discontinuities in the trajectory (main chart). These discontinuities represent the differences between the simulated results (computed just upstream of the real time nodes) and the recorded conditions (at the real time nodes). These differences are due to calibration errors and the effects of unmodelled events which have affected the flow conditions. For example the increase in flow at Katlani represents the unmodelled (and as yet unmeasured) inflows from the Vaal River, which occur just upstream of Katlani.

As the real time simulation progresses towards 380 hours the recorded flows are propagated down the river, resulting in the differences between the thick and thin lines in the main chart. At the next downstream real-time station the recorded data for that station, if available, once again replace the simulated values, resulting in another (although smaller) vertical discontinuity in the flow projection. The simulated conditions at any point on a reach are therefore dependent on the recorded data at the closest upstream telemetry station for the relevant time history of flows at that station. If for some reason the flows are not recorded at a real time station the simulated flows at the station are propagated further downstream as indicated by the thinner blue line. In essence the up-dates are propagated down the river from the real time stations as far as the next real time station, the adjustments travelling with the flow hydrograph, whose speed will vary from place to place depending on the flow and channel geometry.

In more detail, consider the crests of waves **A** and **B** on the hydrograph at Katlani and the “points” **C** and **D** on Neusberg’s hydrograph. The waves at **A** and **C** occurred earlier in time than **B** and **D** and have therefore progressed further down the river as shown in the main chart by A’, B’ C’ and D’. In the real time simulation **A’** will have reached Neusberg where the flows will again be corrected to the values recorded at Neusberg. The differences between the shapes of the hydrographs at the telemetry stations and the corresponding sections of the flow profiles are due to the fact that x-axes show time and distance respectively. In fact the hydrograph from A to B is stretched and reversed in the profile from B’ to A’.

Note that the distance travelled by crest A, to A’, is 450 km in 96 hours, that is at a speed of just over 100 km/day. In the lower reach where the river is wider and evaporation has taken its toll, the “cross-over” point C has travelled 220 km to C’ in 80 hours, slowing to about 70 km/day.

6.2 DETERMINATION OF THE RELEASE SCHEDULE

The releases from Vanderkloof Dam are currently made according to an agreement between ESKOM (the South African electricity supply utility) and the South African Department of Water Affairs and Forestry (DWAF). This agreement defines the quantity of water which can be released in the medium to long term. The current agreement is that a quota of water for a year is allocated for generation purposes on 1 May every year. This quota is dependent on the amount of water in the Orange River System on the decision date and it must be supplied at a 99.5 % assurance of reliability at a 10% load factor. This allocation for power supply can be increased within the year in the event of the dams in the system filling due to high runoff in the wet season (October to April). The Water Resource Planning Model (WRPM) is used to assess the resources of the Orange River in order to determine these quotas (BKS, 1997).

In the short term the current policy is that the releases from Vanderkloof Dam are finally determined a day in advance of their implementation and they may include 2 peaks within 24 hours. Emergency releases, however, can be made through the turbines to correct the phasing of electricity on the National Grid. These releases can be at full load for a total of 4 hours in every 24 hours, but occur with random timing.

It is envisaged that the tool developed in this study will be used for the short-term operation of the river so that the releases can be modelled and their effect on downstream flows can be assessed before a final decision on the release (not the

emergency release) is made. The effect of any emergency releases can be assessed, after they have been made and the release scenario adjusted, if necessary, to account for the effect of the emergency releases.

When a decision is to be made regarding the release schedule, the forecasting model obtains the most recent results from the real time simulation. The simulation therefore starts from the best available initial conditions. The operator of the forecasting model is able to define the hydrographs for the proposed releases from Vanderkloof Dam through each turbine in terms of quantities released and the timing of the releases.

For the practical operation of the dam, decisions regarding operation need to be made one day in advance of their implementation. The release schedule for the first day of the forecast simulation is therefore fixed by the decision taken the day before and cannot be altered, only future releases can be optimised.

As most of the demands to be satisfied are located a considerable distance from Vanderkloof Dam it will take several weeks for the releases to reach them depending on the flow conditions in the river. It is therefore necessary to simulate several weeks of flows before the forecasting runs can be assessed.

Another important effect of these travel times should be noted. If during the evaluation of a proposed release scenario, it is seen that unexpected hot and dry weather has increased the expected evaporation from the river, with the result that there will be a shortage of water at the river mouth, say for example in a week's time, it will not be possible to correct the shortfall as the next releases which are to be made tomorrow will only reach the mouth in 6 weeks time (depending on the flow conditions in the river). The users at the river mouth will however be forewarned about this shortfall, which is an improvement on the current situation.

In fact it is important to note when determining the release schedule that the results of the simulation consist of 2 parts, the forecast flows ("actual" forecast flows) resulting from the releases which have already been made and the ("proposed") flows which result from the proposed release scenario which is being evaluated. The "actual" forecast flows resulting from the releases which have already been made propagate down the river ahead of the "proposed" flows which are being evaluated. The period of the release scenario must be long enough for all the "actual" flows to propagate out of the system so that the full effect of the proposed scenario can be evaluated.

6.3 SUMMARY

A decision support system was devised to help decide how to optimise the release scenarios from Vanderkloof Dam. This decision support system consists of:

- A hydraulic model of the Orange River which is used to evaluate the proposed release scenarios from Vanderkloof Dam,
- A real time hydraulic model which is used to update the simulated conditions of the river with recorded telemetry data so that a best estimate of actual flow conditions is available for the whole river
- A graphical user interface (GUI) which controls the flow of data between the various components of the decision support system and the user.

The forecast simulations are conducted on an iterative basis, adjusting the flow releases until all the downstream flow requirements are met. During the evaluation of these proposed release scenarios it is important to simulate a long enough period so that the full effect of the proposed schedule on the most downstream users can be evaluated.

Before these forecast simulations are conducted their initial conditions must be determined. The initial conditions are a mathematical requirement of the model and represent the physical condition of the river at the start of the flow simulation. The closer the initial conditions are to the actual conditions of the river the more accurate the forecast simulations. These initial conditions are determined by updating the real time simulation of the river with the data recorded at several gauging stations along the river.

This chapter describes the concepts of the decision support system and the interaction between its various components. The practicalities of real time modelling and the forecast simulations are also discussed. These concepts are further developed in the following chapter where the GUI is described in detail.

7 GRAPHICAL USER INTERFACE

The function of the Graphical User Interface (GUI) is to co-ordinate the transfer of data between the real time telemetry network, the ISIS engines and the user. The ISIS software and the GUI are two separate software components. The GUI acts as a link between the real time data and the ISIS software. The ISIS software performs the hydraulic computations and provides the tools to view the results of the computations. The interface is just that – an interface between the data, the ISIS engine and the user.

The decision support system can be further divided into the real time station and the forecasting station. It is important to remember that the real time simulation is a separate process from the forecast simulation (See Chapter 6) and is independent of it. The results of the real time simulation are used as the initial conditions of the forecast simulations. The GUI provides the link between the real time results and the forecast simulation.

There are also two types of forecasting simulations. The first is referred to as the “Real Forecast”. This forecast is performed either manually or automatically after a real time simulation has been completed, updating the current conditions. It uses proposed releases from Vanderkloof Dam and demand information which have already been assessed and accepted. The results of this simulation represent the best estimate of what will happen on the river in the near future (in the next 24 hours), given that the releases and demands are not changed from those previously determined and included in the simulation. The second type of forecast simulation is used to test the effect of the demand scenarios and release patterns on a “what if” basis. The releases and/or demands are adjusted, the simulation is conducted and the results are assessed. The process is repeated until the user is satisfied with the release or demand scenario, as described in the introduction to Chapter 6.

The decision support system therefore consists of the following modules:

- A real time model. This is the ISIS software which uses the real time data to perform simulations. The initial conditions for the real time simulation are taken from the results of the previous real time simulation.
- A forecast model. This is the ISIS software which simulates the proposed releases from Vanderkloof Dam, which the user has supplied. The initial conditions are the results of the most recent real time simulation. There are two types of forecast simulations, one which updates the 24 hour forecast

conditions on the river and one which the user can use to assess and modify the proposed release scenarios.

- The Graphical User Interface (GUI). The GUI facilitates the exchange of data between the real time network and the real time model, between the real time model and the forecast models and between the user and the real time and forecast models and was illustrated in Figure 6.1.

For any river system which is modelled using the decision support system, it is envisaged that only one real time simulation will be conducted. Forecasting simulations can be set up on the same machine as that used for the real time simulation or at one or more remote workstations. The only limitation on the number of forecasting work stations is the number of valid ISIS licences available for use by the project team.

Before the processes in the GUI are explained in more detail it is necessary to give a basic outline of the files which are used and produced by the system.

7.1 DESCRIPTION OF THE FILE STRUCTURE

Both the file name and the file extension describe the files used by the decision support system. The file extension describes the type of file and the file name describes the scenario being analysed. The file names are input by the user but for the sake of this description the following naming convention will be used:

- REAL.*- files used in and produced by the real time model
- REAL_FORECAST.* - files used in and produced by the update of the (short-term) forecast after a real time simulation
- SCEN.*- files used in and produced by forecast simulations.

The * represents the “wild-card” character representing any file extension or filename. The files are described in this section only in terms of the data which they contain and the way in which they are used in the decision support system.

7.1.1 The ISIS Data File (*.DAT)

This is the ISIS data file that describes the hydraulic units and the physical characteristics of the river, including the cross-sectional data, the distance between the cross-sections and the roughness of the river reaches. The determination and calibration of these properties are explained in Chapter 9. Copies of this file (REAL.DAT etc) are used for the real time and forecast simulations so that the basic

data describing the physical properties of the river are identical for all simulations of the river.

7.1.2 The Remote Data File (*.RTM)

These files contain time series data used by the real time units in the ISIS simulations. The data describe the stage-time series and flow-time series for the real time units. The data contained in this file can either be the real time data as recorded on the river or they can describe the release and demand scenarios which the user wants to assess. In other words, it contains the data which are changed over the course of various analyses and can therefore not be kept in the Isis *.DAT file.

In particular, the **REAL.RTM** file is used in the real time simulation and contains the real time recorded data as well as the hydrographs describing the demands which are being abstracted from the river. The file **REAL_FORECAST.RTM** contains additional information on future releases and demands. It is expected that the proposed future demands included in the **REAL_FORECAST.RTM** will eventually be used in the **REAL.RTM**

The **SCEN.RTM** files used in the forecast simulations contain the demand hydrographs as well as the scheduled and estimated releases from Vanderkloof Dam, but exclude any real time data.

7.1.3 The Policy Files (*.POL)

The policy file describes the demand policy which is to be analysed in the system. The demand policy is a time series of abstractions. The **REAL.POL** file contains the demands which most accurately describe what is currently being abstracted from the river and what will be most likely abstracted at a future date. The **real.pol** file is used in the update of the forecast conditions. The **SCEN.POL** file contains the demands which are being abstracted during the forecast runs as well as the releases from Vanderkloof Dam which are being assessed. The GUI enables the user to easily modify the data in the **SCEN.POL** file before the simulation is made and the results assessed.

It should be noted that ISIS cannot directly access the information in the *.pol file. The information is first written by the GUI into the ISIS compatible format in the *.rtm file, which is then read by the ISIS data file.

7.1.4 The result files (*.ZZS, *.ZZN, *.ZZL, *.ZZR, *.ZZD)

The *.ZZ* files are all ISIS output files.

- **Snapshot results (*.ZZS)** describe the flow conditions at all computational nodes for a particular point in time. The **REAL.ZZS** file describes the conditions on the river at a point in time. It is the best estimate of the actual conditions at the time. These are the initial conditions for the next real time simulation or forecast simulation.
- **Unsteady results (*.ZZN)**. This file is the output from the unsteady flow simulations. It contains the flow, stage, Froude number and velocity for every saved timestep at every node.
- **Label File (*.ZZL)**. This file contains information regarding the labels used in the ISIS data file. It is produced in the unsteady simulations and is used in viewing the simulated results
- **Diagnostic File (*.ZZD)**. This file contains diagnostic information about the simulation.
- **Tabulated Results (*.ZZR)** The *.ZZR is produced during result processing and consists of tabular output of flow conditions as a function of time and position.
- **Rating Diagnostic File (*.ZZU)**. This file contains information relating to the stage discharge relationships at the control sections as well as at the Real Time units.

The **REAL.ZZ*** files describe the results of the real time simulations, whereas the **SCEN.ZZ*** files describe the results of the forecast simulations. The **REAL_FORECAST.ZZ*** files are renamed to the date at which the real time simulation ended. For example 200006012_1030.ZZN is the unsteady result file which was generated starting from the initial conditions which best describe the actual conditions at 10h30 on 12 June 2000.

7.1.5 Calibration File (*.CAL).

The time series data in the calibration file can be compared to the simulated result. The real time data are written to the REAL.CAL file so that the recorded data can be compared to the simulated data.

7.1.6 The log file (*.LOG)

A log of the processes performed by the GUI during a simulation are written to the *.log files.

7.2 THE PROCESSES OF THE DECISION SUPPORT SYSTEM

The processes performed by the GUI are illustrated in Figure 7.1, which is based on Figure 6.1 but gives more detail. The four bubbles in Figure 7.1 represents four separate locations where the required files are to be stored. The top left bubble represents the real time telemetry network and the location of the real time files. These files can be stored on either the forecast or real time workstations on a local area network. The real time files for the various weirs where data are recorded do not have to be stored in the same location.

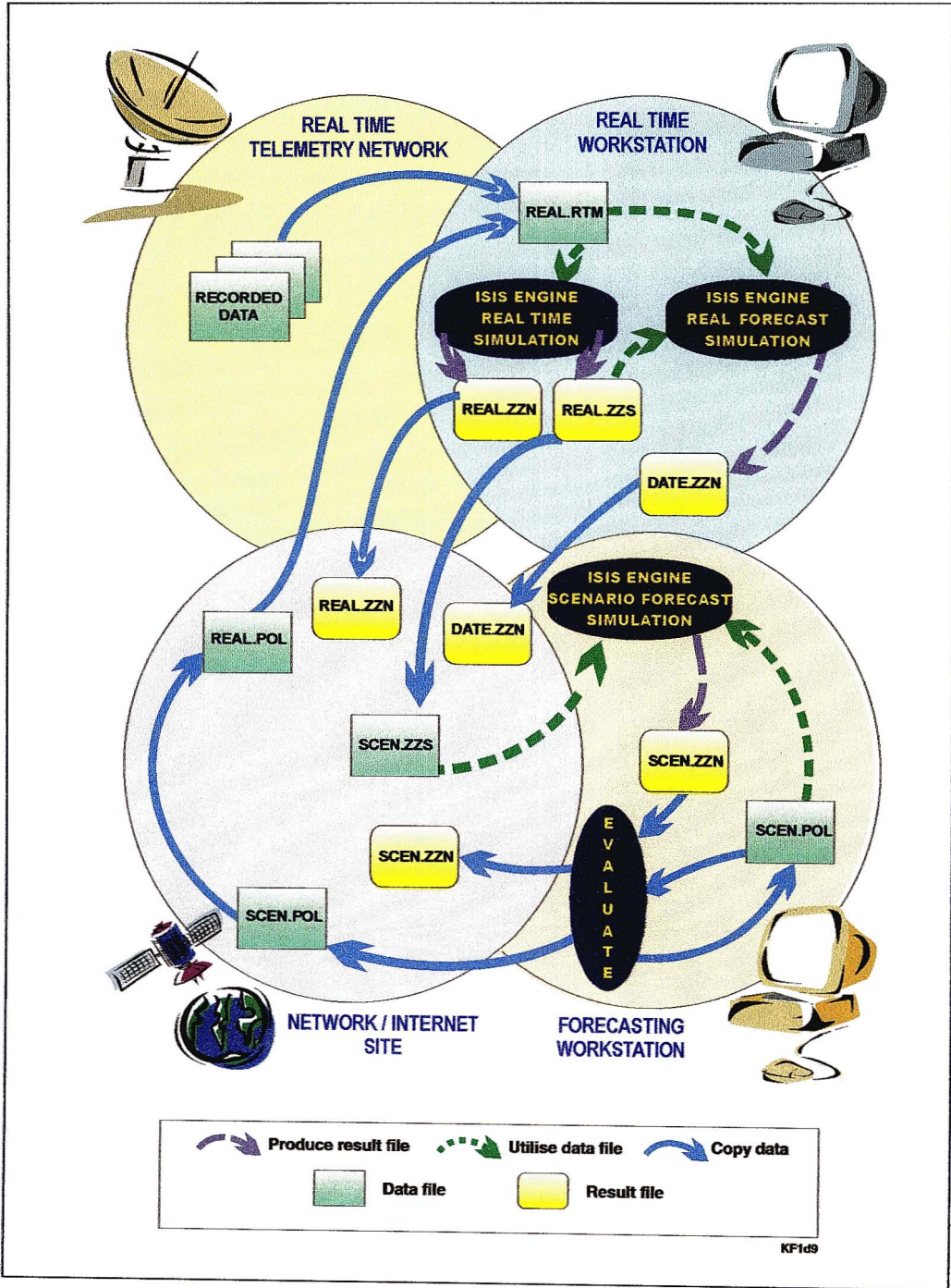


Figure 7.1: Processes of the Decision Support System

The top right bubble represents the real time work station. This is where the real time ISIS model performs the real time simulations and then updates the forecast of flow conditions.

The bottom left bubble represents the remote directory which is used by the GUI to store the result files. This directory is referred to as the “Link Directory” in the GUI.

The bottom right bubble represents the forecast workstations, where the release scenarios are simulated and evaluated.

7.2.1 Real Time Simulation

The recorded real time data files are copied to the Real Time Workstation. The files are read, the data checked and incorporated into the REAL.RTM file. Any errors and possible inconsistencies detected during this process are flagged and logged in the file REAL.LOG.

The official policy file, REAL.POL is also downloaded automatically from the Link Directory. This file contains the demands appropriate to the current month and weather conditions as well as the confirmed and estimated forecast releases from Vanderkloof Dam. Only the policy data describing the demands are incorporated into the REAL.RTM file. Any data in the REAL.RTM, which corresponds to that in the REAL.POL file is overwritten and updated in this process. If the policy file, describing the demands is altered, the changes are automatically included in the next real time simulation. It is assumed that the file is correct and the data are automatically entered into the REAL.RTM file.

Once the REAL.RTM file has been updated, ISIS simulates the flow in the river in real time mode, incorporating all real time data into the computations, up to the time corresponding to the period for which real time data are available. The results of this simulation are the initial conditions for the forecast simulations.

The real time stations are defined either as ‘essential’ or ‘non-essential’. If a station is ‘essential’ the real time simulation will not proceed beyond the period for which real time data are available. If the station is ‘non-essential’ the simulation can continue without the data. This allows the simulation to continue in cases where a station ceases transmission for some reason and no real time data are available.

Once the real time simulation has proceeded as far in time as real time data availability allows, the computed results of the ISIS simulation are written to the Link directory. The results in the REAL.ZZS file record the conditions in the river at all computational nodes for the time when the real time simulation paused. The dynamic results for the real time simulation in the REAL.ZZN file are also updated. These results can be viewed through the ISIS software.

