# Real-Time Model Development for the Full River System

by

James Alexander Melvill

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## Declaration

I, the undersigned, hereby declare that the work contained in this thesis is my own original work and that I have not previously in its entirety or in part submitted it at any university for a degree.

Signed: …………………….

Date: …………………….

# Summary

Increased water demand coupled with limited resources has increased the need for greater control over our water resources. The Orange-Fish-Sundays Transfer Scheme, which transfers 700 million $m^3$/yr (as measured from 2002 to 2005) from the Orange River to the water scarce Fish and Sundays River valleys in the Eastern Cape province of South Africa, is losing an estimated 200 million $m^3$/yr to the sea. These estimated losses are equal to half the current water supply to the Western Cape water supply system for 2005 (E. Van den Berg). For this reason the Department of Water Affairs and Forestry requested a new hydrodynamic model to simulate the requested irrigation demands and forecast the required flow release from the control structures to assure supply as well as limit losses.

This thesis outlines the development of the new hydrodynamic model along with the data control systems setup to manage the real-time data, the inflow forecasts as well as the requested demands for the river system. These systems were developed to assist with the operation of the model by non-technical operators, thus reducing the training and operation costs of the river system. The completed model will be tested during 2007 and further improvements are to be made during this period.

# Opsomming

Die verhoogde aanvraag na die alreeds beperkte plaaslike waterbronne het die behoefte vir beter beheer oor ons waterbronne laat ontstaan. Die Oranje-Vis-Sondagsrivier Oordrag Skema verplaas sowat 700 miljoen $m^3$/jr (soos bereken tussen 2002 tot 2005) vanaf die Oranjerivier na die waterskaars Vis en Sondagsrivier valleie in die Oos-Kaap provinsie. Suid-Afrika verloor omtrent 'n geskatte 200 miljoen $m^3$/jr aan die see. Hierdie geskatte verliese is gelykstaande aan die helfte van die huidige watertoevoer na die Wes-Kaap watertoevoerstelsel vir 2005 (E. Van den Berg). Om hierdie rede het die Departement van Waterwese en Bosbou 'n nuwe hidrodinamiese model versoek wat aan die bestaande bespoeings behoeftes kan voldoen. Die verlangde watervloeistroomvrystelling vanaf gekontroleerde strukture moet ook voorspel kan word ten einde te verseker dat genoegsame water voorsien word, maar waterverliese terselfdetyd beperk word.

Hierdie tesis beskryf die ont wikkeling van die nuwe hidrodinamiese model asook die datakontrole sisreem wat opgestel is om die reele tyddata, die instroom voorspellings en die behoefte vereistes vir die rivier sisteem te bestuur. Hierdie sisteme is ontwikkel om die hantering van die model deur nie-tegniese werkers te vergemaklik en dus sodoende opleidings koste van werkers en die operasiekoste van die rivier sisteem te verminder.

## Acknowledgement

I would like to give thanks to the following people who gave me much assistance, guidance and support during my thesis.

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I would like to thank my friends and family for their support and patience during my thesis, especially during the final writing.

I would like to dedicate this thesis to the memory of Mr Dirk Crafford of the Department of Water Affairs and Forestry who passed away during the development of the new Orange Fish Sundays Model.

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# List of Symbols

| Symbol | Description |
|---|---|
| A | Cross sectional area of flow |
| g | Gravitational constant (9.81 m/s$^2$) |
| I | Inflow |
| P | Wetted Perimeter |
| Q | Outflow or discharge |
| q | lateral discharge per unit length of channel |
| R | Hydraulic Radius |
| S | Storage |
| $S_f$ | Friction slope |
| $S_o$ | Bed slope |
| t | Time step |
| T | Time |
| V | Flow velocity |
| x | Distance step |
| y | Depth of flow |
| $F_f$ | Flow Factor |
| $B_f$ | Base flow |
| $K_U$ | High flow rate storage constant |
| $K_L$ | Low flow rate storage constant |
| $T_f$ | Transition flow rate between high flow and low flow rate |
| $E_m$ | Modelled evaporation |
| a | Amplitude factor |
| b | Frequency factor |
| c | Vertical shift |
| s | Horizontal shift |
| d | Date in days |

# Glossary of Terms

| | |
|---|---|
| ASCE | American Society of Civil Engineers |
| CSV | Comma Delimited |
| DTM | Digital Terrain Model |
| DWAF | Department of Water Affairs and Forestry |
| EC | Electro Conductivity |
| FISUN | Fish Sundays |
| FSC | Full Supply Capacity |
| FSL | Full Supply Level |
| HFL | High Flood Level |
| ICE | Institute of Civil Engineers |
| IDZ | Industrial Development Zone |
| MAF | Mean Annual Flow measured at Flow Gauging Station |
| MAR | Mean Annual Runoff as calculated from Catchment |
| masl | Meters Above Mean Sea Level |
| MOL | Minimum Operating Level |
| OFS | Orange-Fish-Sundays |
| SMS | Short Message Service |
| TDS | Total Dissolved Salts |

# 1 Introduction

## 1.1 Background

The Orange Fish Sundays (OFS) scheme transfers 700 million $m^3$ per year of water from the Orange River Catchment in the Free State to the water scares Fish River and Sundays River Catchments of the Eastern Cape Province in South Africa (see Figures 1.0-1 and 1.0-2). The scheme was initiated for irrigation purposes along the banks of the Great Fish River but over the years its role has changed. Increased urban growth along the coast due to rural to urban migration as well as increased industrial development in Port Elizabeth and the new Coega IDZ has increased the demand for water to this region. With additional demand being placed on the Orange River from Namibia as well as water transfer schemes in Lesotho reducing the Mean Annual Flow (MAF) at gauging sites on the Orange River, it has become imperative that South African water resources are managed with care and accuracy.

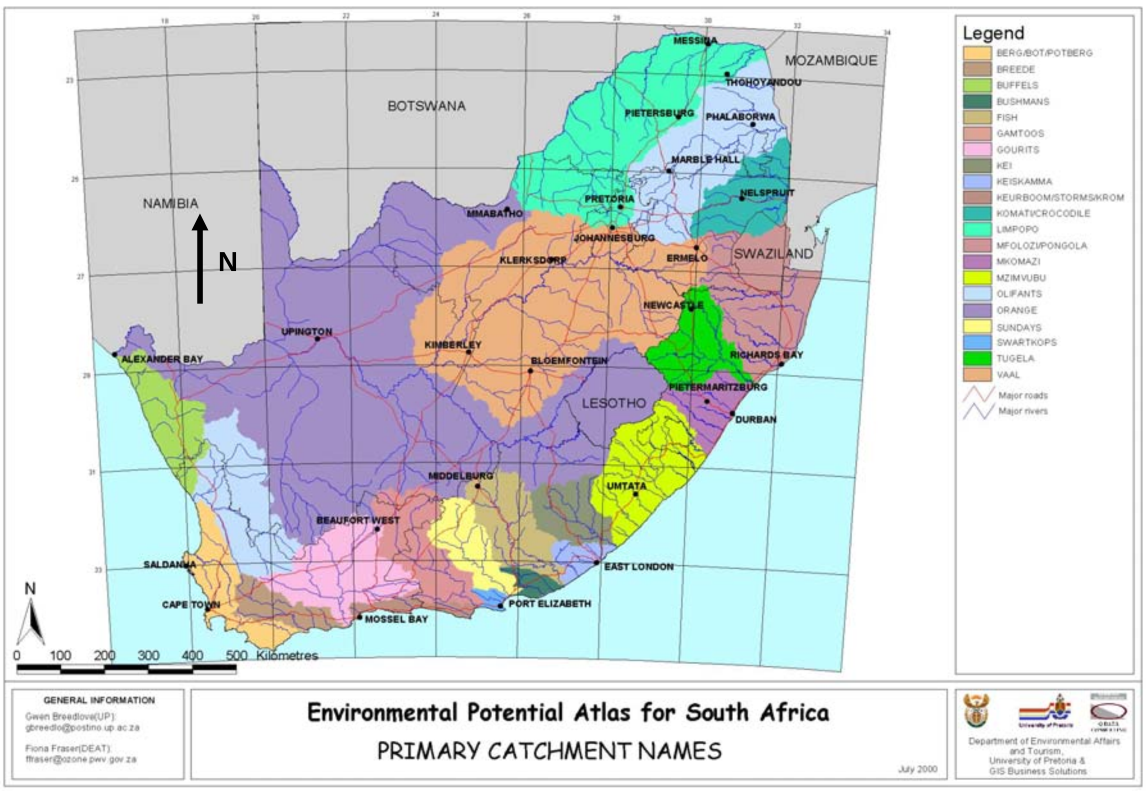



**Figure 1.1-1: Primary Catchment Areas of South Africa (Department of Environment and Tourism)**

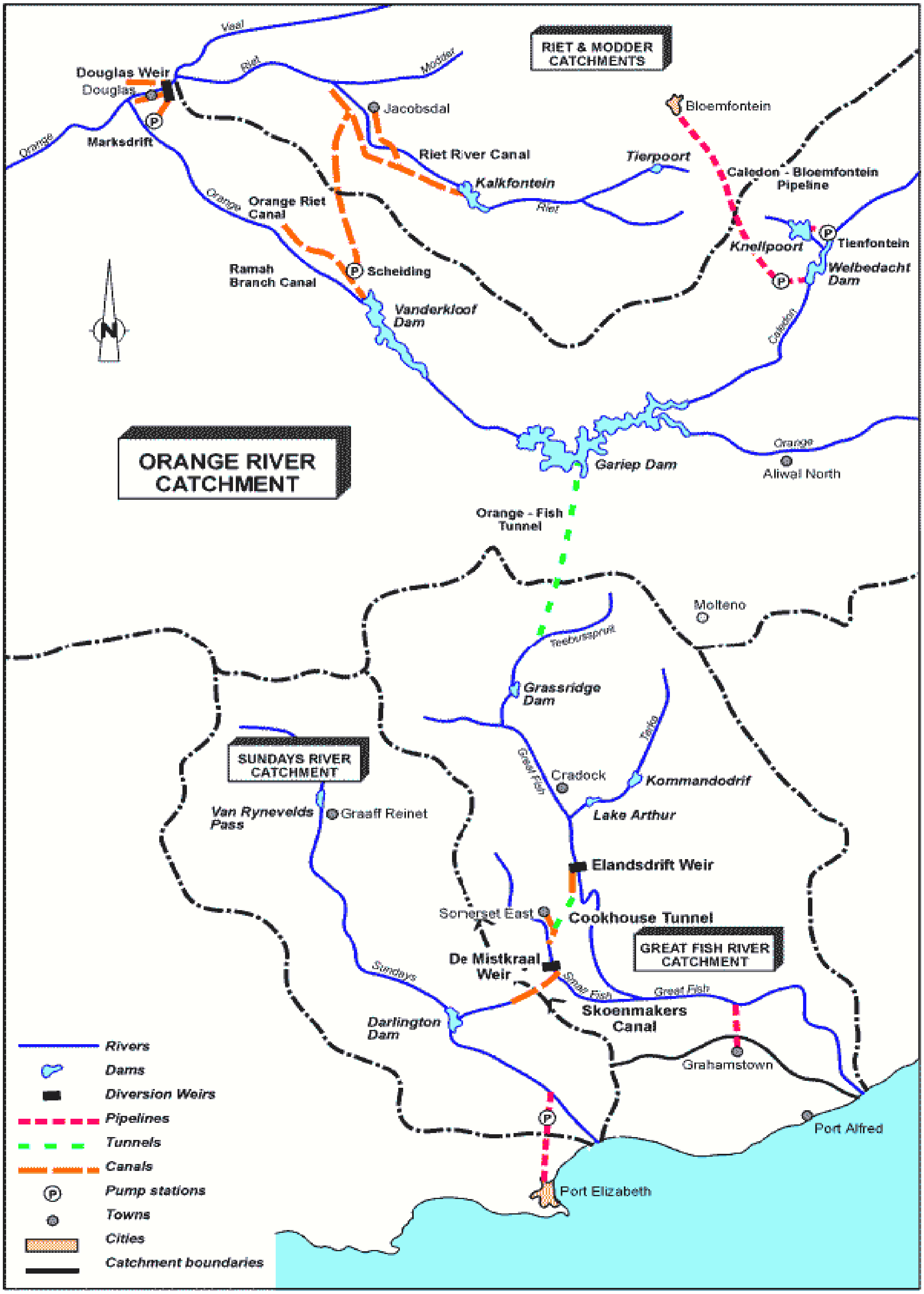



**Figure 1.1-2: Schematic of the Orange River Transfer Project (Department of Water Affairs and Forestry Website)**

The OFS scheme is currently modelled by the FISUN (Fish Sundays) model, which was developed in the late 1980's and uses a simplified flow routine to lag the demands up river to the control structures where the required flow release pattern is calculated. In addition to the flow routing, the water quality was also modelled and if required, refreshing releases were recommended by the program. This program, although sophisticated for its time is unable to take into account the variations in tributary inflow or stream flow that results in a river system over time. Thus over compensation of flows and conservative operation has lead to an estimated 200 millions of $m^3$ of flow per annum, which is transferred from the Orange River at great cost, being lost to the ocean.

## 1.2 Motivation and Objective

The model that has been developed as part of this thesis and as part of a Department of Water Affairs and Forestry (DWAF) contract is aimed to operate this scheme more efficiently and with the use of Real-Time hydrodynamic river modelling be able to store more tributary inflows in order to minimise the flows out of the system at the downstream boundary of the model, in this way providing more of this most precious resource to other users in South Africa.

The motivation for this thesis was given by the need to limit the losses from the river system due to the scare supply of water within the country along with increased transfers of water from the Orange River to as far away as Gauteng in the north and additional agriculture along the banks of the Lower Orange River.

The objective of the thesis was to develop a hydrodynamic flow routing model that could simulate the flows in the river system. This model is to be operated in real-time with data being received from field stations measuring the stage, flow and water quality in the river system. This model is to be used to forecast the flow releases from the control structures required to satisfy the water demands of both the irrigators along the river system as well as the domestic requirements while reducing the volume of water transferred from the Orange River as well as the losses from the downstream boundaries. This model is to be user friendly and to be able to be operated by non-technical staff with as little training costs as possible.

## 1.3 Methodology

In order to achieve these objectives, a commercially available fully hydrodynamic model is to be evaluated and then calibrated to represent the seasonal flow conditions of the river system. This calibrated model is then to be used to optimise the required flow releases from the control structures. A system of field gauging stations is to be setup to send data in real time to a central station which is to evaluate and check the data. The tributary inflows are to be forecasted for the forecast period. Pre-Process programs will be needed to manage the real-time data and prepare it for use within the optimisation model as well as to allow for the easy data entry of the requested abstractions along the river system by a non-technical operator.

## 1.4 Literature Review

The internet as well as the journals of the ASCE was the source for the literature review. In addition a request was put to the ICE for other sources of information on the thesis topic. The value of the journals and articles retrieved from the search was not nearly as satisfactory as was hoped. The majority of real-time river management systems in the world are primarily concerned with flood forecasting and the prediction of flood stage levels as shown by the papers by S. Sinclair and G. Pegram as well as by L.D. James and S.F. Korom. While these bear some similarity to this thesis, the main aspect of this thesis is the daily operation of the control structures to assure supply of water to the irrigators along the river system as well as to reduce the loss of transferred water out of the downstream boundaries.

The paper by R. Wardlaw and J. Barnes details their work on the development of a model to determine the optimum operation of the control structures for a complex river system to minimise the crop losses. The system was driven by the required soil moisture conditions which are determined by the crop type as well as the recent rainfall and the operation of the system. The OFS scheme is driven by the irrigator's requests for water which are dependant purely on the whims of the irrigators. As such there was little true correlation between the two studies.

The article on the real-time management of the San Joaquin River in California discussed the development of a system to control refreshing releases to maintain the water quality of the river within the limits set by the authorities. Although the water quality aspect of the model has been developed in conjunction with the hydrodynamic model, water quality does not form part of this thesis as its inclusion would be too cumbersome.

The evaluation of the various models was done in conjunction with the developers of the models along with the relevant user manuals and references. The details of these evaluations are provided in Chapter 3.

The structural data for the development of the model was attained from various reports on the dam capacity determination studies as well as operation and maintenance manuals from the Department of Water Affairs and Forestry (DWAF). The 1:50 000 Maps and 1:10 000 Orthophotos were obtained from the Department of Land Affairs.

# 2 Orange-Fish-Sundays Scheme Case Study

The Orange Fish Sundays Scheme is located in the Eastern Cape Province in South Eastern South Africa. This large and complex system includes a catchment of more than 75 000 $km^2$ and over 800 km of rivers, canals and tunnels. The scheme has been broken up into 9 basic reaches as shown in Table 2.0-1.

**Table 2.0-1: Reaches of the Orange-Fish-Sundays Scheme**

| Reach # | Upstream Station | Downstream Station | Length |
|---|---|---|---|
| 1 | Teebus OVIS Outlet | Grassridge Dam | 59 km |
| 2 | Grassridge Dam | Waaikraal Weir | 70 km |
| 3 | Waaikraal Weir | Elandsdrift Dam | 74 km |
| 4 | Elandsdrift Dam | Sheldon Weir | 146 km |
| 5 | Sheldon Weir | Fort Brown Bridge (End of model) | 164 km |
| 6 | Elandsdrift Canal | De Mistkraal Dam | 95 km |
| 7 | De Mistkraal Dam | Junction Drift (Great Fish River Confluence) | 58 km |
| 8 | De Mistkraal Canal | Darlington Dam | 87 km |
| 9 | Darlington Dam | Korhaansdrift Weir (End of model) | 49 km |

A schematic layout is shown in Figure 2.0-1.

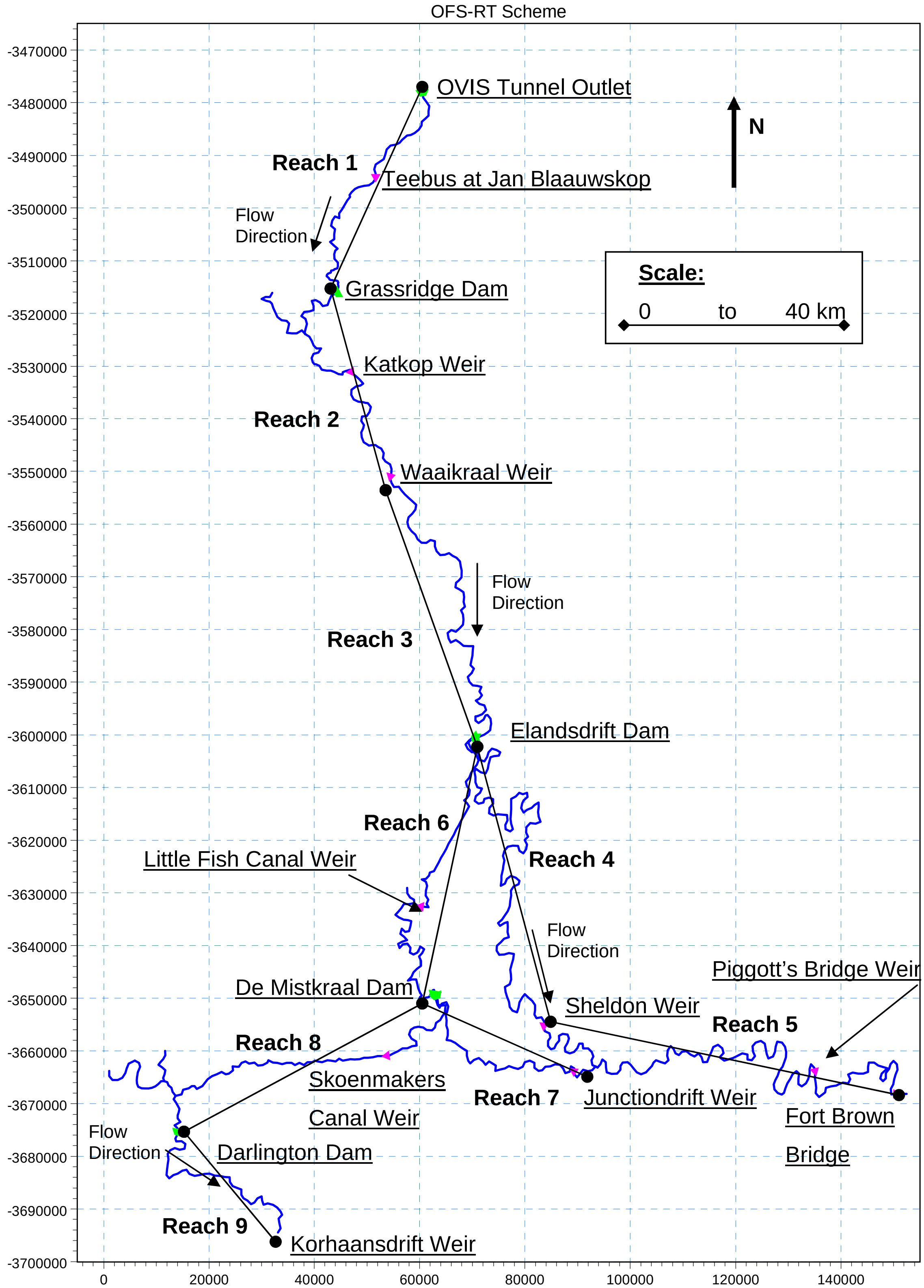



**Figure 2.0-1: Schematic Layout of the Orange-Fish-Sundays Scheme Showing the Locations of each Reach**

## 2.1 Reach 1: Teebus OVIS Outlet to Grassridge Dam

The idea for the Orange-Fish-Sundays scheme was first devised by Dr. A.D. Lewis in 1928 (DWAF, 2002) when he planned to divert the Orange River under the Bosberg Mountains to the Teebus River. The scheme was to supply irrigation water to the Groot Vis River valley increasing the arable land and providing relief from droughts. The scheme was eventually implemented in the 1970's with the completion of the Gariep Dam in 1971. The 82.5 km long OVIS Tunnel was then completed in 1975 to divert a maximum of 25% of the mean annual runoff (MAR) of the Orange River to the Groot Vis River valley via the Teebus and Groot Brak rivers. This is the starting point of the model. The region is arid Karoo scrubland where the predominant agriculture is sheep farming. The annual average rainfall is 303 mm per annum almost half the national average of 550 mm per annum. Figure 2.1-1 shows the daily rainfall for the last 18 months from January 2005.

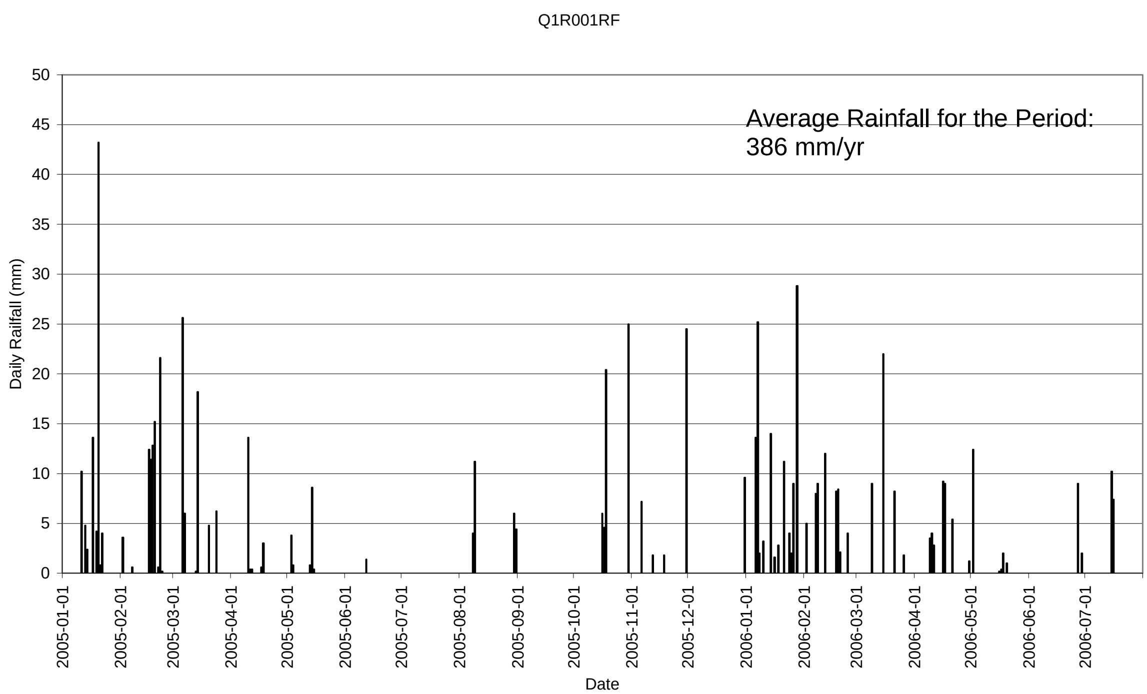



**Figure 2.1-1: Daily Rainfall as recorded at Grassridge Dam rain gauge from January 2005 to July 2006**

Figure 2.1-2 shows Reach 1 from a satellite image taken from the Google Earth website. One can clearly see that the green areas indicate that irrigation is limited to the banks of the river whilst the surrounding country is dry and arid. The mean annual flow values are calculated for the 2002 to 2005 period unless otherwise noted.

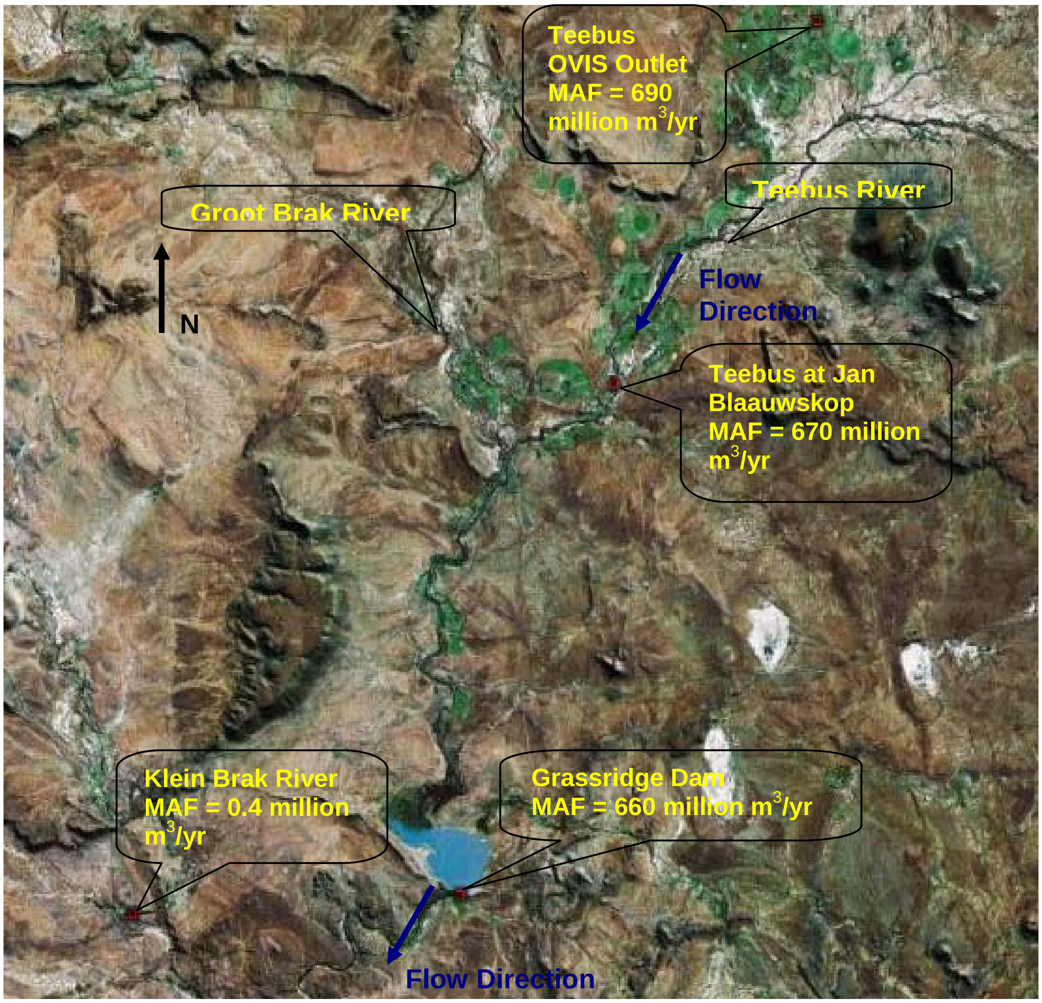



**Figure 2.1-2: Satellite Image of Reach 1 from Teebus OVIS Outlet to Grassridge Dam**

The Teebus flows into the Groot Brak River which is un-gauged and only flows for a short duration during infrequent storm rainfall events. Along the Teebus and Groot Brak River banks the local farmers abstract an average of 62.5 million $m^3$/yr for irrigation. This is 9% of the Mean Annual Flow (MAF) from the OVIS Outlet at Teebus. MAF is defined as the mean annual flow through a gauging weir as opposed to MAR which is the mean annual runoff from a catchment. The reach ends at Grassridge Dam completed in 1924 by the Great Fish Irrigation Board. Grassridge Dam had an as built capacity of 90.9 million $m^3$ which has reduced due to sedimentation to 46 million $m^3$ according to the capacity determination study conducted in 2000 (DWAF, 2000). Due to restrictions imposed by the Dam Safety Office of the Department of Water Affairs and Forestry, the dam can only operate at a full supply level of 1054.64 masl with a capacity of 14.3 million $m^3$. Grassridge Dam acts as a storage dam to supply water to the irrigators downstream if there is an interruption in supply through the OVIS tunnel.

## 2.2 Reach 2: Grassridge Dam to Waaikraal Weir

From Grassridge the flow to the Groot Brak River is controlled via the diversion works at the dam. Along this reach the Klein Brak tributary joins the river and then the Groot Brak flows into the Groot Vis River. The Pauls River flows into the Groot Vis River just upstream of the flow gauging station at Waaikraal. There is an average 91 million $m^3$/yr of abstractions along Reach 2 during 2005 and 2006 which is 14% of the release from the OVIS Tunnel. Figure 2.2-1 shows Reach 2 from a satellite image taken from the Google Earth website.

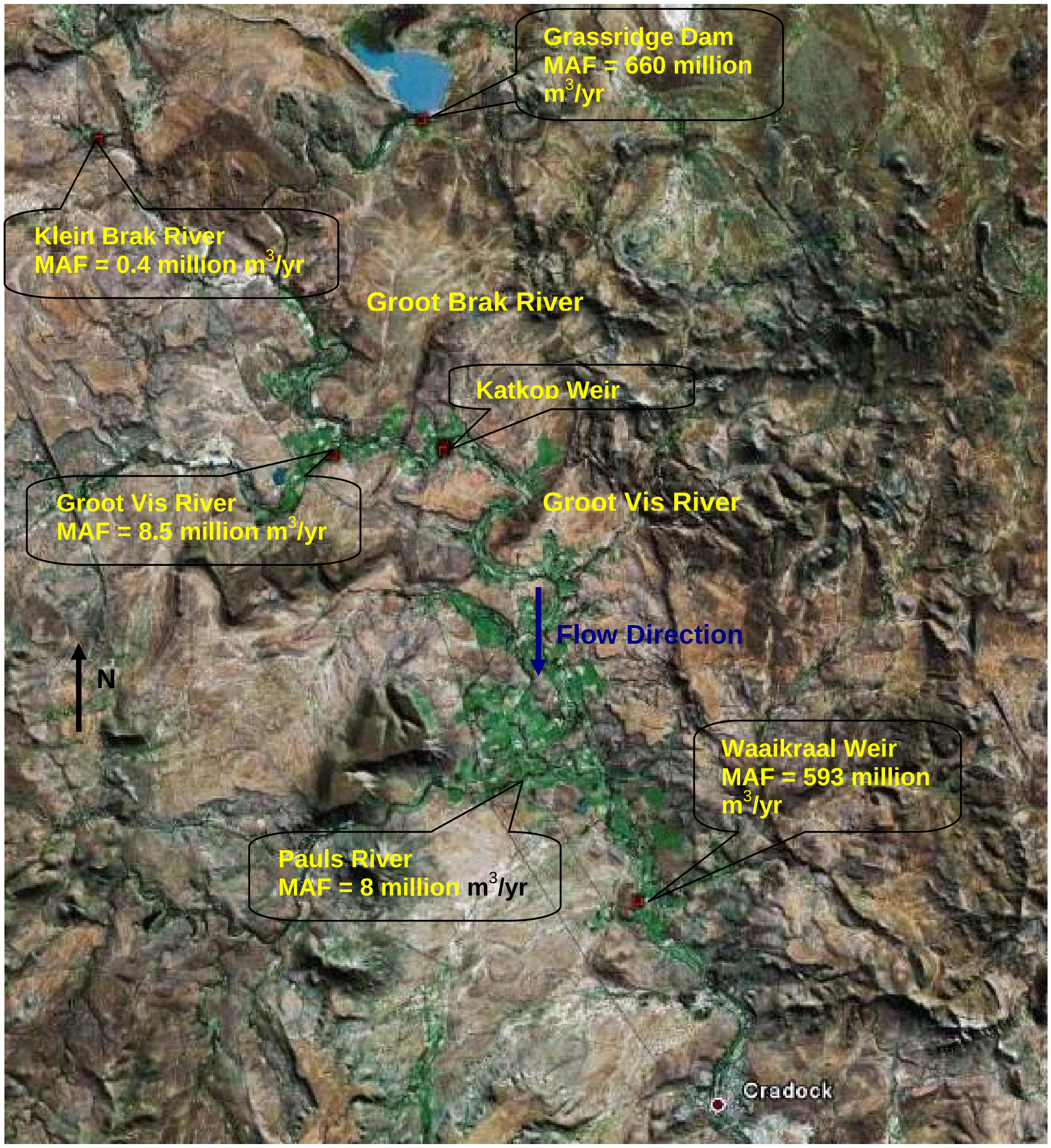



**Figure 2.2-1: Satellite Image of Reach 2 from Grassridge Dam to Waaikraal Weir**

## 2.3 Reach 3: Waaikraal Weir to Elandsdrift Dam

From Waaikraal Weir the flow continues through the town of Cradock towards Elandsdrift Dam. The Tarka River joins the Groot Vis River 41 km downstream of Waaikraal Weir. There are two Private Irrigation Board dams built on the Tarka River, these are Kommandodrift and Lake Arthur. These dams are not operated as part of the Orange Fish Sundays Scheme as they are privately owned and operated, thus they are not included in the model although the inflow from the Tarka River is gauged and included in the model. Figure 2.3-1 shows Reach 3 from a satellite image taken from the Google Earth website.

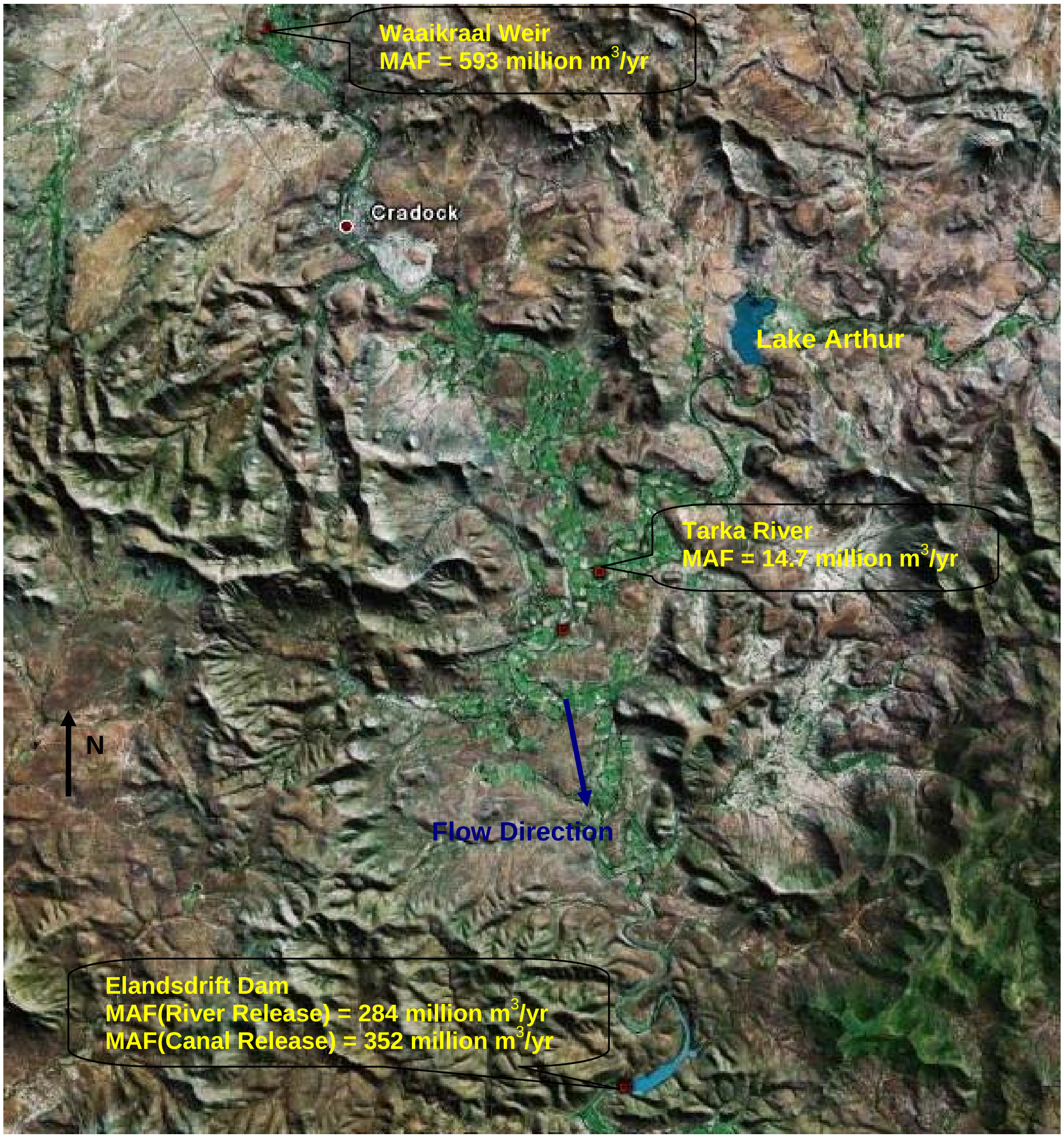



**Figure 2.3-1: Satellite Image of Reach 3 from Waaikraal Weir to Elandsdrift Dam**

The area receives a higher rainfall that that of reaches 1 and 2, this is due to the orographic rainfall that falls against the escarpment. Reach 3 accounts for 169 million $m^3$/yr of abstraction from the scheme which is 26% of the flow from the OVIS Tunnel. This is the largest abstraction of all the reaches. Reach 3 receives an average of 401 mm of rainfall per annum as recorded at Elandsdrift Dam. The daily rainfall for the last 18 months from January 2005 is shown in Figure 2.3-2.

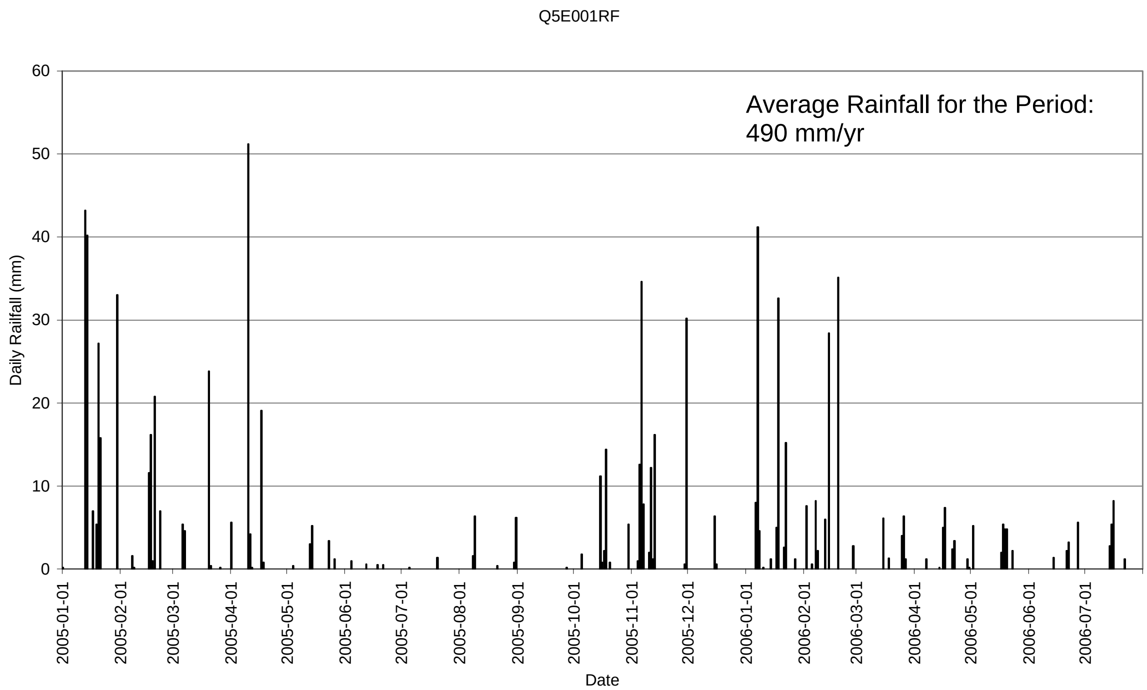



**Figure 2.3-2: Daily Rainfall as Recorded at Elandsdrift Dam Rain Gauge from January 2005 to July 2006**

Elandsdrift Dam was built in 1973 with a Full Supply capacity of 12 million $m^3$. The dam was built as a flood control structure and as a diversion works for the Little Fish Canal (see Figure 2.3-3) which starts at the dam. The Canal has a maximum capacity of 20 $m^3$/s. Elandsdrift Dam is fitted with 4 large radial flood gates each with a fish belly flap (See Figures 2.3-4 and 2.3-5). These flaps can be used to pass debris as well as lower the water level in the dam though these are un-gauged. The dam has three sluice valves which can divert a total maximum of 12 $m^3$/s of flow to the Groot Vis River. One of the Sleeve Valves is shown in Figure 2.3-6. There are three additional vertical sluice gates from the canal to the Groot Vis River shown in Figure 2.3-7. These can be used if additional flow is required to be passed to the Groot Vis River without having to raise one of the large radial flood gates which have a minimum release of 30 $m^3$/s at full supply level (FSL).

Elandsdrift is the pivotal structure on the Orange Fish Sundays Scheme. From here the water from the OVIS Tunnel is either released to reaches 4 and 5 on the Groot Vis River or diverted to Reach 6 on the Little Fish towards De Mistkraal Dam and on to Darlington Dam on the Sundays River.

**Figure 2.3-3: Elandsdrift Dam Vertical Sluice Gates to Canal**


**Figure 2.3-4: Elandsdrift Dam Radial Gate with Fish Belly Flaps lowered and Stop-Log in Place**


**Figure 2.3-5: Close up Picture of the Fish Belly Flap on the Radial Gates of Elandsdrift Dam**

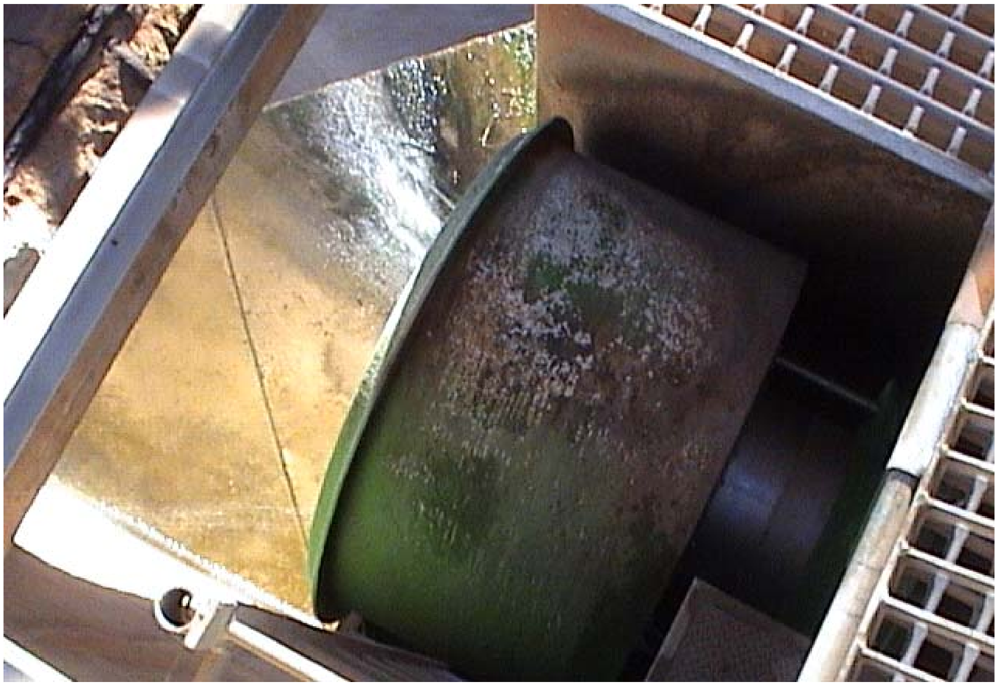


**Figure 2.3-6: A Close up Picture of one of three Sleeve Valves to Release Water from Elandsdrift Dam to the Groot Vis River**

**Figure 2.3-7: Elandsdrift Dam Vertical Sluice Gates from Canal to Groot Vis River**

## 2.4 Reach 4: Elandsdrift Dam to Sheldon Weir

Reach 4 accounts for 17% of the abstraction from the Orange Fish Sundays Scheme, an average of 114 million m$^3$/yr. The Groot Vis River winds its way through a narrow valley then opens up to a large floodplain. Figure 2.4-1 shows a satellite image of Reach 4 taken from the Google Earth website. The Baviaans River flows into the Groot Vis River 67 km downstream of Elandsdrift Dam. Middleton Weir at chainage 104 km is the last major irrigation abstraction on the Groot Vis River. The reach ends at Sheldon Weir 146 km downstream from Elandsdrift Dam.

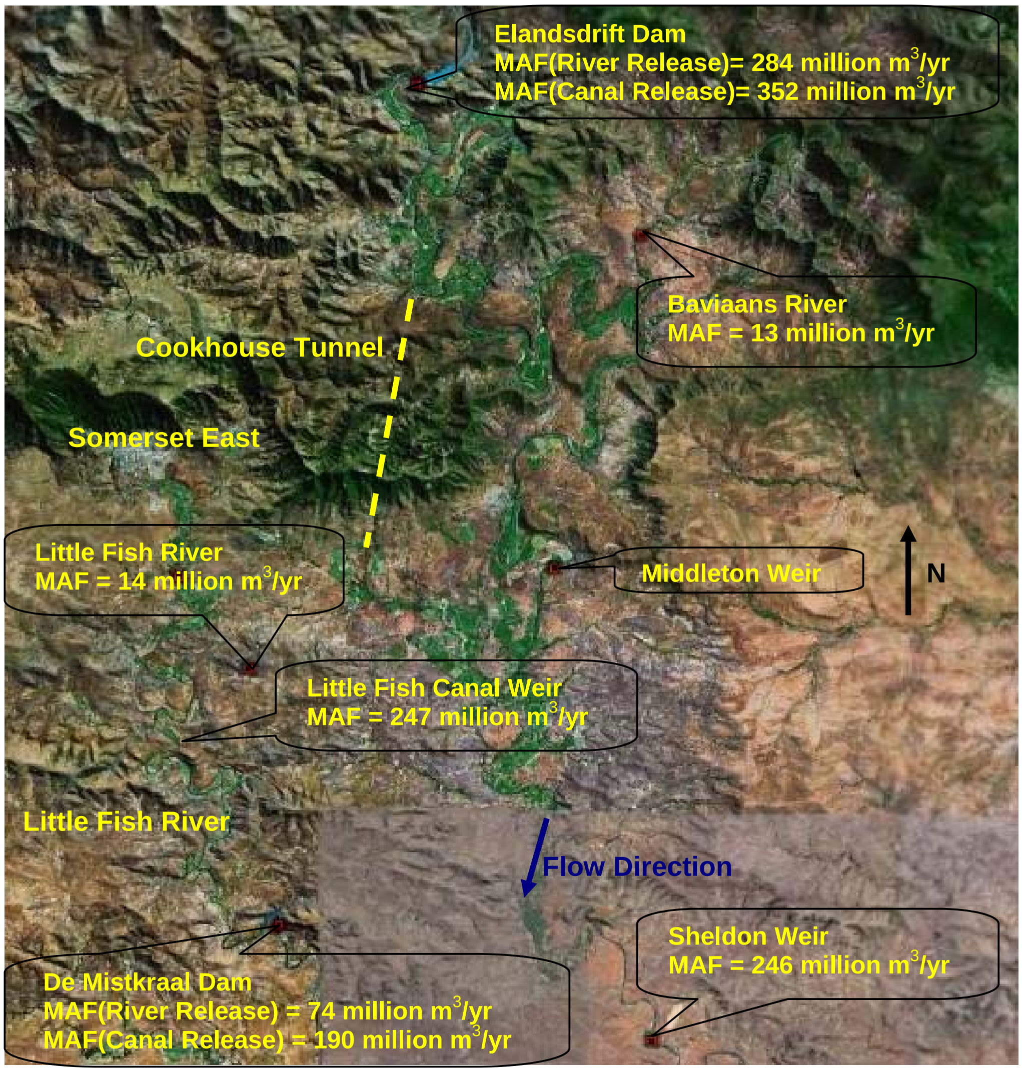



**Figure 2.4-1: Satellite Image of Reach 4 from Elandsdrift Dam to Sheldon and Reach 6 from Elandsdrift Dam to De Mistkraal Dam**

## 2.5 Reach 5: Sheldon to Fort Brown Bridge

There are no licensed abstractions below Sheldon Weir. The flow is joined by the Little Fish River 28 km downstream of Sheldon Weir and continues through land which is used mostly for Game farming. Reach 5 is the longest of the reaches at 164 km. The flow can be diverted at Hermanuskraal Diversion Weir (see Figure 2.5-1) into the Ecca Tunnel which diverts flow to Glen Melville Dam, an of channel storage dam for domestic and irrigation use. This dam is usually only filled 2 to 3 times a year as and when required. There is no flow measurement possible at Hermanuskraal Diversion Weir due to an un-gauged sluice gate and fish ladder. Figure 2.5-2 shows a satellite image taken from the Google Earth website.

**Figure 2.5-1: Hermanuskraal Diversion Weir on the Groot Vis River**

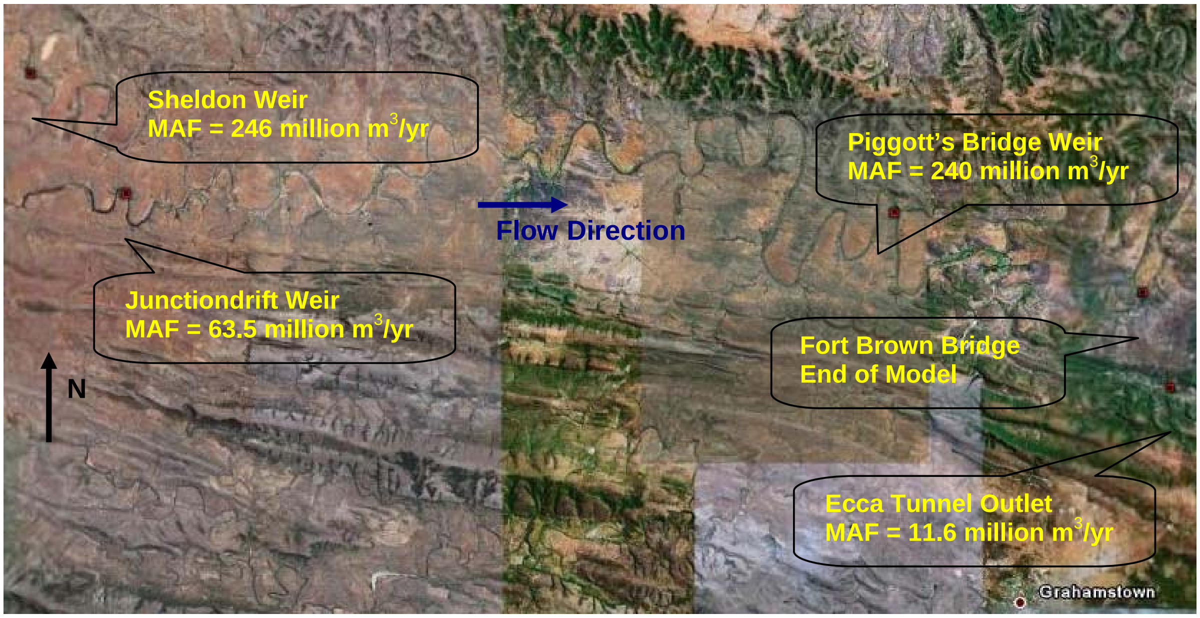



**Figure 2.5-2: Satellite Image of Reach 5 from Sheldon to Fort Bown Bridge**

## 2.6 Reach 6: Elandsdrift Dam to De Mistkraal Dam

From Elandsdrift Dam, the flow can be diverted into the Little Fish Canal. This canal follows the eastern side of the valley and then passes under a mountain range via the Cookhouse Tunnel to be transferred to the Little Fish River just South of Somerset East. Figure 2.4-1 shows a satellite image of this reach. The 95 km reach incorporates 45 km of canal and the 13.1 km of the Cookhouse Tunnel completed in 1978. The annual volume of abstraction along this reach is 64.8 million $m^3$/yr, which is 10% of the total flow from the OVIS Tunnel.

De Mistkraal Dam was completed in 1987 enabling water from the Orange River to be transferred via the Groot Vis River to the Sundays River. The Initial Storage capacity was 4 million $m^3$. This has reduced to 2.6 million $m^3$ by 2002 due to sediment deposition. De Mistkraal Dam is primarily a diversion weir, as it has limited storage capacity. It has an un-gated free overflow spillway as shown in Figure 2.6-1. The Diversion works are on the left hand side with the free overflow spillway in the centre.

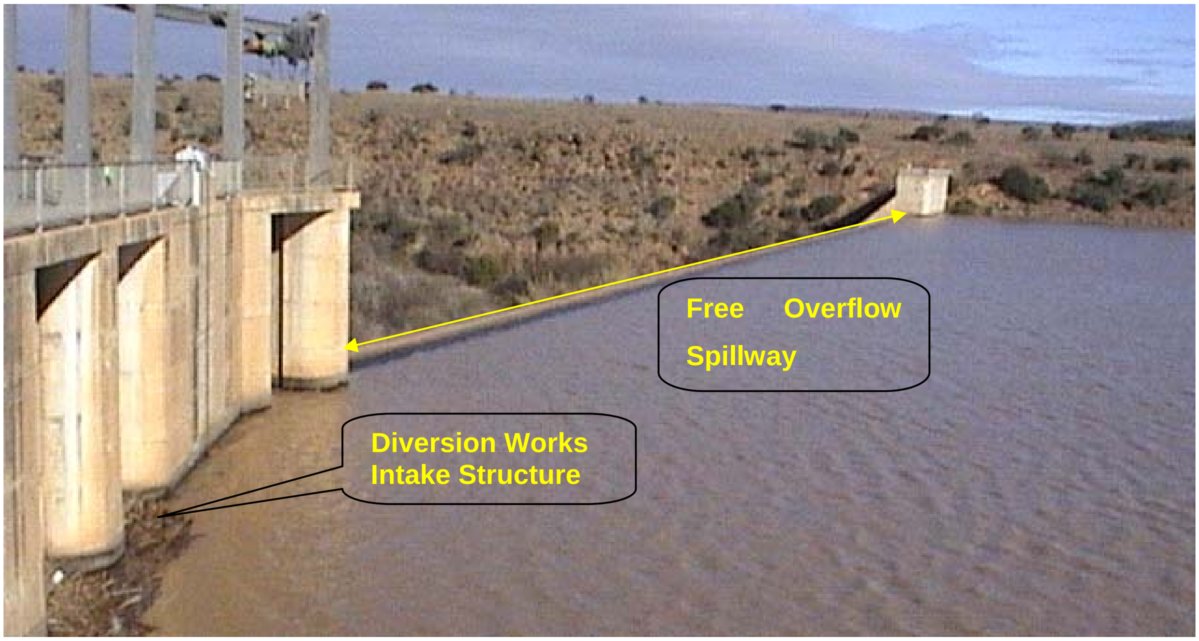



**Figure 2.6-1: De Mistkraal Dam as seen from Upstream**

Flow can be released to the Little Fish River via two sleeve valves, one of which is shown in Figure 2.6-2. A maximum flow of 19 $m^3$/s can be diverted via the canal at De Mistkraal Dam to the Skoenmakers River which is a tributary of the Sundays River upstream of Darlington Dam.

**Figure 2.6-2: One of the Sleeve Valves to Release Flow to the Little Fish River at De Mistkraal Dam**

The Little Fish River valley is similar to the Groot Vis River valley and receives an average of 425 mm of rainfall per year. Figure 2.6-3 shows the daily rainfall recorded at De Mistkraal Dam from January 2005 to July 2006.

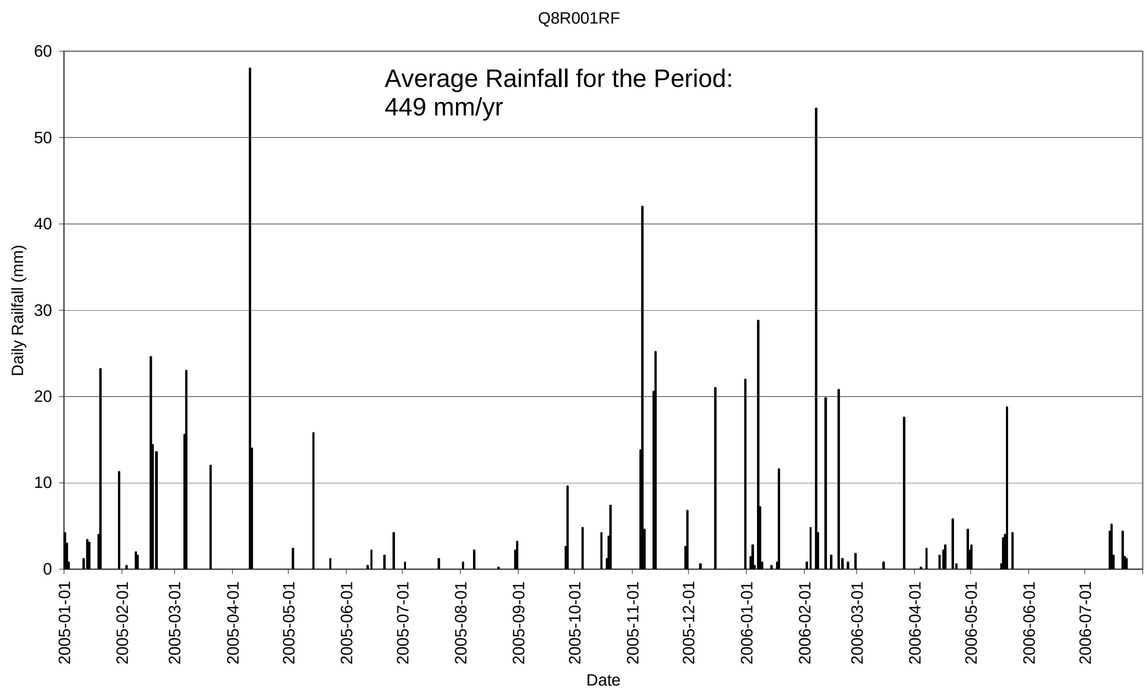



**Figure 2.6-3: Daily Rainfall as Recorded at De Mistkraal Dam Rain Gauge from January 2005 to July 2006**

## 2.7 Reach 7: De Mistkraal Dam to Junctiondrift Weir

At De Mistkraal Dam, flow can be released through the sleeve valves or pass over the spillway into the Little Fish River. This Reach is 58 km long and has an average abstraction of 22 million $m^3$/yr, which is 3% of the total transfer from the OVIS Tunnel. The water that is released to the Little Fish River at De Mistkraal flows back to the Groot Vis River below Sheldon Weir.

## 2.8 Reach 8: De Mistkraal Dam to Darlington Dam

Up to 19 $m^3$/s can be transferred via the Skoenmakers Canal from De Mistkraal Dam to the Skoenmakers River. This 28 km long canal was the last phase in the construction of the current Orange Fish Sundays Scheme. Figure 2.8-1 shows a Satellite image of reaches 8 and 9 taken from the Google Earth website.

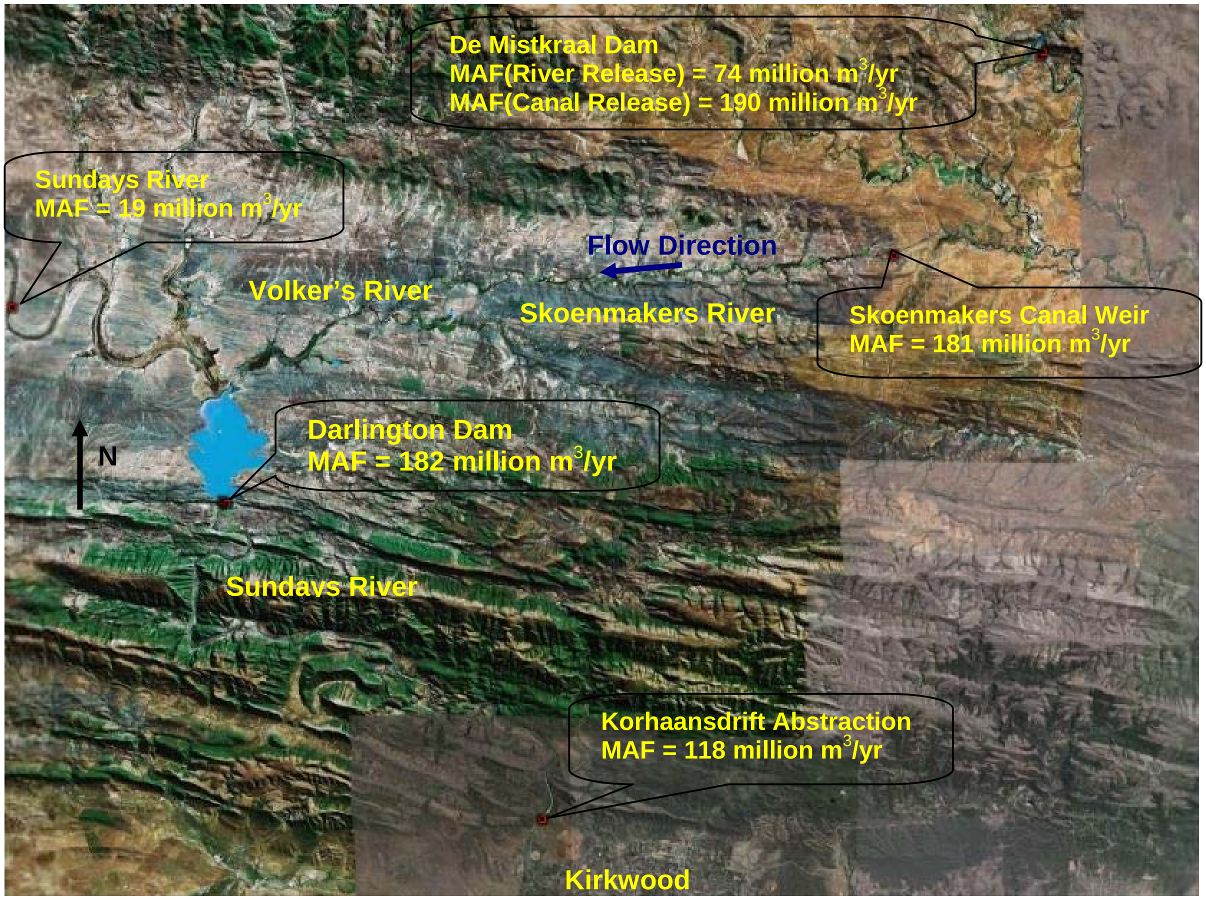



**Figure 2.8-1: Satellite Image of Reach 8 from De Mistkraal Dam to Darlington Dam and Reach 9 from Darlington Dam to Korhaansdrift Weir**

An average of 15 million m$^3$/yr is abstracted from Reach 8 annually, which accounts for 2% of the total transfer from the OVIS Tunnel.

Darlington Dam is on the Sundays River which flows through the arid Little Karoo. Darlington Dam was completed in 1922 (DWAF, 2002), see Figure 2.8-2, with an initial full supply capacity of 327 million m$^3$. The current capacity has been reduced to 180 million m$^3$ by sedimentation, but further restrictions have been imposed by the Dam Safety Office of the Department of Water Affairs and Forestry due to poor maintenance of the Stoney flood gates, effectively restricting the Full Supply Capacity to 80 million m$^3$.

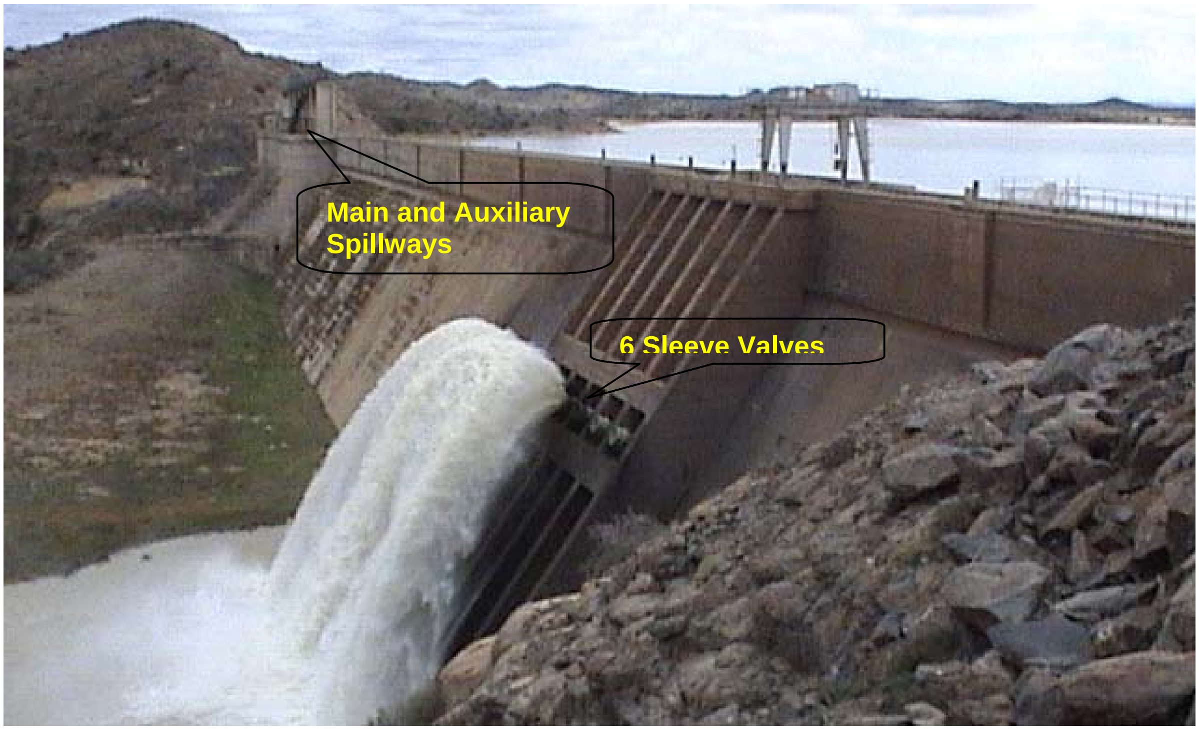



**Figure 2.8-2: Darlington Dam Showing the Position of the Diversion Works and Spillways**

The annual rainfall in the region is just 256 mm per year as measured at Darlington Dam. See Figure 2.8-3 for the daily rainfall recorded at Darlington Dam from January 2005 to June 2006. The inflow from the Sundays River accounts for just over 10% of the total inflow into Darlington Dam for the period from 1980 to 2005. This shows the schemes strong reliance on the water from the Orange River.

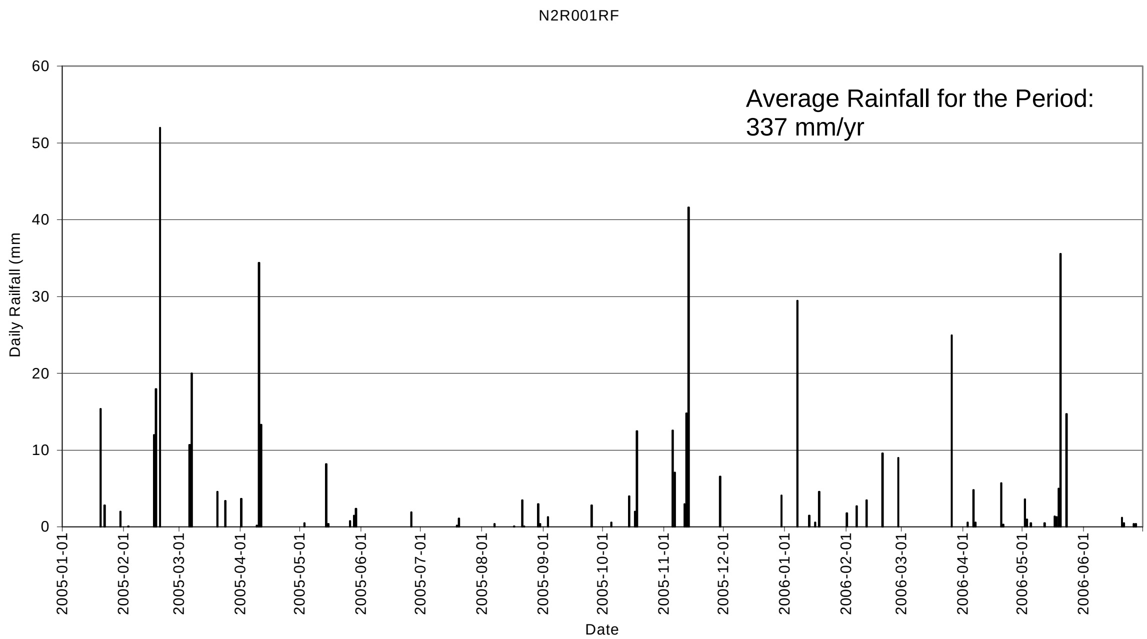



**Figure 2.8-3: Daily Rainfall as Recorded at Darlington Dam Rain Gauge from January 2005 to June 2006**

## 2.9 Reach 9: Darlington Dam to Korhaansdrift Weir

Darlington Dam has 6 Sleeve Valves that can release water to the Sundays River with a total discharge of 60 $m^3/s$ at High Flood Level (HFL). In addition to this there are two gated spillways. The Main Spillway with a crest elevation of 238.0 masl and a maximum discharge capacity of 3025 $m^3/s$ at HFL. The Auxiliary Spillway's gates are left open to reduce the risk of the dam suffering damage in an extreme event. This spillways crest is at 242.925 masl and has a maximum discharge capacity of 496 $m^3/s$ at HFL.

Flow released from the dam to the Sundays River is diverted into a canal at Korhaansdrift 45 km downstream from Darlington Dam. Virtually all the water that flows in the Sundays River at this point is diverted into the canal. The flow that passes over the weir at Korhaansdrift is seen as a loss to the system. Figure 2.9-1 shows the weir at Korhaansdrift.

**Figure 2.9-1: A View of the Weir at Korhaansdrift on the Sundays River**

# 3 Hydrodynamic Model Evaluation

A numerical model is a set of equations that model the behaviour of some physical element such that under a set of conditions the modelled event is similar to the actual physical event. In this case the physical event is the flow of water in rivers, canals and tunnels as well as the operation of the structures that control the flow. There are numerous models that are available, either commercially or developed for a specific project or for research purposes.

A desk study was conducted to determine what type of model would best suit the physical element that was to be modelled. It was necessary in the study to first establish the characteristics of the different types of available models and then to determine which are best suited to the purpose. The various models available are Kinematic or Dynamic models which can be either Lumped or Distributed. These are explained below.

## 3.1 Different Types of Numerical Models

### 3.1.1 Kinematic vs Dynamic

In reality the flow of water is in 3 dimensions, however for practical applications the flow across the channel and with respect to depth can be ignored. Thus only the flow along the channel is modelled, ie: one dimensional flow. Flow can also be steady or un-steady. Steady flow is defined as flow with a constant rate, while with un-steady flow the flow rate may vary with time.

The Saint-Venant Equations, first developed in 1871 by Barre de Saint-Venant describe one dimensional unsteady, open channel flow. As with any theoretical equation used to model nature, various assumptions have to be made. In the case of the Saint-Venant Equations these are:

- The flow is one dimensional; depth and velocity vary only in the longitudinal direction of the channel.
- Flow is assumed to vary gradually along the channel maintaining hydrostatic pressure and ignoring vertical accelerations.
- The longitudinal axis of the channel is approximated as a straight line.
- The bottom slope of the channel is small.
- Resistance coefficients used for steady open channel flow are applicable.
- The fluid is incompressible and uniform.

The Saint-Venant Equations are made up of two parts, the Continuity Equation:

$$V\frac{\partial y}{\partial x}+y\frac{\partial V}{\partial x}+\frac{\partial y}{\partial t}=0$$ **Equation 3.1-1**

And the Momentum Equation:

$$\frac{\partial V}{\partial t}+V\frac{\partial V}{\partial x}+g\frac{\partial y}{\partial x}-g(S_o-S_f)=0$$ **Equation 3.1-2**

Each term in the equations relates to a specific characteristic of the flow. The equations can be simplified by ignoring certain terms, making substitutions or assumptions. The first term in the Momentum Equation defines the local acceleration and the second term defines the convective acceleration. These two terms represent the inertial effect of the flow. The third term defines the pressure force acting on the fluid, while the second to last term define the gravitational force and the last one defines the frictional force acting on the fluid.

If one were to ignore the acceleration and pressure terms in the Momentum Equation and assume that $S_o = S_f$, only the forces associated with mass and force would be taken into account. As kinematics is the study of motion exclusive to the influence of mass and force, this new equation would define the kinematics of the flow or the Kinematic wave.

By including the effect of inertia on the fluid by including all the terms of the Momentum Equation we take into account back water effects. That is, if there is a change in water level or flow at a point in the channel and the flow is subcritical, the effects of this change in water level or flow rate can cause changes in the flow upstream of this point. This is called the Dynamic wave model. Simply put the difference between the Kinematic wave model and the Dynamic wave model is that the Kinematic wave model can not take into account backwater effect and the Dynamic wave model which can. If the backwater effects can be ignored, the simple Kinetic wave model can be used.

The way that the two models describe the motion of a wave or change in the flow differs for each of the models. As the Kinematic wave model ignores the inertial and pressure effects on the fluid, as such the gravity and frictional forces are balanced ($S_o = S_f$), the flow does not take into account acceleration changes, such as happen with the movement of a large flood. During an increase if flow, the greater flow rate moves faster than the lesser flow rate and as such wants to overtake it. This change in velocity or acceleration is not considered in a Kinematic wave model.

### 3.1.2 Lumped flow routing vs Distributed Flow Routing

Flow routing can be defined as the tracing of a flow through a system. Lumped flow routing calculates the hydrograph as a function of time while distributed flow routing calculates the hydrograph as a function of time and space.

In a lumped flow routing system, the Input I(t), Output Q(t) and storage S(t) are related by the continuity equation (V. te Chow, D.R. Maidment, L.W. Mays):

$$\frac{dS}{dt} = I(t) - Q(t)$$ **Equation 3.1-3**

I(t) is known but Q(t) and S(t) are not, one needs a storage function to relate Q(t) to S(t) to give the two unknowns and two equations so that one might solve them simultaneously. The storage function is often written as a function of I, Q and their time derivatives. The specific form of the storage function depends on the nature of the system being analysed. One can have an invariable or a variable relationship for the outflow and the storage. An invariable relationship is one where the outflow is determined by the storage level, such as level pool routing. This applies to open tracts of water where the level is considered to be horizontal and the outflow is determined as an invariable relationship to the storage level as shown in Figure 3.1-1a. Figure 3.1-1b shows how the maximum outflow intersects the receding inflow at the point of maximum storage.

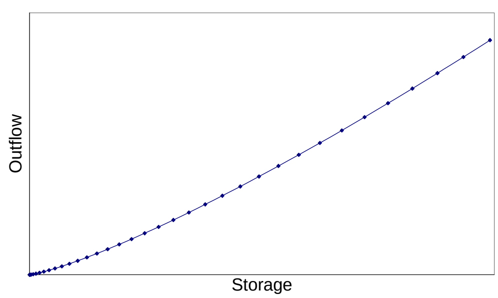



**Figure 3.1-1a: Invariable Relationship between Outflow and Storage (V. te Chow, D.R. Maidment, L.W. Mays)**

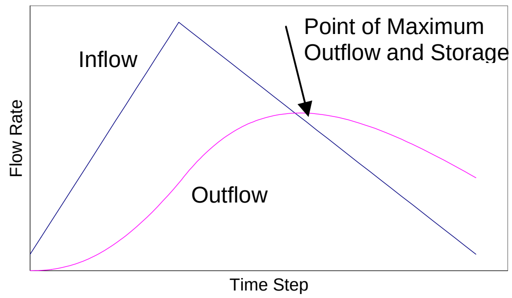



**Figure 3.1-1b: Invariable Relationship between Inflow and Outflow (V. te Chow, D.R. Maidment, L.W. Mays)**

A variable relationship applies to narrow dams or river channels where the water surface could be sloped or curved significantly due to back water effects. The volume of storage due to backwater effects can vary with time rate of change of flow as can be shown in Figure 3.1-2a. The relationship between Outflow and Storage is no longer a single value relationship but a single or twisted loop as shown, this results in a shifted outflow hydrograph as shown in Figure 3.1-2b. The peak outflow is not intersected by the Inflow recession as in the invariable case.

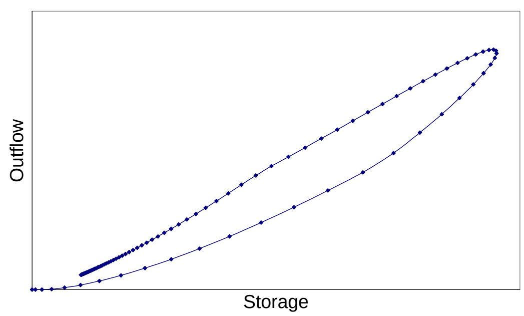



**Figure 3.1-2a: Variable Relationship between Outflow and Storage (V. te Chow, D.R. Maidment, L.W. Mays)**

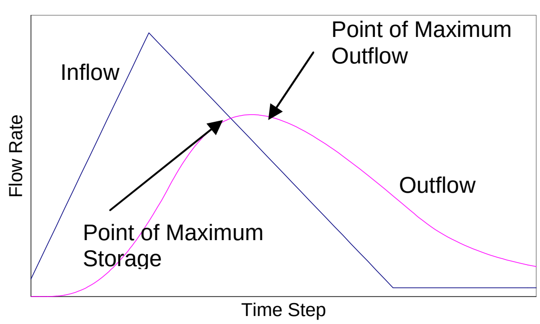



**Figure 3.1-2b: Variable Relationship between Inflow and Outflow (V. te Chow, D.R. Maidment, L.W. Mays)**

The Muskingum method is the most popular hydrologic routing method for variable outflow-storage relationships. The method models the combination of wedge and prism storage. During the advance of a flood wave the inflow is greater than the outflow thus creating a wedge of storage on top of the steady state prism storage in the river channel as shown in Figure 3.1-3.

Constants are used to define the characteristics of the prism and wedge storage and how they vary for different conditions. This is done on a reach for reach basis lumping the flow from point to point. These simple calculations have proved very popular while computer processing ability was in its infancy, with more powerful computers, the full Saint-Venant Equations can be solved.

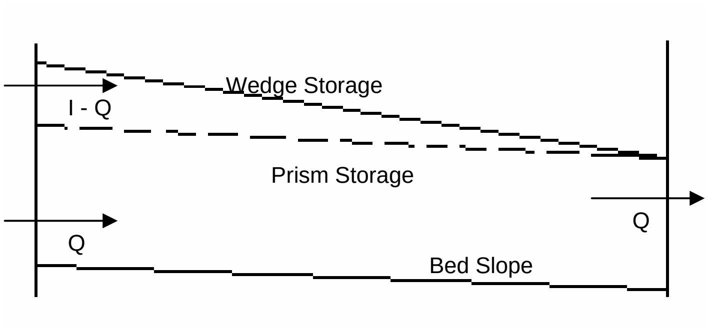



**Figure 3.1-3: Schematic of the Muskingum Wedge and Prism Storage Principle (V. te Chow, D.R. Maidment, L.W. Mays)**

As water flows through a system, the flow rate, velocity and depth vary in space and time. Estimates of flow rate and depth can be obtained by using a distributed flow routing routine. Distributed flow routing is based on partial differential equations, for example the Saint-Venant Equations. These partial differential equations allow the flow rate and depth to vary as functions of space and time. Simple distributed schemes make use of the Kinematic wave model as discussed while more complex models find solutions for the Dynamic wave model. The partial differential equations can be solved using either explicit or implicit schemes. An explicit scheme uses a forward-difference scheme for the time derivative and a central-difference scheme for the special derivative. This can be explained with the aid of Figure 3.1-4.

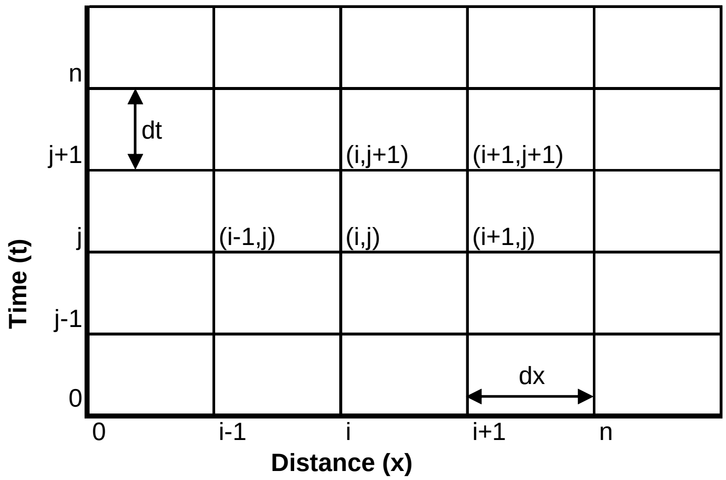



**Figure 3.1-4: A Grid on the x-t plane Used for the Numerical Solution of the Saint-Venant Equations by Finite Difference (V. te Chow, D.R. Maidment, L.W. Mays)**

If one assumes that at time (j) one knows the solution for flow rate and depth, a difference method including (i-1, j), (i, j) and (i+1, j) could be used to find the partial derivative at point (i, j+1). An implicit scheme uses a finite difference approximation for both the time and space derivatives to determine the solution at point (i+1, j+1). There are numerous variations of finite difference approximations that can be used; the simplest options have been briefly explained here in the text.

In summery, a lumped flow routing scheme calculates the outflow from a fixed reach based on assumptions that characterise that reach. A distributed flow routing scheme finds solutions for finite difference equations based on a grid in the time-space frame, thus finding a distributed solution at discrete points all along the reach.

## 3.2 OFS-RT Model Requirements

The Department of Water Affairs and Forestry operates the Orange-Fish-Sundays Scheme and wished to upgrade their existing flow routing model. The new model was to perform certain pre-defined tasks which were the basis of the desk study to determine which model would best suit the requirements of the client. The new model was to exhibit the following abilities:

- The primary task of the model is to determine the optimal flow hydrographs that are to be released from each of the control structures to meet the demands that are given for the following week's irrigation and domestic use.
- The model is to operate in a user-friendly manner.
- Perform one-dimensional hydraulic flow routing.

- Simulate both the spatial and temporal variability of the system.
- Simulate the variation of water quality with regards to salinity as a function of the models flow characteristics.
- The model is to be able to run in real-time, providing updates according to the changes in the use of the water, the changes in water quality as well as variations in the inflow from tributaries.
- The model is to provide warnings of incoming floods and provide possible solutions for the safe and efficient management of these flood events.

With these requirements, one was able to narrow down the possible options to a hydraulic, distributed, fully dynamic model. As the new model was to take over from the existing model, it was important to understand how the existing model operated. To ease the transition from the existing model to the new model, one wanted to limit the number of operational changes required for the implementation of the new model.

## 3.3 The Existing FISUN Model

The existing FISUN (Fish Sundays) model was developed in house by the Department of Water Affairs and Forestry by Cameron Tylcoat (1990).

### 3.3.1 Theory of the FISUN Model

From discussions with the regional office of the Department of Water Affairs and Forestry in the Eastern Cape, it was decided that for week to week forecasts, the variation of base flows, tributary inflows, rainfall and irrigation return flow was insignificant. This was substantiated by a statistical analysis. An in channel approach was taken in such that the channel characteristics would be modelled by the model as opposed to a more simplistic Lumped flow approach. The following characteristics of the scheme were considered:

- The physical factors affecting the flow travel time and attenuation.
- The base flow and salinity
- The combination of released flow with its salinity to the previous release with the associated changes in salinity.
- Additions and losses to the system due to external factors such as return flow and riparian vegetation.

Although a large amount of historical data was available along with local knowledge, two special releases were made and these were comprehensively monitored to provide additional data for the calibration and verification of the model.

1:50,000 Maps were used to determine the distance and gradient of the rivers and canals, while the travel time was estimated based on the Manning's equation with an n value of 0.035 from the modellers experience and supported by V. te Chow and H.H. Barnes jnr. A parabolic section was assumed for the river channel. Despite these assumptions, results within 2 to 5 % were claimed between the model and observed results for the flow rate.

The system was divided into reaches, where one reach would be from control structure to the next control structure and into sections, which were from gauging stations to the next gauging station. And this was divided further into subsections. The reaches were used to determine the release pattern, flow and volumetric back calculations for a particular week. This enabled the modeller to determine two constants A and B with the Manning's equation where A and B are used in the following equation to calculate flow times as a function of the reach length:

$$Traveltime = A \times Q^{B}$$ **Equation 3.3-1**

Where Traveltime is given in hrs/km and Q is the flow rate in $m^3/s$.

Sections were used to determine the values of other flow parameters such as, flow loss, base flow and base flow TDS as well as model parameters such as Alpha, Beta, a and b. A subsection was a length of channel between nodes spaced roughly 12km apart. This unit was based upon the system characteristics as well the desired calculation interval and flow routing theory used.

A modified Saint-Venant Equation was used for the flow routing routine. As discussed there are two equations that make up the Saint-Venant Equations, the momentum equation and the continuity equation. As the pure Saint-Venant equations were too complex to calculate at the time of this model, simplifications were researched and tested. The Momentum equation was replaced with a simpler function that modelled the complex interactions of the full equation.

$$A = Alpha \times Q^{Beta}$$ **Equation 3.3-2**

This was taken from Li et al. (1977) on his work on Kinematic wave approximations.

An implicit scheme was used and the solution was found via the Newton-Raphson iteration procedure to within a desired minimum error.  Thus the model was a simplified Saint-Venant, distributed Kinematic wave model with a spatial step (dx) of 12km and a time step (dt) of 2 hours. As such backwater effects were not taken directly into account as well as there was difficulty in model stability during large increases in flow such as during the start of a flow release or flood flow routing.

A salt mass balance was used for the calibration of the water quality routine scheme.  This was due to the unsuitability of a steady state salt mixing approach due to the dynamics in the mixing of the flow new released to the tail of the old release.  The steady state salt mixing approach could only be applied after a steady state had been reached in the model.  For this reason, the water quality component of the FISUN model was run as a separate routine after the Hydraulic Routine.

### 3.3.2 Weekly operation of the FISUN Model

The FISUN model was developed at a time when the largest portable storage disk had less than 1MB of space.  The data for the spatial characteristics of the model were accessed along with the system data files from the hard drive on the personal computer.  The initial conditions were taken from the results of the last run made by the model.  The user interacted with the model through a series of menus which gave them access to the various sub routines that made up the model. The user could update the models spatial and system data through these menus as well as input the data required for the weekly forecast.  To determine the release hydrograph for the following week, the user would select the menu option to allow them to enter in the following week's abstraction requests provided by the irrigation boards, municipalities and private irrigators that make use of the water transferred from the Orange River.  The model would be run on a Thursday and would forecast the release hydrograph from the next Monday 06h00 to the following Monday 06h00.  After inputting the abstraction requests the user would update the maximum operating flows as well as the dam operating levels and TDS.  The program would be run and would produce a theoretical release pattern for the control structures.  The operator would check and adjust the releases to a simpler release pattern then run this release pattern to determine if the final release pattern was acceptable.  The operator was able to view graphical output from the model to assist them in their decisions.

### 3.3.3 Summary Remarks

Although the FISUN model was highly effective in its role of forecasting the required hydrograph for the control structures as well as additional releases if required to maintain the modelled TDS to within specified maximum limits, the model was unable to operate as a real-time model and perform dynamic flow routing. This ability would be required to be added to the model if the model was to operate as a real-time flood forecast model as required by the client. This would have resulted in a new model being developed. It was decided to look for a commercially available model with a proven track record in both fully hydrodynamic flow modelling as well as salinity or TDS modelling.

Two packages were found to have the required ability as set out by DWAF's requirements. These were the Danish Hydraulic Institutes (DHI) Mike 11 model and Wallingford's InfoWorks RS model. The two models were evaluated and recommendations were made.

## 3.4 Wallingford's InfoWorks RS model

Wallingford's InfoWorks RS (Wallingford Software Ltd.) model combines the reliable and tested ISIS model integrated with model builders, geographical analysis and model data as a database in one model. Thus the model can be built, simulations run and results viewed from one program.

Wallingford's InfoWorks RS model is based on the ISIS hydrodynamic model which has been in operation for the past two decades. This robust model runs from a database platform which has certain advantages. All the model data files are stored in the Models Database. This database, if required, can be accessed by other users from remote locations. Changes to the database can only be made of the model is "checked out", these can then be saved separately or deleted after a simulation without changing the original model.

Wallingford's helpfulness and quick response to support queries was most apparent and one was able to set up the model in the GeoPlan window with the use of the CSV editor relatively quickly. The model is specifically designed with GIS interface and data in mind, as this is to be used in preference to a more dated fixed cross section approach as with other packages. Unfortunately as a full Digital Terrain Map (DTM) was not available, this advantage was not able to be taken.

### 3.4.1 Setup of the Model

The Tutorials are simple, if not too simple but outline the necessary steps required to set up a model via the graphic model builder. Again the emphasis here is on the availability of a DTM. The model is also built around its most popular use, that of flood forecasting and determination of high flood water levels due to proposed changes to the river course or future development. The unavailability of a detailed DTM required the cross section and network data as taken down by hand from 1:50 000 maps to be processed via the CSV data import option. This aspect is mentioned as some time was taken getting to grips with how the cross sections were to be entered into the model. The basic input window for the cross sections requires the X and Y co-ordinates for each Z (elevation) chainage point which was not available as an elevation-chainage section was used. See Figure 3.4-1 for Cross Section Property Sheet.

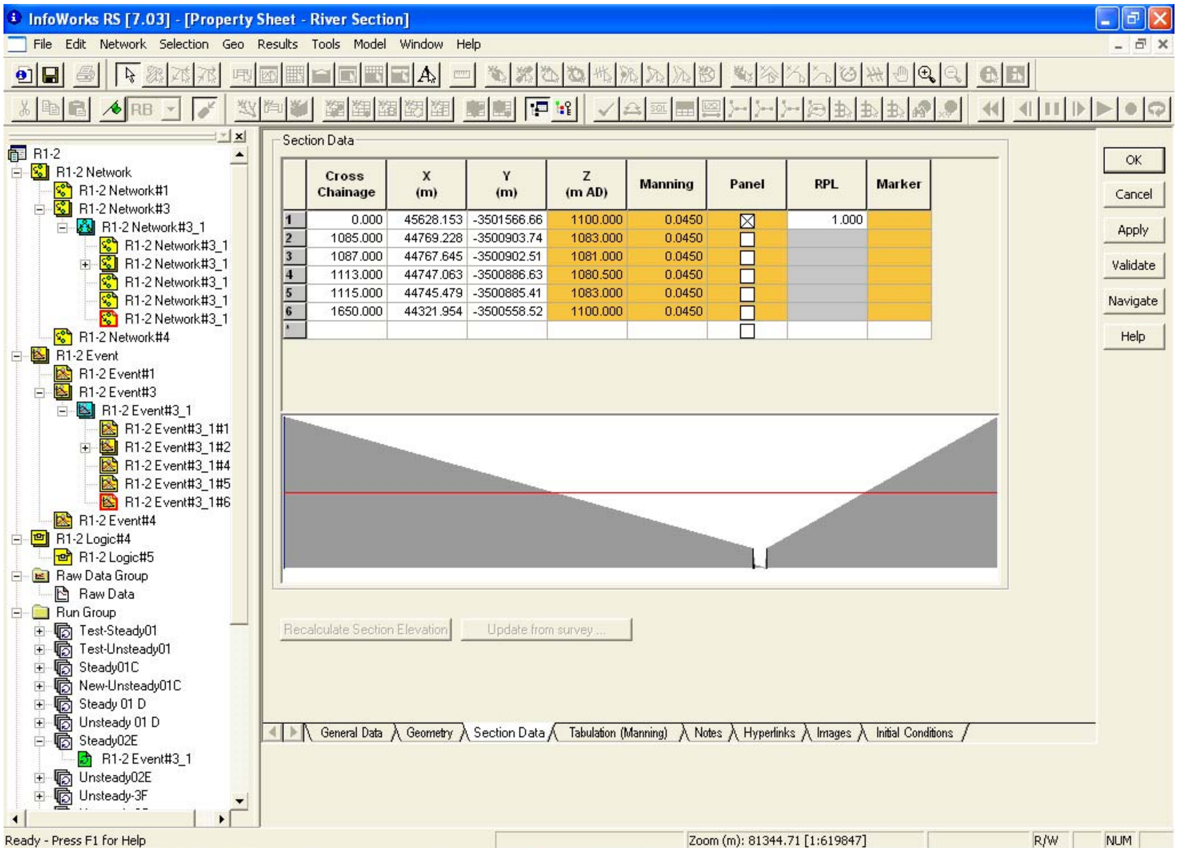



**Figure 3.4-1: Screenshot of InfoWork's Cross Section Property Sheet Showing Chainage, X, Y, Z and Manning's n Values Required for each Chainage of the Cross Section**

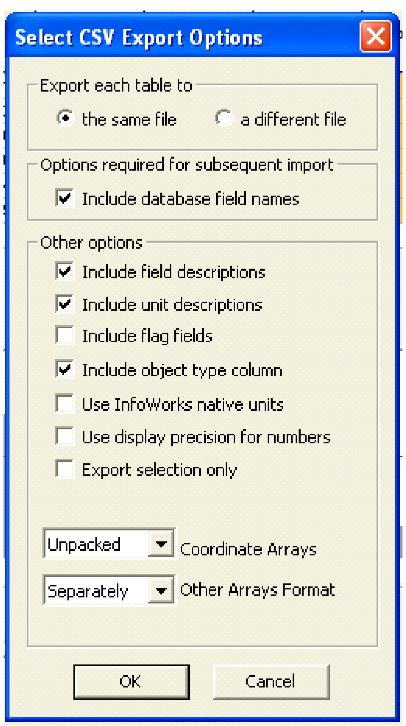



**Figure 3.4-2: Pop-up Window for CSV Export**

This was made more complex by the limitation of the model with regards to its copy paste ability with external data. In other words, copied data from and excel spreadsheet was not always able to be pasted into the cross section properties page. This aspect is currently under review and should be upgraded in the next version of InfoWorks RS. This was solved by exporting the basic model to a Comma Delimited (CSV) file (see Figure 3.4-2), editing the CSV file and then updating the model with the new CSV file. This system works well and is easy enough to manage. Flow time data is also imported into the model via this method with the Events file which holds all the data associated with the flow boundaries, and time dependant aspects of the model.

Once the cross sections from the OVIS Outlet at Teebus to Waaikraal Weir including Grassridge Dam were inputted into the model the network file needs to be added to the system. The Network file provides information on the location of the cross sections. This was a complex task to complete because as mentioned the data available was not in the format that Wallingford required. Just the co-ordinate of the lowest point of the cross section, was available for the position of the cross section. Wallingford required the co-ordinate of each point where an elevation is taken. As this data would be too complex to re-calculate or re-measure from the 1:50,000 maps, this problem was given to Wallingford who were swift to offer advice and options that could be followed. The river network points were entered into AutoCAD and this file was imported into the GeoPlan directly with the AutoCAD import tool. This data was then regarded as a river centre line. The auto reach layout tool was used to associate the simplistic setup of the cross sections to their actual location along the river reach. Once the correct series of manoeuvres was taken, this was successful in creating the model setup as shown in Figure 3.4-3.

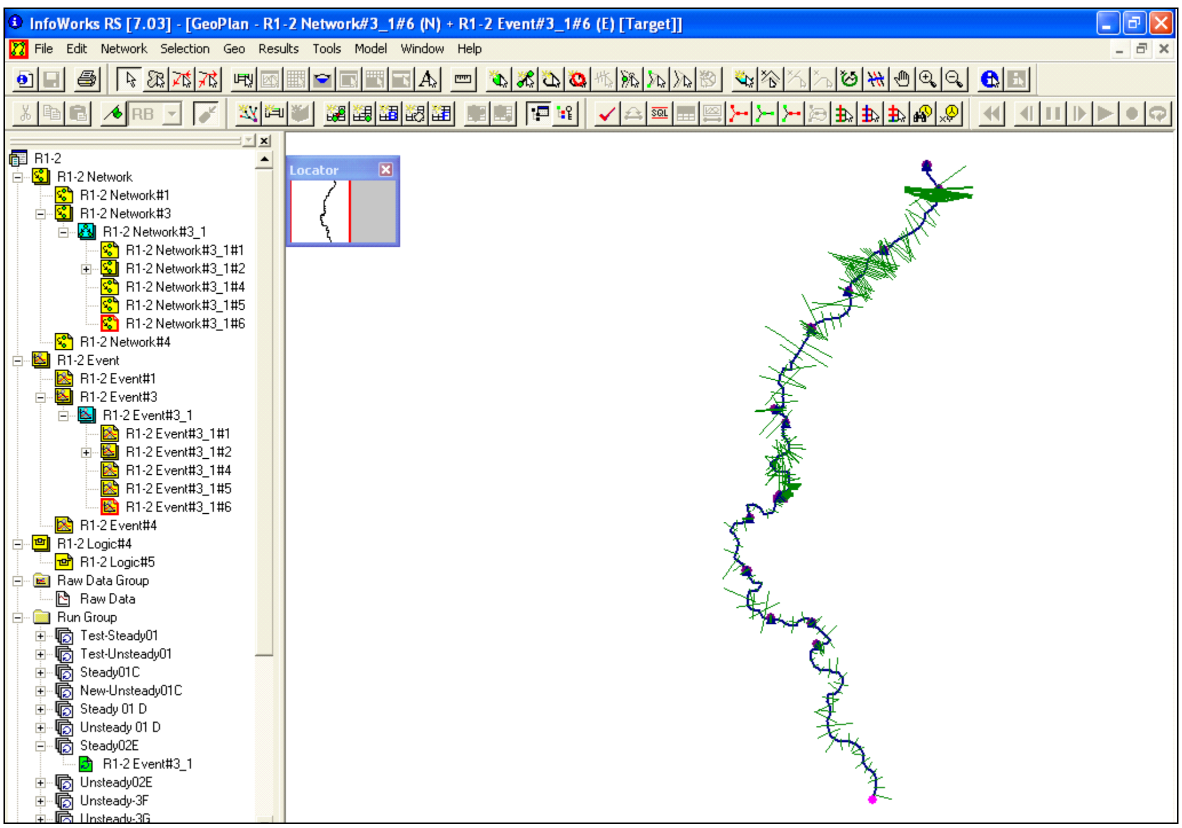



**Figure 3.4-3: Screenshot of InfoWork's GeoPlan Window showing the Network from OVIS Outlet to Waaikraal Weir**

Grassridge Dam was modelled in a very complex manner. As there was no facility to release a set flow-time hydrograph as a control structure, the dam was modelled as a weir but the release to the Groot Brak River was modelled as an abstraction from the dam and as a separate inflow at the start of the Groot Brak River below Grassridge Dam.

### 3.4.2 Running Simulations on the Model

Once the model's network information and Event data had been entered into the model, it was time to run simple stable flow through the model to test its stability. An initial constant flow of 10 $m^3/s$ was chosen but this was found to be too low and caused instabilities in the model thus a constant flow of 40 $m^3/s$ was suggested by the Wallingford support personnel. Figure 3.4-4 shows the set up page to start a steady state simulation run.

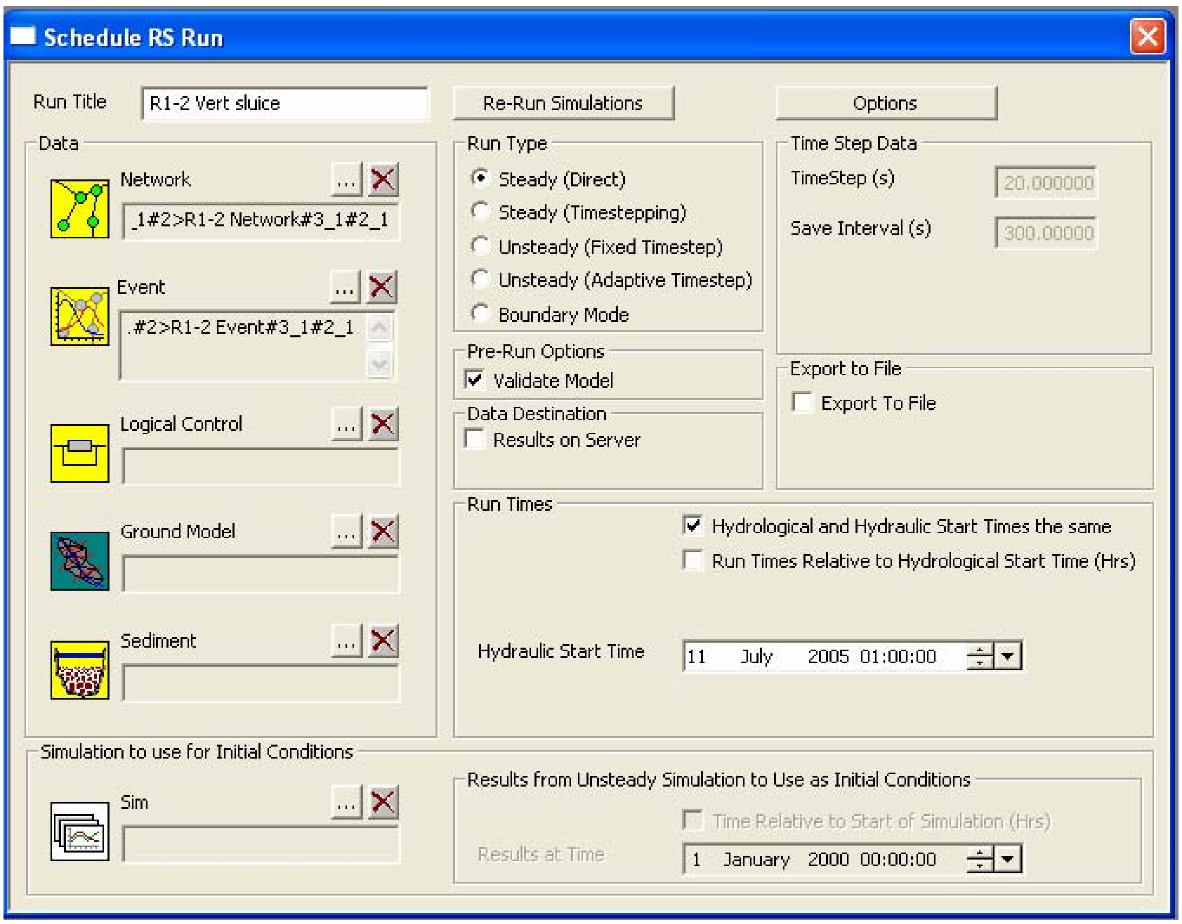



**Figure 3.4-4: InfoWork's Run Schedule Page as Setup for a Steady State Simulation Run**

After a successful steady state simulation run had been completed, the diagnostic file was read to see if greater stability can be achieved if more cross sections are added to the network. Table 3.4-1 shows a section of this diagnostic report describing the number of additional sections added by the model during the direct step method and then the recommended number of sections to be added by the modeller for the reach described.

Those nodes with a "_int###" suffix are interpolated sections that have already been added to the network by the modeller prior to this simulation. This data assists the modeller in pinpointing trouble spots and in getting the network stable in less time. In Figure 3.4-3 these interpolated sections can be seen as more closely spaced sections.

**Table 3.4-1: Data from the Diagnostic Report of a Steady State Run Simulation by InfoWorks**

| Direct method extra sections information (W2017) | | | |
|---|---|---|---|
| Reach between labels | | | |
| upstream | downstream | Sections added by the direct method | Sections recommended to be added by the user |
| Teebus_22000_D | Teebus_23000 | 3 | 0 |
| Teebus_23000 | Teebus_24000 | 1 | 0 |
| Teebus_24000 | Teebus_25000 | 3 | 0 |
| Teebus_25000 | Teebus_26000 | 3 | 0 |
| Teebus_26000 | Teebus_27000 | 3 | 0 |
| Teebus_27000 | Teebus_28000 | 3 | 0 |
| Teebus_28000 | Teebus_29000 | 3 | 0 |
| Teebus_29000 | Teebus_30000 | 7 | 1 |
| Teebus_30000 | Teebus_30000_int228 | 1 | 0 |
| Teebus_30000_int456 | Teebus_30000_int684 | 3 | 0 |
| Teebus_30000_int684 | GrootBrak_31000 | 3 | 0 |
| GrootBrak_31000 | GrootBrak_31000_int228 | 1 | 0 |
| GrootBrak_31000_int228 | GrootBrak_31000_int456 | 3 | 0 |
| GrootBrak_31000_int456 | GrootBrak_31000_int684 | 3 | 0 |
| GrootBrak_31000_int684 | GrootBrak_32000 | 3 | 0 |
| GrootBrak_32000 | GrootBrak_33000 | 7 | 1 |
| GrootBrak_33000 | GrootBrak_33000_int228 | 1 | 0 |
| GrootBrak_33000_int228 | GrootBrak_33000_int456 | 1 | 0 |
| GrootBrak_33000_int456 | GrootBrak_33000_int684 | 1 | 0 |
| GrootBrak_33000_int684 | GrootBrak_34000 | 3 | 0 |
| GrootBrak_34000 | GrootBrak_35000 | 7 | 1 |
| GrootBrak_35000 | GrootBrak_35000_int228 | 1 | 0 |
| GrootBrak_35000_int228 | GrootBrak_35000_int456 | 1 | 0 |
| GrootBrak_35000_int456 | GrootBrak_35000_int684 | 1 | 0 |
| GrootBrak_35000_int684 | GrootBrak_36000 | 3 | 0 |
| GrootBrak_36000 | GrootBrak_37000 | 1 | 0 |
| GrootBrak_37000 | GrootBrak_38000 | 1 | 0 |
| GrootBrak_38000 | GrootBrak_39000 | 3 | 0 |
| GrootBrak_39000 | GrootBrak_40000 | 3 | 0 |

Once the modeller is satisfied with the stability of the model, the steady state simulation is used as the initial conditions for an unsteady simulation. The time step is chosen along with the start and end times for the simulation run. If the model is stable the run will complete after a while and the results can be viewed and comparisons made between various simulations and observed data. Figure 3.4-5 shows the setup page for an unsteady simulation run.

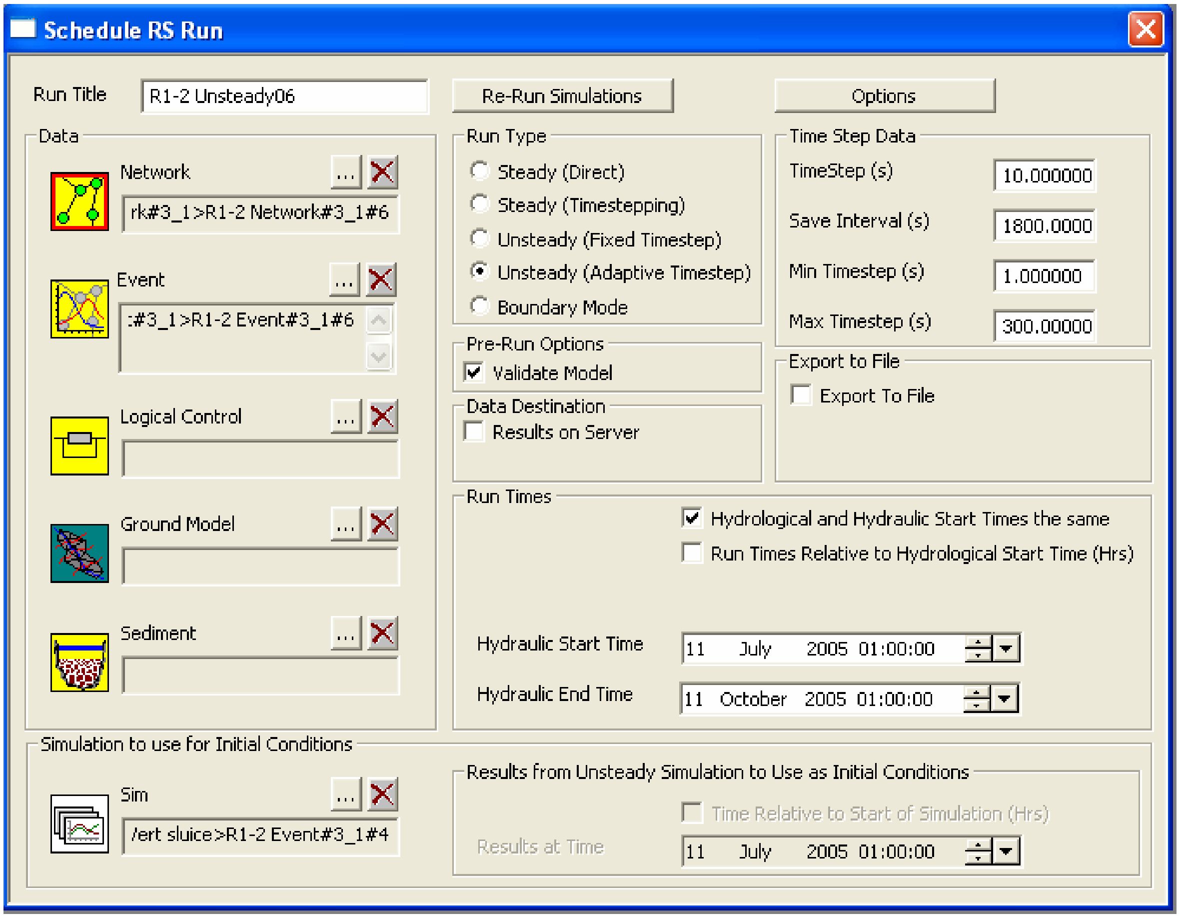



**Figure 3.4-5: Run Schedule Page for an Unsteady State Simulation Run in InfoWorks**

### 3.4.3 Viewing of Results

The results can be viewed by dragging the result file into the workspace. Once loaded, the file can be viewed as either a time series of flow or stage at a particular location or as an animation of flow or stage along a selected reach. Figures 3.4-6 and 3.4-7 show the result file for a simulation of the model with each reach set to change colour and width according to the rate of flow along that reach in Figure 3.4-6. These can be adjusted manually in the program by the operator. The flow rate is shown in each reach graphically as a colour and a line width. A table is provided on the right hand side of the screen for easy identification of the flow rate. In Figure 3.4-7 the view can be animated to show the variation of water levels over time along the reach.

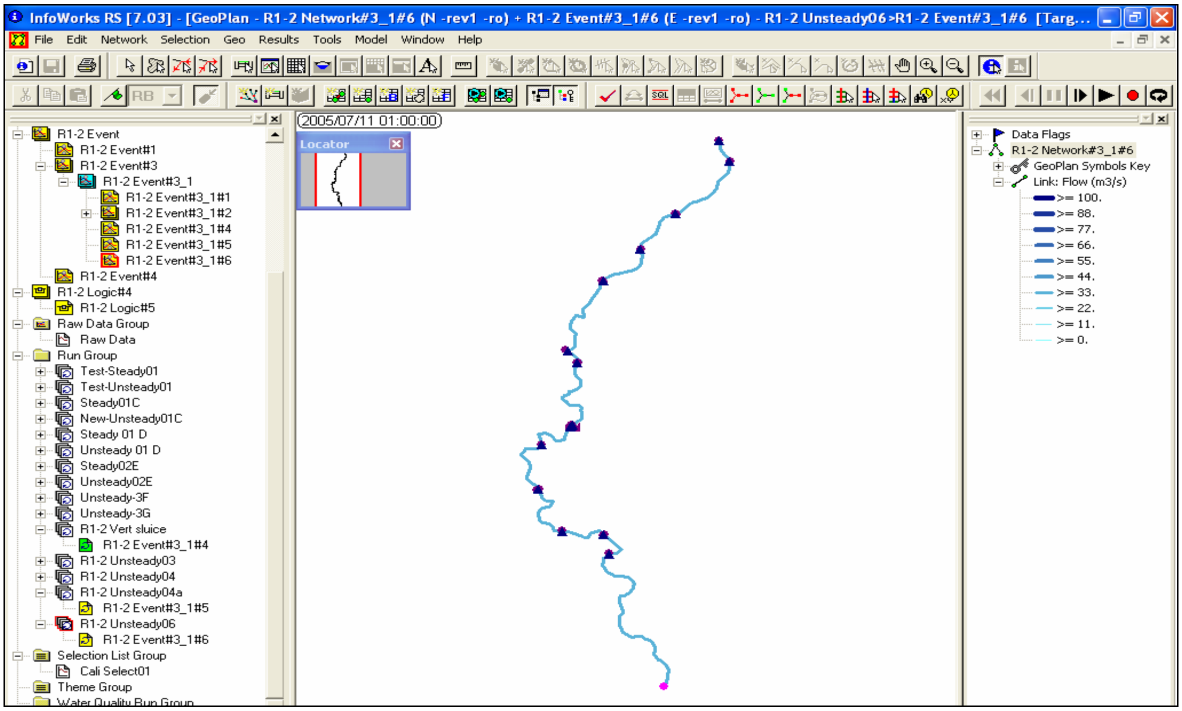



**Figure 3.4-6: Simulation Results for the Model**

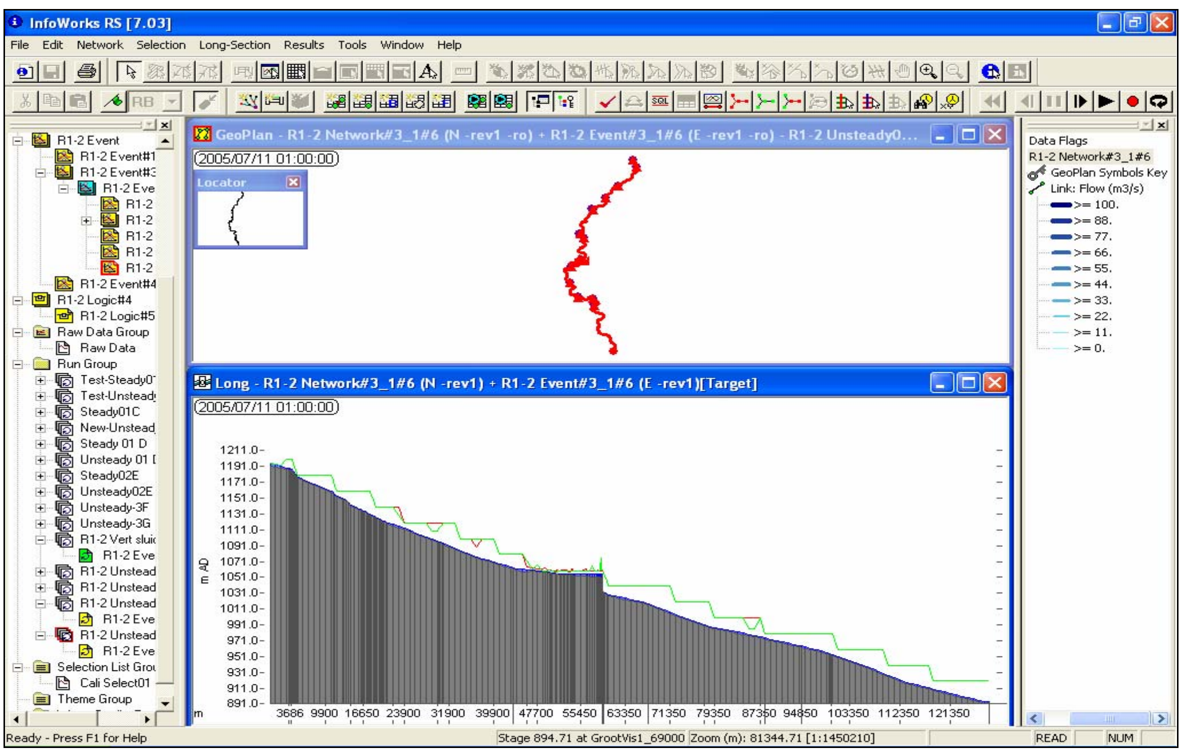



**Figure 3.4-7: Longitudinal View of the Selected Reaches with the Water Level**

The longitudinal plot can be recorded to an AVI file which can be viewed with the appropriate software. To view the attached AVI file for this simulation on the CD, please click the following link: Play Simulation 6 AVI.

Figures 3.4-8 to 3.4-11 are time series plots taken of the results of simulation 5 and simulation 6 with the observed data at the point of interest. The Manning's n value was adjusted on the reach from Grassridge to Waaikraal in Simulation 6 as there was too much attenuation of the flow at Waaikraal.

The blue line shows the results for simulation 6, the green line shows the results for simulation 5 and the yellow line shows the observed record. The simulation did not complete and as such the simulation data does not continue to the end of the graphic output. The simulation did not complete due to a rapid drop in flow rate that the model has not been fully adjusted to deal with. Unfortunately due to time constraints, the model has not been set up to its optimum operational status. At Teebus at Jan Blaauwskop no change has been made to the Manning's n value along this reach as such there is no discernable difference between the results of the simulations.

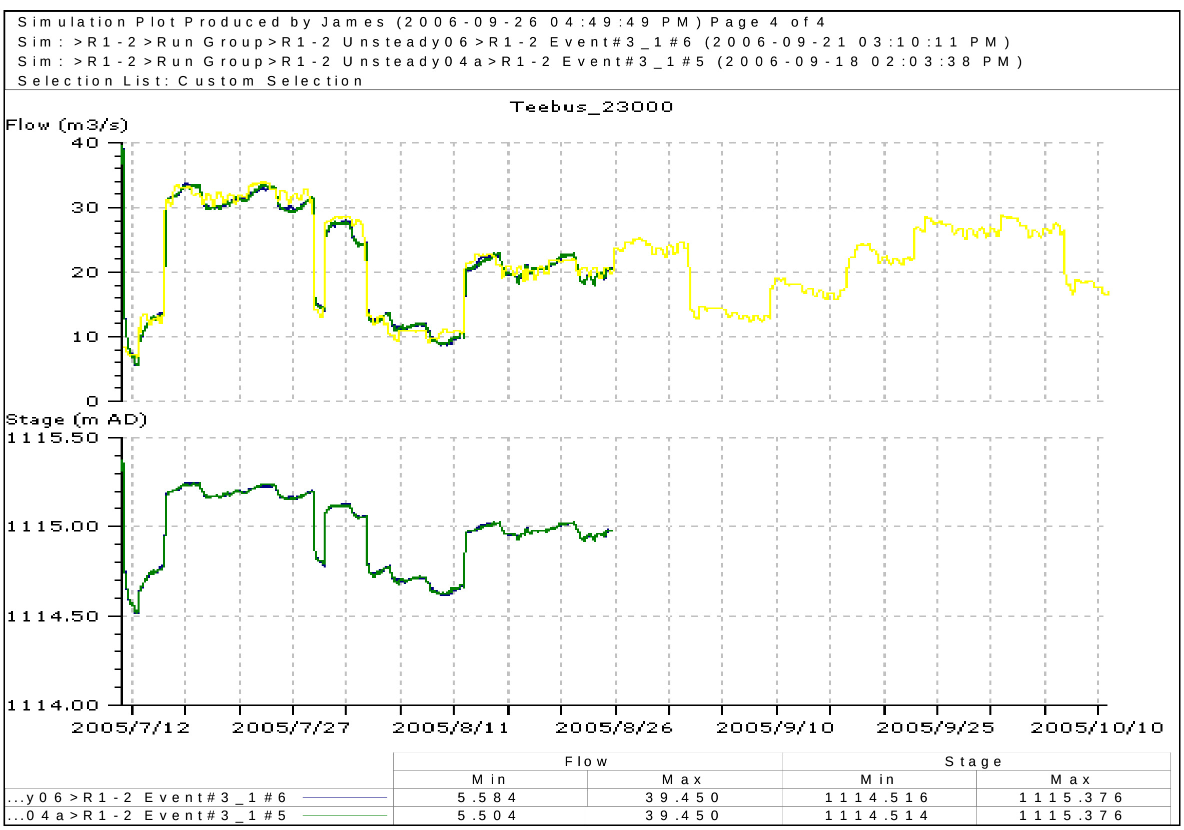



**Figure 3.4-8: Flow Rate and Stage at Jan Blaauwskop Gauging Weir on the Teebus River**

Figure 3.4-9 shows the stage and flow rate at Grassridge Dam. This is just upstream of the dam wall. This location suffered from instability due to the large change in cross sectional area with a small change in water level due to the large width of the valley.

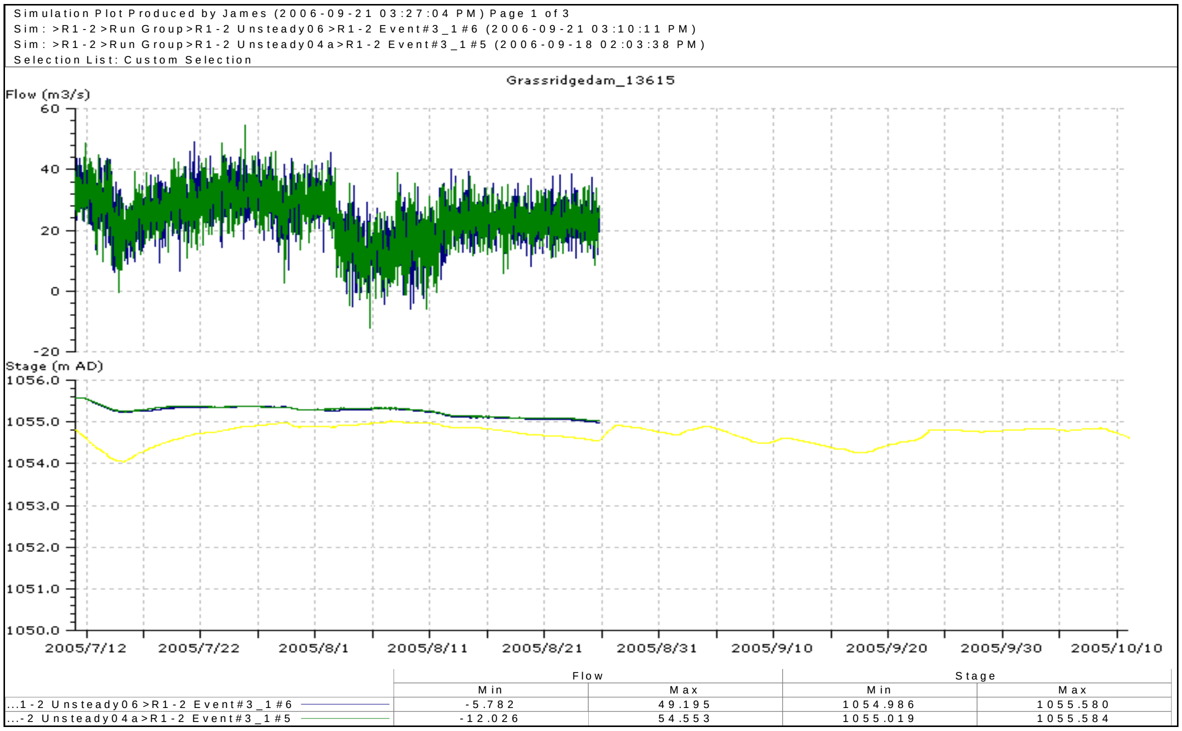



**Figure 3.4-9: Flow Rate and Stage in Grassridge Dam**

Figure 3.4-10 shows the stage and flow rate just below Grassridge Dam. The instability at flow rates below 12 $m^3/s$ is demonstrated by the change in simulation 6 (blue line) on the stage-time relationship. The cause for this instability was uncertain at the time when the simulations were run.

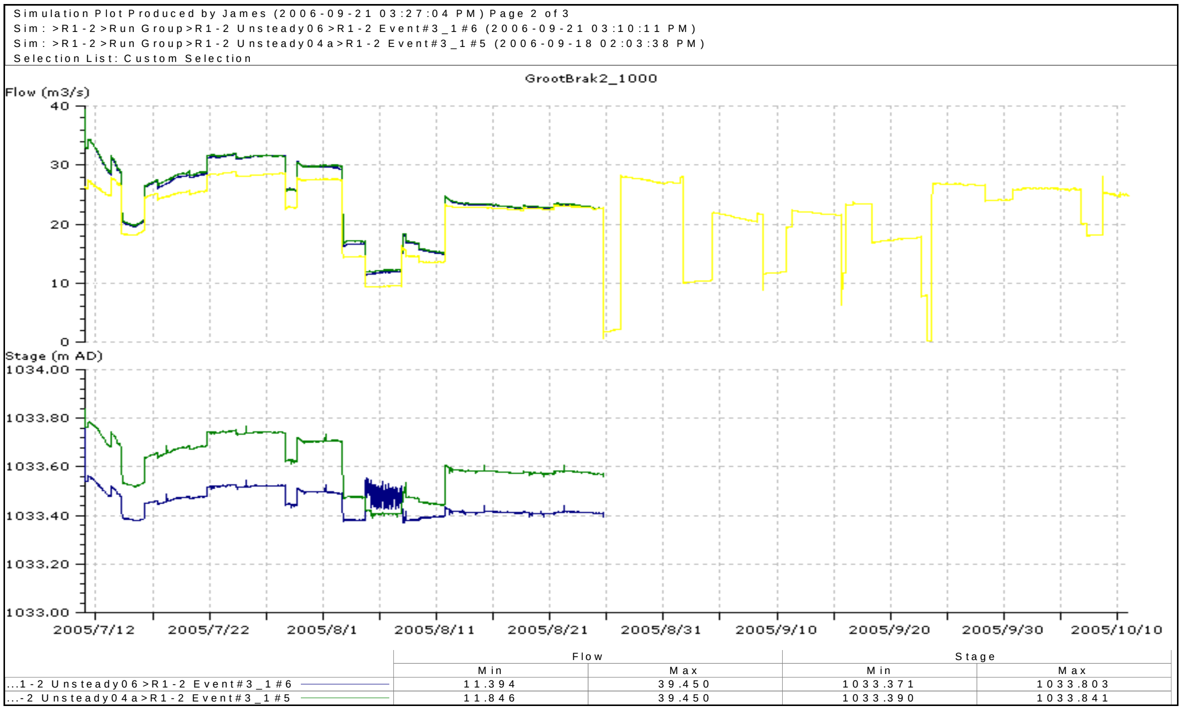



**Figure 3.4-10: Stage and Flow Rate just below Grassridge Dam**

Figure 3.4-11 shows the stage and flow rate at Waaikraal Weir. There is an interruption in the record of flow at Waaikraal at the end of July into early August 2005 due to the re-building of the weir along with routine maintenance. The variation in the Manning's n value from Simulation 5 of 0.045 to the n value for Simulation 6 of 0.03 for the reach from Grassridge down to Waaikraal is evident in the difference between the results of simulation 5 and 6. The decrease in the Manning's n value had reduced the time lag in the flow from simulation 5 to 6. Due to time constrained further simulation runs were not possible, but the model's ability to react to changes in the section data given by the modeller is demonstrated.

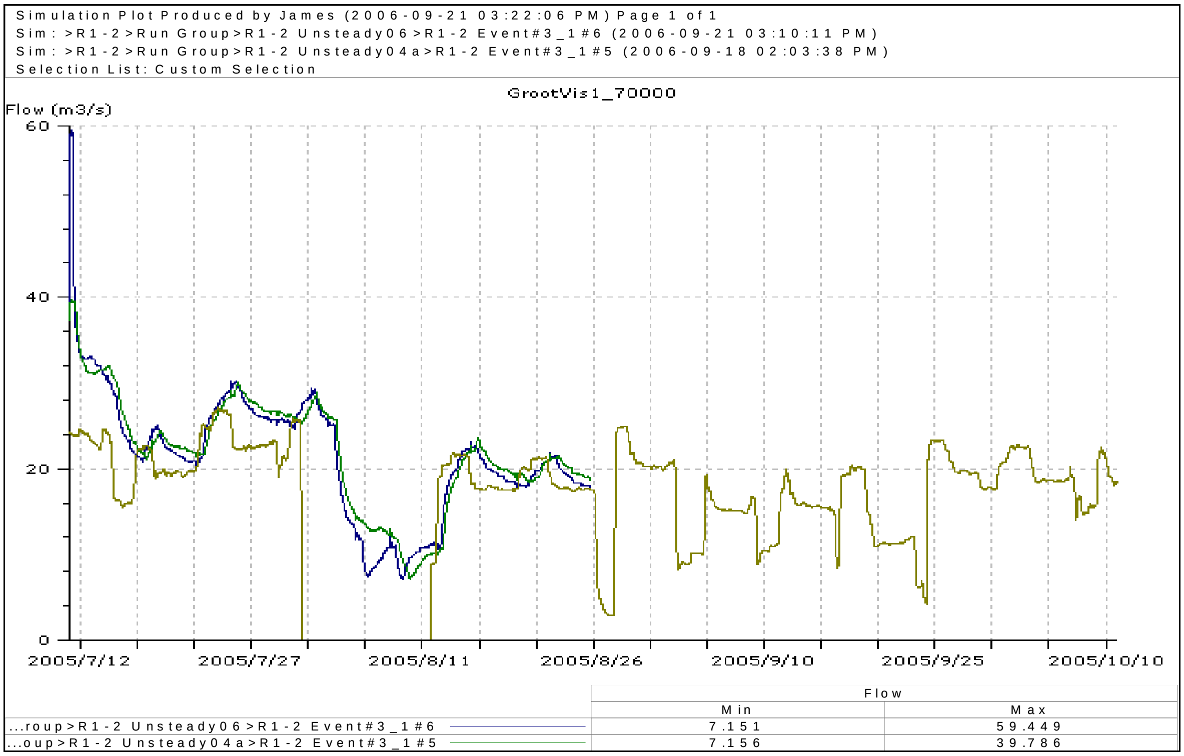



**Figure 3.4-11: Stage and Flow Rate Relationship at Waaikraal Weir**

Table 3.4-2 and Table 3.4-3 provide data on the statistical fit of the data from simulation 5 and simulation 6 respectively. The high correlation at Teebus_23000 shows that accurate calibration is achievable with this model while the lower correlation at GrootVis1_70000 shows that more simulation runs are required to improve the correlation between the simulated results and the observed values from Grassridge Dam to Waaikraal Weir at GrootVis1_70000. The increase of the Pearson Correlation Coefficient from 0.585 to 0.603 is an indication that simulation 6 is more accurate than simulation 5.

**Table 3.4-2: Statistical Results of Observed vs Simulated data from Simulation 5**

| Name of Node where Observed Data is Available | Variable | Mean Difference | Standard Deviation of Difference | Pearson Correlation Coefficient | Number of Data-points |
|---|---|---|---|---|---|
| Grassridgedam_13615 | stage (m AD) | 0.552 | 0.274 | 0.026 | 1200 |
| GrootBrak2_1000 | flow ($m^3$/s) | 2.444 | 1.568 | 0.961 | 2354 |
| GrootVis1_70000 | flow ($m^3$/s) | 3.087 | 4.118 | 0.585 | 3304 |
| Teebus_23000 | flow ($m^3$/s) | -0.398 | 1.702 | 0.978 | 2657 |

**Table 3.4-3: Statistical Results of Observed vs Simulated data from Simulation 6**

| Name of Node where Observed Data is Available | Variable | Mean Difference | Standard Deviation of Difference | Pearson Correlation Coefficient | Number of Data-points |
|---|---|---|---|---|---|
| Grassridgedam_13615 | stage (m AD) | 0.533 | 0.275 | 0.042 | 1200 |
| GrootBrak2_1000 | flow ($m^3$/s) | 2.234 | 1.570 | 0.961 | 2354 |
| GrootVis1_70000 | flow ($m^3$/s) | 2.894 | 4.618 | 0.603 | 3304 |
| Teebus_23000 | flow ($m^3$/s) | -0.366 | 1.699 | 0.978 | 2657 |

The control structure that regulates the flow in the model at Grassridge Dam was most difficult to simulate in this model. As mentioned, there is no option for a control structure that discharges flow according to a flow-time relationship. The closest one can come to this is a gated structure where the gate opening can be defined as a function of time. This would be the usual way of modelling a control structure in a normal situation but as structural data on the size and position of the sleeve valves and flood gates were not available to the model; this could not accurately be simulated in the model. The solution came with the help of abstraction nodes which can take or add water to a system according to a time series of flow vs time. Upstream of the dam an abstraction was placed that took water from the system while downstream of the dam a second abstraction was placed to release flow to the system. The fluctuations in the simulated water level follow the trend of the observed water level even though the modelled water level is somewhat greater than the observed water level. This is due to a miss alignment at the start of the simulation. As the water level fluctuations followed the trend of the observed water level, the simplified model of the control structure was considered to work satisfactorily for the requirements of this evaluation.

If this model was to be used as the model for the project, the exact size and location of each of the gates in the structure would have to be known and entered into the model. The flow would then be controlled via rules and operations within the logical control dialogue. The logical control dialogue is used to enter in logical code that controls the operation of structures on the model. For example: if the water level in the reservoir was to increase past a set limit, the program would open the gate by a set amount to increase the outflow and bring the water level back within the limits set by the modeller. Table 3.4-4 provides an example of the logical code.

**Table 3.4-4: Example of the Logical Code Used to Control the Operation of Flood Gates**

| | |
|---|---|
| HEAD(GrootBrak2_0_Dam).LT.1051.00 | MOVE=-0.001 |
| HEAD(GrootBrak2_0_Dam).GE.1051.00.AND.HEAD(GrootBrak2_0_Dam).LT.1053.00 | MOVE=-0.0005 |
| HEAD(GrootBrak2_0_Dam).GE.1053.00.AND.HEAD(GrootBrak2_0_Dam).LT.1055.80 | MOVE=0.0 |
| HEAD(GrootBrak2_0_Dam).GT.1055.80 | MOVE=0.001 |

Each line in the code is an IF statement. The first asks if the water level at node grootBrak2_0_Dam is less than 1051 m above datum (AD). If it is, the gate will be closed by 0.001 m. The second line asks if the water level is between 1051 and 1053. If the statement is true, the gate will be closed by a smaller amount as shown by "MOVE=-0.0005". And so on for the other lines of code. With this method, any attribute of the flow at any location within the model can be used to control the operation of a control structure.

## 3.5 DHI Water and Environment Model: Mike 11

The DHI package (DHI Water and Environment) consists of various modules that work together to supply a client's needs. The Mike 11 package consists of various modules or editors that work together within a workspace called Mike Zero. For the purpose of the new OFS-RT Model, DHI recommended their Mike 11, 1-D fully hydrodynamic flow routing package with the additional Advection-Dispersion (AD) module and Flood Forecast (FF) module which would all be integrated in their Flood Watch (FW) module. The Mike 11 module consists of various separate editors that allow the user to interact with different aspect of the model within a Microsoft Windows platform. Each of the editors is dealt with separately in the following sections.

### 3.5.1 The Simulation Editor

One is able to access the separate editors individually or via the Simulation editor opened from the Mike zero platform. The Simulation editor is the central component that contains the simulation and model controls parameters, is used to start the simulations and links the network editor to all the other editors required for the simulation. Figure 3.5-1 shows the Module selection page of the simulation editor. From this page the different modules required for the simulation are selected; note that the Hydrodynamic and Advection-Dispersion modules have been selected.

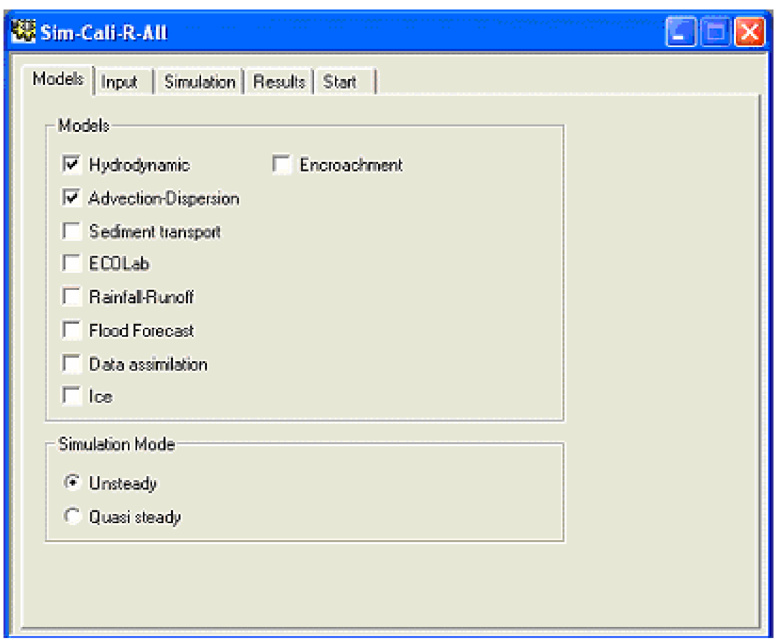



**Figure 3.5-1: Mike 11's Simulation Editor's Model Page**

Figure 3.5-2 shows the next page, which is the input of the various editors that are required for the simulation. Here the files for the Network data, Cross Section data, Boundary data, Hydraulic parameters (HD) and Advection-Dispersion (AD) parameters are loaded into the simulation editor, so that they are linked together and can be accessed as required by the model.

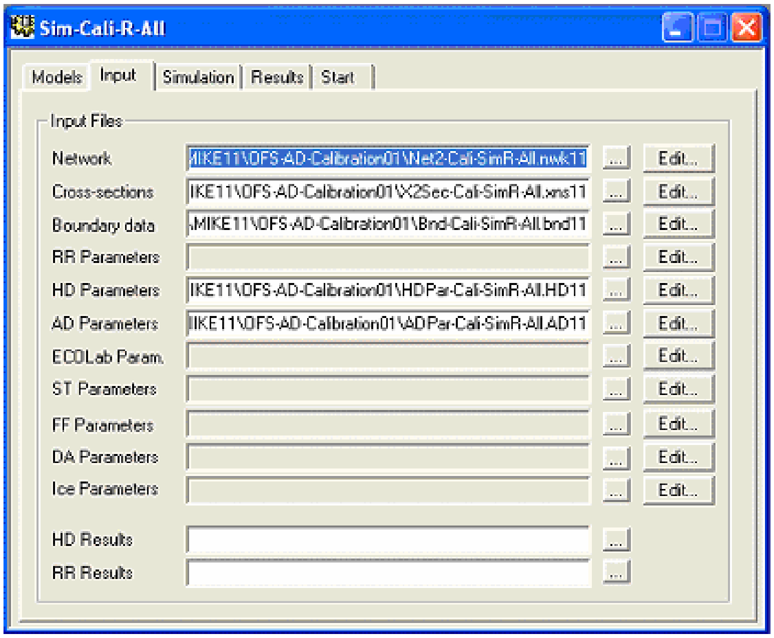



**Figure 3.5-2: Mike 11's Simulation Editor's Input Page**

The third page in the simulation editor allows the operator to define the time step for the simulation, the simulation period (start and finish times) as well as allow the operator to select which type of initial conditions to run. A Hot Start initial condition uses the flow conditions at a certain time from a previous simulation to begin the current simulation. The simulation page of the simulation editor is shown in Figure 3.5-3.

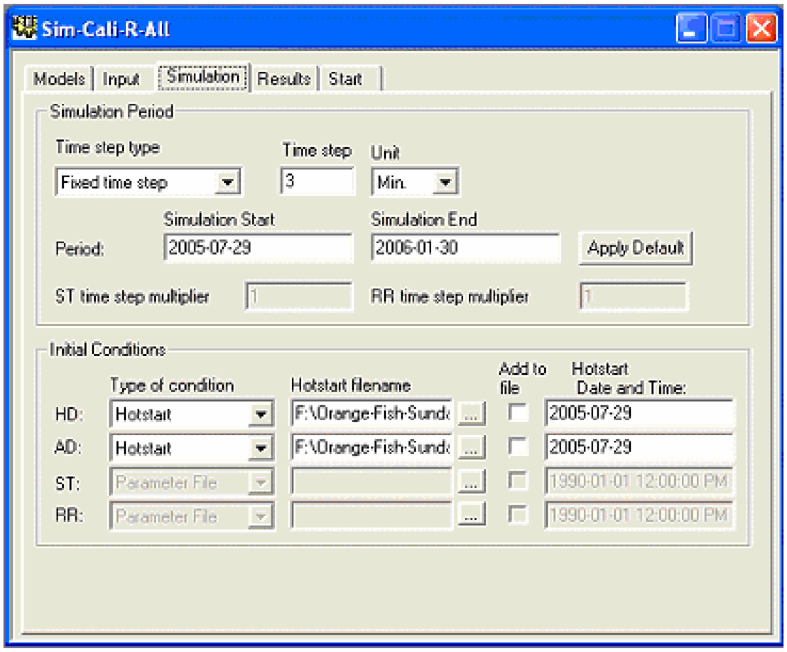



**Figure 3.5-3: Mike 11's Simulation Editor's Simulation Page**

The Results page of the simulation editor shows the location of the result file. This is where the results from the simulation will be saved as well as it allows the operator to define the time interval for the results. Even if the simulation is carried out at a 5 minute time step the results can be saved at hourly intervals. The final page of the simulation editor provides warnings if some of the data is not acceptable to start a simulation, allows for correction and if all is green for go, the simulation can be run. Figure 3.5-4 shows the start page of the simulation editor. The page shown in Figure 3.5-4 shows the status of each of the modules required for the simulation and alerts to any problems preventing a simulation from being started. As can be see in this figure, the Hot Start times are not in phase with simulation start time.

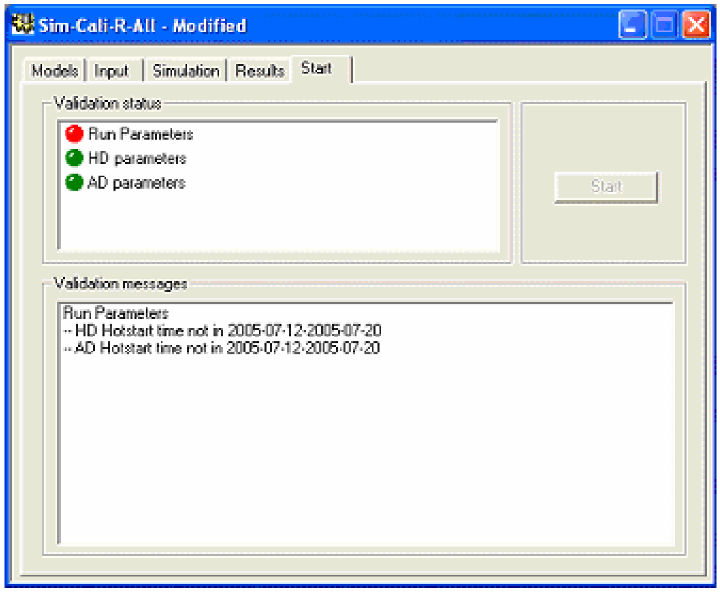



**Figure 3.5-4: Mike 11's Simulation Editor's Start Page**

Each of the editors must be created individually and saved with the required data before it can be called into the simulation editor. Figure 3.5-5 shows the selection pop-up window for creating a new editor file. For example, one would select River Network to create a new network editor file.


**Figure 3.5-5: Shows the Pop-up Window where one selects the Type of New Editor one Wishes to Create in Mike 11**

### 3.5.2 The Network Editor

The network editor contains the spatial data that makes up the physical nature of the scheme. The network editor allows for the input of data pertaining to:

- River networks and branch connections.
- Definition of hydraulic structures.
- Allows an over view of all model information in the current simulation and can access these other editors as well.

Figure 3.5-6 shows the network editor for the calibration model of the OFS-RT model. The location of various structures and parameters are shown by symbols in the graphical view.

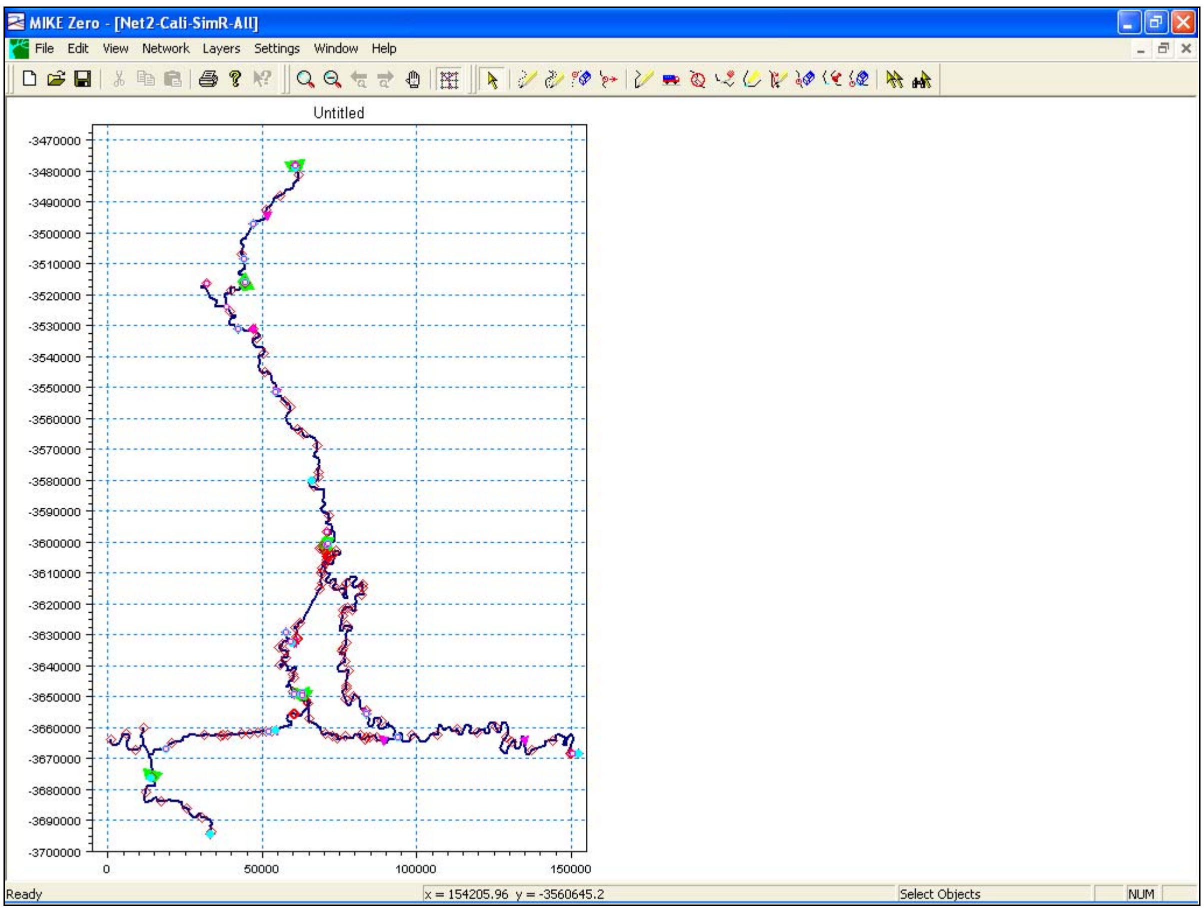



**Figure 3.5-6: Graphical View of the Mike 11 Network Editor**

The graphical view of the network editor allows for the inclusion of a DTM as well as aerial photos of the network for easier user interface with the model. The network editor can also be viewed in more detail in the tabular view shown in Figure 3.5-7. The tabular view allows for the editing of network data as well as the entering or editing of branch, structure or rainfall catchments detail as required.

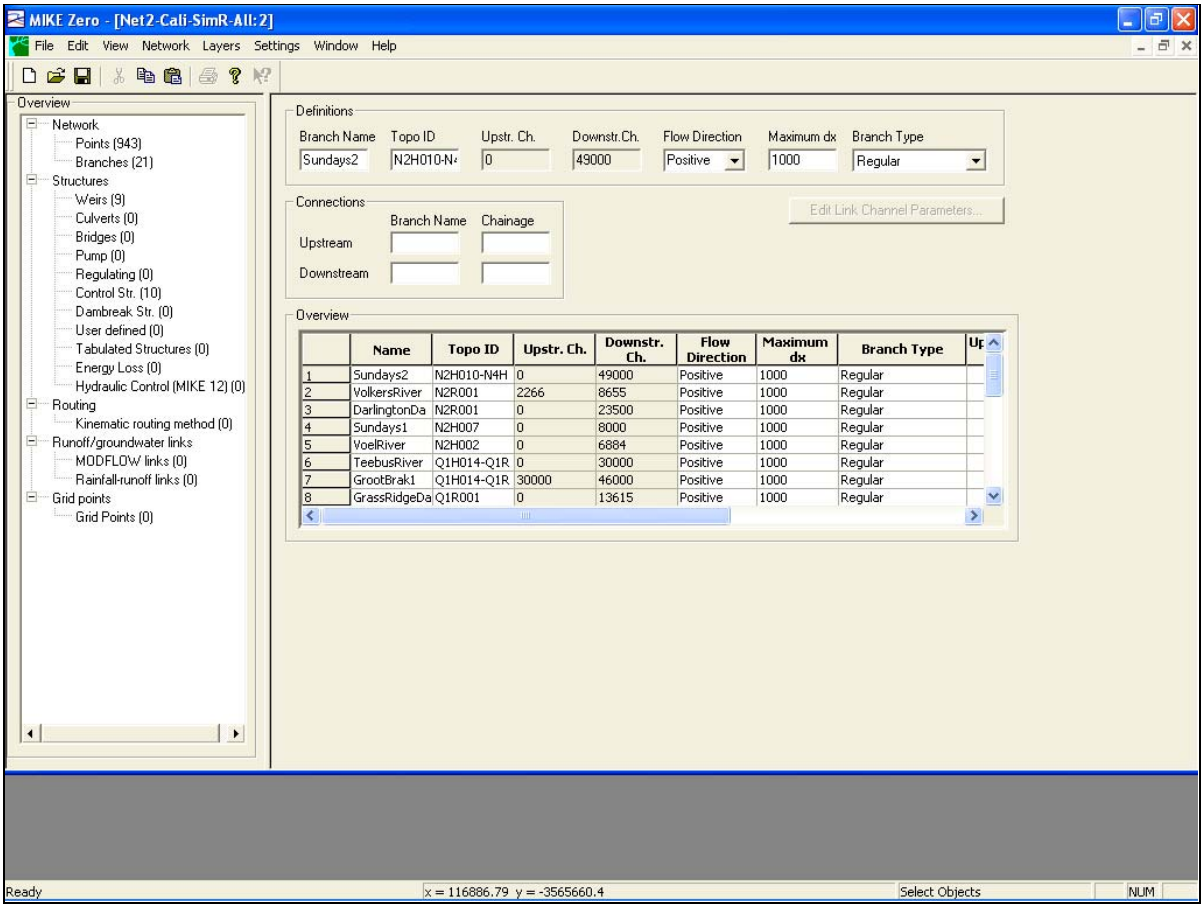



**Figure 3.5-7: Tabular View of the Mike 11 Network Editor**

The most efficient way to input network data with regards to node points and branch connections is via the import function, shown in Figure 3.5-8. This allows for a text file coded in the correct manner to easily enter in the X;Y co-ordinates as well as the branch names and chainage for the river network. Figure 3.5-9 shows some of the network data prepared for the import of the Teebus River network.

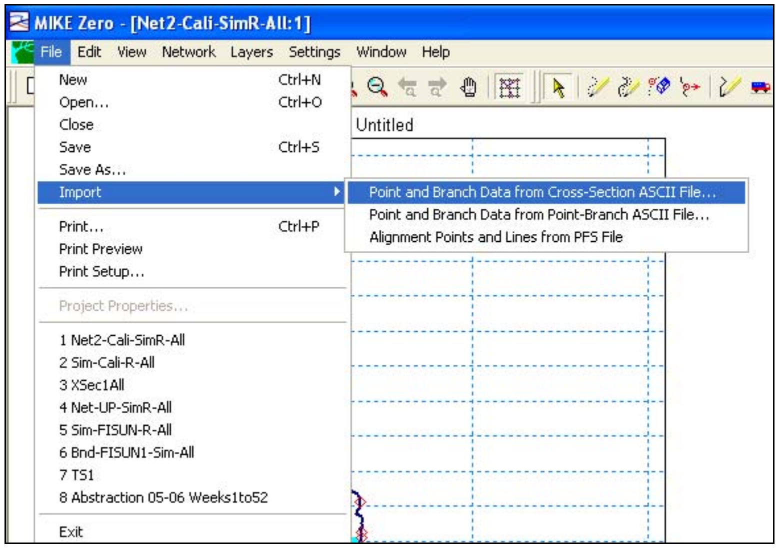



**Figure 3.5-8: Dropdown Windows to Import Data from Text Files Prepared with Network Data**

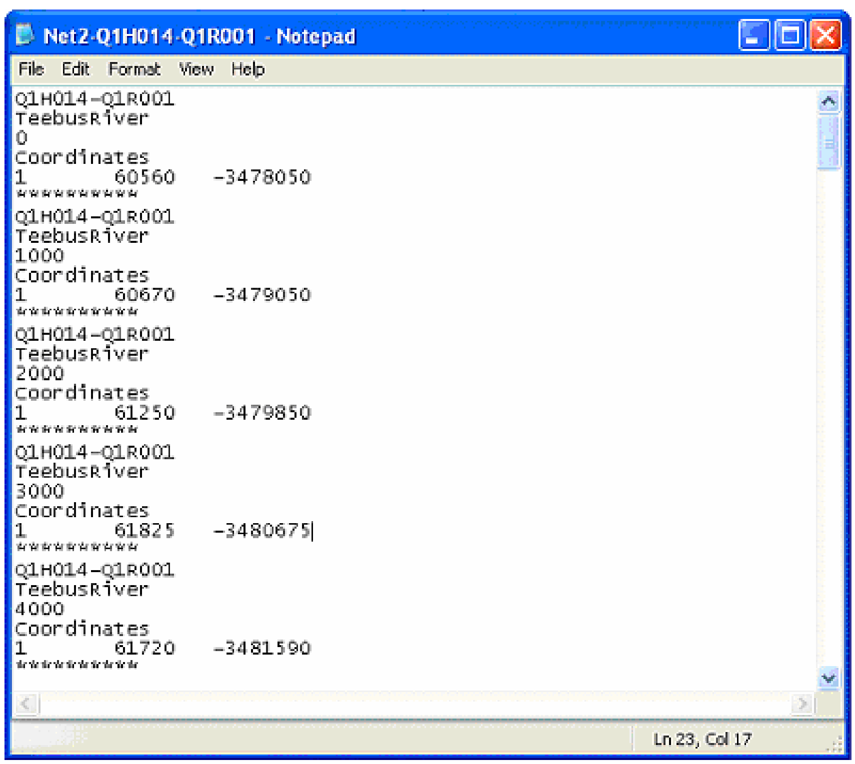



**Figure 3.5-9: Text File with the Data for the First 5 Nodes of the Teebus River for Import into Mike 11**

The first line defines the ID code for the reach, the second line defines the branch name, and the third line is the chainage for that co-ordinate point. The fifth line defines that there is one co-ordinate point and gives the co-ordinates in X then Y. The line of *'s designates the end of this string of code and begins the next nodes data.

### 3.5.3 The Cross Section Editor

Cross section data is taken from left bank to right bank while looking downstream. The lowest point of the river channel corresponds to the co-ordinates of the network node that it is associated with. Figure 3.5-10 shows a view of the cross section editor. The left hand side provides a tabular view of the data while the right hand side shows a graphical representation of the selected cross section.

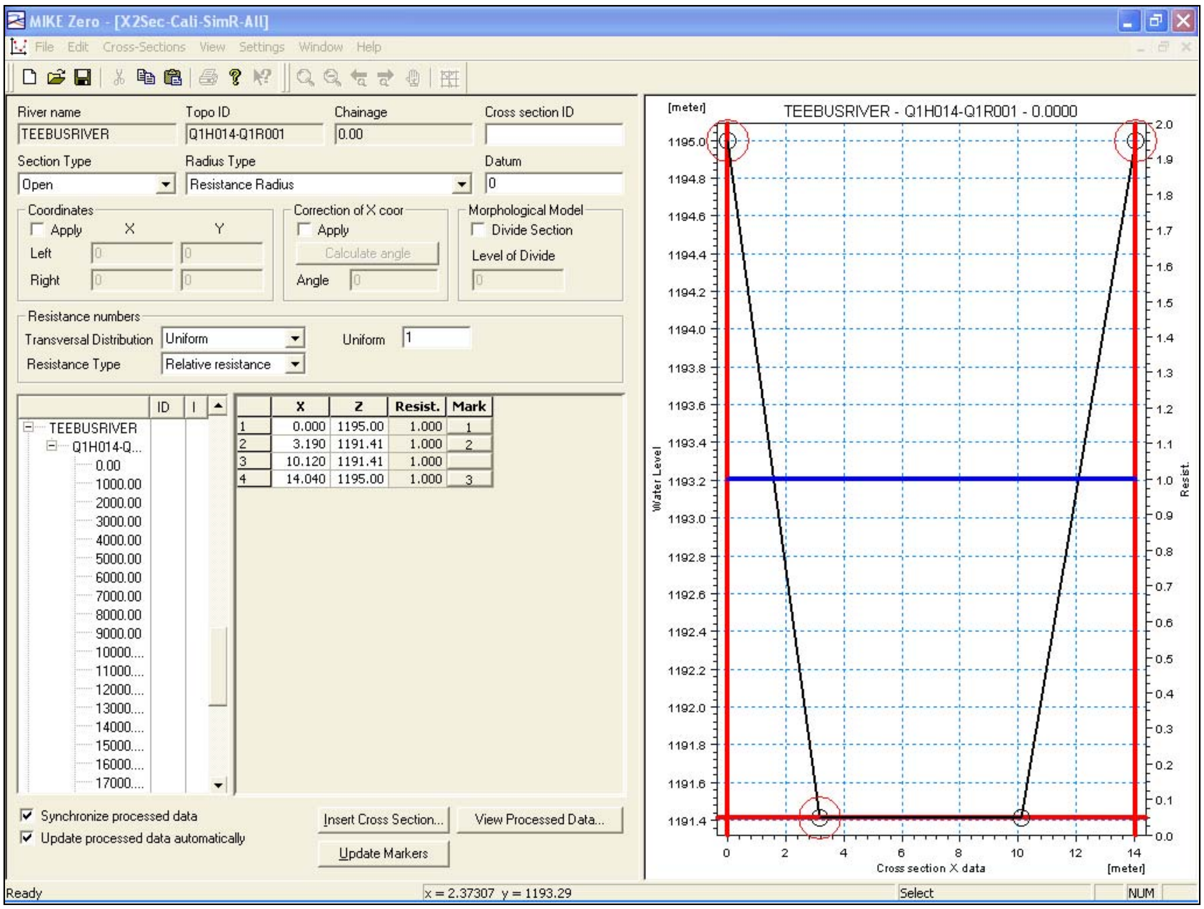
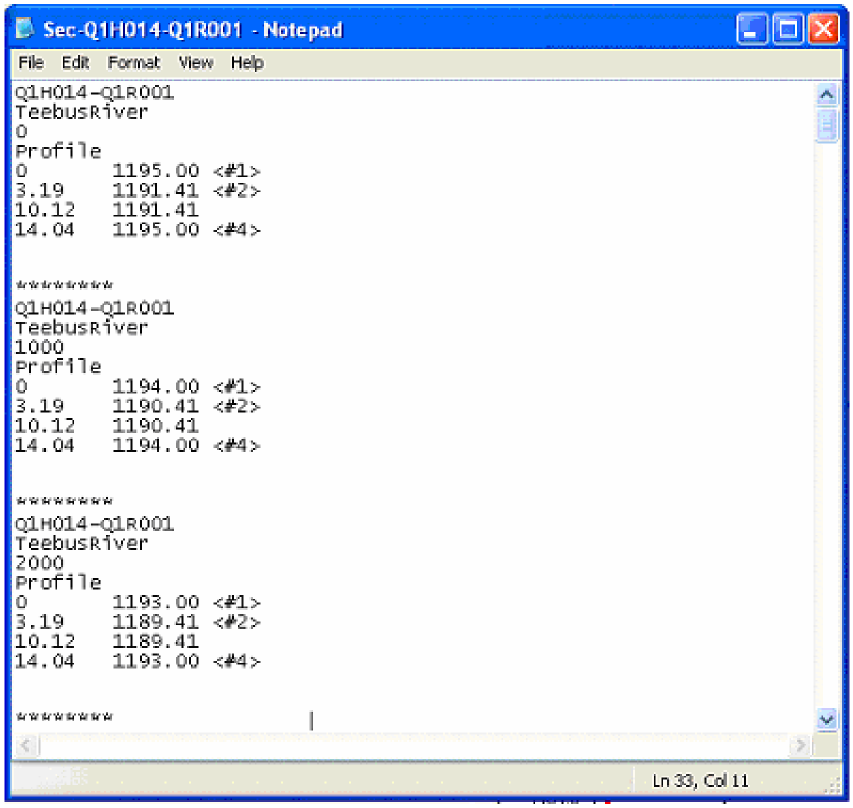



**Figure 3.5-10: A View of the Mike 11 Cross Section Editor**

The canal section at the start of the Teebus River Reach is shown in Figure 3.5-10. In this editor different characteristics of the cross section can be edited such as the type of section, hydraulic radius calculation method and roughness properties. The roughness for each cross section can be edited in three ways; uniform across the whole section; left bank, right bank and channel section; or the roughness for each chainage point across the section can be defined individually. The roughness can be defined here or as a reference to the roughness for the reach in the Hydraulic parameters editor.

The cross section data can be entered into this page or imported from a text file. Figure 3.5-11 shows an example of the text file format for the importing of cross section data.




```
Q1H014-Q1R001
TeebusRiver
0
Profile
0        1195.00 <#1>
3.19     1191.41 <#2>
10.12    1191.41
14.04    1195.00 <#4>


********
Q1H014-Q1R001
TeebusRiver
1000
Profile
0        1194.00 <#1>
3.19     1190.41 <#2>
10.12    1190.41
14.04    1194.00 <#4>


********
Q1H014-Q1R001
TeebusRiver
2000
Profile
0        1193.00 <#1>
3.19     1189.41 <#2>
10.12    1189.41
14.04    1193.00 <#4>


********
```

**Figure 3.5-11: Example of a Text File for the Importing of Cross Section Data into Mike 11**

As with the text file for the importing of network data, the first three lines define the ID of the reach, the river branch name and the chainage. The cross section must correspond to a node in the network to be recognised as corresponding to that node. The lines after the word "Profile" are the chainage and elevation of the cross section from left bank to right bank as seen from upstream. The symbols <#1>, <#2> and <#4> refers to the markers which define the left bank, channel bottom and right bank respectively.

### 3.5.4 The Boundary Conditions Editor

The boundary conditions editor is divided into three sections as can be seen in Figure 3.5-12. The top section lists all the boundaries that are in the model. The top and bottom ends of the model must have a boundary condition before a simulation can be completed. In the boundary conditions editor, one will chose the description of the boundary, whether it be an open boundary, a point source, such as an abstraction or addition to the flow, or a distributed source. The boundary type can be inflow, water level or stage-discharge relationship amongst others. The branch and chainage must be specified to define where the boundary is and an ID can be given to the boundary to identify it if there are many similar boundaries on the system.

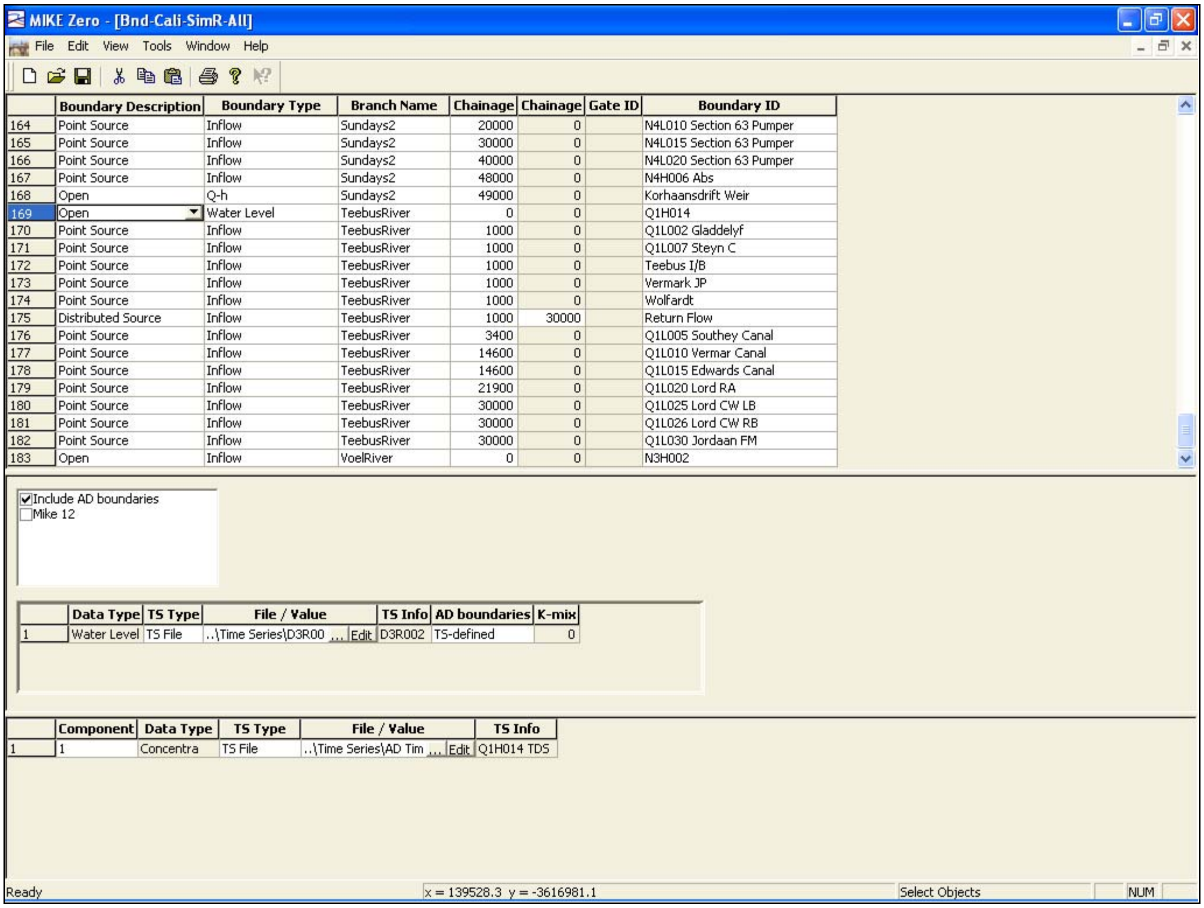



**Figure 3.5-12: View of the Mike 11's Boundary Conditions Editor**

In the middle section of the editor window, the corresponding time series file for the selected boundary is selected or a constant value assigned. In the box alongside this one can select whether the AD boundary is to be included with this hydraulic boundary. By selecting this box, the lower section is available for the operator to define the corresponding time series file for the AD module or a constant value can be assigned.

### 3.5.5 The Hydraulic Parameters Editor

The editor is used to define the roughness characteristics for each branch of the river network. The initial conditions for each branch can be entered if a hot start file is not available. Additional results can also be selected. The standard result file provides for just the water level and discharge. If one was interested in the velocity or Froude number along the reach, this can be selected from the page shown in Figure 3.5-13. There are verious pages into which different information pertaining to initial conditions or various forces acting on the hydraulic flow can be edited.