Once the real time simulation is complete the real_forecast simulation is updated. The REAL.POL file is once again used as it describes the proposed demand and release scenarios. The REAL.ZZS file is used as the initial conditions for the run. This REAL.ZZS file represents the conditions on the river at a specific point in time, say 16 June 2000 at 00h30. The results of the real forecast are re-named to 20000616_0030.ZZN so that the date and time of the start of the forecast simulation are known. These results are then copied back to the Link Directory, where they can be accessed by interested parties.

In order to give the operator of the system control over the processes, the errors and warnings are not dealt with explicitly by the model. If, after reviewing the simulated results and the logged errors, the operator decides that incorrect data were incorporated into the simulation, the real time run can be stopped and the real time data corrected or excluded as appropriate by manually editing the REAL.RTM file. The real time simulation can then be restarted at an appropriate point in time prior to the corrected error. Using the ISIS software the REAL.ZZN, which holds the results for the simulated period, can be interrogated to produce the initial conditions for the re-started simulation.

7.2.2 Forecast simulations

Several forecast models can be set up and run independently on condition that each model has its own valid ISIS licence. Under normal circumstances, however, only one ISIS model will be needed and the forecast simulations will be performed on the same computer as the real time simulations. The forecast models are based on ISIS data files identical to that used by the real time model, although the names of these files must differ to prevent the real time results being overwritten by the forecast results.

At the start of the forecast simulation the REAL.ZZS file is downloaded and renamed by the GUI to match the relevant ISIS (Forecast) data file name. These data describe

the most up to date conditions of the river and are used as the initial conditions of the simulation.

Various release policies and demand conditions can be written into *.POL files using the GUI. These scenarios can then be simulated and the results assessed using the ISIS workbench.

The *.POL file, describing the release scenarios and demand patterns, along with the corresponding simulated results can then be copied to the link directory so that other users can also evaluate them. Included with these files is an ASCII data file containing information regarding the author of the files and the run descriptions.

Once a scenario has been selected the *.POL file can be copied to the REAL.POL file so that the appropriate demand data can be incorporated into the real time simulation. This is necessary because information describing the abstractions to satisfy the demands is not included in the real time telemetry network.

7.3 SCREENSHOTS OF THE GRAPHICAL USER INTERFACE

The graphical user interface was designed to be as simple as possible. Before a run can take place the user is only prompted for the data file and directory information and the ISIS run parameters.

As the program initialises the user must log on and choose the type of run which is to be performed. If the first option is chosen, only the real time simulation will be updated and the results will be stored on the Network Site. If the second option is chosen only forecast simulations will be performed. The most up to date results from the real time simulation will be downloaded from the Network Site, where they are stored before the forecast simulation takes place. The third option updates the real time simulation. Immediately after this simulation is complete the "real forecast" simulation is conducted.

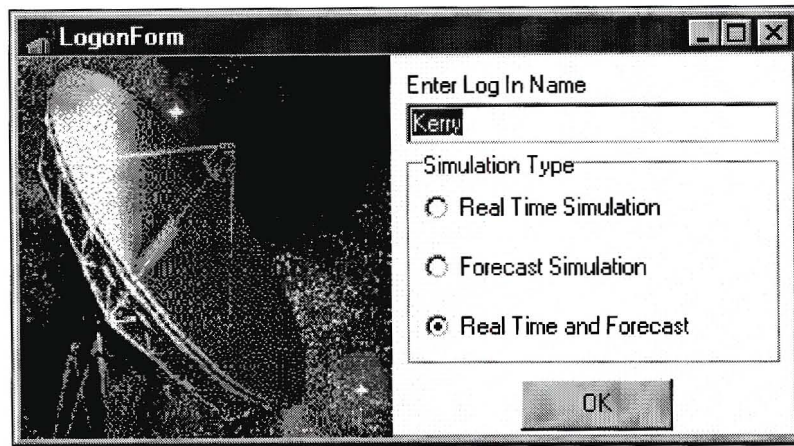


Figure 7.2: The logon Form

After the user has logged on the Task Bar appears (Figure 7.3). The task bar gives the user access to the tasks which must be performed. The tasks for setting up the simulations must be performed in the sequence explained below or left to right within the groups on the task bar. The menu items for some of the tasks are inactive until all the required preceding tasks have been performed.

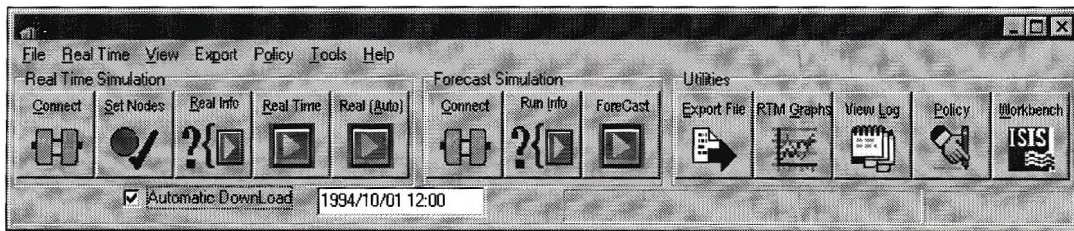


Figure 7.3: The Task Bar

The Task Bar is divided into three sections: the real time simulation, the forecast simulation and the utilities. The procedures to start the real time and forecast simulations are shown in Figures 7.4 and 7.5 and are discussed in Sections 7.3.1 and 7.3.2 respectively.

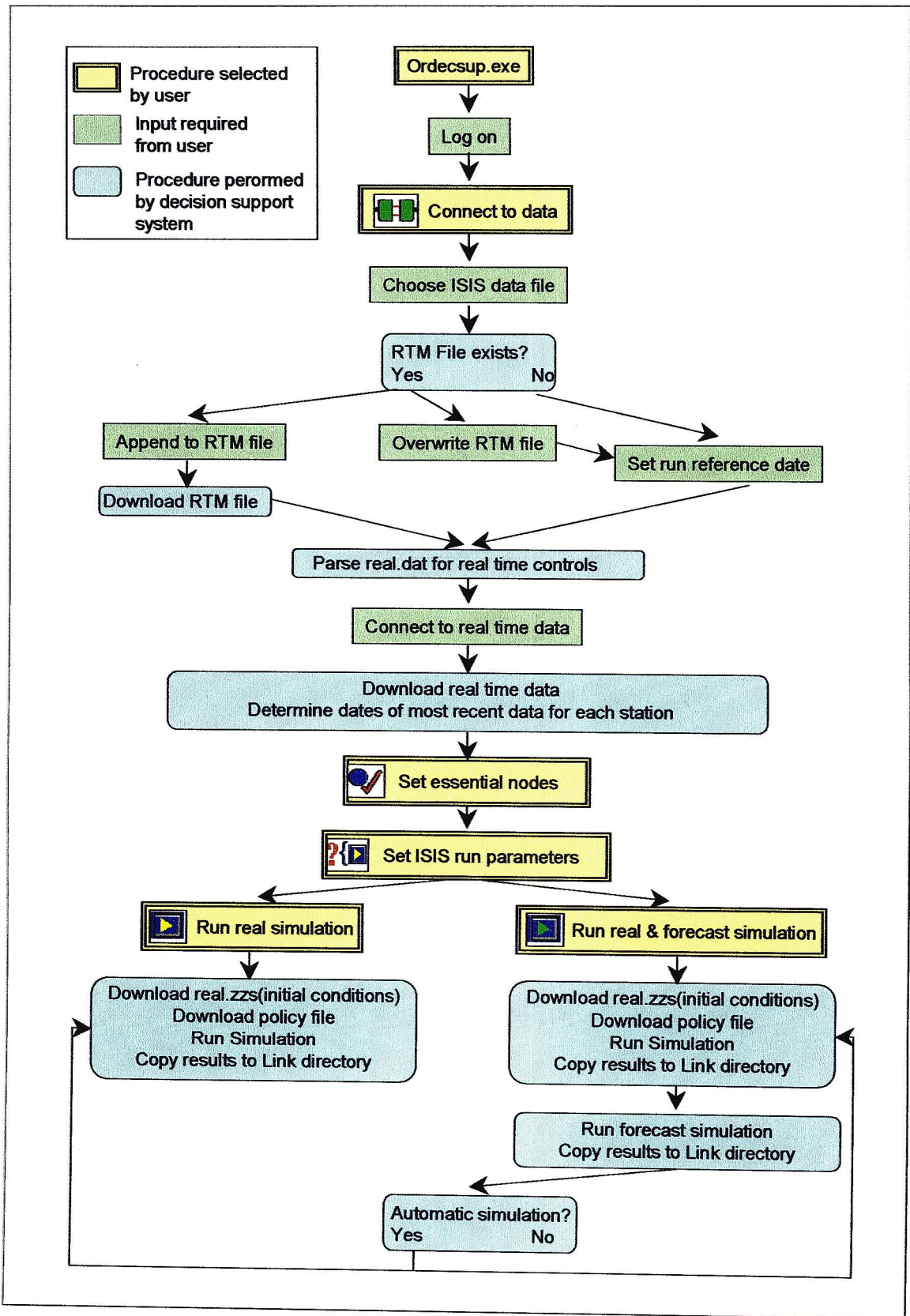


Figure 7.4: Procedure to start a real time simulation

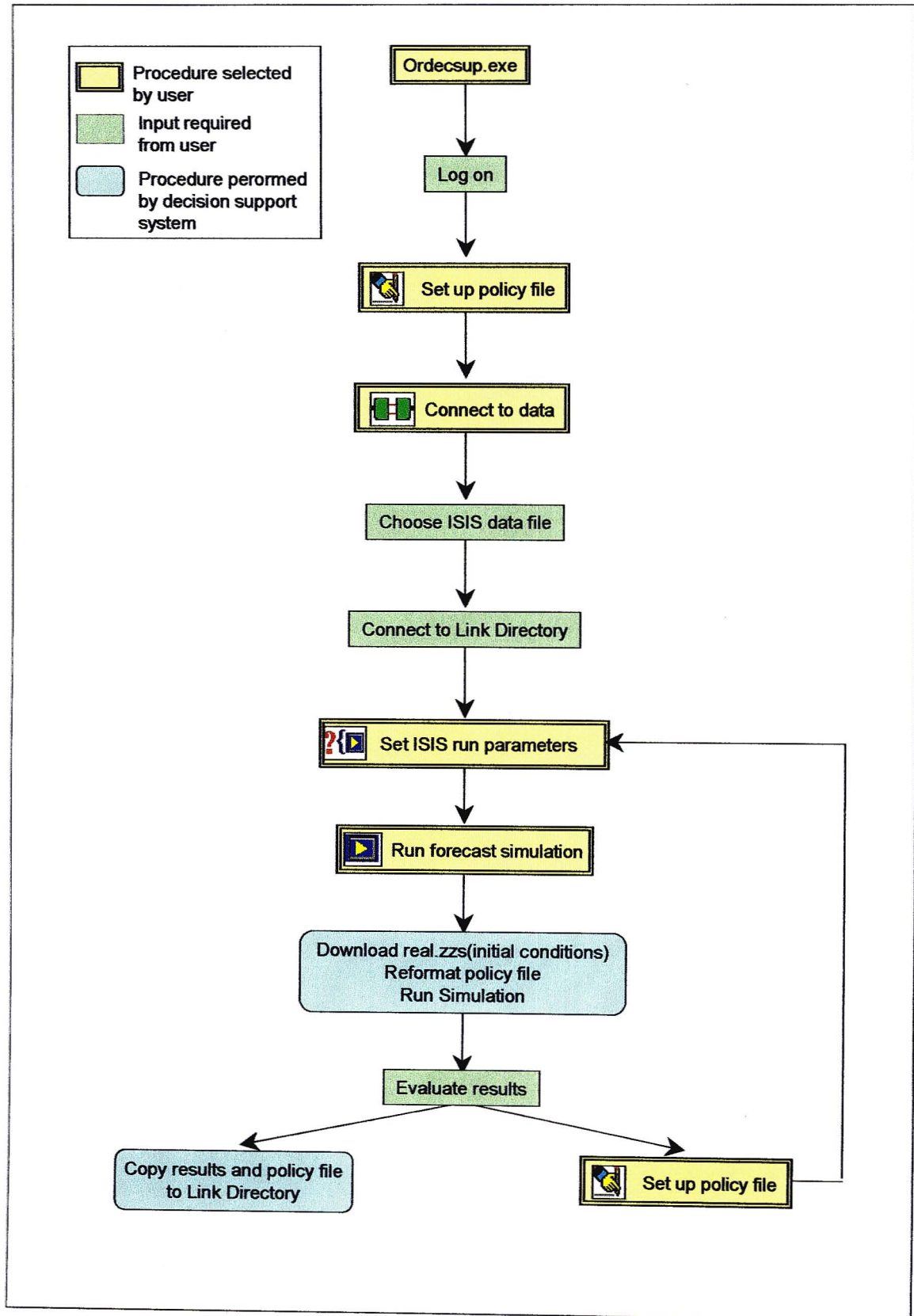


Figure 7.5: Procedure to start forecast simulation

7.3.1 The real time simulation

The first task is to connect to the relevant data files. This is initiated by clicking on the



Connect Button. The user is required to select the relevant ISIS data file as shown in Figure 7.6

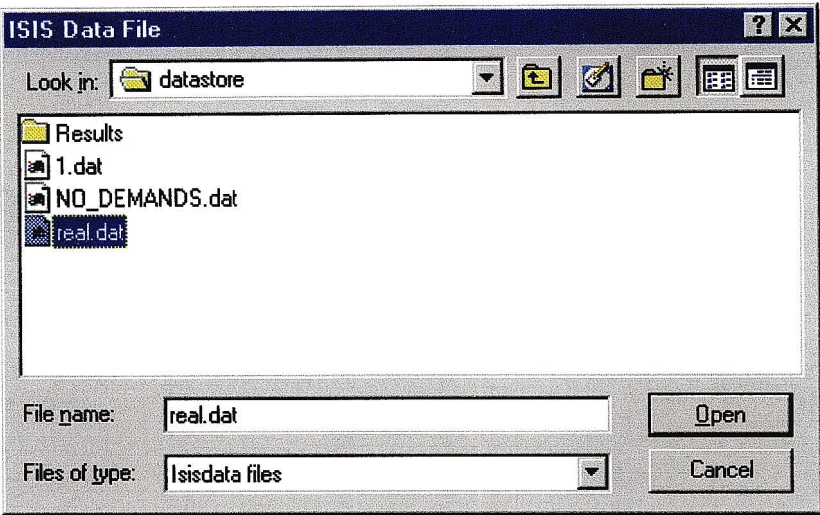


Figure 7.6: Selecting the data file

Once the data file is selected the program searches for a corresponding RTM file. If a file is found the user can choose to either overwrite the RTM data, or to append the new information to the existing RTM file (Figure 7.7). The date shown on the form is the reference date of the initial real time simulation. All results given by and data input to the ISIS hydraulic engine are in hours from the start of the initial run, or hours from the reference date. If the user chooses to overwrite the RTM file, or if no RTM file was found, a new reference date must be input. This date should then correspond to the date and time that the simulation is to start. This would correspond with the date and time of the most recent real time simulation update (the previous update), which would provide the initial conditions for the start of this real time simulation. The date shown on the task bar (Figure 7.3) corresponds to the date on this form.

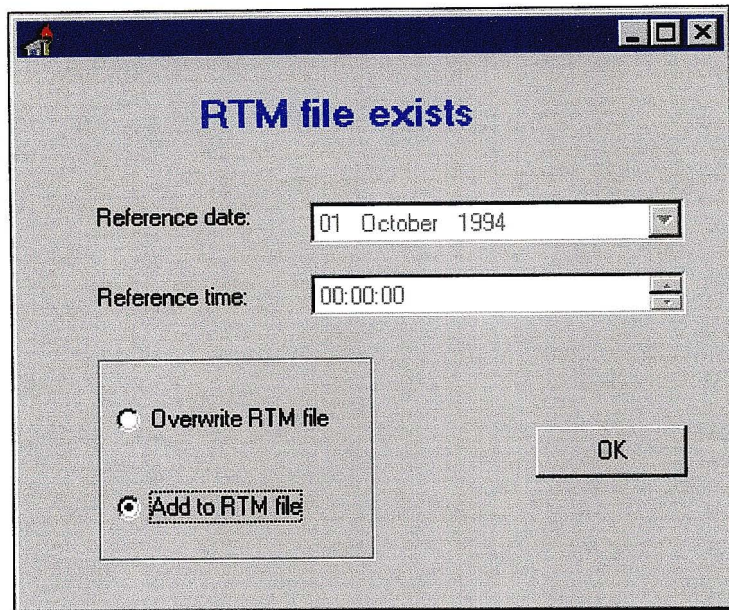


Figure 7.7: Setting the RTM file data

The next form which appears (Figure 7.8) requires the user to connect the real time nodes in the model to the relevant data directories. All the real time nodes as well as the “Link” directory are listed in the top left box (Note that the top left box is empty in the example as the nodes have already been connected as described below). The “link” directory is the Network Site where all the real time results and policy files are stored (Figure 7.1).


The user then selects some (or all of these nodes) and by clicking on the >> (or >>>> for all the nodes) transfers them to the top right hand box. The nodes which appear in the top right hand box are the nodes which will be connected to the directory selected when the “Network” button is clicked. Once the “Network” button has been clicked and the relevant directory has been selected, the nodes and the directories are displayed in the bottom box. The program will then search for the real time data files in the directory displayed. It is therefore evident that the program is very flexible in terms of where the real time data is kept. This was to facilitate the fact that the data might not all come from the same source (for example the real time data for the gauging stations is obtained from DWAF, whereas the data for the releases from Vanderkloof Dam may be supplied from ESKOM)

Connected Nodes			Connection
4DC010M1	(D3H023M1)	C:\Delphi\ods-oopV\datastore	
4DC010M2	(D3H023M2)	C:\Delphi\ods-oopV\datastore	
3BC015G	(D3H008)	C:\Delphi\ods-oopV\datastore	
2AA090G	(D7H008)	C:\Delphi\ods-oopV\datastore	
0DA005G	(D7H014)	C:\Delphi\ods-oopV\datastore	
7DC010G	(D8H003)	C:\Delphi\ods-oopV\datastore	

Figure 7.8: Connecting the model to the real time data

If one or more connected nodes are selected from the bottom box and the “Disconnect” button is clicked the node will be disconnected from its real time data and it will not receive any real time data for the simulation. It will then appear back in the top left hand box, indicating it has not been connected to any real time data.

The second step in setting up the real time simulation is to select the real time nodes for which real time data is required for the simulation to continue. On selecting this

button  on the taskbar the Essential Node Selection Form is displayed (Figure 7.9)

Each of the real time nodes is listed on this form, along with the most recent date for which real time data is available. The DWAF gauge number and the name of the gauging station are also shown. An essential node is considered to be a node where additional real time data must be available before a simulation can continue. The user must select these essential nodes from the list.