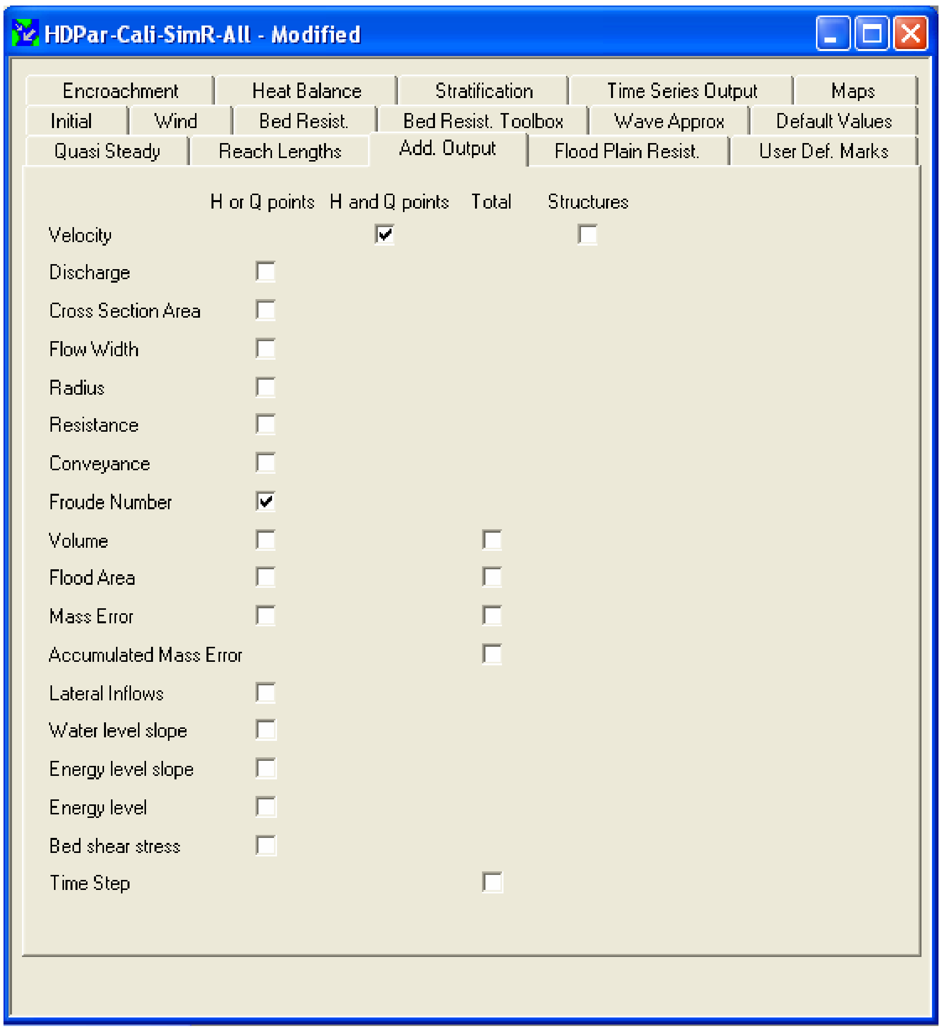



**Figure 3.5-13: Mike 11's Hydraulic Parameters Editor Showing the Additional Information Page**

### 3.5.6 The Advection-Dispersion Editor

The last of the editors required for the calibration run of the Mike11 model is the AD Editor. This Editor is used to enter in the data with regards to the type of advection or dispersion required as well as the characteristics of these as well. Figure 3.5-14 shows the AD editor page where the operator defines the dispersion factor and exponent for the dispersion equation.

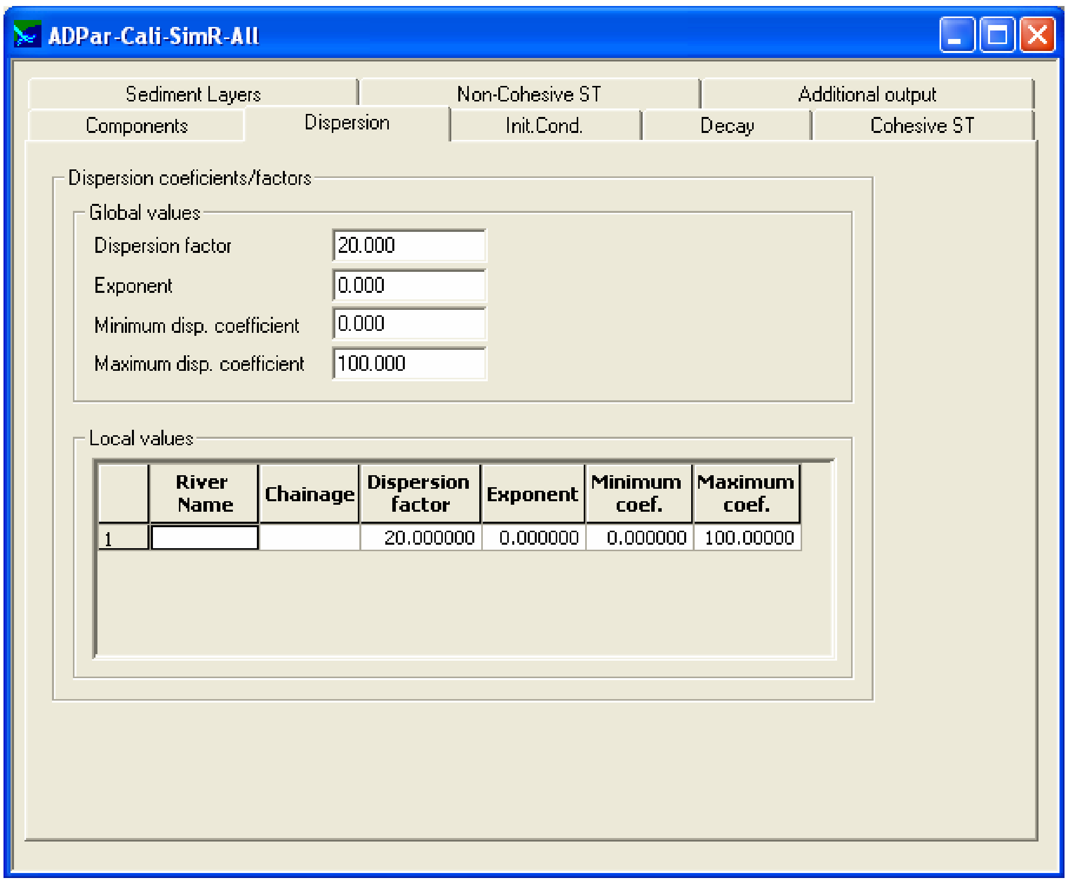



**Figure 3.5-14: Mike 11's Advection-Dispersion Editor Showing the Dispersion Page**

One is able to define the initial conditions for the AD model for each of the components. This AD editor is also able to calculate sediment transport through the river network as well as other water quality components that may require modelling.

Each of the editors discussed here are saved as separate files and can be modified as required during the calibration process. It is important to note that changes made, once saved are not reversible. It is advisable to keep track of all changes that are made to the various editors so that comparisons can be made. Additionally, changed files can be saved with different file names for later reference.

### 3.5.7 Viewing of Results

Unlike Wallingford's InfoWorks, Mike 11 is unable to view the results of the simulations directly from the Mike 11 platform. To view the results one must open MikeView, a program developed specifically by DHI to view all the result files for their different programs. Although this is useful if one is interacting between many different DHI products and wished to compare, for example, the ground water results against the rainfall runoff results, but for most applications, one would rather view various simulations and observed results for the same program. Figure 3.5-15 shows the opening Pop-up window from which the operator would select which file they wish to view. Here one would select which result file one would like to open from the browser.

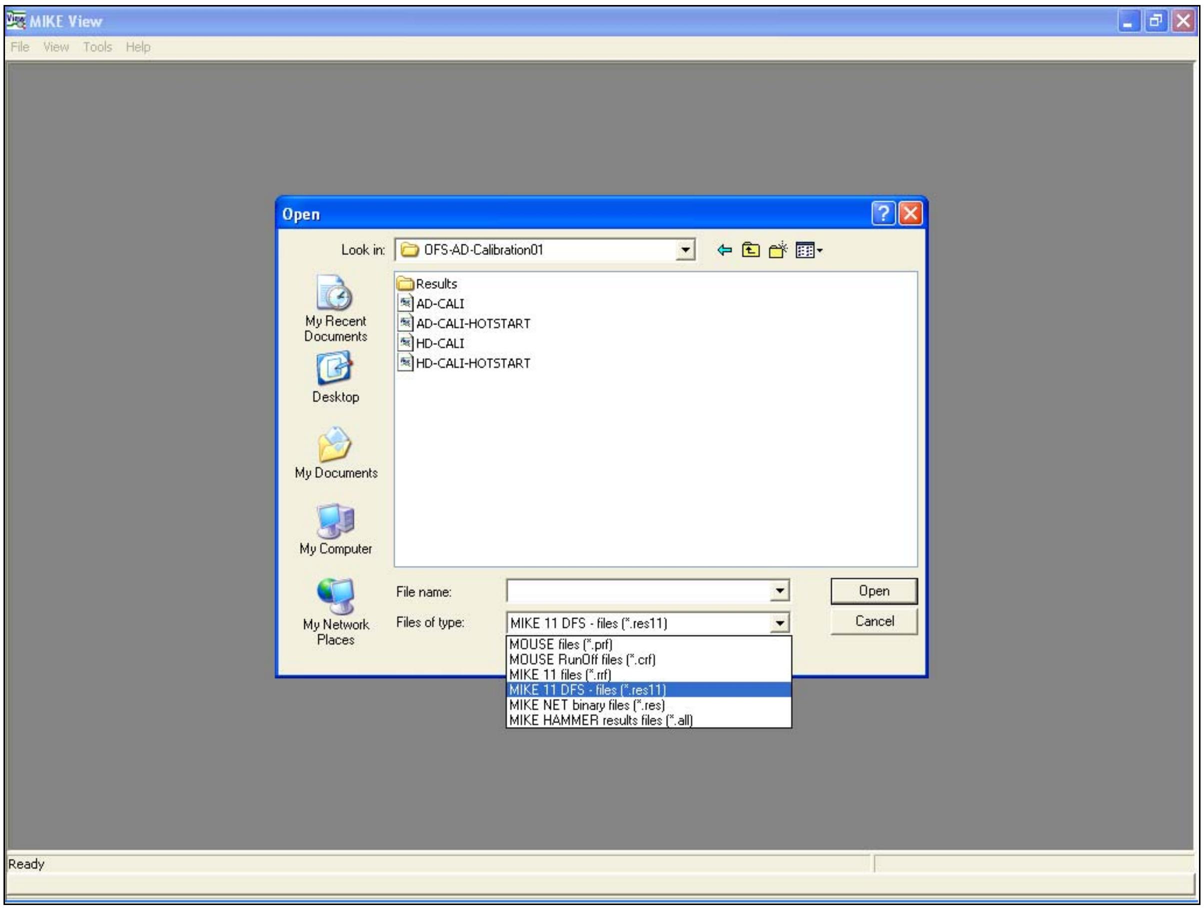



**Figure 3.5-15: The MikeView Result Viewer Showing the Selection Pop-up Window**

Once the result file has loaded, the network is shown graphically. This graphic can be set to display different properties such as water depth, discharge, flow velocity etc. Note: to view the flow velocity, this result must be selected from the additional information page in the Hydraulic Parameters editor.

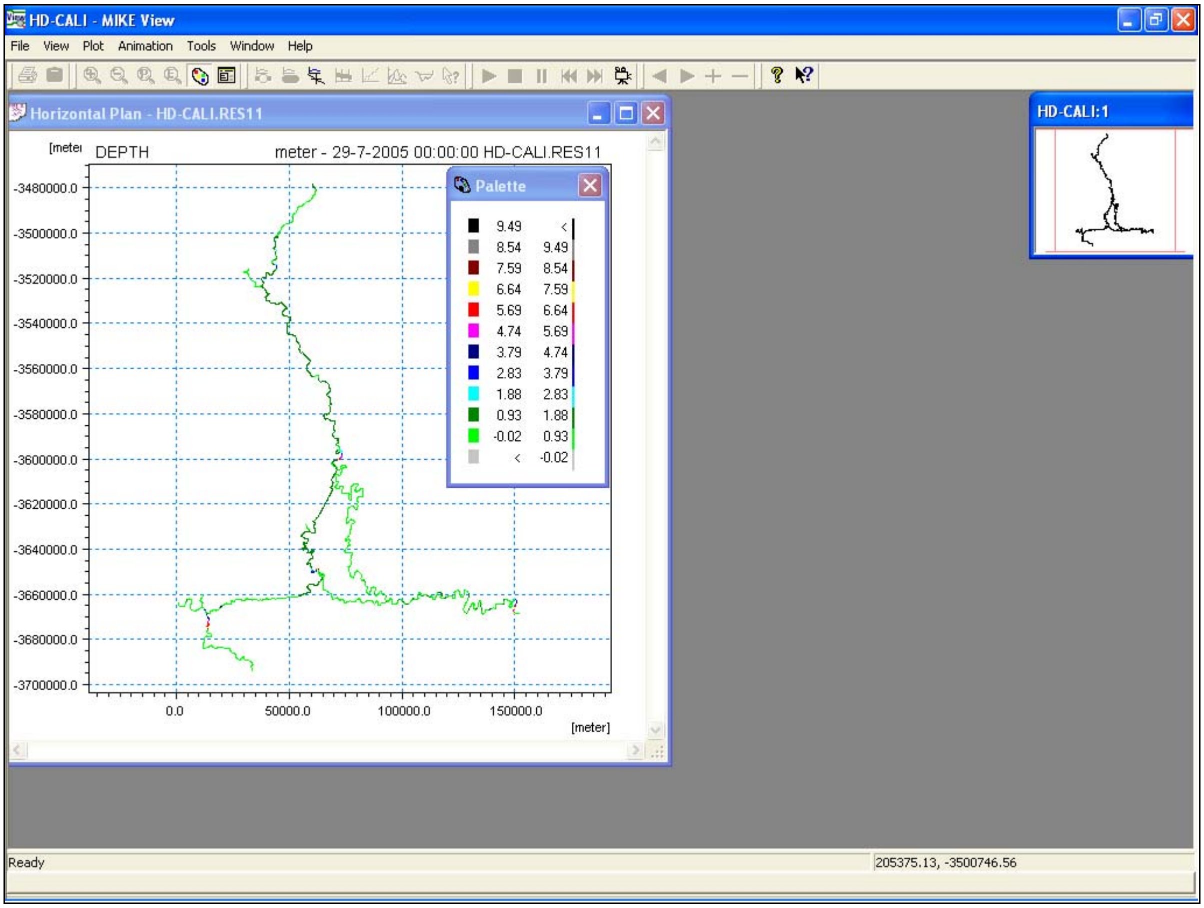



**Figure 3.5-16: MikeView with the Graphical View of the Network Displaying the Water Depths in the Network at the Start of the Simulation**

Results can be viewed in various forms. A longitudinal section can be selected and the variation of any of the result variables can be viewed as an animation. Alternatively the variation of one or more of the results at any point along the network can be viewed. Figure 3.5-17 shows the longitudinal section with the variation of discharge from Teebus OVIS Outlet to Elandsdrift Dam. The discharge is shown in blue while the ground line of the network is shown in the background. To view animation on the CD, please click here.

Figure 3.5-18 shows the simulated results for the Water level and discharge at Teebus at Jan Blaauwskop, chainage 23600 on the Teebus River. The observed discharge at this gauging weir is shown in green. This is the most useful display of the results as the simulated results can be displayed against the observed data at a location. Statistical correlations can be calculated from this data and these can be displayed in both text and graphical form.

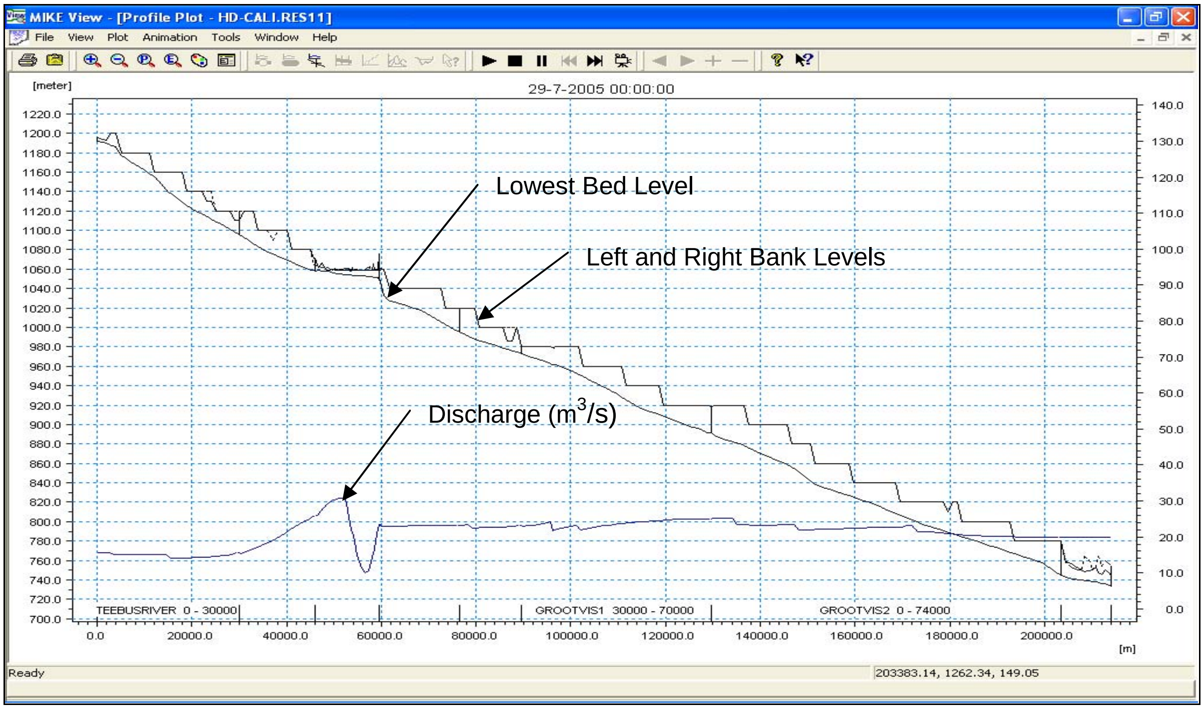



**Figure 3.5-17: MikeView Longitudinal Section from Teebus OVIS Outlet to Elandsdrift Dam Showing the Variation of Discharge along the Reach**

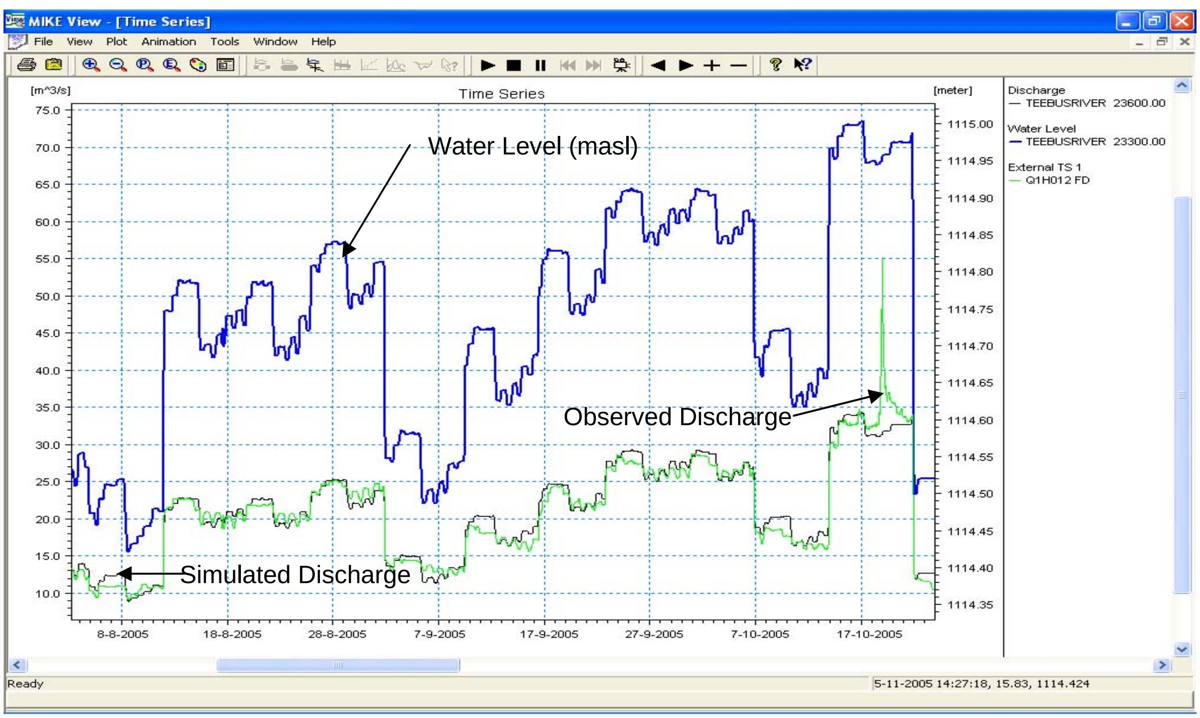



**Figure 3.5-18: Shows the Results for Simulated Discharge and Water Level against the Observed Values of Discharge at Q1H012 Teebus at Jan Blaauwskop in MikeView**

Figure 3.5-19 shows some of the statistical results that can be produced by the MikeView program. The plot on the top left hand side shows the modelled vs observed data and as is shown there is a flood event that is observed but not modelled. This is due to the inflow from the Teebus tributary, which is mostly zero, being un-gauged upstream of the confluence with the canal that transfers the flow from the OVIS tunnel to the Teebus River.

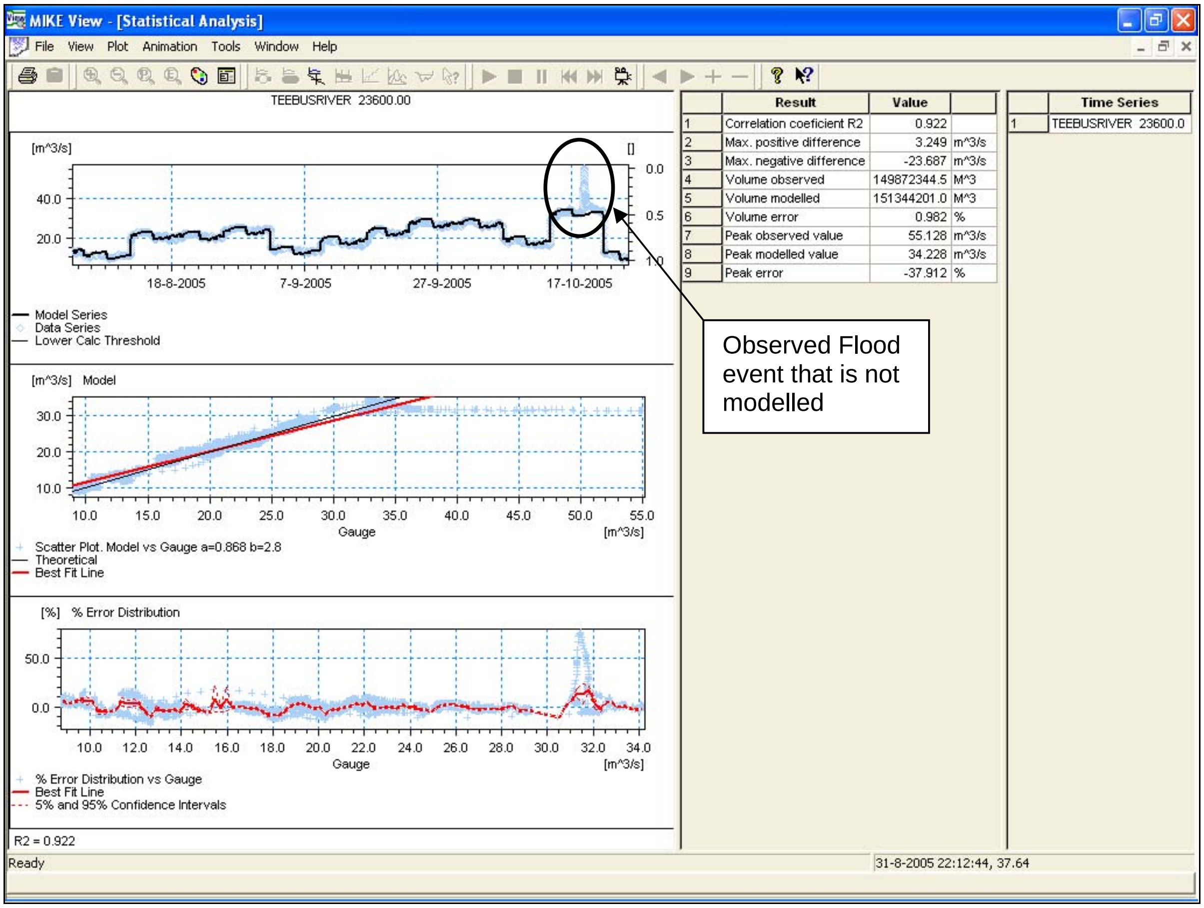



**Figure 3.5-19: MikeView's Statistical Results for Simulated vs Observed Data at Teebus Jan Blaauwskop on the Teebus River at Chainage 23600**

The statistical analysis package is most useful when trying to calibrate the network as the correlation between the simulated and observed values can be calculated for a number of time lags forward and backward from the simulated time. This allows one to more accurately determine the correct Manning's n value for the reach. Figure 3.5-20 shows the statistical result page with this option. The Lag R2 relationship on the lower left hand side is of most interest as this shows what the correlation would be if the simulated flow was lagged forward or backwards.

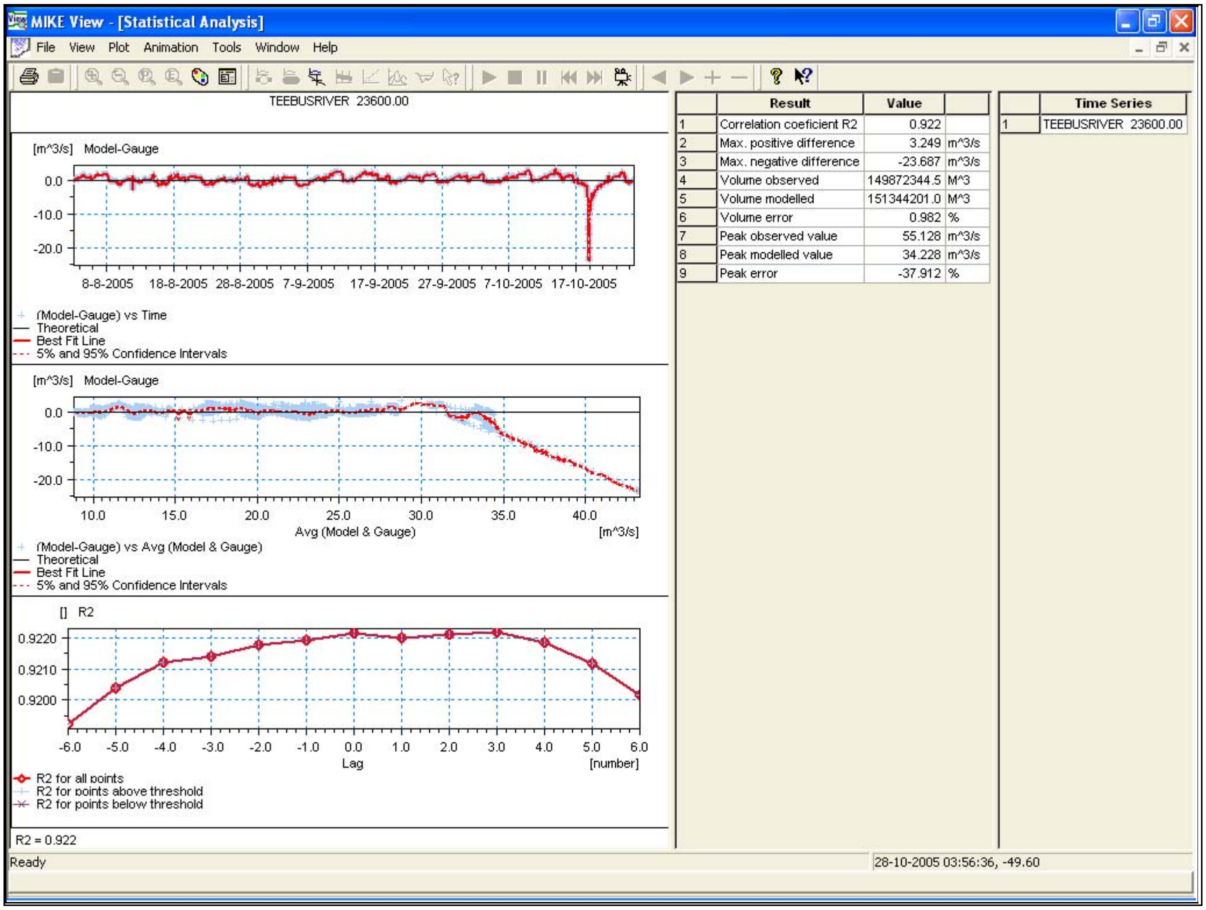



**Figure 3.5-20: Shows a Different Set of Statistical Results for the Same Set of Data as Shown in Figure 3.5.-19**

As Figure 3.5-20 shows there is a close correlation between the simulated and observed data for this run, but also if the flow was lagged forward by 1 to 3 steps in time.

## 3.6 Summary Conclusions for the Evaluation of the Models

The database format of the Wallingford InfoWorks RS model is most unique and as such, took some time to get used to as the modeller has had more experience with HEC-RAS and DHI models in the past which run on a more Windows based operation. Although this can be seen as a disadvantage, it is believed that if a technician is trained with InfoWorks first, this would be their preferred choice in the future. The method of checking out networks to be edited in the Wallingford model is most useful as this reduces the likelihood of important changes to the model being lost, though it can result in a long list of network or other files which can cluster the workspace. During the evaluation period, the most significant factor that was a drawback to the smooth operation of the InfoWorks model was that the data available to the model was collected with a DHI or HEC-RAS model in mind and as such there was difficulty in adapting the type of data to fit into the InfoWorks setup. The lack of a DTM file was also a significant drawback to the InfoWorks model which is designed to operate with this data in addition to standard cross section data.

The lack of a simple discharge-time structure in the Wallingford model was a further drawback to the basic setup that was being tested. This exclusion is surprising as its addition would be simple and popular in many instances where accurate knowledge of the gates in a control structure is not available. Naturally certain limits would have to be coded into the structure to limit the maximum discharge etc.

The InfoWorks model's processing speed was much slower than other models tested. This can be attributed to the set up of the model. More time was needed to improve the initial conditions as well as to refine the cross sections to be able to evaluate flow rates less than 10 $m^3/s$. This would allow the time step used to be increased and thus the model would simulate a selected period in much less time. The long simulation periods was the main cause for the calibration of the model at Waaikraal (GrootVis1_70000) not being completed as only one simulation was able to be run per day after basic changes had been made. For further stability changes to the model as suggested by the support team of Wallingford, much time would have been required to re-evaluate the cross section setup and the initial conditions which would have taken time that was unfortunately not available.

The support from the Wallingford personnel far exceeded expectations. Their quick and accurate response to queries shows that they have an intimate knowledge of their software and the problems that are most common. The ability to create a transportable model as a copy of the model from the database is most useful and aided in the resolution of many of the problems.

It is the view of the modeller that this software is not as suitable for the proposed use as the DHI model. It is felt that the model would better suit its original purpose of calculating flood inundation areas and high flood levels. The ease of use with regards to the DTM and the ability to super-impose a graphical view of the terrain would be most applicable to high detail, small area studies as opposed to a scheme like the OFS-RT Project which firstly does not have the digital information as well as being over too great and area.

DHI's Mike 11 package has all the functions offered by the Wallingford InfoWorks RS model. The difference between the models is that the stability and speed of calculation of Mike 11 was seen to be far superior to that of InfoWorks RS. The modeller has had more time and experience with the Mike 11 package which aided the modeller in the set up of the network. The import function of Mike 11 was simpler and more efficient that that of Wallingford's InfoWorks RS model, this lead to a better model setup which ultimately had an effect on the performance of the simulation calculations. This being said, the ability to view results from the same program that runs the simulations was seen to be an advantage that InfoWorks RS had over the Mike 11 package. The presentation of the results between the two models was virtually identical. The interaction with DTM and graphics was easier to understand with InfoWorks. Thus if a full DTM was available it is felt that the InfoWorks RS model could have had a significant advantage but as the most important aspect of the model was user friendliness and computational accuracy, the Mike 11 package produced better, more stable runs and was simpler to set up than InfoWorks.

The optimisation of the model from a series of simulations designed to optimise a set of conditions in the river system was best done with the Mike 11 model. Its Autocal module can be set up to run a number of evaluation simulations to optimise a set of conditions determined by the modeller such as the water level in the dam or the environmental flow release at the downstream boundary of the model. The number of evaluation runs requires is a function of the number of objective functions required to be optimised. This can lead to many simulation runs which may require the use of an office grid of computers running in parallel to reduce the simulation time. This facility is available with the Autocal module. The InfoWorks RS model does not have this option as an add-on but this would have to have been programmed into the model via the Logical Controls dialogue. These rules or optimisation parameters are used to determine the optimum flow release pattern for each of the control structures to operate the system as required.

These optimisation routines are too complex to be set up for just an evaluation and as such were not included during the evaluation process. The final decision on which model was best equipped to deal with the requirements as set out by DWAF was based on the evaluation of the calibration results, model stability and computational speed. The optimisation routine was also considered though this was based on a desk study. For these reasons, DHI's Mike 11 model was selected for the project. This selection was supported by DWAF as they had already been using previous models from DHI.

# 4 Setup of the Hydrodynamic Model

## 4.1 Data used for the OFS-RT Model

One of the factors in the decision to use DHI's Mike 11 as opposed to Wallingford's InfoWorks was due to the unavailability of suitable digital terrain maps and models of the catchments and notably rivers and floodplains. Many hours were spent searching web sites and chasing leads for high resolution, high accuracy satellite or aerial images that had been digitised and normalised to a co-ordinate system. The Department of Land Affairs is in the process of digitising the 1:10,000 Orthophoto's for those parts of South Africa which have previously not been done and unfortunately the Sundays and Fish River catchments are in these regions. It is the misfortune of this thesis that at the time of this study, none of these had been completed. As such the only accurate spatial data available to the modelling team at a reasonable cost and time frame was 1:50,000 maps with contours at 20 m intervals. This data was only useful in providing an indication of the valley shape, the reach length and gradient and to a lesser degree the normal width of the river. Figure 4.1-1 gives an example of the 1:50,000 maps that were used to collect data about the nature of the river network.

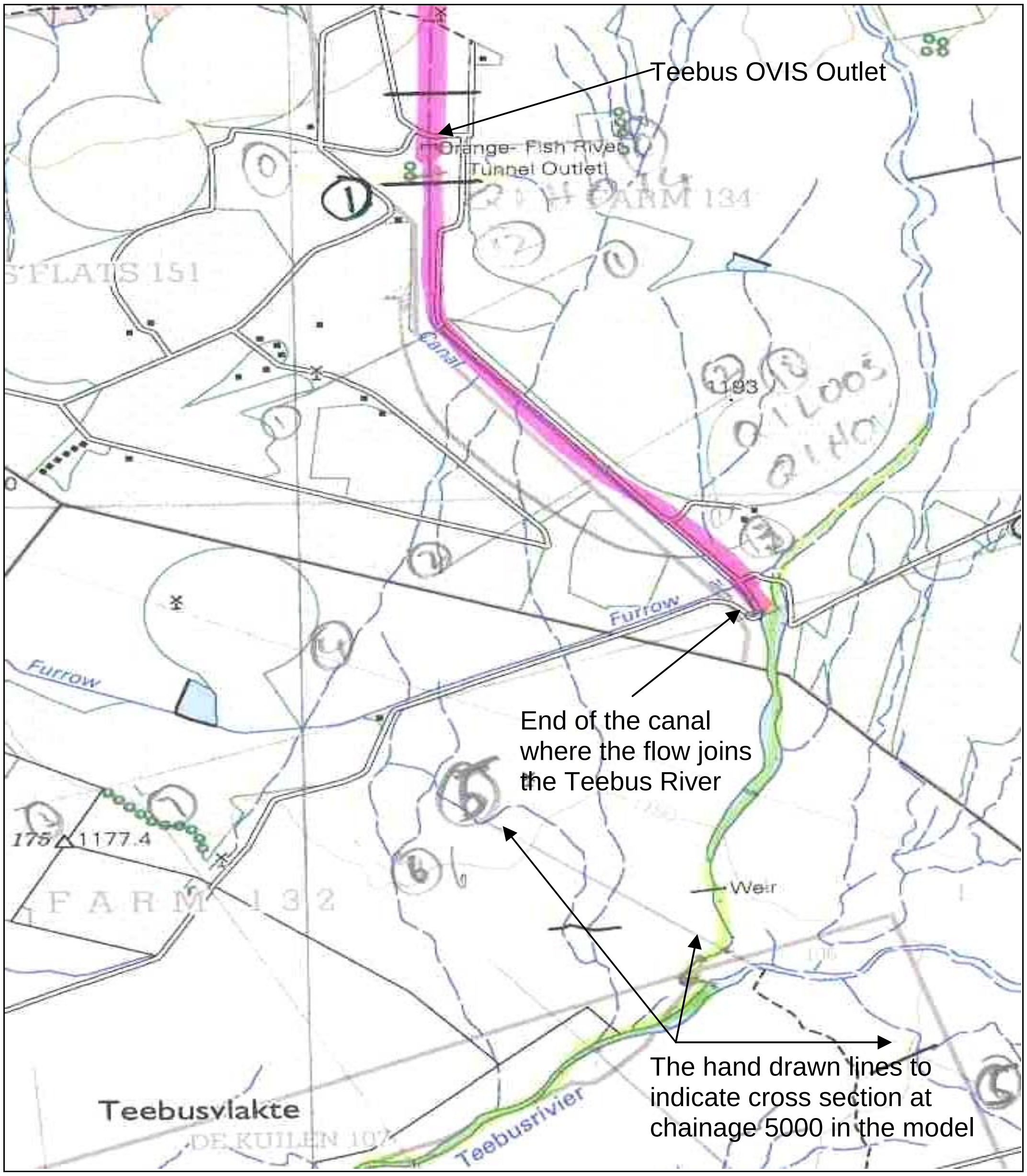
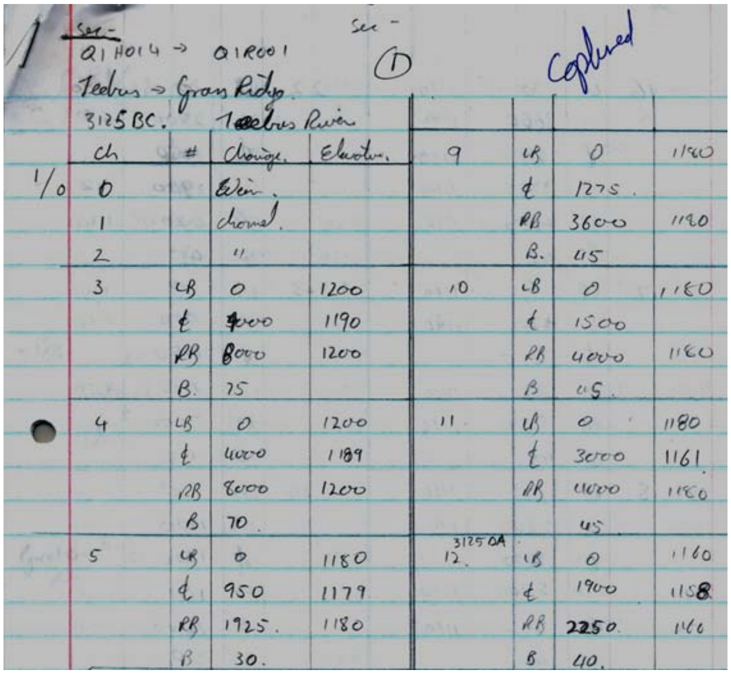



**Figure 4.1-1: Shows a Portion of the 1:50,000 Map Used to Create the Network Data for the Teebus River Branch**

The data was taken down by hand, Figures 4.1-2 and 4.1-3 show the hand written sheets with the contour chainage and values along with the co-ordinates for each chainage along the river reach. Three to five cross section points were taken, these were the left bank, the next contour down (if taken) the centre of the river, the next contour up (if taken) and the right bank. The co-ordinate of the point where the cross section line intersected the river was taken down as the position of the cross section on the river at that chainage.


Sec- Q1 H014 → Q1R001 Sec - (1) Captured

Teebus → Grass Ridge.

3125BC. Teebus River

| ch | # | Chainage | Elevation | | | | |
|---|---|---|---|---|---|---|---|
| 1/0 0 | | Main. | | 9 | LB | 0 | 1180 |
| 1 | | channel. | | | ℄ | 1275. | |
| 2 | | " | | | RB | 3600 | 1180 |
| | | | | | B. | 45 | |
| 3 | LB | 0 | 1200 | 10 | LB | 0 | 1180 |
| | ℄ | 4000 | 1190 | | ℄ | 1500 | |
| | RB | 8000 | 1200 | | RB | 4000 | 1180 |
| | B. | 75 | | | B | 45. | |
| 4 | LB | 0 | 1200 | 11 | LB | 0 | 1180 |
| | ℄ | 4000 | 1189 | | ℄ | 3000 | 1161 |
| | RB | 8000 | 1200 | | RB | 4000 | 1180 |
| | B | 70. | | | | 45 | |
| 5 | LB | 0 | 1180 | 3125DA 12. | LB | 0 | 1160 |
| | ℄ | 950 | 1179 | | ℄ | 1900 | 1158 |
| | RB | 1925. | 1180 | | RB | 2250. | 1160 |
| | B | 30. | | | B | 40. | |

**Figure 4.1-2: Hand Written Notes for Some of the Chainage and Contour Elevation of the Cross Sections for the Teebus River Reach**

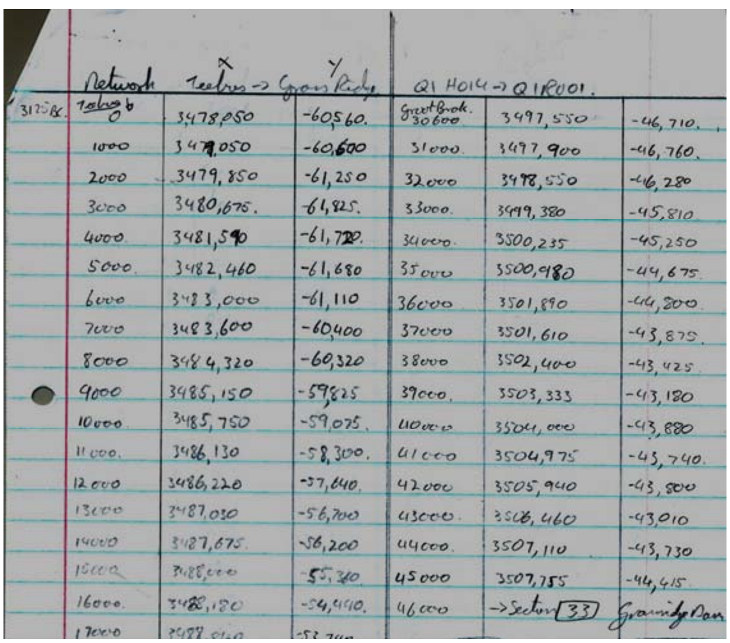

| Network Teebus → Grass Ridge | X | Y | Q1 H014 → Q1R001 | | |
|---|---|---|---|---|---|
| 3125BC Teebus 0 | 3478,050 | -60,560. | Great Brak. 30600 | 3497,550 | -46,710. |
| 1000 | 3479,050 | -60,600 | 31000. | 3497,900 | -46,760. |
| 2000 | 3479,850 | -61,250 | 32000 | 3498,550 | -46,280 |
| 3000 | 3480,675. | -61,825. | 33000. | 3499,380 | -45,810 |
| 4000. | 3481,590 | -61,720. | 34000. | 3500,235 | -45,250 |
| 5000 | 3482,460 | -61,680 | 35000 | 3500,980 | -44,675. |
| 6000 | 3483,000 | -61,110 | 36000 | 3501,890 | -44,800. |
| 7000 | 3483,600 | -60,400 | 37000 | 3501,610 | -43,875. |
| 8000 | 3484,320 | -60,320 | 38000 | 3502,400 | -43,425. |
| 9000 | 3485,150 | -59,825 | 39000. | 3503,333 | -43,180 |
| 10000 | 3485,750 | -59,075. | 40000 | 3504,000 | -43,880 |
| 11000. | 3486,130 | -58,300. | 41000 | 3504,975 | -43,740. |
| 12000 | 3486,220 | -57,640. | 42000 | 3505,940 | -43,800 |
| 13000 | 3487,030 | -56,700 | 43000. | 3506,460 | -43,010 |
| 14000 | 3487,675. | -56,200 | 44000. | 3507,110 | -43,730 |
| 15000 | 3488,000 | -55,340. | 45000 | 3507,755 | -44,415. |
| 16000. | 3488,180 | -54,490. | 46000 | → Section [33] | Grassridge Dam |
| 17000 | [illegible] | [illegible] | | | |

**Figure 4.1-3: Hand Written Co-ordinates as Read from the 1:50,000 Maps Used to Locate the Position of the River at the Chainage Point**

The data was then captured into a spreadsheet. From here the data was arranged and ordered, this allowed for a Macro to be run that entered a river channel section at the centre line point of the river.

There was little data available as to the exact shape of the river channel, and this could not be gleamed from the 1:50,000 Maps. Thus the channel had to be assumed from what data was available to the modeller for this purpose.

For the four dams on the system, the capacity determination surveys (DWAF - 1990, 1994, 2000, 2002) were used in each case. These provided accurate and detailed sections through the dam, which could be entered into the model.

## 4.2 Creation of the Assumed River Channel Section

The cost and time delay to perform a complete survey of the entire Orange-Fish-Sundays scheme river network with comprehensive surveyed cross sections of the rivers and canals at regular intervals was prohibitive. Due to the limited detailed data available with regards to the river channel shape, 34 surveys were requested to be done at various selected sections along the scheme. These included some of the main weirs as well as the canals and key rivers. The sites were selected to have easy access for the survey team and to provide the most amount of data for the least amount of effort. The survey was able to complete 30 sites and provide historical survey data for two additional sites. The two sites that were not surveyed were due to the Ecca tunnel from Hermanuskraal to Glen Melville Dam being flooded at the time. Table 4.2-1 shows the list of sites surveyed. The data and pictures of these sites are available in Appendix A.

**Table 4.2-1: List of Sites to be Surveyed for Accurate River / Canal Channel Sections.**

| Site # | River Name | Comments / Details | Completed |
|---|---|---|---|
| 1 | Ovistunnel Outlet | Elevation and Section of Canal Start after Gauging Weir Q1H014 | Yes |
| 2 | Teebus | Maryland | Yes |
| 3 | Groot Brak | Brak River Off take.  100m Down Stream of Weir | Yes |
| 4 | Groot Brak | Woolwyn | Yes |
| 5 | Groot Vis | Katkop Weir.  Up Stream | Yes |
| 6 | Groot Vis | Katkop Weir.  Over Weir | Yes |
| 7 | Groot Vis | Katkop Weir.  Down Stream | Yes |
| 8 | Groot Vis | 100m Downstream from Weir at Marlow | Yes |
| 9 | Groot Vis | Cloverfield | Yes |
| 10 | Canal from Elandsdrift Weir | Elevation and Section of Canal Start after Gauging Weir Q5H006 | Yes |
| 11 | Cookhouse Tunnel Intake | Elevation and Section of Tunnel Intake | Yes |
| 12 | Cookhouse Tunnel Outlet | Elevation and Section of Canal from Tunnel Outlet | Yes |
| 13 | Groot Vis | Eastpoort | Yes |
| 14 | Groot Vis | Harefield | Yes |
| 15 | Little Fish | Kranskloof | Yes |
| 16 | Canal from De Mistkraal Dam | Elevation and Section of Canal Start after Gauging Weir Q8H013 | Yes |
| 17 | Little Fish | Ripon | Yes |
| 18 | Groot Vis | Junction Drift.  100m Upstream of confluence | Yes |
| 19 | Little Fish | Junction Drift.  100m Upstream of confluence | Yes |
| 20 | Great Fish | Junction Drift.  100m Downstream of confluence | Yes |
| 21 | Great Fish | Junction Drift.  Over Weir | Yes |
| 22 | Skoenmakers | Middlewater | Yes |
| 23 | Sundays | Schiet Hoogte (by Plat Berg) | Data from previous survey |
| 24 | Sundays | Breeknek | Data from previous survey |
| 25 | Canal from Korhaansdrift | Elevation and Section of Canal Start after Gauging Weir N4H006 | Yes |
| 26 | Canal to Scheepers Vlakte | Elevation and Section of Canal Start before it drops into the Dam | Yes |
| 27 | Great Fish | Carlisle Bridge | Yes |
| 28 | Great Fish | Hermanus Kraal Weir. Tunnel Elevation and Section | No-Tunnel under water |
| 29 | Great Fish | Hermanus Kraal Weir. Up Stream | Yes |
| 30 | Great Fish | Hermanus Kraal Weir. Over Weir | Yes |
| 31 | Great Fish | Hermanus Kraal Weir. Down Stream | Yes |
| 32 | Glen Melville Tunnel Outlet | Elevation and Section of Tunnel Outlet | No-Tunnel under water |
| 33 | Glen Melville Tunnel Outlet | Elevation and Section of Canal to Ecca River | Yes |
| 33 | Groot Vis | Kleinkanaan | Yes |

The most common river channel section assumption is a trapezoidal section as shown in Figure 4.2-1.  This section is virtually a canal section but becomes unstable during simulations as the hydraulic radius, flow depth and flow width at low flow rates creates instabilities in the finite difference routines.

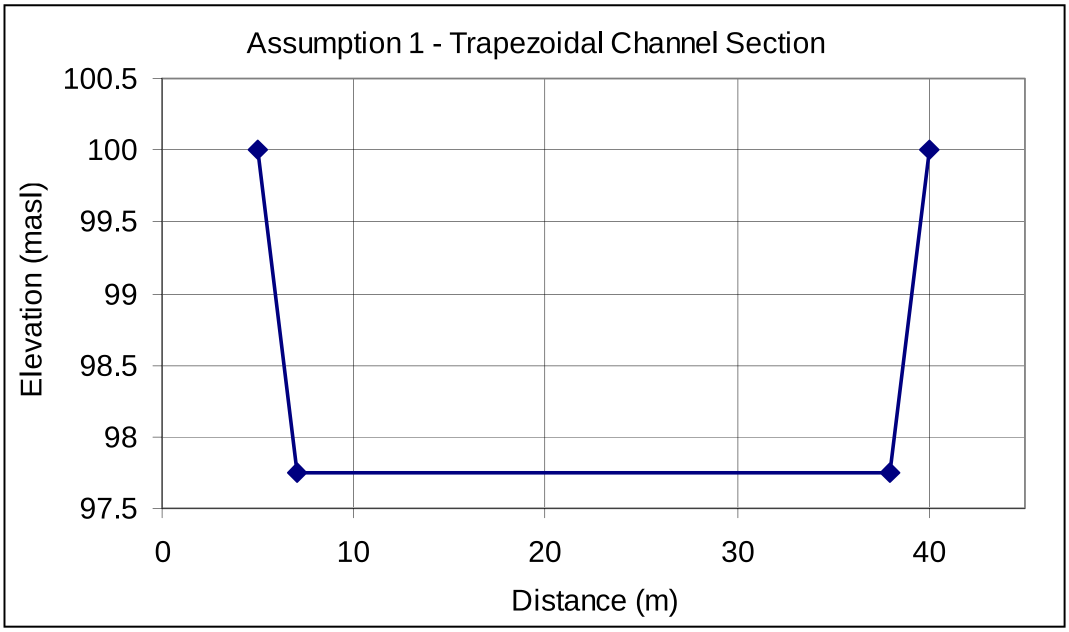



**Figure 4.2-1: An Example of a Simple Trapezoidal Section for an Assumed River Channel (Assumption 1)**

To assist in the solution of low flow conditions, a low flow slot is often added to the assumed section. This practice is recommended by most modelling programs and has been used previously as a solution to low flow instabilities. This is simply a schematized smaller trapezium added to the bottom of the simple trapezium section as shown in Figure 4.2-2. This section is more stable during low flow conditions as the relationship between the cross sectional area and the wetted perimeter is adjusted by the low flow slot.

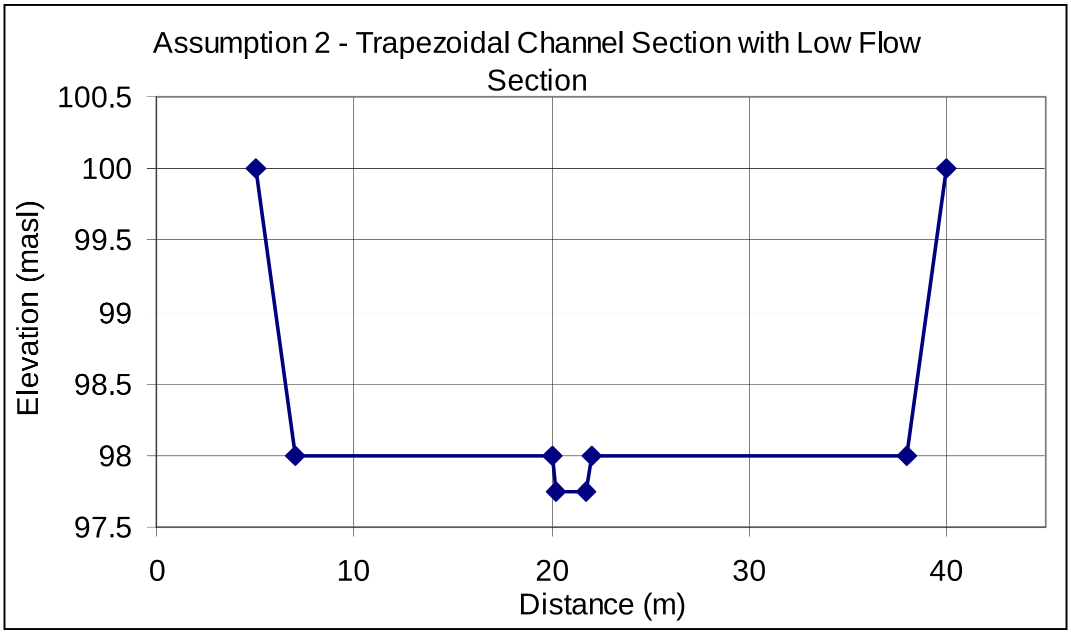



**Figure 4.2-2: An Example of a Simple Trapezoidal Section with a Low Flow Slot (Assumption 2)**

It was considered that the transition from the low flow section to the main channel section was going to create stability problems and was not necessarily an efficient solution to the problem. Observations of various river cross sections taken from the Orange-Fish-Sundays Scheme showed that the river cross section would be better modelled with a skewed trapezoidal cross section. That is that the one corner of the lower section is dropped lower than the other.

The theory behind this section is based on the pattern of scour and deposition that occurs in a river bed as the river winds its way through the earth, the flow velocity on the outer edge of the bend in a river is greater than that on the inner edge, thus there are patterns of scour due to increased velocity and transportation on the outer edge as opposed to deposition of transported material due to the lesser velocities on the inner edge. This results in the one side of the river bed being somewhat lower than the other. Figure 4.2-3 shows a surveyed river section in blue with the assumed Skewed Trapezoid super imposed on top in magenta.

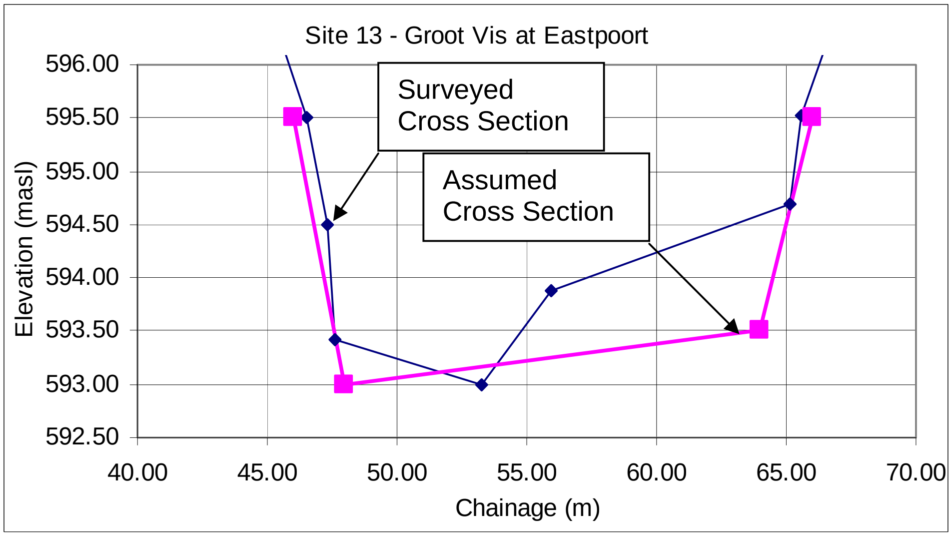



**Figure 4.2-3: A Typical Surveyed River Section Showing Skewed Trapezoidal River Channel Assumption (Assumption 3)**

With this section there is always a triangular shaped flow section during low flows, thus smoothing out the relationship between the wetted perimeter and the cross sectional area of flow. Thus the variation of hydraulic radius with flow depth is smooth, along with the variation of flow width with flow level. As Figure 4.2-4 shows, there is a discontinuity through the level of the low flow slot in Assumption 2, while this is smoothed out by Assumption 3.

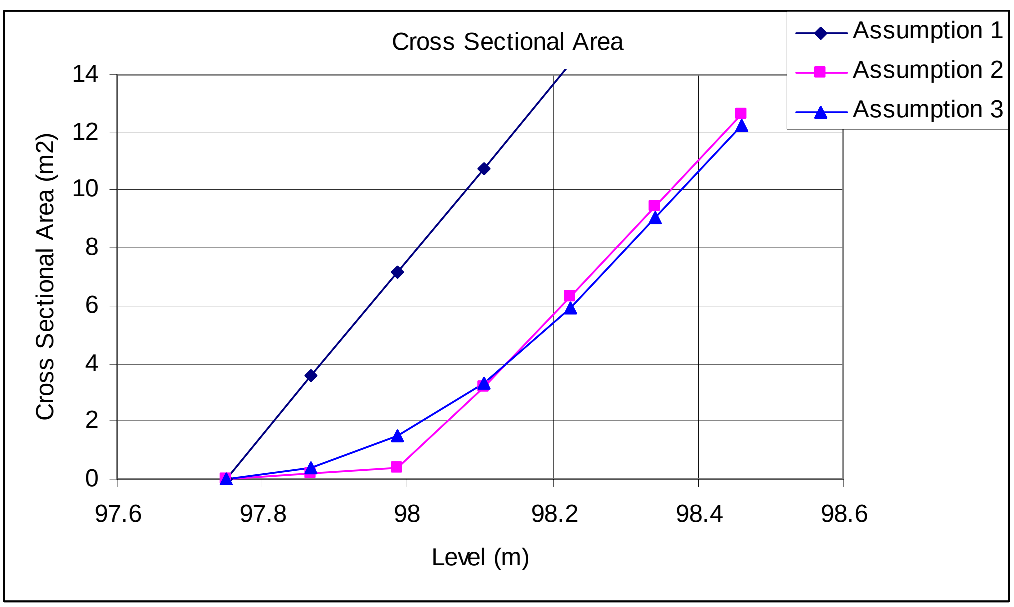



**Figure 4.2-4: Variation of Cross Sectional Area of Flow with Flow Level for the 3 Assumed Channel Sections. Note the Discontinuity in Assumption 2 with Respect to Assumption 3.**

This discontinuity in Assumption 2 is further illustrated in Figure 4.2-5. There is a sharp jump in the flow width as the flow level rises out of the low flow slot and into the main channel. This has an effect on the Hydraulic Radius used in the Manning's Equation (Equation 4.2-1) to model the bed friction ($S_f$) term in the Momentum Equation (Equation 3.1-2) of the Saint-Venant Equations.

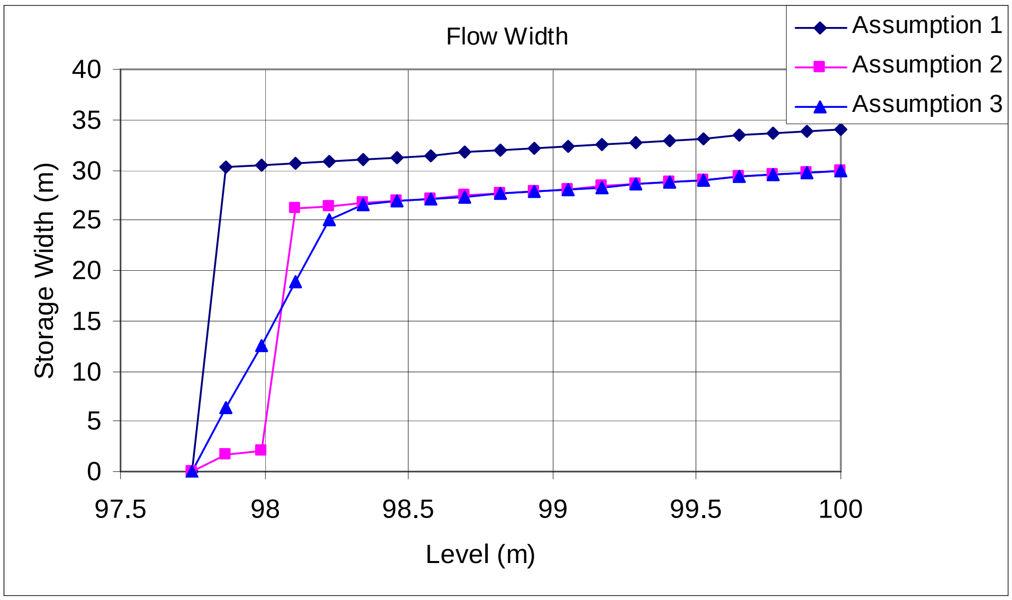



**Figure 4.2-5: Variation of Flow Width with Flow Level for the 3 Assumed Channel Sections**

The Hydraulic Radius is the relationship between the area of flow and the wetted perimeter. It is essentially a function of the shape of the flow (Equation 4.2-2). An economical section will have a larger relationship between the area and the wetted perimeter; this means that there is less flow surface being affected by the roughness of the bed thus the flow is less influenced by friction losses from the bed. As Equation 4.2-2 shows, the larger the value of R, the greater the value of V.

$$V = \frac{R^{\frac{2}{3}}\sqrt{S_f}}{n}$$ **Equation 4.2-1**

Where:

$$R = \frac{A}{P}$$ **Equation 4.2-2**

The variation of hydraulic radius with flow level for the three assumed channel sections is shown in Figure 4.2-6. It is important to note how in Assumption 2, the hydraulic radius increases along with that of Assumption 1, then drops significantly to below that of Assumption 3. This rapid change in the hydraulic radius can result in the flow rate and flow level fluctuating in this flow region resulting in instabilities that could decrease the time step needed for the simulation to run and thus reducing the efficiency of the model.

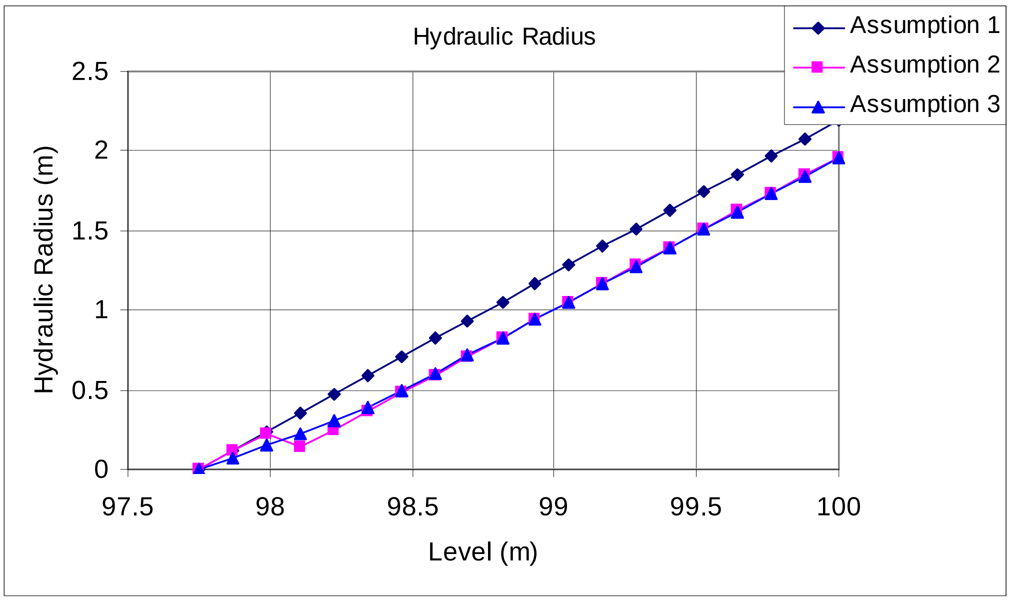



**Figure 4.2-6: Variation of Hydraulic Radius with Flow Level for the 3 Assumed Channel Sections**

To test the stability of the three sections, each was used as the assumed section for an 18 km long reach with a slope of 1:1000 and a Manning's n value of 0.04 which was the general Manning's n value from the calibration of the Model. The sections were all similar in that the top width was 30m and the maximum depth was 2m, the sides were 1:1 (except for the one side of Assumption 3 which was steeper at 1:0.8). The Figures 4.2-7 to 4.2-10 provide the details of the relationships between Cross Sectional Area, Flow Width and Hydraulic Radius against Flow Level for each of the three assumed channel sections. The values for these variables are calculated for twenty equidistant level increases.

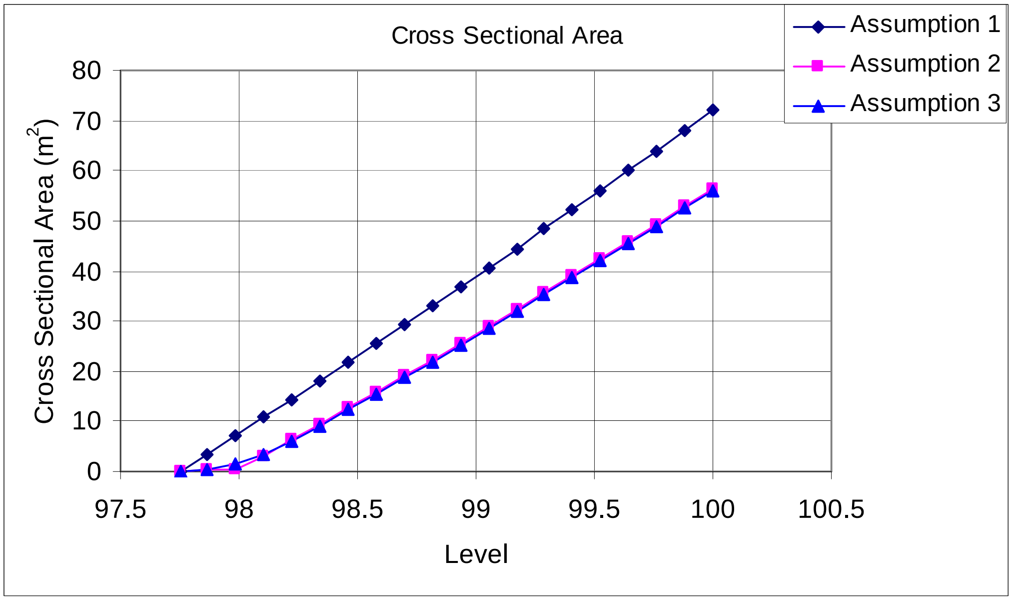



**Figure 4.2-7: Cross Sectional Area against Flow Level for the 3 Assumed Channel Cross Sections**

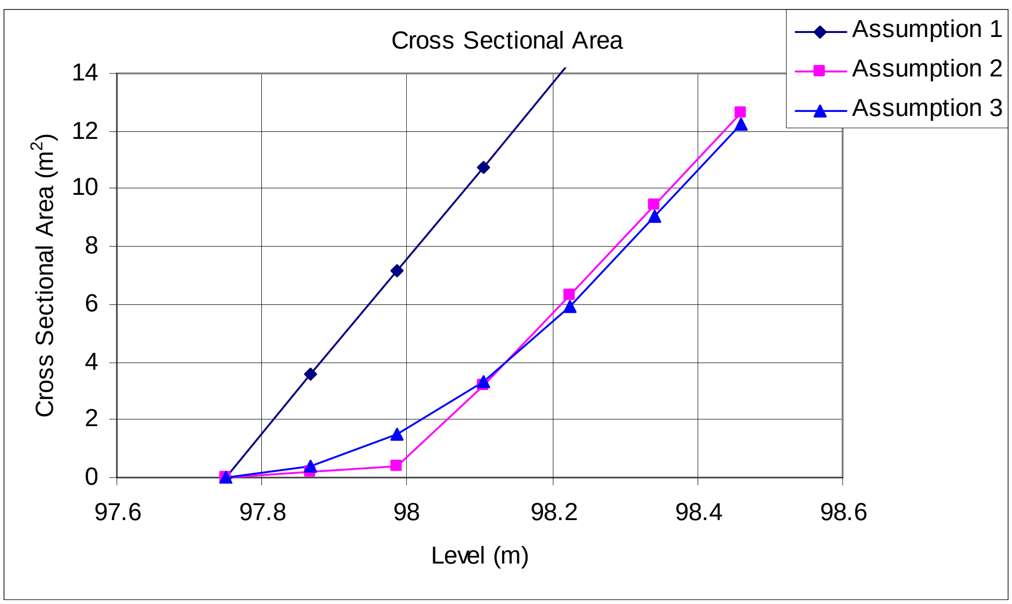



**Figure 4.2-8: Close View of the Low Level Cross Sectional Area against Flow Level for the 3 Assumed Channel Cross Sections**

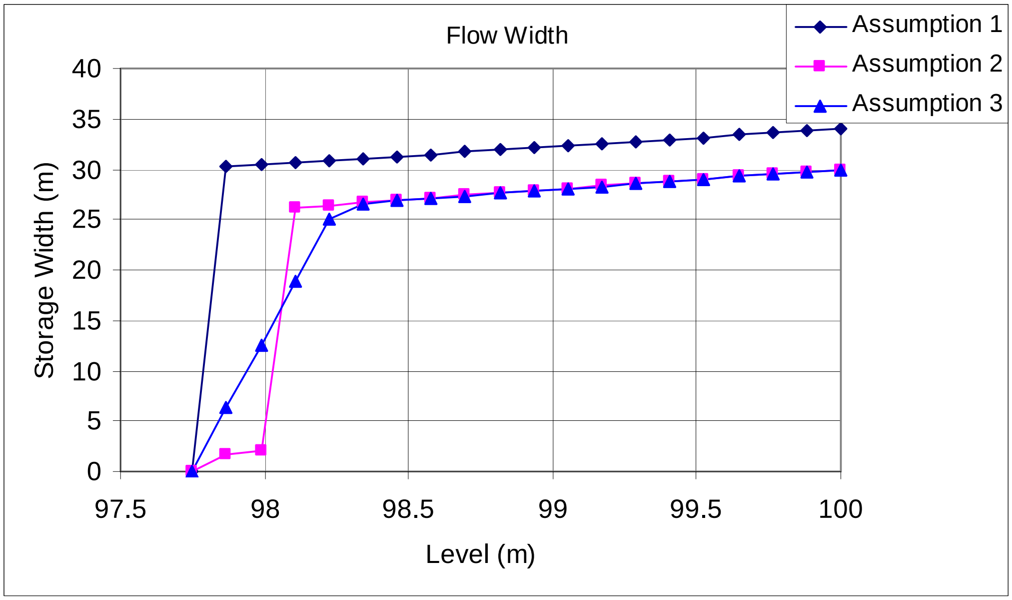



**Figure 4.2-9: Flow Width against Flow Level for the 3 Assumed Channel Cross Sections**

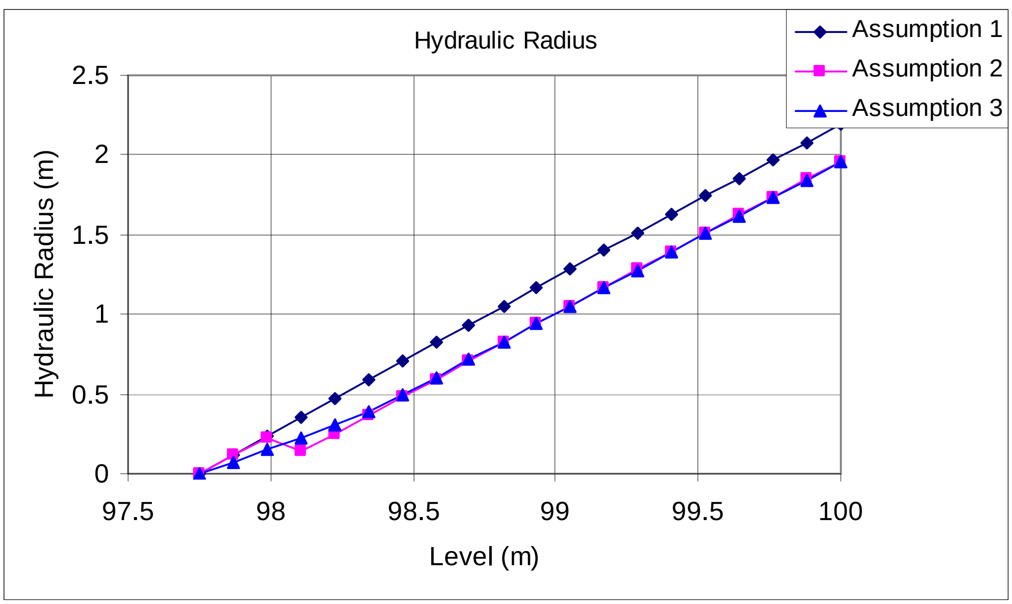



**Figure 4.2-10: Hydraulic Radius against Flow Level for the 3 Assumed Channel Cross Sections**

These sections were tested with a hydrograph designed to test the limit of low flow as well as the ability of the channel section to calculate a rapid increase in flow from a low flow. From the modeller's experience, this is one of the most unstable flow conditions due to its dynamic nature. The test hydrograph is shown in Figure 4.2-11. The flow rate is reduced to 0,001 $m^3/s$.

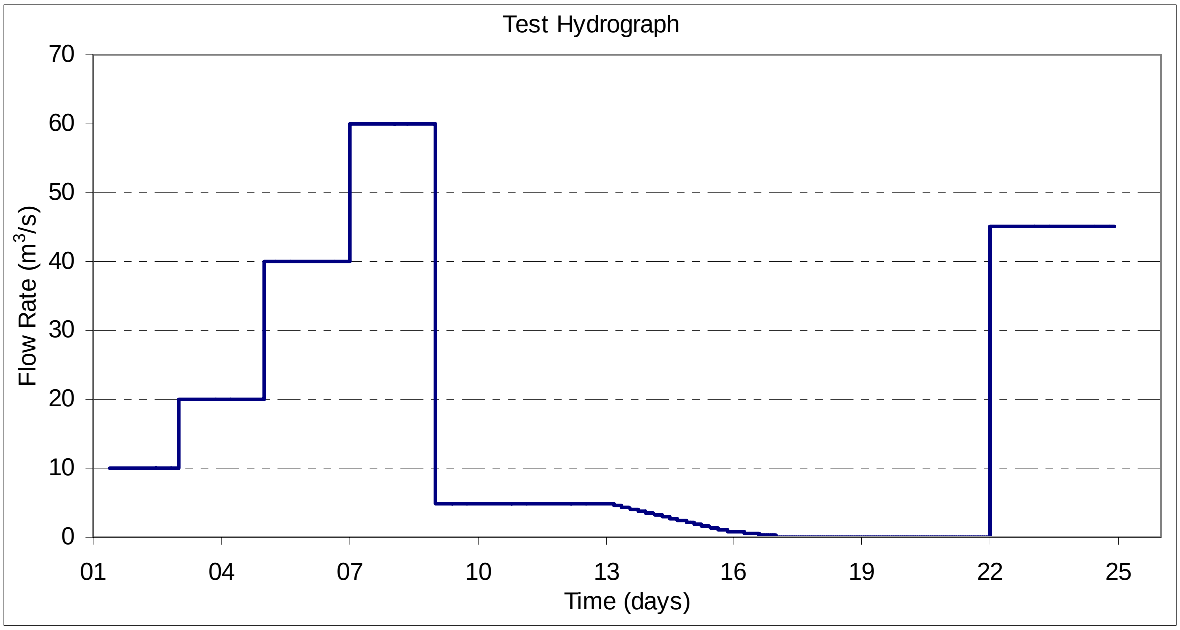



**Figure 4.2-11: Test Hydrograph used to Test the Stability of the Assumed Channel Sections**

The time step was increased after each successful simulation until all the assumptions had suffered permanent stability problems. The following Figures 4.2-12 to 4.2-14 show the discharge for each of the 3 assumptions for an increase in flow from 0.001 $m^3/s$ to 45 $m^3/s$. For each figure the black line represents Assumption 1, the blue line represents Assumption 2 and the green line represents Assumption 3. The red line is the inflow hydrograph displayed as a reference. The Time Series plots are taken at chainage 4500.

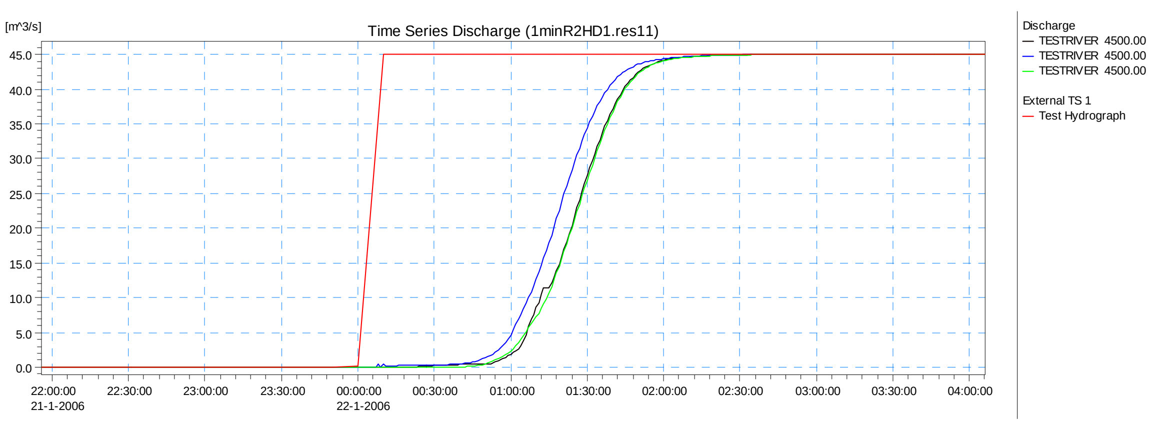



**Figure 4.2-12: Discharge for the 3 Assumed Channel Sections for a Time Step of 1 min**

For a time step of one minute, all three assumptions are stable and reliable. There is some noticeable difference between Assumption 2 and the others as it is affected by the flow increase before the other channel sections. This leads one to conclude that the flow velocity is greater.

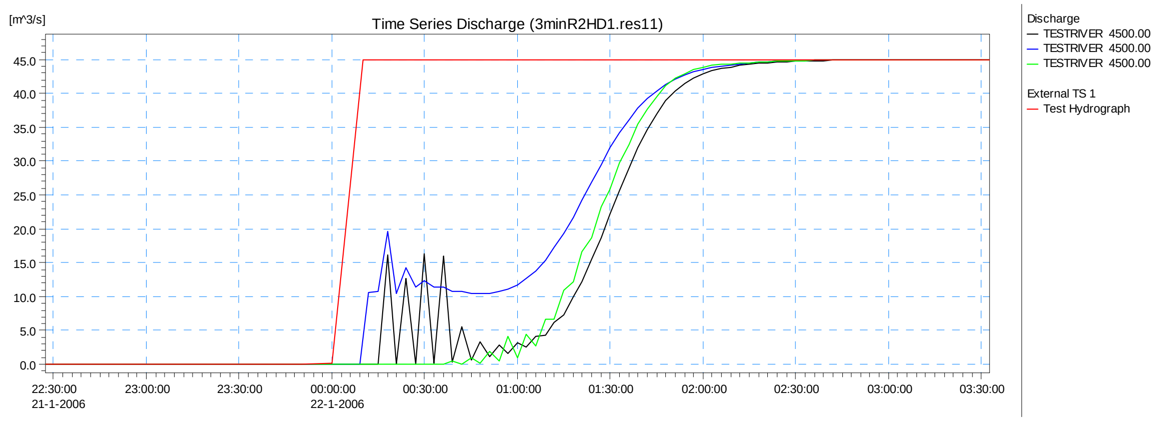



**Figure 4.2-13: Discharge for the three Assumed Channel Sections for a Time Step of 3 min**

At a time step of 3 minutes, there is instability in the finite difference solution for all three sections. This is most pronounced for Assumption 1. Assumption 2 shows a strange behaviour, as there is an initial jump followed by some instability then a smooth increase in the flow rate. This section again experiences the increase in flow rate earlier than the other sections. Assumption 3 shows only a slight instability which decreases gradually as the flow rate increases, yet this divergence from the solution is small and far more in line with a practical observed result.

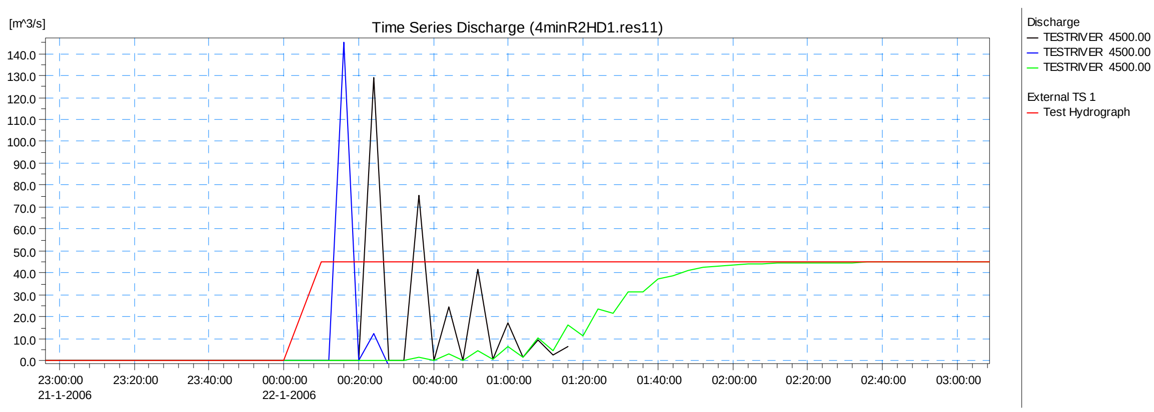



**Figure 4.2-14: Discharge for the three Assumed Channel Sections for a Time Step of 4 min**

At a time step of 4 min, both Assumption 1 and 2 have suffered a permanent error, while Assumption 3 has completed the simulation. Thus Assumption 3 is the more resilient and stable channel section of the three tested. Further results for Velocity, Froude Number and Hydraulic Radius are presented in Appendix B.

From the tests conducted on the different assumed channel cross sections, it was decided to use Assumption 3. This decision was taken in the light that the macro that inserted the channel section was far simpler to implement. As shown in Figure 4.2-15, the assumed channel section requires just two variables, the width of the section and the minimum depth of the section. These are given in the line that started with “b” on the left hand side, the “30” is the width while the “2” is the lesser depth. On the right hand side, one can see how the new section has been inserted at the level of the river bank (1139 masl).

| | | | | | | |
|---|---|---|---|---|---|---|
| Q1H014-Q1R001 | | | | Q1H014-Q1R001 | | |
| TeebusRiver | | | | TeebusRiver | | |
| 16000 | | | | 16000 | | |
| Profile | | | | Profile | | |
| LB | 0 | 1160 | <#1> | 0 | 1160 | <#1> |
| | 3600 | 1140 | | 3600 | 1140 | |
| CL | 3650 | 1139 | <#2> | 3635 | 1139 | |
| | 3725 | 1140 | | 3637 | 1137 | |
| RB | 4650 | 1160 | <#4> | 3663 | 1136.5 | |
| b | 30 | 2 | | 3665 | 1139 | |
| | | | | 3725 | 1140 | |
| | | | | 4650 | 1160 | <#4> |
| ***** | | | | ******** | | |

**Figure 4.2-15: Shows the Raw Data and the Processed Data for the Insertion of the Assumed Channel Section**

A difference in bottom depth of 0.5 m was deemed sufficient. This assumption was not tested, but was shown by the results to work. There is scope for further research into the relationship between the slope of the bottom and the stability of the finite difference solution.

## 4.3 Boundary Conditions

Each of the tributaries which had historic flow data was included into the network of the model. There were eight tributaries as well as the inflow to the model from the OVIS Tunnel outlet. These constituted the inflow boundaries for the model. Data was collected from the Department of Water Affairs and Forestry Hydrological department in Cradock in the Eastern Cape. The Department had installed a new stage logging system in 2001 which was operational by 2002. As such, reliable stage and flow data was available for each of these stations along with the flow gauging stations along the scheme. This data from 2002 to 2005 was planned to be used for the calibration and verification of the model. In addition to this data, the water level fluctuations in the four dams were collected. The discharge from Elandsdrift Dam and De Mistkraal Dam to the Groot Vis and Little Fish respectively was not available for the period from 2002 to 2005 as there are not flow gauging stations downstream of these structures to measure the outlet flow release. The required flow release from these structures was available for the modelling from the hand kept records of flow discharge. These seldom made accurate allowance for additional flow release to the river during times of flooding. In the case of Elandsdrift Dam this was most severe as there are 4 different methods to release the flow from the dam to the river. These have been mentioned in Chapter 2. The location of each of the inflow boundaries, downstream boundaries, control structure outlets and flow gauging stations are shown in Figure 4.3-1.

The flow through the downstream boundaries was modelled with the weir stage – discharge relationship at Korhaansdrift, though data was not available during the calibration period. At Fort Brown Bridge, the controlled section under the bridge had just been installed and no stage – discharge relationship was available during the calibration period. This is available now for the operation of the model. The weir at Hermanuskraal does not have a stage – discharge relationship either, due to the sluice gate and fish ladder built into the structure. For the purpose of the calibration of the downstream boundaries of the model, Darlington Dam's water level and the flow at Piggott's Bridge Weir were used.

The calibration reaches were defined as follows in Table 4.3-1.

**Table 4.3-1: Flow Calibration Reaches**

| | Upstream Station | Downstream Station | Length (Km) |
|---|---|---|---|
| A | Teebus OVIS Outlet | Teebus at Jan Blaauwskop | 23.6 |
| B | Grassridge Dam | Waaikraal Weir | 70 |
| C | Elandsdrift Dam | Sheldon Weir | 146 |
| D | Sheldon Weir | Piggott's Bridge Weir (End of Calibration) | 121 |
| E | Elandsdrift Canal | Little Fish Canal, Parshall 2 | 43 |
| F | De Mistkraal Dam | Junction Drift (Great Fish River Confluence) | 58 |
| G | De Mistkraal Canal | Skoenmakers Canal, Parshall 3 | 27 |
| H | Darlington Dam | Korhaansdrift Abstraction (End of Calibration) | 48 |

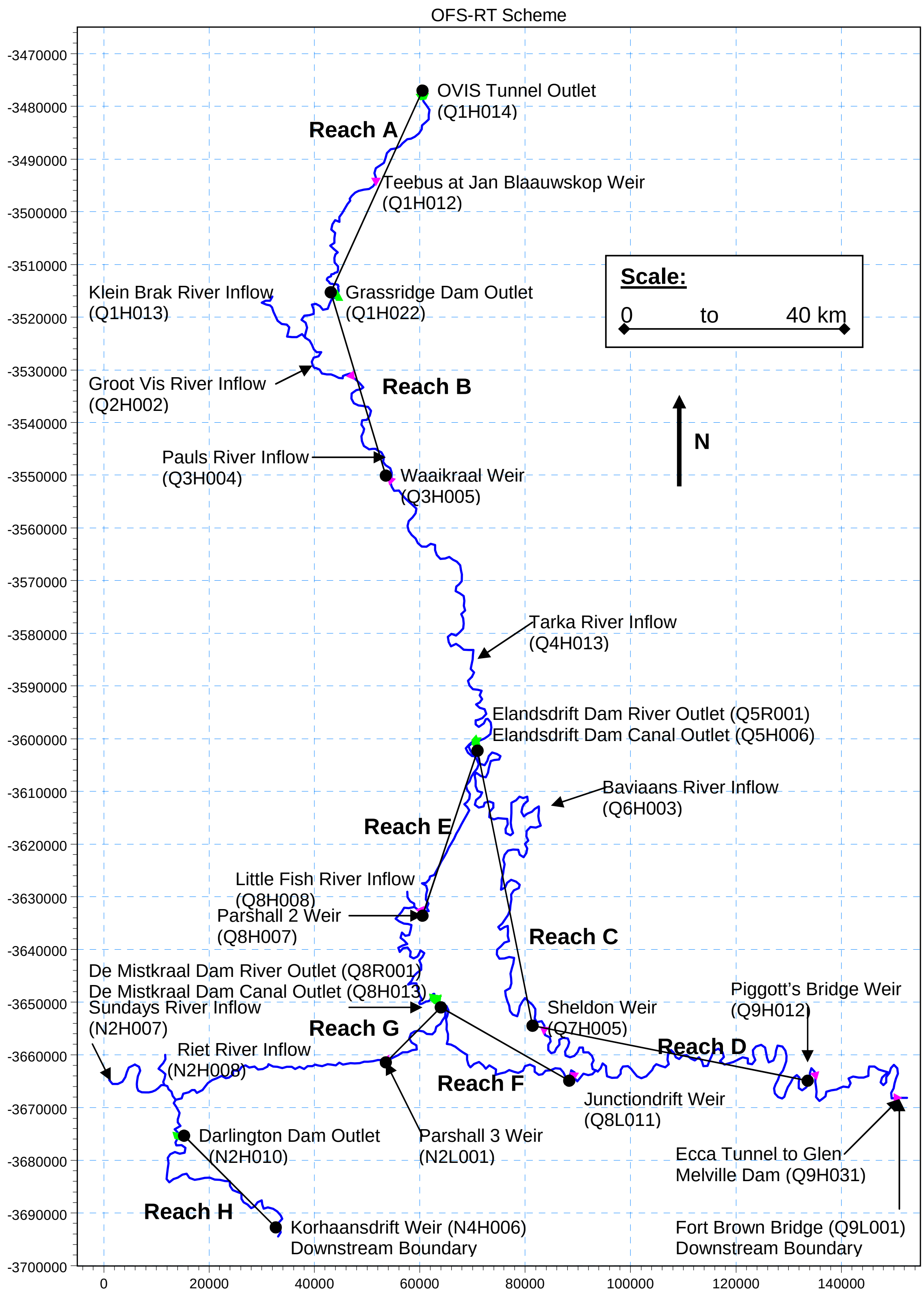



**Figure 4.3-1: Inflow Boundaries, Control Structure Outlets, Downstream Boundaries, the Flow Gauging Stations and the Calibration Reaches Available for the Model Setup**

The Mean Annual Flow (MAF) during the period from 1980 to 2005 for each of the flow gauging stations is provided in Table 4.3-2. This provides an indication of which flow stations have the greatest influence on the system. From the table one can determine that 51% of the transfer from the OVIS Tunnel is diverted to the Little Fish River at Elandsdrift Dam and 54% of this flow (or 28% of the total transfer) is diverted to the Sundays River at De Mistkraal Dam.

**Table 4.3-2: Mean Annual Flow through the Flow Gauging Stations on the OFS Project from 1980 to 2005**

| **Flow Gauging Stations** | | | |
|---|---|---|---|
| **Station #** | **Station Name** | **MAF ($m^3$/yr)** | **% of OVIS Tunnel Outlet** |
| Q1H014 | Teebus OVIS Outlet | 689,720,776 | 100% |
| Q1H012 | Teebus at Jan Blaauwskop | 669,191,892 | 97% |
| Q1H022 | Grassridge Dam Release | 660,864,153 | 96% |
| Q3H005 | Waaikraal Weir | 593,509,620 | 86% |
| Q5H006 | Elandsdrift Canal Release | 352,456,000 | 51% |
| Q5R001 | Elandsdrift River Release | 284,234,874 | 41% |
| Q8H007 | Little Fish Canal, Parshall 2 | 246,987,649 | 36% |
| Q7H005 | Sheldon Weir | 246,860,821 | 36% |
| Q9H012 | Piggott's Bridge Weir | 240,676,275 | 35% |
| Q8H013 | De Mistkraal Canal Release | 190,768,400 | 28% |
| N2H010 | Darlington River Release (Including Spillway) | 182,682,899 | 26% |
| N2L001 | Skoenmakers Canal, Parshall 3 | 181,453,500 | 26% |
| N4H006 | Korhaansdrift Abstraction | 118,651,085 | 17% |
| Q8R001 | De Mistkraal River Release | 74,931,563 | 11% |
| Q8L011 | Junction Drift Weir | 63,466,770 | 9% |
| Q9H031 | Ecca Tunnel to Glen Melville Dam | 11,646,985 | 2% |
| **Tributary Gauging Stations** | | | |
| **Station #** | **Station Name** | **MAF** | **% of Total Tributary Inflow** |
| Q3H004 | Pauls River upstream of Waaikraal Weir | 8,161,853 | 10% |
| Q8H008 | Little Fish River upstream of De Mistkraal Dam | 13,997,582 | 18% |
| N2H007 | Sundays River upstream of Darlington Dam | 19,696,890 | 25% |
| Q1H013 | Klein Brak River downstream of Grassridge Dam | 440,752 | 1% |
| Q2H002 | Groot Vis River downstream of Grassridge Dam | 8,485,318 | 11% |
| Q4H013 | Tarka River upstream of Elandsdrift Dam | 14,717,876 | 19% |
| Q6H003 | Baviaans River downstream of Elandsdrift Dam | 12,935,957 | 16% |

Requests were made for all the abstraction data from the irrigation boards, private pumpers and municipalities available to the Department of Water Affairs and Forestry to be made available. This data is either compiled annually from the licence allocation for the irrigator based on the area of land under irrigation or received weekly by fax or phone call to the DWAF office in Uitkeer. A full set of this data was not available in its raw format and as such the text files produced by the existing FISUN model were the only reliable source of data on the volume of weekly flow abstractions. Unfortunately this data is deleted at the beginning of the next irrigation year and as such, during the initial calibration, only the abstraction data from July 2005 to the end of December 2005 was available.

Time series files in ".DSF0" format are required by Mike 11, these files had to be created from the data that was available. The data can be imported from a text file, though the format must conform to a specific layout. The date and time must be in "yyyy-mm-dd hh:mm:ss" format. Table 4.3-3 shows an example of the text file format for a time series import into the Mike 11 format. The text file is tab delimitated. The station name is in the first line and the fourth column, the headers for the two columns follow in the second line. The third line provides the import tool with information about the type of data that is being imported. This is not necessary but saves time if available. This example is for discharge with the units of $m^3/s$ and that the data is instantaneous. The word "Unit" defines this line of code as describing the type of data. The following lines provide the time in the correct format and the value for that time.

**Table 4.3-3: Format of Text File for Time Series Import**

| | | | | |
|---|---|---|---|---|
| | | | Station name | |
| | | Date | Data name | |
| Unit | 100001 | | 1800 | 0 |
| | | 2006-01-01 09:00:00 | | 10 |
| | | 2006-01-01 09:20:00 | | 10 |
| | | 2006-01-01 09:40:00 | | 10 |
| | | 2006-01-01 10:00:00 | | 10 |
| | | 2006-01-01 10:20:00 | | 10 |
| | | 2006-01-01 10:40:00 | | 10 |

All the inflow and abstraction time series are created as a text file in this format and then imported into the Mike 11 format and saved as ".DSF0" files. The model requires that the control structures are entered into the model to control the flow discharge from the dams allowing for the water level in the dams to be calibrated against the observed water levels.

## 4.4 Structural Data used in the Model

There are seven control structures in total. These are listed below:

- OVIS Tunnel Outlet
- Grassridge Dam Outlet
- Elandsdrift Dam River Outlet
- Elandsdrift Dam Canal Outlet
- De Mistkraal Dam River Outlet
- De Mistkraal Dam Canal Outlet
- Darlington Dam Outlet

Each one requires a control structure to release flow to the designated reach according to the time series created from the historical data for the structure. Two model setups were created; the first was the Hot Start simulation used to setup the initial conditions in the reaches as well as the correct water levels in the dams. The data assimilator module of Mike 11 was used to adjust the water level in the dams to the correct level. Once this had been completed, the start time of the full simulation run could be determined. The second model was the full simulation model. Here the initial conditions were determined by the Hot Start result file. The simulation was run until the end of the available time series data.

Data was requested from the Department of Water Affairs and Forestry on the size and stage – flow characteristics for each of the outlets of each of the dams. This data was for the most part not available or obsolete. Darlington and Grassridge Dams were both operating under restrictions imposed by the dam safety office of the Department of Water Affairs and Forestry. For Darlington Dam this was due to the maintenance of the Stoney gates installed on the auxiliary and main spillways not being done, thus the reliability of these gates being able to be opened in times of adverse flooding was not satisfactory. There were also concerns over the uplift pressures under the 80 year old dam. Due to these concerns, the auxiliary spillway Stoney gates were left open, thus reducing the FSL from 247.200 masl to 242.925 masl. The auxiliary spillway now acts as a free overflow crest.

For Grassridge Dam, there were concerns over the riprap that was placed on the upstream face of the earth embankment. This rock had weathered down from over 1 m in diameter to 20 mm in diameter in some instances as shown in Figure 4.4-1.

**Figure 4.4-1: The Weathering of the Riprap Protection on the Grassridge Dam Embankment**

There were also concerns with increased seepage through the left abutment in the vicinity of the outlet works during periods of high water level. The FSL as designed was 1057.84 masl, the water level is now restricted to a maximum of 1054.5 masl. This is approximately 30% of the original FSC. All the structural data available as well as the data that could be concluded from observations and local knowledge of the structures is presented in the Appendix C. These were the operational limits as used in the model to restrict the operational optimisation of the model.

# 5 Calibration of the Mike 11 Model

## 5.1 Initial Dry Period Calibration

Initial calibration was carried out with data from 2002 to 2003. This data was incomplete due to missing or unavailable abstraction data, thus the simulated results at the downstream check points were showing an excess of water. This data was used to provide an idea of possible Manning's n values for the reaches as well as determine the simulation stability of the assumed river channel sections and their low-flow section assumptions.

A global value of n = 0.05 was assumed for all river reaches and n = 0.035 for all canals. This initial assumption proved to be close to the final values, but as the flows are not exact, it was difficult to verify the results.

Upon receiving a set of full abstraction data as well as flow gauging station data for the period from 11 July 2005 to 31 December 2005, calibration of the flow could continue and estimates of the return flow could be made as a more exact idea of the abstractions was available. The abstraction data was gathered from the digital text files produced from the FISUN model simulations which are saved for each irrigation week and over-written each year. The irrigation year begins in July each year.

The data provided was of a reasonable quality and was complete. The first month was used to generate initial conditions in the Hot Start simulation run that would be used as the initial conditions for the full simulation runs. The Hot Start simulation was found to be stable and with flow matching that observed at flow gauging stations along the reach and with water levels in the dams matching those observed on the 2$^{nd}$ August 2005, this was taken as the start date for the calibration simulation. A flood event was observed after 3 November 2005 which could not be modelled due to large additional inflows of rainfall runoff that was not gauged. It was decided that a dry calibration would be done with the available data between 2$^{nd}$ August 2005 and 3$^{rd}$ November 2005. This provided a three month period with little to no additional rainfall supplying flow to the tributaries and additional rainfall runoff adding to the return flow along the reach lengths, thus the irrigation return flows could be more accurately modelled.

The calibration was done by observing the shape of the simulated flow rates at the downstream check points and the water levels in the dams against the observed values. MikeView has a statistical tool that compares these two files and calculates various statistical values. Of primary use was the correlation lag plot which lags the simulated results forwards and backwards against the observed values and determines the correlation ($R^2$) for each lag. From this it is possible to determine if the flow is early or late and make the necessary adjustments to the Manning's n value for that reach.

The following section gives the results of these dry period calibration runs providing details of the final Manning's n values for each calibration reach as well as comments on accuracy and reliability of the results.

## 5.2 Initial Calibration of Reach A

The upstream boundary is defined as the flow through the weir on the canal from the OVIS Tunnel Outlet (Q1H014). The flow is routed along the reach for 23.6 km to a flow gauging weir at Teebus at Jan Blaauwskop (Q1H012), which serves as the downstream check point. The reach continues on for a further 22.4 km to Grassridge Dam, but there were no other gauging weirs upstream of the reservoir. A water level recorder has been installed at Ellion Bridge, just upstream of Grassridge Dam. Further calibration will be done at this check point when the stage – discharge relationship has been established along with a period of reliable data.

The assumed width and depth for the river channel was taken as 30 m wide and 2 to 2.5 m deep as shown in Figure 5.2-1. The assumed widths and depths of the river channel are derived from various sources. Firstly, where available, cross sections taken in the field were used, 34 sections were requested at various places along the entire scheme including all the canals. These provided actual information on which to base our assumptions. The observed shape and dimensions were also used from the field trip undertaken in July 2005 by the modelling team. Lastly, measurements of river width from available maps were used. The surveyed cross sections are available in Appendix A.

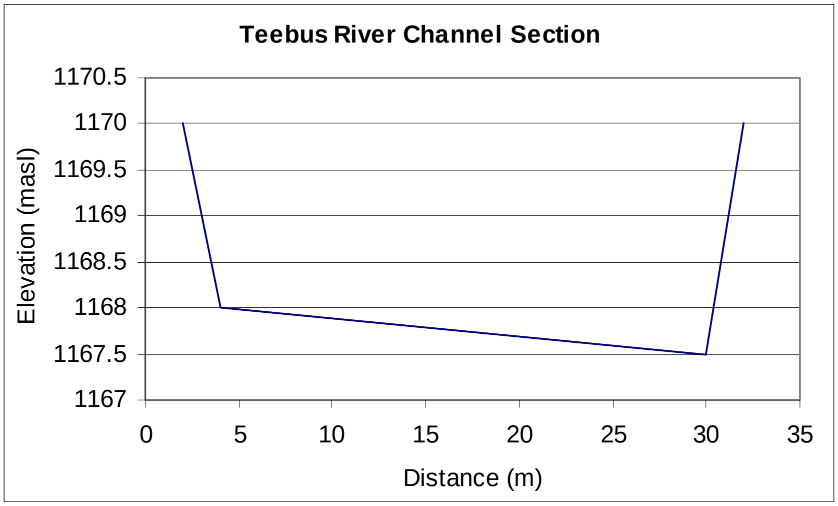



**Figure 5.2-1: Assumed Typical River Section for Teebus and Groot Brak Rivers**

Figure 5.2-2 shows the simulated flow rate against the observed flow rate. Note that for all plots of simulated against observed data, the simulated values are shown as a black line while the observed values are shown as a blue line.

A correlation of 0.956 was found for the calibration period. Although there are small differences in flow rate, these can be attributed to un-gauged abstractions, mistimed abstractions, errors in flow calibration of the gauging stations or additional flow from tributaries. Notice is made of the spike in observed flow rate between the 11-10-2005 and 21-10-2005. This can be attributed to a flood event on the Teebus River which is not gauged upstream of the confluence with the OVIS Tunnel Outlet canal as this river is non-perennial.

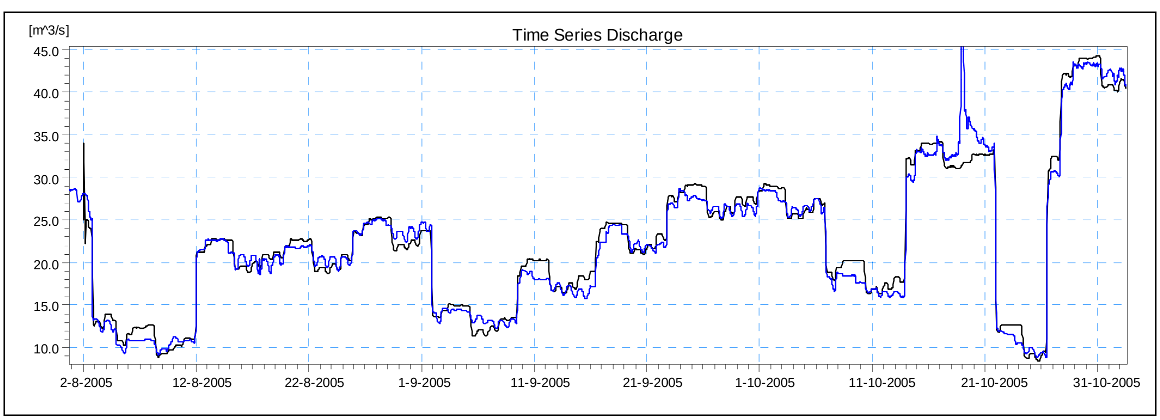



**Figure 5.2-2: Plot of Simulated (black) vs. Observed (blue) Flow at Teebus Jan Blaauwskop (Q1H012)**

Figure 5.2-3 shows a graphical representation of the statistical analysis between the simulated and observed flow rates over the calibration period. The error distribution has a very small band and shows a strong relationship between the simulation and the observed results. The anomaly that occurs between 31 and 32 $m^3/s$ on the gauge scale is as a result of the un-gauged flood, with most of the readings fitting in between 10% to -10%.

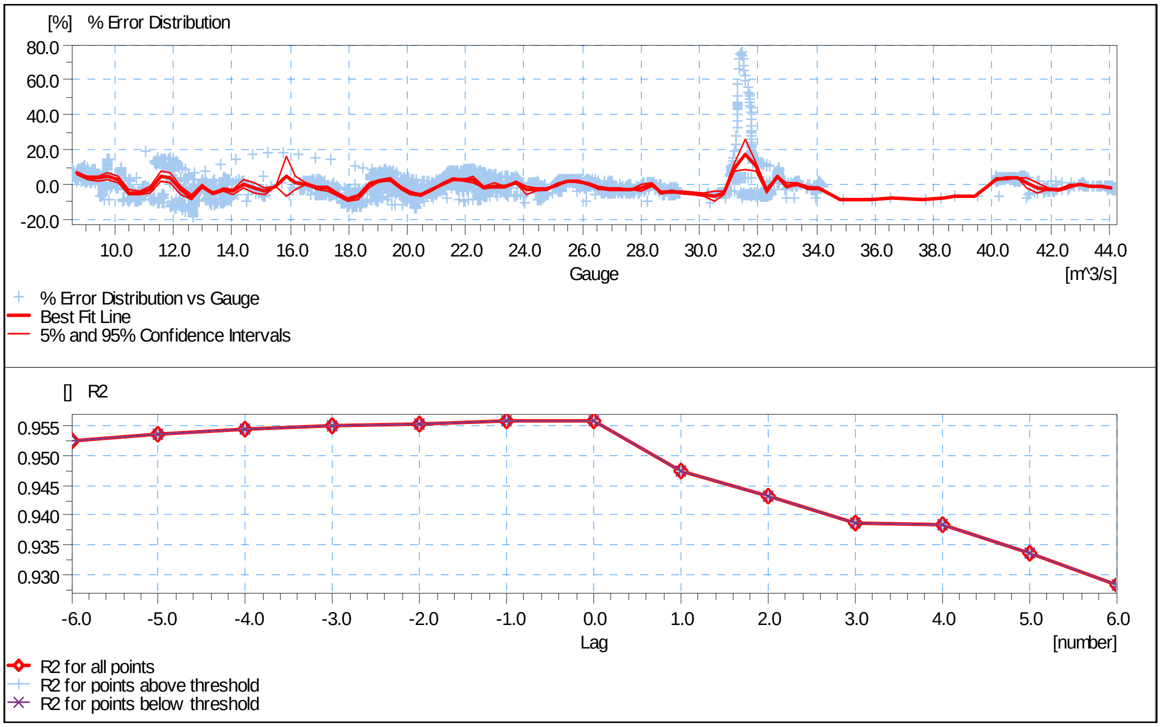



**Figure 5.2-3: Statistical Results for Reach A Showing the Error Distribution between Simulated and Observed Data and the Correlation with a Variation in Time Lag Forwards and Backwards**

From the calibration statistical results the Manning's n was found to be 0.037. See Table 5.2-1 for statistical results for reach A. Note the large maximum negative difference due to the flood event.

**Table 5.2-1: Statistical Results for the Initial Calibration of Reach A**

| Teebus Q1H012 | | |
|---|---|---|
| Correlation coefficient $R^2$ | 0.956 | |
| Max. positive difference | 3.292 | $m^3/s$ |
| Max. negative difference | -23.692 | $m^3/s$ |
| Volume observed | 173028037.9 | $m^3$ |
| Volume modelled | 174744410.5 | $m^3$ |
| Volume error | 0.992 | % |
| Peak observed value | 55.128 | $m^3/s$ |
| Peak modelled value | 44.25 | $m^3/s$ |
| Peak error | -19.732 | % |

## 5.3 Initial Calibration of Reach B

Reach B begins at the river outlet of Grassridge Dam (Q1H022) and ends at Waaikraal Weir (Q3H005) 70 km downstream. Waaikraal is a key measuring station between Grassridge Dam and Elandsdrift Dam and has recently undergone an upgrade and so the discharge table relating water elevation to flow was being re-calibrated at the time of this calibration run, thus the simulated flow discharge calculated at Waaikraal does not accurately reflect the observed measured flow. There is also concern at Grassridge Dam, as the un-corrected calculated mean annual flow (MAF) is 6.5% greater than the MAF through the OVIS Tunnel Weir (Q1H014). The difference is exaggerated further by the sum of the abstractions between OVIS Tunnel Weir and Grassridge Dam totalling 65.5 million $m^3/a$. The inflow from the Groot Brak River is not gauged as it is small and infrequent, thus is not included in the model. The discharge from Grassridge Dam was reduced to 90% of its measured value to provide a more reasonable flow in the reach. The reduction factor was optimised during the calibration process. A correlation was found as the fluctuation in water level is registered by the water level recorder so the time of flow and thus Manning's n can be calibrated.

The channel section width and depth was assumed as shown in Figure 5.3-1. This assumption was derived from the same process as for reach A. The Manning's n value was found to be 0.04 for the 30 km Groot Brak 2 reach and 0.033 for the 40 km Groot Vis 1 reach. The weir at Katkop shown in Figure 5.3-2 was included in the model as this weir has been re-instated as a flow gauging station. This large weir has the effect of reducing the time of flow due to level pool routing through this weir. The result is that the resultant Manning's n value is lower for this reach but is still within the expected range for a natural river of 0.03 to 0.06. Figure 5.3-3 shows the plot of simulated vs. observed flow data (note: no data available from 3 August 2005 to 12 August 2005 as no readings were able to be taken at Waaikraal Weir (Q3H005) during the re-construction of the weir.

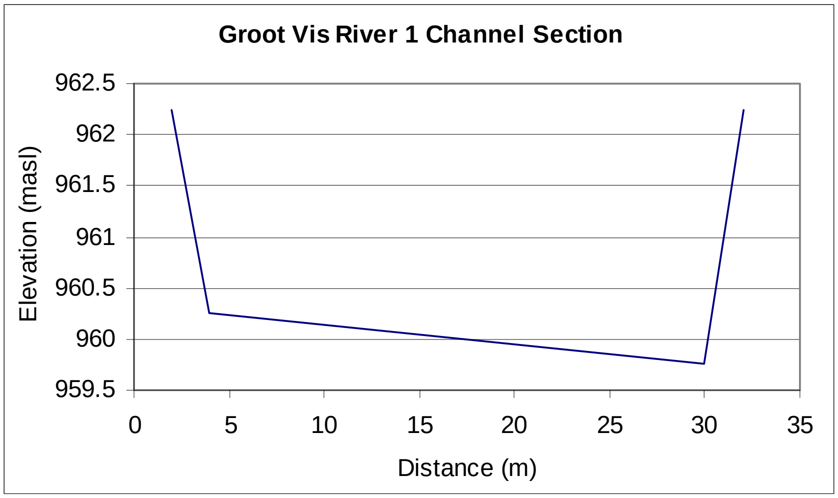



**Figure 5.3-1: Assumed Typical River Section for Groot Vis River Reach B**

**Figure 5.3-2: Flow over the Diversion Weir at Katkop on the Groot Vis River**

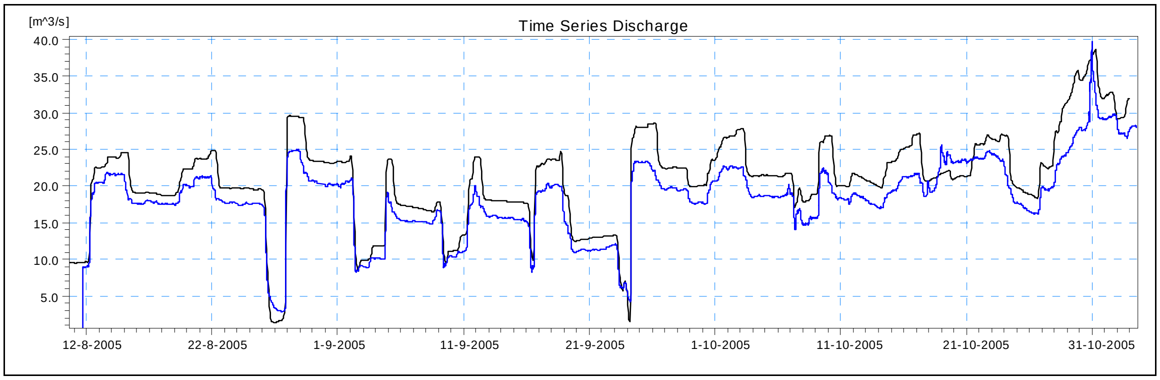



**Figure 5.3-3: Plot of Simulated (black) vs. Observed (blue) Flow at Waaikraal Weir (Q3H005)**

The difference between the observed and the simulated values plotted in Figure 5.3-3 are a function of the flow rate. The difference is greater for larger flows and smaller for lesser flows. This observation leads to the conclusion that the stage – flow relationship for the weir at Waaikraal is not accurate. One should also bear in mind that the flow in this reach has already been reduced to 90% of the measured release from Grassridge Dam (Q1H022). Once further calibration of this weir has been carried out, these results will be verified.

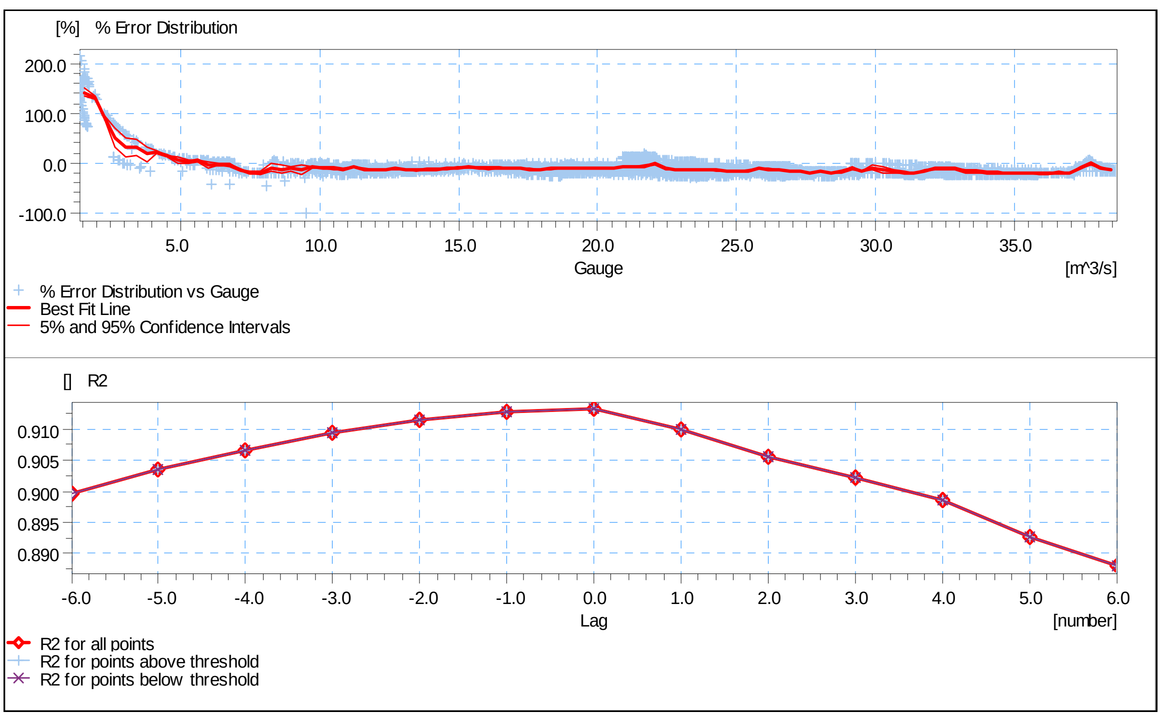



**Figure 5.3-4: Statistical Results for Reach B Showing the Error Distribution between Simulated and Observed Data and the Correlation with a Variation in Time lag Forwards and Backwards**

The plots in Figure 5.3-4 show that the correlation between the simulated results and the observed flow is 0.913. This is a good result given the problems with the calibration of the gauging weirs as mentioned above. There is a large error distribution for low flows, though this is more as a result of a small margin of error divided by a small value. But from 5 $m^3/s$ the error distribution is within 0% to -30%. This is due to a calibration error of the weir. Table 5.3-1 gives the results of the statistical analysis between the simulated and observed flow for reach B. Note the volume error of 14% which suggests that the already reduced flow from Grassridge Dam should be reduced by a further 14% if the observed flow at Waaikraal is believed to be accurate.

**Table 5.3-1: Statistical Results for the Initial Calibration of reach B**

| **Waaikraal Q3H005** | | |
|---|---|---|
| Correlation coefficient $R^2$ | 0.913 | |
| Max. positive difference | 9.535 | $m^3/s$ |
| Max. negative difference | -3.89 | $m^3/s$ |
| Volume observed | 130237020.7 | $m^3$ |
| Volume modelled | 148600798.1 | $m^3$ |
| Volume error | 14.1 | % |
| Peak observed value | 39.67 | $m^3/s$ |
| Peak modelled value | 38.64 | $m^3/s$ |
| Peak error | -2.596 | % |

## 5.4 Initial Calibration of Reach C

At Elandsdrift Dam (Q5R001) the flow is released to the river through various means (sleeve valve to the river from the dam structure, vertical sluice to the river from the diversion canal, belly flaps on the gated spillway section and in times of adverse flow the radial gates can be opened). These have been discussed in Chapter 2.

It was not clear what the actual flow released into the river was. A flow gauging station has been installed at a controlled section downstream of Elandsdrift Dam, though this was not operational at the time of the calibration. For this reason the modeller had to rely on the written log of what flow discharges were supposed to have been released as recorded by the dam control officer stationed at the dam. This is further complicated by the possibility of the gate operation levers being out of calibration. Thus the calibration of the flow from this point onward down the Groot Vis River is not as accurate as one would hope.

For reach C from Elandsdrift Dam (Q5R001) to Sheldon (Q7H005), the assumed width for the river channel of 25 m and depths of 2.5 to 3 m are shown in Figure 5.4-1. This assumption was made in the same manner as the other reaches. The Manning's n value was calculated to be

0.075 for reach C. This is much higher than the other n values found. The 146 km reach is the longest of the reaches with some of the largest abstractions along its length. The sum of the mean annual abstractions along this reach is 118.1 million $m^3/a$. These abstractions often have diversion structures built in the river which would attenuate the flow as well as slow down the time of flow. Thus there is a need to simulate this delayed flow by increasing the Manning's n value for this reach. Only key weirs were included at Middleton and Sheldon. Middleton was included as this weir is to be upgraded to a flow gauging station in the future. The inclusion of weirs can lead to instabilities in the flow and longer simulation time steps, which is why only weirs currently used for flow gauge measurement or those that are to be used in the near future are included. The cross section for Middleton Weir was assumed while the section used for Sheldon Weir was taken from the survey data provided by the DWAF survey team.

Figure 5.4-2 shows the simulated vs. observed discharge at Sheldon (Q7H005), indicating the poor fit between observed and simulated data. Figure 5.4-3 shows that there is a better error distribution for flow greater than 5 $m^3/s$. The $R^2$ vs. lag plot is just off the mark but close enough considering the poor correlation in relation to the other reaches and that this equates to a change in Manning's n in the order of a thousandth of a point.

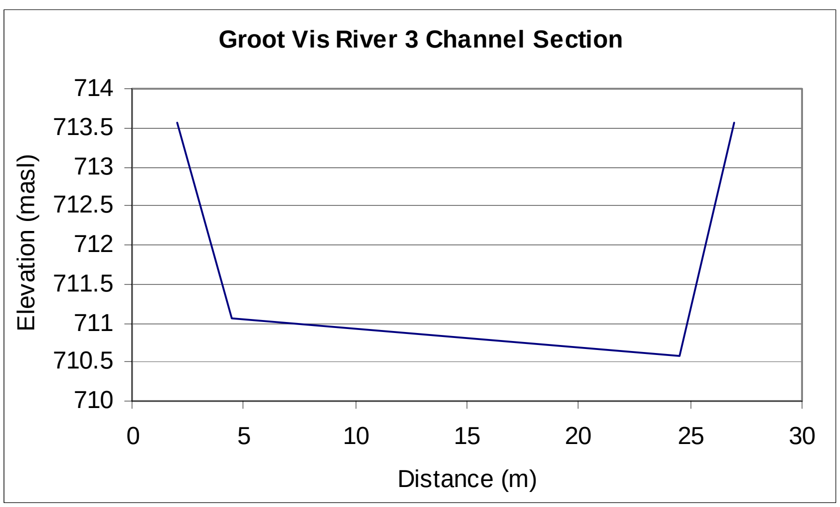



**Figure 5.4-1: Assumed Typical River Section for Groot Vis River 3 Reach C**

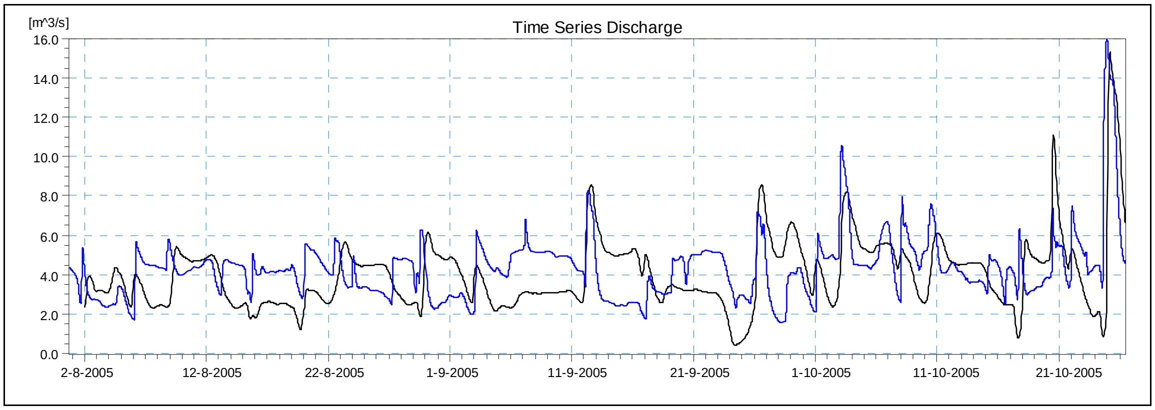



**Figure 5.4-2: Plot of Simulated (black) vs. Observed (blue) Flow at Sheldon Weir (Q7H005)**

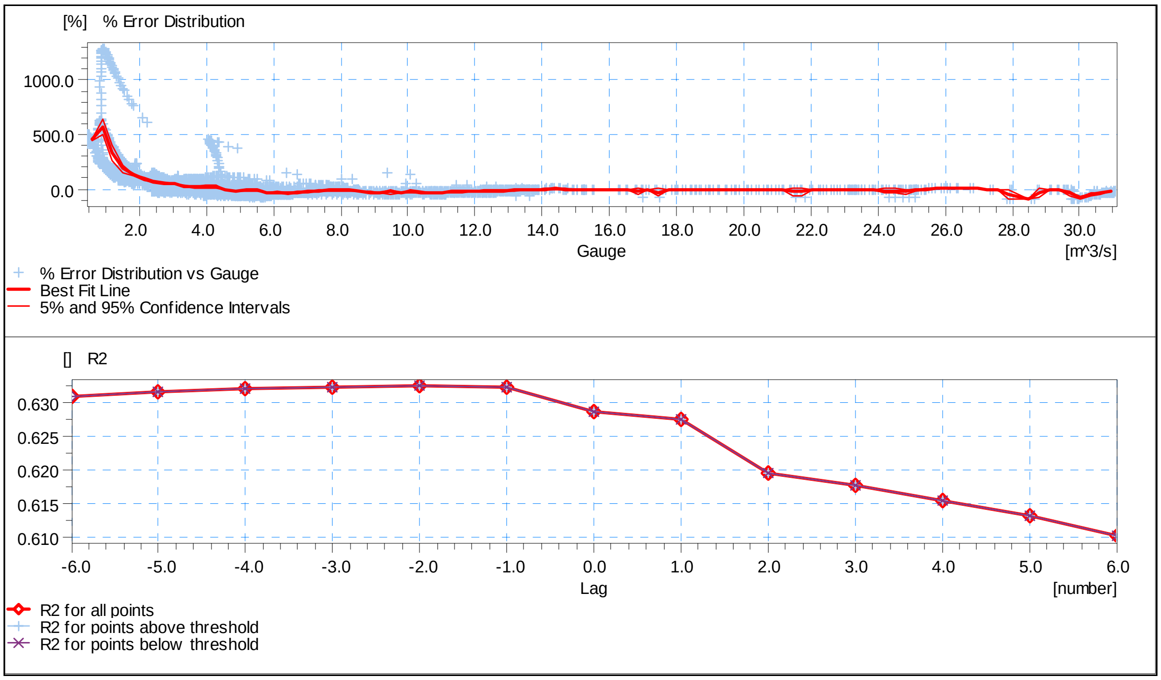



**Figure 5.4-3: Statistical Results for Reach C Showing the Error Distribution between Simulated and Observed Data and the Correlation with a Variation is Time Lag Forwards and Backwards**

The calibration of reach C at Sheldon is not as accurate as for other reaches. The main reason is due to the reach length. The 146 km meanders through an open flood plain with many abstractions and associated structures as well as many small un-gauged tributaries and return flows along the reach. A planned release from Elandsdrift Dam was considered but due to the reach length, the volume of water required would not be cost effective and thus it has been decided to continue the calibration process of this reach and reach D into the future as better results become available from the new flow gauging station at the controlled section downstream of Elandsdrift Dam as well as the additional weir re-instated at Middleton.

Table 5.4-1 gives the results of the statistical analysis between the simulated and observed flow for reach C at Sheldon. A volume error of 2.4% can be considered small while the maximum positive and negative difference is cause for further information for better calibration.

**Table 5.4-1: Statistical Results of the Initial Calibration of reach C**

| **Sheldon Q7H005** | | |
|---|---|---|
| Correlation coefficient $R^2$ | 0.629 | |
| Max. positive difference | 25.382 | $m^3/s$ |
| Max. negative difference | -18.538 | $m^3/s$ |
| Volume observed | 40505946.47 | $m^3$ |
| Volume modelled | 39546162.14 | $m^3$ |
| Volume error | -2.369 | % |
| Peak observed value | 27.65 | $m^3/s$ |
| Peak modelled value | 31.098 | $m^3/s$ |
| Peak error | 12.469 | % |

## 5.5 Initial Calibration of Reach D

There is no control structure regulating the flow at Sheldon (Q7H005) thus any error in the flow at Sheldon (Q7H005) is passed on to Piggott's Bridge (Q9H012). The system has been calibrated as a whole entity from Elandsdrift Dam through Sheldon on to Piggott's Bridge to better simulate the real possibilities of errors being passed on to the next check point in this case. There are no metered or known abstractions along the 121.6 Km of river in reach D. The area is mostly game farm land with little need of intense irrigation. There is undoubtedly some abstraction for domestic and small scale agricultural use but these are unknown so are not taken into account in the model as abstractions but as losses weighed against inflows from tributaries and ground water. As there are fewer abstractions along this reach, there is a reduction in the return flow from irrigated fields and as such there is a greater chance of river losses being the dominant factor to be considered.

The flow has not been well calibrated along this reach due to the uncertainty of flow release from Elandsdrift and the long reach lengths with many unknowns. The flow from the Little Fish River is gauged just upstream of the confluence of the two Fish Rivers at Junction Drift (Q8L011). This gauge had recently been re-instated and as such calibrated flow data was not available at the time of this calibration. Future calibration will be done during the operation of this model which will take this gauging station into account. This additional information will be used during the verification to re-calibrate reach C and D as well as reach F from De Mistkraal Dam (Q8R001) to Junction Drift (Q8L011).

The assumed river channel width of 30 m and depth of 2 m to 2.5 m was taken from the combination of selected cross sectional surveys, observations and river widths measured from 1:10,000 maps from the Department of Land Affairs. The typical section is shown in Figure 5.5-1. The results of the calibration are shown in Figures 5.5-2 and 5.5-3.

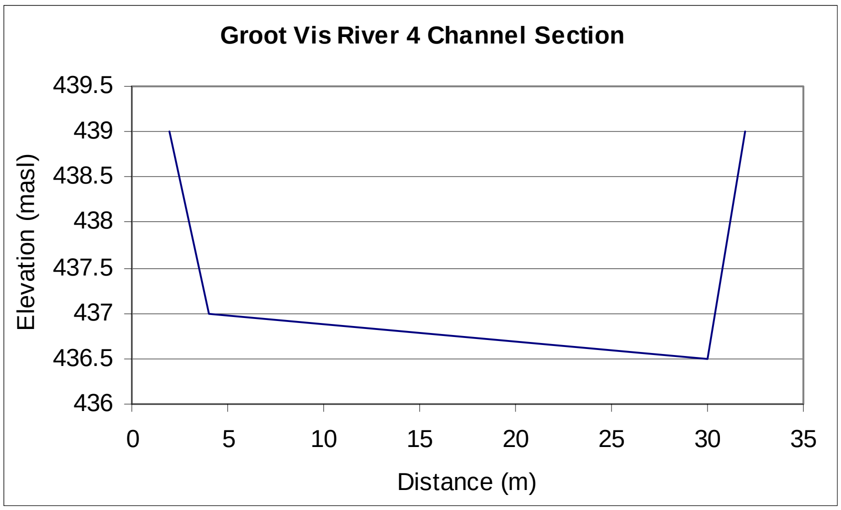



**Figure 5.5-1: Assumed River Section for Groot Vis River 4 Reach D**

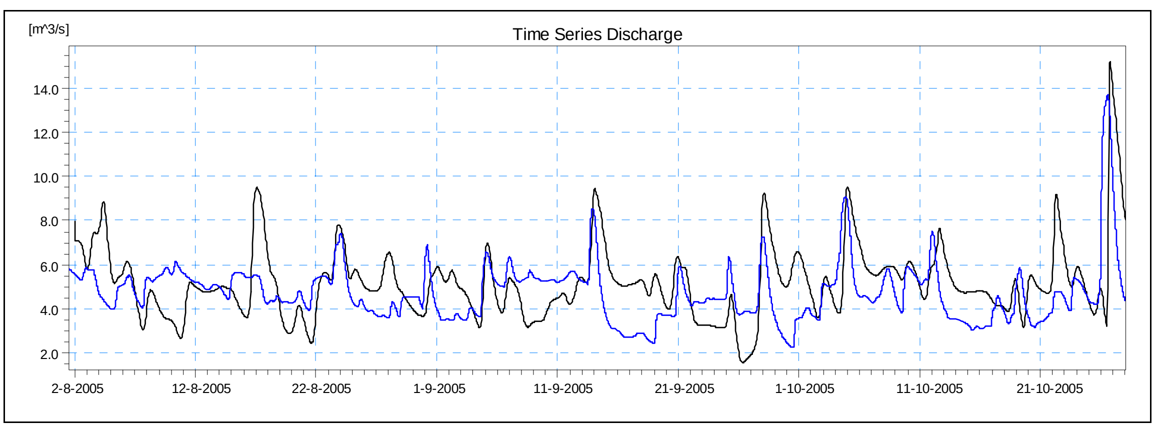



**Figure 5.5-2: Plot of Simulated (black) vs. Observed (blue) Flow at Piggott's Bridge (Q9H012)**

Figure 5.5-2 plots the simulated flows against those observed during the calibration period. It is clear from this plot that there is little correlation. With the lines crossing over each other so frequently, it is difficult to determine any sort of pattern and thus the calibration is not as good as for reaches A or B. Table 5.5-1 gives the results of the statistical analysis between the simulated and observed flow for reach D at Piggott's Bridge. There are many problems in the results with large maximum positive and negative differences, volume error and peak error.

**Table 5.5-1: Statistical Results for the Initial Calibration of reach D**

| **Piggott's Bridge Q9H012** | | |
|---|---|---|
| Correlation coefficient $R^2$ | 0.617 | |
| Max. positive difference | 27.571 | $m^3/s$ |
| Max. negative difference | -10.659 | $m^3/s$ |
| Volume observed | 40406078.17 | $m^3$ |
| Volume modelled | 46131886.72 | $m^3$ |
| Volume error | 14.171 | % |
| Peak observed value | 21.163 | $m^3/s$ |
| Peak modelled value | 31.441 | $m^3/s$ |
| Peak error | 48.568 | % |

Figure 5.5-3 shows the error distribution and correlation for reach D. As the majority of the flow is between 3 to 6 $m^3/s$, it is possible to see that the error distribution for this range of flows is between 50% to -20%. The correlation of 0.617 as given in Table 5.5-1 is confirmation of the poor calibration. Once further data has been obtained, better results are expected.

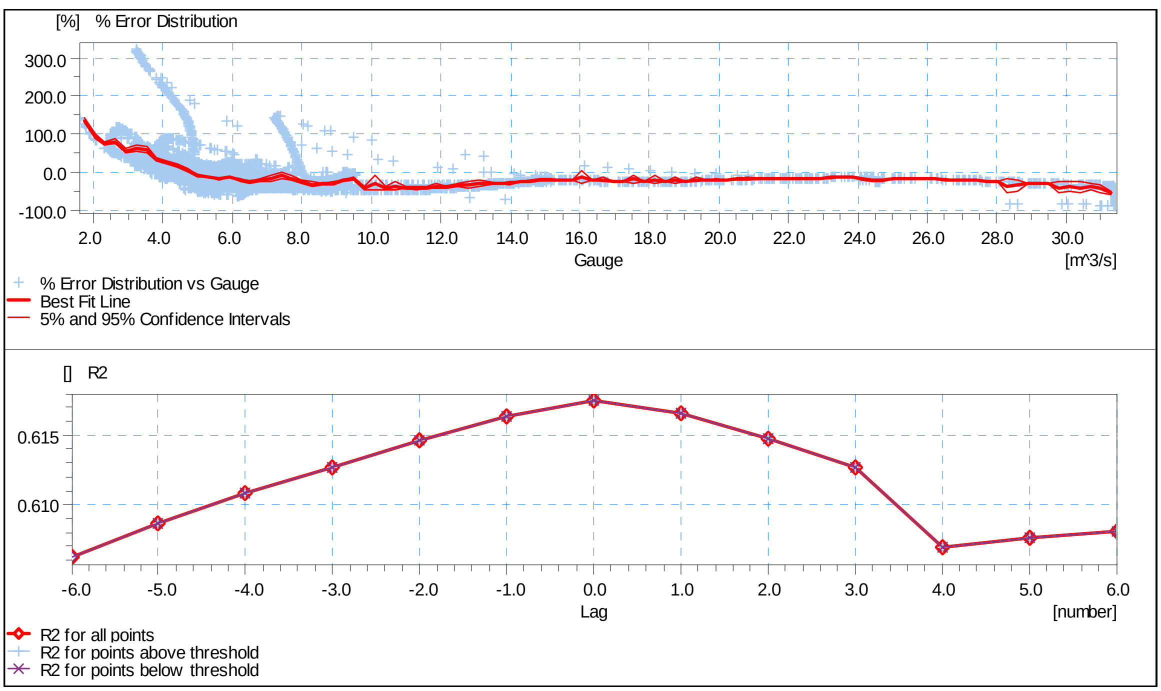



**Figure 5.5-3: Statistical Results for Reach D Showing the Error Distribution between Simulated and Observed Data and the Correlation with a Variation in Time Lag Forwards and Backwards**

## 5.6 Initial Calibration of Reach E

The 45 km canal from Elandsdrift (Q5H006) diverts water from the Groot Vis River to the Little Fish River. Part of this diversion is the 13.1 km Cookhouse Tunnel from chainage 19 km to 31 km. The reach is gauged at Parshall 2 (Q8H007) before the canal drops down 51.5 m in elevation over 1 km to the Little Fish River as shown in Figure 5.6-1. This steep decent can cause instability in the calculations and as such a control structure had been placed in the model at this point to aid the computation. This control structure discharged the same flow into the Little Fish River as that which is flowing just upstream of the drop in elevation, thus the instability through the drop is removed from the calculations. This has since been removed and the weir at Q8H007 is the last structure or node in the model before the Little Fish Canal joins the Little Fish River as recommended by DHI.

**Figure 5.6-1: View from the Top of the Drop from the End of the Little Fish Canal to the Little Fish River**

The canal cross sections were surveyed by the DWAF survey department and the typical cross section is shown in Figure 5.6-2. As the sides of the canal are made from concrete which has a layer of algae growth, the Manning's n value was determined as 0.02. The calibration was extremely accurate with a $R^2$ value of 0.993.

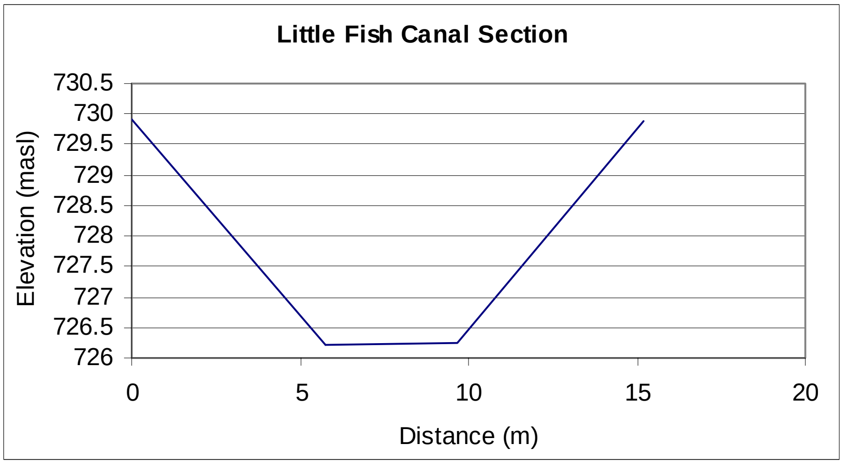



**Figure 5.6-2: Typical Canal Cross Section of the Little Fish Canal**

The discharge from Elandsdrift Dam into the canal was increased by 12% as there was a discrepancy with the observed flow at Parshall 2 which was showing that there was more water passing through Parshall 2 than through the gauging station at Elandsdrift Dam. The decision was taken to factor up the flow at Elandsdrift as this water was required at De Mistkraal to supply the demand from this dam. This flow factor of 12% was optimised during the calibration process.

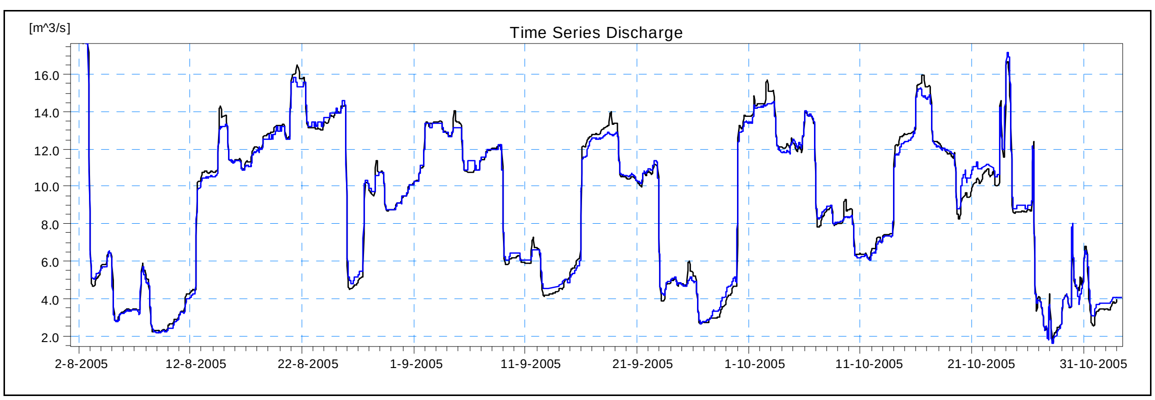



**Figure 5.6-3: Plot of Simulated (black) vs. Observed (blue) Flow at Parshall 2 (Q8H007)**

Figure 5.6-3 shows the strong relation between the simulated and observed values at Parshall 2 (Q8H007). This is after the flow factor has been applied. Table 5.6-1 shows the results of the statistical analysis between the simulated flow and the observed flow before the flow factor was applied. The volume error is given as 12% and when this was applied to the discharge into the canal, the results were more positive.

**Table 5.6-1: Results of Early Calibration Run Showing Volume Error Between Simulated and Observed flow**

| Result | Value | |
|---|---|---|
| Correlation coefficient $R^2$ | 0.99 | |
| Volume observed | 104619741.4 | $m^3$ |
| Volume simulated | 92021667.19 | $m^3$ |
| Volume error | -12.042 | % |
| Peak observed value | 24.022 | $m^3/s$ |
| Peak modelled value | 22.357 | $m^3/s$ |
| Peak error | -6.93 | % |

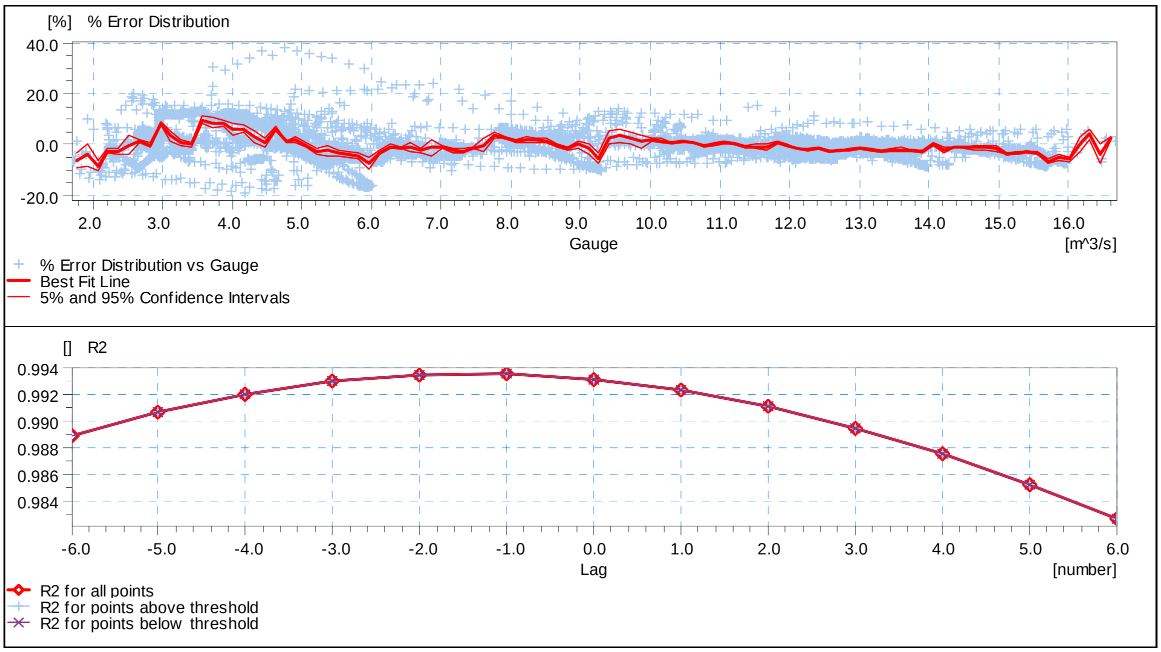



**Figure 5.6-4: Statistical Results for Reach E Showing the Error Distribution between Simulated and Observed Data and the Correlation with a Variation in Time Lag Forwards and Backwards**

Figure 5.6-4 shows the results of the statistical analysis which gives the error distribution as between -10% to 10%. Table 5.6-2 gives the results of the statistical analysis between simulated and observed values for reach E. The quality of the calibration is shown by the small values for maximum positive and negative difference, volume error and peak error.

**Table 5.6-2: Statistical Results for the Initial Calibration of reach E**

| **Parshall 2 Q8H007** | | |
|---|---|---|
| Correlation coefficient $R^2$ | 0.993 | |
| Max. positive difference | 1.308 | $m^3/s$ |
| Max. negative difference | -1.839 | $m^3/s$ |
| Volume observed | 70823624.06 | $m^3$ |
| Volume modelled | 70970900.57 | $m^3$ |
| Volume error | 0.208 | % |
| Peak observed value | 17.202 | $m^3/s$ |
| Peak modelled value | 16.683 | $m^3/s$ |
| Peak error | -3.019 | % |

## 5.7 Initial Calibration of Reach G

From De Mistkraal Dam the water is diverted to the Skoenmakers Canal (Q8H013) to the Skoenmakers River which is a tributary of the Volker's River which flows into the Sundays River at Darlington Dam. The Skoenmakers Canal is 28 km long and is gauged at Parshall 3 (N2L001) before it drops 22 m in elevation into the Skoenmakers River. Again there is a possibility of some instability in the calculations at this point but these have been solved with help from DHI.

The canal was also surveyed and a typical section is shown in Figure 5.7-1. The flow was well calibrated due to the short reach and constant section and slope. The Manning's n value found from the calibration is similar to that found for the Little Fish Canal and is 0.022.

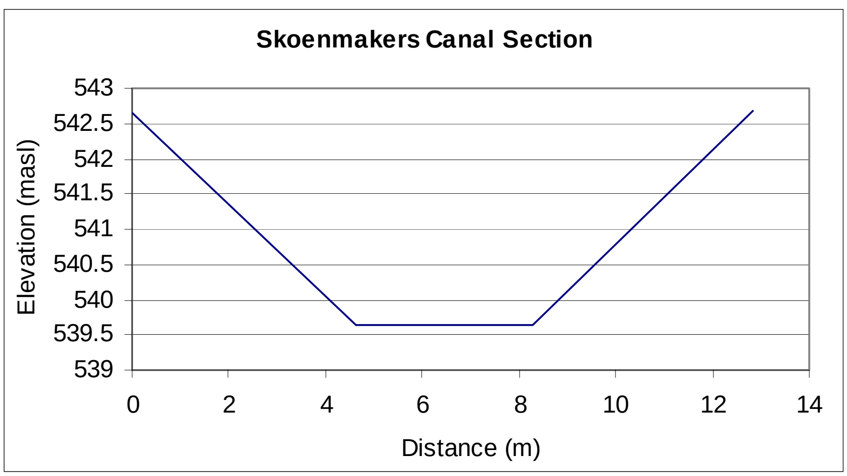



**Figure 5.7-1Typical Canal Cross Section of the Skoenmakers Canal**

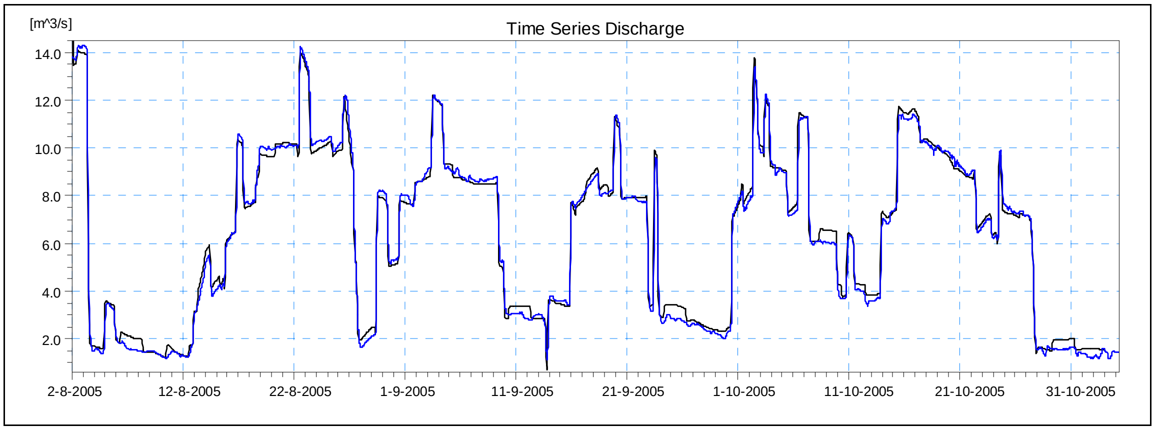



**Figure 5.7-2: Plot of Simulated (black) vs. Observed (blue) Flow at Parshall 3 (N2L001)**

Figure 5.7-2 shows how well the simulated flow compares to the observed flow at Parshall 3. There was a discrepancy between the volume observed at Parshall 3 and that simulated from the discharge given into the canal and as such a flow factor of 0.95 has been required for the flow into the canal at De Mistkraal Dam.

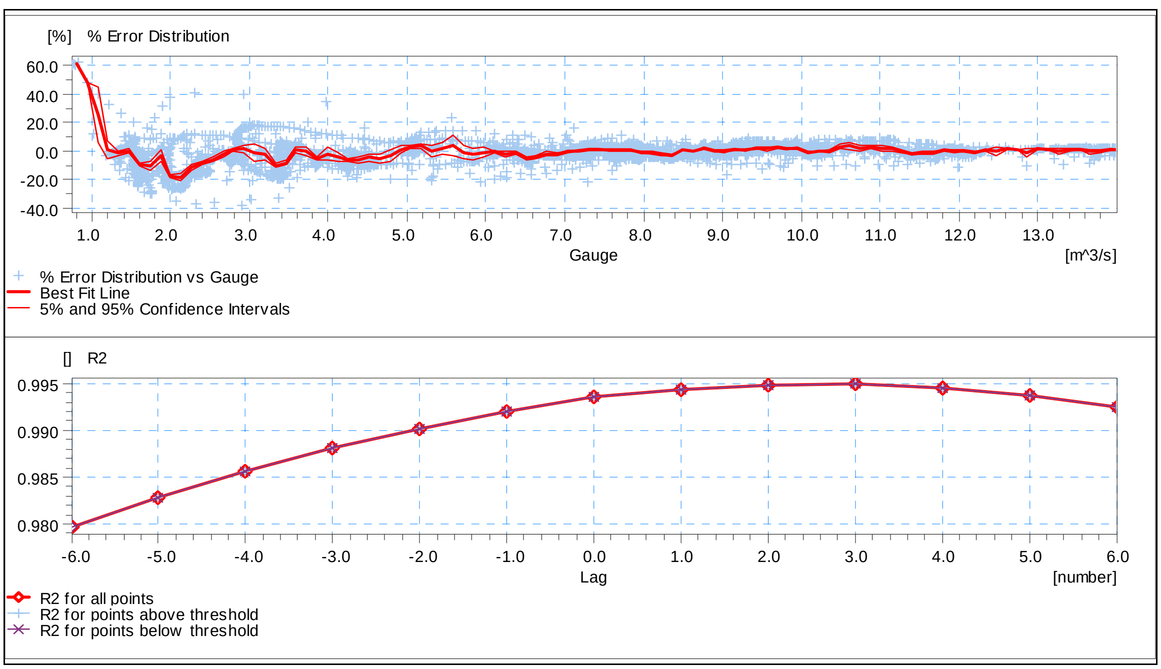



**Figure 5.7-3: Statistical Results for Reach G Showing the Error Distribution between Simulated and Observed Data and the Correlation with a Variation in Time Lag Forwards and Backwards**

Figure 5.7-3 shows how the error distribution is between 10% to -20% for small flows and between 10% and -10% for flows over 3 $m^3/s$. The slightly poorer results for flows less than 3

$m^3/s$ can be attributed to the calibration of the gauging station or un-gauged abstractions as the simulated flow is more often greater than the observed flow during these conditions. Table 5.7-1 gives the results for the statistical analysis between the simulated and observed flow for reach G at Parshall 3. Again the good calibration is shown by small values for the maximum difference and error terms.

**Table 5.7-1: Statistical Results for the Initial Calibration of reach G**

| **Parshall 3 N2L001** | | |
|---|---|---|
| Correlation coefficient $R^2$ | 0.993 | |
| Max. positive difference | 1.563 | $m^3/s$ |
| Max. negative difference | -1.378 | $m^3/s$ |
| Volume observed | 49392688.95 | $m^3$ |
| Volume modelled | 49904250.2 | $m^3$ |
| Volume error | 1.036 | % |
| Peak observed value | 14.293 | $m^3/s$ |
| Peak modelled value | 13.999 | $m^3/s$ |
| Peak error | -2.057 | % |

## 5.8 Calibration of Flood Events

The initial calibration period was concentrated around the dry period from August to November 2005. As more flow data became available during 2006, the flood events during the period from November 2005 to May 2006 were included into the calibration. The initial results showed that the mass balance at the dams was not being accurately modelled. As only a few tributaries are being gauged an area flow factor was required to take into account all the un-gauged tributaries contributing to the flow into the dams. Table 5.8-1 shows the area factor used in the model for each flow gauging weir.

Figure 5.8-1 is a schematic depicting a catchment area A with sub catchments B and C. To find the area factor of catchment B you find the square root of the ratio of the total area (A) less that other sub catchments (C) to the area of the catchment you want (B). Equation 5.8-1 shows the equation for the area factor for catchment B.

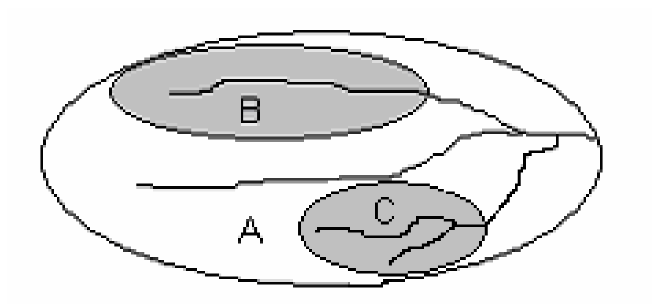



**Figure 5.8-1:Schematic for the calculation of the area factor.**

The area factor is calculated as follows (GR Basson):

$$A_f = \sqrt{\frac{A-C}{B}} \qquad \textbf{Equation 5.8-1}$$

Where:

| | | |
|---|---|---|
| $A_f$ | : | Area Factor for the flow gauging station for catchment B |
| $A$ | : | The Total Area for that catchment |
| $B$ | : | The area for catchment B (Part of catchment A) |
| $C$ | : | The area for catchment C (Part of catchment A) |

The area factor calculated from Equation 5.8-1 for the various tributary flow gauging stations are used as an indication and as such provide information to the modeller of which tributaries have a greater effect on the total runoff flow affecting a catchment. The final values used in the model were refined through calibration and are shown in the last column of Table 5.8-1.

**Table 5.8-1: Area Factors for Flow Gauging Stations.**

| Weir Number | Weir Name | River Name | Catchment Area (km$^2$) | Area Factor for the Catchment ($A_f$) | Factor Used in the Model |
|---|---|---|---|---|---|
| **Sundays River** | | | | | |
| N2H007 | De Draai | Sundays River | 13428 | 1.090 | 2.5 |
| N2H008 | Riet River | Riet River | 341 | 2.897 | 1.1 |
| N2H009 | Volker's River | Volker's River | 536 | 2.388 | 1 |
| N2H010 | Darlington Dam Release | Sundays River | 16826 | 1.072 | 1 |
| **Brak River** | | | | | |
| Q1H012 | Jan Blaauwskop | Teebus River | 1567 | 1.095 | 1 |
| Q1H022 | Grassridge Dam Release | Groot Brak River | 4325 | 1.036 | 0.9 |
| **Upper Fish River** | | | | | |
| Q1H013 | Klein Brak | Klein Brak River | 2445 | 1.836 | 1 |
| Q2H002 | Great Fish River | Groot Vis River | 1713 | 2.094 | 1.7 |
| Q3H004 | Pauls River | Pauls River | 872 | 2.766 | 1.25 |
| Q3H005 | Waaikraal | Groot Vis River | 10830 | 1.239 | 1 |
| **Middle Fish River** | | | | | |
| Q3H005 | Waaikraal | Groot Vis River | 10830 | 1.058 | 1 |
| Q4H013 | Tarka River | Tarka River | 4742 | 1.128 | 1.1 |
| Q5L002 | Elandsdrift Dam Release | Groot Vis River | 16864 | 1.038 | 1 |
| **Lower Fish River** | | | | | |
| Q6H003 | Baviaans River | Baviaans River | 814 | 2.198 | 1.1 |
| Q7H005 | Sheldon | Groot Vis River | 19134 | 1.078 | 1 |
| Q9H012 | Piggott's Bridge | Groot Vis River | 23067 | 1.065 | 1 |
| **Little Fish River** | | | | | |
| Q8H008 | Little Fish River | Little Fish River | 1512 | 1.113 | 2.8 |
| Q8HR001 | De Mistkraal Dam Release | Little Fish River | 1873 | 1.092 | 1 |

## 5.9 Calibration of Return Flows

During the initial dry period calibration, the water levels in the 4 dams were simulated and plotted against the observed data to determine if the mass balance was correct and determine the return flows from irrigation abstractions without additional rainfall runoff and groundwater return flow. Estimates of return flow were added based on abstractions by both a mass balance and a crop water balance method (W. Kamish, S Maharaj). These are given in Table 5.9-1.

**Table 5.9-1: Results of Crop Water Balance to Estimate Return Flows per Reach (W. Kamish, S Maharaj)**

| Reach No. | From | To | Total Volume per annum ($Mm^3$) | Average monthly volume ($1000m^3$) | Average return flow rate ($m^3/s$) | Return flow as % of reach abstractions (%) |
|---|---|---|---|---|---|---|
| **1** | Teebus River | Grassridge Dam | 2.7 | 333 | 0.12 | 7 |
| **2** | Grassridge Dam | Waaikraal | 20.1 | 2515 | 0.94 | 22 |
| **3** | Waaikraal | Elandsdrift Dam | 24.2 | 3022 | 1.13 | 26 |
| **4** | Elandsdrift Dam | Sheldon | 12.2 | 1531 | 0.57 | 16 |
| **5 & 6** | Sheldon | Hermanuskraal | *No abstractions* | | | |
| **7** | Elandsdrift Dam | Little Fish River | 17.7 | 2214 | 0.83 | 44 |
| **8** | Q8H008 Little Fish Tributary | De Mistkraal Dam | 0.0 | 0 | 0.00 | 0 |
| **9** | De Mistkraal Dam | Groot Vis River | 0.0 | 0 | 0.00 | 0 |
| **10** | Skoenmakers Canal | Darlington Dam | 7.5 | 941 | 0.35 | 71 |
| **11** | Darlington Dam | Korhaansdrift | *No abstractions* | | | |

These values were used as guidelines for estimation of return flows. The Mike 11 model was used to calibrate these return flows by comparing the simulated water level in the dams as well as flow rates at gauging stations to determine what changes, if any, were required.

## 5.10 Final Calibration of the Mike 11 Model

Once reliable data had been collected for the model covering the period from July 2005 to the middle of May 2006, the model was re-calibrated. There had also been significant changes to the model during the period from the first dry period calibration and the final calibration. These included the removal of some of the minor weir structures, the removal of the short tributary inflow reaches which were replaced with point sources and the addition of the spillways stage – flow relationships for Darlington Dam and Elandsdrift Dam. The spillway for Elandsdrift was calculated from observations of the dam as no data on the stage – flow relationship over the top of the Fish Belly Flaps was available from DWAF. The spillway was assumed to be 30 m long with a standard free overflow broad created weir equation for the relationship between stage and discharge. This simple assumption is sufficient for the current model and may be updated with a full stage – flow relationship when one becomes available.

These changes amongst other small improvements to the model resulted in the calculation time step being increased from 2 minutes to 5 minutes for the Hydrodynamic calculations and from 30 seconds to 3 minutes for the full model including the AD module for Salinity simulation. Thus a simulation of the period from the 29$^{th}$ July 2005 to the 20$^{th}$ May 2006 takes 41 minutes for a Hydrodynamic simulation and for a full simulation with the AD module included from the 20$^{th}$ July 2005 to the 30$^{th}$ January 2006 takes 58 minutes, a single week takes just over 2 min per simulation.

Table 5.10-1 shows the final calibration values for the various flow factors and return flow values used during the calibration process as well as the Manning's n value for each reach. Constant values are used for the return flow rates. In reality, the return flows are a function of the volume of irrigation water placed per Ha of irrigated land, the distance of the land from the river and the saturation of the soil. As only the volume of irrigation water is known, a reliable function for the return flow due to irrigation could not be created. As such a constant value for the dry period was assumed. During the wet season, there is additional return flow from rainfall runoff; this has been taken into account by the tributary inflow factors. The return flow was calibrated for data during the dry season to reduce the effect of the additional rainfall runoff return flow.

**Table 5.10-1: Final Calibration Values Used for the Mike 11 Model**

| Reach Name: | Return Flow Values ($m^3/s$): | Manning's n Value: |
|---|---|---|
| Teebus | 0.2 | 0.037 |
| Groot Brak1 | 0.7 | 0.040 |
| Groot Brak 2 | 0.70 | 0.040 |
| Groot Vis 1 | 1.0 | 0.033 |
| Groot Vis 2 | 2 | 0.045 |
| Groot Vis 3 | 1 | 0.070 |
| Groot Vis 4 | 0 | 0.050 |
| Little Fish 1 | 0.25 | 0.055 |
| Little Fish Canal | 0 | 0.020 |
| Little Fish 2 | -0.5 (Riparian Losses) | 0.055 |
| Skoenmakers Canal | 0 | 0.025 |
| Skoenmakers | -0.5 (Riparian Losses) | 0.055 |
| Sundays2 | 0 | 0.075 |
| Darlington Spillway | Yes | From DT in Maintenance manual |
| Elandsdrift Spillway (assumed) | Yes | Assumed and reduced from earlier |
| | | |
| Gauge Factors: | | |
| OVIS Tunnel Outlet (Q1H014) | 1 | |
| Grassridge Dam (Q1H022) | 0.9 | |
| Elandsdrift River (Q5R001) | 1 | |
| Elandsdrift Canal (Q5H006) | 1.12 | |
| De Mistkraal River (Q8R001) | 1 | |
| De Mistkraal Canal (Q8H013) | 0.95 | |
| Darlington Dam (N2H010) | 1 | |
| | | |
| Tributary Factors: | | |
| Klein Brak (Q1H013) | 1 | |
| Groot Vis (Q2H002) | 1.7 | |
| Pauls (Q3H004) | 1.25 | |
| Tarka (Q4H013) | 1.1 | |
| Baviaans (Q6H003) | 1.1 | |
| Little Fish (Q8H008) | 2.8 | |
| Sundays (N2H008) | 2.5 | |
| | | |

The final calibration statistics for the model are provided in the following text, for the full set of results for the calibration of the model, see Appendix D.

### 5.10.1 Calibration of Flow Rate at Teebus at Jan Blaauwskop for Reach A

There was little change in the calibration of reach A due to there being no major influences on the flow besides the rainfall runoff from the Teebus catchment. As there was little effect of additional inflow during rainfall periods, there was no alteration to the inflow received at Teebus at Jan Blaauwskop. Figure 5.10-1 shows the simulated flow rate against the observed flow rate at Teebus at Jan Blaauwskop (Q1H012). Simulated flow is shown as a black line while observed flow is shown as a blue line.

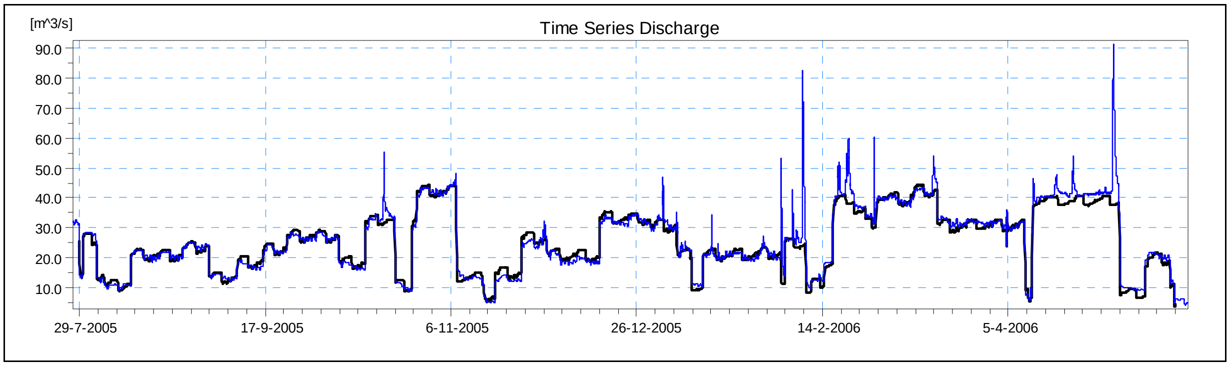



**Figure 5.10-1: Simulated (black) vs Observed (blue) Flow Rate at Teebus at Jan Blaauwskop (Q1H012)**

The observed flow rate at Teebus at Jan Blaauwskop shows many spikes in flow rate which equate to numerous small flood events. These flows are not incorporated into the calibration model. Figure 5.10-2 shows the graphical results of the statistical analysis between the simulated and observed Data.

The first plot shows the variation with time of the difference between the simulated and the observed flow at the flow gauging station. One can see the magnitude of the additional flow due to rainfall runoff observed at Teebus at Jan Blaauwskop. The second plot shows the variation with average flow of the difference between the simulated and observed flow at the gauging weir. As can be seen, the variation is slight up to 45 $m^3/s$. There is no simulated flow greater than 45 $m^3/s$ as this has not been released from the Teebus OVIS Outlet which has a maximum discharge capacity of 57 $m^3/s$. The last plot is the most useful when calibrating the Manning's n value for the reach. This is the time lag correlation plot which shows the correlation between the simulated and observed data at different time lags both forward in time and backwards in time. Thus one can determine if the simulated flow is arriving too soon or too late and adjust the Manning's n value for the reach to correct this. This had been fixed during the initial calibration simulation runs.

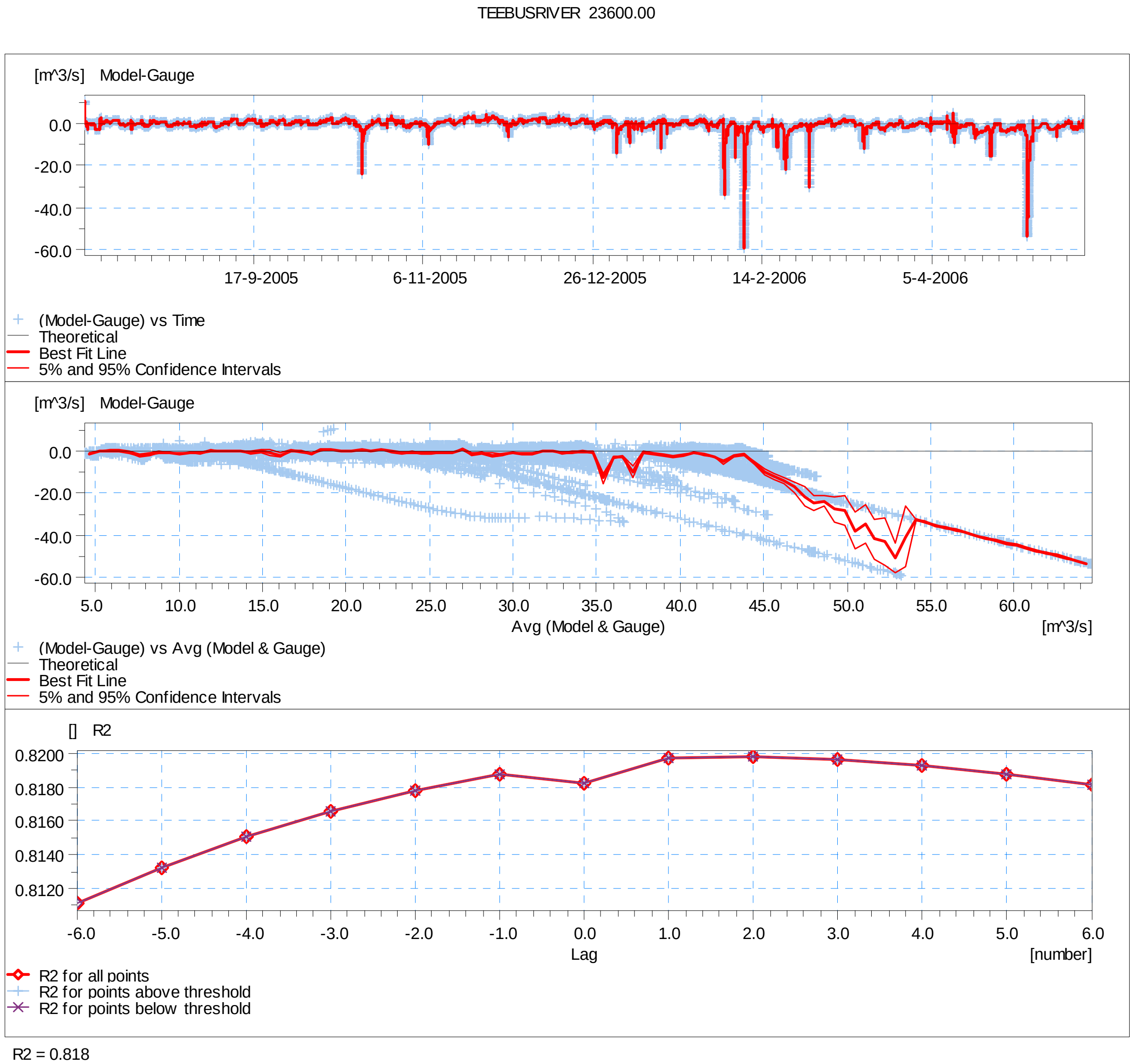



**Figure 5.10-2: Graphical representation for the Statistical results for the data at Teebus at Jan Blaauwskop (Q1H012).**

Table 5.10-2 shows the results of the statistical analysis between the simulated and observed flow rates at Teebus at Jan Blaauwskop (Q1H012).