If a node is not selected the simulation will continue even if there is no data available for this node. When the simulation is begun the program downloads the latest real time data and determines the latest data for each of the essential nodes. The program then loops through the list of selected nodes to determine the length of the simulation. If there is no new real time data available at a selected gauging station a message will appear saying that the simulation cannot continue until more real time information is available.

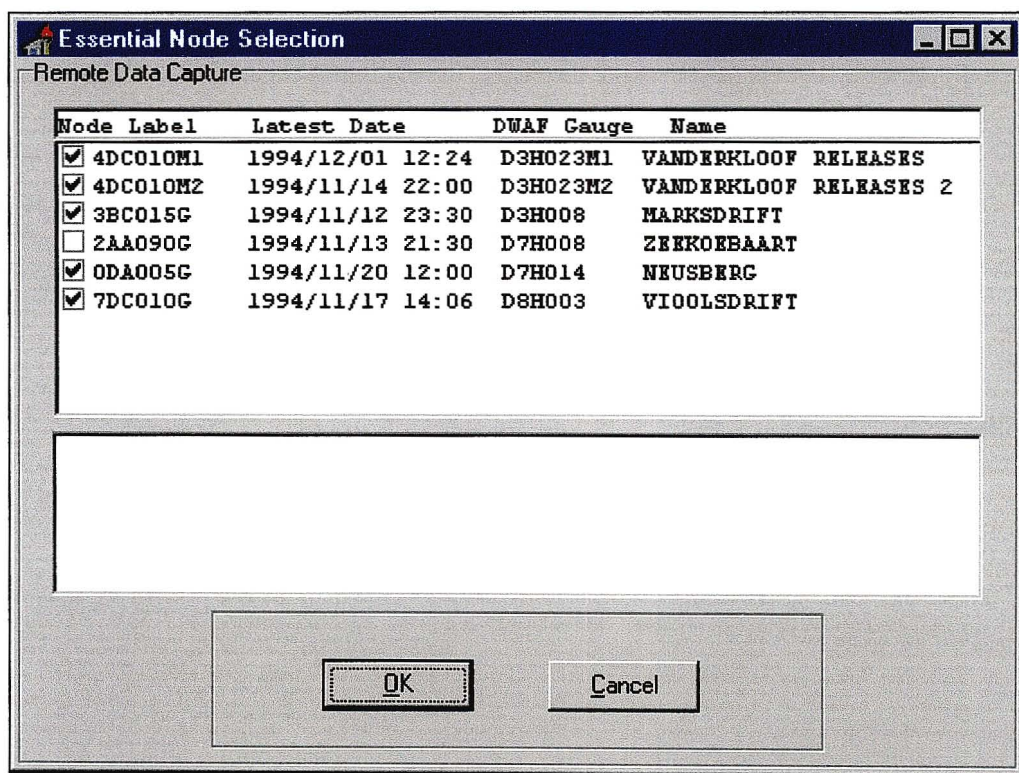


Figure 7.9: Selecting the nodes for which real time data is required

The next step accessed from the toolbar is to set the ISIS run time parameters



This is shown in Figure 7.10.

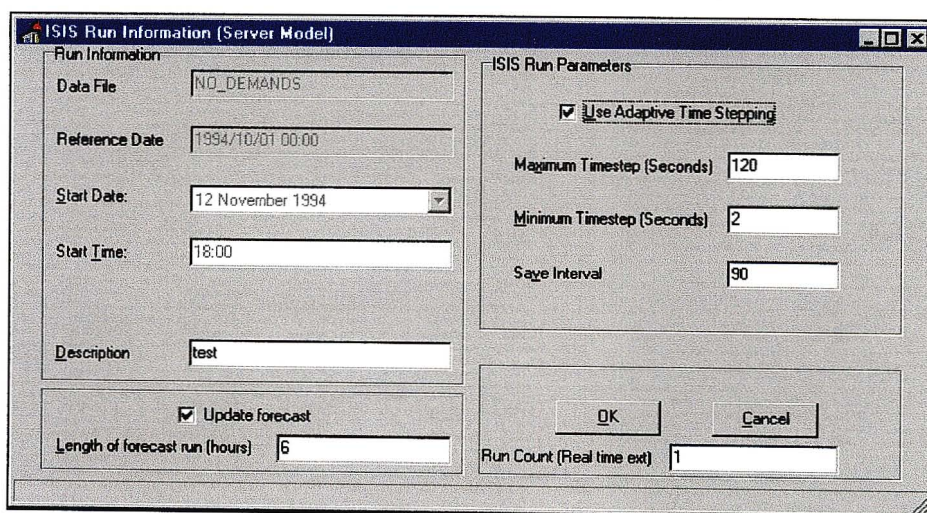


Figure 7.10: Setting the ISIS run time parameters

The data file, reference date and the start date and time are displayed but cannot be changed as they are dependent on the previous simulations. The user can enter a brief description of the simulation and can choose whether the real forecast simulation should be updated after the real time simulation is complete. If the real

forecast simulation is to be updated the length of this forecast simulation must be specified (in hours).

The user must also specify whether adaptive timestepping must be used, the timestep information (in seconds) and the interval for which the results must be stored (in multiples of the maximum timestep). The run count indicates how many times the real time simulation will have been updated when this update is complete.

Finally the real time update simulation can be started by clicking on the “Run Button”



on the toolbar. If this button is selected only one update is completed.



Alternatively the “Auto Run Button” can be selected. If this button is selected the program automatically downloads the real time information and updates the real time simulation (and real forecast simulation) at user specified intervals.

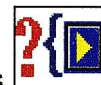
7.3.2 The forecast simulation

The forecast simulation is set up in much the same way as the real time simulation.



First the user must connect to the Network Site, where the real time simulation results are stored (Figure 7.1). The same form as shown in Figure 7.8 is used to do this, except that in the case of the forecast simulation only the “Link” to the real time results needs to be connected as shown in Figure 7.11.

Once the connection is made the results of the most recent real time simulation are automatically downloaded from the project network site (“Link” Directory). The



following step is to set up the ISIS run time parameters, in the same way as shown in Figure 7.10. The start date and time for the forecast simulation cannot be set by the user as it is automatically set to the date and time of the latest real time simulation results. The user needs to set the timestep parameters and the length of the forecast simulation.



The forecast simulation can then be started

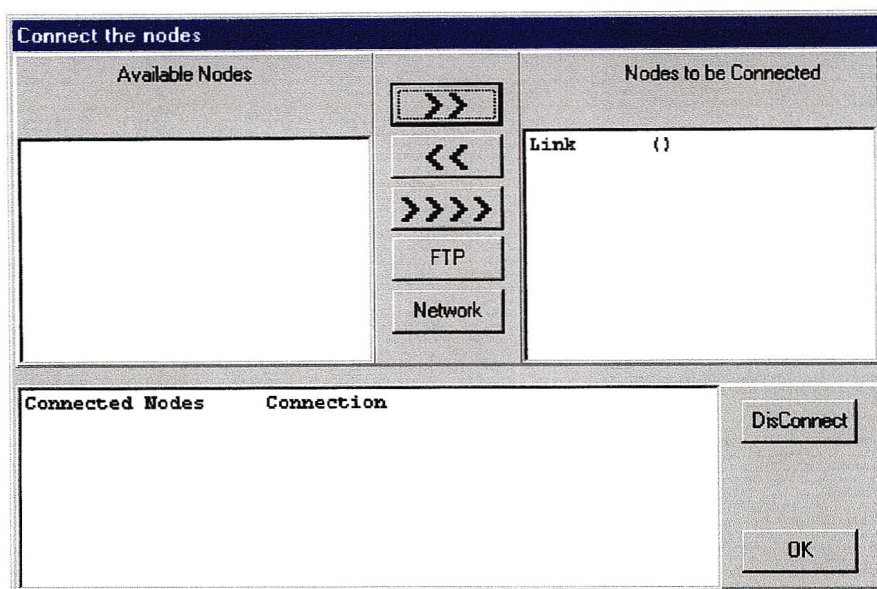




Figure 7.11: Connecting the forecast simulation to the real time results

7.3.3 The Utilities

Several utilities were coded to enable the user to easily manipulate the data and review the simulation results.

The “Export Button”  allows the user to copy the results and policy files to a directory other than the link directory.

During the real time and forecast simulations a log of all the events is kept. On


viewing this log  the user can determine if any errors occurred and get the details of these errors as shown in Figure 7.12.

A policy can be set up for any of the nodes in ISIS defined as QTREAL. These nodes are listed in the top left hand drop down box (currently showing “4DC010M1 VANDERKLOOF”). On selecting one of these nodes the data for the policy is displayed in a grid format and the chart is drawn.

The users must select the unit of time for the information and define whether the information should be repeated in cycles or extended past the time period defined.

There are several buttons below the information grid allowing the user to easily edit the data and if the “Advanced” button is selected the user can edit the ASCII policy data file directly.

The user can also choose to graphically view any of the data records in the RTM file

 Figure 7.14 shows the releases from the two turbines at Vanderkloof Dam, which are included in the RTM file.

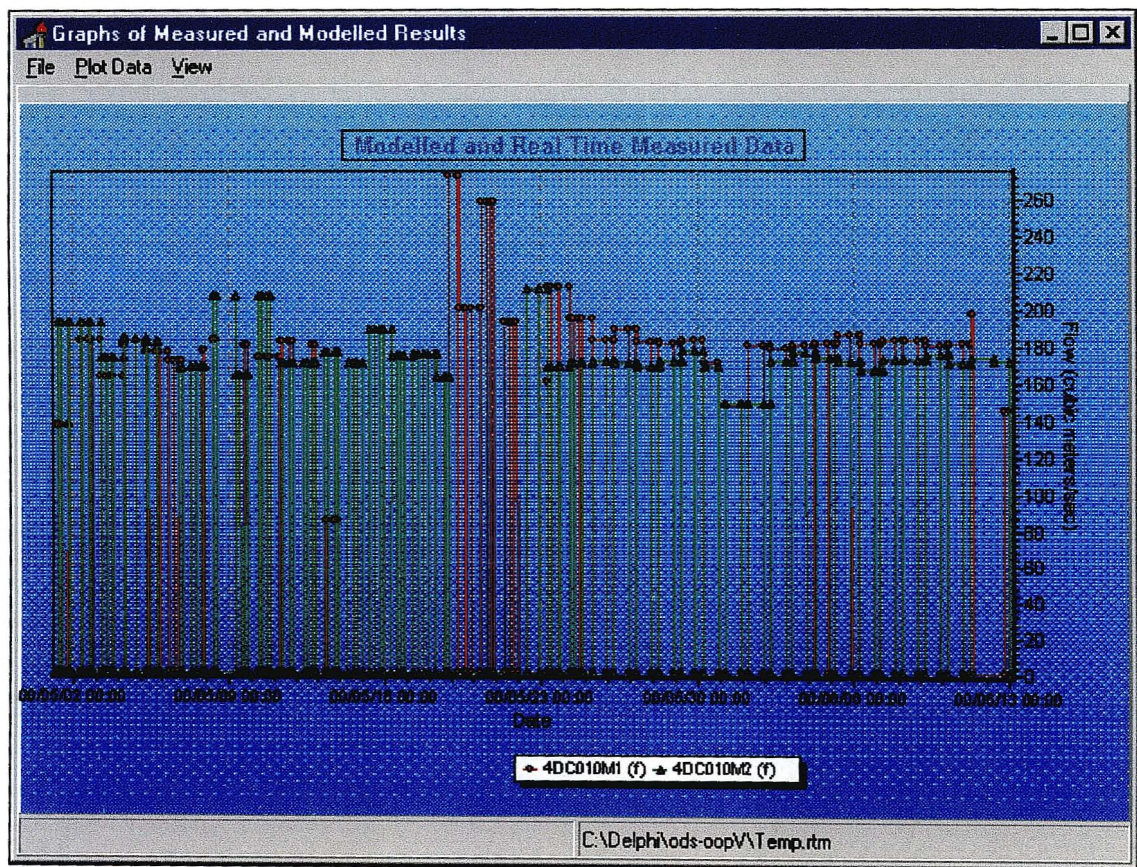



Figure 7.14: Vanderkloof Dam data recorded in the RTM file



Finally the ISIS workbench can be viewed . The ISIS workbench enables the user to interrogate the ISIS results. The user is referred to the ISIS manual for more information on the functioning of the workbench (Halcrow & HR Wallingford, 1995).

7.4 SUMMARY

This chapter described the functioning of the Graphical User Interface (GUI) and follows on from the previous chapter where the basic concepts of the decision support system are explained.

The decision support system consists of the following modules:

- A real time model, which is the ISIS model which uses real time data to perform hydraulic simulations to produce the initial conditions for the forecast simulations.
- A forecast model, which is the ISIS model used to simulate the proposed releases from Vanderkloof Dam. There are two types of forecast simulations, one which updates the 24 hour forecast conditions on the river and one used to assess and modify the proposed release scenarios.
- The Graphical User Interface (GUI), which co-ordinates the transfer of data between the real time telemetry network, the ISIS engines and the user.

The nomenclature of the data and results files used in the GUI is described. The file name describes the scenario being analysed and the file extension describes the type of result or data file.

The processes performed by the GUI are described in this chapter. The data and result files are stored in 4 separate locations:

- The real time telemetry network and the location of the real time data files
- The real time work station, where the real time ISIS model performs the real time simulations and then updates the forecast flow conditions.
- The "Link" directory, where the result files are stored
- The forecast work station, where the release scenarios are simulated and evaluated.
- The transfer of data and results between these locations is described. Finally screenshots are used to describe how to use the GUI.
- In the preceding chapters the theory of modelling open channel flow is described (Chapters 2 to 4) and the developments made to the hydraulic

model in order to model the Orange River in real time are discussed (Chapter 5). The basic concepts of the decision support system are explained (Chapter 6) and the functioning of the GUI is described (Chapter 7). In the following chapters the real time telemetry network on the Orange is described (Chapter 8) and the population and calibration of the hydraulic model of the Orange River is discussed (Chapter 9) before the decision support system is assessed (Chapter 10).

8 THE REAL TIME TELEMETRY NETWORK

In 1993 the World Meteorological Organisation (WMO), in association with the World Bank, started to promote a World Hydrological Cycle Observing System (WHYCOS) to enable the development of continuously updated data bases of various hydrological parameters one of which is river flow. The system is based on a global network of reference stations with real-time satellite-based data transmission.

WHYCOS is made up of interconnected regional Hydrological Cycle Observing Systems (HYCOS) sub-components which are managed and supported by the national hydrological authorities in the regions. The national authorities are the custodians of the information which they create. South Africa is included in the Southern African Development Countries (SADC) HYCOS with the Department of Water Affairs and Forestry (DWAF) being in control of all hydrological data in South Africa. One of the main HYCOS components is a network of key stations used for the collection and transmission of several variables related to water resource monitoring. Several stations on the Orange River are included in the HYCOS agreement.

These stations are equipped with multi sensor Data Collection Platforms (DCP's) which transmit data by means of satellite technology. The data are collected and recorded in Germany and then re-transmitted to the Department of Water Affairs and Forestry (DWAF) head office in Pretoria every hour, containing 5 readings for the hour.

The use of cell phone technology has provided another means of data transmission and DWAF has implemented this technology at several sites on the Orange River. The result is that a database of real-time flow conditions is available at the DWAF head offices in Pretoria.

The real-time data which are collected at DWAF head offices are available on the Internet. The data recorded at the real time gauging stations are verified before they are displayed on the internet. The internet data only include the average reading for every hour and are therefore not suitable for the purposes of this model where, given the nature of the releases from Vanderkloof Dam, shorter time intervals are required. However, the original recorded data are available from DWAF and can be e-mailed to any interested parties. The process of data acquisition is illustrated in Figure 8.1. Although the original intention was for the Graphical User Interface to obtain the data via the Internet using FTP, the data are not yet available on the Department of Water

Affairs and Forestry's FTP site. This is mainly due to security reasons as the department's firewalls limit the access of the FTP site.

The stations included in this real-time network are indicated in Figure 8.2 and their details are given in Table 8.1

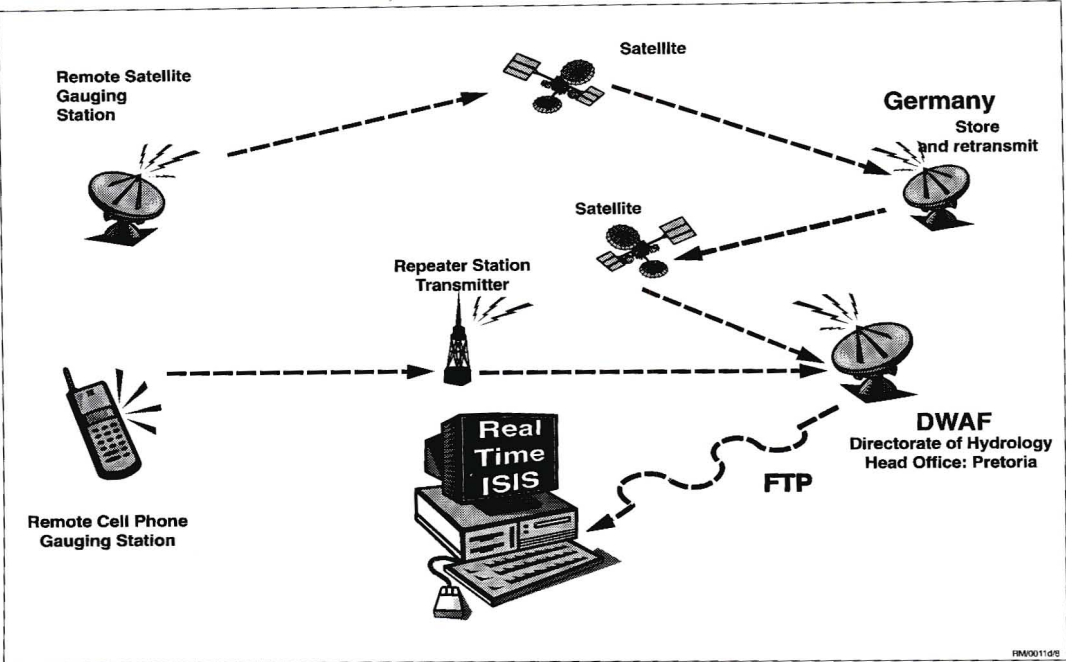


Figure 8.1: The real time telemetry network

Table 8.1:Stations on the Orange River included in the telemetry network

Station name	DWAF Gauge Number	Distance from Vanderkloof Dam (km)	Measured component	Type of transmission
Vanderkloof Dam	D3H023	0	Flow	Satellite
Doorenkuilen	D3H012	0.7	Stage	Satellite
Marksdrift Weir	D3H008	174	Stage	Cellular
Douglas Weir	C9R001	(Vaal River)	Stage	Cellular
Katlani	D7H007	190	Stage	Cellular
Boegoeberg	D7R001	471	Stage	Satellite
Zeekoebaart	D7H008	473	Stage	Satellite
Neusberg	D6H014	703	Stage	Cellular
Violsdrift	D8H003	1 097	Stage	Satellite
Brandkaros	D8H007	1 362	Stage	Satellite

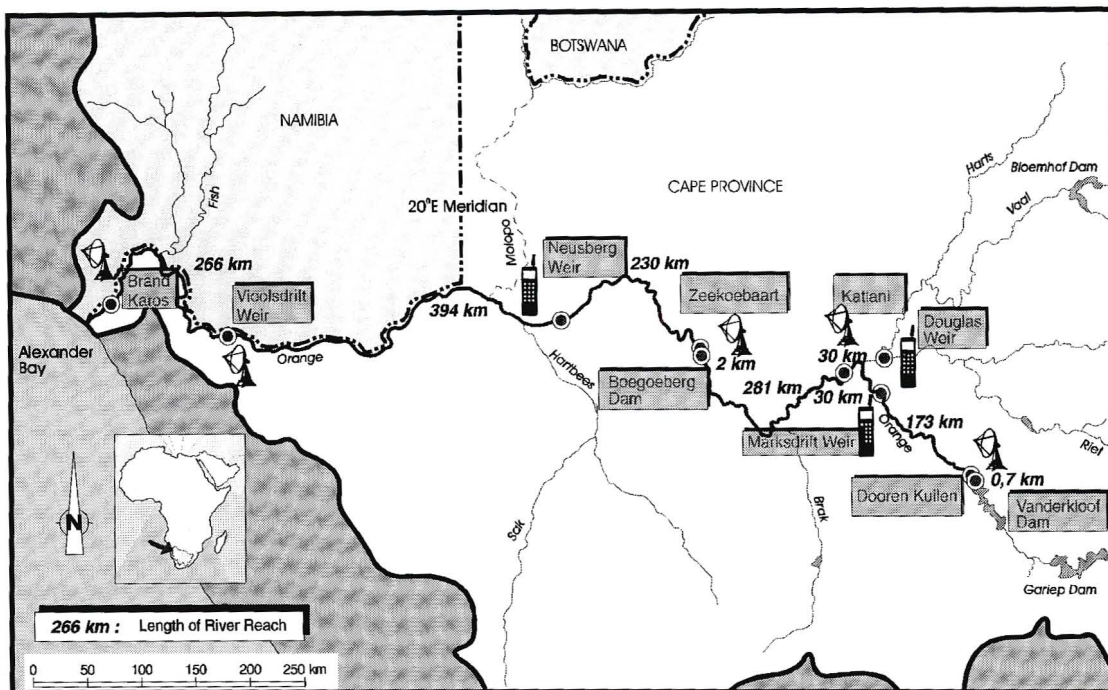


Figure 8.2: Position of real time gauging stations

The real time data which were available for the period 1 Jan 2000 to 1 December 2000 are illustrated in Figure 8.3. It can be seen that there are problems with the data stream and an error check was included in the Graphical User Interface, where a reading is flagged if it differs from the previous reading by more than a user specified amount. The zero readings are also discarded during this checking process.