**Table 5.10-2: Statistical Results for Teebus at Jan Blaauwskop (Q1H012)**

| **Q1H012 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.818 | |
| Max. positive difference | 10.315 | $m^3/s$ |
| Max. negative difference | -58.823 | $m^3/s$ |
| Volume observed | 662662292.8 | $m^3$ |
| Volume modelled | 648795404.4 | $m^3$ |
| Volume error | -2.093 | % |
| Peak observed value | 91.522 | $m^3/s$ |
| Peak modelled value | 44.328 | $m^3/s$ |
| Peak error | -51.565 | % |

The 2% volume difference can be explained by the additional flow during flood events in the Teebus River catchment. With this taken into account the correlation of 0.818 is most satisfactory.

### 5.10.2 Calibration of Water Levels at Grassridge Dam

As one moves down the scheme, the flow from Teebus at Jan Blaauwskop flows into Grassridge Dam. The calibration at the dams was based on the dry period return flow calibration and as such there are increasing discrepancies between the observed water level and the simulated water level in the later part of the simulation period. This is due to the exclusion of additional water which is not modelled as it is not taken into account through tributary area factors or as the irrigators along the reaches have not taken the water from the system that they had requested as the rains would have supplied the necessary water for their crops. This results in additional flow observed in the rivers which can not be simulated. Due to these factors, the decision was taken to under simulate the water level as in the field this water would be provided naturally.

The measured flow discharge from Grassridge Dam was reduced to 90% of the original value for the model as the water level in the dam was seen to be dropping. As there was insufficient cause to increase the inflow to the dam, the gauging weir downstream of the dam was deemed to be over estimating the flow rate. The weir was calibrated with a scale model at the DWAF hydraulic laboratory. As can be seen in Figure 5.10-3, there is still turbulent flow with trapped air bubbles still being released from the flow after it has passed the energy dissipater. This bulking of the flow due to the additional air retained in the flow would cause the stage logger to over estimate the water depth and so the flow rate.

**Figure 5.10-3: The Energy Dissipater and Flow Gauging Station below the Outlet Gates of Grassridge Dam**

Figure 5.10-4 shows the variation in water level at Grassridge Dam for the calibration period.

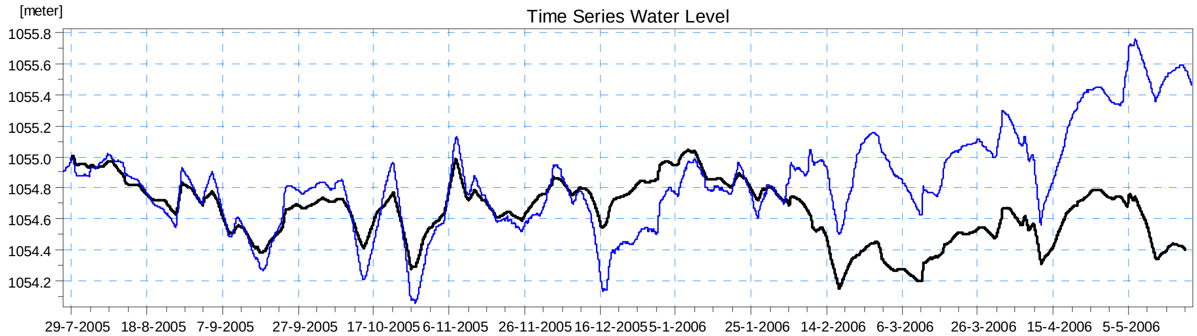



**Figure 5.10-4: Variation of Simulated (black) vs Observed (blue) Water Level at Grassridge Dam (Q1R001)**

One can observe in Figure 5.10-4 how well the water level is calibrated during the first two thirds of the simulation period. From then on the additional un-simulated inflow into the dam causes the two lines to diverge, but the trend in the variations still hold. Figure 5.10-5 shows the variation in water level between the simulated and observed data for the dry period calibration.

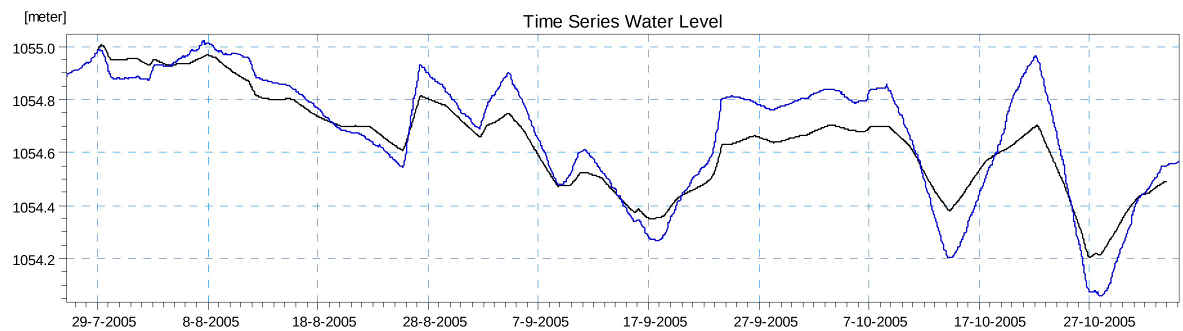



**Figure 5.10-5: Variation of Simulated (black) vs Observed (blue) Water Level at Grassridge Dam during the Dry Period Calibration**

Here one can observe how accurate the correlation between the two sets of data is. Table 5.10-3 shows the statistical data for the dry period calibration.

**Table 5.10-3: Statistical Results for Dry Period Calibration at Grassridge Dam (Q1R001)**

| Q1R001WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.866 | |
| Max. positive difference | 0.188 | meter |
| Max. negative difference | -0.267 | meter |
| Volume observed | 8838260618 | |
| Volume modelled | 8837970134 | |
| Volume error | -0.003 | % |
| Peak observed value | 1055.023 | meter |
| Peak modelled value | 1055.005 | meter |
| Peak error | -0.002 | % |

One can deduce that an increased return flow value for reach A would result in an incorrect correlation during the Dry Period. Table 5.10-4 shows the statistical results between the simulated and observed data for the full calibration period. One can see that the maximum difference between the simulated and observed data is 1.18 m which is at the end of the simulation period.

**Table 5.10-4: Statistical Results for the Full Calibration Period at Grassridge Dam (Q1R001)**

| **Q1R001 WL** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.019 | |
| Max. positive difference | 0.438 | meter |
| Max. negative difference | -1.18 | meter |
| Volume observed | 26884821780 | |
| Volume modelled | 26880076475 | |
| Volume error | -0.018 | % |
| Peak observed value | 1055.76 | meter |
| Peak modelled value | 1055.043 | meter |
| Peak error | -0.068 | % |

### 5.10.3 Calibration of Flow Rate at Waaikraal for Reach B

The 70 km from the outlet of Grassridge Dam to the flow gauging station at Waaikraal Weir (Q3H005) includes three tributaries and accounts for 17% of the total abstraction from the system on 9% of the total system length.

**Figure 5.10-6: Repairs of Waaikraal Weir during July 2005**

The weir at Waaikraal was being re-built / repaired during the site visit to the OFS scheme in July 2005, see Figure 5.10-6. The calibration of the stage – flow relationship has since been uncertain. There is a difference between the flow rate observed at the weir and that simulated by the model. Although there is a difference in the actual flow rate the fluctuations are still able to be modelled and so the Manning's n value can be determined for this reach. Figure 5.10-7 shows the variation of flow rate for the simulated in black and the observed flow rate in blue at Waaikraal for the full calibration period.

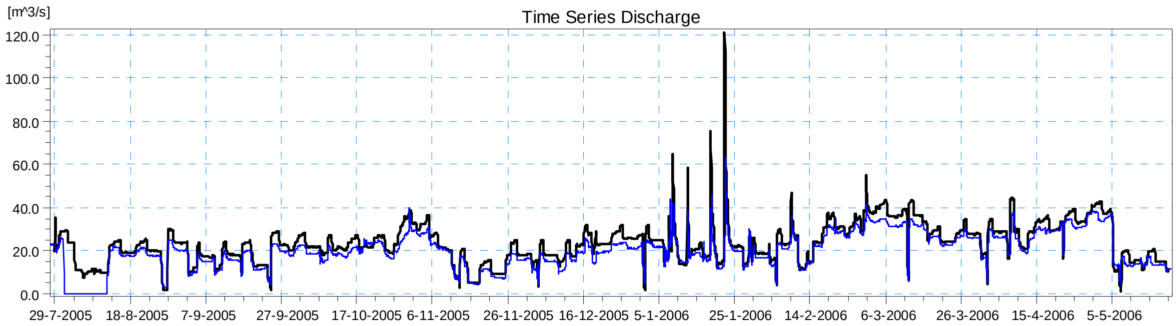



**Figure 5.10-7: Variation of Simulated (black) vs Observed (blue) Flow Rate at Waaikraal Weir (Q3H005)**

As Figure 5.10-7 shows, the simulated flow is greater than the observed flow, one must bear in mind that the flow discharge from Grassridge Dam has already been reduced to 90% of the measured discharge. One can also see that the difference between the simulated and observed data increases for an increased flow rate. This supports the theory that the stage – flow relationship calculated for the new Waaikraal weir is not as accurate as it should be. Note that the tributary flood events have been modelled and have been factored up to be greater than that observed to account for additional inflow not gauged. Figure 5.10-8 shows the graphical representation of the statistical data between the two sets of data.

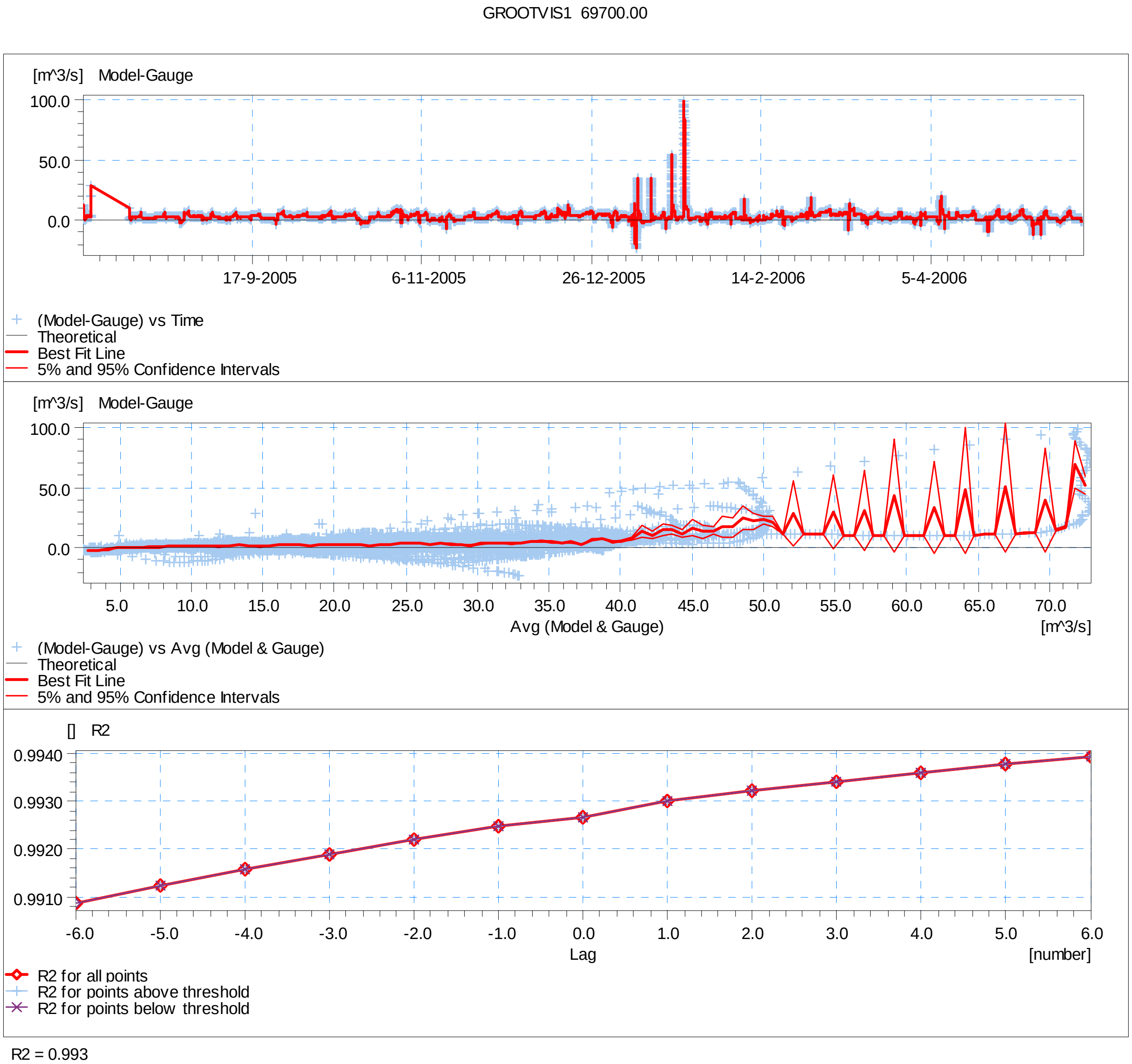



**Figure 5.10-8: Graphical Representation of the Statistical Analysis between the Simulated and Observed Data at Waaikraal Weir (Q3H005)**

There is a very high correlation between the simulated and observed data of 0.993. As the lag correlation plot shows, there could be a greater correlation if the Manning's n value was adjusted but this would skew the model towards the infrequent flood flow events. As such the more stable and common dry period calibration was used for the determination of the Manning's n value. The lag correlation plot is shown in Figure 5.10-9 and shows that for the dry period, the Manning's n value is the most correct. Mike 11 does have the ability to cater for different Manning's n values for different sections of the river channel, but as the modeller did not have accurate data pertaining to the cross sections of the flood plain, it was decided that, the slight error in flow simulated time of travel would not have a severe impact of the operation of the real-time model.

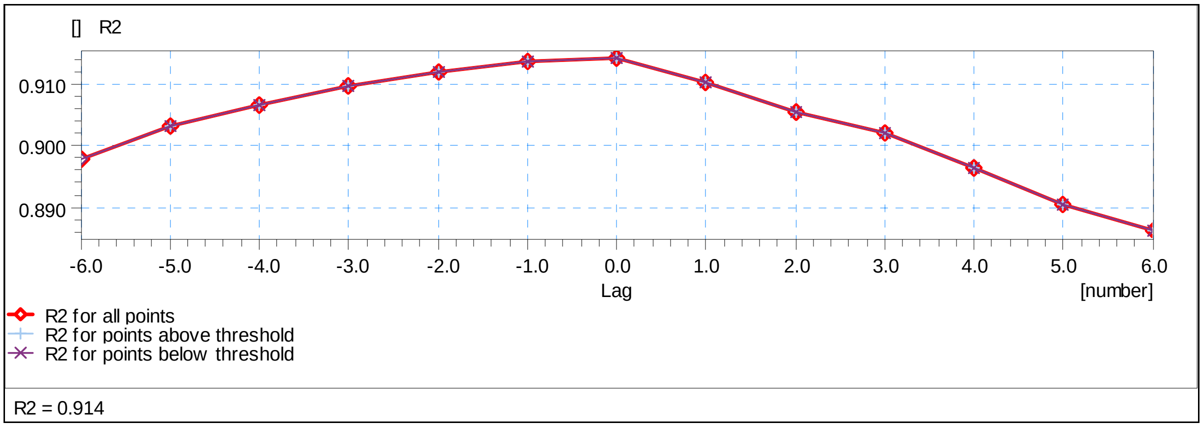



**Figure 5.10-9: Lag - Correlation Plot for the Simulated against Observed Flow Rate Data at Waaikraal Weir for the Dry Period Calibration**

Table 5.10-5 shows the statistical results for the full dry period calibration. The volume error of 17% provides an indication of the level of error in the measurement of the flow rate at Waaikraal Weir.

**Table 5.10-5: Statistical Results for the Full Calibration Period at Waaikraal Weir (Q3H005)**

| **Q3H005 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.993 | |
| Max. positive difference | 98.14 | $m^3/s$ |
| Max. negative difference | -22.566 | $m^3/s$ |
| Volume observed | 511746177.6 | $m^3$ |
| Volume modelled | 602984798.2 | $m^3$ |
| Volume error | 17.829 | % |
| Peak observed value | 63.1 | $m^3/s$ |
| Peak modelled value | 121.044 | $m^3/s$ |
| Peak error | 91.829 | % |

It is unfortunate that there is such a strong difference between the simulated and observed flow rates at Waaikraal Weir (Q3H005) when the flow travel times are so well correlated. Once further work has been conducted to correct the stage – discharge relationship at Waaikraal, a more conclusive result would appear.

### 5.10.4 Calibration of Water Level at Elandsdrift Dam

Elandsdrift Dam is more of a diversion weir than a storage dam with storage capacity of just under 7 million $m^3$ as determined at the last capacity determination in 1994. As discussed in the text, there is no stage – flow relationship for the Fish Belly Flaps on the top of the 4 radial flood gates. These are primarily used to clear debris from the dam, but the water level records that the FSL has been exceeded during the full calibration period and as such these must have come into operation. This can be seen in Figure 5.10-10 which shows the variation of simulated and observed water levels at Elandsdrift Dam for the full calibration period.

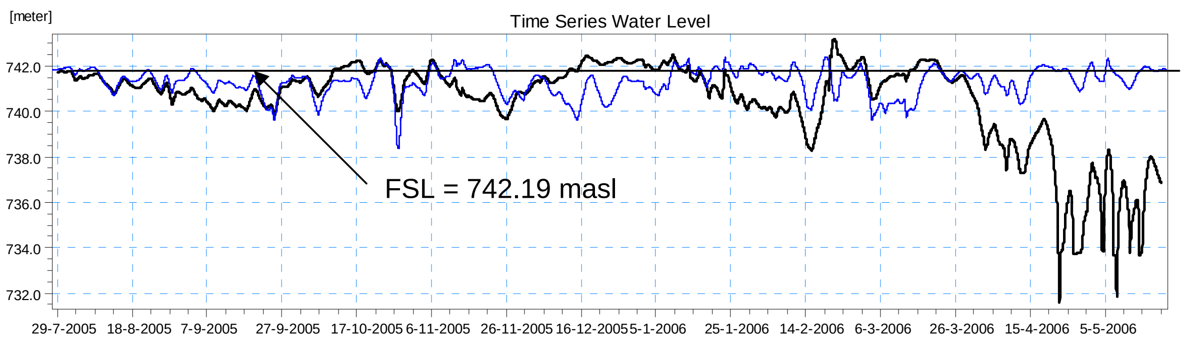



**Figure 5.10-10: Variation between Simulated (black) and Observed (blue) Water Levels for the Full Calibration Period at Elandsdrift Dam (Q5R001)**

There was no flow gauge downstream of Elandsdrift Dam to accurately determine the flow release from the dam to the Groot Vis River. One has been installed at a control section though this was not operational during the calibration period. Thus the only data available for the calibration of the model was the record of what was supposed to have been released to the Groot Vis River. There was a full set of historical data for the flow gauging station on the Little Fish Canal release shown in Figure 5.10-11.

**Figure 5.10-11: Elandsdrift Dam Showing the Flow Gauging Station on the Little Fish Canal as well as the Four Radial Flood Gates**

Figure 5.10-12 shows the effect of not incorporating a free overflow spillway at Elandsdrift Dam. The effect is greatly magnified by the accumulative effect but nevertheless illustrates the fact that there is either an error in the observed flow release to the Groot Vis River or that there are occasional spill events. This data is substantiated by the 12% increase in the measured discharge from the dam to the Little Fish Canal (Q5H006). This flow factor was required due to discrepancies in the volume modelled at the end of the Little Fish Canal at Parshall 2 (Q8H007).

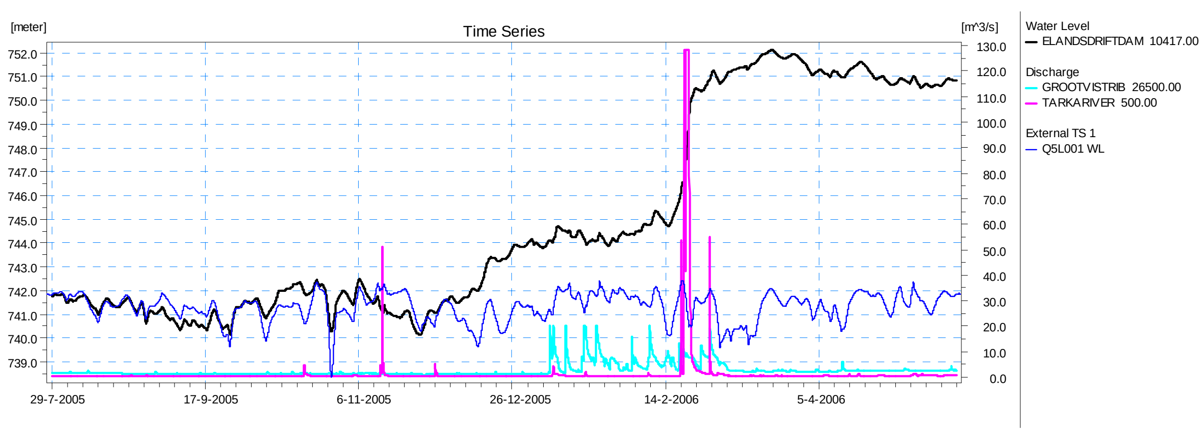



**Figure 5.10-12: Early Calibration Simulation Run Showing the Variation between Simulated (black) and Observed (blue) Water Levels at Elandsdrift Dam as well as the Tributary Inflows above Elandsdrift Dam [Groot Vis (Cyan) and Tarka (Magenta)]**

The inflow to Elandsdrift Dam was increased at little as possible in light of the area factors for the tributary inflows. The factors for the tributaries to Elandsdrift Dam are far less than for those into De Mistkraal or Darlington Dam. This could have something to do with the variation in topography and micro climate in the region. Yet as can be seen in Figure 5.10-13, the variation between the simulated in black and the observed in blue is slight for the dry period calibration. Thus the values for the return flow and tributary base flows are suitable for this flow period.

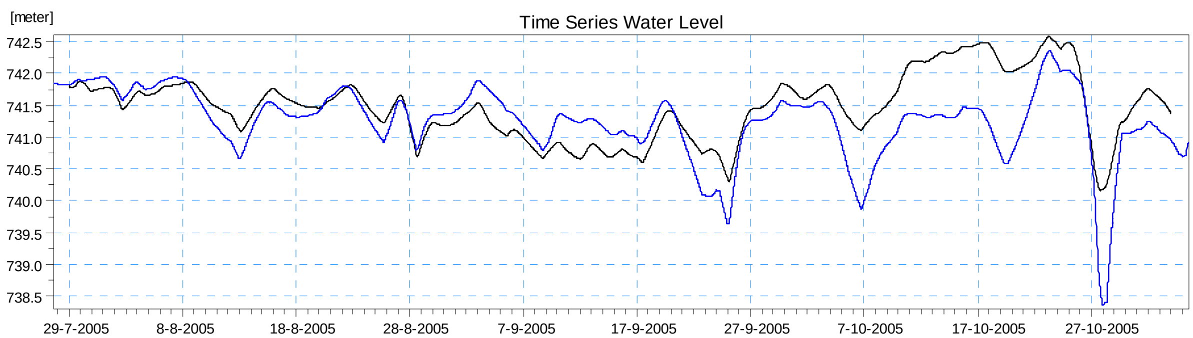



**Figure 5.10-13: Variation of Simulated (black) against Observed (blue) Water Levels during the Dry Period Calibration at Elandsdrift Dam (Q5R001)**

As the dam has such a small storage capacity and there is some doubt over the reliability of the data for the flow release to the river, the correlation of the water levels is not as good as one would have hoped. Many simulation runs were completed to accurately model the water level at this dam as this dam is the fulcrum of the scheme as from here the flow can be diverted to Darlington Dam which has the largest storage capacity of the four dams. This can assist in the efficient operation of the model, reducing the demand for water to be transferred from the Orange River.

Table 5.10-6 shows the statistical results for the calibration of the water level at Elandsdrift Dam. The correlation is very poor though with the many unknowns and un-reliable flow release data for a dam with such a small storage, this is not so surprising. The pleasing result is the volume error term which shows just a 0.106 % error in the volume modelled against that observed.

**Table 5.10-6: Statistical Results for the Full Calibration Period for the Water Level at Elandsdrift Dam (Q5L001)**

| **Q5R001 WL** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.014 | |
| Max. positive difference | 2.689 | meter |
| Max. negative difference | -10.199 | meter |
| Volume observed | 18894533126 | |
| Volume modelled | 18874465176 | |
| Volume error | -0.106 | % |
| Peak observed value | 742.41 | meter |
| Peak modelled value | 743.199 | meter |
| Peak error | 0.106 | % |

The lower water level simulated near the end of the simulation period is a function of the same factors mentioned in 5.10-2. The base flow in the tributaries is greater after the wet season as opposed to that calibrated during the dry period but is far more unstable due to the variations in the volume and seasonality of the rainfall for any given year at any location. This is coupled with the likelihood of irrigators not taking their allocated irrigation water released to them resulting in less water being modelled and thus negatively effecting the calibration of the model. There are plans to re-evaluate the manner in which the irrigators pay for the water that they use to include a penalty for not using the water that they request.

### 5.10.5 Calibration of Flow Rate at Sheldon Weir

Figure 5.10-14 shows the variation between the simulated and observed flow rates at Sheldon Weir (Q7H005). There is a greater correlation (0.709) between the two data sets compared to that found during the dry period calibration (0.629). This is due to the improved results obtained from the calibration of the flood events. The simulated flood events are somewhat greater in peak flow rate than the observed flow rate. This could be due to poor assumptions of the flow release from Elandsdrift Dam including the assumed free overflow created for the Fish Belly Flaps.

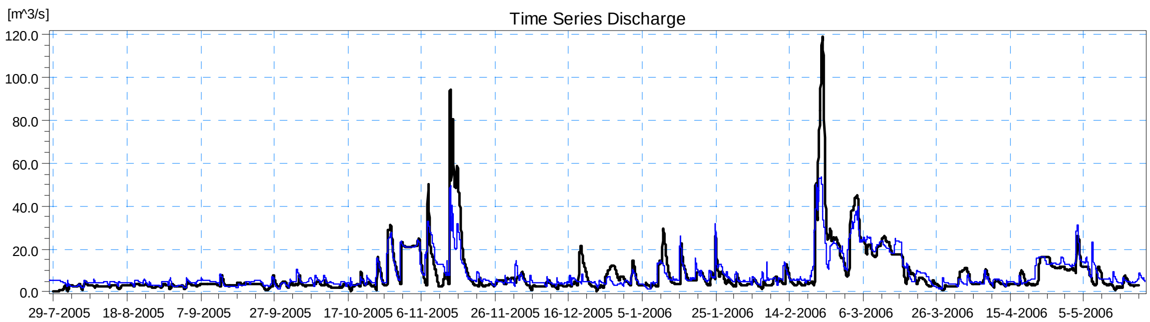



**Figure 5.10-14: Variation between the Simulated (black) and Observed (blue) Flow Rate at Sheldon Weir (Q7H005)**

Table 5.10-7 provides the statistical data for the calibration between the simulated and observed flow rate at Sheldon. Due to the length of this reach, the amount of attenuation and smoothing out of the flow results in a great difficulty in producing accurate correlations.

**Table 5.10-7: Statistical Results for the Full Calibration Period for the Flow Rrate at Sheldon Weir (Q7H005)**

| **Q7H005 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.709 | |
| Max. positive difference | 80.569 | $m^3/s$ |
| Max. negative difference | -25.181 | $m^3/s$ |
| Volume observed | 213991296 | $m^3$ |
| Volume modelled | 207665533 | $m^3$ |
| Volume error | -2.956 | % |
| Peak observed value | 53.569 | $m^3/s$ |
| Peak modelled value | 119.258 | $m^3/s$ |
| Peak error | 122.625 | % |

An old weir at Middleton has been re-established and it is hoped that this additional data will aid in the calibration of this reach. The Middleton Weir is situated 104 km downstream from Elandsdrift Dam which is 42 km upstream from Sheldon Weir. The greater accuracy from the flow gauging station located just downstream of Elandsdrift will also be a great benefit to increasing the accuracy of the calibration.

### 5.10.6 Calibration of Flow Rate at Piggott's Bridge Weir

The calibration at Piggott's Bridge Weir has been improved with the inclusion of flood events and this is shown in Figure 5.10-15. The correlation has been improved from 0.617 to 0.67 as shown in Table 5.10-8. There is some concern over the peak flow rate simulated between February and March of 2006. This peak flow rate can be attributed to the lack of a full flood plain being modelled due to the un-availability of suitable spatial data which would attenuate the flow. A proper digital survey of the river reaches would allow for far greater accuracy in the assumption of the Manning's n value for the left and right banks. The maximum of this flood peak is also considered to be incorrect due to the assumptions made at Elandsdrift Dam 267 km upstream from this flow gauging weir. In future, the data collected from the additional weirs along with increased knowledge of the flow from Elandsdrift Dam will be useful in improving the calibration of the model for the lower Groot Vis River reach from Elandsdrift to Piggott's Bridge and on to Fort Brown Bridge.

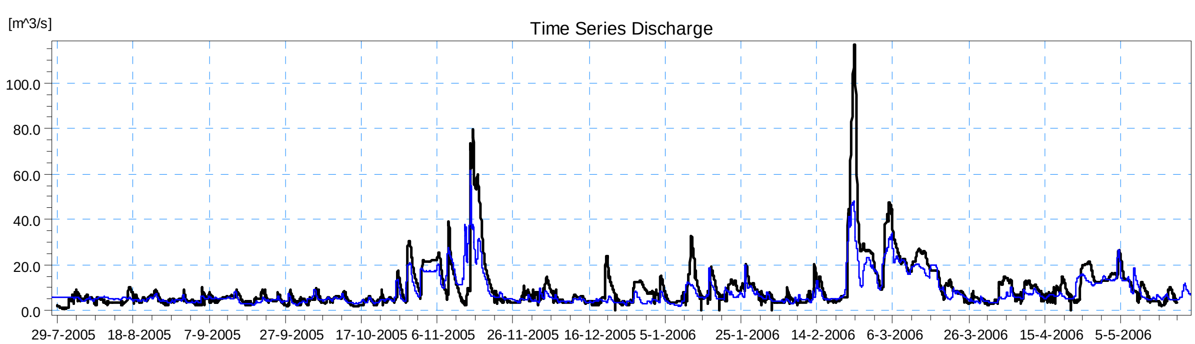



**Figure 5.10-15: Variation between Simulated (black) and Observed (blue) Flow Rate at Piggott's Bridge (Q9H012)**

With the many uncertainties associated with this long and remote reach, the statistical data presented in Table 5.10-8 shows the poor calibration of this reach from Sheldon to Piggott's Bridge.

**Table 5.10-8: Statistical Results for the Full Calibration Period for the Flow Rate at Piggott's Bridge (Q9H012)**

| Q9H012 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.67 | |
| Max. positive difference | 76.337 | $m^3/s$ |
| Max. negative difference | -41.188 | $m^3/s$ |
| Volume observed | 208478242.2 | $m^3$ |
| Volume modelled | 253430349.8 | $m^3$ |
| Volume error | 21.562 | % |
| Peak observed value | 61.587 | $m^3/s$ |
| Peak modelled value | 116.877 | $m^3/s$ |
| Peak error | 89.775 | % |

## 5.10.7 Calibration of Flow Rate at Parshall 2 on the Little Fish Canal

The calibration of the flow rate at the end of the Little Fish Canal (Q8H007) is little different from that for the dry period. This is due to the non existent influence of rainfall or rainfall runoff to the flow rate in the canal. The controlled flow releases from Elandsdrift Dam are operated according to the demand in the Little Fish and Sundays Rivers. As such the calibration of this reach was unchanged. Figure 5.10-16 shows the variation of flow rate between the simulated and observed data, while Table 5.10-9 shows the statistical results for the flow rate at Parshall 2 at the end of the Little Fish Canal.

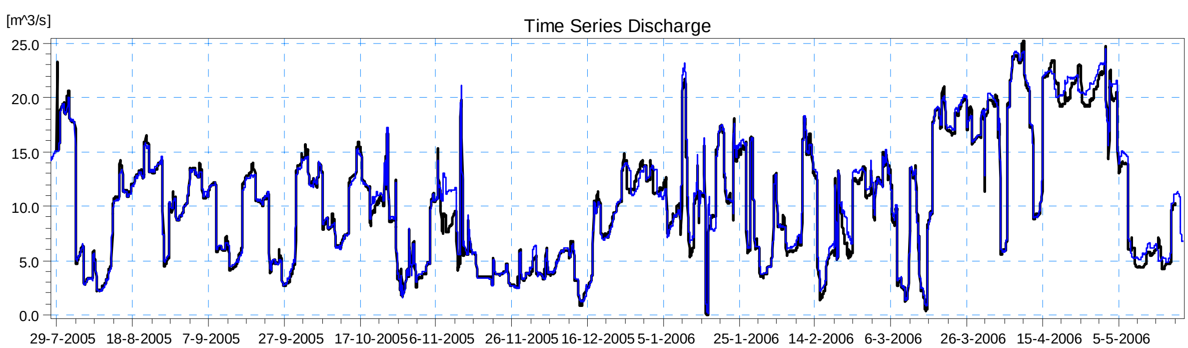



**Figure 5.10-16: Variation between the Simulated (black) and Observed (blue) Flow Rate at Parshall 2 on the Little Fish Canal (Q8H007)**

**Table 5.10-9: Staristical Results for the Full Calibration Period for the Flow Rate at Parshall 2 on the Little Fish Canal (Q8H007)**

| **Q8H007 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.987 | |
| Max. positive difference | 8.144 | $m^3/s$ |
| Max. negative difference | -4.562 | $m^3/s$ |
| Volume observed | 267387847.3 | $m^3$ |
| Volume modelled | 262488679.2 | $m^3$ |
| Volume error | -1.832 | % |
| Peak observed value | 24.469 | $m^3/s$ |
| Peak modelled value | 25.227 | $m^3/s$ |
| Peak error | 3.099 | % |

## 5.10.8 Calibration of the Water Level at De Mistkraal Dam

De Mistkraal Dam is the only dam on the scheme which has a true free overflow spillway crest. The stage – flow relationship for the free overflow spillway was used in the model to release flow to the Little Fish River during periods of high water level in De Mistkraal Dam. As with Elandsdrift Dam, the dam is more of a diversion weir as the storage volume of the dam is just 2.6 million $m^3$ as calculated from the Capacity Determination Study conducted in 2002. Figure 5.10-16 shows the variation between the simulated and observed water levels at De Mistkraal Dam. For the most part the correlation is very good. The Little Fish tributary inflow was factored to account for the additional un-gauged inflow into the dam. This factor was calibrated to 2.8 as opposed to the calculated value of 1.113 as shown in Table 5.8-1. This greater value for the area factor can be attributed to it replacing a greater return flow figure but the return flow was calibrated successfully for the dry period.

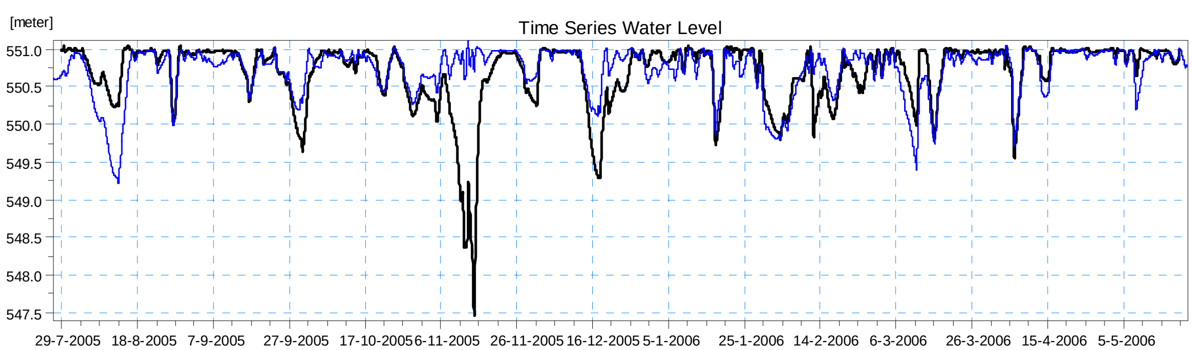



**Figure 5.10-17: Variation between the Simulated (black) and Observed (blue) Water Levels at De Mistkraal Dam (Q8R001)**

Table 5.10-10 shows the statistical results for the full calibration period. The correlation is low thought the volume error and peak error terms show a high degree of accuracy. There is no cause for the simulated water level dropping so far below the observed water level in early November 2005. This could be due to an error in the recording of the flow release records used due to the lack of a flow gauging station downstream of De Mistkraal Dam. This could also be the result of a storm that fell outside of the catchment of the Weir on the Little Fish tributary at Q8H008.

**Table 5.10-10: Statistical Results for the Full Calibration Perion for the Water Level at De Mistkraal Dam (Q8R001)**

| **Q8R001 WL** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.162 | |
| Max. positive difference | 1.029 | meter |
| Max. negative difference | -3.244 | meter |
| Volume observed | 14036846911 | |
| Volume modelled | 14036002738 | |
| Volume error | -0.006 | % |
| Peak observed value | 551.114 | meter |
| Peak modelled value | 551.048 | meter |
| Peak error | -0.012 | % |

## 5.10.9 Calibration of the Flow Rate at Junctiondrift Weir

Junctiondrift Weir is situated on the Little Fish River just upstream of the rivers' confluence with the Groot Vis River. The weir is approximately 55 km downstream of De Mistkraal Dam and has recently been re-calibrated and had the instrumentation re-installed as part of the upgrading of the flow gauging station on the OFS Scheme. Data for this station was not available during the dry period calibration but has become available for the full calibration period. Figure 5.10-18 shows the variation between the simulated and observed flow rates at Junctiondrift Weir.

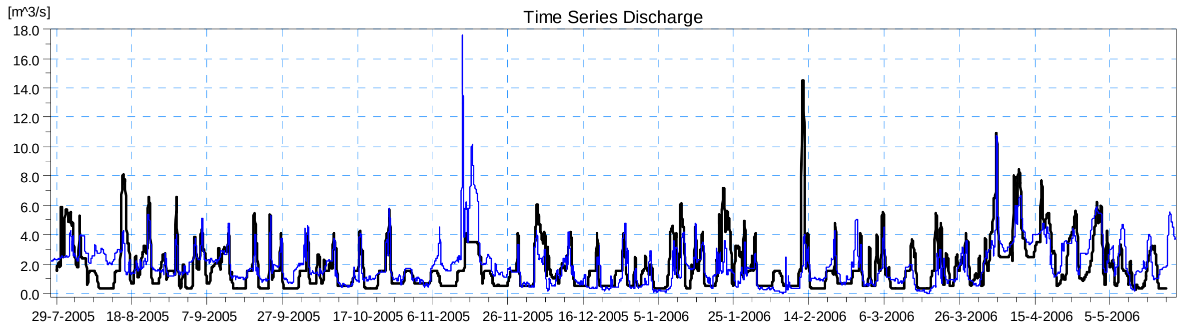



**Figure 5.10-18: Variation between Simulated (black) and Observed (blue) Flow Rate at Junctiondrift Weir (Q8L011)**

The large difference between the simulated and observed flow rates in November 2005 correlate with the large difference in simulated water levels against observed water levels during the same period. This points to a large flood event occurring during this period that was not included in any of the tributaries. Regardless the remaining months exhibit a good correlation for flow travel times and flow rates.

## 5.10.10 Calibration of Flow Rate at Parshall 3

As with the calibration of Parshall 2, there is little difference between the calibrations for the dry period as opposed to the full period as there is no variation with respect to rainfall runoff or return flow. Figure 5.10-19 shows the variation between the simulated and observed flow rate at Parshall 3 on the Skoenmakers Canal (N2L001).

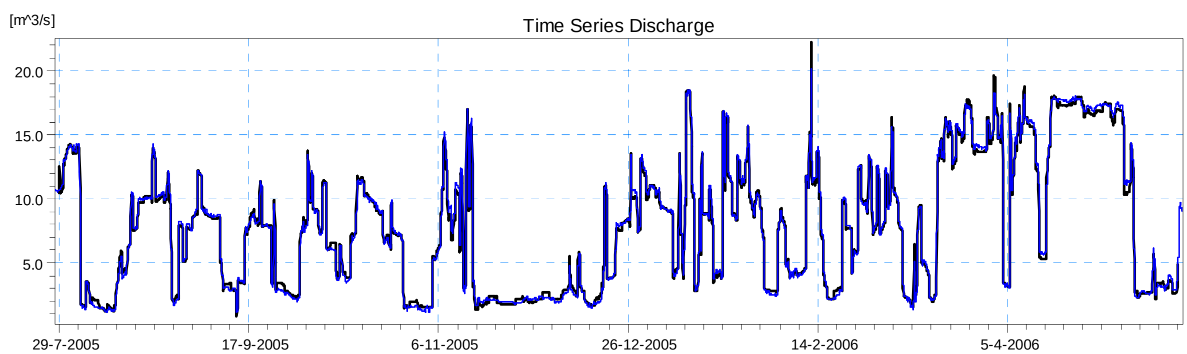



**Figure 5.10-19: Variation between the Simulated (black) and Observed (blue) Flow Rate at Parshall 3 (N2L001)**

Table 5.10-11 shows the high correlation between the simulated and observed flow rates at Parshall 3. The flow factor found during the dry period calibration was found to sill be accurate with just a 0.521% volume error.

**Table 5.10-11:Statistical Results for the Full Calibration Period for the Flow Rate at Parshall 3 (N2L001)**

| **N2L001 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.996 | |
| Max. positive difference | 2.874 | $m^3/s$ |
| Max. negative difference | -1.608 | $m^3/s$ |
| Volume observed | 200514374.7 | $m^3$ |
| Volume modelled | 199469227.1 | $m^3$ |
| Volume error | -0.521 | % |
| Peak observed value | 20.168 | $m^3/s$ |
| Peak modelled value | 22.255 | $m^3/s$ |
| Peak error | 10.349 | % |

## 5.10.11 Calibration of Water Level at Darlington Dam

The Sundays River was factored by 2.5 as shown in Table 5.8-1. This change was effected to improve the wet season calibration of the water level of Darlington Dam. The addition of the Auxiliary spillway to the model was also an improvement to the model that shows a good correlation between the simulated and observed water level as shown in Figure 5.10-20. An effort was made to more closely match the high water level during November 2005 but the lack of all the tributary inflow data restricted the ability for better results to be obtained. As such the actual difference in water level is slight. The crest level of the Auxiliary spillway is displayed in the figure. One can see how often the dam is spilling under the current operation resulting in a loss to the system.

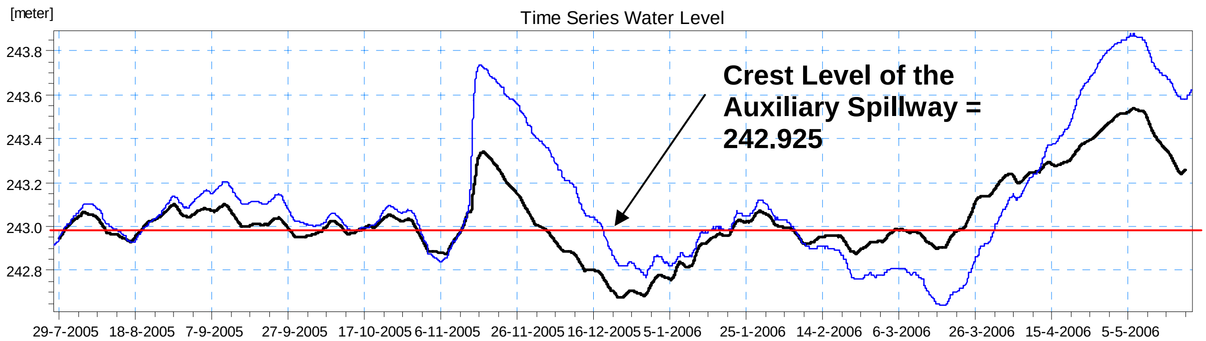



**Figure 5.10-20: Variation between Simulated (black) and Observed (blue) Water Levels at Darlington Dam (N2R001)**

The location of the Auxiliary and Main Spillways are shown in Figure 5.10-21 along with the location of the sleeve valves. The flow release from the sleeve valves is recorded at a flow gauging station located downstream of Darlington Dam. The flow rate from the Auxiliary Spillway whose gate is left open due to dam safety considerations is not measured. The release from the auxiliary spillway is modelled from the stage – flow relationships provided in the maintenance and operation manual (DWAF, 2000).

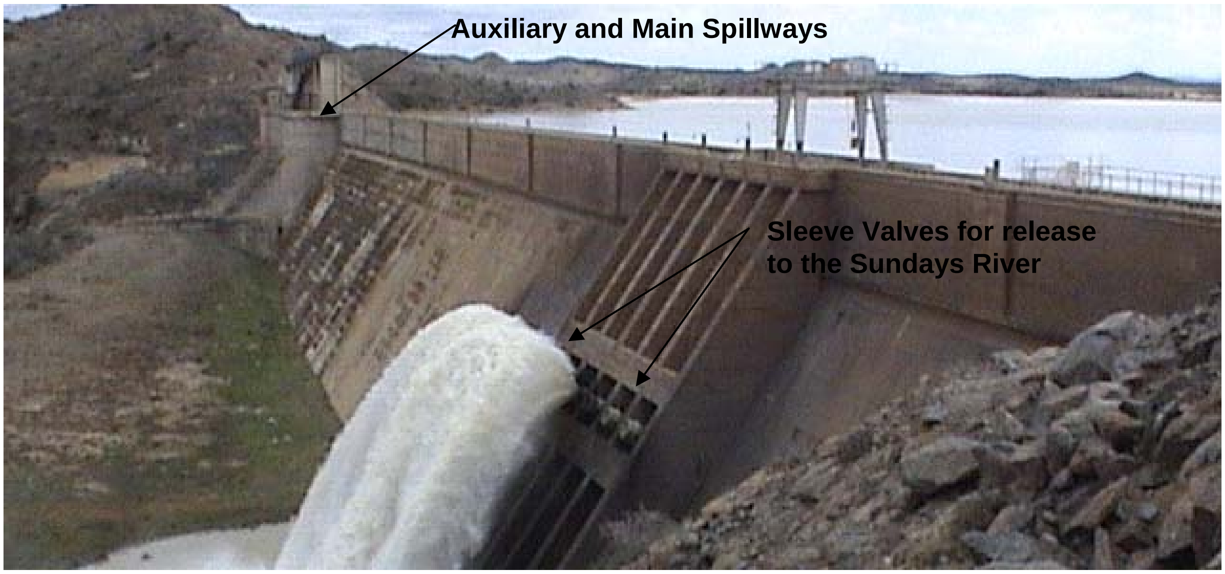



**Figure 5.10-21: Darlington Dam showing the Location of the Outlet Works**

The correlation between the simulated and observed water level is acceptable compared to the low results for the other dams. The maximum and minimum differences are very small and point towards an accurate calibration.

**Table 5.10-12:Statistical Results for the Full Calibration Period of the Water Leval at Darlington Dam (N2R001)**

| **N2R001 WL** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.682 | |
| Max. positive difference | 0.28 | meter |
| Max. negative difference | -0.449 | meter |
| Volume observed | 6193899602 | |
| Volume modelled | 6192123542 | |
| Volume error | -0.029 | % |
| Peak observed value | 243.88 | meter |
| Peak modelled value | 243.538 | meter |
| Peak error | -0.14 | % |

### 5.10.12 Calibration of Flow Rate at Korhaansdrift Weir and Abstraction

The calibration at Korhaansdrift is not possible as the measured value available is for the abstraction from the Sundays River as opposed to the flow over the weir. The weir at Korhaansdrift is 150 m long and as such highly inaccurate for small changes in flow level. As such the data available for the discharge over the weir is un-reliable. The current operation of the weir structure at Korhaansdrift is aimed at diverting all flow from the Sundays River into the Canal at Korhaansdrift which transfers this water to the Kirkwood Citrus Farmers and Skeepersvlakte dam which is an off-channel storage dam for domestic water use for Port Elizabeth on the South East Coast of South Africa. The model is taken up to the diversion weir at Korhaansdrift as no detailed data was available for the demand or abstractions along the Canal as Darlington Dam and the Korhaansdrift weir and canal are operated by a the Lower Sundays Water Use Association.

## 5.11 Concluding Summery of the Model Calibration

The model has been afflicted by a severe lack of reliable and accurate river cross section and flow data. This is not restricted to data concerned with the flow rates for water levels observed in the scheme, but structural limitations and capabilities of the scheme as well. For this reason the calibration is not as accurate as one would like to expect. The limit to the length of data available has also caused a limit to the accuracy of the calibration.

This being said, the model reacts to flood events and dry periods with an expected result and can be used to show strong trends with regards to how the water is used and allocated within the scheme. It is considered that with more calibration data and time to conduct the calibrations, the values for the return flow, Manning's n value for the channel section and that for the flood plains can be improved upon within the construction of the current model.

# 6 Verification of the Mike 11 Model

The verification of the model was completed with data collected from the DWAF website, the local Hydro Office in Cradock as well as from data received from the stations directly via SMS. There period used was from the 5th May 2006 to the 29th October 2006. This period has been remarkably different from the period used during the initial calibration from July to November 2005. The initial calibration period was very dry with few rainfall or flood events while the verification period has seen a 1:20 yr flood event on the Fish Rivers. Figure 6.0-1 shows the inflow into Elandsdrift and De Mistkraal Dams for 2005, while Figure 6.0-2 shows the inflow into the two dams for the same season for 2006. The inflow into Elandsdrift Dam is given as the black line while the inflow into De Mistkraal Dam is the blue line.

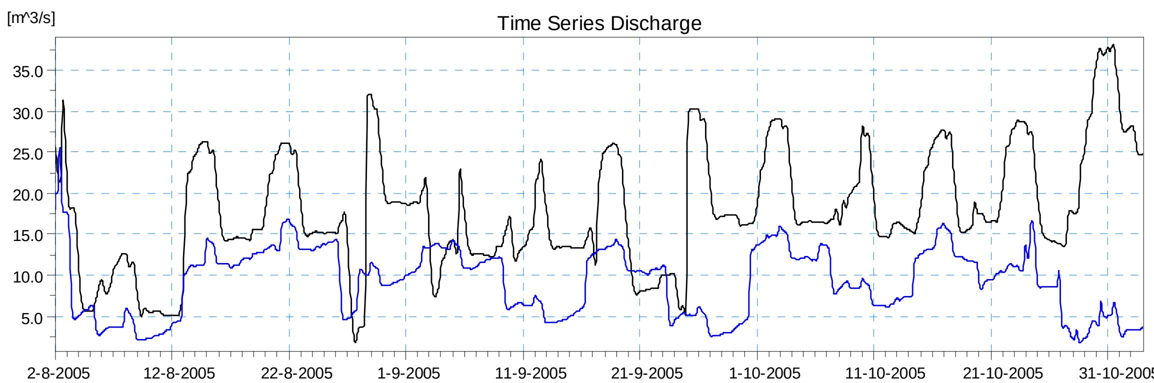



**Figure 6.0-5.11-1: Inflow into Elandsdrift (black) and De Mistkraal (blue) Dams for the Initial Calibration Period from July to November 2005**

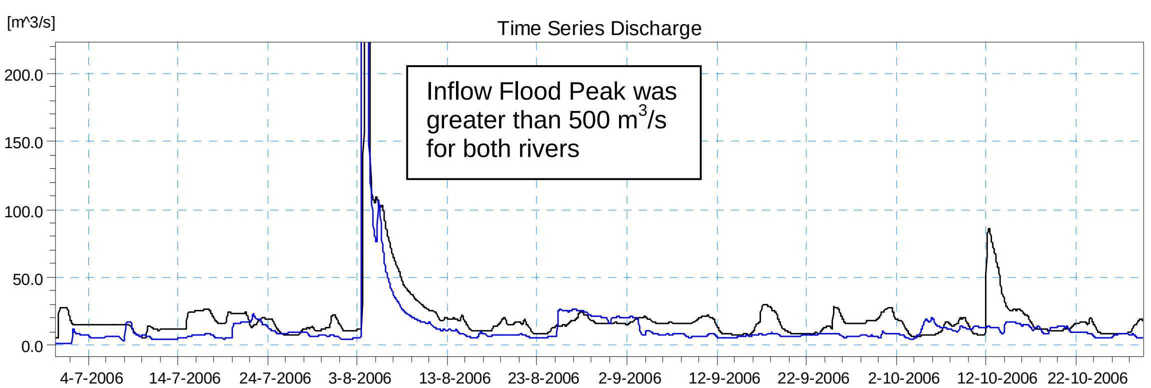



**Figure 6.0-5.11-2: Inflow into Elandsdrift (black) and De Mistkraal (blue) Dams for the Verification Period from July to November 2006**

This unseasonably wet spring has resulted in greater than expected return flows and runoff which have not been allowed for in the model due to the return flow values being calibrated based on the dry 2005 season. This effect is further enhanced with irrigators not taking water that they have requested as it is not needed due to the additional rainfall in the region. This variation between the model and the prototype has resulted in the simulated water levels in the four dams falling below that which has been observed.

## 6.1 Verification of Water Level at Darlington Dam

Figure 6.1-1 shows the simulated and observed water levels for the verification period. The black line represents the simulated results while the blue shows the observed results. This is true for all plots of simulated vs observed water levels and flow rates.

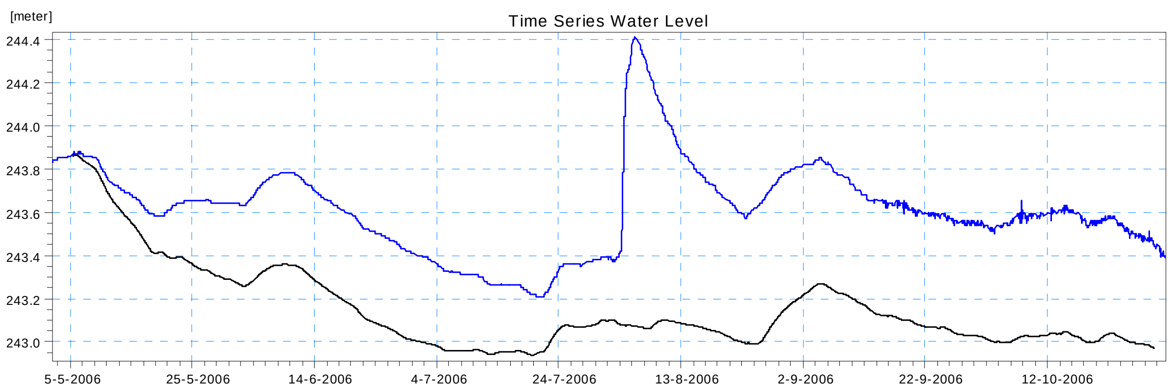



**Figure 6.1-1: Simulated (black) and Observed (blue) Water Levels at Darlington Dam for the Verification Period**

The inflow of the Sundays River seems to have been missed by the flow gauging station N2H007 at De Draai with just over 14 $m^3/s$ being observed as the inflow which is clearly not sufficient to cause the 1 m increase in observed water level at the dam. This can be seen in Figure 6.1-2. One can only assume that there is some error in the logging of the flow data or with the stage – flow relationship at this gauging station which could have been altered due to sedimentation through the years.

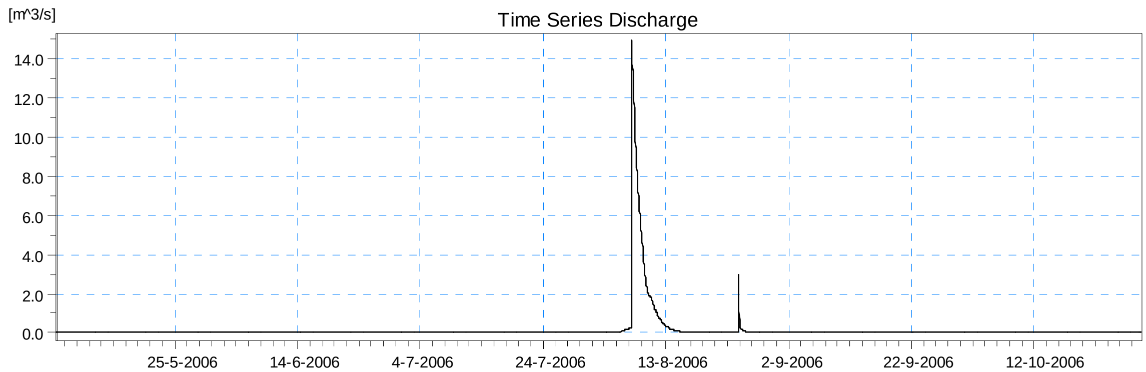



**Figure 6.1-2: Observed inflow to Darlington Dam from the Sundays River**

Despite this error, the trend in the rise and fall of the simulated and observed water levels is consistent and given that the total error over the whole verification period is just 0.5 m, this is a small consideration. Table 6.1-1 shows the statistical results for the verification of the water level at Darlington Dam. Although the correlation is poor, this is due to the unaccounted for flood inflow into the dam. Despite this, the volume error is just 0.19%.

**Table 6.1-1: Statistical Results for the Verification of the Water Level at Darlington Dam**

| **N2R001 WL** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.142 | |
| Max. positive difference | -0.006 | meter |
| Max. negative difference | -1.337 | meter |
| Volume observed | 3722321318 | |
| Volume modelled | 3715253360 | |
| Volume error | -0.19 | % |
| Peak observed value | 244.41 | meter |
| Peak modelled value | 243.864 | meter |
| Peak error | -0.223 | % |

## 6.2 Verification of Water Level at Grassridge Dam

Figure 6.2-1 shows the variation of the simulated and observed water levels at Grassridge Dam for the verification period.

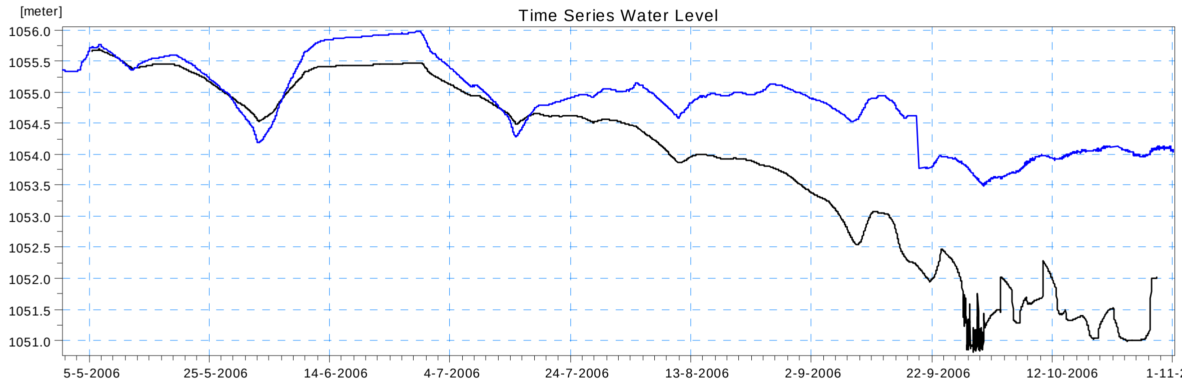



**Figure 6.2-1: Simulated (black) and Observed (blue) Water Level at Grassridge Dam for the Verification Period**

The simulated water level shows a consistent falling rate against the observed water levels. This is an indication that the modelled inflow is less than the observed inflow. This discrepancy is explained by the lower return flow values found during the calibration period which was much drier than the verification period. The instability in the simulated water level after the 22nd of September 2006 is due to the water level in the dam reaching the minimum level of the dam. Over a shorter period of time the simulation would show a far lesser error and as such the possibility of the simulated water level dropping to the bottom level of the dam is most unlikely.

Table 6.2-1 shows the statistical results for the verification of the water level at Grassridge Dam. The correlation value is high which shows that although there is a gradual increase in the difference between the simulated and observed water levels the variation between the two is consistent supporting the calibration results. The volume error is also sufficiently insignificant.

**Table 6.2-1: Statistical Results for the Verification of the Water Level at Grassridge Dam**

| **Q1R001 WL** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.746 | |
| Max. positive difference | 0.36 | meter |
| Max. negative difference | -3.08 | meter |
| Volume observed | 16124868058 | |
| Volume modelled | 16109761442 | |
| Volume error | -0.094 | % |
| Peak observed value | 1055.97 | meter |
| Peak modelled value | 1055.685 | meter |
| Peak error | -0.027 | % |

## 6.3 Verification of Water Level at Elandsdrift Dam

Figure 6.3-1 shows the simulated and observed water levels at Elandsdrift Dam for the Verification period. There is a break in the observed water levels from the 4th July to the 3rd August 2006.

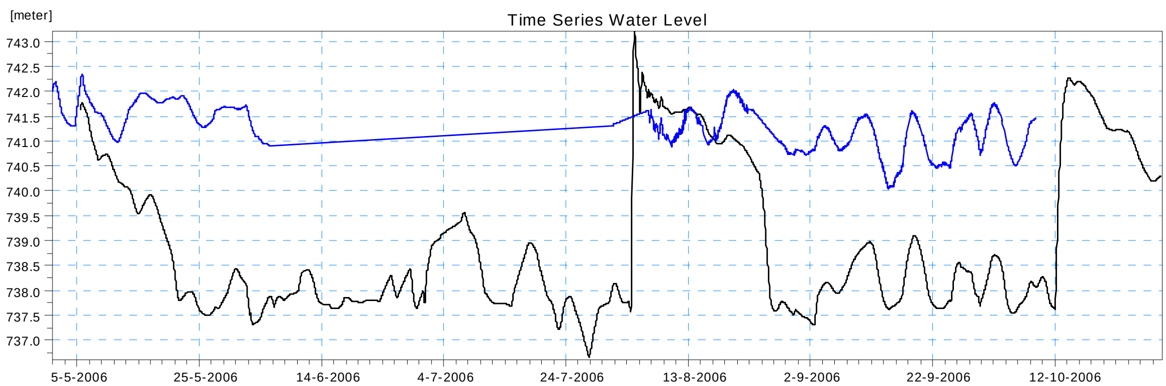



**Figure 6.3-1: Simulated (black) and Observed (blue) Water Levels at Elandsdrift Dam for the Verification Period**

The variation between the simulated and observed water levels at Elandsdrift is remarkable variable. With the small storage capacity of the dam, small variations in the inflows or outflows from the dam have a greater impact on the simulated water levels. The validation period used the observed flow release to the Groot Vis River as opposed to the reported flow release as used in the previous calibration simulations as this data was now available for this period.

The verification of the water levels is very poor at Elandsdrift, though one must take into account the complex operation of this dam as well as the large areas of un-gauged rainfall runoff that is not able to be included into the model. Figure 6.3-2 shows the daily rainfall observed at Elandsdrift which provides an indication of the rainfall in the region of Elandsdrift Dam. One can clearly observe the remarkable increase in rainfall during the verification period from May 2006 to November 2006 as compared to the same period during 2005 when the calibration of the return flows was conducted. This trend is repeated for all the rainfall gauges used in the model. The full set of data is available in Appendix G.

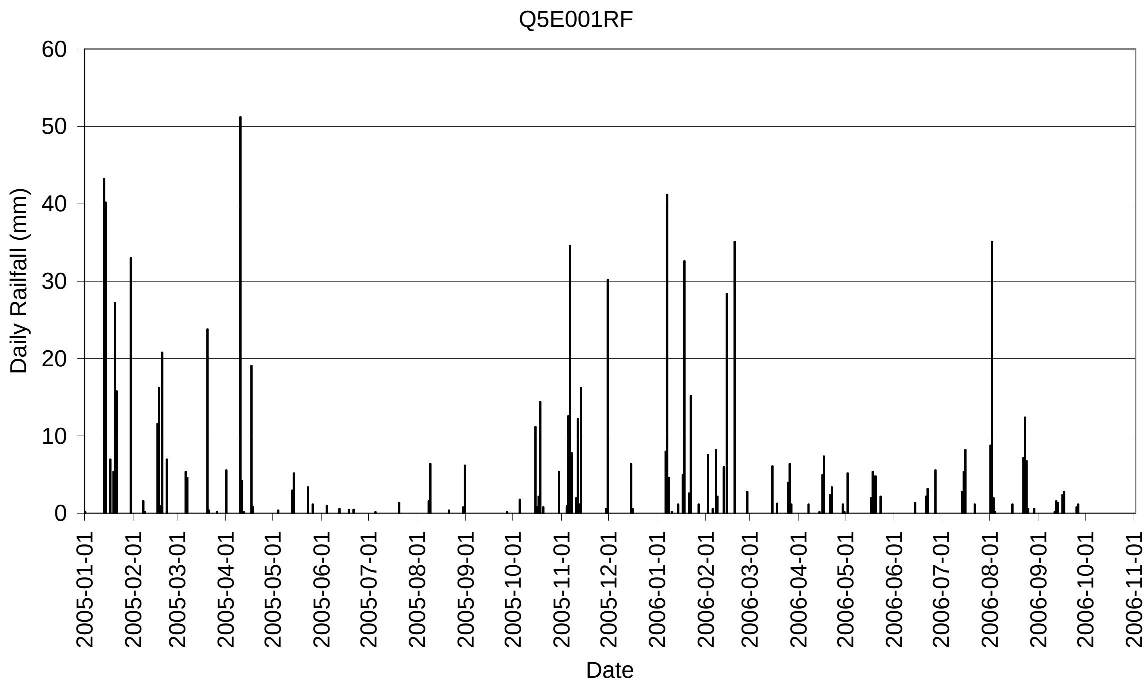



**Figure 6.3-2: Observed Daily Rainfall at Elandsdrift Dam from January 2005 to November 2006**

## 6.4 Verification of Water Level at De Mistkraal

Figure 6.4-1 shows the variation in the simulated and observed water levels at De Mistkraal Dam for the verification period.

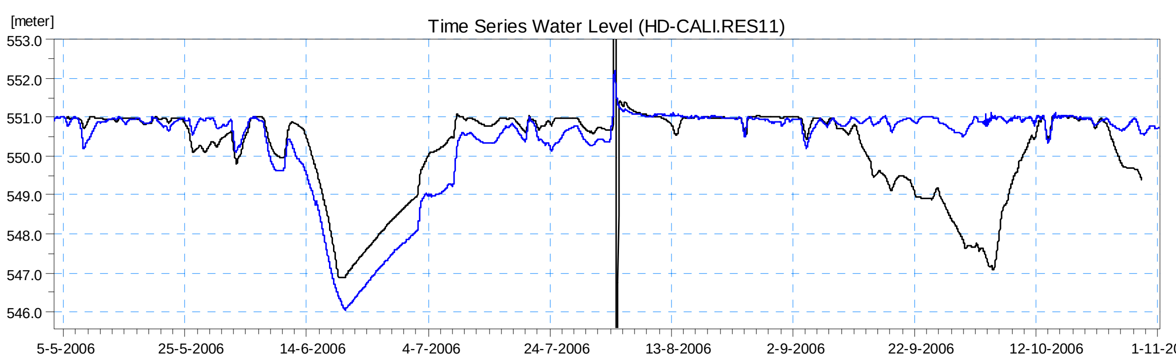



**Figure 6.4-1: Simulated (black) and Observed (blue) Water Level at De Mistkraal Dam for the Verification Period**

De Mistkraal Dam also has a small storage capacity though the dam is operated at near FSC and as the dam has an uncontrolled free overflow spillway, additional water is passed through the Little Fish River. The most noticeable event during the Verification is the 1:20 year flood that passed through the dam on the 3rd August 2006. The peak flow rate observed at the gauging station on the Little Fish Tributary upstream of the confluence with the Little Fish Canal was 550 $m^3/s$. This was factored up by 2.8 in accordance with the tributary flow factors found for the catchment from the full calibration period from July 2005 to May 2006, thus the final modelled inflow was 1540 $m^3/s$. Due to this large inflow into the dam, the model became unstable and as such was unable to accurately pass this flow through the dam and as such there is a jump and drop in the simulated water level at De Mistkraal Dam. Despite this the remaining variations in the simulated and observed water levels are concurrent, thought the correlation as calculated by MikeView was virtually non existent as shown in Table 6.4-1.

**Table 6.4-1: Statistical Results for the Verification of the Water Level at De Mistkraal Dam**

| **Q8R001 WL** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.003 | |
| Max. positive difference | 10.125 | meter |
| Max. negative difference | -309.47 | meter |
| Volume observed | 8408691600 | |
| Volume modelled | 8403236712 | |
| Volume error | -0.065 | % |
| Peak observed value | 552.21 | meter |
| Peak modelled value | 562.331 | meter |
| Peak error | 1.833 | % |

## 6.5 Verification of Flow Rate at Teebus at Jan Blaauwskop

Figure 6.5-1 shows the simulated and observed flow rates at the flow gauging station at Jan Blaauwskop on the Teebus River.

The additional flow in the river can be seen by the observed flow rate in blue being greater than the simulated flow in black. This is due to un-gauged rainfall events as well as additional runoff contribution to return flow. Of importance, is the clear indication that certain irrigators have not taken the water that has been allocated to them. These abstraction allocations are taken in the model and are shown by the black line dropping below the blue during a period of constant flow such as between the 18th July and the 9th September 2006. This is one of the contributing factors to the poor correlation of the water levels in the dams.

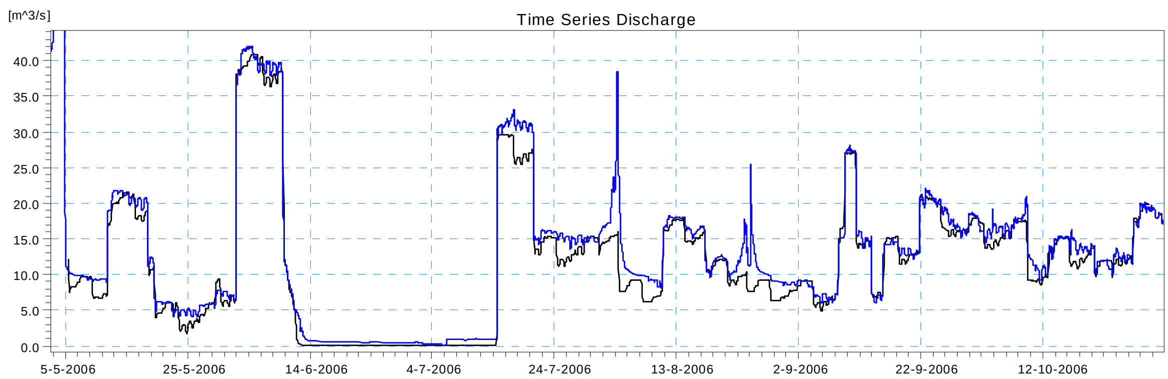



**Figure 6.5-1: Simulated (black) and Observed (blue) Flow Rate at Teebus Jan Blaauwskop for the Verification Period**

Despite the variation in the flow rates due to these un-abstracted flows and the rainfall runoff that is un-gauged Table 6.5-1 shows that the statistical correlation between the simulated and observed flow rates is very strong.

**Table 6.5-1: Statistical Results of the Verification of Flow Rate at Teebus Jan Blaauwskop**

| **Q1H012 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.943 | |
| Max. positive difference | 6.864 | $m^3/s$ |
| Max. negative difference | -22.749 | $m^3/s$ |
| Volume observed | 194024504.2 | $m^3$ |
| Volume modelled | 176913046.5 | $m^3$ |
| Volume error | -8.819 | % |
| Peak observed value | 41.986 | $m^3/s$ |
| Peak modelled value | 40.888 | $m^3/s$ |
| Peak error | -2.615 | % |

There is an eight percent volume error which if included would have improved the relationship found between the simulated and observed water levels at Grassridge Dam.

## 6.6 Verification of Flow Rate at Waaikraal Weir

Figure 6.6-1 shows the simulated and observed flow rate at the flow gauging station at Waaikraal Weir on the Groot Vis River. There is still some concern over the calibration of the stage – flow relationship at Waaikraal Weir as the simulated flow rate is approximately 20% greater than the observed flow rate as shown in Table 6.6-1.

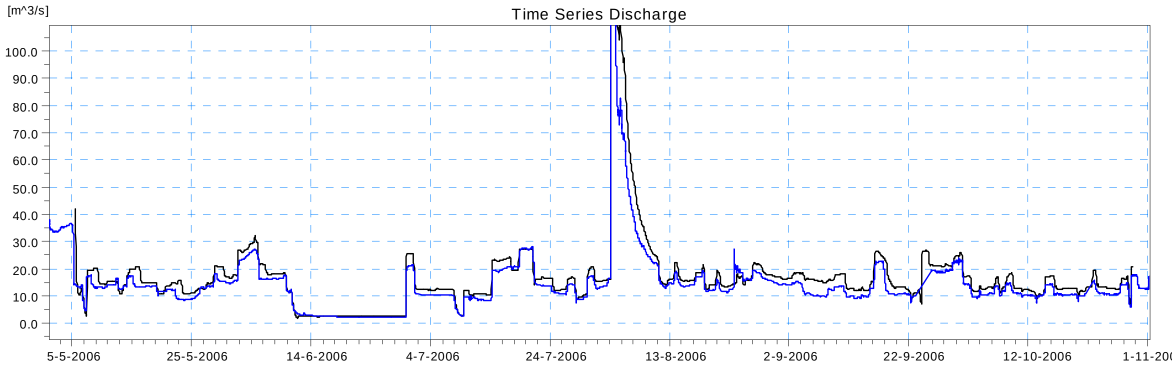



**Figure 6.6-1 : Simulated (black) and Observed (blue) Flow Rate at Waaikraal Weir for the Verification Period**

The return flow for the reach from Grassridge to Waaikraal is 1.7 $m^3/s$, which is on top of the reduced flow release from Grassridge Dam due to the entrapped air in the flow through the flow gauging station at the dam's outlet works. The difference between the simulated and observed flow volumes is calculated as a mean flow rate of 2.9 $m^3/s$ over the verification period. If we are to believe the observed flow rate, then the nett return flow would be – 1.2 $m^3/s$, which is not consistent with the observations at other flow gauging stations.

**Table 6.6-1: Statistical Results of the Verification of Flow Rate at Waaikraal Weir**

| **Q3H005 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.975 | |
| Max. positive difference | 192.747 | $m^3/s$ |
| Max. negative difference | -43.875 | $m^3/s$ |
| Volume observed | 224745301.8 | $m^3$ |
| Volume modelled | 269693082.1 | $m^3$ |
| Volume error | 19.999 | % |
| Peak observed value | 423.471 | $m^3/s$ |
| Peak modelled value | 583.46 | $m^3/s$ |
| Peak error | 37.78 | % |

Regardless the correlation is very strong and shows that despite the possible error in the calibration of the stage – flow relationship the simulated and observed flow rates react in a similar manner to the changed in the flow rate in the model.

## 6.7 Verification of the Flow Rate at Sheldon Weir

Figure 6.7-1 shows the simulated and observed flow rates at Sheldon Weir 146 km downstream of Elandsdrift Dam.

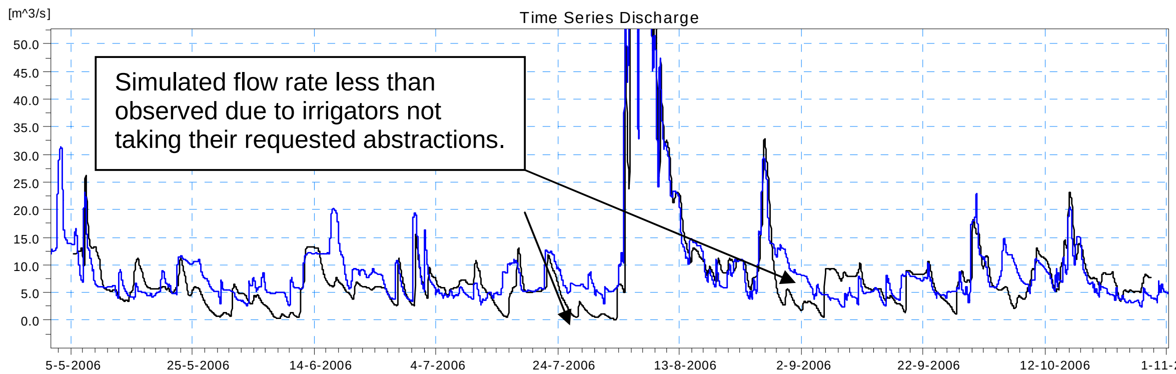



**Figure 6.7-1: Simulated (black) and Observed (blue) Flow Rate at Sheldon Weir for the Verification Period**

Despite the inclusion of the observed flow rate released into the Groot Vis River from Elandsdrift Dam at the new station downstream of Elandsdrift Dam. There is little improvement in the correlation between the simulated and observed flow rates. There are more instances of requested abstractions not having been taken as shown in Figure 6.7-1, though in some instances these have possibly been taken as a later date. Table 6.7-1 shows that there is some correlation with R2 found to be 0.556, though this is not as significant as the 0.7 correlation found during the full calibration period.

| Q7H005 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.556 | |
| Max. positive difference | 135.971 | $m^3/s$ |
| Max. negative difference | -299.265 | $m^3/s$ |
| Volume observed | 164286497.6 | $m^3$ |
| Volume modelled | 151744345.2 | $m^3$ |
| Volume error | -7.634 | % |
| Peak observed value | 368.515 | $m^3/s$ |
| Peak modelled value | 283.225 | $m^3/s$ |
| Peak error | -23.144 | % |

Despite the slight correlation, the volumes are just 7.6 % out and this is mostly due to the additional return flow and un-abstracted flow.

## 6.8 Verification of the Flow Rate at Parshall 2 on the Little Fish Canal

Figure 6.8-1 shows the simulated and observed flow rate at Parshall 2 on the Little Fish Canal for the Verification period.

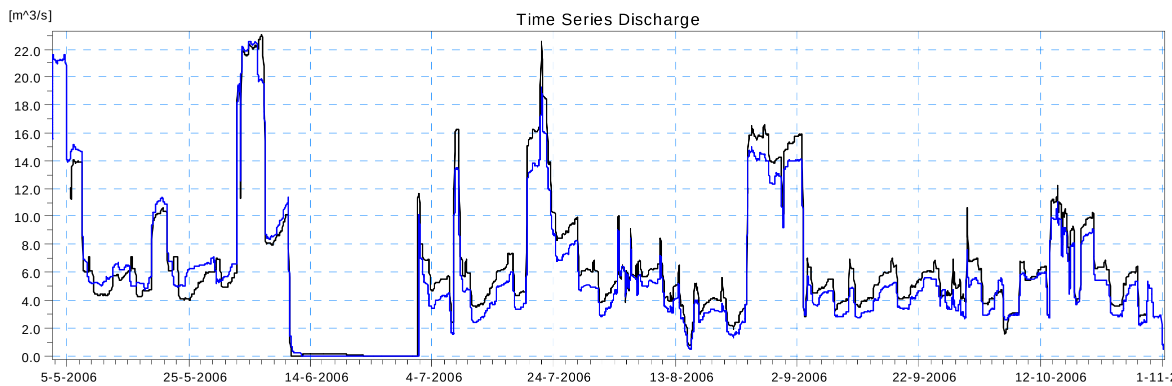



**Figure 6.8-1: Simulated (black) and Observed (blue) Flow Rate at Parshall 2 on the Little Fish Canal for the Verification Period**

There is a definite difference between the relationship between the observed and simulated flow before and after the maintenance period during June 2006. The initial calibration was conducted with a flow factor of 1.12 applied to the release from Elandsdrift to the canal. This now shows an increase in the simulated flow against the observed flow. Thus the error in the flow – stage relationship at this weir has been corrected and as such the simulated flow after the maintenance period in June does not require the flow factor. Figure 6.8-2 shows the simulated and observed flow rates for Parshall 2 on the Little Fish Canal after the flow factor has been removed.

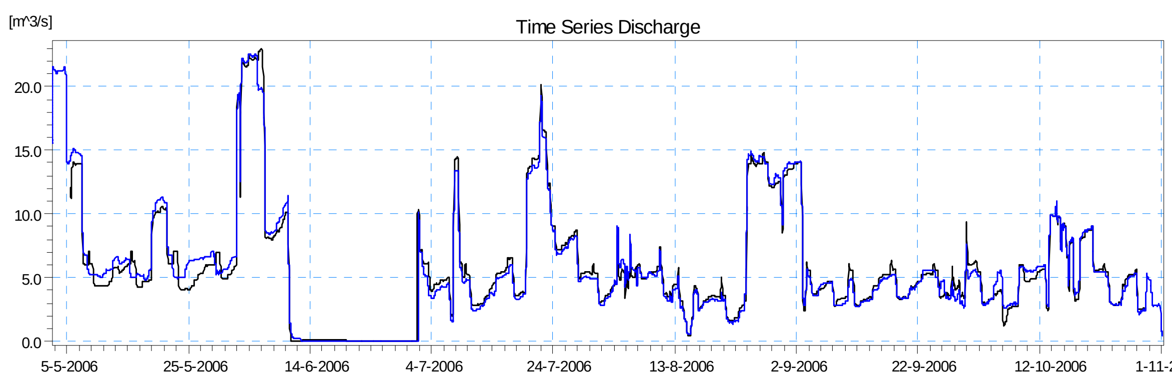



**Figure 6.8-2: Simulated (black) and Observed (blue) Flow Rate at Parshall 2 on the Little Fish Canal after Removing the Flow Factor**

One can observe that the simulated and observed results are more closely aligned. Table 6.8-1 shows the statistical results for the verification period.

**Table 6.8-1: Statistical Results for the Flow Rate at Parshall 2 for the Verification Period**

| **Q8H007 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.974 | |
| Max. positive difference | 10.271 | $m^3/s$ |
| Max. negative difference | -3.534 | $m^3/s$ |
| Volume observed | 85979466.96 | $m^3$ |
| Volume modelled | 86332843.32 | $m^3$ |
| Volume error | 0.411 | % |
| Peak observed value | 22.579 | $m^3/s$ |
| Peak modelled value | 23.008 | $m^3/s$ |
| Peak error | 1.9 | % |

The volume error is just 0.4% which is negligible. This is a demonstration of the need for a constant vigil over the various stage – flow relationships and the simulated and observed results.

## 6.9 Verification of the Flow Rate at Junctiondrift Weir

Figure 6.9-1 shows the simulated and observed flow rate at Junctiondrift Weir on the Little Fish River downstream of De Mistkraal Dam.