It is interesting to note that the flows at Katlani are greater than the flows at Marksdrift. This is due to the inflows from the Vaal River.

A further important point to consider is that the releases from Vanderkloof Dam are not included in the network of real time stations. The releases from Vanderkloof Dam used in this study were obtained from records kept by Eskom. Due to the position of the turbines at the dam wall, it is apparently not physically possible to record the releases in real time. For this reason the gauge at Doorenkuilen (it is not a weir) was linked to the network as it is immediately downstream of the dam.

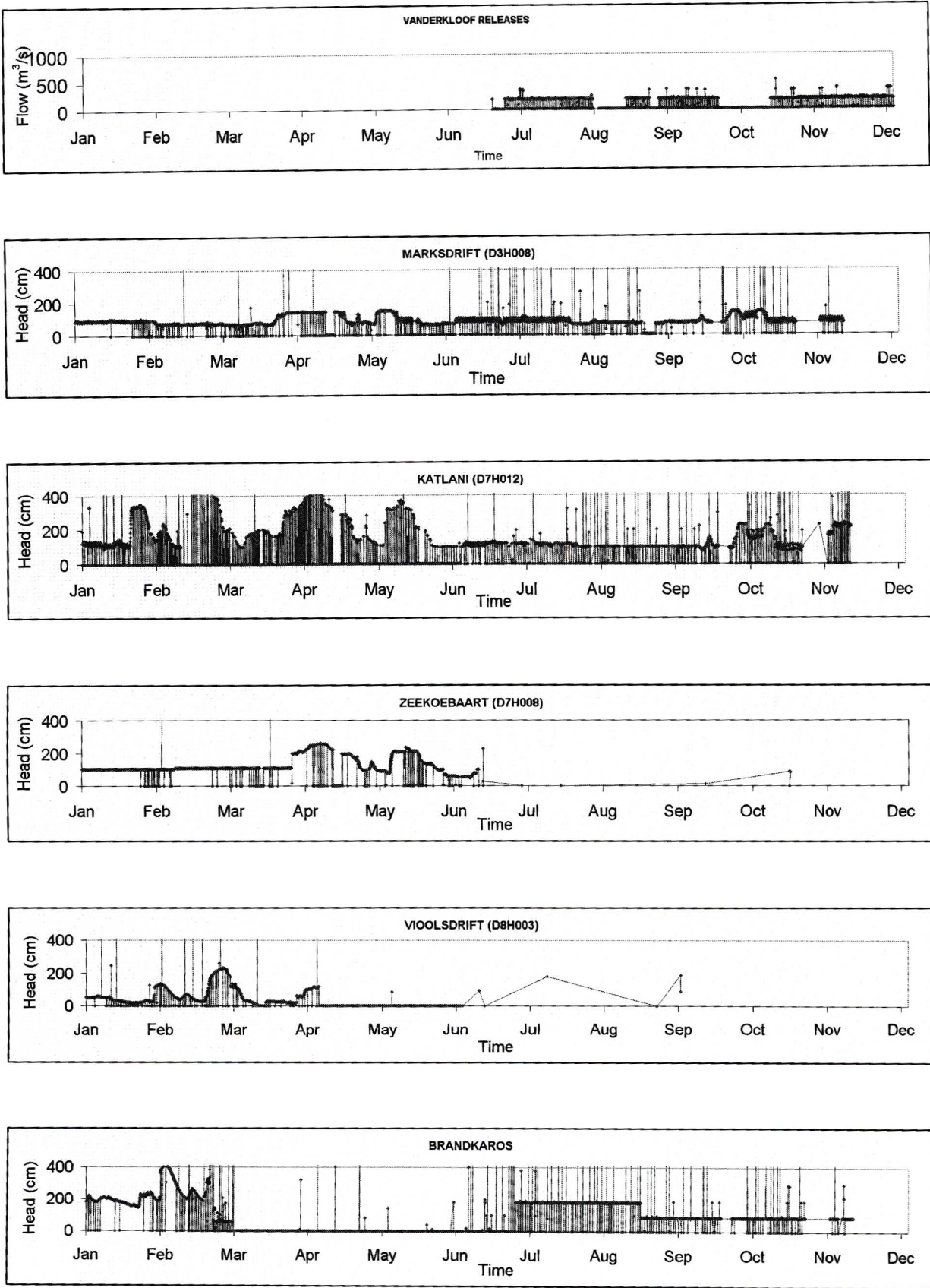


Figure 8.3: Real time data for period 1 January - 1 December 2000

However, there are two problems with utilising the records obtained from Doorenkuilen. Firstly the readings are available only in hourly increments. The exact times that the turbines at Vanderkloof Dam are opened and closed are therefore not recorded. Secondly, there is no record available from the station yet.

Doorenkuilen is not a weir but a gauging site on the river. It was modelled in ISIS as a RIVER section, 100m downstream of Vanderkloof Dam. The modelled stage discharge relationship was extracted from the simulated results and is shown in Figure 8.4. It is clearly evident that the stage-discharge relationship at this site is influenced by the releases from Vanderkloof Dam. As is usual in rapidly varying flows, the stage-discharge relationship differs on the rising and falling limb of the hydrograph. It is therefore of concern that using the recorded stage will not be sufficiently accurate for determining the actual releases.

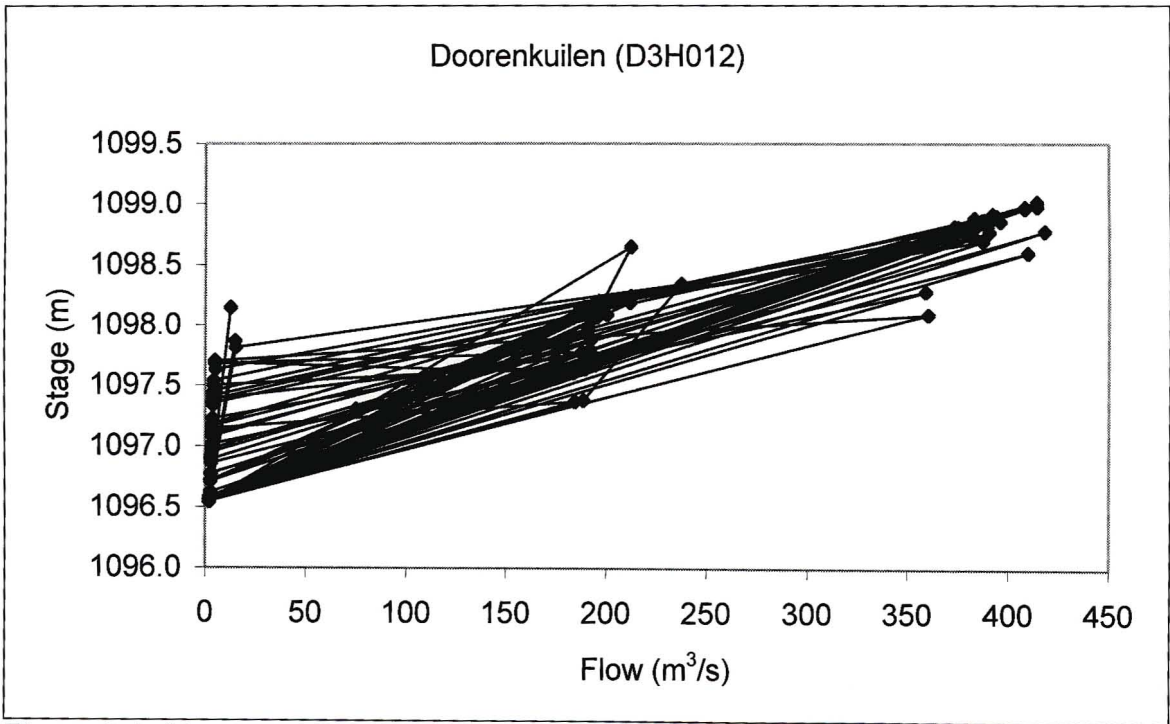


Figure 8.4: Simulated stage discharge relationship at Doorenkuilen

The releases from Vanderkloof Dam are not included in the real time telemetry network. As an alternative it was intended by DWAF that the stage recorded at Doorenkuilen, which is 100m downstream of Vanderkloof Dam, should be used to determine the releases from Vanderkloof Dam. However, to date, Doorenkuilen has not been included in the real time network and as the stage discharge relationship at Doorenkuilen is not linear (due to the rapidly varying flows released from Vanderkloof Dam) it may not be possible to use this station to determine the releases from

Vanderkloof Dam. The releases from Vanderkloof Dam used in this thesis were obtained directly from ESKOM, but they are not available in real time.

8.1 SUMMARY

This chapter describes the real time telemetry network in operation on the Orange River. Real time data are recorded at several gauging stations on the Orange River and the data is transmitted via satellite to the Department of Water Affairs head office in Pretoria where they are archived.

The data recorded at the real time gauging stations contain 5 readings in every hour. These data are verified before they are displayed on the internet. The internet data only include the average reading for every hour and are therefore not suitable for the purposes of this model where, given the nature of the releases from Vanderkloof Dam, shorter time intervals are required. However, the original recorded data are available from DWAF and can be e-mailed to any interested parties.

There are several problems with the real time data which are discussed in this chapter. These problems include the fact that the data recorded at Doorenkuilen cannot be used to record the releases from Vanderkloof Dam and the Vanderkloof releases are not part of the real time telemetry system. Other problems include incomplete records due to faulty transmitters.

Up to this point in the dissertation various aspects of the decision support system have been described:

- The theoretical aspects of modelling open channel flow were discussed, in Chapters 2 to 4
- The modifications made to the ISIS hydraulic engine were described in Chapter 5.
- The conceptual aspects of the decision support system and the working of the Graphical User Interface (GUI) were described in chapters 6 and 7.
- The real time telemetry network of gauging stations was described in this chapter.

The final component of the decision support system, the hydraulic model of the Orange River, is described in detail in the following chapter and its assessment will be made in Chapter 10.

9 THE HYDRAULIC MODEL

This chapter describes the hydraulic model of the Orange River as defined for ISIS. The selection of the mathematical model used for the study is justified in Chapter 3. The method used to set up the model and the calibration of the model are discussed in this chapter.

9.1 PREPARATION OF THE HYDRAULIC MODEL

In order to model the river using the full St Venant equations, data describing the river are required. These data include cross sectional information as well as information on the roughness of the river reaches. Information on abstractions from the river and losses from the river are also required.

A considerable amount of the data capturing for the model was done during the Orange River Losses Study (McKenzie and Craig, 1999). This included setting up and calibrating a model of the lower reaches of the Orange River. The cross-sectional data for the remaining portion of the river was captured by a team of BKS (Pty) Ltd employees. The candidate was responsible for combining this information into one model of the entire length of the Orange River downstream of Vanderkloof Dam, including the detail of the real time stations and calibrating the model against the recorded flow at each of the real time stations.

9.1.1 Cross sectional data

The spacing and resolution of the cross sectional information has an influence on the accuracy of the simulated results. Cross sectional information should be included at the points in the river where flow conditions change, such as at weirs or where large changes in cross section occur. Where the hydraulic properties of the river remain constant, interpolated sections were used in the model. Although these sections contain no new cross sectional data they are included to improve the calculation of the water surface level.

Considering that the section of the river being modelled is 1 400km long, relatively few sections have been physically surveyed. The surveyed information was used where available but in general most of the cross section information was determined from contour maps and aerial photographs.

The drawback of using contour maps and aerial photography is that the shape of the river below the water surface is not known. This information had to be interpolated from the information available above the water surface level and from local changes

to flow which were visible in the aerial photographs. The channel properties were then calibrated by comparing the width of the water surface of the simulated flows to those shown in the aerial photographs which were taken during similar flow conditions.

9.1.2 Channel roughness

Channel roughness was estimated using information obtained from photographs as well as visits to sections of the river. In general, however, the channel roughness, expressed in terms of Manning's equation (Manning's n) was calibrated against recorded flow.

9.1.3 Real Time Units

The real time units include a stage-discharge relationship of the weirs. These relationships were determined by the Department of Water Affairs and Forestry by means of flow gauging exercises and are shown in Figure 9.1.

It can be seen from the charts that for several of the weirs the flow changes considerably for a small change in stage. This is due to the fact that these weirs were not designed for low flow measurements and have wide cross sections. Neusberg Weir is wide enough to require two gauging stations. The stage-discharge relationship for these stations were combined for this study. The initial flows at Neusberg are due to a fish ladder in the weir.

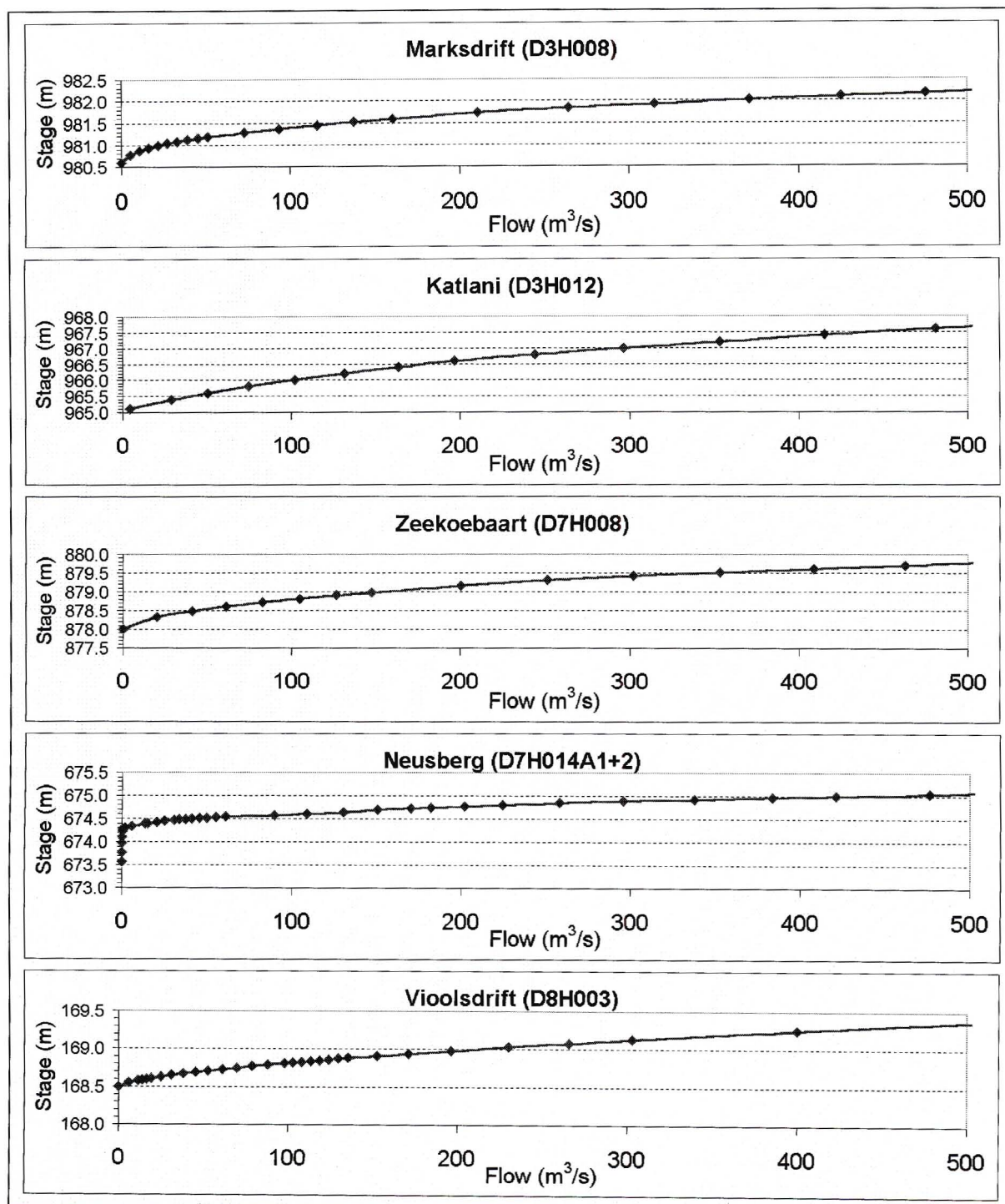


Figure 9.1: Stage-Discharge Relationship for Real Time Gauging Stations

9.1.4 Summary of the physical nature of the Orange River downstream of Vanderkloof Dam

The Orange River downstream of Vanderkloof Dam is divided into a number of reaches by control structures. The physical characteristics of these reaches are listed in Table 9.1 and a longitudinal sections of the river showing the bed level and roughness (Manning's n) are shown in Figure 9.2. The maximum and minimum velocity and Froude numbers are shown in Figure 9.3. (These data correspond with

the flows of Figure 9.20). Photographs of some of the weirs and typical river sections are shown at the end of this section.