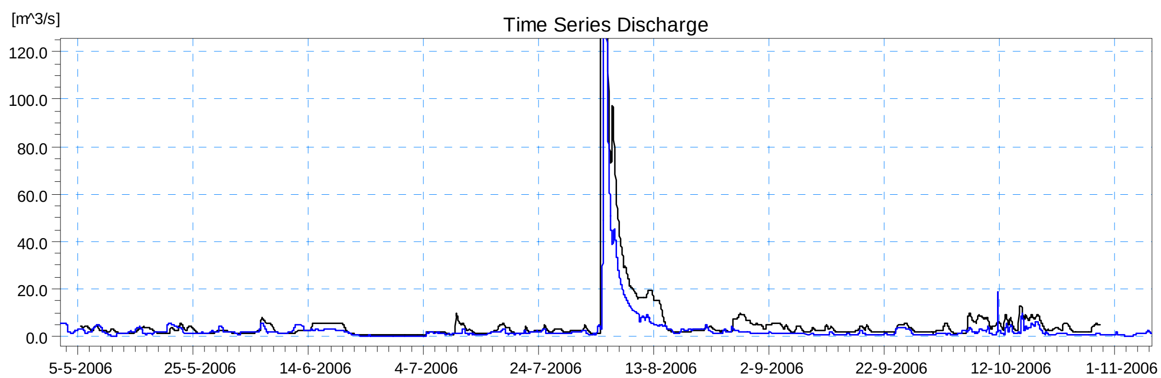



**Figure 6.9-1: Simulated (black) and Observed (blue) Flow Rate at Junctiondrift Weir for the Verification Period**

There is very little information on the flows in the Little Fish River. There is no observed record for the flow releases to the Little Fish River from De Mistkraal dam and as such the only data available was the recorded release as claimed by the operator at the dam. These are tabled by hand as and when changes are made to the sleeve valves that release the flow to the river. No record is kept of how this flow changes as a function of the water level over time. In addition to this the available data was not complete as it ended at the end of August 2006. At the release pattern is mostly repetitive, this cycle was repeated to provide a standard release for the model. Table 6.9-1 shows the statistical results for the flow rate at Junctiondrift. The correlation between the simulated and observed flow rates is strong though the volume error does not support a good result. The large flood that was experienced on the Little Fish on the 3rd August 2006 has been factored up by 2.8, this was to take into account the additional, un-gauged flow in the river. This factor appears to over compensate for these additional inflows and will need to be adjusted once more data is available.

**Table 6.9-1: Statistical Results for the Flow Rate at Junctiondrift Weir for the Verification Period**

| **Q8L011 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.734 | |
| Max. positive difference | 600.795 | $m^3/s$ |
| Max. negative difference | -12.809 | $m^3/s$ |
| Volume observed | 56089937.34 | $m^3$ |
| Volume modelled | 127702364.3 | $m^3$ |
| Volume error | 127.674 | % |
| Peak observed value | 380.14 | $m^3/s$ |
| Peak modelled value | 690.473 | $m^3/s$ |
| Peak error | 81.636 | % |

As such no firm conclusions can be drawn from the results obtained at Junctiondrift for this period. These errors and assumptions have an effect on the results at Piggott's Bridge as well as the flow from the Little Fish River joins into the Groot Vis River downstream of Sheldon Weir.

## 6.10 Verification of the Flow Rate at Piggott's Bridge Weir

Figure 6.10-1 shows the simulated and observed flow rate at Piggott's Bridge Weir for the verification period.

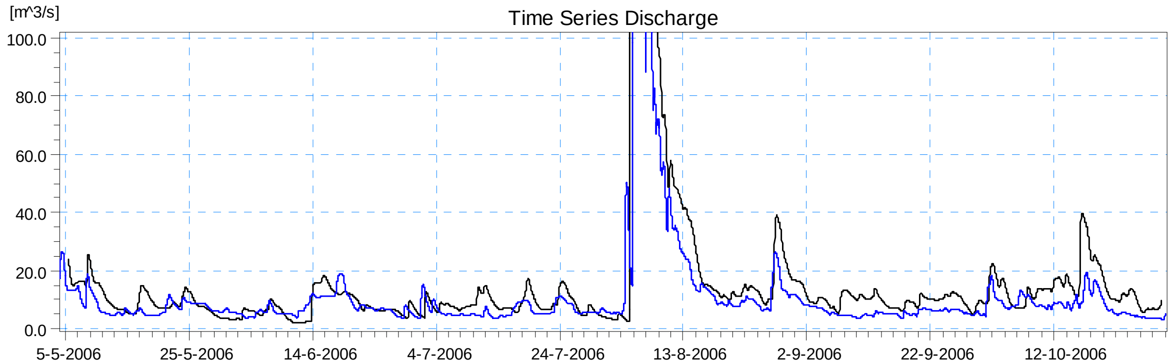



**Figure 6.10-1: Simulated (black) and Observed (blue) Flow Rate at Piggott's Bridge Weir for the Verification Period**

The correlation of the flow rates at Piggott's Bridge Weir is complex due to the long reach lengths encountered without any control in the flow release. As such, large events have been reasonably modelled though the time of flow is slightly less accurate. The smaller, normal operation flows, are far more complex to model, especially with the inflow from the Little Fish River. The flow travel times have been calibrated to suit the low, operational flows as flood events are less frequent and will have a smaller effect on the daily running of the model. Table 6.10-1 shows the statistical results from the verification run at Piggott's Bridge.

**Table 6.10-1: Statistical Results for Flow Rate at Piggott's Bridge Weir for the Verification Period**

| **Q9H012 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.5 | |
| Max. positive difference | 687.784 | $m^3/s$ |
| Max. negative difference | -509.004 | $m^3/s$ |
| Volume observed | 207423773.7 | $m^3$ |
| Volume modelled | 290440953.1 | $m^3$ |
| Volume error | 40.023 | % |
| Peak observed value | 695.605 | $m^3/s$ |
| Peak modelled value | 724.588 | $m^3/s$ |
| Peak error | 4.167 | % |

## 6.11 Verification of Flow Rate at Parshall 3 on the Skoenmakers Canal

Figure 6.11-1 shows the simulated and observed flow rate at Parshall 3 on the Skoenmakers Canal. The observed data is incomplete as it ends at the end of September 2006 while the verification period ends at the end of October 2006.

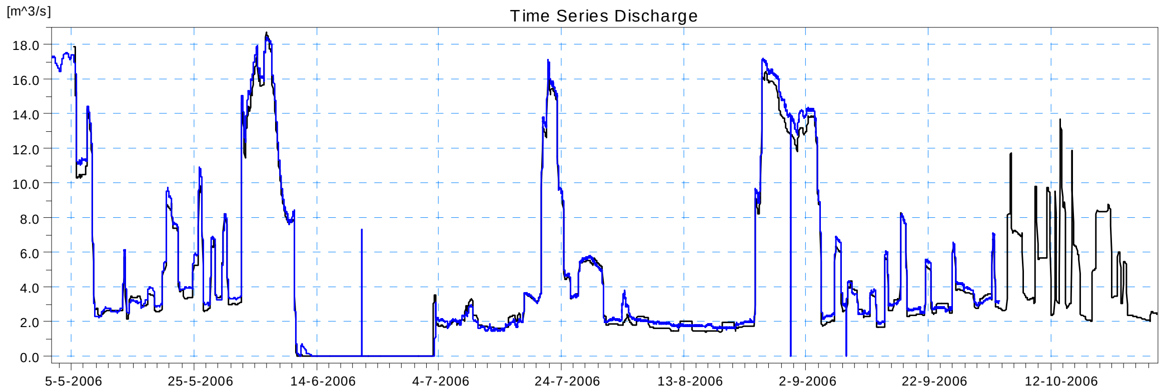



**Figure 6.11-1: Simulated (black) and Observed (blue) Flow Rate at Parshall 3 for the Verification Period**

Table 6.11-1 shows the statistical results for the verification at Parshall 3 on the Skoenmakers Canal. The short reach and predictable channel shape allows for good correlation and easy calibration.

**Table 6.11-1: Statistical Reuslts for the Flow Rate at Parshall 3 for the Verification Period**

| **N2L001 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.994 | |
| Max. positive difference | 12.859 | $m^3/s$ |
| Max. negative difference | -7.338 | $m^3/s$ |
| Volume observed | 58993335.51 | $m^3$ |
| Volume modelled | 56472548.53 | $m^3$ |
| Volume error | -4.273 | % |
| Peak observed value | 18.461 | $m^3/s$ |
| Peak modelled value | 18.696 | $m^3/s$ |
| Peak error | 1.271 | % |

A flow factor of 0.95 was applied to the flow from De Mistkraal Dam into the Skoenmakers Canal. The volume error of -4.3% suggests that this flow factor is no longer required and should be re-evaluated. This adjustment will be required throughout the operation of the OFS Model to improve the calibration as certain system parameters and physical constants vary with time.

## 6.12 Verification of Flow Rate at Korhaansdrift

The only reliable data available at Korhaansdrift is the abstraction to the Sundays Irrigation Boards canal which is set to abstract all of the flow from the Sundays River. Due to the high rainfall during the verification period, there is far more flow being released from Darlington Dam and as such there is little correlation between the flow released from Darlington Dam and the flow abstracted at Korhaansdrift Weir. This is shown if Figure 6.12-1.

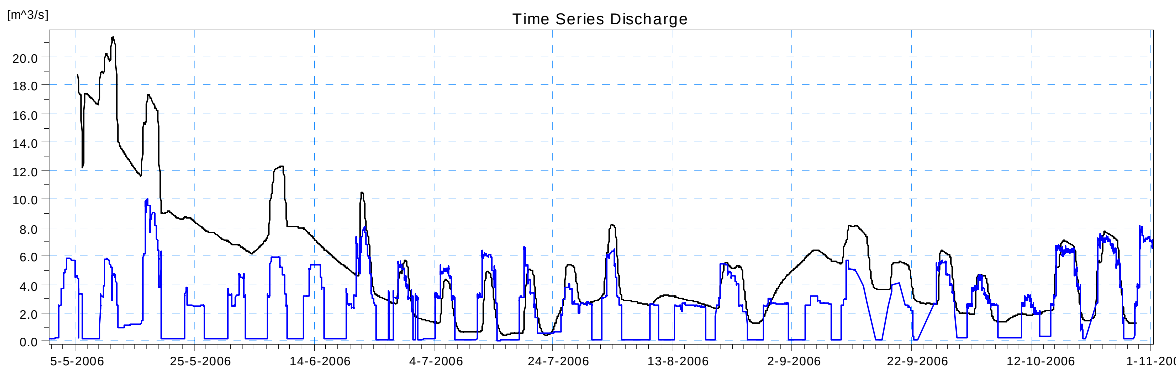



**Figure 6.12-1: Simulated (black) and Observed (blue) Flow Rate at Korhaansdrift Abstraction Weir for the Verification Period**

The verification has shown that the initial and full calibration of the model does model the flow through the Orange-Fish-Sundays scheme to a high level of certainty with only the lower Fish River having poor correlations for flow rate. These stations are affected by the long reaches and many un-gauged inflows that disrupt the ability for them to be accurately modelled.

The variation in return flows from the calibration period to the verification period shows clearly that this variable has a large effect on the long term simulations for water level at the dams. Fortunately the simulation period for the forecast model will be in days not months and the effect on the water levels at the dams will be negligible.

# 7 Setup of the Real-Time Data Retrieval

For a real time system to operate as efficiently as possible there are two key factors that must be born in mind; the first is the quality of the data, the second is the delay in acquiring the data. Having up to the minute data is only worthwhile if the data is accurate and reliable where as having data which is perfect but is days late defeats the real-time definition. To define what is required for the model, one must know how frequently the model will be updated. This depends on the run time of the model and the operation of the system. For the OFS-RT project, an update is required every four hours. This was decided from discussions with the modelling team as well as the operators of the scheme. It was felt that the model would be able to make an optimised simulation within four hours while the operators were prepared to implement changes to the operation of the model at four hour intervals. If the operation of the scheme was to be automatically operated as in some situations in the world, the update time could be reduced to further increase the efficiency of the model.

The various stages involved in providing the latest data with regards to the current conditions in the scheme are discussed in the following text.

## 7.1 Field Gauging Stations

There are currently 34 stations in the field. These stations monitor various physical aspects from flow stage to Electrical Conductivity (EC). These sensors log the variation in these physical properties of the flow every 3 min and record the average to the logger depending on the time interval set by the operator. For EC the readings are logged every hour. Rainfall is accumulated every 15 min and stored in the logger for each hour. Stage is recorded every 6 min.

This data is stored in the logger until such time as it is down loaded into a PDA by one of the technicians and the memory cleared. While this data is in the logger, it can be accessed remotely via a modem connected to a computer or the logger can be set to transmit the data via SMS on the GSM network to a GSM modem connected to a computer. The SMS method of retrieving data is the most popular as the cost is very much reduced as well as this is the simplest handling of the data. As such where ever possible, this system has been used. Table 7.1-1 shows the complete list of all current stations in the field collecting data.

**Table 7.1-1: List of all Stations Used to Log Data for the OFS-RT Project**

| | Station Number | Station Name | Lat. South | Long. East |
|---|---|---|---|---|
| | **Dams - GSM Modems** | | | |
| 1 | N2R001 | Darlington Dam | 33°12'22" | 25°08'50" |
| 2 | Q1R001 | Grassridge Dam | 31°46'10" | 25°28'25" |
| 3 | Q5L001 | Elandsdrift Dam | 32°31'45" | 25°45'10" |
| 4 | Q8R001 | De Mistkraal Dam | 32°58'00" | 25°40'25" |
| | **River Gauging Sites - SAT Modems** | | | |
| 5 | Q1L001 | Great Brak at Ellion bridge | 31°41'25" | 25°28'09" |
| 6 | Q1H012 | Teebus at Beaconsfield | 31°34'04" | 25°32'37" |
| 7 | N2L001 | Parshall no. 3 at Skoenmakers river | 33°04'31" | 25°34'43" |
| | **River Gauging Sites - GSM Modems** | | | |
| 8 | N2H010 | Darlinton Dam Outlet to Sundays | 33°12'22" | 25°08'50" |
| 9 | N2L009 | Volkers at Skoenmakers | 33°06'28" | 25°13'43" |
| 10 | N4H001 | Sondags at Courhans drift | 33°22'43" | 25°21'17" |
| 11 | Q1L002 | Great Fish at Kat Kop | 31°54'12" | 25°28'54" |
| 12 | Q1H022 | Outlet to Great Brak | 31°46'12" | 25°28'20" |
| 13 | Q3H005 | Great Fish at Waaikraal | 32°05'09" | 25°34'33" |
| 14 | Q5L002 | Great Fish current gauging site | 32°31'41" | 25°44'54" |
| 15 | Q5L003 | Great Fish at Mortimer | 32°20'11" | 25°43'19" |
| 16 | Q7L003 | Great Fish at Middleton | 32°46'48" | 25°50'04" |
| 17 | Q7H005 | Great Fish at Sheldon | 33°01'40" | 25°53'37" |
| 18 | Q8L011 | Little Fish at Junctiondrift | 33°05'29" | 25°57'14" |
| 19 | Q9L002 | Great Fish at Fort Brown | 33°07'41" | 26°36'49" |
| 20 | Q9H012 | Great Fish at Piggot's bridge | 33°05'53" | 26°26'41" |
| | **Tributary Gauging Sites - GSM Modems** | | | |
| 21 | N2H007 | Sondags at De Draai | 33°06'02" | 25°00'44" |
| 22 | N2H008 | Riet at Groen Leegte | 33°04'49" | 25°04'41" |
| 23 | Q1H013 | Little Brak at Zevenfontein | 31°46'40" | 25°19'07" |
| 24 | Q2H002 | Great Fish at Zoutpansdrift | 31°54'15" | 25°25'49" |
| 25 | Q3H004 | Pauls at Coutzenburg | 32°02'09" | 25°31'14" |
| 26 | Q4H013 | Tarka at Bridge Farm | 32°18'45" | 25°44'23" |
| 27 | Q6H003 | Baviaans at de Klerksdal | 32°36'21" | 25°53'05" |
| 28 | Q8H008 | Little Fish at Doornkraal | 32°47'06" | 25°36'54" |
| | **Irrigation Canal Sites - GSM Modems** | | | |
| 29 | N4H006 | Canal from Sondags at Courans drift | 33°22'47" | 25°21'14" |
| 30 | Q1H014 | Teebus tunnel out-let | 31°25'29" | 25°38'15" |
| 31 | Q5H006 | Canal from Elandsdridft Dam | 32°31'43" | 25°45'15" |
| 32 | Q8H007 | Little Fish Canal - Parshall section 2 | 32°49'58" | 25°39'21" |
| 33 | Q8H013 | Canal from De Mistkraal Dam | 32°58'07" | 25°40'19" |
| 34 | Q9H031 | Tunnel outlet at Glen Melville Dam | 33°11'25" | 26°37'28" |

Where the GSM network is not operational or the signal is often too weak to transmit the data, a Satellite Modem is used. These are very costly and are unable to send a SMS. As such these stations have to be contacted via a modem and the data downloaded directly. As the satellite stations are operated by a European agency, the connection to the satellite is made via Europe to the satellite and back to the station. At present all the satellite stations that are operated within South Africa are set to dial through to DWAF head office each night when the call rates are less expensive. This is not acceptable for the purpose of the OFS-RT project and as such these satellite stations will have to be contacted every 4 hours to update the model. There is also a problem making contact with the GSM Stations through a standard analogue modem. Experience has shown that this is not possible from outside the dialling code of that region. This problem is to do with the noise on the landlines between the different dialling codes.

### 7.1.1 A Typical Flow Gauging Station

Waaikraal Weir was recently refurbished and fitted with the most recent sensors and loggers to monitor the stage, rainfall and water quality in the Groot Vis River 70 km downstream of Grassridge Dam. The main weir section is shown in Figure 7.1-1 and consists of a crump weir at two levels, the lower level helps increase the accuracy of low flow measurements below 10 $m^3/s$, while the higher section increases the width allowing for a greater range of flow rates.

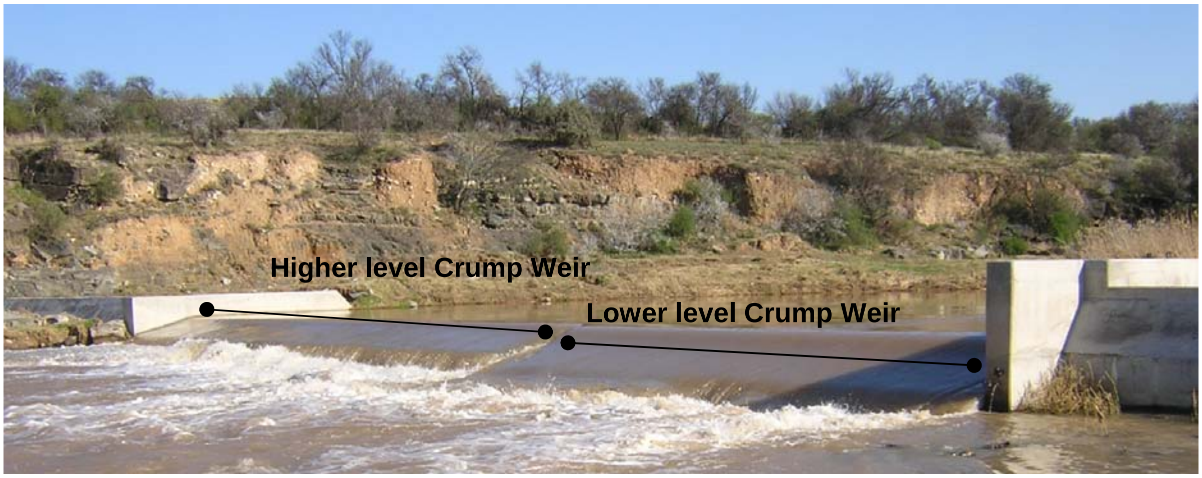



**Figure 7.1-1: Downstream View of Waaikraal Weir**

During significant flood events the flow may rise to a level where it flows over the side wall sections shown in Figure 7.1-2.


**Figure 7.1-2: Downstream View of the Side Walls of Waaikraal Weir**

A gauge plate is bolted to the side of the weir to indicate the stage level in the river. This is the level of flow above the lowest section on the weir crest. This is shown if Figure 7.1-3, the flow stage at the time of this picture was 0.64 m. On the side of the flow gauge plate is a chamber with the logger; this is shown in Figure 7.1-4. The chamber has a plate with slots to allow the water elevation to be the same inside the chamber as that in the stream. The logger is safe within this chamber from debris that could cause it damage if the logger was in the stream.


**Figure 7.1-3: Gauge Plate on the Weir**

**Figure7.1-4: Slotted Plate over the Chamber**

There is a small pipe that diverts some of the water passed the EC meter which records a measure of electrical conductivity. This is an indication of the level of dissolved salts in the water. Figure 7.1-5 shows the water trickling out of this pipe from the end of the side wall.

Figure 7.1-6 shows the side wall with the remote water level indicator and rain gauge at the end. The last flood level is indicated by the flattening down of the reeds and grass on the banks. The sediment that the flood brought can be seen in the lower left of the picture. If the level of sediment gets too high it affects the accuracy of the stage – flow relationship and needs to be removed or the stage - flow relationship needs to be revised. Figure 7.1-7 shows Tarka Bridge Weir which has been over grown by reeds over the years. This flow gauging station therefore has lost some of its accuracy as the reeds change the flow patterns through the weir, but it is still in operation as an indication of the flow coming in from this tributary.

Figure 7.1-5: Water from EC Meter trickling out for the End of the Side Wall

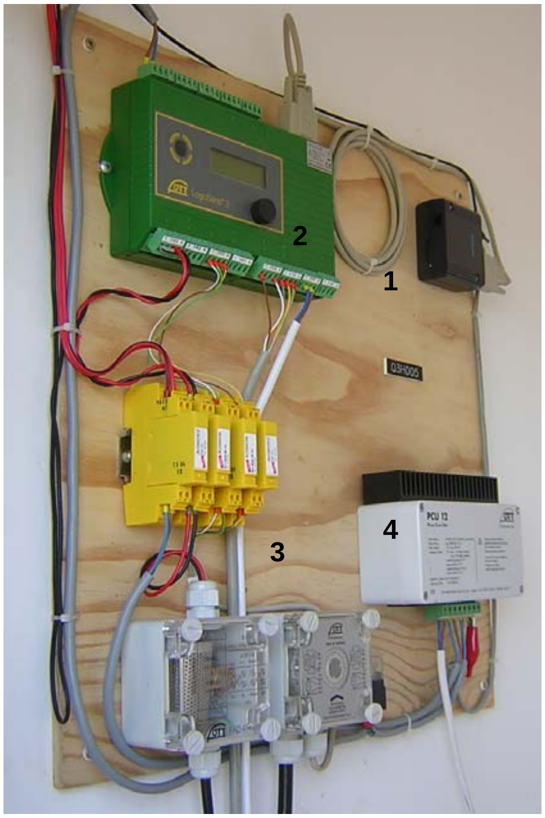

Figure 7.1-6: Waaikraal Weir, showing the Side Walls and the Last Flood Level at the Weir

**Figure 7.1-7: Tarka Bridge Weir showing the Reeds that have grown on the sediments that have been deposited on the Banks and Behind the Weir**

**Figure 7.1-8: A Stage Logger being taken out of its Chamber**

The stage loggers are simply pressure sensors that register hydrostatic pressure. They are suspended in the chamber at the correct level, as shown in Figure 7.1-8, and send the data that they read through to the control box which is often some distance up on the bank to protect the sensitive electrical instruments from the high flood waters. These control boxes also need to be secure to protect from theft and vandalism as the batteries and solar panels are a commodity within the local communities.

Cables are run from the loggers to the control box in pipes buried in the ground for protection. Figure 7.1-9 shows the instrumentation kept within these control boxes. The items on the board are listed as follows:




1. GSM Modem to send and receive data.

2. Logger Control Box, this stores the data and the software to manage the data and communications ability.

3. These are the cables from the Loggers coming into the Logger Control Box

4. Power Management Box, this monitors the voltage in the batteries and supplies the DC current to the Modem and Logger Control Box.

**Figure 7.1-9: Electronic Instrumentation Kept Inside the Control Box**

## 7.2 Hydras 3 and the Receiving of the Data from the Field

The logger system has been supplied for this project by the German Company, OTT. They have a long and successful history in instrumentation with a proven track record. The data sent via SMS from the field stations is sent to a GSM modem connected to a computer with the OTT Hydras 3 software. The Hydras 3 software is a computer program which collects and stores in a data base the logger readings and can provide graphical output of the results. These SMS's are received and automatically read into the raw database for that particular station by the software. Figure 7.2-1 shows the Hydras 3 workspace window with the OTT SMS Receiver program in the top right hand corner.

The software allows the operator to view the data that has been sent through from a station as sensors. Sensors are either primary in that they represent the physical measurements taken by the loggers in the field, or Virtual, in that the sensor converts the real sensor value via a stage – flow relationship or similar transformation into flow rate or TDS. The stage reading at a dam can be converted into the % of FSC or the actual water level above mean sea level. For example, Waaikraal (Q3H005) sends data for stage (10) and EC (55), where the number in the brackets represents the station number of the sensor number. These are added to the data base along with two virtual sensors for flow rate (30) and TDS (56). Table 7.2-1 shows the numbers for the different sensors.

**Table 7.2-1: List of Sensor Numbers used for the OFS-RT Project**

| Property Being Measured | Primary or Virtual | Sensor Number |
|---|---|---|
| Upstream Water Level | Primary | 10 |
| Dam Water Level (MASL) | Virtual | 20 |
| Flow Rate | Virtual | 30 |
| Conductivity | Primary | 55 |
| TDS | Virtual | 56 |
| Rainfall | Primary | 70 |
| Full Supply Capacity (%) | Virtual | 220 |
| Nett Capacity | Virtual | 320 |

The virtual sensors are calculated from one or more primary or virtual sensors by a conversion table or equations. The stage – flow relationships are given as a table which is loaded into Hydras 3. The conversion from EC to TDS is given as an equation which is dependant on the stage being positive. This is to restrict false readings during times of no flow. This is especially important for the tributary stations.

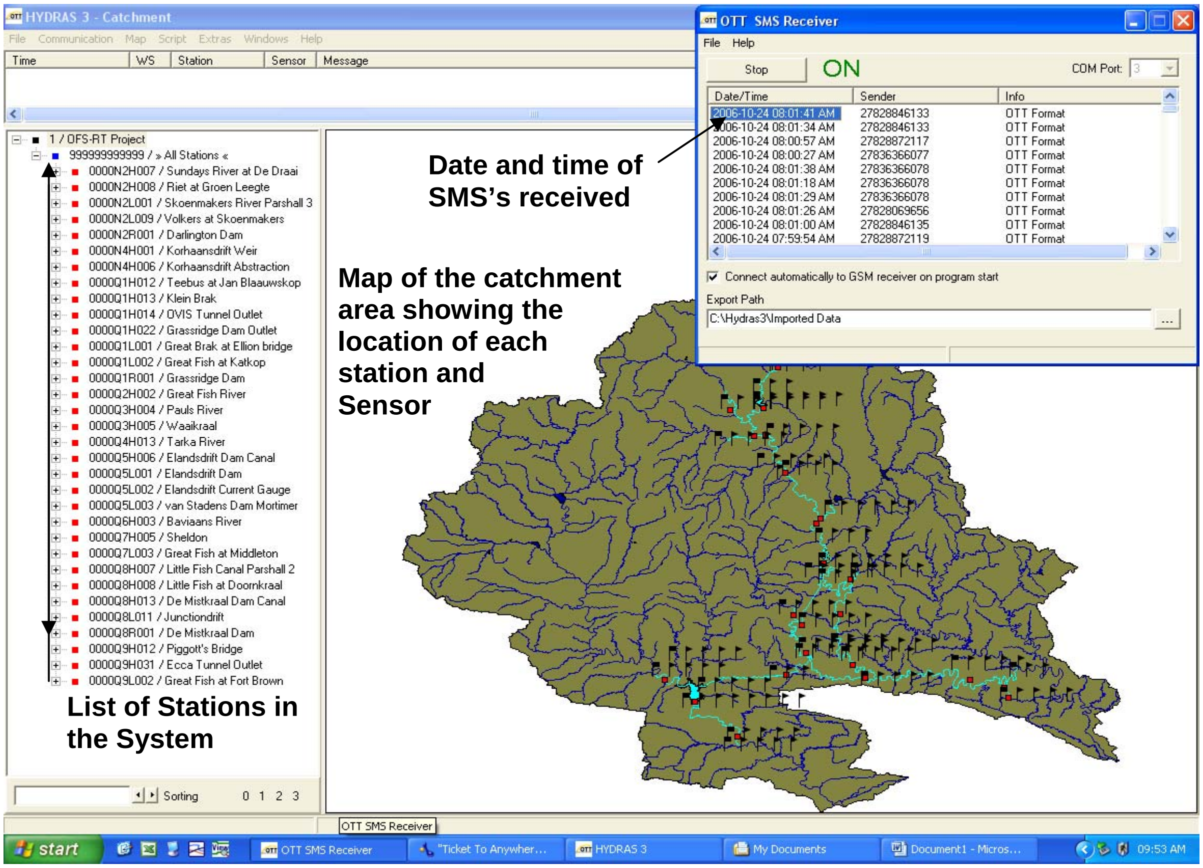



**Figure 7.2-1: Hydras 3 Workspace with OTT SMS Receiver Open**

Each of these sensors can be plotted as a time dependant variable. Up to 6 different sensors can be plotted as shown in Figure 7.2-2. Here the two raw sensors and the two virtual sensors for Waaikraal Weir are shown for the week from the 18$^{th}$ October 2006 to the 24$^{th}$ October 2006.


**Figure 7.2-2: Plot of Sensors for Waaikraal Weir as presented by Hydras 3**

If a station is unable to send SMS's or one wishes to download the data from a satellite station, this is possible with the read/operate function within Hydras 3. The selected station is displayed as shown in Figure 7.2-3 with the information regarding the number to dial, which sensors to download and for what time period on the left hand side. The right hand side has the communication details with regards to what modem and protocol to use to make the connection. Data which is downloaded from the field station is added to the raw database.

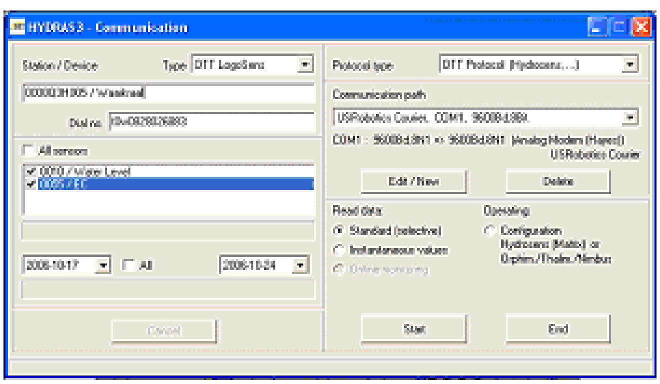



**Figure 7.2-3: Read / Operate Page for Downloading Data from a Field Station in Hydras 3**

The data available in Hydras is not immediately accessible to other programs, one must first set up a data export. A collection of sensors can be exported to a text file from Hydras 3, this can be set up to run automatically at any time interval. This allows for the data to be handled by different program for conversion and use in a model.

# 8 Pre-Processing of the Real-Time Data

The text file exported from Hydras 3 is not in the format required by the Mike 11 model for conversion to a Mike 11 time series file. The data also needs to be checked and processed as the tributary inflows have to be forecasted 7 days into the future, and the flow releases from the control structures have flow factors that are to be applied.

## 8.1 Data Checking and Correction

The Pre-Processor program is executed as a batch file. This is run after the Hydras 3 text file export has completed exporting the last 7 days worth of data. All the sensors are exported to one text file sequentially. If a station's sensors have not logged any data due to an error or trouble with communication, the Pre-Processor sets the value to 0 for the last 7 day period from the Time of Forecast (TOF). The Time of Forecast is the current time as given by the computer clock.

The pre-processor program separates each sensor into a separate sheet and names then with a specific name for its station and sensor type. The station's number is followed by the sensor number, for example, Waaikraal Weir has the station number Q3H005 and the sensor for the discharge through this weir is given the number 30. Thus the new sheet is called Q3H005-30. The data is also converted to the format required by Mike 11 to be made into a Mike 11 time series file. At this time the data can be saved as individual text files and is ready to be converted to the Mike 11 times series file format. But before this is done, the tributary inflow and TDS are forecasted 7 days into the future.

## 8.2 Inflow Forecast Method

As rainfall runoff forecasting is not considered in this model due to a limitation on the availability of rainfall stations as well as detailed data on the catchments, the tributary inflow is forecasted by a simple regression method.

There are eight tributaries that have real time data being sent to the operator. The values for the Mode, Mean, Minimum and Maximum for each of the stations along with the final base flow value to be used are presented in Table 8.2-1.

**Table 8.2-1: Statistical Results for Tributary Inflow Data in $m^3/s$**

| Station Number | Station Name | Mode | Mean | Min | Max | Base Flow |
|---|---|---|---|---|---|---|
| N2H007 | Sundays at De Draai | 0.002 | 1.072 | 0.000 | 74.774 | 0.001 |
| N2H008 | Riet at Groen Leegte | 0.000 | 0.219 | 0.000 | 71.801 | 0.001 |
| Q1H013 | Klein Brak at Zevenfontein | 0.005 | 1.367 | 0.000 | 130.586 | 0.001 |
| Q2H002 | Groot Vis at Soutpansdrift | 0.177 | 1.124 | 0.018 | 185.785 | 0.150 |
| Q3H004 | Pauls at Coutzenburg | 0.101 | 0.619 | 0.000 | 102.100 | 0.150 |
| Q4H013 | Tarka at Bridge Farm | 0.319 | 0.893 | 0.000 | 106.910 | 0.250 |
| Q6H003 | Baviaans at de Klerksdal | 0.028 | 2.114 | 0.000 | 241.260 | 0.025 |
| Q8H008 | Little Fish at Doornkraal | 0.178 | 1.774 | 0.000 | 545.121 | 0.150 |

The inflow hydrographs for each of the tributary stations for the last 20 years, where available, were examined and 6 to 10 random flood events including a range of flood peaks were taken for analysis of the regression of the peak flow rate back down to the base flow rate. The base flow rate was also determined from observations of the Hydrographs and values of the mean and mode for the data set.

The recession of a flood hydrograph is related to the runoff characteristics of the catchment. The curve can be estimated by the following equation (G. Pegram, J.A Melvill).

$$q_2 = q_1 \times e^{(-dt/K)}$$ **Equation 8.2-1**

Where:

$q_2$ : Flow rate in the second time step

$q_1$ : Flow rate in the first time step

$dt$ : Time step

$K$ : Coefficient relating to the catchment characteristics

It is important to note that the value of K has a relationship to the unit of time used. For this model the value of K is related to Hours.

K is a coefficient that is unique to any catchment and characterises the response of the catchment to a rainfall event. A catchment with a long reach, lots of vegetation and low flat slope, will have a high K value while a catchment with a short reach, little vegetation and a steep gradient will have a small K value. So for a catchment with a small K value, the water that falls on the land flows swiftly to the rivers and is soon passed out of the system as opposed to a catchment with a high K value where the rainfall runoff will take longer to reach the rivers and recede to the base flow. There are many factors that influence the recession curve for a catchment, such as the saturation of the soil and the level of the water table; these are related to the amount of rain that has fallen during the recent period. In addition, for large catchments, there could be a spatial variation in that a rainfall event might cover the whole catchment region for one event then just a portion of the catchment for a different event. This localised rainfall could then be located close to the gauging weir which would reduce the recession time while if this occurred some distance from the gauging weir, the attenuation of the flow would increase the recession time. These variations could not be added to the model as there was no data for the analysis and the proposed model was required to be simple.

Figure 8.2-1 shows the assumed recession curves for five of the tributaries. This shows the variation in the recession of the flow from the same flow rate over the same period of time. The K value for each is given in the figure. One can see that the higher the value for K the longer the time of recession becomes.

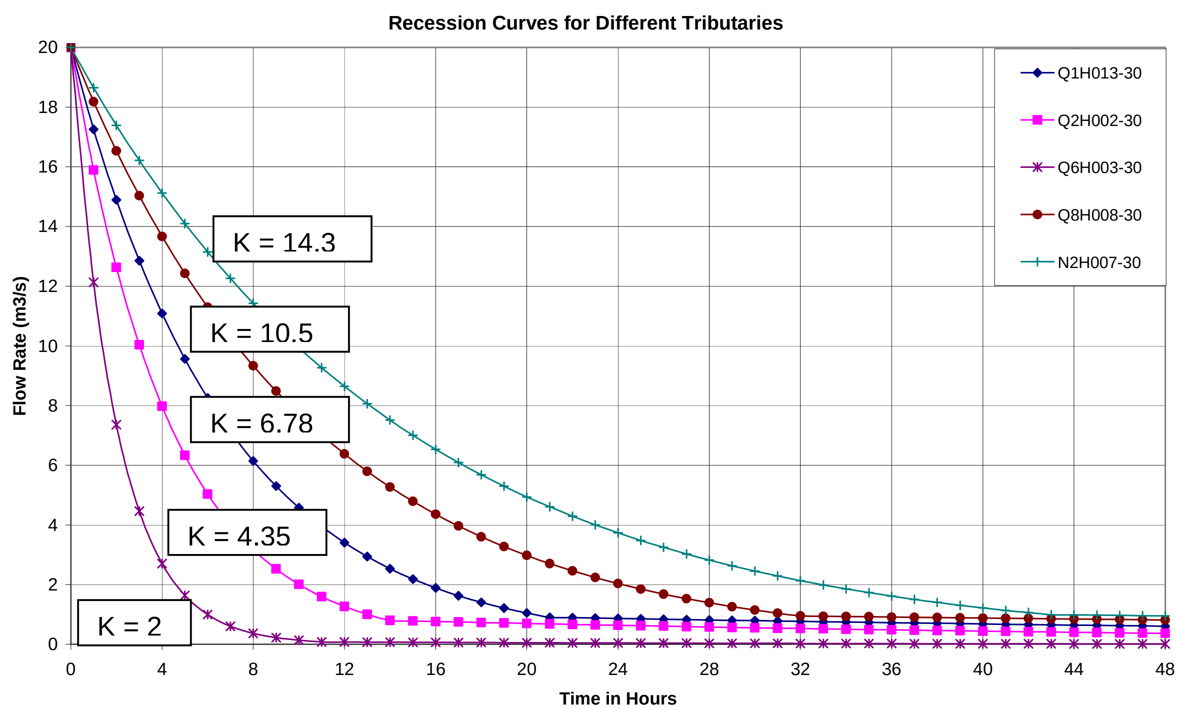
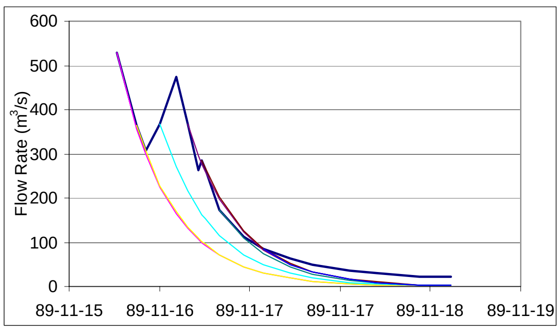



**Figure 8.2-1: Assumed Recession Curves for Various Tributaries**

To demonstrate how the calibration of the K values was done, some of the results for the calibration of K for one of the tributaries are shown in the text. The Little Fish River (Q8H008) has a catchment area of 1512 km$^2$. The vegetation is dense in parts with a moderate gradient. For various times based on the measured flow rate, a forecast was made. These are shown in Figures 8.2-2a and 8.2-2b. The blue line represents the measured flow rate at the gauging station while the other coloured lines are the forecasts made at different times. One can see in Figure 8.2-2a that the early forecasts do not take into account the additional rainfall runoff that was measured later.

A K value of 10.5 was found to limit the error between the measured and the assumed values. The error was found as the sum of the absolute value of the difference between the measured and assumed value for each of the various assumed forecast plots. Figure 8.2-3 shows the fit of the assumed flow recess curve for the Little Fish River at Q8H008.



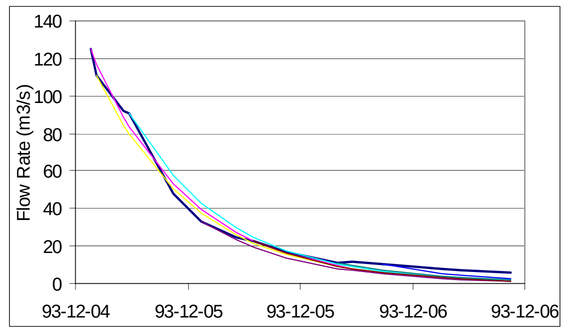




**Figure 8.2-2a: Recession Forecast for Q8H008**

**Figure 8.2-2b: Recession Forecast for Q8H008**

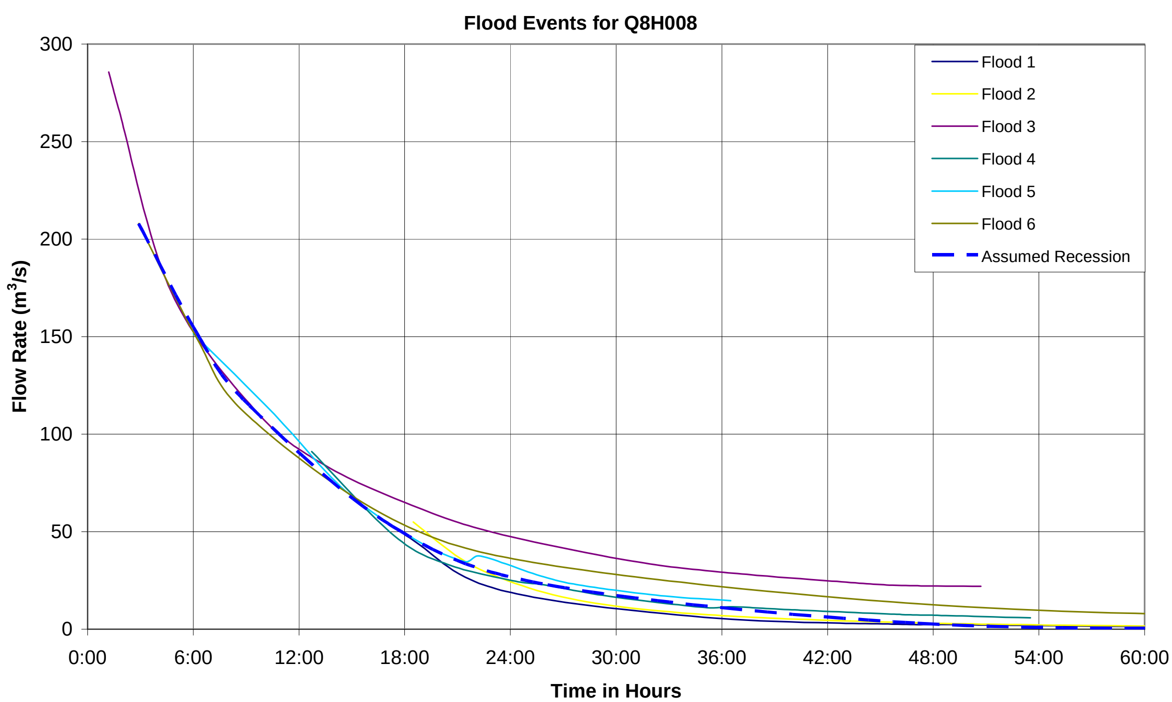



**Figure 8.2-3: Various Flood Events Plotted against the Assumed Recess Curve for Q8H008**

One will notice that flood 3, which is the largest, has a slower recess than the other smaller floods. As the peak flow rate is larger, there must be more rainfall runoff and this would take longer to return to the base flow than a smaller flood event. Although there are these variations, the general ability of the single K value to represent the recession curve for the catchment is valid.

Appendix F has the calibration and fit plots for the other tributary inflows.

The recession of the flow was seen to slowdown as the flow receded towards the base flow. For this reason, a secondary K values was introduced. This $K_L$ is used to reduce the flow once it had gone below a certain transition flow rate $T_f$, usually taken as 10 times the base flow. The $K_L$ is 10 times greater than the $K_U$, this assumption was tested and found to work satisfactorily. Figure 8.2-4 shows the Flood Events for Pauls River (Q3H004) which has a high base flow as some abstraction return flow is diverted into the Pauls River upstream of the flow gauge.

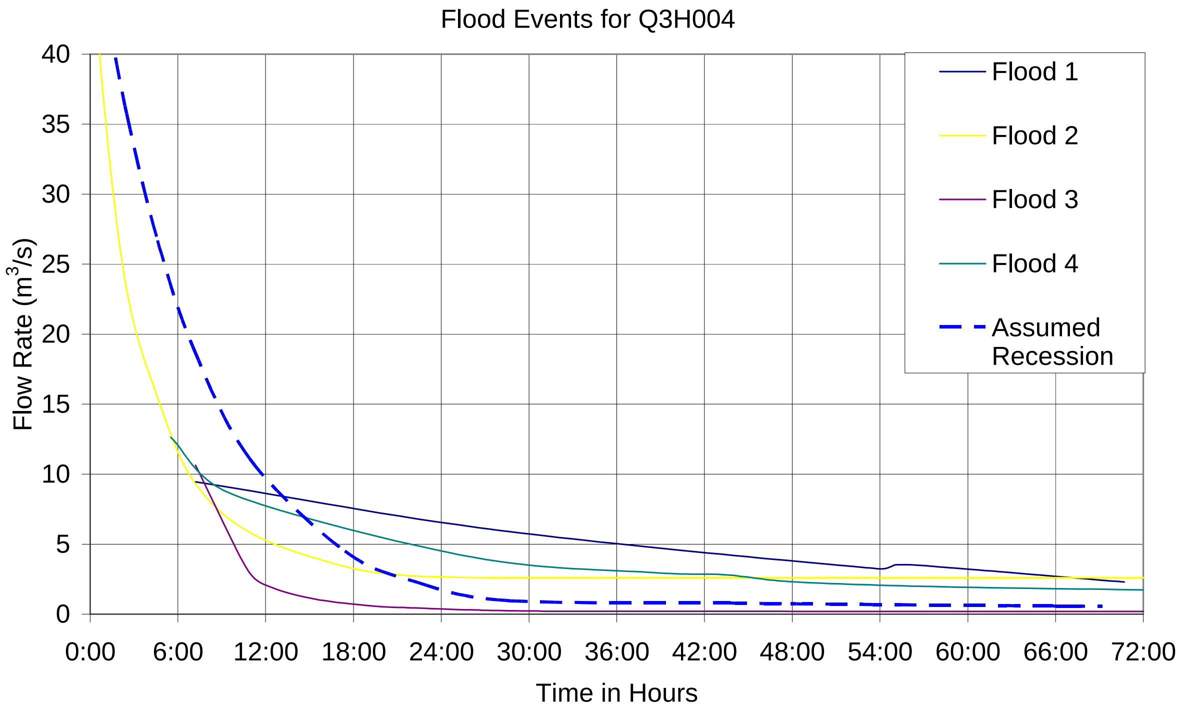



**Figure 8.2-4: Various Flood Events Plotted against the Assumed Recession Curve for Q3H004**

The assumed recession curve follows flood 2 almost precisely, one can note that the assumption for the lower flow rates is most applicable for the reduction in recession rate as the flow reduces towards the base flow rate. The final equations used are given by Equations 8-1 to 8-3.

$$If\left(q_1 > T_f\right) \rightarrow then \rightarrow q_2 = q_1.e^{(-(t_2-t).24/K_U)}$$ **Equation 8-2**

$$If\left(q_1 > B_f\right) \rightarrow then \rightarrow q_2 = q_1.e^{(-(t_2-t).24/K_L)}$$ **Equation 8-3**

$$Else \rightarrow q_2 = B_f$$ **Equation 8-4**

The final K values for each tributary inflow station are shown in Table 8.2-2.

**Table 8.2-2: K Values for the Tributary Inflow Stations**

| Station Number | Station Name | Upper K value | Lower K value | Base Flow | Transition Flow from $K_U$ to $K_L$ | Catchment Area (km$^2$) |
|---|---|---|---|---|---|---|
| | | $K_U$ | $K_L$ | $B_f$ | $T_f$ | A |
| Q1H013 | Klein Brak at Zevenfontein | 6.78 | 67.8 | 0.001 | 1 | 2445 |
| Q2H002 | Groot Vis at Soutpansdrift | 4.35 | 43.5 | 0.1 | 1 | 1713 |
| Q3H004 | Pauls at Coutzenburg | 4.5 | 45 | 0.25 | 1.5 | 872 |
| Q4H013 | Tarka at Bridge Farm | 3.2 | 32 | 0.25 | 5 | 4742 |
| Q6H003 | Baviaans at de Klerksdal | 2 | 20 | 0.01 | 0.1 | 814 |
| Q8H008 | Little Fish at Doornkraal | 10.5 | 105 | 0.1 | 1 | 1512 |
| N2H007 | Sundays at De Draai | 14.3 | 143 | 0.001 | 1 | 13428 |
| N2H008 | Riet at Groen Leegte | 5.52 | 55.2 | 0.001 | 1 | 341 |

These values are to be revised from time to time as part on the continual maintenance of the model.

There are many complex and intricate rainfall runoff models available nowadays, though these all require a significant amount of data with regards to the catchment and its characteristics. Thus this simple yet effective routine was devised to solve a problem in the most efficient manner with the available data at hand. Figures 8.2-3 and 8.2-4 show that for large or small flow rates the results provide an indication of how the flow will recede in the following hours and days. It is important to note that this model can not take into account additional rainfall that might have fallen already or be falling in the future. This additional rainfall will be forecasted during the following update of the model as indicated in Figure 8.2-2a.

## 8.3 TDS Forecasting for Tributary Inflows

The flow can be forecasted as it is a function of the catchment characteristics for the station. The TDS on the other had is a function of the catchment as well as the flow rate. Generally there is a negative relationship between the flow rate and the water quality in the rivers. This was studied by Wageed Kamish and Simone Maharaj of Ninham Shand Consulting Engineers Inc. as part of the OFS – RT Project.

They were able to establish a log-log relationship between the flow rate and the TDS measured in the tributaries at the flow gauging stations. The equation for the relationship is as follows:

$$\log(TDS) = B + A \times \log(Q)$$ **Equation 8.3-1**

Where A and B are constants for the catchment at that station.

The results of their research and calculations are presented in Table 8.3-1.

**Table 8.3-1: Coefficients for the relationship between flow and TDS**

| Gauge | River Name | A | B |
|---|---|---|---|
| Q1H013 | Klein Brak River | -0.1183 | 923.42 |
| Q2H002 | Groot Vis River | -0.2989 | 455.09 |
| Q3H004 | Pauls River | -0.3680 | 612.35 |
| Q4H013 | Tarka River | -0.3313 | 1178.41 |
| Q6H003 | Baviaans River | -0.3359 | 385.03 |
| Q8H008 | Little Fish River | -0.2049 | 972.30 |
| N2H007 | Sundays River | -0.0574 | 1518 |
| N2H008 | Riet River | -0.0372 | 4760 |

Figure 8.3-1 shows an example of the relationship between Flow and TDS for Tarka Bridge Weir (Q4H013). For low flows the TDS is high as there is less water for the salts to be diluted with.

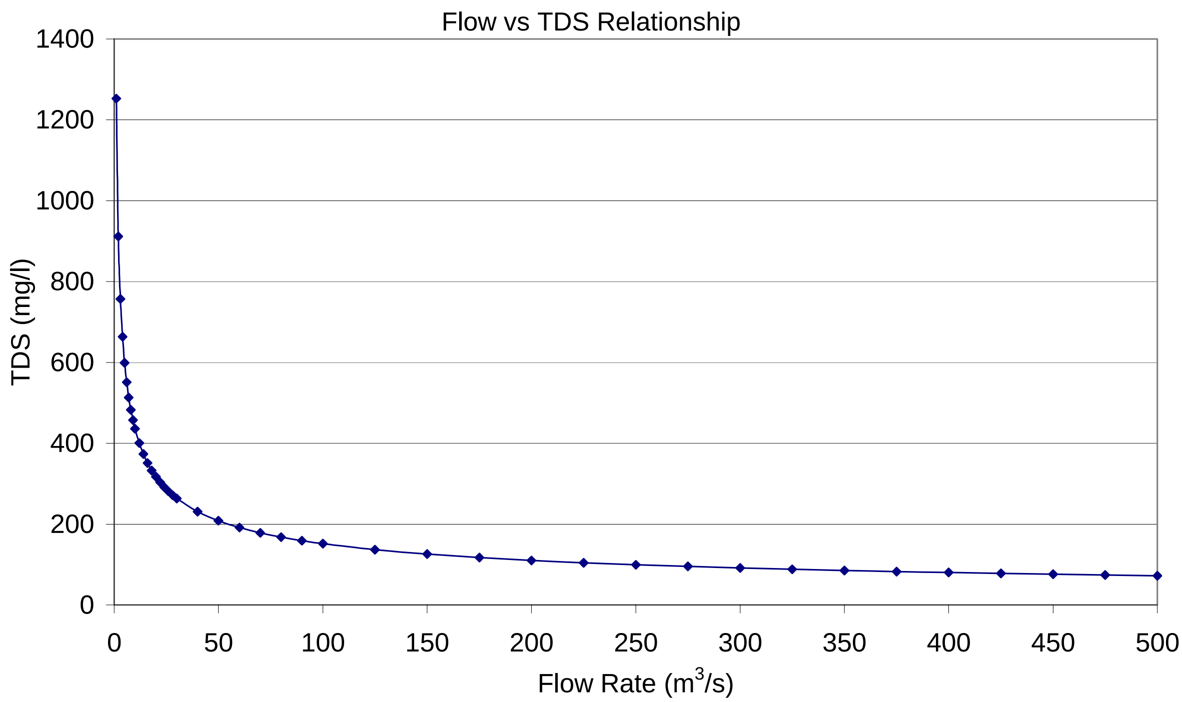



**Figure 8.3-1: Flow vs TDS Relationship for Tarka Bridge Weir (Q4H013)**

As the flow increases, an increase in the TDS can be seen as shown in Figure 8.3-2 taken from Hydras3 for a flood event on the Tarka River. This is the flushing out of the salts from the soil than have been deposited through evaporation. As these salts pass through with the flow the additional flow dilutes the concentration. As the flow recedes, the concentration of salts in the flow increases.

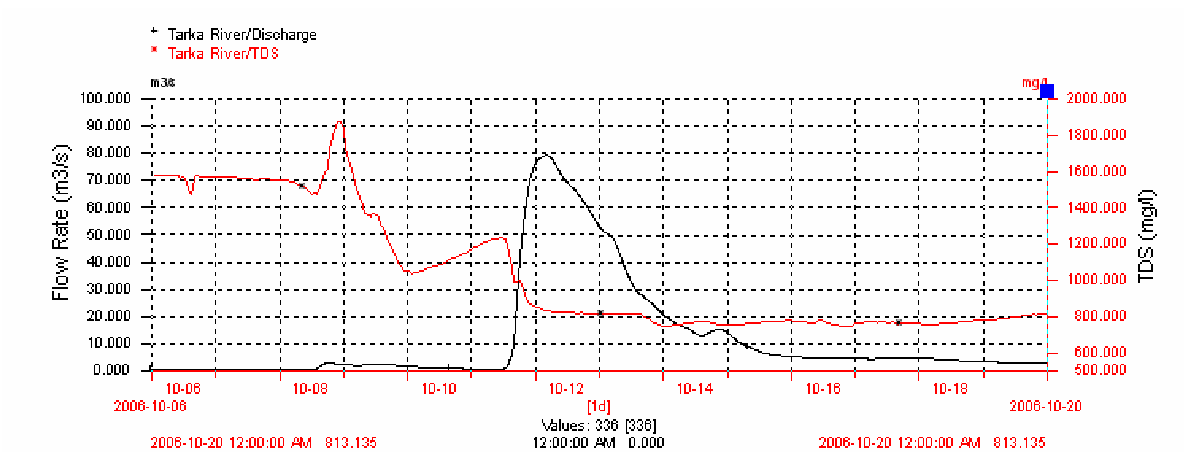



**Figure 8.3-2: Variation of Flow and TDS at Tarka Bridge Weir**

Although the flow – TDS relationship is far more complex than the simple log-log relationship shows, the basic nature of the relationship is modelled. Thus the log-log relationship is used to forecast the TDS from the forecasted flow. Figure 8.3-3 shows the forecast done with this method for the Tarka River tributary. The TOF is the 19th October 2006; the flow recedes smoothly while the TDS increases to the standard base flow TDS given by the log-log relationship.

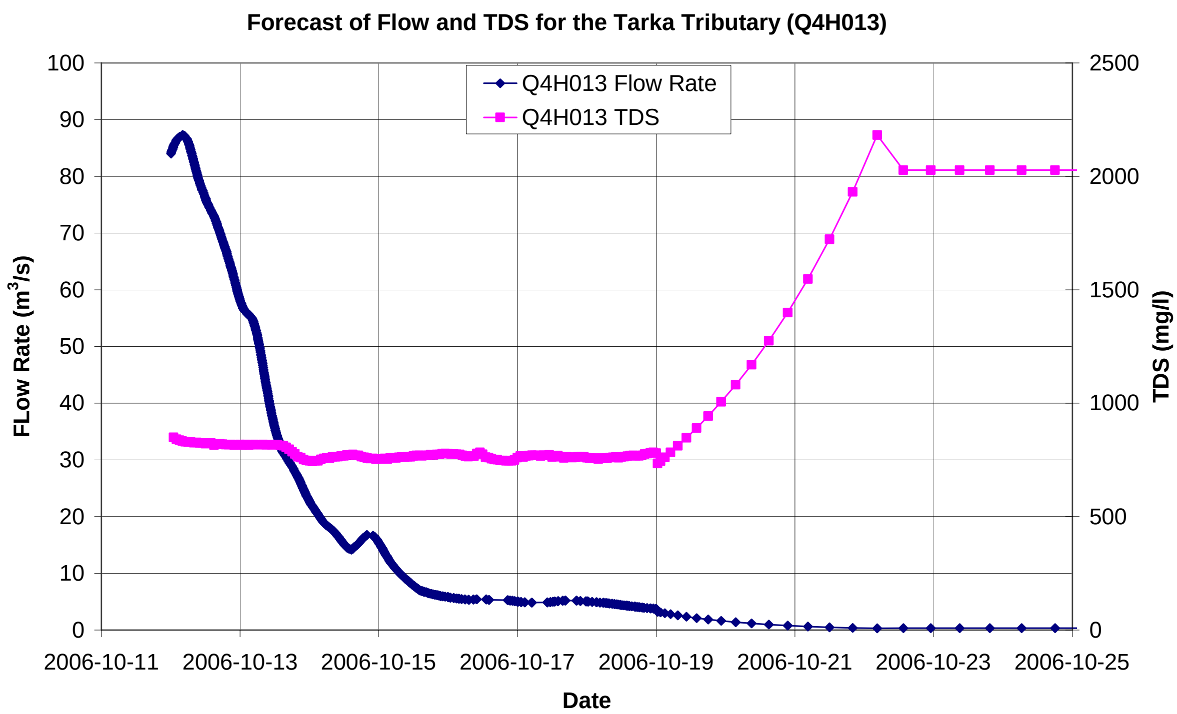



**Figure 8.3-3: Forecast of Tributary Inflow Flow and TDS at Tarka Bridge (Q4H013)**

In this manner, continuous data is provided for the inflow boundaries for the full simulation period as required by the Mike 11 Model.

## 8.4 Data Correction and Adjustment

The measured release from the control structures flow gauging stations have been calibrated by the model to require a different flow rate provided by a flow factor $F_f$. The Real-Time Pre-Processor applies these flow factors to the flow for these stations. These flow factors have been discussed in Chapter 4. Table 8.4-1 shows the stations which have a flow factor along with the factor used for the current model calibration.

**Table 8.4-1: Shows the Flow Factor $F_f$ used for the Measured Release from the Control Structures**

| Station Number | Station Name | $F_f$ |
|---|---|---|
| Q1H014-30 | Teebus OVIS Tunnel Outlet | 1 |
| Q1H022-30 | Grassridge Dam Outlet | 0.9 |
| Q5L002-30 | Elandsdrift Dam River Outlet | 1 |
| Q5H006-30 | Elandsdrift Dam Canal Outlet | 1 |
| Q8R001-30 | De Mistkraal Dam River Outlet | 1 |
| Q8H013-30 | De Mistkraal Dam Canal Outlet | 0.95 |
| N2H010-30 | Darlington Dam Outlet | 1 |

These factors are applied to the flow before the files are saved as a text file in the format required by the Mike 11 model. Each text file is updated every four hours and the previous text file for that stations sensor is over written. An archive is not required as the real-time flow data is stored in the Hydras 3 database where is can be extracted as and when required.

# 9 Pre-Processing of the Abstraction Data

Besides the inflow into the model or the supply side, the demands from the system have to be entered into the model so that the required flow releases from the control structures can be optimised. The demands are given as abstractions at points along the reaches of the scheme and are setup in the network of the Mike 11 model as negative point sources. Each of these abstractions has a flow time series that defines the demanded flow rate for a time period. These demands are requested by the irrigation boards or private users that have licences to abstract water from the Orange-Fish-Sundays Scheme. These licences are issued by the Department of Water Affairs and Forestry and are related to the area of land to be irrigated.

## 9.1 The Current Abstraction Request Process

The abstractors place their weekly order for water by 10am on a Thursday every week. These orders are either faxed or telephoned through to the DWAF office at Uitkeer outside Somerset East. Once all the relevant abstractors have handed in their data, the operator enters this data into the existing FISUN model along with the dam water level readings as per Chapter 3.3. The abstraction input data is kept as a DW683.txt file. These files are overwritten each year. This database has been the basis for the input to the Mike 11 model for the calibration of the new model.

The new model has been designed to limit the operator's transition from the existing model to the new model. For this reason, the collection and input of the abstraction data on a weekly basis will remain as is. As such the new model will require a pre-process program which will be used by the operator to interact with the Mike 11 model. This will simplify the training required as well as the level of skill required to operate the model.

## 9.2 The New Abstraction Request Process

The new pre-processor has been designed within Excel with VBA Macro's running the operations. The user can access different input screens through a main menu. In the main menu page, the operator will select which irrigation week they are working in and the start date. The start date for the week is the Sunday before the week at 12 noon. This is to facilitate certain abstractions that start before the original start time of Monday at 6 am.

The operator then goes through to the request abstraction page. Here requests for each of the 149 abstractions currently in the model can be edited. The abstraction for the previous week is assumed as the default abstraction, this aids the input for those stations whose weekly abstraction is constant. An example of such abstractions is the private pumpers who abstract a constant flow for the whole week. Their abstraction rate is assumed based on the volume of their water licence.

A request is defined as a flow rate in $m^3$/hr for a defined duration from a given start time. Thus an abstractor would ask for say 600 $m^3$/hr for 96 hours starting at 6am on Monday morning. It is possible to request different amounts for different times, such as 600 $m^3$/hr for 48 hours then 400 $m^3$/hr for the following 48 hours. The start time for each abstraction must be entered, ie: 6am Monday and 6am Wednesday. These requests are logged and archived. Once the operator has entered the new data for that abstractor, they select the "schedule requests" button. The requests are tiled together with the more recent request over riding previous requests. This allows for changes to be kept track of. For example: if a request of 600 $m^3$/hr for 60 hours from 6am Monday was followed by a request for 400 $m^3$/hr for 48 hours starting at 6am Wednesday. The 60 hours of 600 will be reduced to 48 hours as the two streams of requests can not coincide. Figure 9.2-1 shows a diagram of how this conversion from requests to schedules operates.

| Request | Flow | Duration | Date | Time |
|---|---|---|---|---|
| 1 | 600 | 72 | 02-Oct | 06:00 |
| 2 | 400 | 48 | 04-Oct | 06:00 |
| 3 | 200 | 24 | 06-Oct | 06:00 |

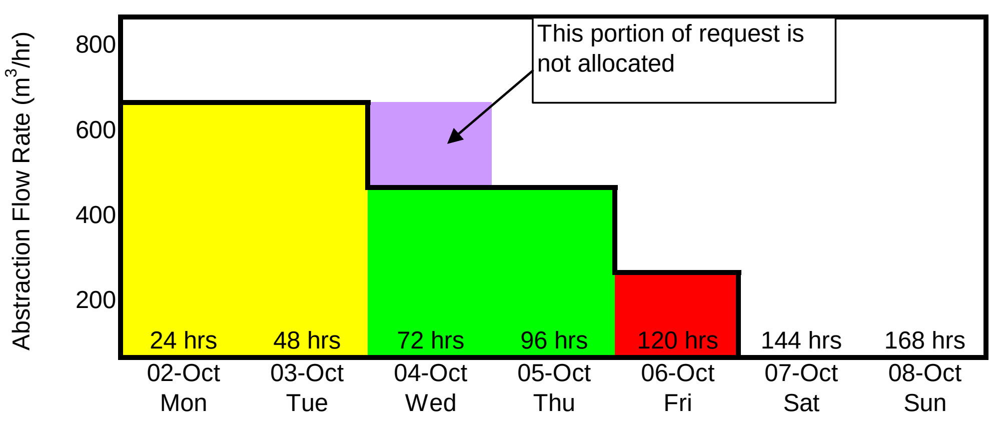



**Figure 9.2-1: Diagram of Requests to Schedules**

The schedules can be checked as a graphic and time series output before they are exported to the Archive.

The last 3 weeks of requests are available to be viewed while the total record of schedules is kept in an archive. This archive can be updated with changes to requests during the irrigation week. Each abstraction is given a lag time. In reality this lag time varies according to the flow rate. The lag time is the average time that a flow release takes to reach the point on the reach where the abstraction is located. The average was taken from average flow velocities for each reach to determine a mean flow travel time or lag. As flow released for an abstraction can not be recalled, no changes are allowed to be made to abstractions that are not given in time. This is an additional ability of the OFS-RT model that the existing model does not possess. Once all the abstractions requests have been edited, the schedule checked, and the schedules exported to the archive, the operator can update the abstraction time series files for the forecast week. This process takes the schedules for each abstraction for the last week and the new week that has just been edited and writes them as a text file in the Mike 11 format. Thus the demand side of the model has been updated for the forecast week.

## 9.3 System Control Parameters

The Abstraction Pre-Processor also allows the operator to edit and interface with certain model parameters. These are: three operation water levels for the dams, maximum TDS limits for different reaches, maximum flow rates for the downstream boundaries, return flow and return flow TDS along the reaches, evaporation characteristics for the dams as well as certain logical operations to turn on or off in the model.

### 9.3.1 Dam Operation Levels

Each dam has three operation levels. There are:

- High Flood Level (HFL)
- Full Supply Level (FSL)
- Minimum Operating Level (MOL)

The HFL is the ultimate highest level that the dam's water level can reach before over topping the Non Overspill Crest (NOC) or endangering the stability and safety of the dam. This level has a high weighting in the optimisation routine as should the model operate the dam above this level, there would be great damage and the dam could fail. The FSL is generally the level of the spillway. If the dam's water level were above this level, the spillway would function and this water would be lost to the system. This is not desirable and should be minimised if possible. During flood events, this might not be possible, though the HFL should never be reached. The MOL is the lowest that the dams water level should be allowed to drop to. If the dam's water level were to drop below this, there would be environmental concerns with regards to fish stocks in the dam and water quality downstream due to the passing of excessive sediments from the dam and/or the level could be too low to release the required discharge.

The model can vary the water level in the dams such that during normal operation the dam should never exceed FSL or drop below the MOL. The operation of the dams is such that the dams should use the water in its storage before calling for more water from upstream dams and ultimately the OVIS Tunnel. Thus if a flood event were to be experienced, the dams would have sufficient capacity to store as much of this "free" water as possible, thus reducing the demand on the OVIS Tunnel and the Orange River catchment.

### 9.3.2 Maximum TDS Limits

DWAF have signed agreements with the various irrigation boards that the water quality in the Little Fish, Groot Vis and Sundays Rivers are not to exceed certain levels during normal operating periods when irrigation is taking place. These levels are set at 600 mg/l for all the rivers in the scheme. To facilitate the operation of the model, these can be adjusted as required to suit changes to the model or specific events such as the filling of Glen Melville Dam.

When Glen Melville Dam is nearing its MOL, or the quality of the water stored has become poor, a re-freshening release is made from Elandsdrift Dam to lower the TDS in the lower Groot Vis at Hermanuskraal to below 300 mg/l. Once this has been obtained, the diversion tunnel from Hermanuskraal to Glen Melville Dam is opened to fill the dam. Thus Glen Melville Dam is re-supplied with good quality water.

### 9.3.3 Maximum Flow Limits for the Downstream Boundaries

The Mike 11 model requires the downstream flow limit, which is the desired flow that should pass out of the system under normal operations. This should be set to supply the environmental flow as well as any additional release that might be required. These limits are set at the downstream model boundaries at Fort Brown Bridge and Korhaansdrift Weir, but at Junction Drift on the Little Fish as well. This is to assist the model in finding a solution. The difference in the flow rate observed at these stations in the model is used by the optimisation routine to re-calculate the flow release from the control structures to limit the release and thus restrict losses from the scheme.

### 9.3.4 Return Flow and Return Flow TDS

The return flow is the ground water that flow into the river system, predominantly from irrigated fields. As such this flow has a high concentration of dissolved salts and can have high TDS values of up to 6000 mg/l. The return flow was one of the calibration variables which do vary seasonably due to the volume of irrigation abstraction and rainfall groundwater return flows. These variations are complex to model and as such have been assumed constant for the entire period based on the dry period calibration. The return flows were calibrated reach by reach as described in Chapter 5.

The concentration of the return flow TDS was used as the calibration factor for the AD module calibration. This was as a result of all the other inputs to the scheme having measured TDS readings. The calibration of the return flow TDS was conducted by Simone Maharaj of Ninham Shand. The values found were based on the studies of the return flow TDS conducted during the 1970's and 1980's (W. Kamish, S. Maharaj) during the development of the Orange-Fish-Sundays Scheme.

These return flow and return flow TDS values are to be re-calibrated from time to time to increase the accuracy of the Mike 11 model.

### 9.3.5 Evaporation Characteristics

The rainfall and evaporation on the dams is taken into account by the Mike 11 model. The hourly rainfall at the dams is received in real-time from the stations at the dams but the evaporation has to undergo some calculations based on the observed water level at the S pan as well as the recorded rainfall and wind etc at the station. For this reason, evaporation can not be given in real-time.

Observations of the variation in evaporation showed that the evaporation varied seasonally as one would expect with higher evaporation being recorded during the summer months and a lower evaporation rate during winter. Thus a sinusoidal function was used to model the fluctuation in the observed evaporation at each of the dams. The equation was as follows:

$$E_m = A \times Sin((x + S) \times B) + C$$ **Equation 9.3-1**

Where:

$E_m$ is the modelled Evaporation

*A* is the Amplitude Factor

*B* is the Frequency Factor

*C* is the Vertical Shift

*S* is the Horizontal Shift

*x* is the Date in units of days

Figure 9.3-1 shows the observed and modelled evaporation at De Mistkraal Dam. One notices that there is some variation due to cloud cover, wind speed, humidity and air temperature, but the suitability of the sinusoidal variation is far more accurate than a constant value. This is confirmed by the smaller standard deviation calculated for the model as opposed to the mean verses the observed data shown in Table 9.3-1.

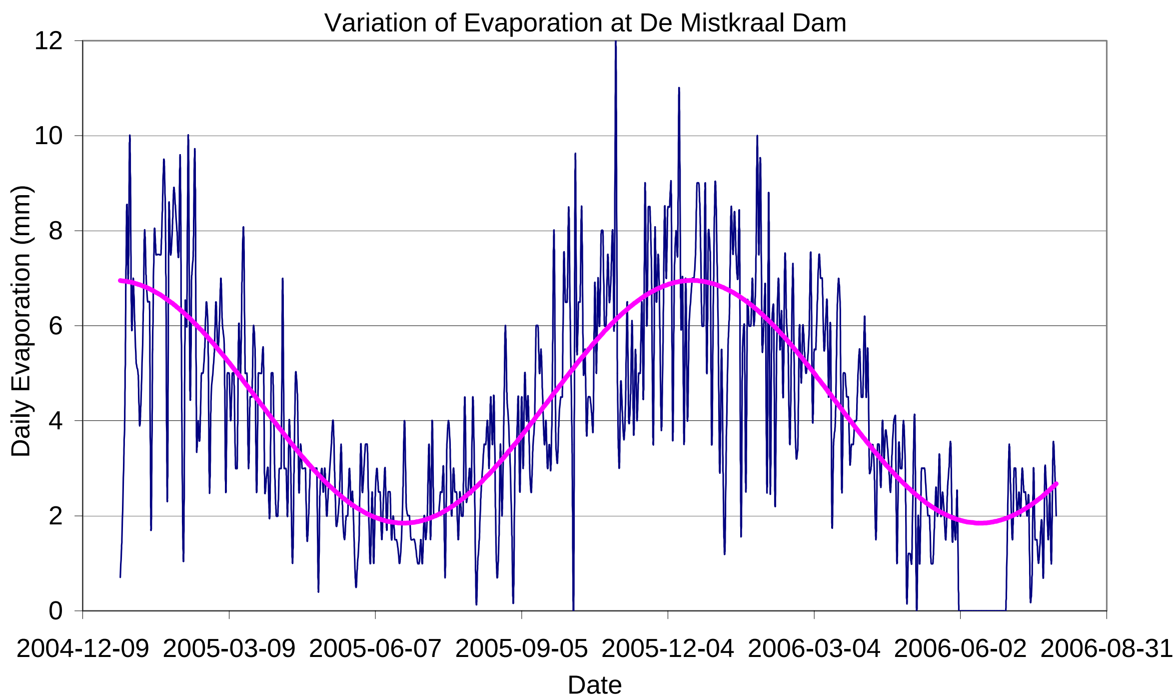



**Figure 9.3-1: Variation of Observed Evaporation vs Modelled Evaporation at De Mistkraal Dam**

**Table 9.3-1: Calibrated Values for each Evaporation Station Model and Statistical Results**

| Evaporation Station | Station Name | A | B | C | S | Mean | Std Dev from Mean | Std Dev from Model |
|---|---|---|---|---|---|---|---|---|
| N2E001 | Darlington Dam | 3.79 | 1.01 | 5.69 | 60 | 5.09 | 1.61 | 1.34 |
| Q1E001 | Grassridge Dam | 2.76 | 1.01 | 4.76 | 60 | 4.56 | 1.44 | 1.11 |
| Q5E001 | Elandsdrift Dam | 2.56 | 1.01 | 5.06 | 60 | 4.73 | 1.30 | 1.07 |
| Q8E002 | De Mistkraal Dam | 2.55 | 1.01 | 4.40 | 60 | 4.05 | 1.35 | 1.04 |

The pan factors for the region have been taken into account and are given in Table 9.3-2.

**Table 9.3-2: Pan Factors for the Eastern Cape Region (B.J. Middleton et al. 1981)**

| Oct | Nov | Dec | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.8 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 0.8 | 0.8 | 0.8 |

As B is the frequency factor and there are 360 degrees to one phase of the Sin curve and 365.25 days in the year, the frequency factor was 365.25 / 360 = 1.01458. The horizontal shift factor was based on finding the average of when the model and the observed data where in phase. This was found to be within 5 days of the optimum for each individual station and so was chosen to be a constant for the model. The factors A and C define the range that the modelled evaporation varies within and can vary from station to station. These parameters are to be re-calibrated against observed data from time to time to confirm that they are still applicable.

### 9.3.6 Logical Controls

There are certain aspects of the model operation that can be varied from the pre-processor. This reduces the need for the operator of the model to interact directly with the Mike 11 model. These logical control operations define if the Tunnel from Gariep Dam is open or closed for maintenance, if the model should divert additional flow from Gariep Dam if Vanderkloof Dam is spilling and whether the Auxiliary Gates on Darlington Dam are permanently open or closed and operational.

For two weeks each year during June, the OVIS Tunnel is closed for routine maintenance of the valves and controls. As such the dams are to be maintained at their FSL before the closure of the Tunnel so that any demand can still be supplied if required. Once the Tunnel is re-opened, the system returns to normal operations.

On the Orange River, any spillage at Vanderkloof Dam downstream of Gariep Dam is seen as a loss to the Orange River system. As such, this excess water could be used to flush out the dams and rivers, thus lowering the TDS levels in the system and reducing the likelihood of additional refreshing releases being made to maintain the TDS levels in the rivers. This additional flow would also aid in maintaining the river channels, keeping them clear of obstructions. One must bear in mind that the maximum discharge of the OVIS Tunnel is in the range of 57 $m^3$/s and the Little Fish Canal is 20 $m^3$/s. These are not excessively high flow rates which could not be seen as being a safety risk to those along the banks of the rivers. The flushing and filling of the dams would be done from the top down. Grassridge Dam would be filled first, from here the additional flow would be passed on to Elandsdrift Dam, here the maximum diversion to the Little Fish Canal would be diverted to De Mistkraal Dam, any additional flow would be released to the Groot Vis River. At De Mistkraal, once the dam was filled to FSC, the flow would be diverted to Darlington Dam.

At Darlington Dam, the Auxiliary Gate is kept open due to concerns over the ability to operate the Main Spillway and Auxiliary gates in time during a flood event. Thus the auxiliary gate is essentially a free overflow spillway. Nevertheless, this situation could change and as such the model needs to be adapted for this eventuality. To save the Mike 11 Model from being edited directly which would be more complex and require expert skills, the operator can switch between having the gate kept open or being operational via the Pre-Processor.

## 9.4 Reports and Statistics

The Pre-Processor keeps track of the water request for each abstractor and provide warnings to the operator when the available volume for the abstractor has neared the maximum allowed according to their licence allocation. This will allow the operator to warn the abstractor that they are running short of their annual allocation. In addition, this pre-processor will keep track of the total volume of water requested for abstraction any period of time, allowing the operator to report on the efficiency of the model and the operation of the OFS-RT scheme.

The abstraction pre-processor is the main focus point of the interaction between the operator and the model. It has been designed to provide the maximum required flexibility and choice with regards to the operation of the model without being too complex and providing too many options and adjustments. Having the pre-processor written in Excel macros allows the code to be easily adjusted and upgraded by any programmer or experienced excel user, thus reducing the projects reliance on external professional.

Any changes to the Mike 11 model would require someone with experience in Mike 11. Simple changes to the Manning's n value for calibration proposes would be simple to conduct but other more complex changes such as the addition of reaches to extend the model, or the addition of abstraction points would require these to be included into the optimisation routine that has been developed by DHI for the OFS-RT project. As such, either a highly skilled Mike 11 operator or DHI would have to make such changes. For this reason, Dummy Abstractions have been inserted along the reaches where there are large spaces between the next abstractions. Thus if a new abstraction is located near to an existing abstraction, these can be summed to model a combined required demand. Else if a new abstraction is not located near an existing abstraction, one should find a Dummy abstraction near (within 5km) of the abstraction point that could be used. This should suffice until such time as there is sufficient need to add them to the model.

# 10 Operation of the Mike 11 FloodWatch Model

FloodWatch is a module that has been developed by DHI to co-ordinate the operation of Real-Time systems. From within a GIS framework, any model and or batch file can be set to run at different time intervals. This program is highly complex and requires intensive training for proper operation. For this reason, the Pre-processor has been designed in conjunction with DHI to interact with the model to limit the need for the operator to interface directly with the FloodWatch module. FloodWatch controls the operation of the Mike 11 model in that it controls when it is run, what time series files are used and where the results are saved.

## 10.1 Mike 11 Optimisation and FloodWatch

The calibrated Mike 11 model was sent to DHI to be improved by their specialists at DHI. Once this was completed, DHI constructed the optimisation routines for the model. This enables the model to adjust flow releases from one simulation to the next to find the optimum flow release for each of the 7 control structures to suit the model limits and requirements. The first stage is to lag each abstraction up the reach to the first control structure and then the lumped flow from here up to the next control structure and so on up to the OVIS Tunnel Outlet. Figure 10.1-1 shows an example of different hydrographs for each control structure for one week's abstraction demands.

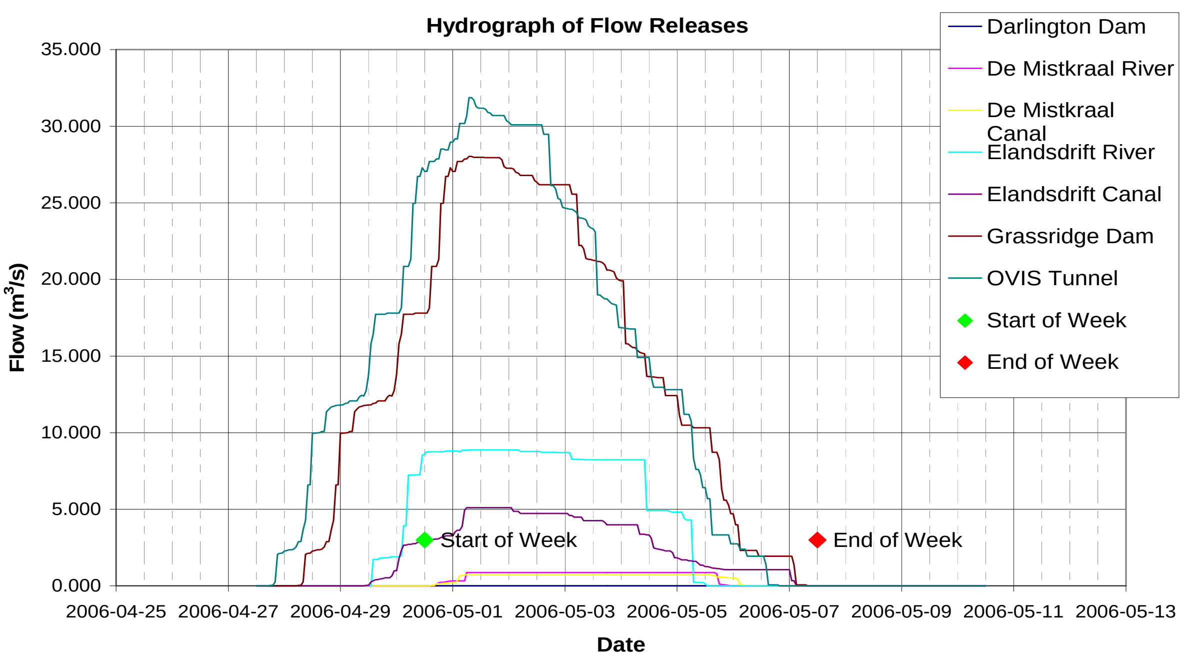



**Figure 10.1-1: Assumed Flow Release Hydrographs for each Control Structure**

One will notice that flow is required to be released three days prior to the start of the irrigation week. This is to accommodate those abstraction points that are a long way downstream of the control structure. These lagged hydrographs do not take into account the variation in flow rate and time of travel associated with various flow rates in a reach, thus the flow released along with many other flows will reach the abstraction point sooner than if it were released along with far fewer releases. This is due to the hydraulic radius for the flow as mentioned in Chapter 4 and shown by Equation 4.2-3.

Due to this variation in travel time with flow rate, the optimisation routine re-configures the release hydrograph according to differences between the simulation and the required flow rates, water levels and TDS levels of the model. These differences are weighted such that those with a greater importance are corrected more strongly than those with a lesser importance. The calibration of these weighing factors is extremely complex and requires years of experience and skill. For example the weighing factor for the positive error on the HFL (simulated water level > HFL) will carry a greater factor than that for the positive error on the FSL difference, yet during a flood event, one can't reduce the release from a dam to keep the lost water flow rate at the lower boundary condition to a minimum. This complex balance is controlled by the weighting factors on the difference terms at various positions along the system.

Once the optimised release hydrograph for each of the control structures has been calculated, this is given as an output file to a website which can be accessed by the operators of the control structures. In addition to this output, other reports and outputs are available from the FloodWatch model. These are given in Table 10.1-1.

These outputs are used to refine the calibration of the model, calculate the volume of water used and allocated for monthly reports and to check on the operation of the system. These reports are available from the FloodWatch user station or uploaded to the website for access by remote users. This ability is necessary for senior officials within the Department of Water Affairs and Forestry to view the effectiveness of the model and the functions of the OFS Scheme.