Table 9.1: Summary of data on the 14 river reaches between control sections

Description	Upstream Control Type	Upstream Chainage (m)	Length (m)	Ave Slope 1:x	Average Rough (n)	No River Sections (measured)	No Inter-mediate (Δx)
Vanderkloof Dam to Marksdrift Weir	Real Flow-Time	0	174400	1469	0.036	6	145
Marksdrift to Vaal River Confluence	Real Flow-Stage	174400	15300	2684	0.032	1	11
Vaal River Confluence to Katlani	Real Flow Time	189700	15000	15	0.038	0	9
Katlani to Boegoeberg Dam	Real Flow-Stage	204700	266300	3201	0.026	4	169
Boegoeberg Dam to Zeekoebaart Weir	Round Nosed Weir	471000	2000	2	0.038	0	5
Zeekoebaart Weir to Gifkloof Weir	Real Flow-Stage	473000	143000	1857	0.038	2	67
Gifkloof Weir to Neusberg Weir	Round Nosed Weir	616000	87000	505	0.052	3	140
Neusberg Weir to Augrabies Falls	Real Flow-Stage	703000	49900	140	0.081	3	93
Augrabies Falls to Stolzenfels Weir	Round Nosed Weir	752900	85300	431	0.045	15	144
Stolzenfels Weir to Orange Falls	Round Nosed Weir	838200	67500	807	0.040	17	200
Orange Falls to Henkries Weir	Round Nosed Weir	905700	140200	1054	0.071	23	233
Henkries Weir to Vioolsdrift Weir	Round Nosed Weir	1045900	50700	1500	0.037	4	67
Vioolsdrift Weir to Fish River Confluence	Real Flow-Stage	1096600	147000	1396	0.038	17	166
Fish River Confluence to Brandkaros	Real Flow Time	1243600	118900	2383	0.037	27	149

It is interesting to note in Figure 9.3 that there are negative minimum values at the upstream end of the model. These are the result of numerical instabilities introduced by the steep hydrographs released from Vanderkloof Dam (computationally the generation of surge waves) which occur in spite of the Courant condition being satisfied by the timestep.

In Figure 9.4 the channel cross sections and weir crests are shown for each of the modelled weirs. Katlani and Doorenkuilen are not weirs and only the cross section is shown. Note the difference in the horizontal scale for Gifkloof and Neusberg. Neusberg Weir has two sections, with two gauging stations. In order to model Neusberg Weir as a real time control an equivalent stage-discharge relationship was determined for the combined weir. The crest level for Neusberg shown in Figure 9.3 is for the combined stage-discharge relationship.

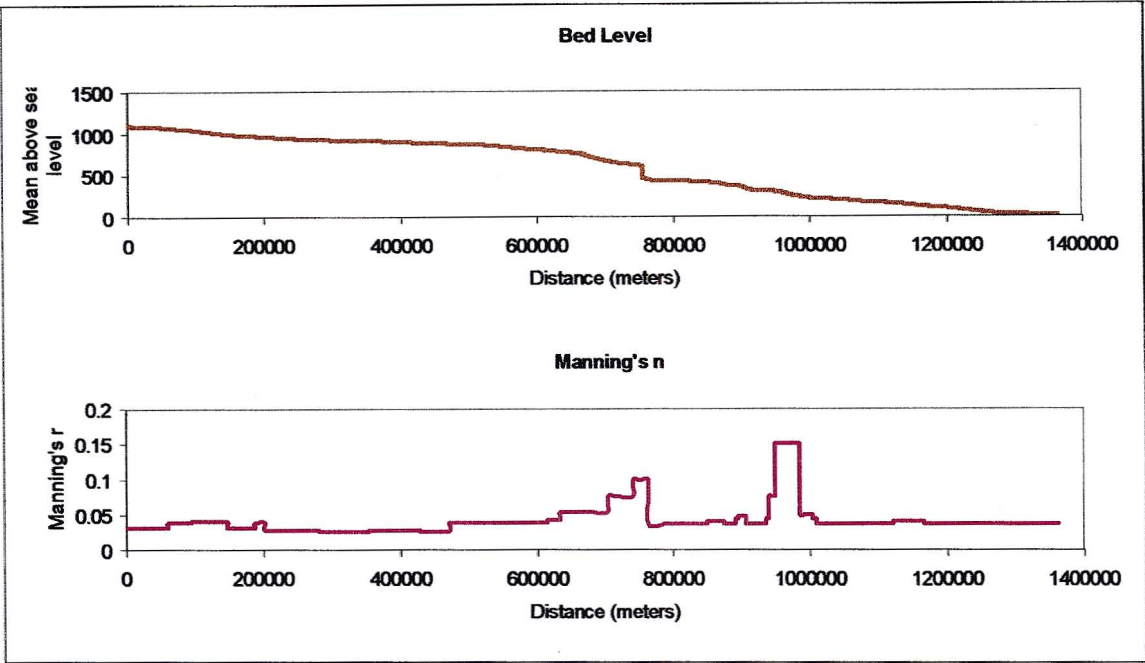


Figure 9.2: Longitudinal section of the river showing bed level and roughness

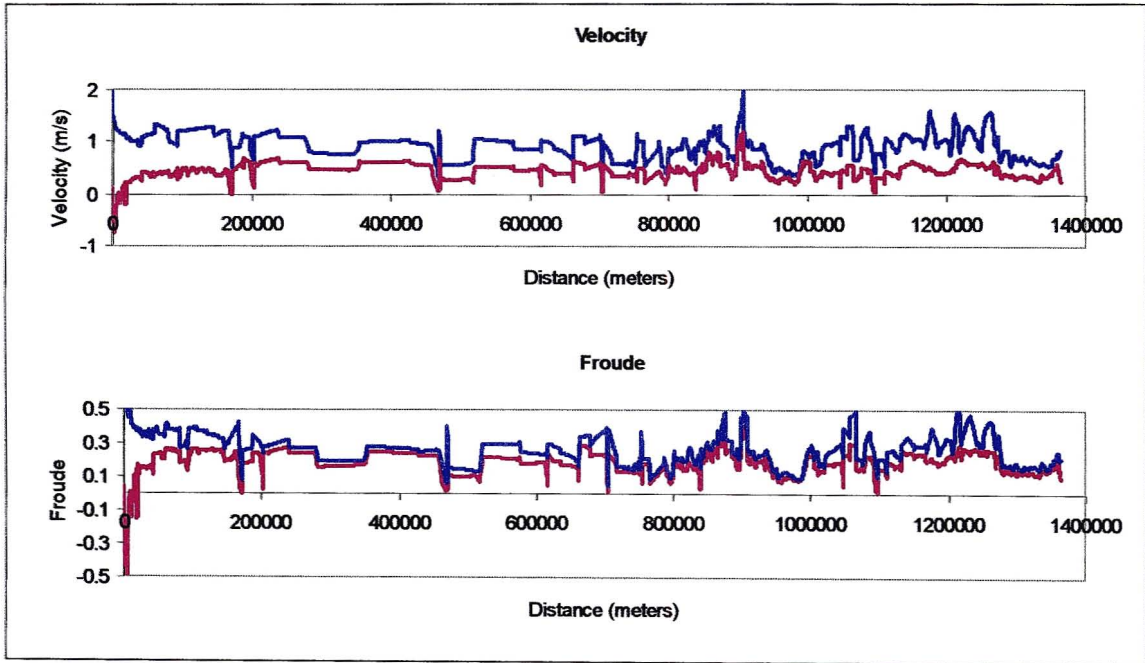


Figure 9.3: Maximum and Minimum Hydraulic properties

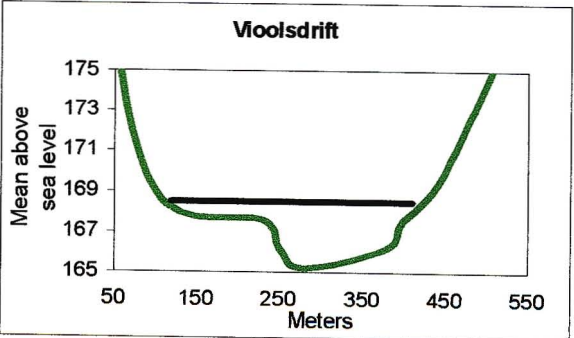
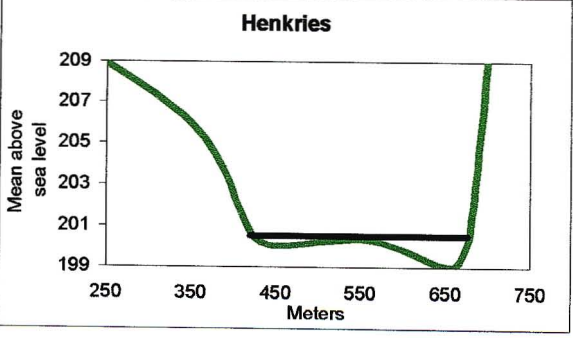
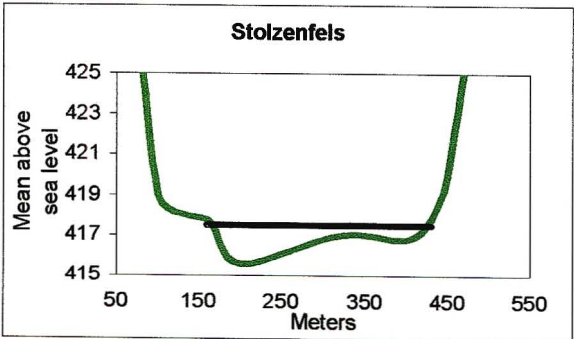
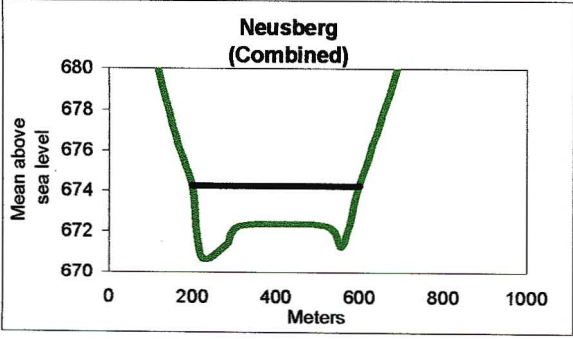
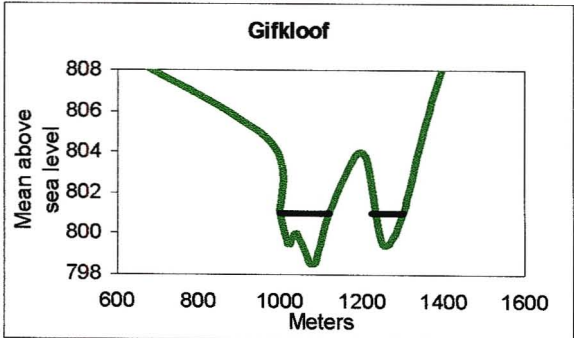
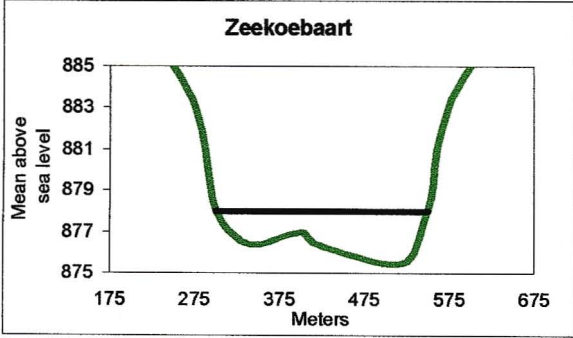
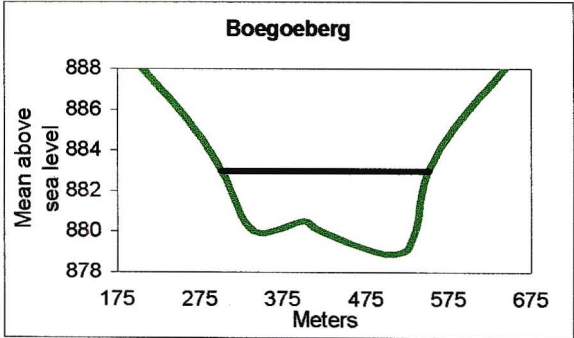
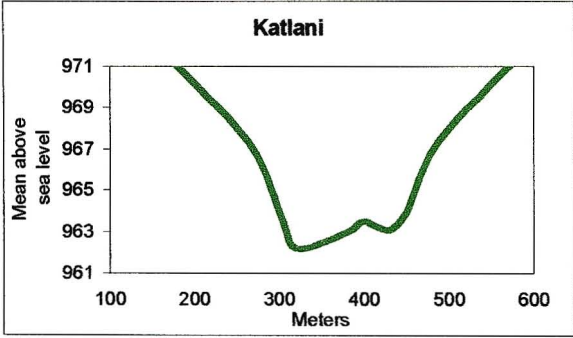
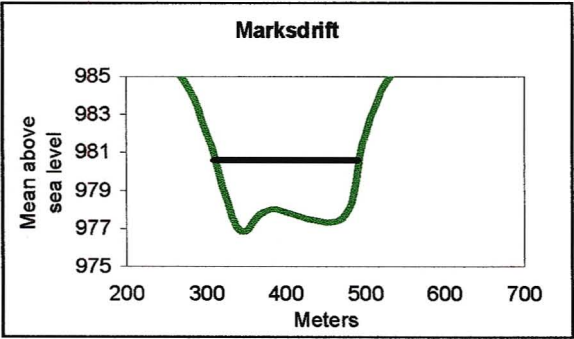
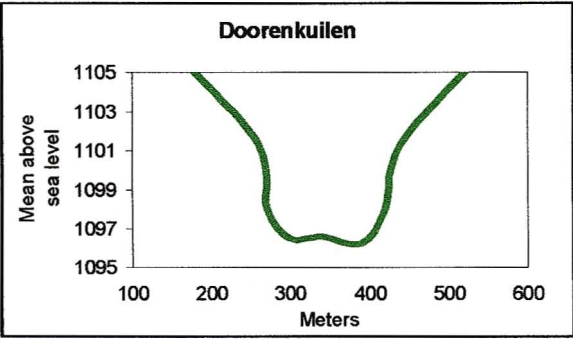


Figure 9.4: Cross Sections of Weirs

The following photographs are reproduced with the kind permission of Dr R McKenzie.

Figure 9.5 shows Vanderkloof Dam, with the turbines on the right hand side. The releases shown in the photograph are scour releases, not turbine releases.

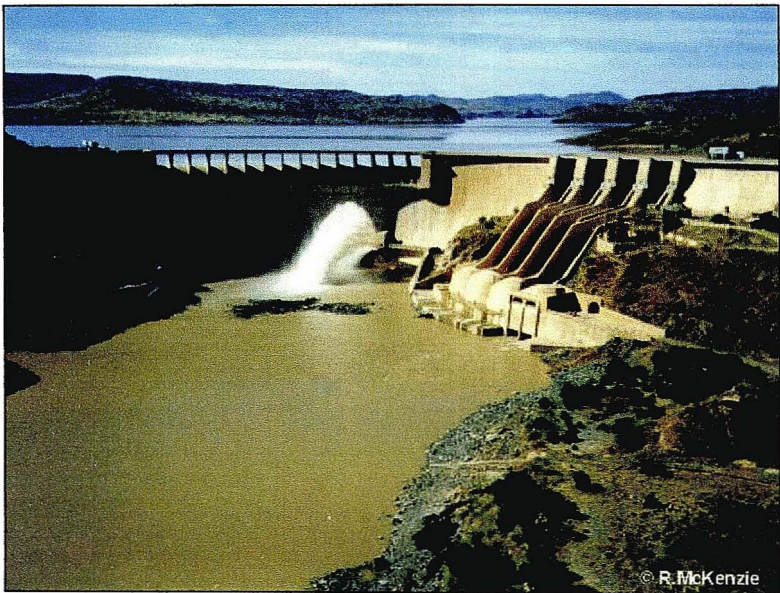


Figure 9.5: Vanderkloof Dam

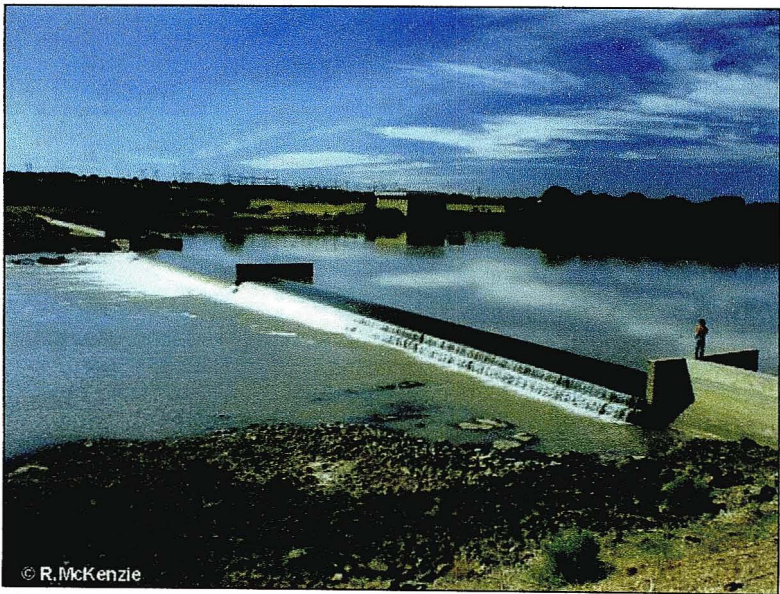


Figure 9.6: Marksdrift Weir

Marksdrift Weir (Figure 9.6) is the last weir on the Orange River before its confluence with the Vaal River. Water is pumped from the weir into a canal and is transferred to Douglas Weir on the Vaal River.

Douglas Weir is on the Vaal River just upstream of the Orange-Vaal confluence. It was originally intended that this weir should be used to record the inflows from the Vaal River. However, as the shape of the weir suggests, it was not designed to measure low flows accurately.



Figure 9.7: Douglas Weir

The Orange-Vaal confluence is shown in Figure 9.8. The Orange River is the larger of the two branches (on the left).