**Table 10.1-1: List of Reports Available from FloodWatch**

| # | Description | Data Presentation | Site Location | Access |
|---|---|---|---|---|
| 1 | Forecast of required discharge hydrograph from control structures | Plotted and table | OVIS Tunnel outlet | User and internet |
| | | | Grassridge Dam outlet | |
| | | | Elandsdrift Dam to river outlet | |
| | | | Elandsdrift Dam to canal outlet | |
| | | | De Mistkraal Dam to river outlet | |
| | | | De Mistkraal Dam to canal outlet | |
| | | | Darlington Dam outlet | |
| 2 | Last 7 days and next 4 days from Time of Forecast (TOF) of flow and TDS | Plotted and table | At any position in model | User |
| | | | At all gauging sites | User and internet |
| 3 | Simulated vs Observed flow and TDS | Plotted and table | At all gauging sites | User |
| 4 | Additional flow released to compensate for water quality | Plotted and table | OVIS Tunnel outlet | User and internet |
| | | | Grassridge Dam outlet | |
| | | | Elandsdrift Dam to river outlet | |
| | | | Elandsdrift Dam to canal outlet | |
| | | | De Mistkraal Dam to river outlet | |
| | | | De Mistkraal Dam to canal outlet | |
| | | | Darlington Dam outlet | |
| 5 | Additional flow released due to spill at Vanderkloof Dam | Plotted and table | OVIS Tunnel outlet | User and internet |
| | | | Grassridge Dam outlet | |
| | | | Elandsdrift Dam to river outlet | |
| | | | Elandsdrift Dam to canal outlet | |
| | | | De Mistkraal Dam to river outlet | |
| | | | De Mistkraal to canal outlet | |
| 6 | Animation of flow, water level and TDS for each reach for last 7 days and next 4 days from TOF | Graphical output | OVIS Tunnel to Grassridge Dam | |
| | | | Grassridge Dam to Elandsdrift Dam | |
| | | | Elandsdrift Dam to De Mistkraal Dam | |
| | | | Elandsdrift Dam to Junction of Groot and Little Fish Rivers | |
| | | | De Mistkraal Dam to Junction of Groot and Little Fish Rivers | |
| | | | Junction of Groot and Little Fish Rivers to Fort Brown Bridge (end of model) | |
| | | | De Mistkraal Dam to Darlington Dam | |
| | | | Darlington Dam to Korhaansdrift (end of model) | |
| 7 | Flow released just to supply abstraction demand | Plotted and table | OVIS Tunnel outlet | User |
| | | | Grassridge Dam outlet | |
| | | | Elandsdrift Dam to river outlet | |
| | | | Elandsdrift Dam to canal outlet | |
| | | | De Mistkraal Dam to river outlet | |
| | | | De Mistkraal Dam to canal outlet | |
| | | | Darlington Dam outlet | |
| 8 | Water level and TDS in each dam (Water level in masl, gauge plate reading and % of FSC) | Plotted and table | Darlington Dam | User and internet |
| | | | Grassridge Dam | |
| | | | Elandsdrift Dam | |
| | | | De Mistkraal Dam | |
| 9 | Accumulated weekly release for abstractions | Table | OVIS Tunnel outlet | User and internet |
| | | | Grassridge Dam outlet | |
| | | | Elandsdrift Dam to river outlet | |
| | | | Elandsdrift Dam to canal outlet | |
| | | | De Mistkraal Dam to river outlet | |
| | | | De Mistkraal Dam to canal outlet | |
| | | | Darlington Dam outlet | |
| 10 | Accumulated monthly release for abstractions | Table | OVIS Tunnel outlet | User and internet |
| | | | Grassridge Dam outlet | |
| | | | Elandsdrift Dam to river outlet | |
| | | | Elandsdrift Dam to canal outlet | |
| | | | De Mistkraal Dam to river outlet | |
| | | | De Mistkraal Dam to canal outlet | |
| | | | Darlington Dam outlet | |

## 10.2 Integration of the Pre-Processors with the FloodWatch Model

Figure 10.2-1 shows the flow of information for the two inputs through the two pre-processors to the FloodWatch model. The flow of information from the real-time stations is controlled automatically by auto jobs set up at each GSM station, in Hydras 3 and through batch jobs in FloodWatch. The data from the abstractions and the system control parameters is entered in by the operator on a weekly basis and as abstraction data is made available to the operator by the abstractors along the reaches. Each Thursday the following weeks' abstractions are entered into the next irrigation week. This data can be edited during the week but changes are only permitted if there is sufficient time for the changed release pattern to be taken into effect.

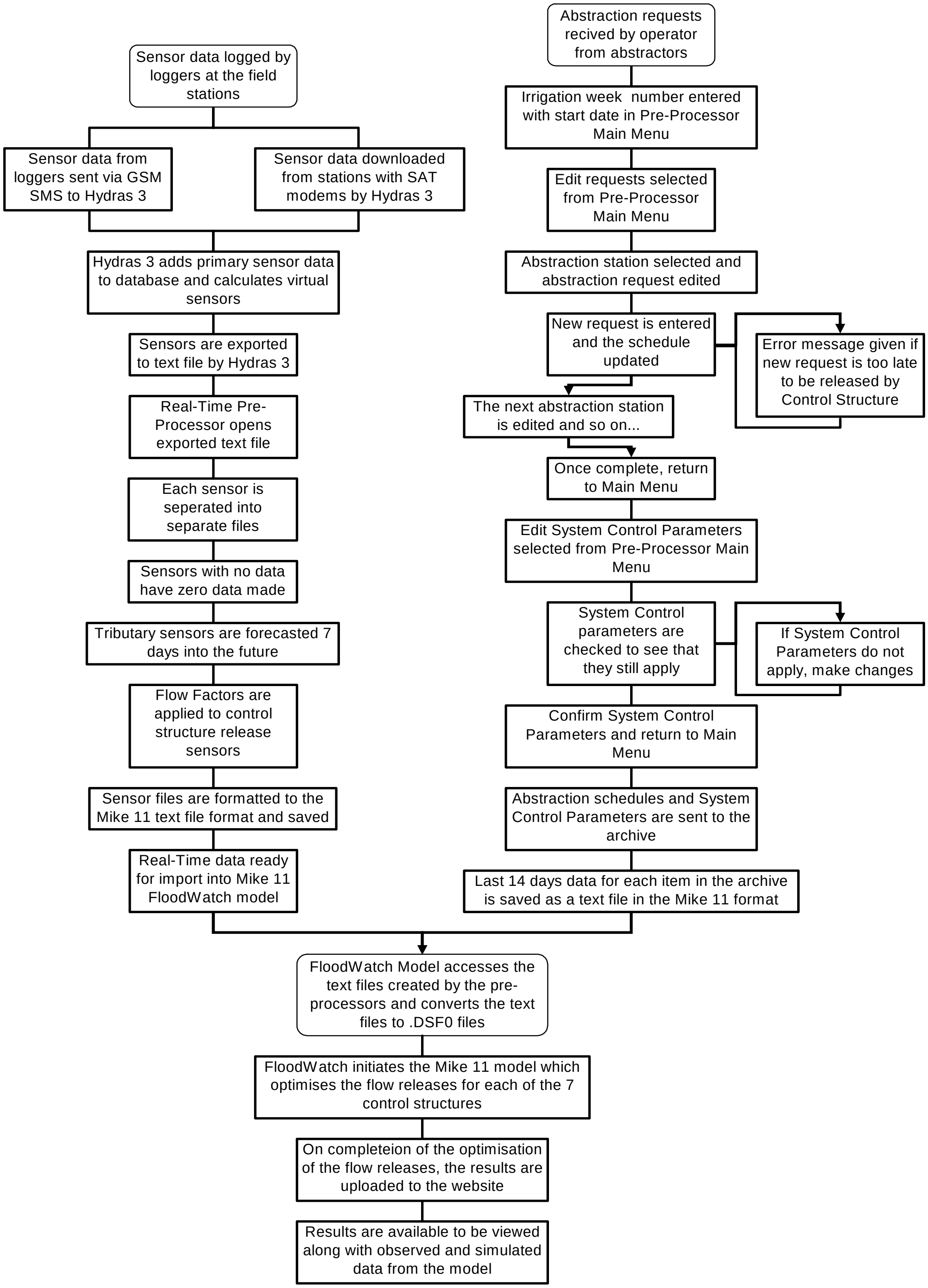



**Figure 10.2-1: Flow Chart of Informationthrough the OFS-RT Model**

## 10.3 Error Checking and Robustness

Errors and poor or impossible data is checked for and eliminated at all stages of the model. This redundancy has been built in to reduce the possibility of incorrect data being introduced into the model.

The raw data from the field stations is checked by Hydras 3 to confirm that the values are within the minimum and maximum allowable values. A maximum gradient of change is also specified to control the possibility of a great change from one level to another. There are further checks on the virtual sensors. Here the discharge can only be positive according to the stage-flow table which defines the flow according to the stage level. For the TDS virtual sensors, the TDS is only calculated if there is positive flow and if the EC raw data is justified. The limits have been set from observed data and structural and physical limitations on the system.

The Real-Time Pre-Processor again checks that the flow is greater than 0. For the tributary inflows, the base flow limit is applied if the flow is equal to or less than 0. This ensures the models stability and robustness through simulations. The model has been calibrated with these base flows included in the model along with the return flows. The Real-Time Pre-Processor creates data for sensors that have no data for the period of evaluation. The data is assumed to be 0. If this is a tributary station then this is transformed to the base flow limit. The FloodWatch model will assume that these 0 values are not valid and ignore them. There are three different types of Real-Time data being provided to the FloodWatch model. The first is input data; this is data which is used as an input to the model such as a tributary inflow. The second is data assimilation data; this is data which the model should try to correct itself to. For example the water level in that dams. The model should always show the actual water level in the dam to prevent incorrect operations based on the dam’s water level from being modelled. These two sets of data are the most important. If a tributary station was to fail to send through data, the real-time pre-processor would assume that the flow was the base flow. If there was actually a flood event, this would not be known until the flow passed through a gauging weir further downstream or when the observed water level in the dam down stream was seem to increase above that of the simulated water level. For this reason the simulated water level has to be corrected to that which is observed at the dams. If the observed water level in the dam was not being sent through to the real-time pre-processor, the model would have to continue based upon the simulated water level. For this reason, manual visual checks on the dam water level are still required to confirm the accuracy of the real-time data. There is no room for complacency.

The last type of real-time data is check data.  These are the flow gauging stations which can be used as data assimilations points but are mostly to assist in the calibration of the model and operations based within the optimisation routines of the model.  Should these stations not send through their data, the model will continue with the simulation.

A check list is available to the operator who must check which GSM stations have sent through their SMS’s to allow those that are not performing to be rectified at the earliest opportunity.  The model outputs are to be examined for variations between the simulated and observed data so that errors in the system can be noticed as soon as possible, any unusual variation if flow rates or water levels are to be investigated and if a problem exists, this should be corrected.

# 11 Mike 11 Optimisation Model

## 11.1 Proposed Optimisation Rules for the Model

The Optimisation rules for the new OFS-RT model have been set up to allow for as much storage in the dams as possible to be available for the storage of the rainfall runoff. The principle is simply that the more natural water used for irrigation reduces the need to transfer more water from the Orange River. To this end, the future operation of the dams will see the MOL of the dams is lowered and the operation level kept closer to the MOL as surety of supply is provided by the storage in Gariep Dam. Currently the dams are conservatively operated near their FSL and the water level is kept as close to the FSL as possible. This reduces the storage available for the storage of tributary inflows. Table 11.1-1 gives the current operational levels along with the proposed levels for each of the four dams.

**Table 11.1-1: Current and Proposed Dam Operation Levels**

| Dam Operation Levels (masl) | Current | | | Proposed | | |
|---|---|---|---|---|---|---|
| | MOL | FSL | HFL | MOL | FSL | HFL |
| **Grassridge Dam** | 1054.64 | 1054.78 | 1056.04 | 1054.00 | 1055.00 | 1056.04 |
| **Elandsdrift Dam** | 741.15 | 742.19 | 744.00 | 741.15 | 742.19 | 744.00 |
| **De Mistkraal Dam** | 550.12 | 550.96 | 556.76 | 548.00 | 550.96 | 556.76 |
| **Darlington Dam** | 243.02 | 243.20 | 247.19 | 240.00 | 243.00 | 247.19 |

In addition to this, excess water in the system is past on down the chain of dams to Darlington Dam. In this way, once a dam's storage capacity has been met, the additional water can be stored in the next dam down the scheme. Once Darlington Dam is at Capacity, the additional water can be used to flush the river reaches below De Mistkraal and Elandsdrift dams. This proposed operation is most useful in the event of there being additional water available in the Orange River. In the event of a spill at Vanderkloof Dam on the Orange River downstream of Gariep Dam, the model will signal for the Outlet works at Teebus OVIS to open, releasing the flow rate of the spill or the maximum flow capacity of the OVIS Tunnel to the top of the scheme. The dams are first filled from Top to Bottom (Grassridge then Elandsdrift then De Mistkraal then Darlington). This will refresh the dams with water with a low salinity as well as flush out the river reaches improving the quality of the water in the scheme.

Once the spill is over, the flow through the Tunnel is reduced back that which is required for abstractions and standard operation. The dams are to be drawn down to their MOL first to provide the additional storage for natural inflows before the additional flow from upstream is brought in to meet the demands. This drawing down of the dams is done by the demand for water downstream of the dam thus the only flow to pass the downstream boundaries should be the required environmental release, additional release to maintain required TDS levels or large flood events during dam spills.

## 11.2 Initial Test Simulations of the Optimisation Model

Four test simulations of the new optimisation model were conducted. These were as tabled below in Table 11.2-1

**Table 11.2-1: Optimisation Test Simulation Conditions**

| Test | Period | Dam Levels |
|---|---|---|
| **Run 1** | Dry period from 12/9/205 to 19/9/2005 | Current Dam Levels |
| **Run 2** | Wet period from 27/2/2006 to 5/3/2006 | Current Dam Levels |
| **Run 3** | Dry period from 12/9/205 to 19/9/2005 | Proposed Dam Levels |
| **Run 4** | Wet period from 27/2/2006 to 5/3/2006 | Proposed Dam Levels |

The same initial conditions were used for each of the four tests. The aims of the four tests were to determine if the optimisation model was operating as planned, and secondly to determine if the new proposed operation levels for the dams would result in a net saving in transferred water.

The results showed that although there were some problems in the weighting of the optimisation terms, the basic management of the scheme was in order. Each of the results are discussed in detail in the following text.

### 11.2.1 Dam Storage Capacity Results

For each of the simulations the water level at Grassridge Dam was observed to drop towards the MOL as shown in Figure 11.2-1. This is in line with the requirements of the optimisation model to draw down the dams to their MOL. In Run 1, the water level is drawn down below the MOL of 1054.64 masl. This shows one that the weighting factor on this optimisation is not great enough to maintain the water level within the correct limits.

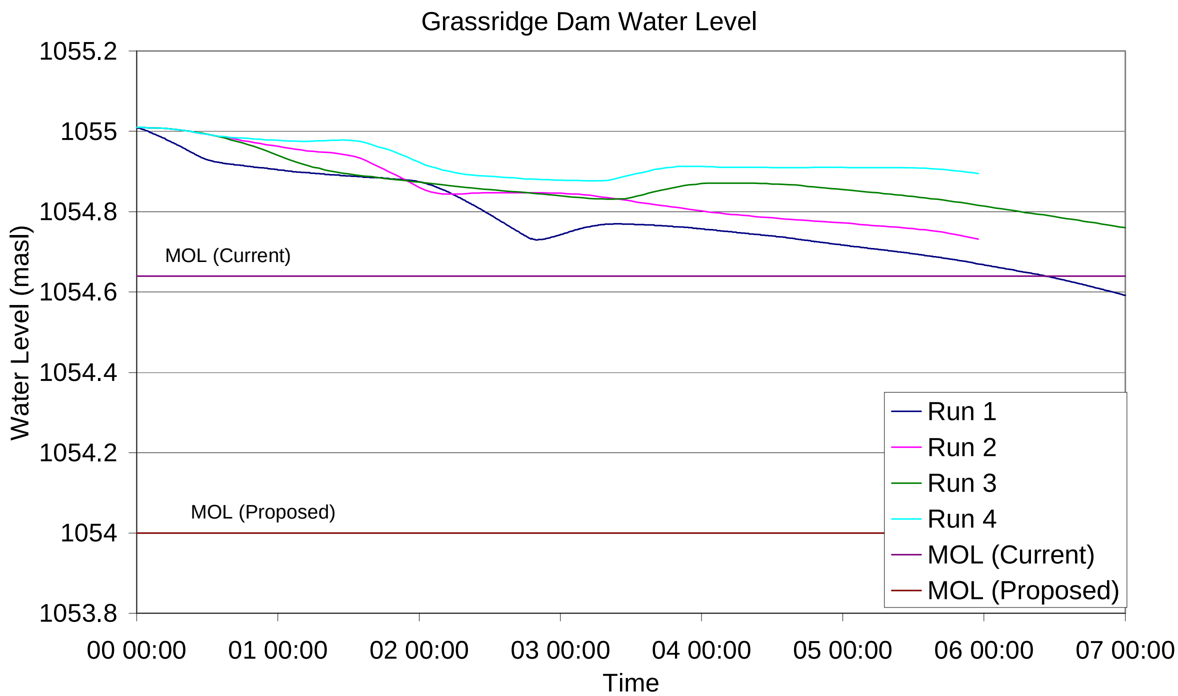



**Figure 11.2-1: Water Level at Grassridge Dam for Simulation Runs of the Optimisation Model**

Figure 11.2-2 shows the water levels for each of the simulation runs at Elandsdrift Dam. Due to the smaller storage capacity of this dam, the simulation shows how the water level is first drawn down to the MOL them allowed to increase up to the FSC. In Run 3 we can see how the model tries to restrict the water level below the FSC. This "bouncing" of the water level is undesirable and further refinement of the optimisation model is required. The increase in the water level back to the FSC in Runs 1 and 3 is not desired. These simulations are during the dry period when there is little to no additional inflow from the tributaries. As such the water level in the dams should be maintained at the MOL to be able to store any additional inflow from tributaries. Simulation Runs 2 and 4 have a tributary inflow upstream of Elandsdrift Dam which would result is the dam water level increasing as shown in Figure 11.2-2.

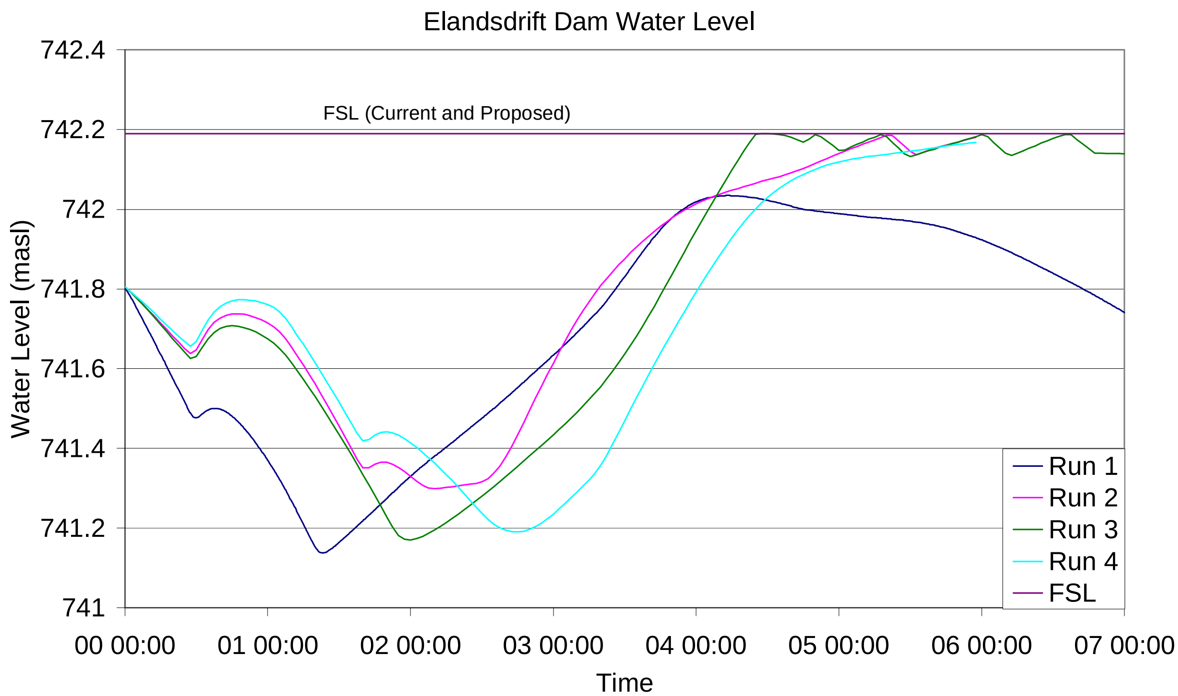



**Figure 11.2-2: Water Level at Elandsdrift Dam for Simulation Runs of the Optimisation Model**

There are problems with the operation of De Mistkraal Dam. The MOL is currently 550.12 masl and the proposed MOL is 548 masl. The initial dam level is below this and, as shown in Figure 11.2-3, falls to well below the bottom level of the dam. This is not desirable and is as a result of the pull of water from Darlington Dam. This is an operational rule to divert extra water to Darlington Dam if available, but the weight on this rule is too high. The Mike 11 model has a powerful numerical processor which can add in water to a system to maintain the numerical stability of the computations. This has upset the natural running of the scheme and is creating an un-natural situation at De Mistkraal Dam.

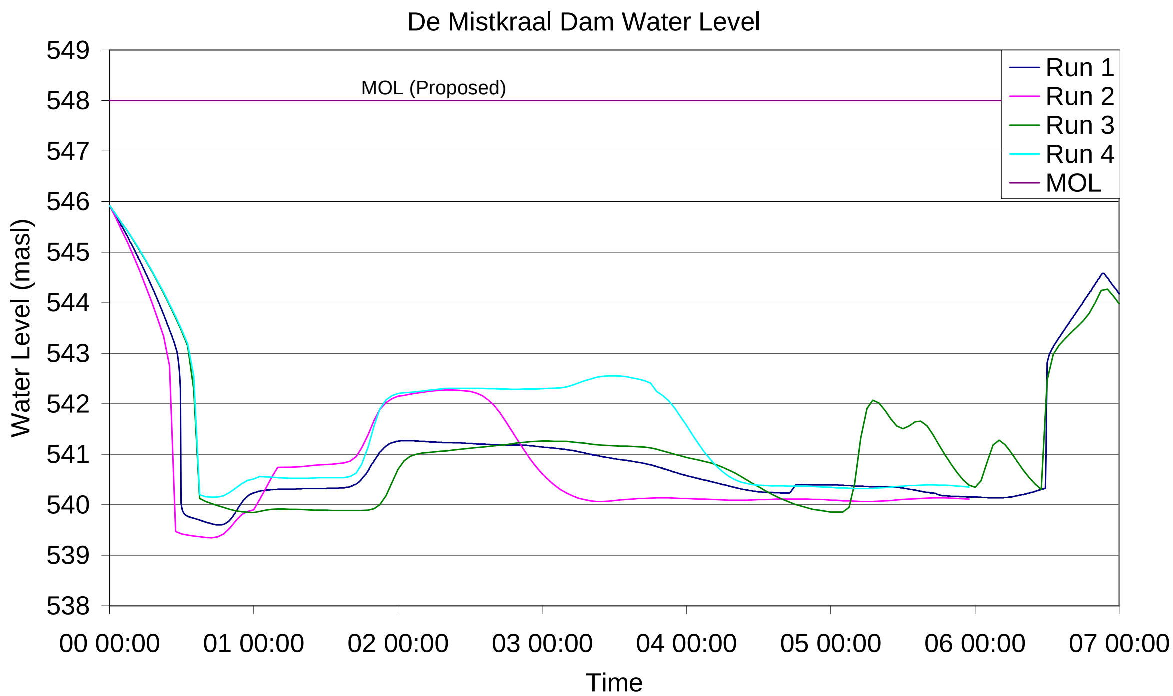



**Figure 11.2-3: Water Level at De Mistkraal Dam for Simulation Runs of the Optimisation Model**

Figure 11.2-4 shows the flow rate into De Mistkraal Dam (shown in Blue) and the flow rates out of the dam (Canal Release in Red, River Release in Green) and the dam water level shown in Black.

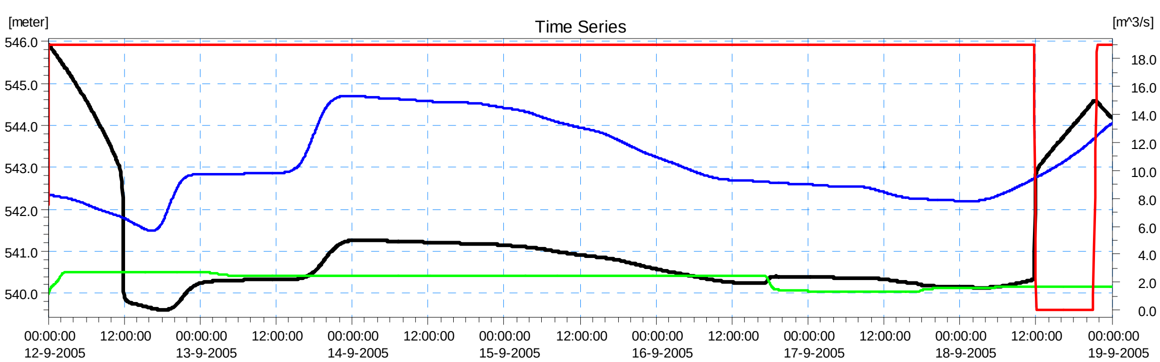



**Figure 11.2-4: Flow Rate Into and Out of De Mistkraal Dam along with the Water Level for Run 1**

It is clear that the outflow from the dam is constantly greater than the inflow which would result in the water level falling continuously over the simulation period, but this is not observed. This problem has been taken up with DHI who are to find a solution.

Figure 11.2-5 shows the increase in water level at Darlington Dam for the four simulation runs. The water level rises from the initial level continues for the duration of the simulation runs. This is due to the continuous transfer of water from De Mistkraal Dam which is seen as un-necessary and incorrect. This transfer should only be made for runs 2 and 4 during the wet period once Elandsdrift and De Mistkraal has been filled to FSC, which has not taken place in the case of De Mistkraal Dam.

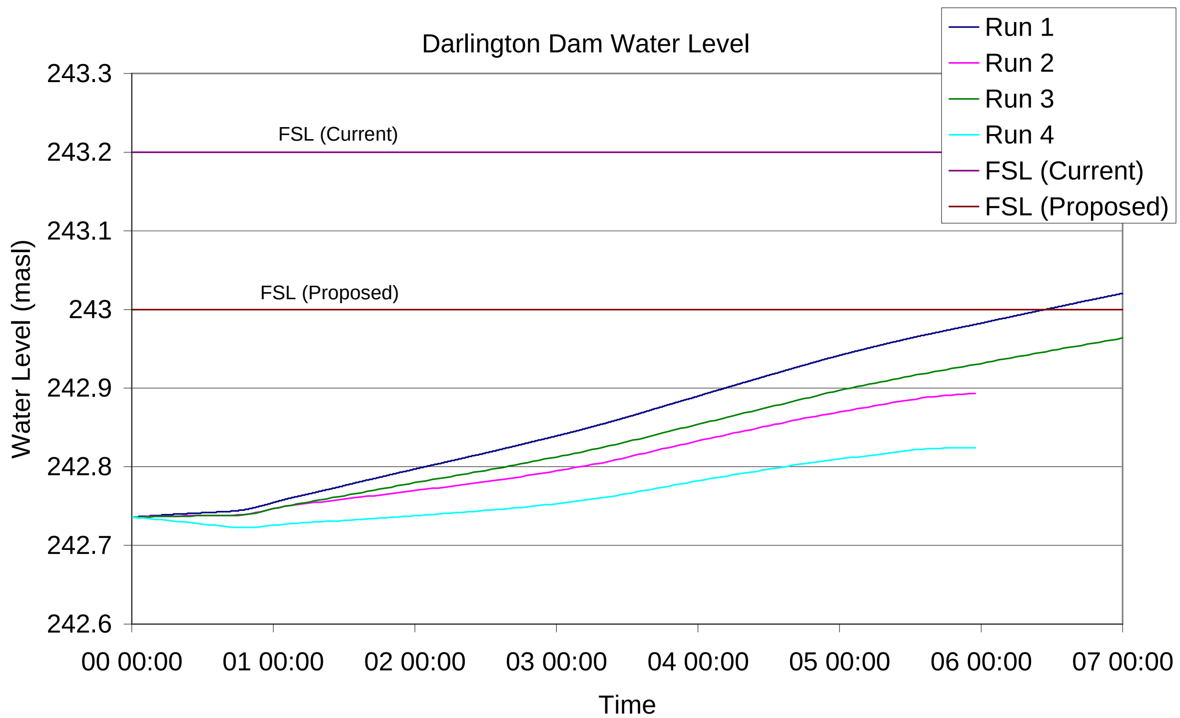



**Figure 11.2-5: Water Level at Darlington Dam for Simulation Runs of the Optimisation Model**

Table 11.2-2 shows the total release from each of the Control Structures for the optimisation simulations. Figure 11.2-6 shows a graphical representation of these releases. It was expected that the values for the releases of runs 3 and 4 would be less than runs 1 and 2, but this was not the case. The evaluation of the use of the flow is not as simple as just looking at the volume transferred from the Orange River as the change in storage at the dams and river reaches need to be taken into account as well. Figure 11.2-7 shows the change in storage for each of the dams for each simulation run. Over all there is greater storage in the scheme at the end of the simulations than at the beginning thus there is a net inflow into the scheme. Interestingly there is a greater increase in storage with the current dam operation levels than for the proposed operation levels along with smaller releases from the control structures. As a result, it is concluded that the current operation levels seem to be more efficient than the proposed operation levels. This is surprising as logic would have the proposed levels being more efficient.

**Table 11.2-2: Volume of Releases Made from each Control Structure for the Four Simulation Runs**

| Name | Run 1 | Run 2 | Run 3 | Run 4 |
|---|---|---|---|---|
| Teebus OVIS | 7,075,818 | 5,745,945 | 10,224,345 | 8,961,342 |
| Grassridge Dam | 12,344,231 | 8,595,546 | 12,300,681 | 9,408,090 |
| Elandsdrift Dam River Release | 1,673,053 | 300,398 | 1,298,484 | 5,490 |
| Elandsdrift Dam Canal Release | 7,712,584 | 4,899,273 | 6,707,333 | 6,292,642 |
| De Mistkraal Dam River Release | 1,331,788 | 463,180 | 33,804 | 21,618 |
| De Mistkraal Dam Canal Release | 10,839,805 | 9,739,973 | 10,765,972 | 9,739,973 |
| Darlington Dam | 1,997,279 | 3,314,617 | 3,483,410 | 5,068,696 |
| **Total** | 42,974,558 | 33,058,932 | 44,814,029 | 39,497,851 |

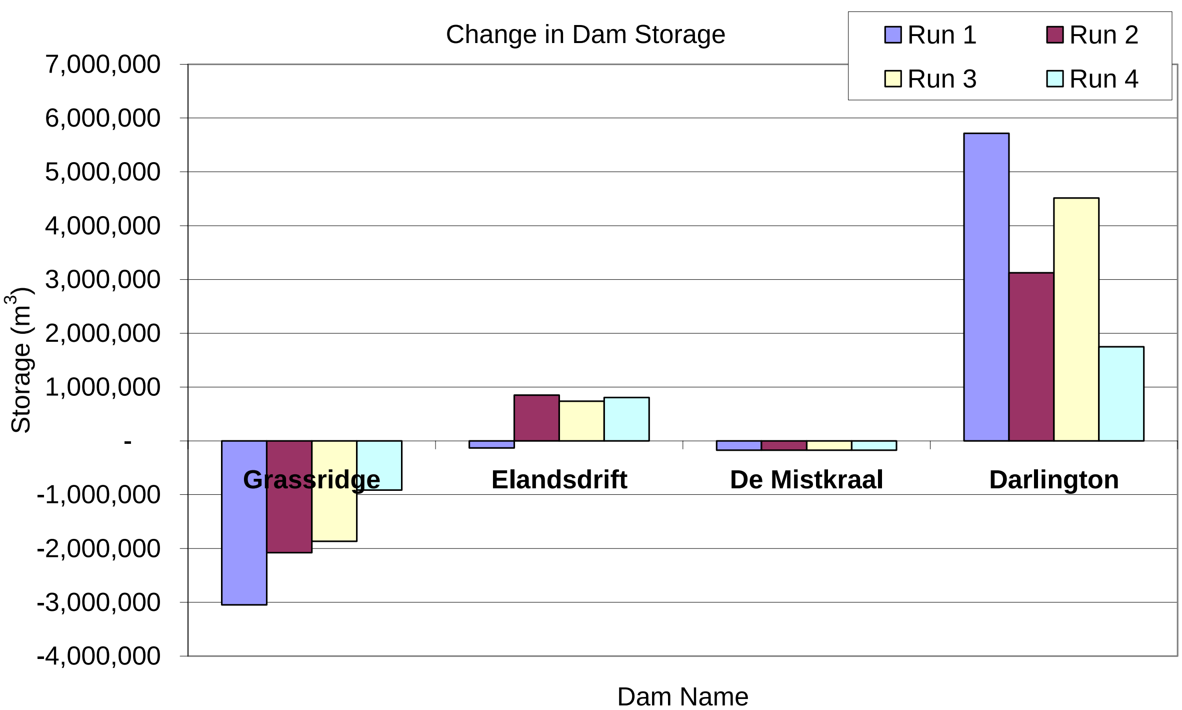



**Figure 11.2-6: Change in Storage for each of the Dams for the Four Simulation Runs**

The volumes of the abstractions from the reaches for the two periods used in the optimisation simulation runs have very similar values which is consistent with the use along the scheme. These are shown in Table 11.2-3. The abstractions have been subtracted from the inflow into the reach. The return flow and tributary inflows have also been taken into account in the analysis.

**Table 11.2-3: Abstractions for Irrigation and Domestic use for each Reach per Simulation Run ($m^3$)**

| Name | Run 1 | Run 2 | Run 3 | Run 4 |
|---|---|---|---|---|
| Teebus to Grassridge | 1,280,612 | 1,311,296 | 1,280,612 | 1,311,296 |
| Grassridge to Elandsdrift | 4,923,096 | 6,007,746 | 4,923,096 | 6,007,746 |
| Elandsdrift to Sheldon | 2,949,470 | 3,106,423 | 2,949,470 | 3,106,423 |
| Elandsdrift to De Mistkraal | 1,220,040 | 1,090,410 | 1,220,040 | 1,090,410 |
| De Mistkraal to Junctiondrift | 462,600 | 408,600 | 462,600 | 408,600 |
| De Mistkraal to Darlington | 312,840 | 312,840 | 312,840 | 312,840 |
| Darlington to Korhaansdrift | 3,076,370 | 2,472,288 | 3,076,370 | 2,472,288 |
| **Total** | 14,225,028 | 14,709,603 | 14,225,028 | 14,709,603 |

As mentioned the wet period simulation runs 2 and 4 include tributary inflows on the Groot Vis River. Table 11.2-4 shows the variation between the volumes of inflow per tributary for the different simulation runs.

**Table 11.2-4: Tributary inflow volumes per Tributary for the Simulation Runs ($m^3$)**

| Name | Run 1 | Run 2 | Run 3 | Run 4 |
|---|---|---|---|---|
| Klein Brak | - | - | - | - |
| Groot Vis | 61,412 | 4,185,220 | 61,412 | 4,185,220 |
| Pauls | 105,186 | 63,580 | 105,186 | 63,580 |
| Tarka | 214,738 | 1,083,667 | 214,738 | 1,083,667 |
| Baviaans | 771 | 1,740,522 | 771 | 1,740,522 |
| Little Fish | 121,594 | 292,969 | 121,594 | 292,969 |
| Sundays | - | - | - | - |
| Riet | - | - | - | - |
| **Total** | 503,701 | 7,365,958 | 503,701 | 7,365,958 |

This additional inflow is mostly into the Groot Vis upstream of Elandsdrift Dam. The FSC of Elandsdrift dam is 7 million $m^3$ as the dam is initially at approximately 75% of FSC, the volume of tributary inflow is greater than the storage in the dam. This additional inflow is diverted via the Little Fish Canal to De Mistkraal Dam. As seen in Figure 11.2-3, there is an increase in the water level at De Mistkraal Dam at the end of the second day. The true effect of this transfer is not possible due to the trouble in the simulations at De Mistkraal.

The downstream outflows at the three locations on the scheme are shown in Table 11.2-5.

**Table 11.2-5: Outflows from the Scheme at the Three Downstream Boundaries ($m^3$)**

| Name | Run 1 | Run 2 | Run 3 | Run 4 |
|---|---|---|---|---|
| Past Sheldon (Groot Vis) | 616,604 | 40,190 | 59,237 | 39,942 |
| Past Junctiondrift (Little Fish) | 422,507 | 288,534 | 7,218 | 6,876 |
| Past Korhaansdrift (Sundays) | 163,268 | 1,085,917 | 975,132 | 2,240,600 |
| **Total** | 1,202,379 | 1,414,641 | 1,041,587 | 2,287,418 |

The results from the analysis of the outflows from the scheme are mixed. For the outflows on the Little Fish and Groot Vis Rivers the proposed operation of the dam's results in less water being lost out of the system, though on the Sundays River, the flow out of the downstream boundary is far greater. This is not as planned and the model was supposed to stop the transfer of water to Darlington Dam when the dams FSL at 242.925 masl had been reached and spill down the Fish River. This has not happened and as such more investigation into the optimisation model is required and more refinement of the model must take place before it is considered operational.

# 12 Conclusion and Recommendations for Future Study

The Orange-Fish-Sundays Scheme is one of the primary transfer schemes in South Africa which provides the water required to sustain an entire region which would otherwise be arid and unproductive. Nevertheless the volume of water transferred must be restricted to a minimum to allow for this water to be used in other regions where there is also scarce water supply.

## 12.1 Conclusions

The new OFS-RT Model based on Mike 11 fully hydrodynamic, real-time distributed model will provide DWAF with a tool to enable them to maximise the efficiency of the water use is the region. The future use of natural tributary inflows along with closer monitoring of the actual stream flows into and out of the scheme will enable the future generations of irrigators and domestic users a more efficient yet equally reliable source of water. This being said, there are many areas where the new model can be improved upon. Initially the model requires further development on the optimisation routine that has been shown to have flaws in its operation of De Mistkraal and Darlington dams. This work is currently underway at the University of Stellenbosch. Field tests of the optimisation model are to be conducted in January 2007 and the full model with the FloodWatch Real-Time component is to be setup for initial operation, testing and refinement from February through to March of 2007. This stage of the project will continue for the next six months until the model is fully operational and the new operators have been fully trained to deal with the various functions and conditions that the model will need to deal with.

The collection of the real-time data has been automated by the OTT Hydras 3 program and a quality checking procedure is being drawn up to alert the DWAF Hydro office in Cradock of stations that have failed to send data or that are not operating properly. The final development of the Pre-Processors for both the real-time data and the weekly abstraction data have been completed with the user interface currently being upgraded. This front end program will reduce the training requirements for the operators of the scheme leaving the complex Mike 11 and FloodWatch models to be tampered with as seldom as possible.

## 12.2 Recommendations

The ability for the irrigators to change their abstraction requests during the week will have an increased effect on the efficient operation of the scheme. The disciplined use of this function will result in millions of cubic meters of water being saved from needless transfer as more of the natural tributary inflow will be stored for use. This aspect of the future operation of the OFS-RT scheme will require the cooperation of the local irrigation boards. To date these irrigation boards have not yet been included in the development of the OFS-RT scheme. The development of internet access for the irrigators to their requests for water is a further possible development for the scheme. This will allow the irrigators more control over how and when they will have water available to them. The proper use of this system may only be fully attained once the billing practice for the use of the water is changed. The current licence system allocates a total volume per hectare at a fixed annual fee. Those irrigators, whose pumps or sluices are metered, are billed for what they use and not for what they request to a maximum of their annual allocation. The other irrigators who are not metered are assumed to take their water allocation evenly distributed through 50 weeks of the year, allowing for the 2 weeks shutdown period at the end of June. The new OFS-RT model will allow DWAF greater ability to keep track of the difference between the irrigators requests for water and the actual use there of. As mentioned during the calibration and verification of the model, the non use of requested water has a negative impact on the accuracy of the model. The operator will be able to notice the difference in the observed and simulated flow at gauging stations and dam water levels, which would alight to irregular use of the transferred water.

The Real-Time forecasting of the tributary inflows has not taken into account the past and future rainfall on the catchment. Currently just the actual flow last recorded is used for the tributary inflow forecast as detailed in Chapter 8. This is an area where more research and development can take place. Increased knowledge of the tributary inflows will allow for the flow in the system upstream of the point of tributary inflow to be reduced, thus reducing the total transfer from the Orange River. The inclusion of a more developed rainfall runoff model to forecast the tributary inflows more accurately and to predict the rise of the flow due to the measured rainfall will increase the time that the model has to react to this additional inflow and as such store more of this natural water. This will ultimately reduce the annual transfer from Gariep Dam. In addition this will provide more time for flood warning to those along the banks of the river system.

The new model has been developed to satisfy the requirements as set out in the introduction and in Chapter 2. The fully hydrodynamic model provides a full simulation of the entire scheme providing the operator with the tools to more efficiently operate the control structures during standard operation as well as provide a warning and simulation for flood events. This will provide the operator with the ability to mitigate the possible disastrous effects of a flood and keep to the three rules of dam operation during a flood. These are that the Dam must be full at the end of the flood, the outflow must not exceed the inflow to the dam and the resulting flood wave should limit the economic damage and loss of life downstream (J.J. van Heerden, D.B. du Plessis).

The new OFS-RT model has the potential ability to redefine the way that control structures are operated for the release of irrigation flows to downstream users. The ground work for this model has been presented in this thesis.

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**Real-Time Model Development for the Full River System**

# Appendix

# Appendix A: Sites Surveyed as part of the OFS-RT Project

34 sites along the Fish and Sundays Rivers were surveyed to attain information on the river channel sections. This data was used to estimate a river channel cross section assumption to be used in the model. Table A.1 sows a list of which sites were surveyed:

**Table A-1: List of Sites Surveyed for Information on Channel Cross Sectional Shape**

| Site # | River Name | Comments / Details | Completed |
|---|---|---|---|
| 1 | Ovistunnel Outlet | Elevation and Section of Canal Start after Gauging Weir Q1H014 | Yes |
| 2 | Teebus | Maryland | Yes |
| 3 | Groot Brak | Brak River Off take. 100m Down Stream of Weir | Yes |
| 4 | Groot Brak | Woolwyn | Yes |
| 5 | Groot Vis | Katkop Weir. Up Stream | Yes |
| 6 | Groot Vis | Katkop Weir. Over Weir | Yes |
| 7 | Groot Vis | Katkop Weir. Down Stream | Yes |
| 8 | Groot Vis | 100m Downstream from Weir at Marlow | Yes |
| 9 | Groot Vis | Cloverfield | Yes |
| 10 | Canal from Elandsdrift Weir | Elevation and Section of Canal Start after Gauging Weir Q5H006 | Yes |
| 11 | Cookhouse Tunnel Intake | Elevation and Section of Tunnel Intake | Yes |
| 12 | Cookhouse Tunnel Outlet | Elevation and Section of Canal from Tunnel Outlet | Yes |
| 13 | Groot Vis | Eastpoort | Yes |
| 14 | Groot Vis | Harefield | Yes |
| 15 | Little Fish | Kranskloof | Yes |
| 16 | Canal from De Mistkraal Dam | Elevation and Section of Canal Start after Gauging Weir Q8H013 | Yes |
| 17 | Little Fish | Ripon | Yes |
| 18 | Groot Vis | Junction Drift. 100m Upstream of confluence | Yes |
| 19 | Little Fish | Junction Drift. 100m Upstream of confluence | Yes |
| 20 | Great Fish | Junction Drift. 100m Downstream of confluence | Yes |
| 21 | Great Fish | Junction Drift. Over Weir | Yes |
| 22 | Skoenmakers | Middlewater | Yes |
| 23 | Sundays | Schiet Hoogte (by Plat Berg) | Data from previous survey |
| 24 | Sundays | Breeknek | Data from previous survey |
| 25 | Canal from Korhaansdrift | Elevation and Section of Canal Start after Gauging Weir N4H006 | Yes |
| 26 | Canal to Scheepers Vlakte | Elevation and Section of Canal Start before it drops into the Dam | Yes |
| 27 | Great Fish | Carlisle Bridge | Yes |
| 28 | Great Fish | Hermanus Kraal Weir. Tunnel Elevation and Section | No-Tunnel under water |
| 29 | Great Fish | Hermanus Kraal Weir. Up Stream | Yes |
| 30 | Great Fish | Hermanus Kraal Weir. Over Weir | Yes |
| 31 | Great Fish | Hermanus Kraal Weir. Down Stream | Yes |
| 32 | Glen Melville Tunnel Outlet | Elevation and Section of Tunnel Outlet | No-Tunnel under water |
| 33 | Glen Melville Tunnel Outlet | Elevation and Section of Canal to Ecca River | Yes |
| 34 | Groot Vis | Kleinkanaan | Yes |

## Site 1: OVIS Tunnel Outlet Canal

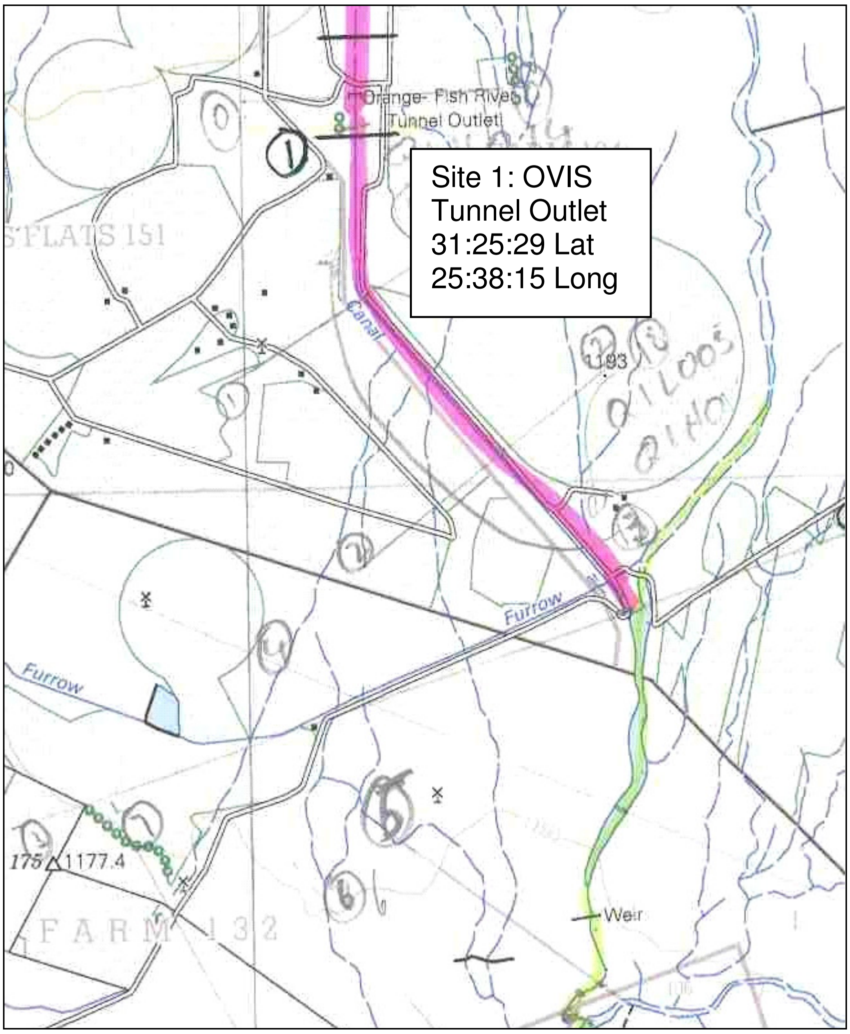
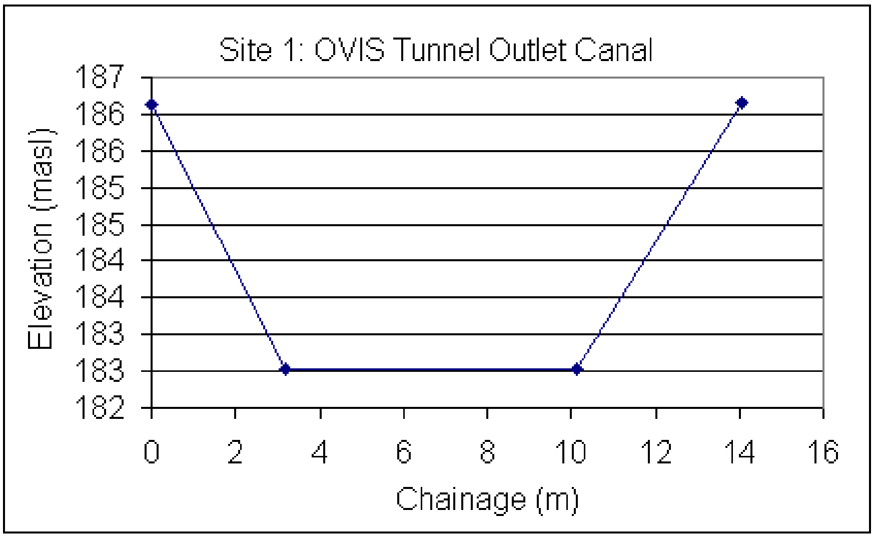



**Figure A-1: Location of OVIS Tunnel Outlet**

**SITE 1**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 186.12 |
| 3.19 | 182.53 |
| 10.12 | 182.53 |
| 14.04 | 186.15 |



# Site 2: Teebus River at Maryland

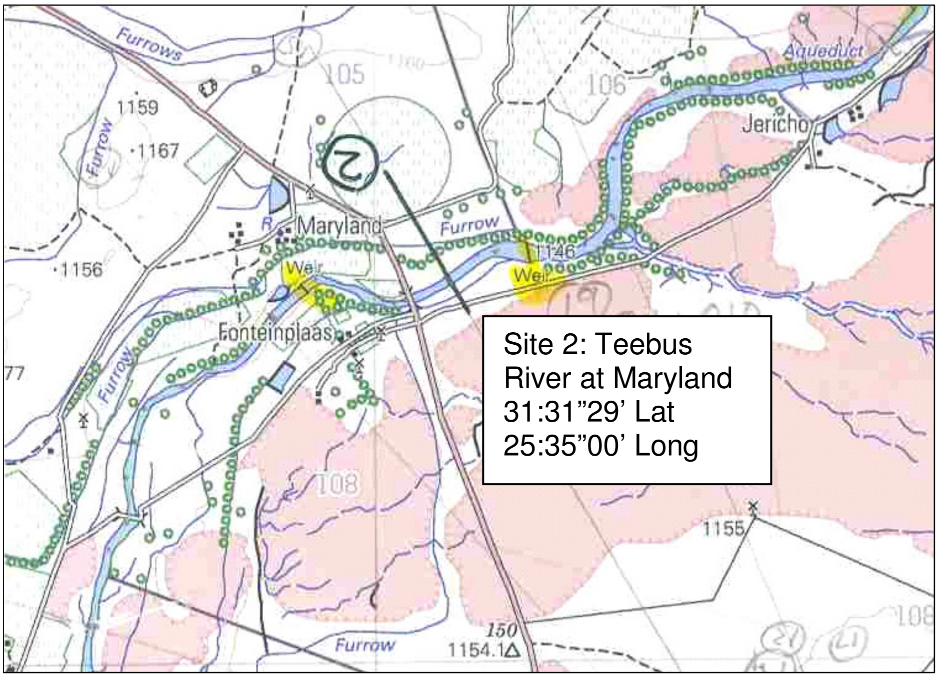
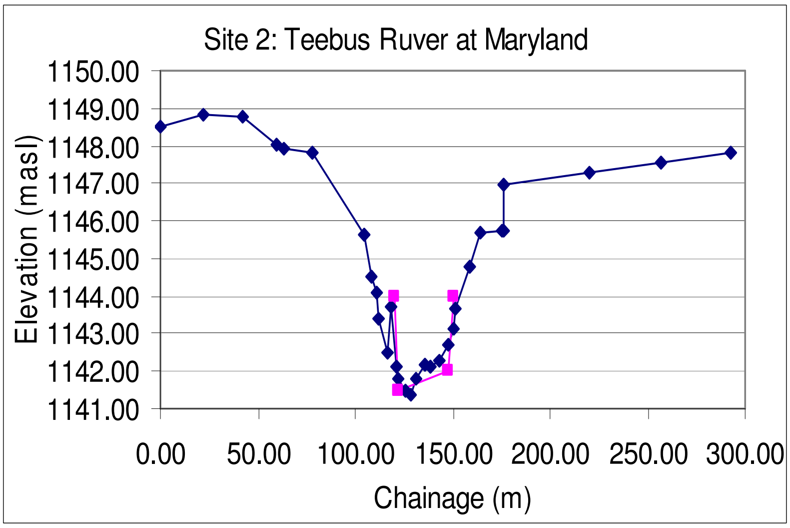



**Figure A-2: Location of Teebus River at Maryland**

**SITE 2**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L. | CH. | G.L. |
| 0.00 | 1148.49 | 120 | 1144.00 |
| 59.35 | 1148.04 | 122 | 1141.50 |
| 77.52 | 1147.84 | 148 | 1142.00 |
| 104.37 | 1145.62 | 150 | 1144.00 |
| 107.81 | 1144.50 | | |
| 111.07 | 1144.10 | | |
| 116.68 | 1142.48 | | |
| 118.63 | 1143.69 | | |
| 120.69 | 1142.13 | | |
| 122.47 | 1141.81 | | |
| 125.76 | 1141.48 | | |
| 128.69 | 1141.39 | | |
| 131.40 | 1141.82 | | |
| 135.69 | 1142.19 | | |
| 138.71 | 1142.11 | | |
| 143.17 | 1142.28 | | |
| 147.59 | 1142.68 | | |
| 150.34 | 1143.16 | | |
| 158.26 | 1144.79 | | |
| 163.81 | 1145.71 | | |
| 176.19 | 1146.98 | | |
| 293.02 | 1147.82 | | |



## Site 3: Groot Brak River at Brak River Off-take

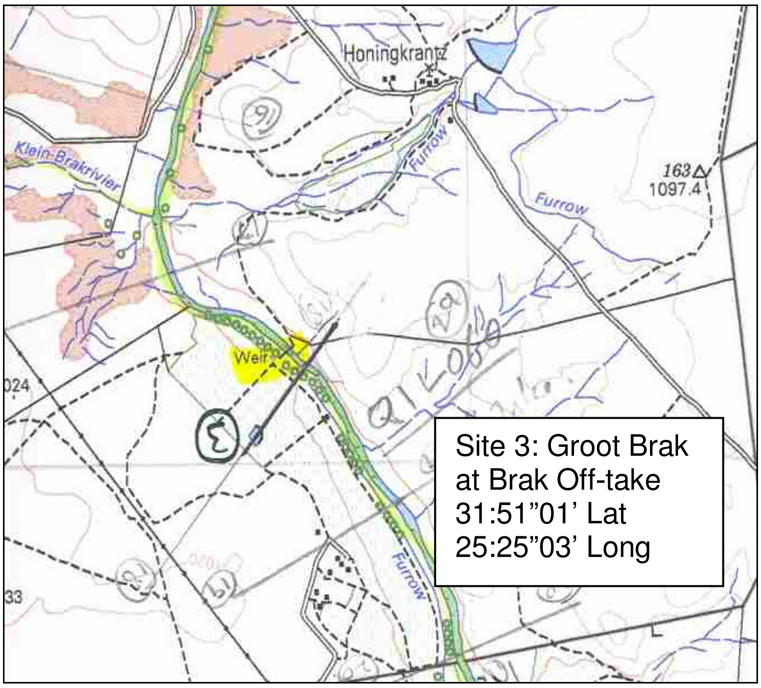
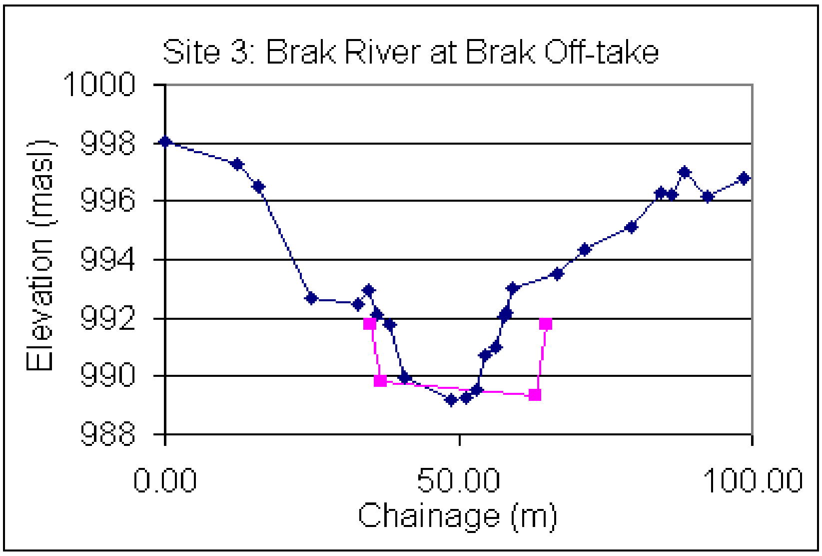



**Figure A-3: Location of Groot Brak River at Brak Off-take**

**SITE 3**

| Measured CH. | Measured G.L. | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 998.03 | 35 | 991.80 |
| 15.93 | 996.53 | 37 | 989.80 |
| 25.03 | 992.65 | 63 | 989.30 |
| 32.94 | 992.44 | 65 | 991.80 |
| 34.54 | 992.96 | | |
| 36.17 | 992.10 | | |
| 38.29 | 991.79 | | |
| 40.97 | 989.98 | | |
| 48.66 | 989.18 | | |
| 51.38 | 989.27 | | |
| 52.98 | 989.54 | | |
| 54.50 | 990.70 | | |
| 56.34 | 990.98 | | |
| 57.86 | 992.05 | | |
| 59.29 | 993.03 | | |
| 66.74 | 993.53 | | |
| 71.34 | 994.37 | | |
| 79.36 | 995.09 | | |
| 84.30 | 996.28 | | |
| 86.29 | 996.21 | | |
| 88.38 | 997.00 | | |
| 92.30 | 996.18 | | |
| 98.69 | 996.77 | | |



## Site 4: Groot Brak River at Woolwyn

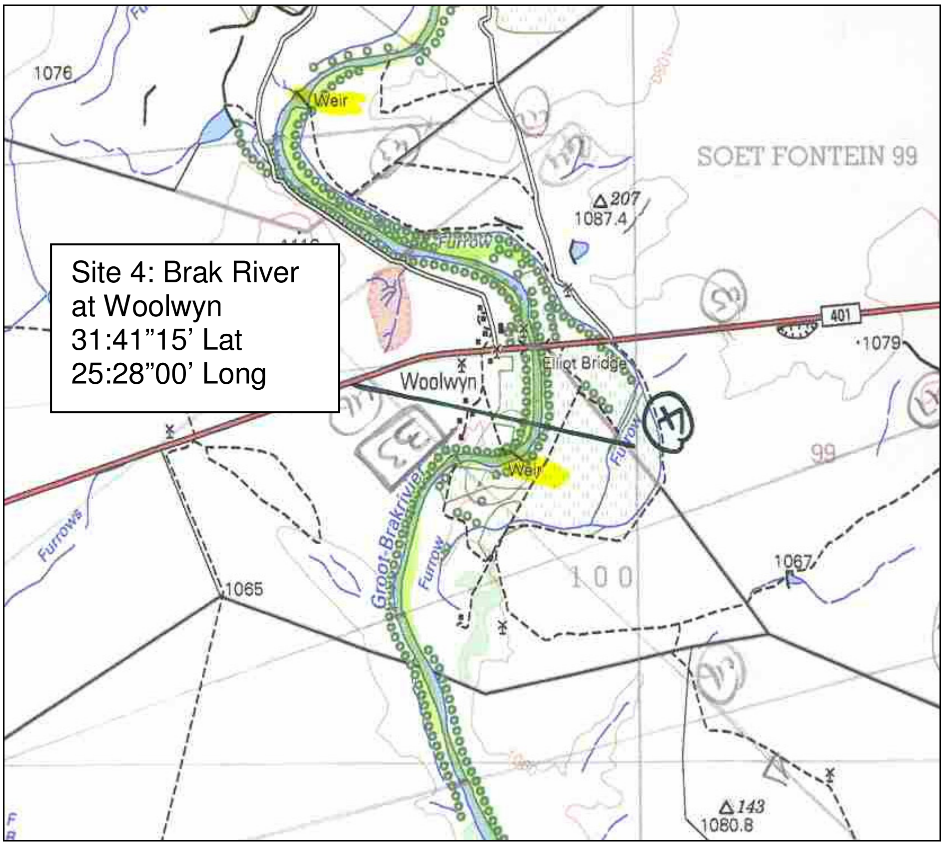



**Figure A-4: Location of Groot Brak River at Woolwyn**

**Site 4**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L. | CH. | G.L. |
| 0 | 1064.47 | 281 | 1060.75 |
| 6.19 | 1062.91 | 283 | 1058.25 |
| 43.79 | 1062.2 | 308 | 1058.75 |
| 75.23 | 1062.04 | 310 | 1060.75 |
| 109.52 | 1062.01 | | |
| 138.62 | 1062.05 | | |
| 178.19 | 1061.84 | | |
| 206.32 | 1062.08 | | |
| 232.57 | 1062.49 | | |
| 253.51 | 1062.24 | | |
| 270.13 | 1062.46 | | |
| 278.3 | 1061.84 | | |
| 281.91 | 1061.92 | | |
| 287.84 | 1057.84 | | |
| 306.27 | 1058.79 | | |
| 311.42 | 1061.3 | | |
| 326.03 | 1061.62 | | |
| 335.94 | 1063.05 | | |
| 357.59 | 1065.99 | | |

## Site 5, 6 and 7: Groot Vis River at Katkop Weir

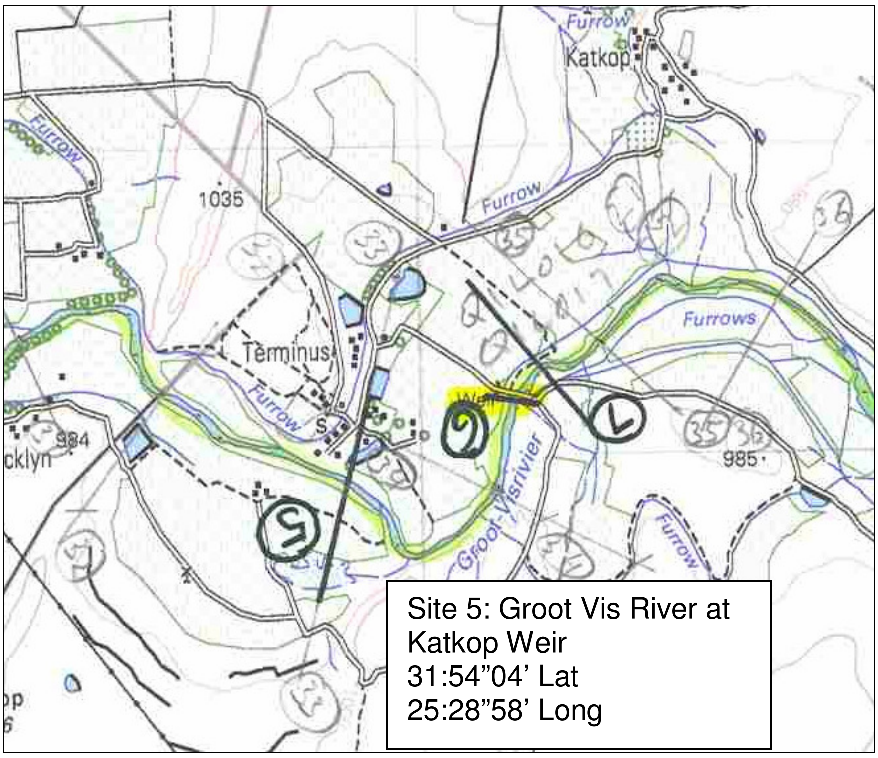
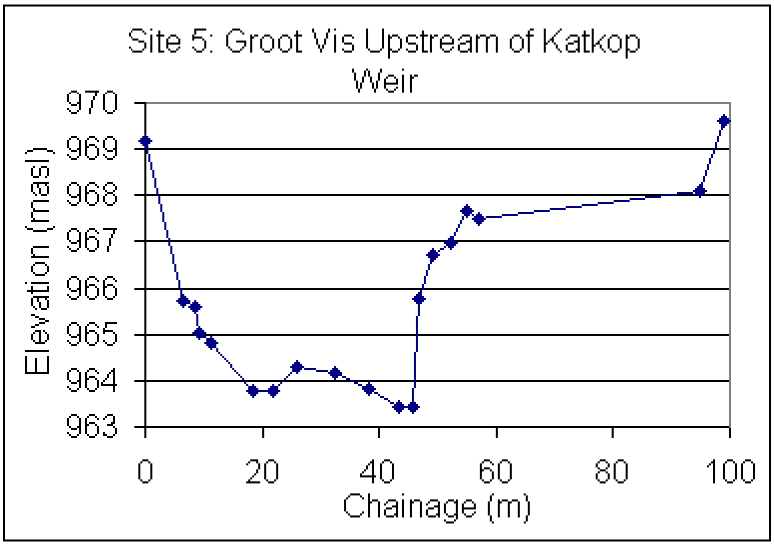



**Figure A-5: Location of Groot Vis River at Katkop Weir**

**SITE 5**

**Measured**

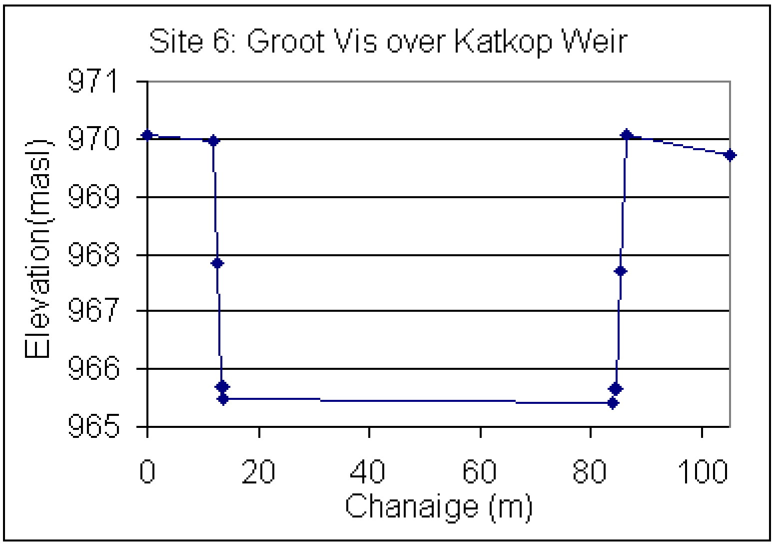
| CH. | G.L. |
|---|---|
| 0.00 | 969.20 |
| 6.58 | 965.71 |
| 8.40 | 965.60 |
| 9.08 | 965.04 |
| 11.26 | 964.82 |
| 18.39 | 963.80 |
| 22.00 | 963.77 |
| 25.99 | 964.28 |
| 32.39 | 964.15 |
| 38.18 | 963.81 |
| 43.40 | 963.44 |
| 45.82 | 963.45 |
| 46.76 | 965.76 |
| 49.21 | 966.72 |
| 52.24 | 966.97 |
| 55.07 | 967.68 |
| 57.09 | 967.48 |
| 95.05 | 968.09 |
| 99.12 | 969.61 |



**SITE 6**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 970.08 |
| 11.69 | 969.98 |
| 12.49 | 967.85 |
| 12.70 | 967.85 |
| 13.42 | 965.71 |
| 13.65 | 965.70 |
| 13.80 | 965.47 |
| 83.76 | 965.43 |
| 84.03 | 965.65 |
| 84.46 | 965.66 |
| 85.06 | 967.70 |
| 85.38 | 967.70 |
| 86.19 | 970.06 |
| 105.00 | 969.72 |




**SITE 7**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 968.32 |
| 16.57 | 967.97 |
| 33.12 | 965.81 |
| 33.58 | 961.36 |
| 34.84 | 960.09 |
| 39.03 | 959.85 |
| 46.50 | 960.73 |
| 55.54 | 960.48 |
| 67.11 | 960.14 |
| 71.56 | 959.76 |
| 80.19 | 960.77 |
| 89.07 | 961.25 |
| 95.30 | 966.01 |
| 95.94 | 965.99 |
| 112.63 | 967.57 |

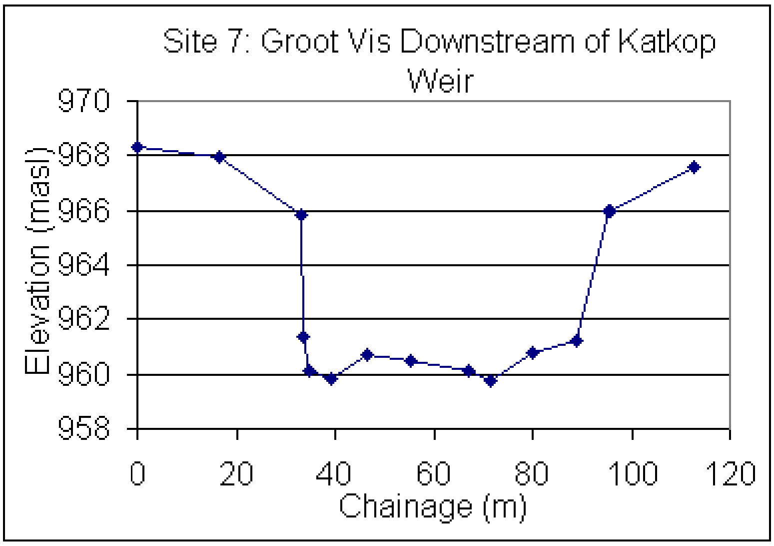



**Figure A-6: Katkop Weir on the Groot Vis River**

## Site 8: Groot Vis River at Marlow

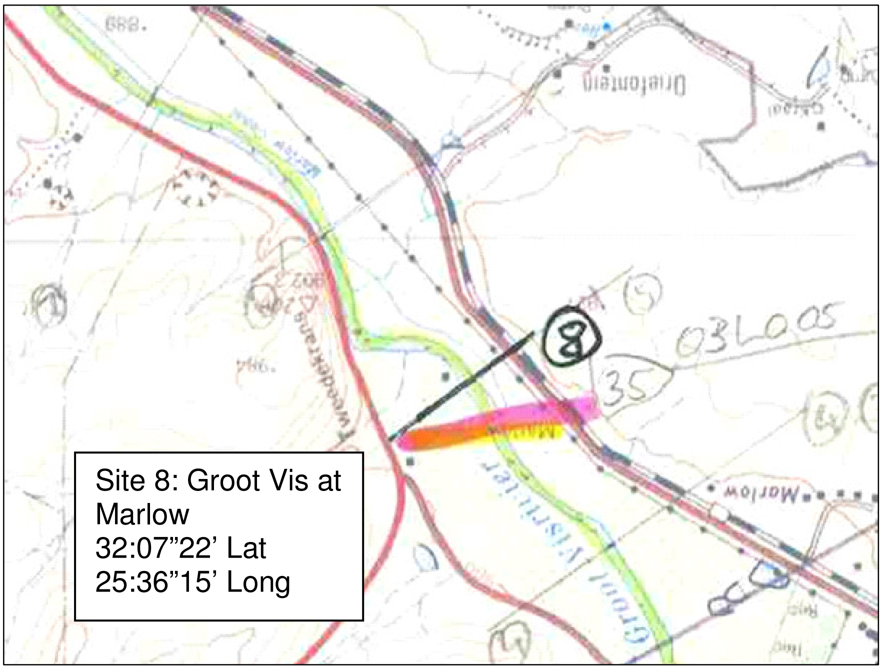



**Figure A-7: Location of Groot Vis River at Marlow**

**SITE 8**

| Measured CH. | Measured G.L. | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 882.07 | 123 | 876 |
| 73.06 | 879.58 | 125 | 874 |
| 110.78 | 878.41 | 155 | 873.5 |
| 113.01 | 877.73 | 157 | 876 |
| 123.22 | 874.68 | | |
| 128.12 | 874.76 | | |
| 130.85 | 874.32 | | |
| 133.70 | 873.83 | | |
| 139.17 | 873.75 | | |
| 143.73 | 873.45 | | |
| 147.43 | 873.52 | | |
| 153.65 | 874.94 | | |
| 158.89 | 874.83 | | |
| 163.63 | 874.23 | | |
| 174.86 | 874.45 | | |
| 184.19 | 875.59 | | |
| 205.47 | 876.28 | | |
| 214.76 | 876.25 | | |
| 223.91 | 876.84 | | |
| 231.84 | 878.20 | | |
| 236.52 | 880.01 | | |
| 255.86 | 880.78 | | |
| 273.16 | 882.20 | | |
| 292.76 | 882.80 | | |

## Site 9: Groot Vis at Cloverfield

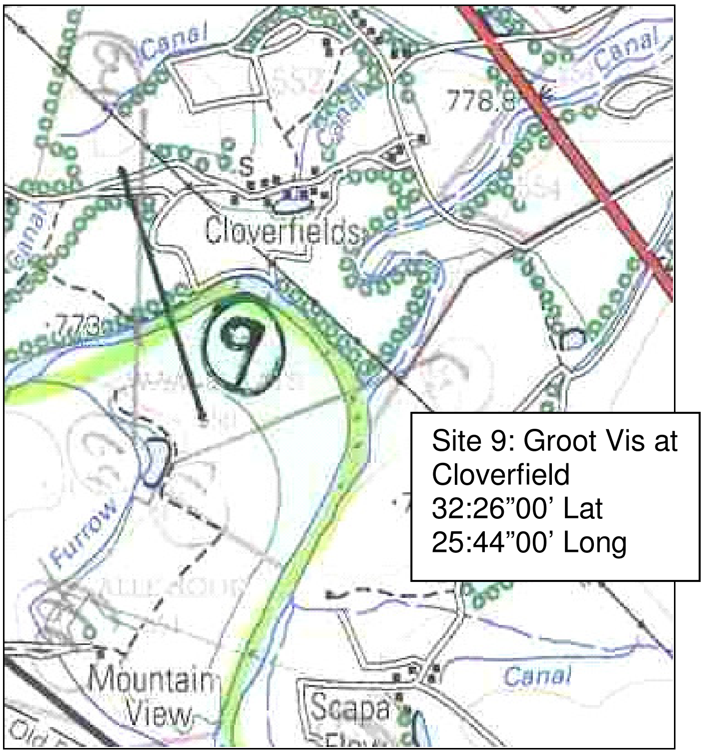
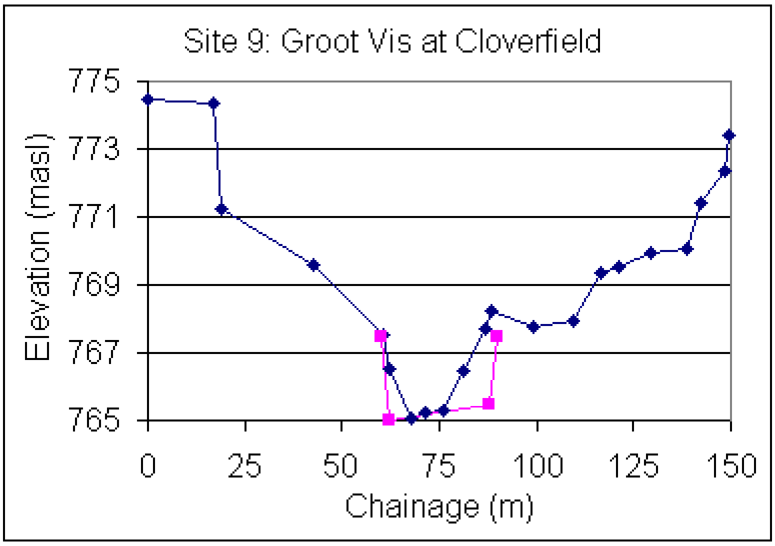



**Figure A-8: Location of Groot Vis River at Cloverfield**

**SITE 9**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L. | CH. | G.L. |
| 0.00 | 774.49 | 60 | 767.50 |
| 17.11 | 774.33 | 62 | 765.00 |
| 18.90 | 771.21 | 88 | 765.50 |
| 42.78 | 769.60 | 90 | 767.50 |
| 60.50 | 767.50 | | |
| 62.00 | 766.52 | | |
| 67.67 | 765.05 | | |
| 71.63 | 765.25 | | |
| 75.99 | 765.27 | | |
| 81.23 | 766.49 | | |
| 86.60 | 767.70 | | |
| 88.33 | 768.25 | | |
| 99.04 | 767.74 | | |
| 109.54 | 767.96 | | |
| 116.72 | 769.38 | | |
| 121.49 | 769.53 | | |
| 129.47 | 769.94 | | |
| 129.52 | 769.94 | | |
| 138.52 | 770.03 | | |
| 142.53 | 771.41 | | |
| 148.43 | 772.34 | | |
| 149.71 | 773.41 | | |



## Site 10: Canal from Elandsdrift Dam

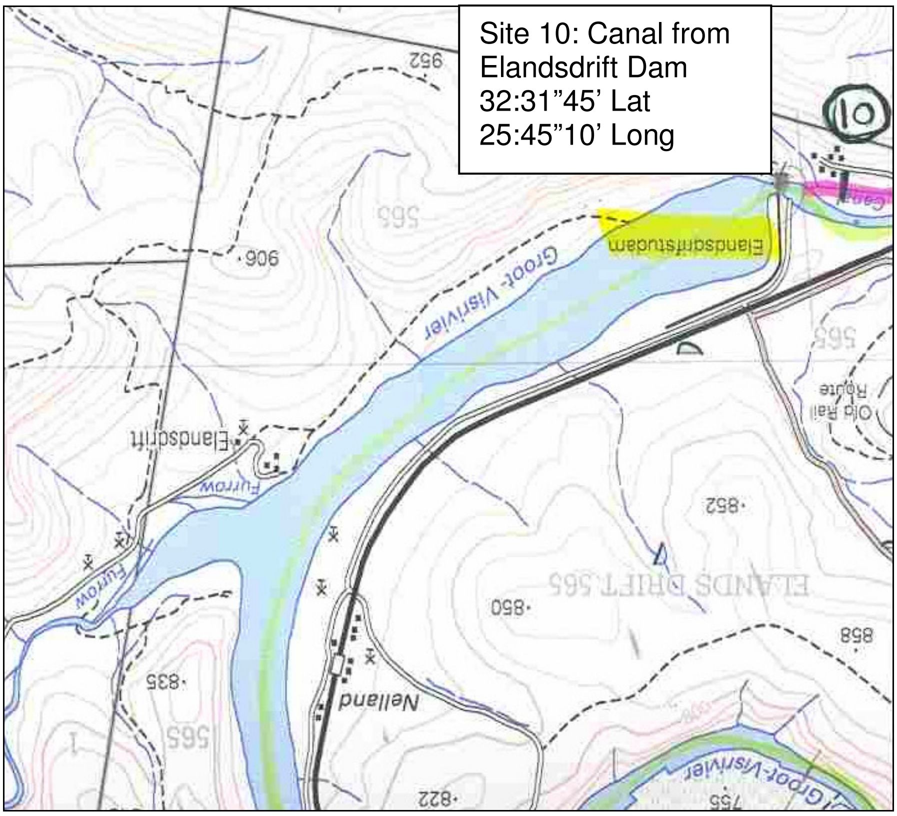
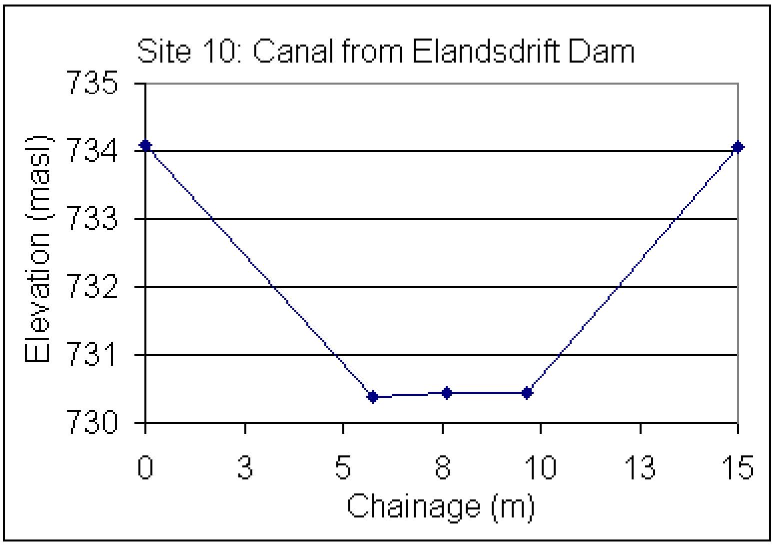



**Figure A-9: Location of Canal from Elandsdrift Dam on the Groot Vis River**

**SITE 10**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 734.09 |
| 5.77 | 730.39 |
| 7.62 | 730.45 |
| 9.66 | 730.43 |
| 15.00 | 734.05 |



## Site 11: Cookhouse Tunnel Intake

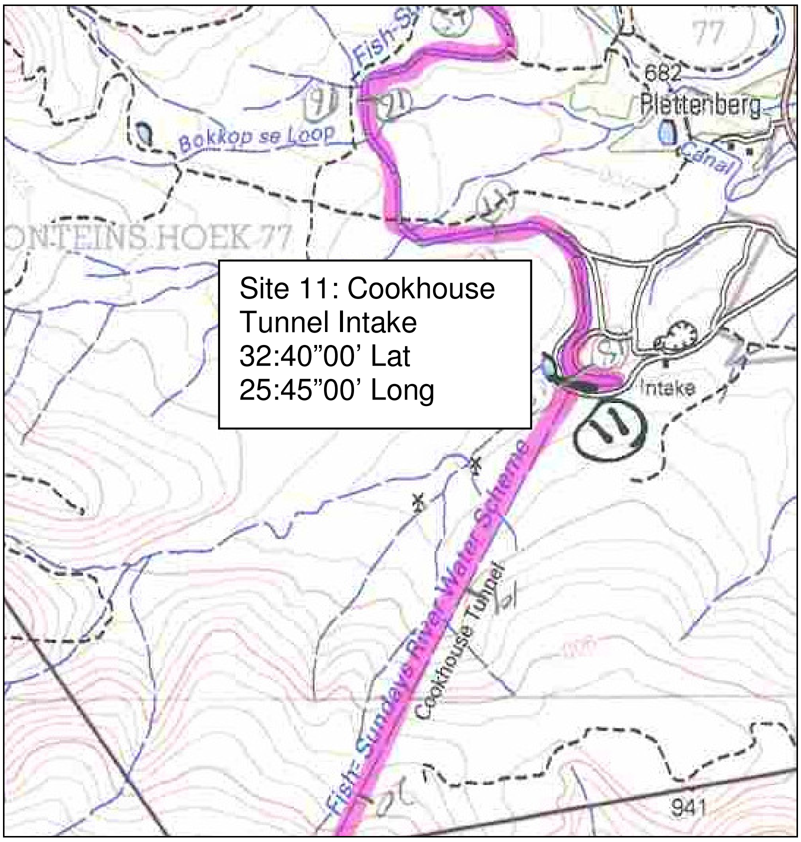



**Figure A-10: Location of Cookhouse Tunnel Intake**

**SITE 11**

| | **Measured** |
|---|---|
| CH. | G.L. |
| 0 | 731.16 |
| 2.42 | 717.89 |
| 4.85 | 731.16 |
| Diameter | 4.85 |
| Invert Level | 717.89 |