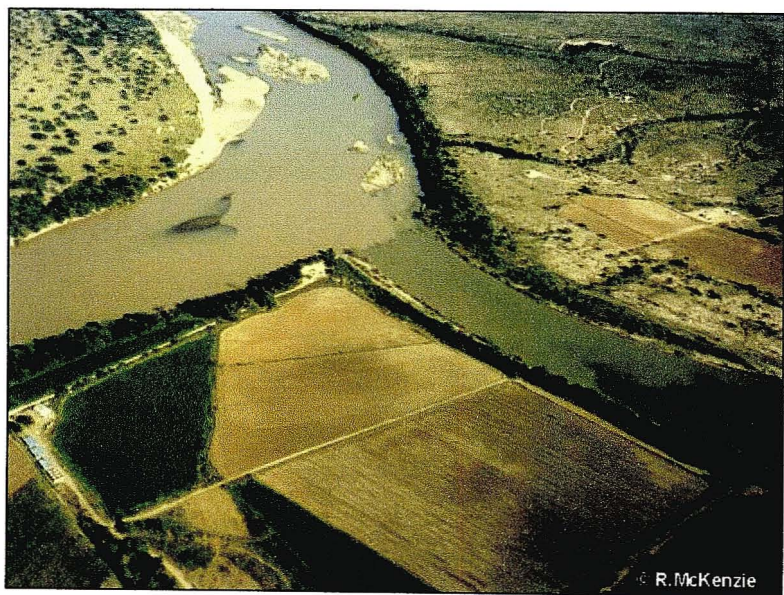


Figure 9.8: Orange-Vaal Confluence

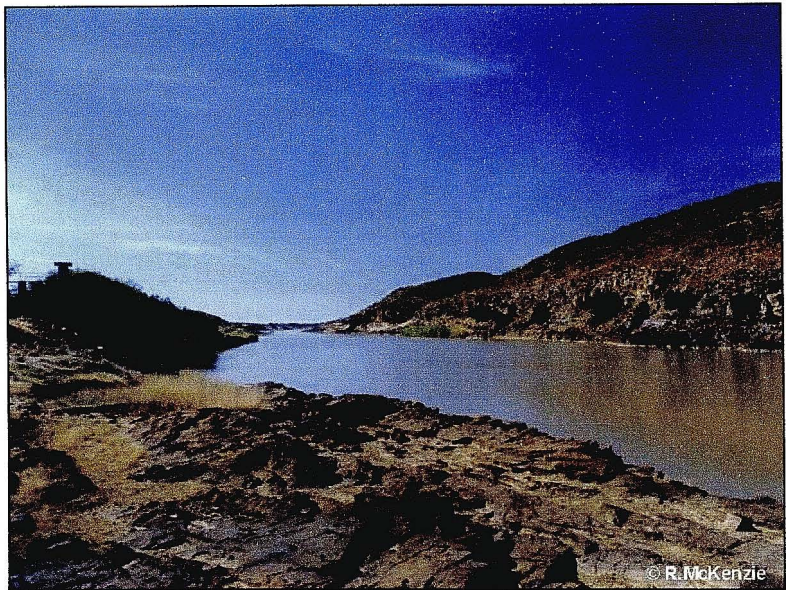


Figure 9.9: Katlani Gauging Station

Katlani is shown in Figure 9.9. It is primarily used as a flood gauging station. Boegoeberg Dam is shown in Figure 9.10. Note that it has a long crest length, making the low flow gaugings inaccurate.

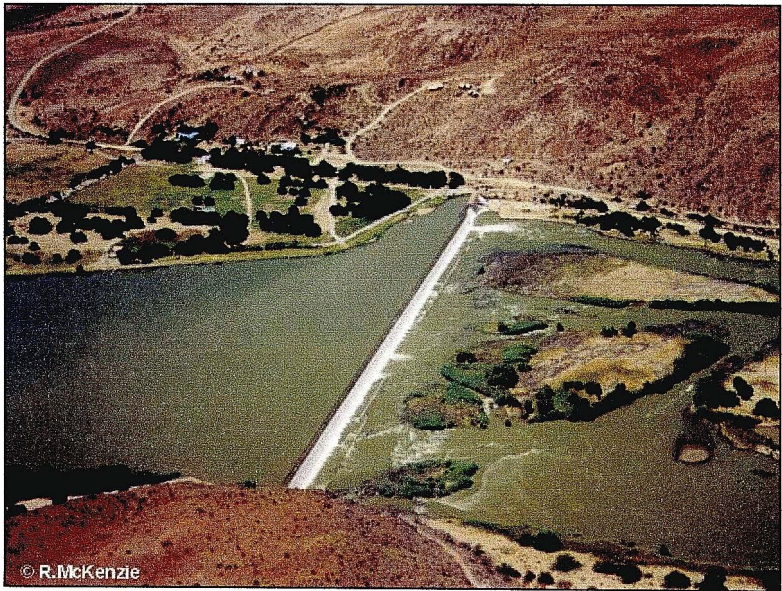


Figure 9.10: Boegoeberg Dam

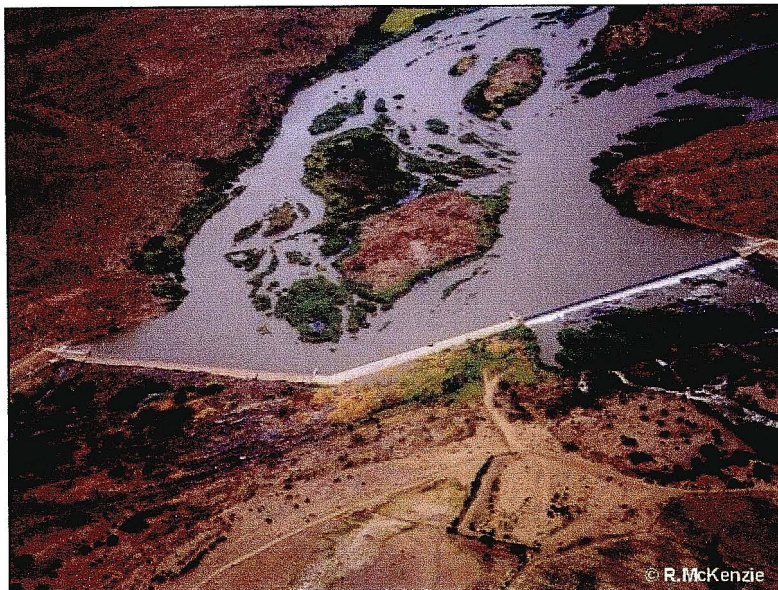


Figure 9.11: Neusberg Weir

Neusberg Weir is shown in Figure 9.11. Once again note that it was not designed as a low flow gauging station and that its long crest length causes low flow gaugings to be inaccurate.

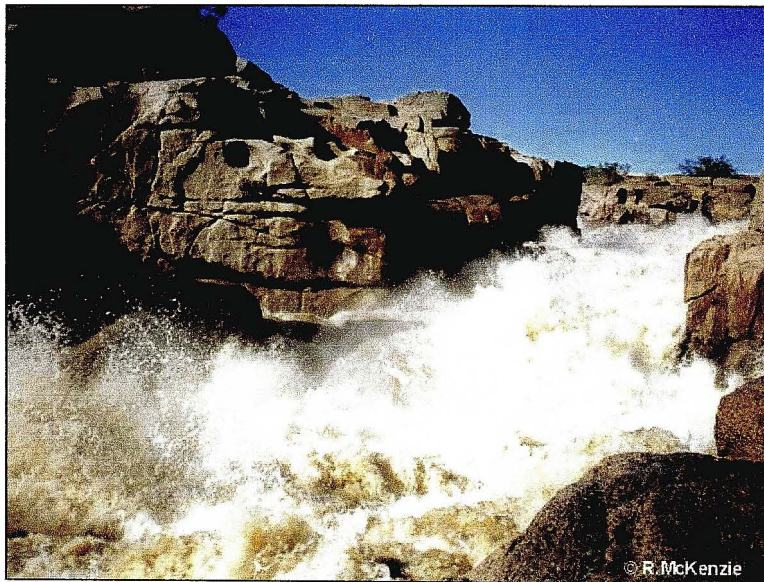


Figure 9.12: The Gorge just upstream of Augrabies Falls

The gorge just upstream of Augrabies Falls is shown in Figure 9.12. The waterfall is modelled as a round nosed weir.

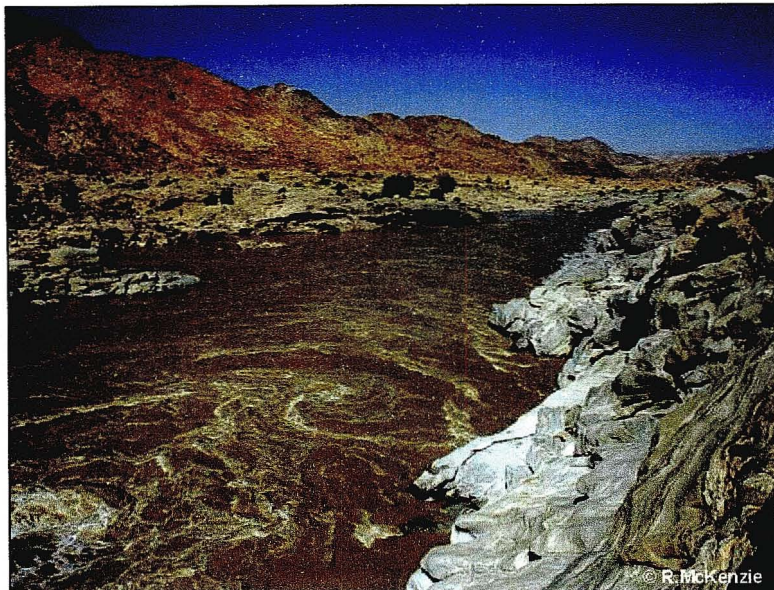


Figure 9.13: The river just downstream of Orange Falls

The Orange Falls, shown in Figure 9.13, are a series of rapids rather than a waterfall as the name might suggest.



Figure 9.14: Pumps at Goodhouse

The abstractions at Goodhouse (Figure 9.14) are used to supply water to the towns of Steinkopf, Okiep, Springbok and Kleinsee.

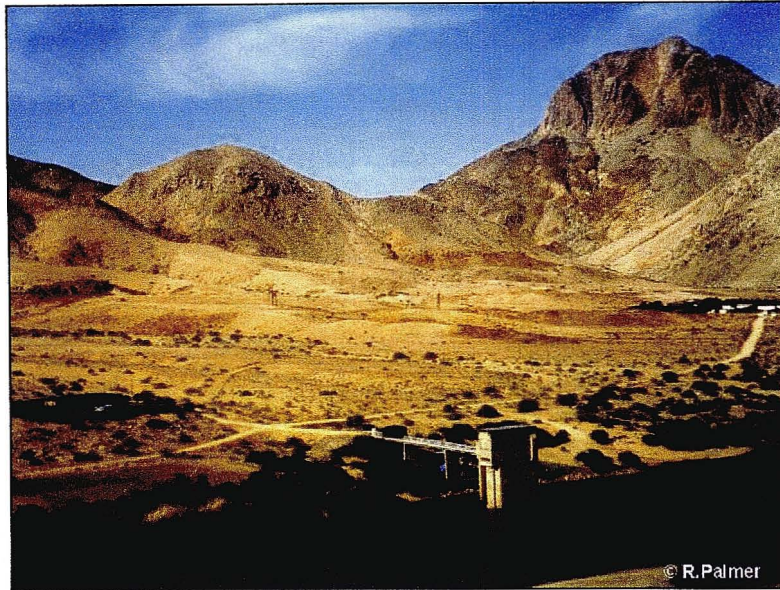


Figure 9.15: Pella Pump Station

Water is pumped from the Orange River at Pella (Figure 9.15) to Pofadder and the large zinc mine at Aggeneys. Almost 5 million m³ per annum are abstracted from the Orange River and treated at a small local purification plant mainly to remove sediment before it is transferred.



Figure 9.16: Vioolsdrift Weir

The Vioolsdrift Weir (Figure 9.16) is effectively the most downstream point on the Orange River where reasonable low flow measurements may be obtained. Approximately 15 million m³ per annum of water is supplied from Vioolsdrift Weir, through a canal to the towns of Vioolsdrift and Noordoewer, where the main development is agricultural.

The very harsh but spectacular terrain of the Richtersveld is shown in Figures 9.18 and 9.19.

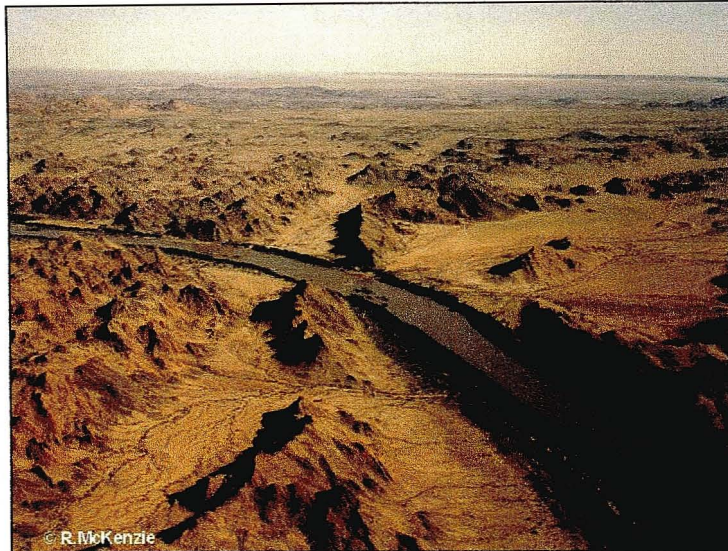


Figure 9.17: The Richtersveld

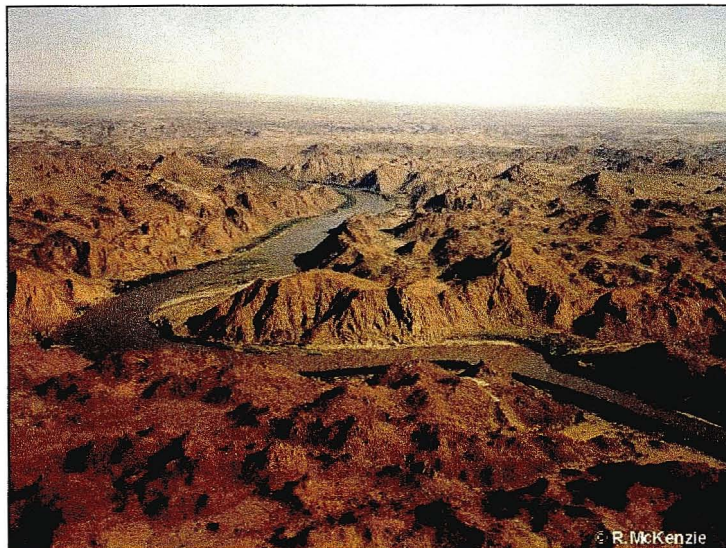


Figure 9.18: The Richtersveld

Water for the diamond mines at the Orange River mouth is abstracted at Brandkaros, shown in Figure 9.19. Irrigation water for the agricultural developments is also abstracted at Brandkaros.

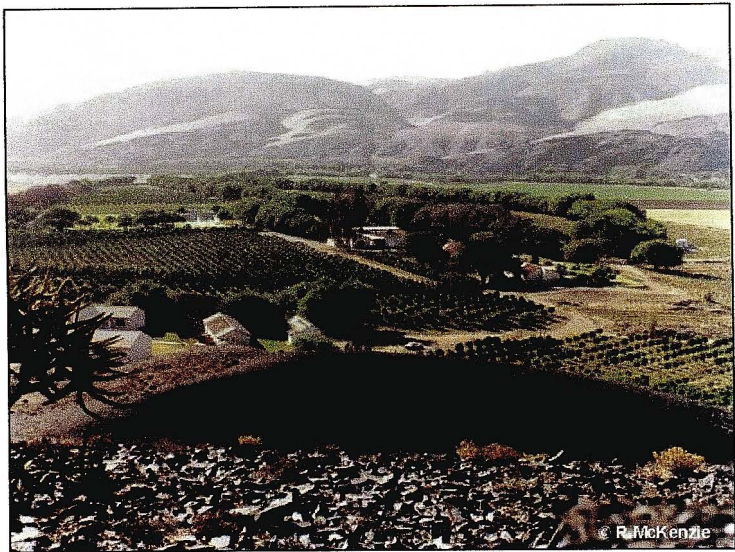


Figure 9.19: Brandkaros

9.1.5 Abstractions from the river

Table 9.2: Abstraction quotas from the Orange River 1994-2000 (not actual abstractions)

Date	Abstraction Quotas from the Orange River (m ³ /s)										
	Vdk – Marks	Marks- Prieska	Prieska -Boeg	Boeg- Uping	Uping- Neus	Neus- Blou	Blou- 20 Deg	20 Deg – Pella	Pella- Viools	Viools- Fish	Fish- BrandK
1 Jan	6.15	3.58	11.55	8.41	11.34	3.58	0.30	1.23	1.49	0.85	0.15
15 Jan	8.32	4.46	11.75	8.43	11.00	3.24	0.28	1.23	1.14	0.20	0.16
1 Feb	7.36	4.14	11.22	7.97	10.16	2.89	0.26	1.20	1.12	0.27	0.16
15 Feb	6.39	3.82	10.69	7.51	9.31	2.54	0.23	1.17	1.09	0.74	0.17
1 Mar	4.97	2.87	8.52	5.77	6.96	1.80	0.18	0.93	0.98	0.65	0.15
15 Mar	3.55	1.92	6.34	4.03	4.61	1.05	0.14	0.70	0.87	0.56	0.13
1 Apr	2.15	1.31	4.40	2.77	3.21	0.82	0.11	0.53	0.78	0.45	0.12
15 Apr	0.74	0.71	2.45	1.51	1.81	0.58	0.09	0.36	0.70	0.34	0.10
1 May	0.36	0.44	1.88	1.38	1.58	0.54	0.07	0.34	0.59	0.20	0.09
15 May	0.02	0.18	1.31	1.25	1.34	0.49	0.06	0.33	0.48	0.06	0.09
1 Jun	0.19	0.19	1.08	1.13	1.23	0.53	0.05	0.32	0.44	0.11	0.09
15 Jun	0.39	0.20	0.85	1.01	1.12	0.57	0.05	0.31	0.40	0.16	0.09
1 Jul	0.96	0.52	1.22	1.35	1.46	0.60	0.05	0.34	0.41	0.19	0.09
15 Jul	1.52	0.84	1.59	1.69	1.79	0.63	0.05	0.37	0.41	0.22	0.10
1 Aug	2.28	1.32	2.31	2.23	2.60	1.02	0.08	0.46	0.47	0.28	0.10
15 Aug	3.04	1.80	3.03	2.76	3.41	1.41	0.10	0.54	0.52	0.33	0.11
1 Sep	5.51	2.73	4.50	3.86	5.11	2.11	0.16	0.71	0.79	1.00	0.12
15 Sep	7.99	3.67	5.97	4.95	6.81	2.81	0.21	0.87	1.07	1.68	0.13
1 Oct	9.04	4.05	7.33	5.87	7.99	3.10	0.24	1.01	1.22	1.69	0.14
15 Oct	10.09	4.42	8.69	6.79	9.16	3.39	0.26	1.14	1.38	1.70	0.14
1 Nov	9.78	4.56	9.90	7.60	10.38	3.80	0.30	1.24	1.59	2.03	0.15
15 Nov	9.48	4.69	11.11	8.41	11.59	4.22	0.33	1.34	1.80	2.35	0.15
1 Dec	6.73	3.70	11.23	8.41	11.63	4.08	0.32	1.29	1.82	2.12	0.14
15 Dec	3.97	2.71	11.35	8.40	11.67	3.93	0.32	1.24	1.84	1.90	0.13
Total (Mm³/a)	143.82	76.26	194.73	147.07	190.88	64.43	5.50	24.88	30.36	26.02	3.9

During the course of various studies considerable work has been done to determine the demands from and the return flows to the Orange River. The information used for

this study was taken from the Orange River Continuous Study. The abstractions quotas are given in Table 9.2.

It should be noted that these are theoretical values which may be abstracted. The amounts which are actually abstracted differ according to the weather and crop requirements. The abstractions quotas from the river have been constant for several years. The modelled abstractions were identical for the 1994 and 2000 simulations.

9.1.6 River Losses

Table 9.3: Losses from the Orange River (averages for the year)

Date	Losses from the Orange River (m ³ /s)									
	Vdk-Marks	Marks-Prieska	Prieska-Boeg	Boeg-Uping	Uping-Neus	Neus-Blou	Blou-20 Deg	20 Deg-Viools	Viools-Fish	Fish-Brand
1 Jan	3.10	4.17	2.57	4.57	2.60	1.92	3.43	7.15	1.89	1.58
15 Jan	3.07	4.11	2.55	4.55	2.60	1.93	3.46	7.25	1.94	1.58
1 Feb	2.63	3.55	2.23	4.04	2.34	1.79	3.19	6.84	1.87	1.52
15 Feb	2.19	2.99	1.91	3.54	2.08	1.64	2.92	6.43	1.80	1.47
1 Mar	1.80	2.47	1.61	3.02	1.77	1.43	2.54	5.68	1.61	1.33
15 Mar	1.40	1.95	1.30	2.50	1.46	1.22	2.17	4.92	1.42	1.18
1 Apr	1.18	1.69	1.09	2.11	1.23	1.03	1.85	4.17	1.21	1.01
15 Apr	0.95	1.44	0.89	1.72	1.01	0.85	1.52	3.41	0.99	0.85
1 May	0.88	1.29	0.79	1.51	0.87	0.74	1.32	2.91	0.84	0.71
15 May	0.81	1.15	0.70	1.30	0.72	0.62	1.11	2.42	0.69	0.57
1 Jun	0.72	1.02	0.61	1.15	0.64	0.54	0.96	2.09	0.59	0.49
15 Jun	0.62	0.89	0.53	0.99	0.55	0.46	0.83	1.75	0.49	0.41
1 Jul	0.68	0.95	0.57	1.06	0.58	0.48	0.86	1.81	0.50	0.42
15 Jul	0.74	1.02	0.61	1.12	0.62	0.50	0.90	1.87	0.51	0.43
1 Aug	0.95	1.30	0.77	1.41	0.78	0.62	1.11	2.30	0.62	0.51
15 Aug	1.17	1.57	0.93	1.71	0.94	0.74	1.32	2.73	0.73	0.60
1 Sep	1.55	2.04	1.21	2.19	1.21	0.93	1.65	3.43	0.92	0.75
15 Sep	1.93	2.51	1.49	2.67	1.49	1.12	1.99	4.13	1.11	0.90
1 Oct	2.17	2.85	1.70	3.06	1.72	1.28	2.29	4.72	1.26	1.02
15 Oct	2.40	3.19	1.91	3.45	1.96	1.44	2.59	5.30	1.41	1.14
1 Nov	2.67	3.58	2.18	3.92	2.21	1.62	2.90	5.99	1.59	1.33
15 Nov	2.93	3.96	2.45	4.38	2.46	1.79	3.21	6.68	1.77	1.53
1 Dec	3.03	4.09	2.52	4.49	2.53	1.85	3.29	6.86	1.80	1.55
15 Dec	3.14	4.23	2.59	4.60	2.60	1.90	3.40	7.04	1.84	1.58
Total	59.33	80.59	49.59	90.22	51.24	39.33	70.25	149.05	40.55	33.72
(Mm³/a)										

An evaporation module was developed for the ISIS software as part of the Orange River Losses Study (McKenzie and Craig, 1999). This model calculated and modelled diffuse evaporation based on the simulated surface area of the river. During the course of that study it was found that the simulations using the evaporation module did not differ significantly from the simulations where evaporation was modelled as discrete abstractions from the river. The evaporation module was therefore not included in the real time model and instead the losses were modelled as discrete

demands abstracted from the lower end of the river reaches. The quantities which were abstracted were obtained from the results of the Orange River Losses Study and are given in Table 9.3.

9.1.7 Calibration of the model

Due to the lack of more recent data the hydraulic model was calibrated against flows recorded in 1994. The first parameter which was calibrated was the Manning's roughness, which affects the attenuation of flow in the river. The average values finally adopted appear in Table 9.1.

The model was also calibrated in terms of numerical diffusion which is introduced during the solution of the St Venant equations. The parameters which can be adjusted are the alpha and theta values. Alpha is the under relaxation factor which is used to accelerate convergence between timesteps (Section 4.1.3). The theta value is the weighting factor used in the Priessman Box method and represents the importance of the previous timestep in the solution of the following timestep. Although Manning's n can be set and therefore calibrated for every cross section in the river the alpha and theta values are applied to the whole river.

In order to calibrate the model in terms of Manning's n , it was necessary to compare the simulated flows to the recorded flows for each of the gauging stations. By introducing the real time recorded data at each of the gauging stations errors in the calibration of the reach upstream of the gauge are not propagated further downstream. In other words the use of the real time unit allows each section of river (between gauging stations) to be calibrated independently of the other sections, while the river is simulated as a whole. If the real time unit were not used it would have been necessary to simulate each section of the river between gauging stations separately.

Each section of the river between gauging stations is made up of a number of RIVER units (Table 9.1 and Figure 9.2) each with a corresponding Mannings n value. It was therefore necessary to adjust all of the Manning's n values between the gauging stations by an appropriate factor, estimated by comparing the simulated and recorded data at the downstream gauge. In this way the relative relationship of Mannings n values for the various reaches is maintained.

Once the basic calibration values were established for the model a closer look at the simulated results was necessary.

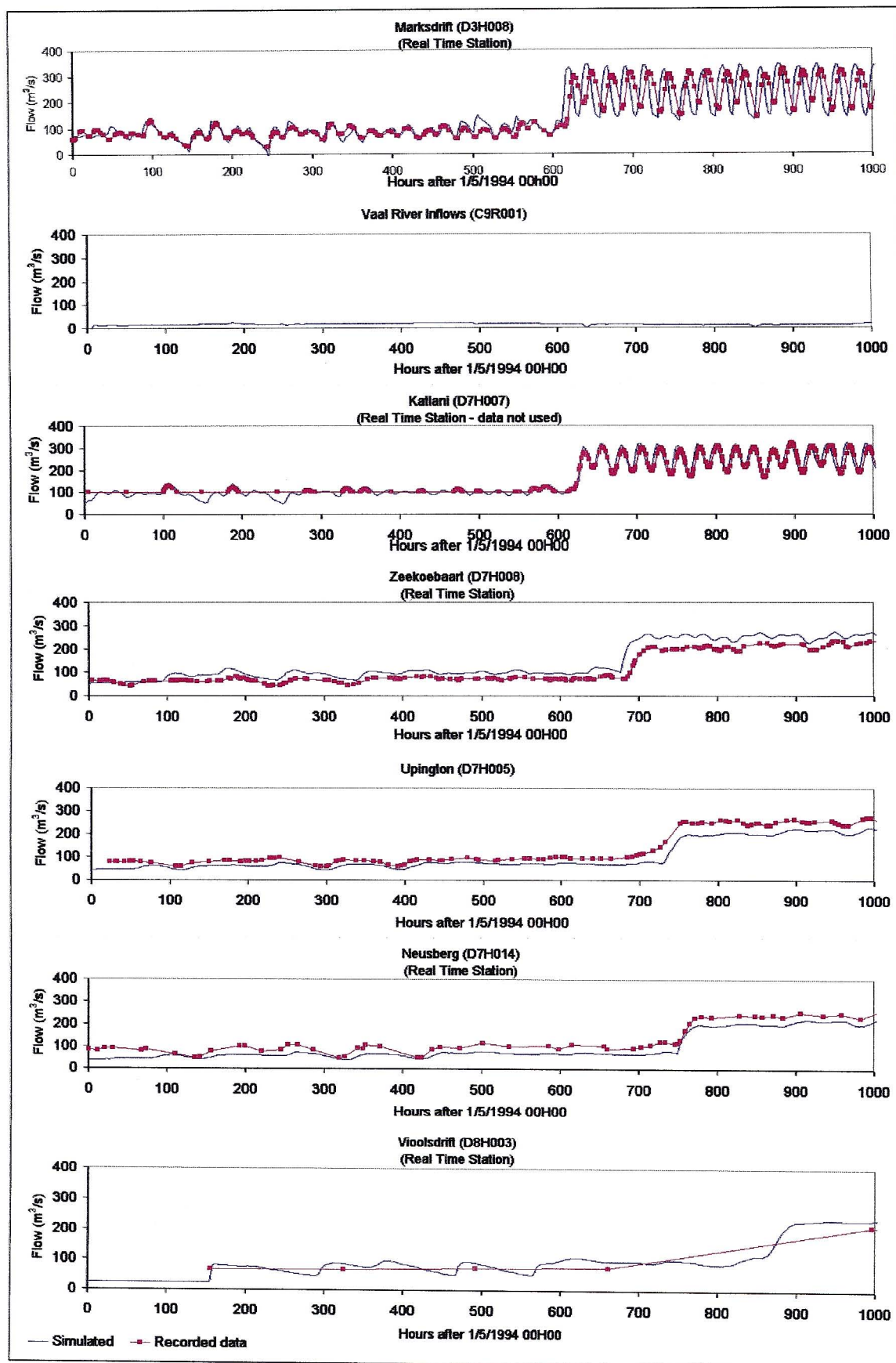


Figure 9.20: Calibration results using all real time data

A statistical breakdown of the results is given in Table 9.4. The statistics were calculated for the periods from 0 to 600 hours and from 600-1000 hours as defined by the different flow regimes which were present in the river.

The results of the first of these simulations are given in Figure 9.20. In this simulation the recorded data were used at all of the real time stations (Marksdrift, Zeekoebaart, Neusberg and Vioolsdrift). As Katlani is not a weir, it could not be used as a real time control in the simulation. The Vaal River inflows which were introduced were those recorded at Douglas Weir. The recorded flows at Upington were not propagated downstream using the real time module because Upington is not included in the real time network of gauging stations. The Upington flows were only used for comparative purposes to assess the accuracy of the model. The simulation started on 1 May 1994 and continued for 1000 hours. The simulated results were assessed for each reach of the river.

In Figure 9.20 the fluctuations in the flow at Marksdrift Weir starting at 600 hours are as a result of the operation of Vanderkloof Dam to meet the electricity demands. It is interesting to note the downstream dissipation of the fluctuations.

The attenuation of the simulated and recorded results differs at Marksdrift Weir. This indicated that Mannings roughness should be increased. There was obviously an error in the recorded flows at Katlani below $100 \text{ m}^3/\text{s}$. The simulated flows at Zeekoebaart exceeded the recorded flows by approximately $50 \text{ m}^3/\text{s}$ on average. This is also indicated by the large average errors at Zeekoebaart given in Table 9.4.

At Upington and Neusberg Weir the recorded flows exceeded the simulated flows by approximately $50 \text{ m}^3/\text{s}$. The data at Vioolsdrift Weir were considered to be too sparse to be used and no conclusion regarding the calibration of the river below Neusberg Weir could be made.

The errors in the simulation at Zeekoebaart, Upington and Neusberg were of concern. The magnitude of the error far exceeds the combined effect of abstractions and losses from the river in the month of May (See Tables 9.2 and 9.3). From Figure 9.1 it can be seen that a change in stage (which is what is actually recorded and input to the model) of less than 2 cm results in a difference in flow between $200 \text{ m}^3/\text{s}$ to $250 \text{ m}^3/\text{s}$ at Zeekoebaart. It was therefore concluded that the data recorded at Zeekoebaart Weir were inaccurate, being too low. Propagating these incorrect low flows down the river resulted in the simulated flows at Upington being too low. As

Upington is not a real time station no recorded flows at Upington were propagated to Neusberg and incorrect data at Zeekoebaart also caused the simulated flows at Neusberg to be less than the recorded flows.

In order to test this conclusion a second simulation was conducted without any recorded data being included at Zeekoebaart Weir. Instead the simulated flows at Zeekoebaart were propagated downstream as far as Neusberg Weir. The recorded data at Zeekoebaart were only used in Figures 9.21 for comparative purposes. Note the increased simulation time to 2000 hours. From the results given in Figure 9.21 it can be seen that the magnitude of the flows at Upington and Neusberg Weir are acceptable, if the real time data at Zeekoebaart Weir are not used. The increase in Manning's roughness between Marksdrift and Neusberg Weirs for this simulation improved the flow attenuation upstream of Neusberg Weir. These improvements are seen in the decrease in the average errors in Table 9.4.

Once the real time model had been calibrated it was necessary to check that the calibration was still acceptable if no real time data were included. This was performed to ensure that the forecast simulations would produce accurate results. The results of this simulation are given in Figure 9.22. From these results it was concluded that the forecast simulations would be acceptably accurate.

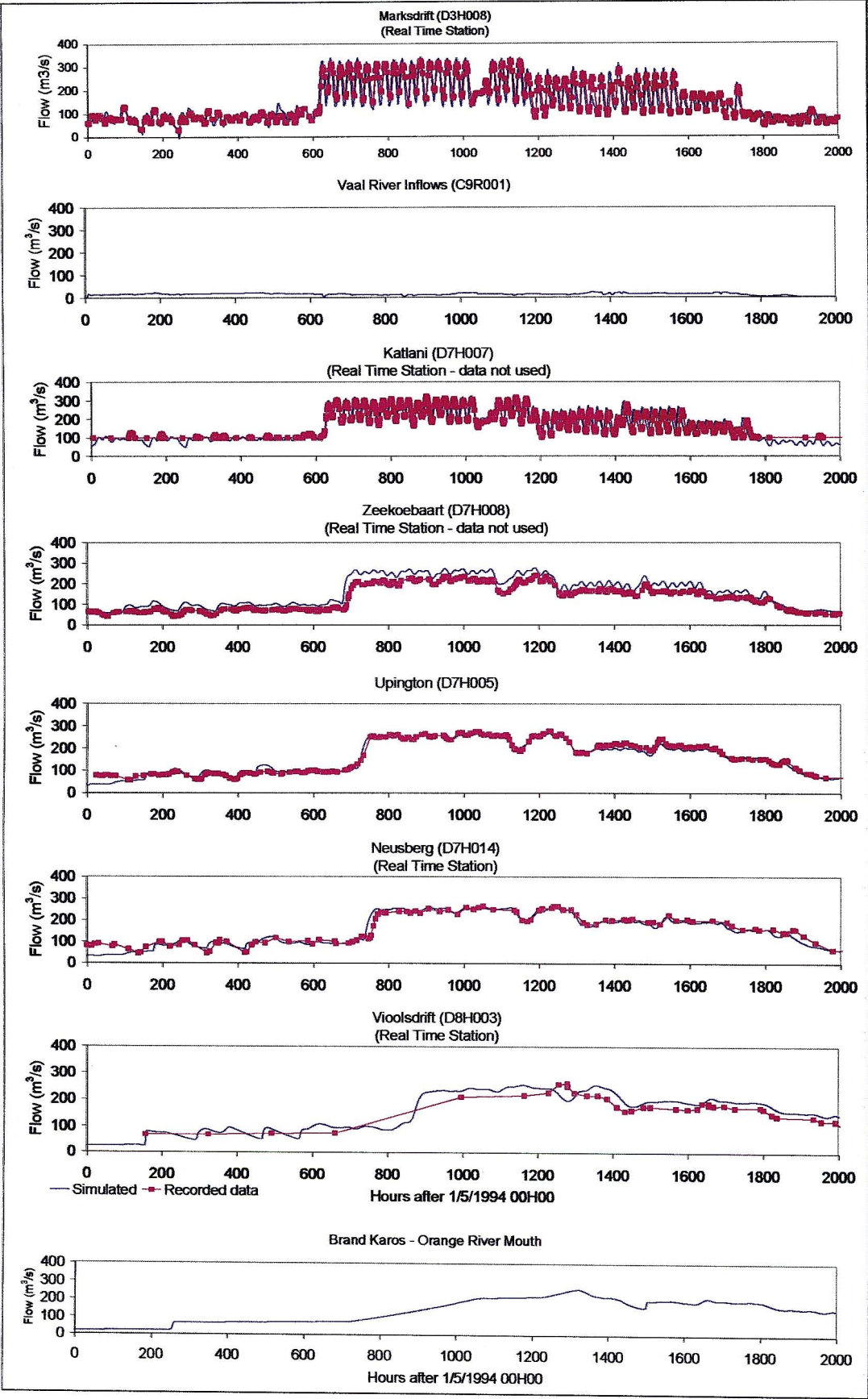


Figure 9.21: Calibration results: No real time data at Zeekoebaart, n increased

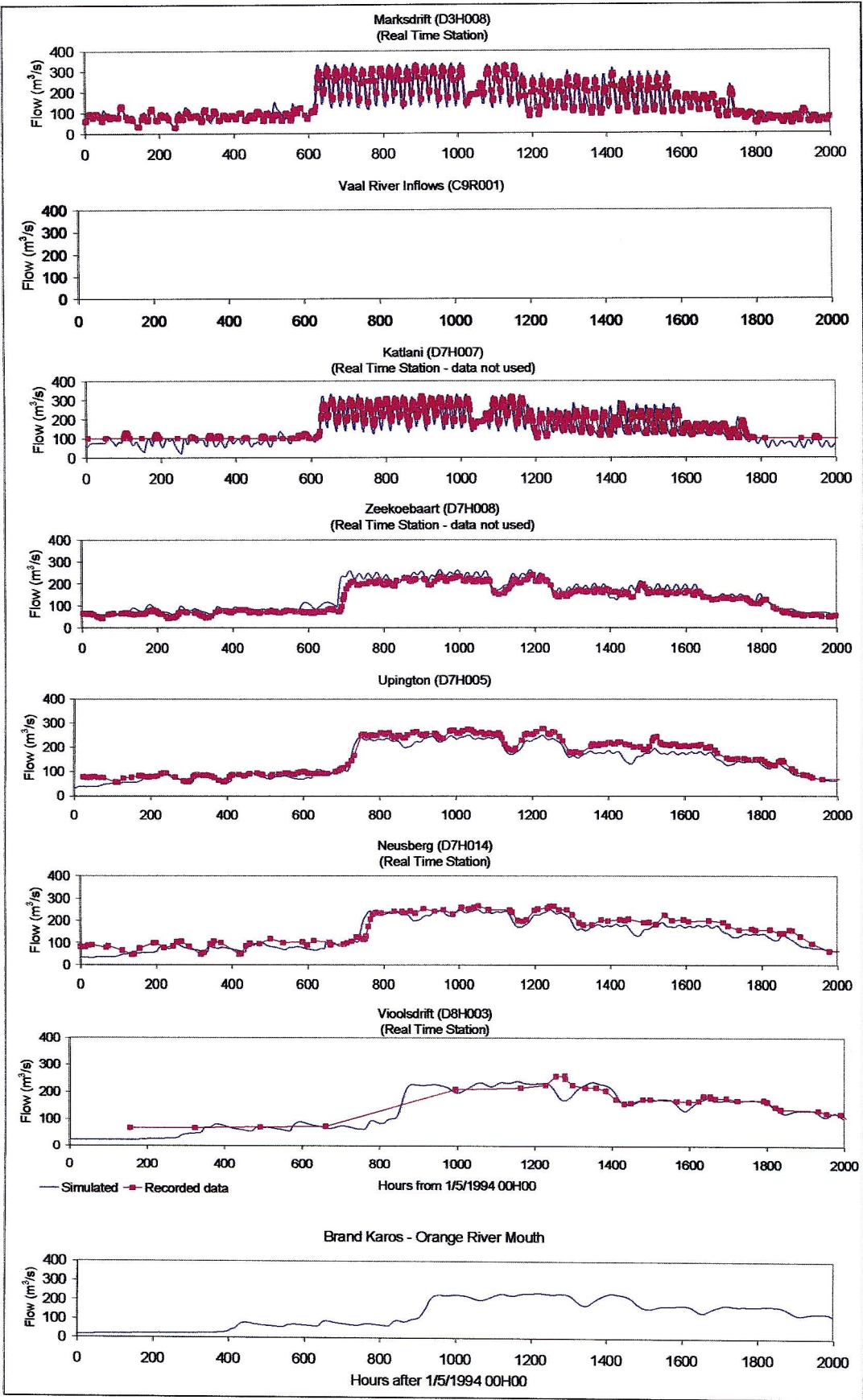


Figure 9.22: Calibration of forecast simulation: No real time data

Table 9.4: Statistical analysis

	0-600 Hours				600-1000 hours			
	Recorded	Fig 9.20	Fig 9.21	Fig 9.22	Recorded	Fig 9.20	Fig 9.21	Fig 9.22
Marksdrift (D3H005)								
Mean	80.8	85.6	85.6	85.7	143.7	144.1	144.1	144.1
stdev	17.7	25.2	25.2	24.4	86.2	88.7	88.7	89.4
Correl		0.7214	0.7224	0.7319		0.9282	0.9281	0.9331
R ²		0.5219	0.5219	0.5219		0.8614	0.8614	0.8614
ave err		4.7	4.7	4.9		0.4	0.4	0.4
Katlani (D7H007)								
Mean	102.2	98.5	98.3	85.3	157.4	158.4	158.2	143.4
stdev	5.4	17.2	17.1	22.1	75.4	83.0	82.8	86.4
Correl		0.6623	0.6629	0.4821		0.9634	0.9744	0.8956
R ²		0.4395	0.4395	0.4395		0.9495	0.9495	0.9495
ave err		-3.6	-3.9	-16.8		1.0	0.8	-14.0
Zeekoebaart (D7H008)								
Mean	66.0	88.2	88.1	75.5	111.0	142.1	141.9	128.2
StDev	10.0	17.0	16.9	14.1	66.5	77.3	77.3	74.0
Correl		0.7644	0.7635	0.5597		0.9784	0.9796	0.9624
R ²		0.5843	0.5830	0.3133		0.9573	0.9596	0.9263
Ave Err		22.2	22.1	9.4		31.1	30.9	17.2
Upington (D7H005)								
Mean	80.9	82.4	82.5	70.0	130.4	131.4	131.3	118.0
StDev	10.6	19.1	22.3	15.3	74.9	75.1	75.6	71.0
Correl		0.6769	0.7052	0.6135		0.9858	0.9824	0.9796
R ²		0.4582	0.4974	0.3763		0.9717	0.9651	0.9596
Ave Err		1.5	1.6	-10.9		1.0	1.0	-12.4
Neusberg (D7H014)								
Mean	84.7	76.2	76.1	65.3	124.8	123.5	123.4	110.6
StDev	17.2	21.3	25.6	18.4	65.7	75.1	75.9	70.7
Correl		0.4610	0.5387	0.4920		0.9590	0.9579	0.9479
R ²		0.2125	0.2902	0.2421		0.9196	0.9176	0.8986
Ave Err		-8.6	-8.6	-19.5		-1.4	-1.5	-14.3
Vioolsdrift (D8H003)								
Mean	69.6	73.3	73.3	54.1	99.3	104.9	104.9	90.4
StDev	2.1	14.3	14.2	20.1	43.7	54.2	54.2	61.9
Correl		0.0935	0.0920	0.7296		0.8944	0.8945	0.9137
R ²		0.0087	0.0085	0.5323		0.7999	0.8001	0.8348
Ave Err		3.8	3.8	-15.4		5.6	5.6	-8.9

9.2 SUMMARY

The physical characteristics of the river downstream of Vanderkloof Dam are described in terms of the ISIS hydraulic model. The procedure used to prepare the model is described and the abstractions and losses from the river are also discussed.

The process used to calibrate the model using the real time recorded data is discussed. During the calibration process it became evident that the accuracy of the

real time recorded data at Zeekoebaart Weir was questionable. The detrimental effect of including inaccurate data in the real time model is clearly indicated in this chapter.

Once the real time model was calibrated against recorded data (excluding the inaccurate data at Zeekoebaart Weir) the calibration was checked for the forecast simulations, when no real time data were included in the simulations.

The various aspects of the decision support model have now been described. The process used to assesses the effectiveness of the decision support model is described in the following chapter.

10 ASSESSMENT OF THE DECISION SUPPORT TOOL

The decision support tool was assessed in regard to two aspects. Firstly the use of the real time model to update the simulated flow conditions on the river (thereby increasing the accuracy of the forecast simulations) was investigated. Secondly, a rough estimate was made of the savings in water that could be gained by using the model to determine flow releases.

10.1 THE EFFECT OF THE REAL TIME MODEL ON THE FORECAST SIMULATIONS

The aim of this exercise was to determine whether the real time modelling made a meaningful improvement to the accuracy of the forecast simulations. The real time model was run over a period of time and at several intervals the conditions on the river were saved and used as the initial conditions for forecast simulations. The results of these forecast simulations were then compared to each other and to the actual flows which were recorded on the river.

In Figure 10.1 the results of several of these simulations are compared. Once again the simulations were conducted using historical data due to the lack of up-to-date recorded information. The period of the simulation was 20 June 1994 to 5 July 1994. The period was chosen in winter which is when the low flows occur in the river and when the model would be of most use.

Due to the inaccuracies of the flows recorded at Zeekoebaart Weir this station was not used as a real time control in the real time simulations (See Chapter 9). The recorded data for Zeekoebaart Weir were not propagated downstream in the real time simulation.

In Figure 10.1 the solid red line represents the results of the real time simulation for 1 May to 5 July. The real time data for the full period were included in this simulation. (Only the results from 20 June are shown). Initial conditions for forecasting runs were produced from the real time simulation on 20 June 00h00, 24 June 00h00 and the 28 June 00h00. These initial conditions were used to start 3 forecast simulations on each of these dates. The forecast simulations continued up until 5 July 1994.

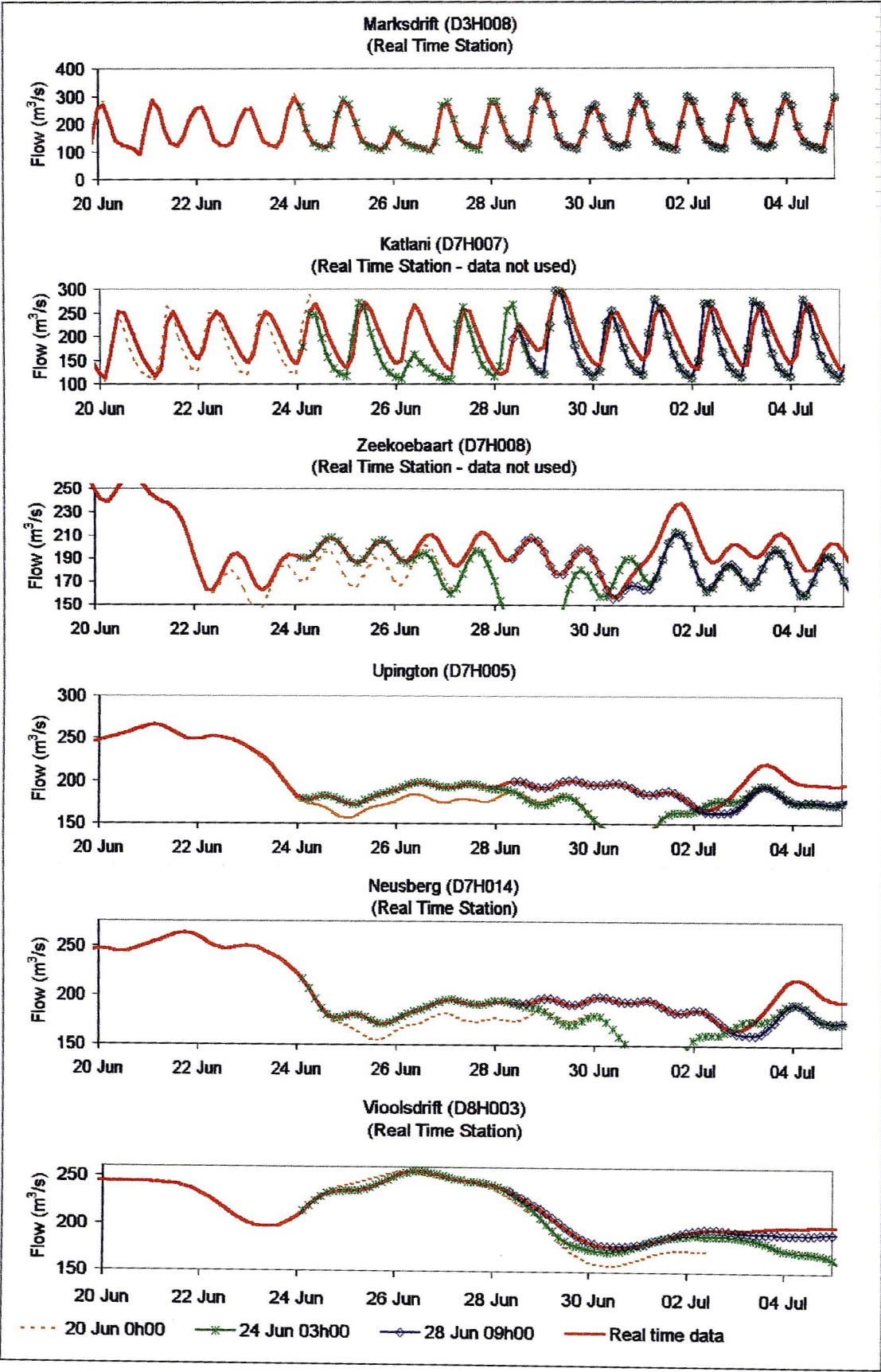


Figure 10.1: The effect of the real time data on the accuracy of the forecast results

The results of the forecast and real time simulations are compared in Figure 10.1. (Note the differences in scale for each set of results.) It can be seen that it takes some time for the flows of the forecast simulation to differ from those of the real time simulation. This is due to the fact that the real time results were used as the initial conditions for every computational node of the model for the forecast simulations. The time taken for the forecast simulation to differ from the real time results is equal to the time it takes for the flow to propagate from just downstream of one real time station to the next real time station. For example consider the flows at Neusberg Weir. The forecast simulation started on the 20 June (dashed red), but the simulated flows only differ from those of the real time simulation after 24 June. Likewise, the simulation which started on the 24 June only differs from the real time simulation after 28 June.

Due to the fact that it takes some time for these flows to differ from the actual flows it can be concluded that the forecast simulations do produce more accurate results if the results of the real time simulation are used for the initial conditions of the forecast simulations.

10.2 ESTIMATE OF SAVINGS IN WATER DUE TO THE DECISION SUPPORT TOOL

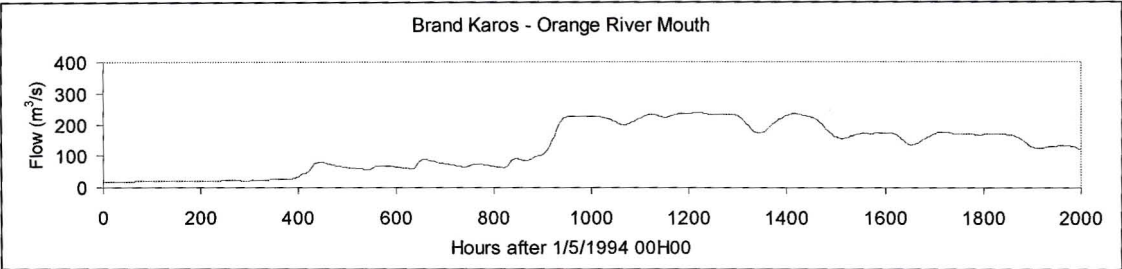


Figure 10.2: Simulated flow at the Orange River mouth

The simulated flow at the Orange River mouth is given in Figure 10.2. The average flow for the full period shown is 125 m³/s. The average flow for the second half of the period is 186 m³/s. The actual environmental demand at the river mouth varies between 6.25 m³/s and 9.33 m³/s depending on the season and drought conditions. Comparing these flows to the required flows it can be seen that the savings in water could be substantial.

It should however be noted that the releases were made to generate hydro-power at Vanderkloof Dam, not to ensure that the minimum required flow was met at the mouth. The comparison is therefore only a very rough estimate of savings that could be made.

10.3 SUMMARY

The decision support tool was assessed in regard to two aspects. Firstly the use of the real time model to update the simulated flow conditions on the river (thereby increasing the accuracy of the forecast simulations) was investigated. Secondly, a rough estimate was made of the savings in water that could be gained by using the model to determine flow releases.

The real time model was run over a period of time and at several intervals the conditions on the river were saved and used as the initial conditions for forecast simulations. The results of these forecast simulations were then compared to each other and to the actual flows which were recorded on the river. When using the results of the real time simulation as the initial conditions for the forecast simulation, it was found that there was an improvement in the level of accuracy of the forecast flows.

The savings in water for the period of May to July 1994, which could have been made had the decision support system been used, was estimated as being in the order of $100\text{m}^3/\text{s}$. It should however be noted that the releases were made to generate hydro-power at Vanderkloof Dam, not to ensure that the minimum required flow was met at the mouth. The comparison is therefore only a very rough estimate of savings that could be made.

11 CONCLUSIONS AND RECOMMENDATIONS

In this dissertation a decision support tool for the operation of the Orange River downstream of Vanderkloof Dam was developed by the candidate. This tool is based on a hydraulic model of the Orange River and enables the decision makers to evaluate the effects of proposed release scenarios from Vanderkloof Dam before they are made.

Several potential challenges in accurately modelling the Orange River downstream of Vanderkloof Dam were identified:

- The length of the river is 1400km.
- Due to the remote terrain through which the river flows the abstractions from the river are not recorded in real time and it is not possible to include accurate abstraction data in the river model
- The return flows from irrigation along the river are also uncertain
- Losses from the river are also significant. Evaporation losses depend on the surface area of the water and therefore the local flow conditions.
- Inflows from tributaries are not recorded in real time.

In order to overcome these difficulties and improve the modelled results, a real time hydraulic model of the river was developed as part of the decision support system. Real time stage and discharge data are recorded at several sites on the Orange River. These recorded data are incorporated into the real time hydraulic model thereby correcting the simulated flows to the actual flow conditions recorded on the river. The initial conditions for the forecast simulations, when the proposed releases are to be assessed, are obtained from the results of the real time model and are therefore a best estimate of the actual flow conditions on the river.

In order to develop the decision support tool, the candidate completed several tasks:

- The theory of open channel flow was studied. This included understanding the derivation of the St Venant equations of flow from Reynolds Transport Theorem and understanding the differences between the various lumped flow routing and distributed flow routing models, so that the correct choice could be made for the model of Orange River. In order to understand the design the real time unit and the adaptive timestepping routine the solution of the St Venant equations were also studied.
- Next the required developments were designed and the changes were made to the hydraulic modelling package (by Dr C Whitlow). The real time hydraulic

unit is a novelty in river modelling and was developed and tested as part of this project. This unit forces the simulated values in the model to match the actual stage and flow conditions recorded at several points on the river, thereby improving the accuracy of the model. The adaptive timestepping routine allows a relatively long timestep to be used for the simulation. If instabilities occur the timestep is reduced until a solution is found, or the minimum timestep is reached. The longer timestep results in shorter computational times. This unit is now available in the commercial releases of ISIS.

- The adaptive timestepping routine and real time unit were tested by the candidate.
- The decision support system as described in Chapter 6 was conceptualised and designed by the candidate. The system is made up of three components: A real time model of the Orange River, which allows the simulations of the river to be continuously updated to the real time data recorded conditions on the river, so that a best estimate of actual flow and stage will be available at every computational node of the model. Once the real time model has been updated with the most recent data, the second component, the forecast model simulates the scheduled releases from Vanderkloof Dam, giving a best estimate of future conditions on the river. The third component can be used to simulate any proposed release scenarios, so that it can be determined whether they meet the required downstream flow requirements.
- The graphical user interface (GUI) as described in Chapter 7 was designed coded and tested by the candidate. The graphical user interface controls the flow of data and results between the various components of the decision support system and between the decision support system and the user.
- The hydraulic model downstream of Vanderkloof Dam was populated by the candidate and a team of support staff.
- The hydraulic model was successfully calibrated against recorded flow by the candidate. During this process the integrity of the real time recorded data was also assessed. From the results of the simulation it was concluded that the data recorded at Zeekoebaart Weir were inaccurate.
- Finally the functionality of the decision support system was tested by the candidate and the affect of the real time model on the forecast simulations was assessed.

The strategy determined in this study will provide a rational basis for the operators of Vanderkloof Dam to determine a discharge release pattern to ensure that the various

demands downstream of Vanderkloof Dam are satisfied. As the model is based on sound hydraulic principles rather than simplified routing methods the users can be confident in the simulated results, provided that the real time data are accurate and available. The use of real time data further improves the simulation as unmodelled events such as localised inflows and abstractions can be catered for and their influence incorporated, even though they are not measured.

One shortcoming of the project is that although the methodology has been developed and tested on historical data it has not been possible, to date, to test the process sufficiently on real time data. This is due to the lack of available accurate real time data.

Secondly, although the system was developed to run in real time the recorded data is not available in real time. The "real time" simulation is therefore updated only when this data becomes available.

Having completed the study the following recommendations can be made:

- The acquisition of real time data from the Orange River be improved. The real time telemetry network must be well maintained so that accurate and up to date information is available.
- It is recommended that the real time data are made available on an FTP site, so that the data do not have to be e-mailed to interested parties. Alternatively the full set of information should be made available on the website. At present only the average readings for every hour are included.
- It is also strongly recommended that the releases from Vanderkloof Dam are made available in real time. This will require co-operation between Eskom and the Department of Water Affairs and Forestry.

The inaccuracy in the data recorded at Zeekoebaart be investigated further and corrected.

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