**Figure A-11: Cookhouse Tunnel Intake on the Little Fish Canal**

## Site 12: Cookhouse Tunnel Outlet

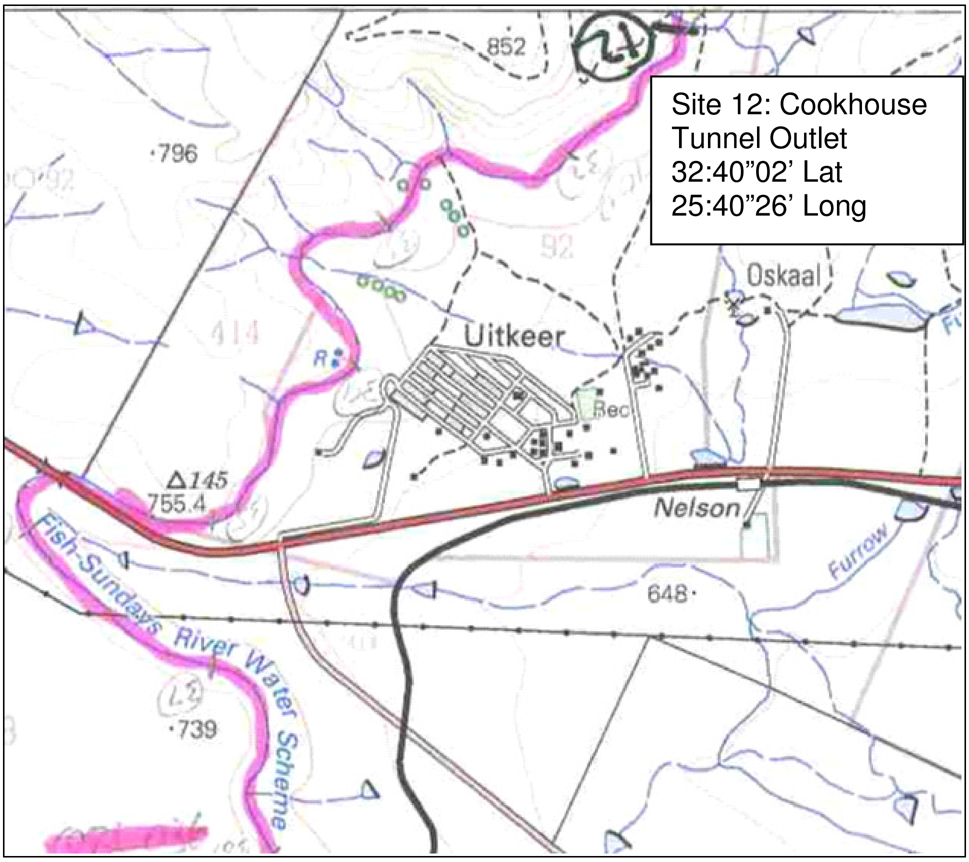



**Figure A-12: Location of Cookhouse Tunnel Outlet**

**SITE 12**

**Measured**

| CH. | G.L. |
|---|---|
| 0 | 716.14 |
| 2.3 | 711.25 |
| 4.61 | 716.14 |
| Diameter | 4.8 |
| Invert level | 711.25 |

**Figure A-13: Cookhouse Tunnel Outlet on the Little Fish Canal**

## Site 13: Groot Vis at Eastpoort

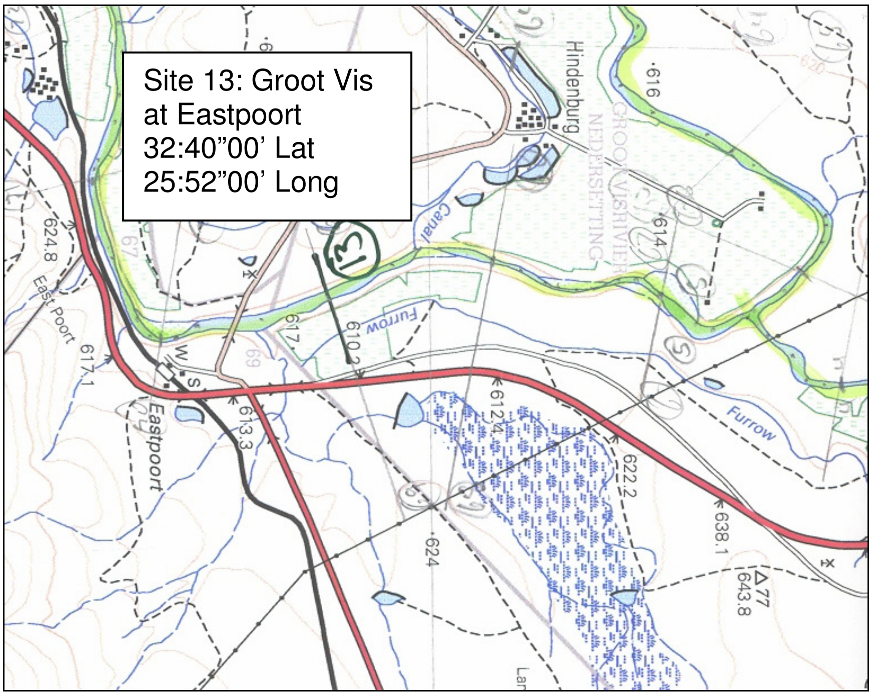
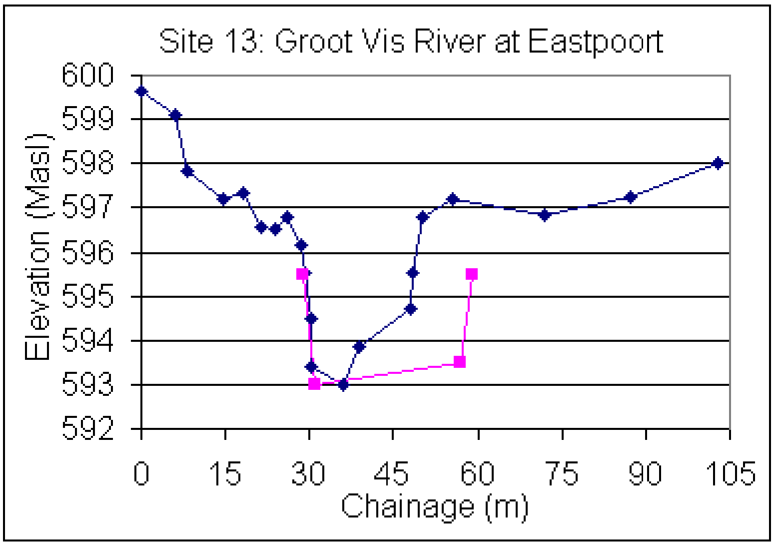



**Figure A-14: Location of Groot Vis River at Eastpoort**

**SITE 13**

| Measured CH. | Measured G.L. | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 599.62 | 29 | 595.50 |
| 6.10 | 599.10 | 31 | 593.00 |
| 8.27 | 597.81 | 57 | 593.50 |
| 14.71 | 597.21 | 59 | 595.50 |
| 18.19 | 597.34 | | |
| 21.26 | 596.56 | | |
| 23.80 | 596.54 | | |
| 25.89 | 596.80 | | |
| 28.45 | 596.14 | | |
| 29.26 | 595.51 | | |
| 30.08 | 594.49 | | |
| 30.35 | 593.42 | | |
| 36.00 | 593.00 | | |
| 38.69 | 593.87 | | |
| 47.94 | 594.69 | | |
| 48.38 | 595.52 | | |
| 50.15 | 596.77 | | |
| 55.44 | 597.18 | | |
| 71.78 | 596.82 | | |
| 87.34 | 597.23 | | |
| 102.86 | 598.02 | | |



## Site 14: Groot Vis at Harefield

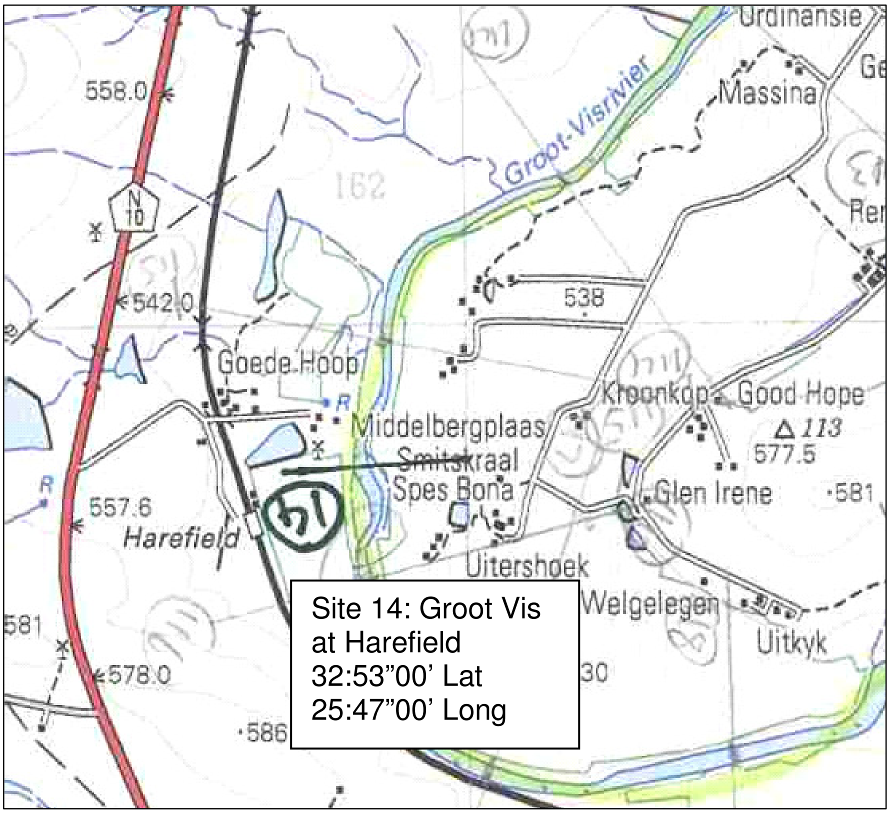
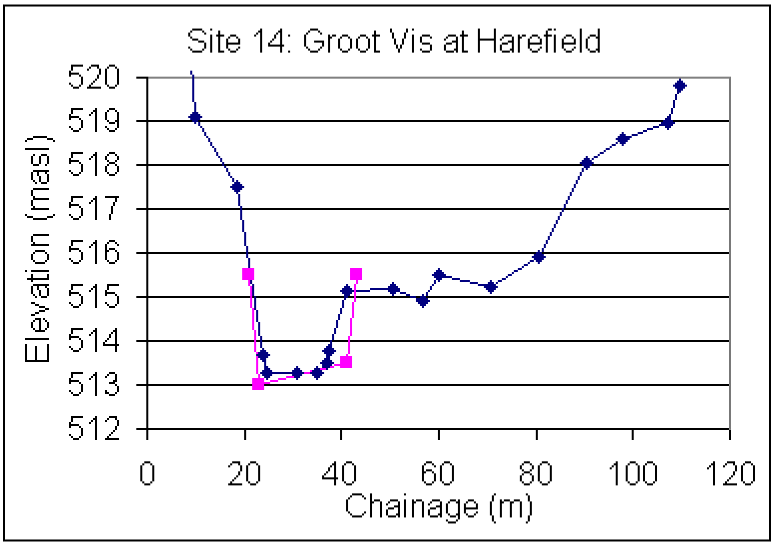


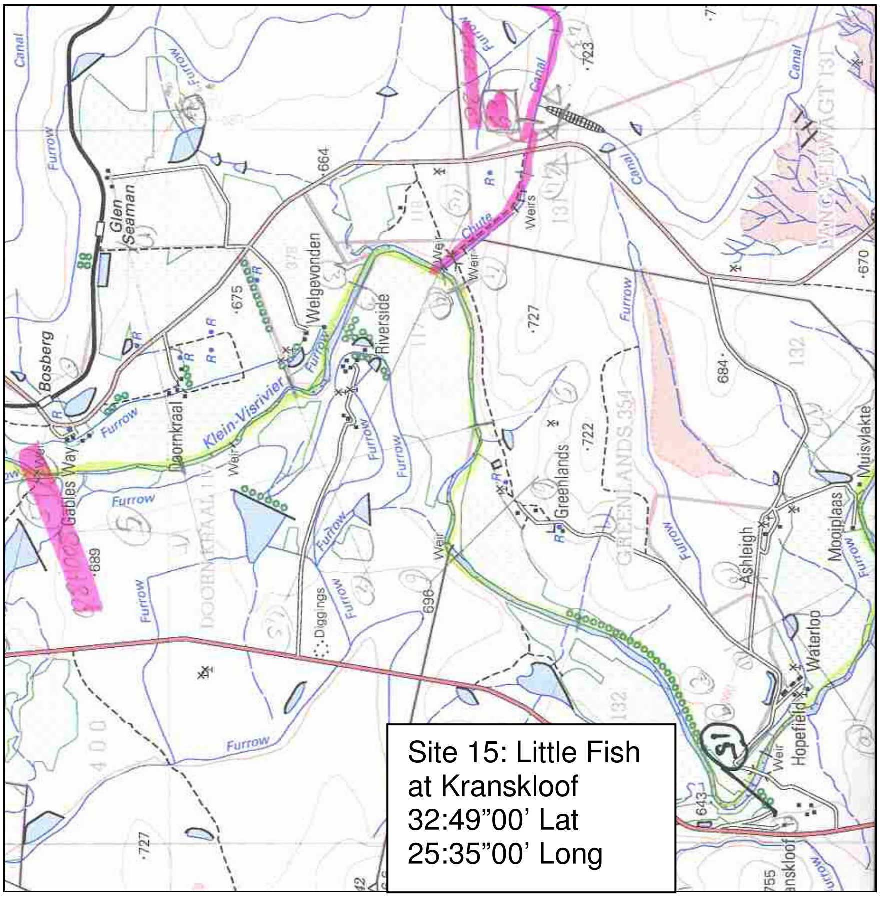

**Figure A-15: Location of Groot Vis River at Harefield**

**SITE 14**

| Measured CH. | Measured G.L. | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 529.34 | 21.00 | 515.50 |
| 3.19 | 528.21 | 23.00 | 513.00 |
| 4.55 | 527.63 | 41.00 | 513.50 |
| 9.96 | 519.08 | 43.00 | 515.50 |
| 18.43 | 517.52 | | |
| 23.88 | 513.70 | | |
| 24.63 | 513.26 | | |
| 30.83 | 513.27 | | |
| 34.77 | 513.29 | | |
| 36.95 | 513.49 | | |
| 37.50 | 513.77 | | |
| 41.04 | 515.12 | | |
| 50.50 | 515.18 | | |
| 56.77 | 514.93 | | |
| 59.81 | 515.49 | | |
| 70.48 | 515.25 | | |
| 80.59 | 515.89 | | |
| 90.59 | 518.05 | | |
| 97.94 | 518.60 | | |
| 109.86 | 519.81 | | |

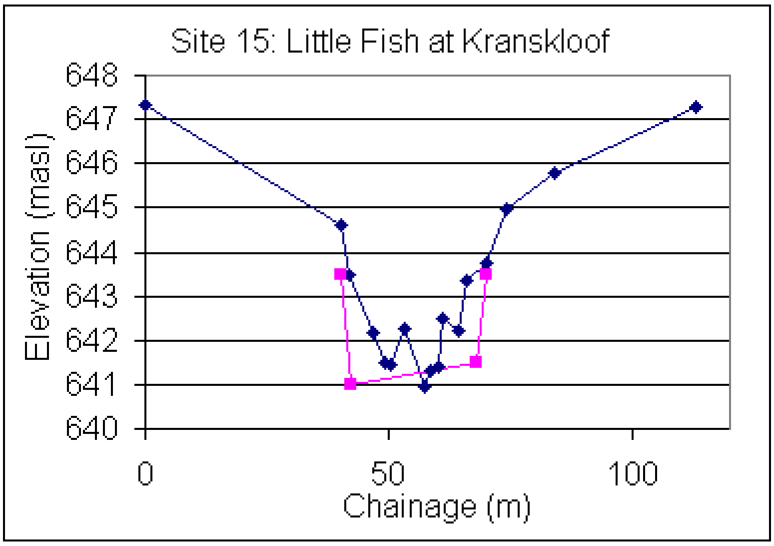


## Site 15: Little Fish at Kranskloof




**SITE 15**

| Measured CH. | Measured G.L. | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 647.30 | 40 | 643.50 |
| 40.00 | 644.61 | 42 | 641.00 |
| 41.89 | 643.49 | 68 | 641.50 |
| 46.63 | 642.16 | 70 | 643.50 |
| 49.26 | 641.47 | | |
| 50.38 | 641.43 | | |
| 53.33 | 642.25 | | |
| 57.23 | 640.97 | | |
| 58.48 | 641.30 | | |
| 60.16 | 641.39 | | |
| 61.02 | 642.47 | | |
| 64.39 | 642.21 | | |
| 65.77 | 643.35 | | |
| 70.13 | 643.74 | | |
| 74.27 | 644.97 | | |
| 84.14 | 645.79 | | |
| 113.19 | 647.26 | | |



## Site 16: Canal from De Mistkraal Dam

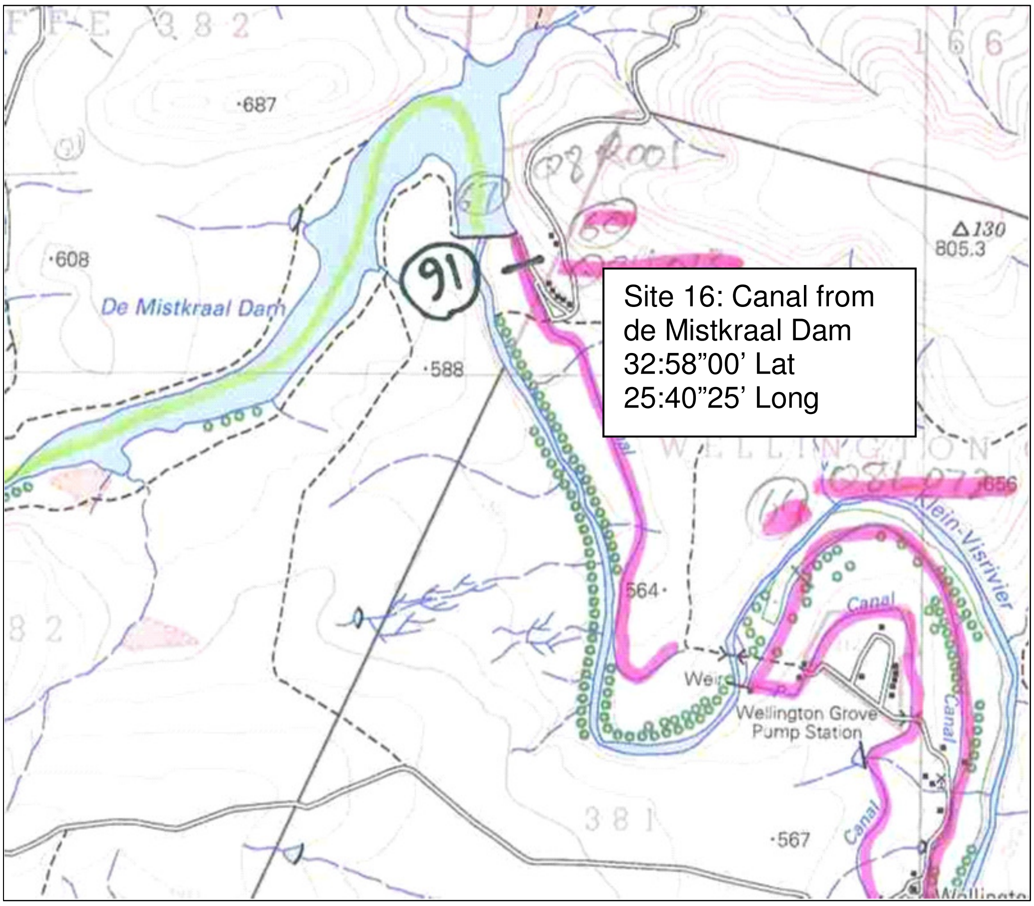



**Figure A-16: Location of Canal from De Mistkraal Dam in the Little Fish River**

**SITE 16**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 549.89 |
| 4.63 | 546.87 |
| 8.27 | 546.88 |
| 12.82 | 549.92 |

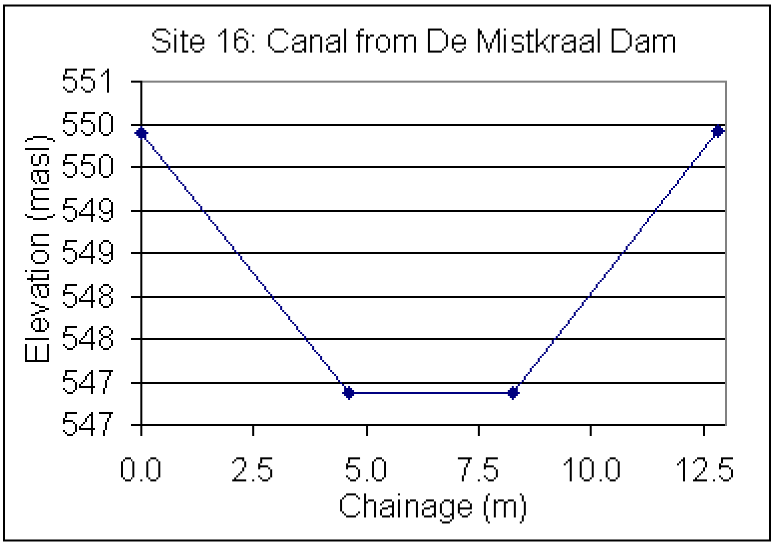



**Figure A-17: Canal at De Mistkraal Dam**

## Site 17: Little Fish at Rippon

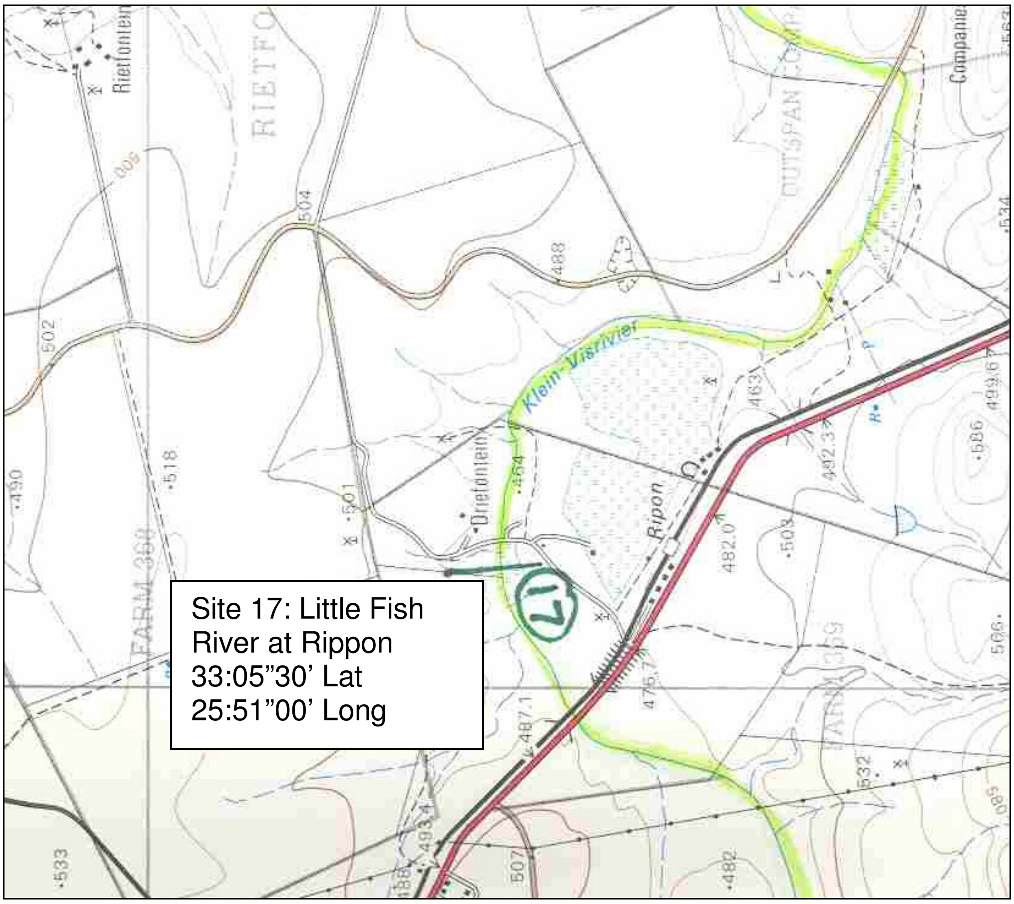
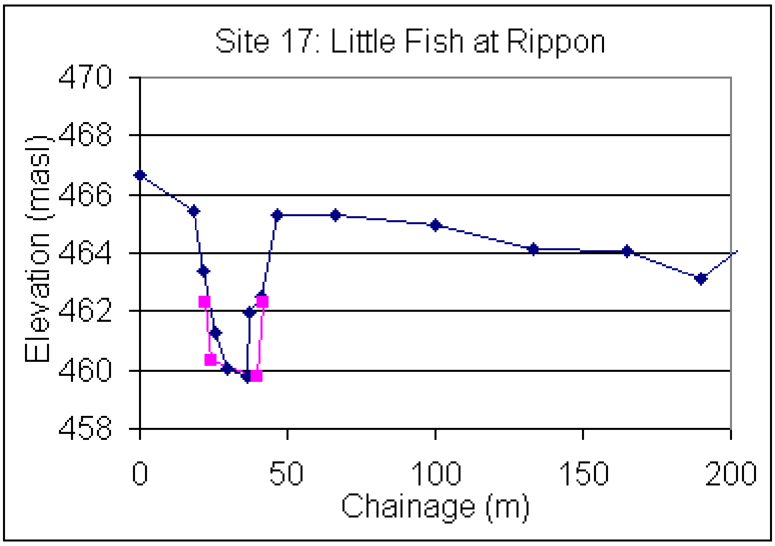



**Figure A-18: Location of little Fish River at Rippon**

**SITE 17**

| Measured CH. | Measured G.L | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 466.65 | 22 | 462.30 |
| 18.13 | 465.41 | 24 | 460.30 |
| 21.29 | 463.37 | 40 | 459.80 |
| 25.95 | 461.30 | 42 | 462.30 |
| 29.58 | 460.02 | | |
| 36.22 | 459.75 | | |
| 37.43 | 461.93 | | |
| 41.19 | 462.48 | | |
| 46.42 | 465.32 | | |
| 66.08 | 465.27 | | |
| 99.87 | 464.93 | | |
| 132.79 | 464.14 | | |
| 164.74 | 464.06 | | |
| 190.04 | 463.13 | | |
| 202.56 | 464.15 | | |
| 223.32 | 468.15 | | |



## Site 18: Confluence of Groot Vis and Little Fish Rivers at Junctiondrift

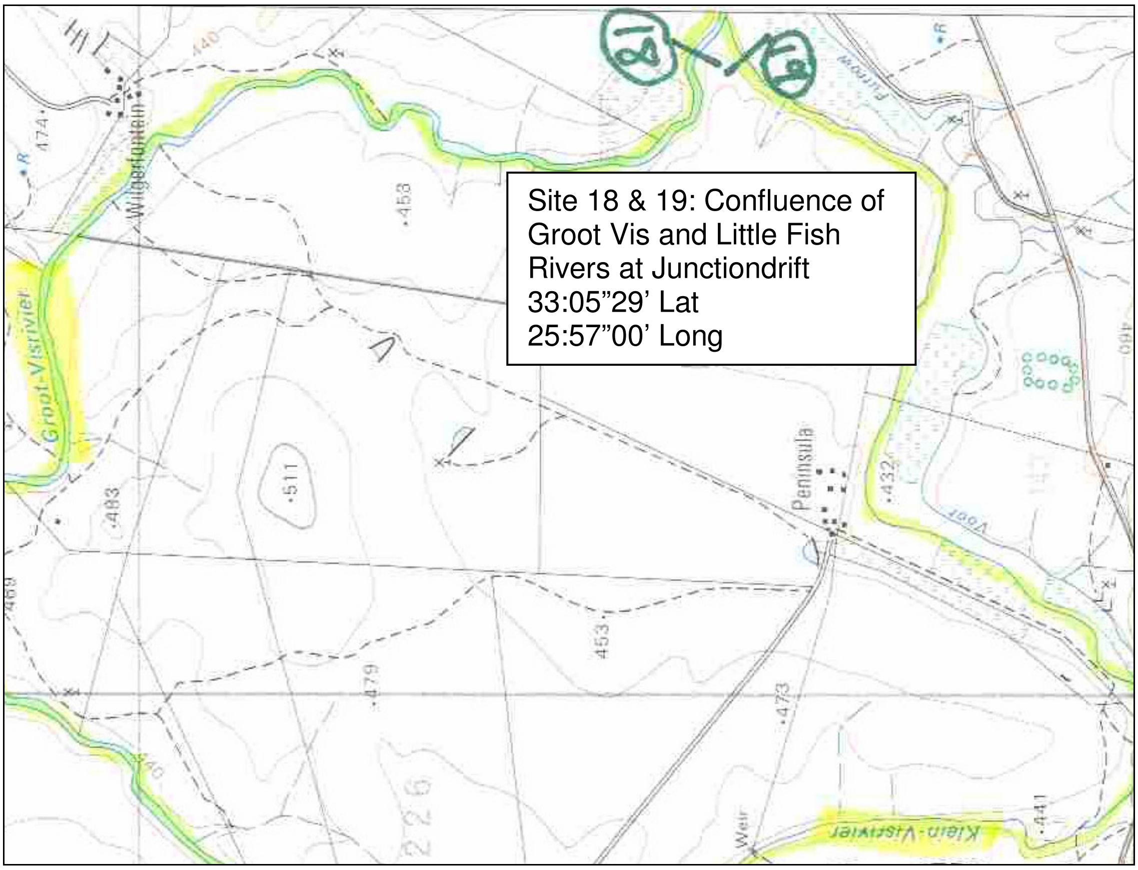



**Figure A-19: Location of Confluence of Groot Vis and Little Fish Rivers at Junctiondrift**

**SITE 18**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L | CH. | G.L. |
| 0.00 | 435.13 | 35 | 419.5 |
| 10.65 | 424.28 | 37 | 417 |
| 18.15 | 420.64 | 65 | 417.5 |
| 36.11 | 419.02 | 67 | 419.5 |
| 37.84 | 418.32 | | |
| 42.85 | 417.24 | | |
| 46.21 | 417.03 | | |
| 50.12 | 416.80 | | |
| 52.49 | 418.03 | | |
| 53.38 | 417.12 | | |
| 59.40 | 417.50 | | |
| 61.79 | 418.11 | | |
| 65.13 | 418.91 | | |
| 73.14 | 420.68 | | |
| 76.66 | 421.22 | | |
| 80.71 | 423.48 | | |
| 92.78 | 425.88 | | |
| 102.89 | 429.42 | | |

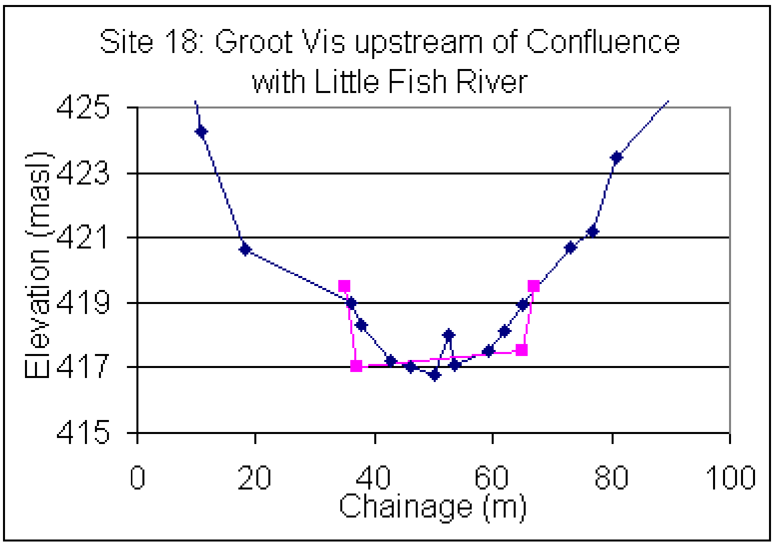
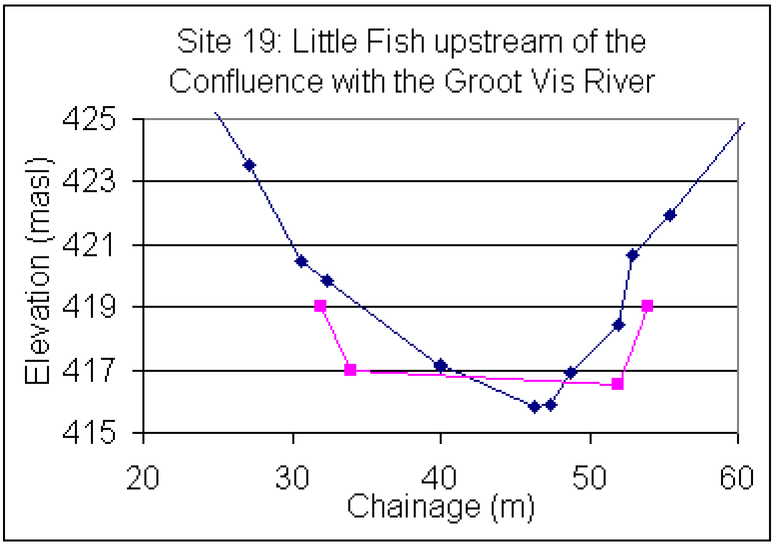


**Figure A-20: Site 18: Groot Vis River Upstream of Confluence with Little Fish River**

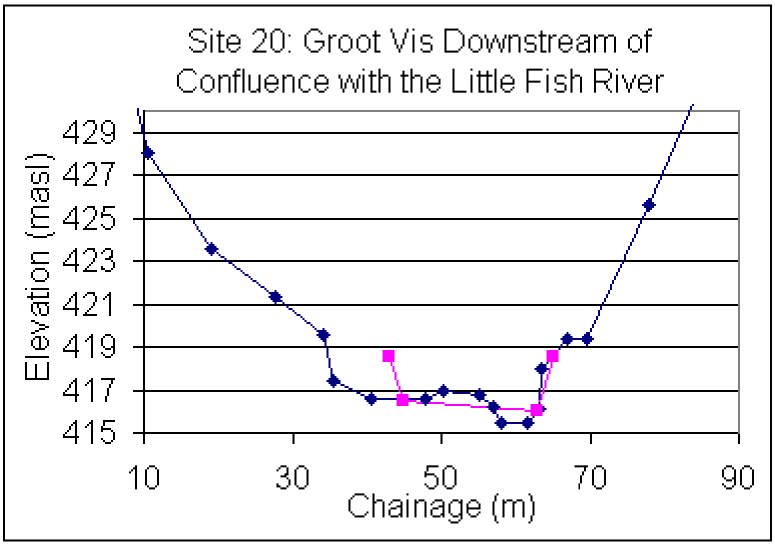
**SITE 19**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L | CH. | G.L. |
| 0.00 | 435.05 | 32 | 419 |
| 8.66 | 433.92 | 34 | 417 |
| 19.93 | 428.93 | 52 | 416.5 |
| 27.08 | 423.52 | 54 | 419 |
| 30.62 | 420.49 | | |
| 32.36 | 419.86 | | |
| 39.98 | 417.10 | | |
| 40.05 | 417.16 | | |
| 46.26 | 415.82 | | |
| 47.34 | 415.87 | | |
| 48.67 | 416.90 | | |
| 51.91 | 418.47 | | |
| 52.85 | 420.69 | | |
| 55.47 | 421.92 | | |
| 64.77 | 427.49 | | |
| 75.35 | 433.98 | | |



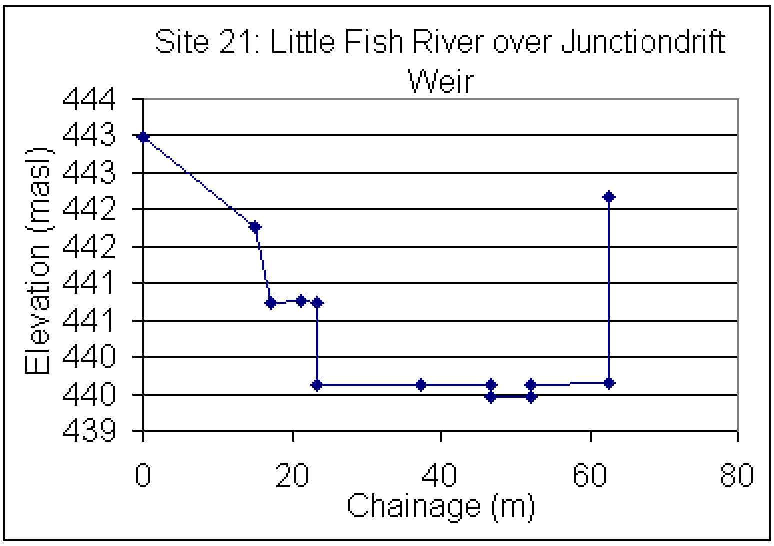
**SITE 20**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L | CH. | G.L. |
| 0.00 | 433.69 | 43 | 418.5 |
| 8.60 | 430.72 | 45 | 416.5 |
| 10.60 | 428.08 | 63 | 416 |
| 19.10 | 423.54 | 65 | 418.5 |
| 27.68 | 421.31 | | |
| 34.28 | 419.58 | | |
| 35.44 | 417.45 | | |
| 40.67 | 416.62 | | |
| 47.92 | 416.61 | | |
| 50.38 | 416.97 | | |
| 55.14 | 416.80 | | |
| 56.95 | 416.22 | | |
| 58.16 | 415.43 | | |
| 61.58 | 415.46 | | |
| 63.11 | 416.10 | | |
| 63.47 | 417.99 | | |
| 66.86 | 419.33 | | |
| 69.61 | 419.41 | | |
| 77.92 | 425.60 | | |
| 85.18 | 431.36 | | |




**SITE 21**

| Measured | |
|---|---|
| CH. | G.L |
| 0.00 | 442.98 |
| 15.00 | 441.75 |
| 17.07 | 440.75 |
| 21.29 | 440.75 |
| 23.32 | 440.75 |
| 23.33 | 439.62 |
| 37.19 | 439.62 |
| 46.69 | 439.62 |
| 46.70 | 439.47 |
| 52.15 | 439.47 |
| 52.16 | 439.62 |
| 62.49 | 439.64 |
| 62.51 | 442.16 |



## Site 22: Skoenmakers River at Middlewater

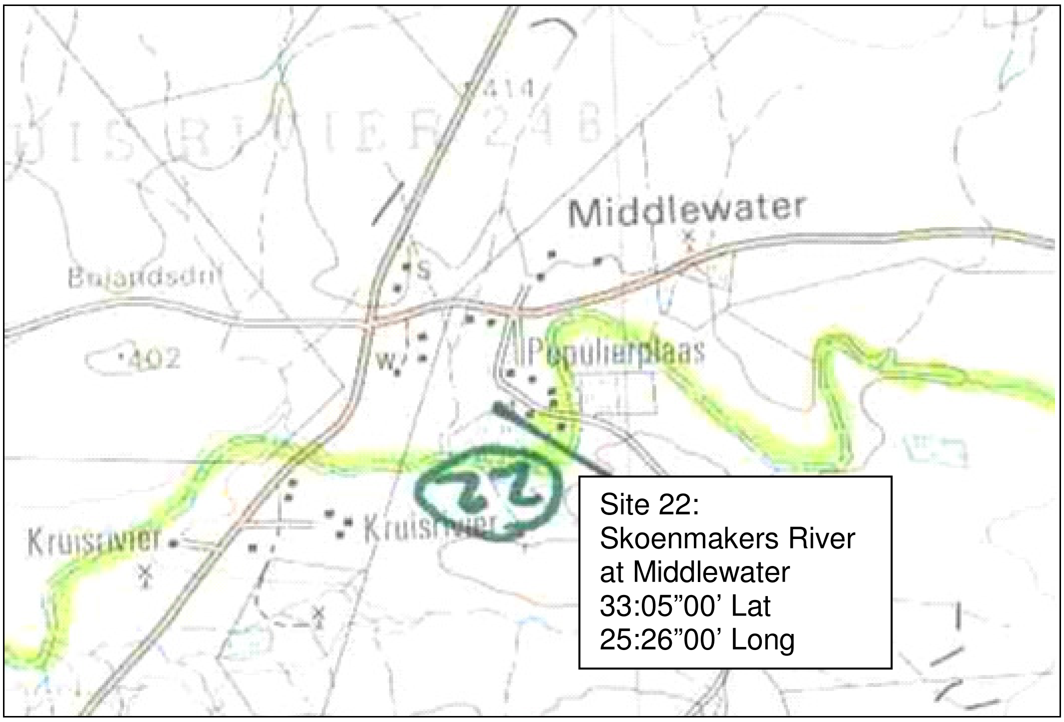
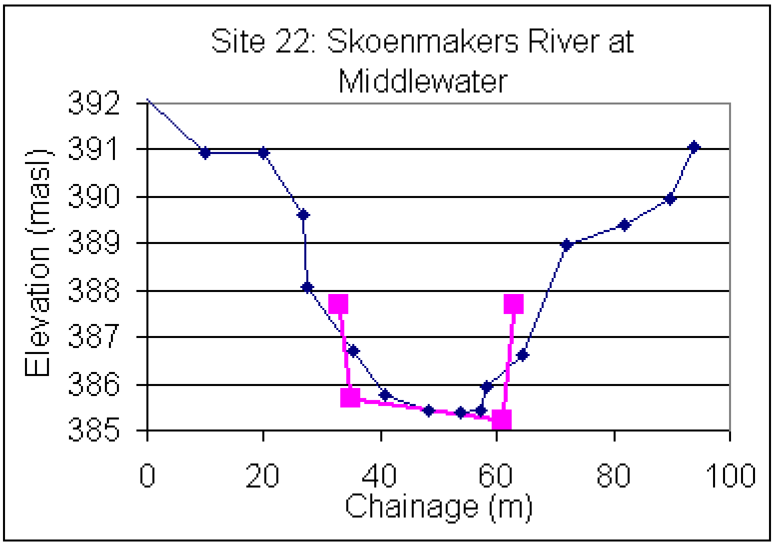



**Figure A-21: Skoenmakers River at Middlewater**

**SITE 22**

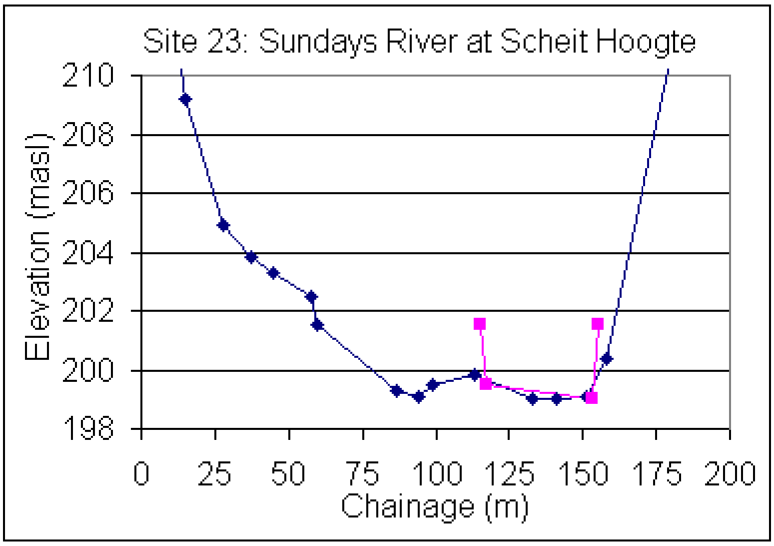
| Measured CH. | Measured G.L. | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 392.10 | 33 | 387.70 |
| 9.86 | 390.94 | 35 | 385.70 |
| 19.85 | 390.94 | 61 | 385.20 |
| 26.87 | 389.61 | 63 | 387.70 |
| 27.53 | 388.09 | | |
| 35.31 | 386.73 | | |
| 40.66 | 385.77 | | |
| 48.30 | 385.41 | | |
| 53.79 | 385.37 | | |
| 57.12 | 385.44 | | |
| 58.27 | 385.93 | | |
| 64.51 | 386.61 | | |
| 71.77 | 388.96 | | |
| 81.73 | 389.41 | | |
| 89.67 | 389.96 | | |
| 93.95 | 391.06 | | |



## Site 23: Sundays River at Scheit Hoogte

**SITE 23**

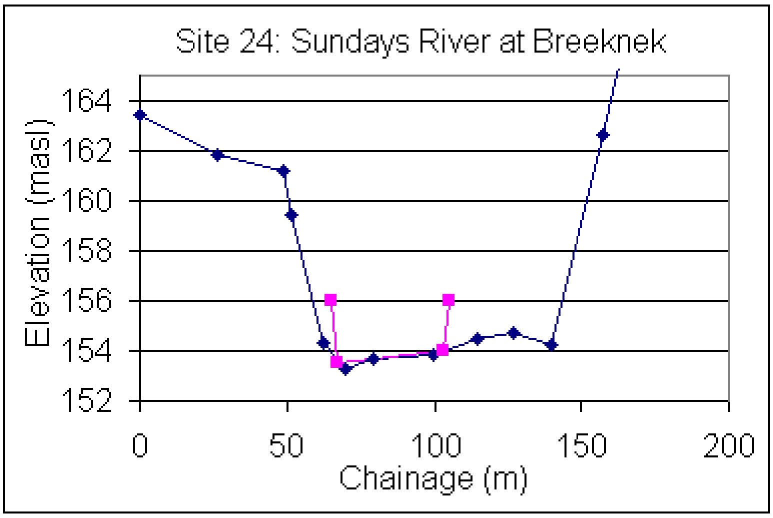
| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L. | CH. | G.L. |
| 0.00 | 220.20 | 115 | 201.50 |
| 9.30 | 213.40 | 117 | 199.50 |
| 15.00 | 209.20 | 153 | 199.00 |
| 28.00 | 204.90 | 155 | 201.50 |
| 37.00 | 203.80 | | |
| 44.80 | 203.30 | | |
| 57.80 | 202.50 | | |
| 59.70 | 201.50 | | |
| 86.90 | 199.30 | | |
| 94.00 | 199.10 | | |
| 99.20 | 199.50 | | |
| 113.00 | 199.80 | | |
| 133.10 | 199.00 | | |
| 141.00 | 199.00 | | |
| 151.40 | 199.10 | | |
| 158.10 | 200.40 | | |
| 180.30 | 210.80 | | |
| 191.20 | 214.60 | | |
| 195.10 | 218.40 | | |




## Site 24: Sundays River at Breeknek

**SITE 24**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L. | CH. | G.L. |
| 0.00 | 163.40 | 65 | 156.00 |
| 26.40 | 161.80 | 67 | 153.50 |
| 49.10 | 161.20 | 103 | 154.00 |
| 51.30 | 159.40 | 105 | 156.00 |
| 62.40 | 154.30 | | |
| 69.70 | 153.30 | | |
| 79.00 | 153.70 | | |
| 99.50 | 153.80 | | |
| 114.40 | 154.50 | | |
| 126.70 | 154.70 | | |
| 139.90 | 154.20 | | |
| 157.30 | 162.60 | | |
| 173.50 | 171.00 | | |
| 495.50 | 178.30 | | |



# Site 25: Canal from Korhaansdrift Abstraction Weir

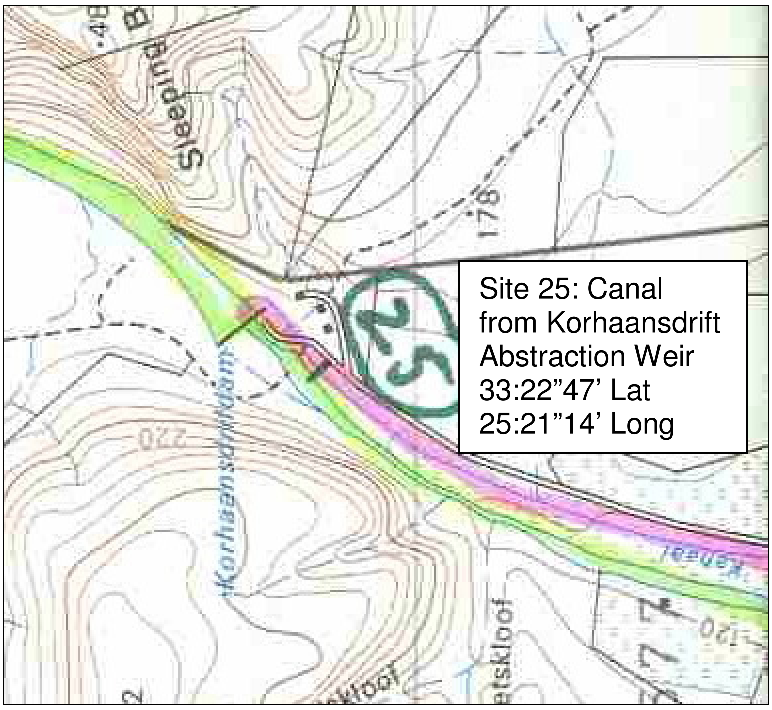
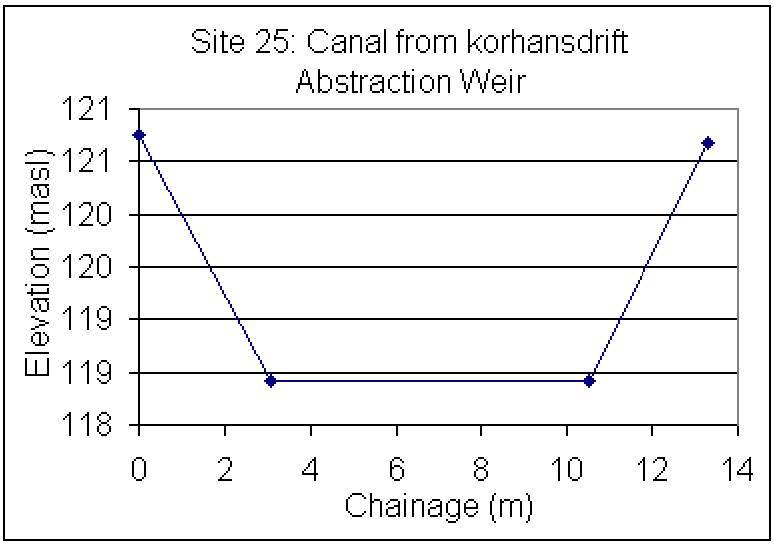



**Figure A-22: Location of Korhaansdrift Abstraction Weir**

**SITE 25**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 120.76 |
| 3.06 | 118.43 |
| 10.48 | 118.41 |
| 13.32 | 120.67 |



## Site 26: Canal to Skeepersvlakte Dam

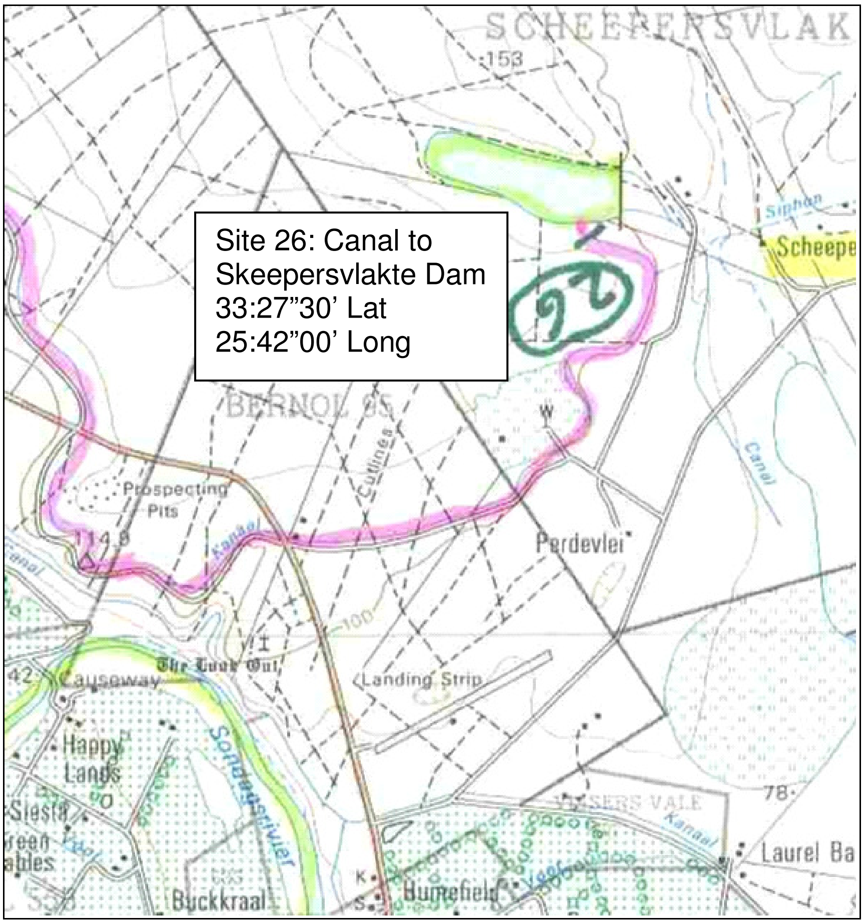
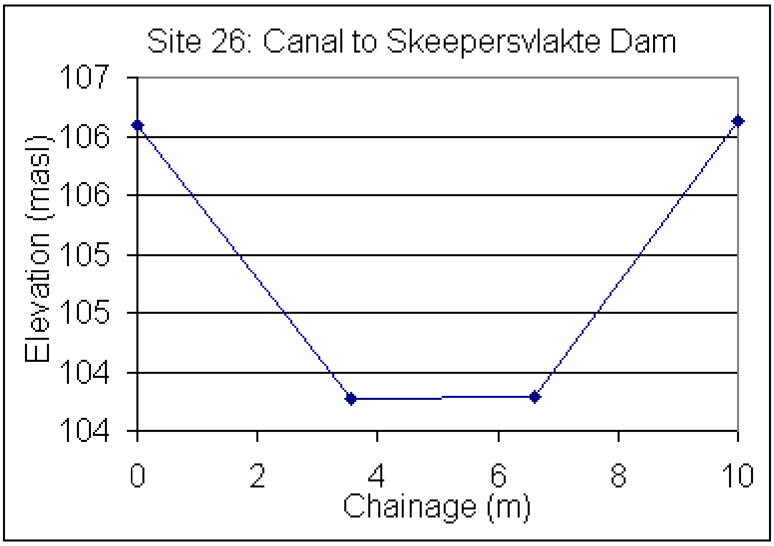



**Figure A-23: Location of Canal to Skeepersvlakte Dam**

**SITE 26**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 106.10 |
| 3.56 | 103.77 |
| 6.60 | 103.78 |
| 10.00 | 106.13 |



## Site 27: Groot Vis at Carlisle Bridge

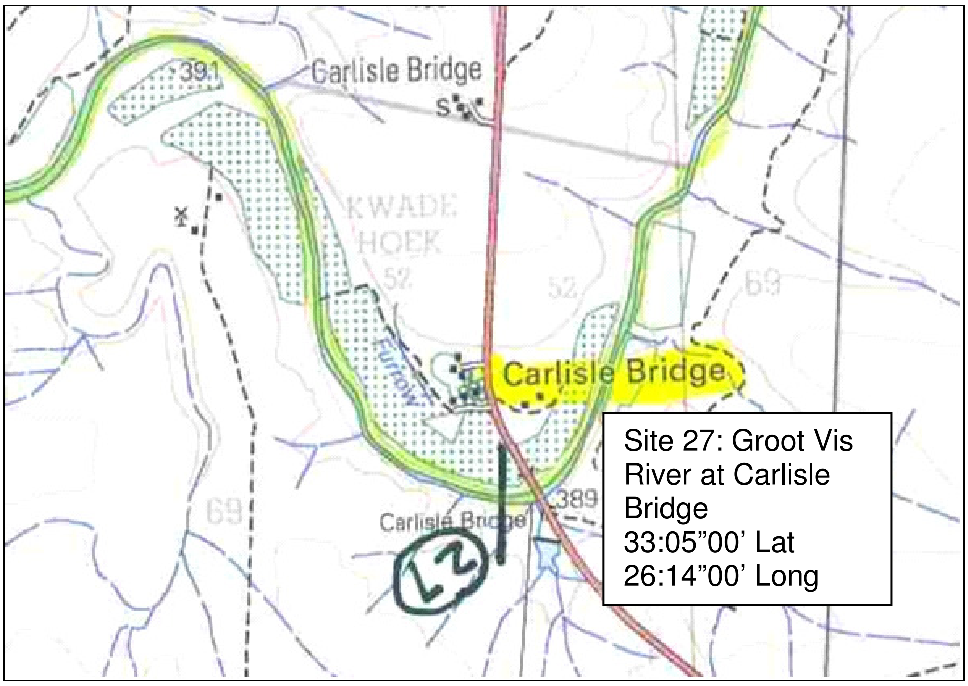
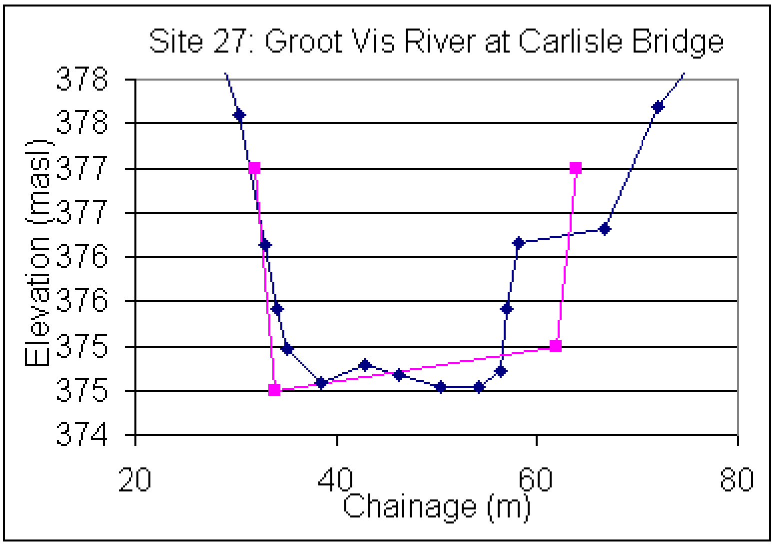



**Figure A-24: Location of Groot Vis River at Carlisle Bridge**

**SITE 27**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L. | CH. | G.L. |
| 0.00 | 386.90 | 32 | 377.00 |
| 1.94 | 386.22 | 34 | 374.50 |
| 5.47 | 385.84 | 62 | 375.00 |
| 19.32 | 381.56 | 64 | 377.00 |
| 30.31 | 377.59 | | |
| 32.85 | 376.14 | | |
| 34.01 | 375.42 | | |
| 35.12 | 374.96 | | |
| 38.53 | 374.59 | | |
| 42.77 | 374.78 | | |
| 46.20 | 374.67 | | |
| 50.44 | 374.53 | | |
| 54.08 | 374.54 | | |
| 56.36 | 374.71 | | |
| 56.94 | 375.41 | | |
| 58.19 | 376.16 | | |
| 66.64 | 376.32 | | |
| 72.13 | 377.68 | | |
| 87.97 | 379.99 | | |
| 96.65 | 381.49 | | |



## Site 28, 29, 30 & 31: Groot Vis River at Hermanuskraal Diversion Weir

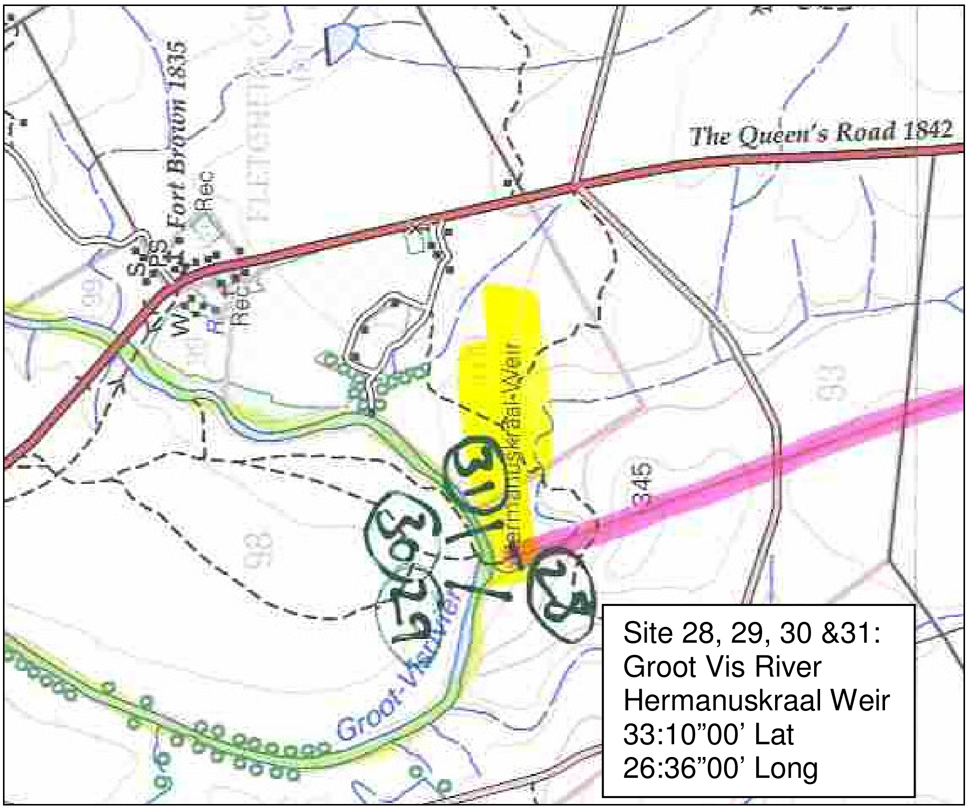
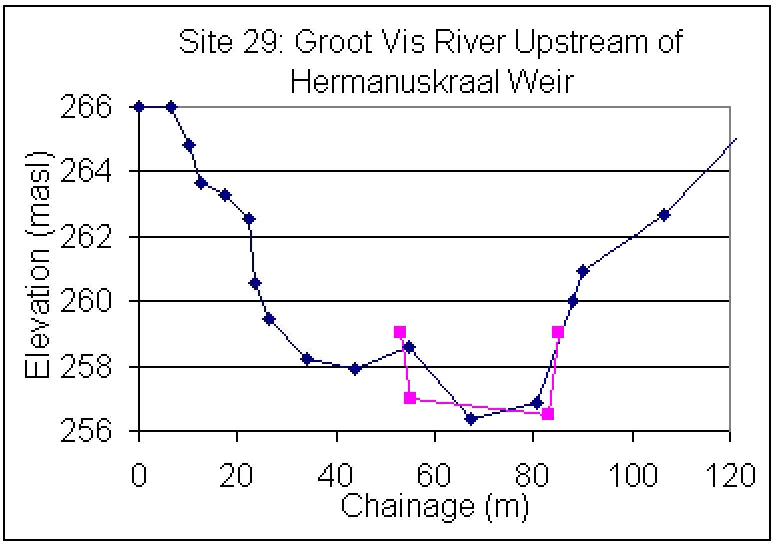



**Figure A-25: Location of Groot Vis River at Hermanuskraal Diversion Weir**

No data available from Site 28 at the Ecca Tunnel Intake

**SITE 29**

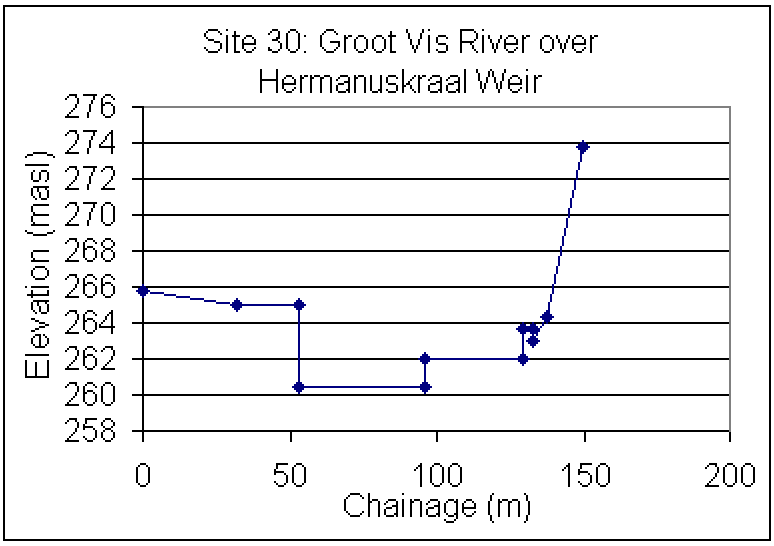
| Measured CH. | Measured G.L. | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 265.99 | 53 | 259 |
| 6.51 | 265.97 | 55 | 257 |
| 9.94 | 264.83 | 83 | 256.5 |
| 12.49 | 263.64 | 85 | 259 |
| 17.59 | 263.29 | | |
| 22.26 | 262.54 | | |
| 23.49 | 260.58 | | |
| 26.30 | 259.48 | | |
| 34.10 | 258.24 | | |
| 43.74 | 257.91 | | |
| 54.90 | 258.59 | | |
| 67.22 | 256.38 | | |
| 80.64 | 256.88 | | |
| 87.96 | 259.99 | | |
| 90.08 | 260.94 | | |
| 106.71 | 262.65 | | |
| 122.71 | 265.26 | | |



**SITE 30**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 265.79 |
| 32.22 | 264.95 |
| 53.25 | 264.97 |
| 53.25 | 260.44 |
| 95.62 | 260.44 |
| 95.63 | 261.96 |
| 95.82 | 261.98 |
| 129.17 | 261.96 |
| 129.31 | 263.68 |
| 132.53 | 263.68 |
| 132.55 | 263.01 |
| 137.15 | 264.33 |
| 149.47 | 273.76 |




**SITE 31**

| Measured | | Assumed | |
|---|---|---|---|
| CH. | G.L. | CH. | G.L. |
| 0.00 | 264.33 | 22 | 257.50 |
| 0.21 | 262.51 | 24 | 255.50 |
| 5.79 | 259.43 | 53 | 255.00 |
| 9.36 | 258.55 | 55 | 257.50 |
| 10.31 | 257.69 | | |
| 19.52 | 256.13 | | |
| 20.90 | 255.85 | | |
| 28.91 | 256.24 | | |
| 33.19 | 255.89 | | |
| 35.36 | 254.86 | | |
| 44.50 | 254.63 | | |
| 48.07 | 254.94 | | |
| 50.03 | 255.61 | | |
| 55.49 | 256.30 | | |
| 63.34 | 260.57 | | |

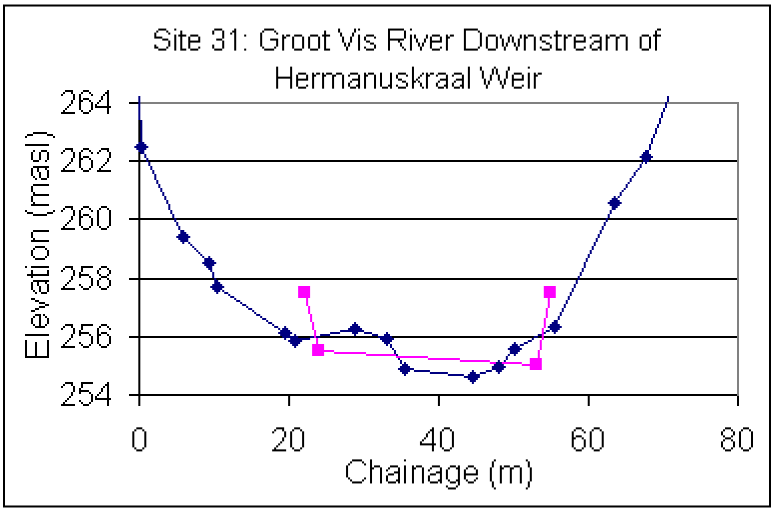


**Figure A-26: Hermanuskraal Weir** **Figure A-27: Groot Vis below Hermanuskraal Weir**

## Site 32 and 33: Ecca Tunnel to Glen Melville Dam

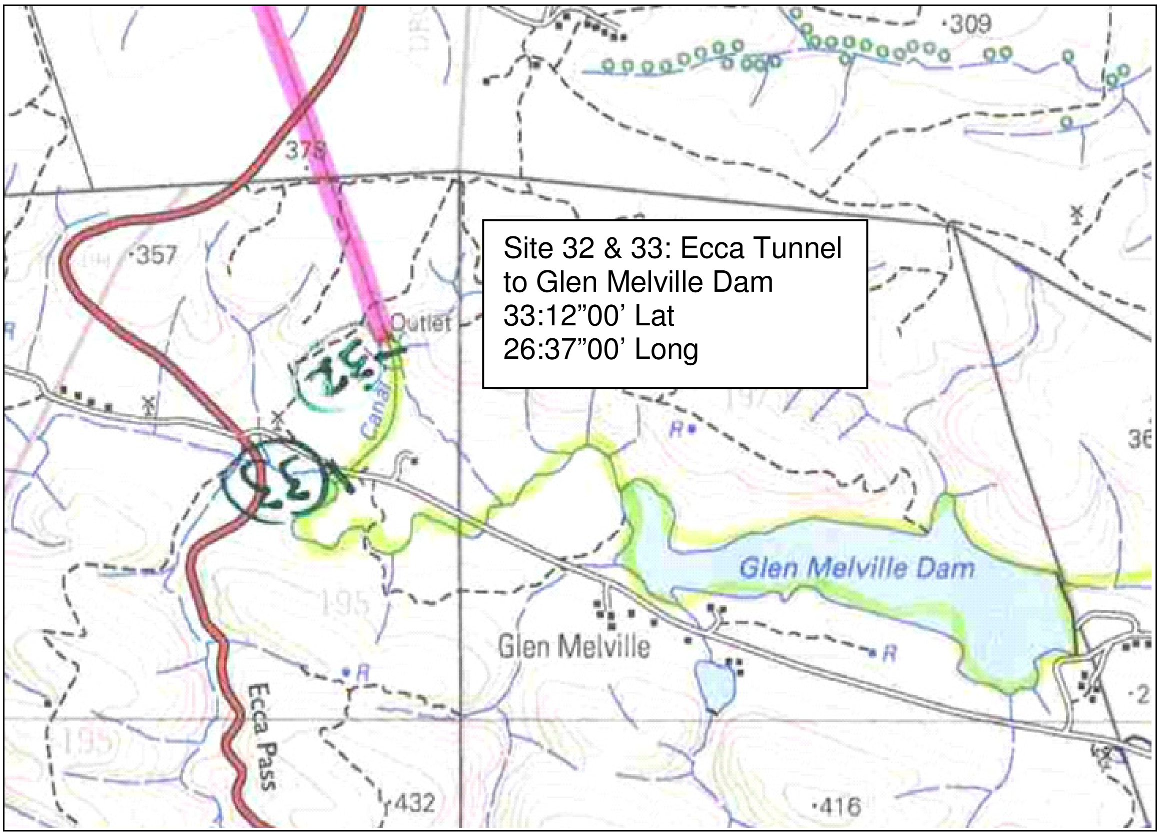


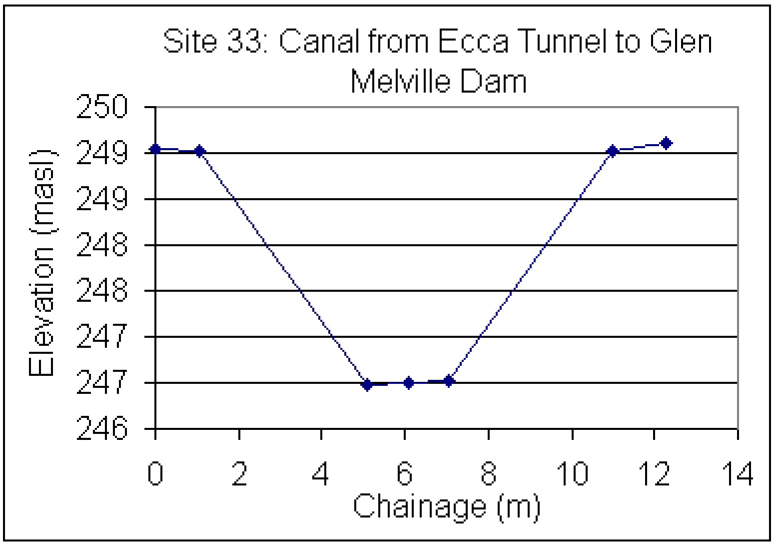

**Figure A-28: Location of Ecca Tunnel to Glen Melville Dam**

No data available from Site 32 at the Ecca Tunnel Outlet

**SITE 33**

**Measured**

| CH. | G.L. |
|---|---|
| 0.00 | 249.04 |
| 1.05 | 249.02 |
| 5.08 | 246.48 |
| 6.11 | 246.50 |
| 7.05 | 246.52 |
| 10.97 | 249.01 |
| 12.27 | 249.11 |




## Site 34: Groot Vis River at Kleinkanaan

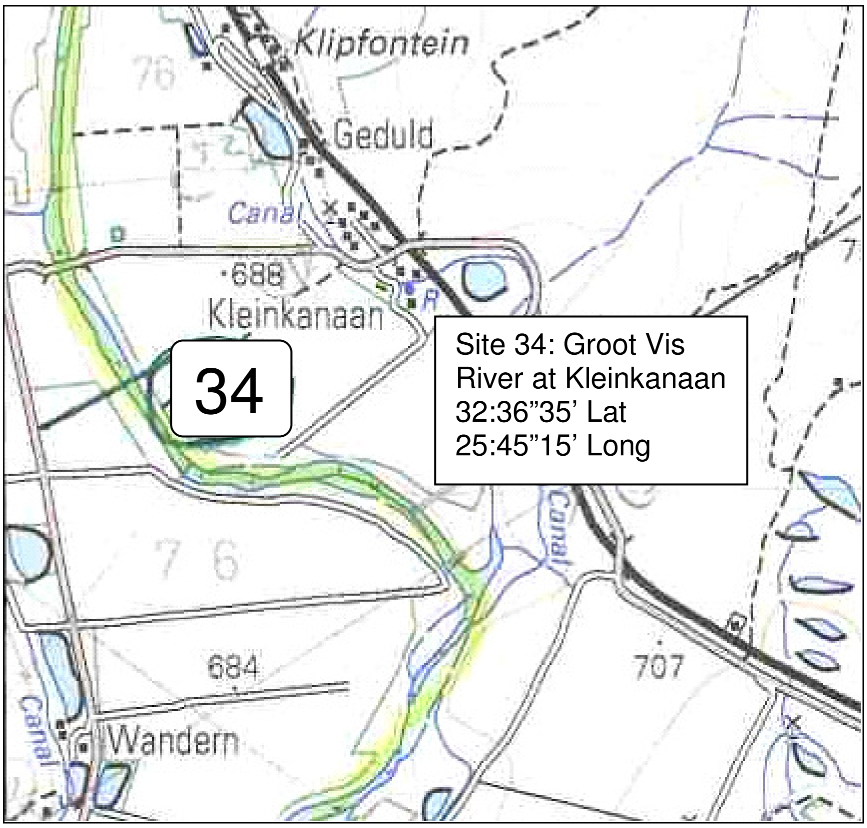
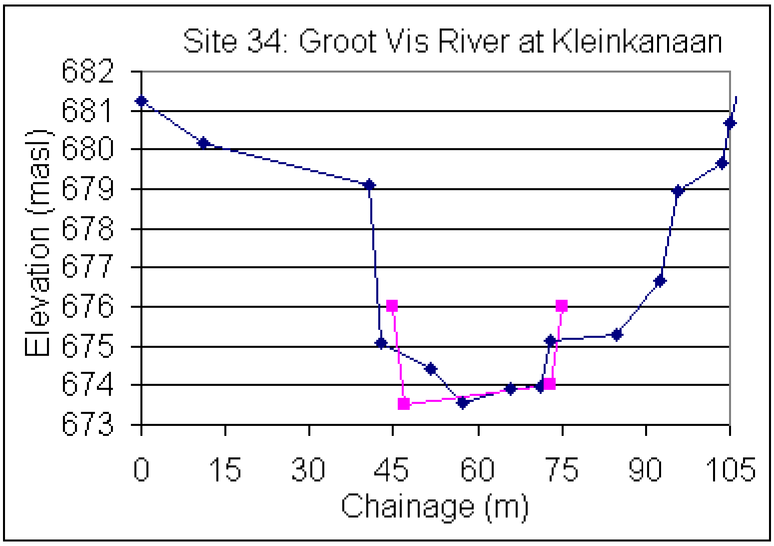



**Figure A-29: Location of Groot Vis River at Kleinkanaan**

**SITE 34**

| Measured CH. | Measured G.L. | Assumed CH. | Assumed G.L. |
|---|---|---|---|
| 0.00 | 681.23 | 45 | 676.00 |
| 11.19 | 680.16 | 47 | 673.50 |
| 40.71 | 679.11 | 73 | 674.00 |
| 42.57 | 675.08 | 75 | 676.00 |
| 51.65 | 674.42 | | |
| 57.26 | 673.56 | | |
| 65.77 | 673.93 | | |
| 71.07 | 673.99 | | |
| 73.00 | 675.16 | | |
| 84.74 | 675.31 | | |
| 92.68 | 676.66 | | |
| 95.84 | 678.96 | | |
| 103.74 | 679.64 | | |
| 105.10 | 680.67 | | |
| 108.32 | 682.43 | | |



# Appendix B: Results for the Analysis of Different Assumed River Channel Cross Sections

Three assumed channel sections were tested:

1. Assumption 1: Simple trapezium section
2. Assumption 2: Simple trapezium with a trapezium low flow slot section
3. Assumption 3: Skewed trapezium section

The following results were obtained from simulations for a standard reach with the same inflow time series, as shown in Figure B-1, while increasing the computational time step for each assumption. For each of the following plots, the Black line represents Assumption 1, the Blue line represents Assumption 2, the Green line represents Assumption 3 and the Red line represents the Input flow time series.

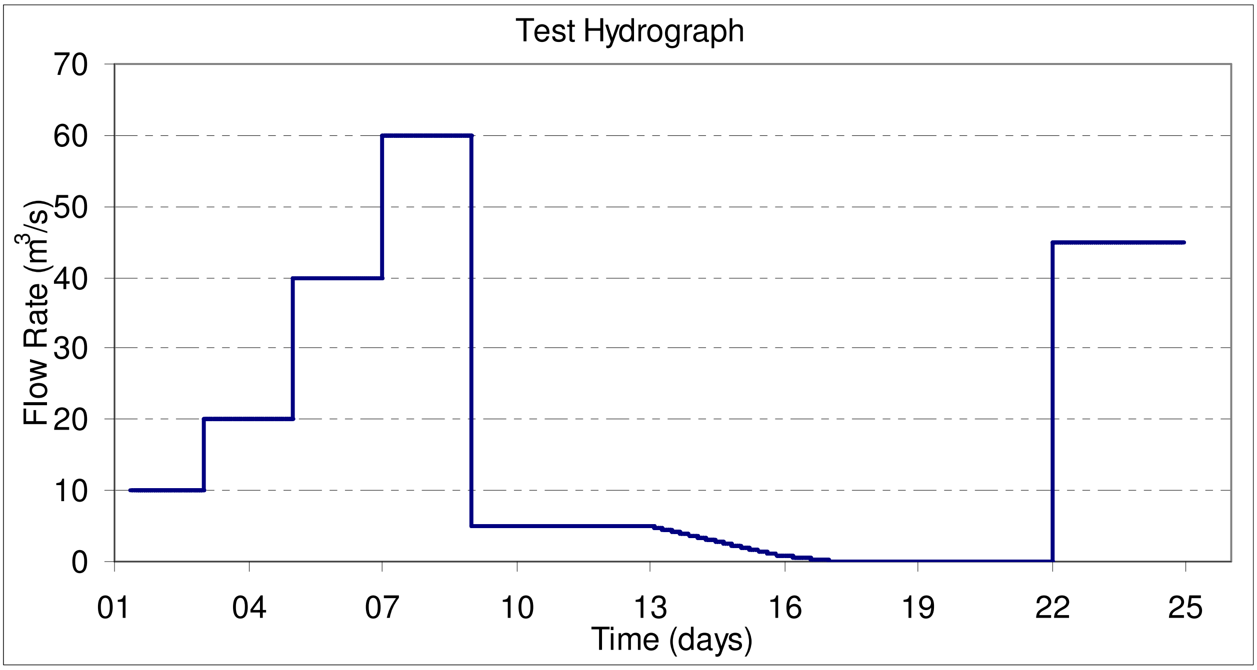



**Figure B-1: Inflow Time Series used to Test the Assumed Channel Sections**

Figure B-2 to B-4 show how each of the Assumed Sections models the reduction in flow to a gradual minimum. The deviation away from the Red (Inflow Time Series) line shows that both Assumptions 1 and 2 have some difficulty is finding the correct solution. Assumption 3 shows the strongest ability to reduce flow to a low slow rate such as is encountered during maintenance shut down periods or when tributaries have no observed inflow.

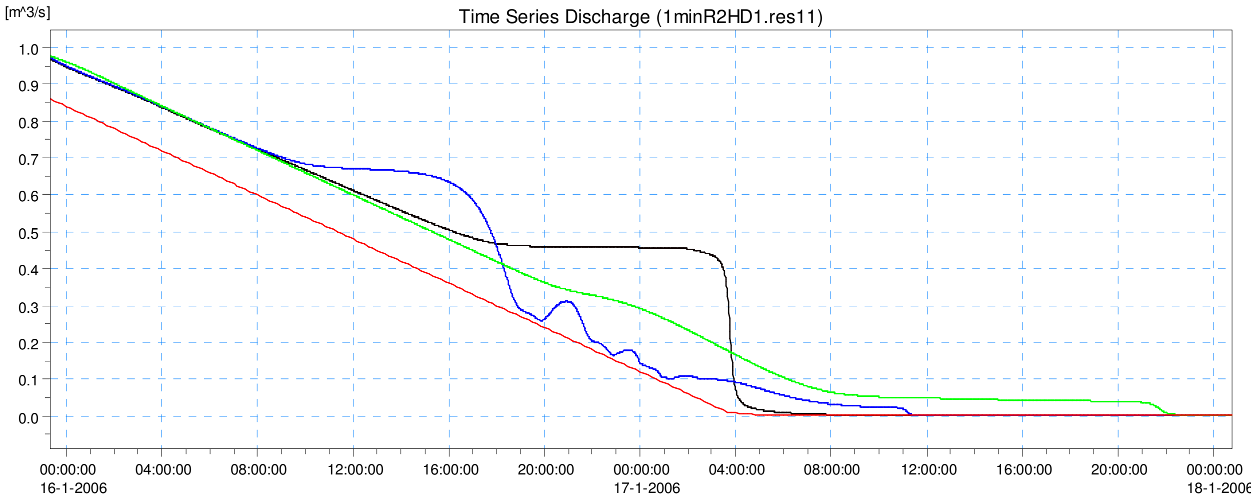



**Figure B-2: View of Flow Rate reducing to 1 L/s with a 1 min Computational Time Step**

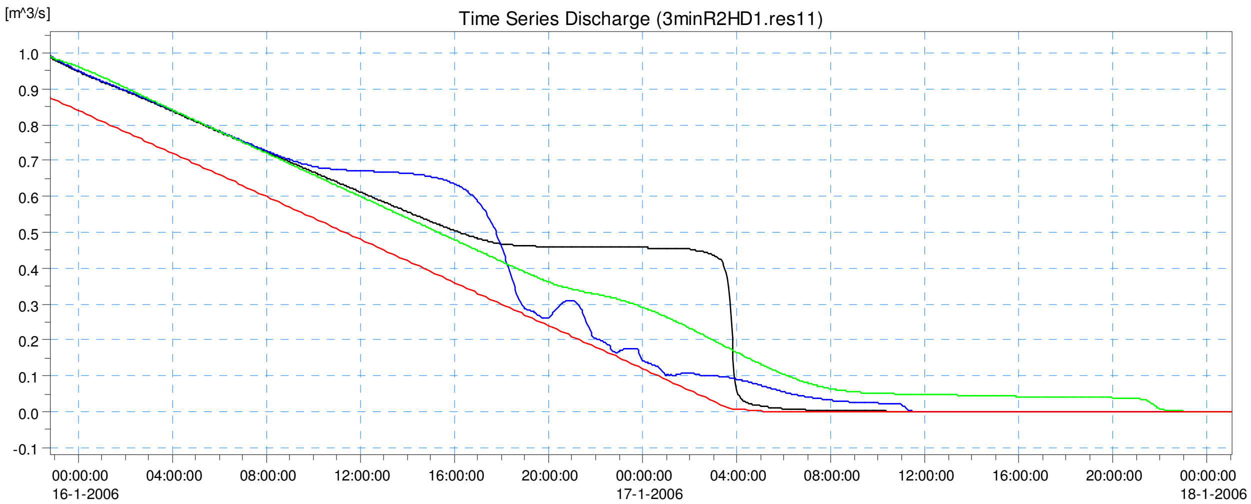



**Figure B-3: View of Flow Rate reducing to 1 L/s with a 3 min Computational Time Step**

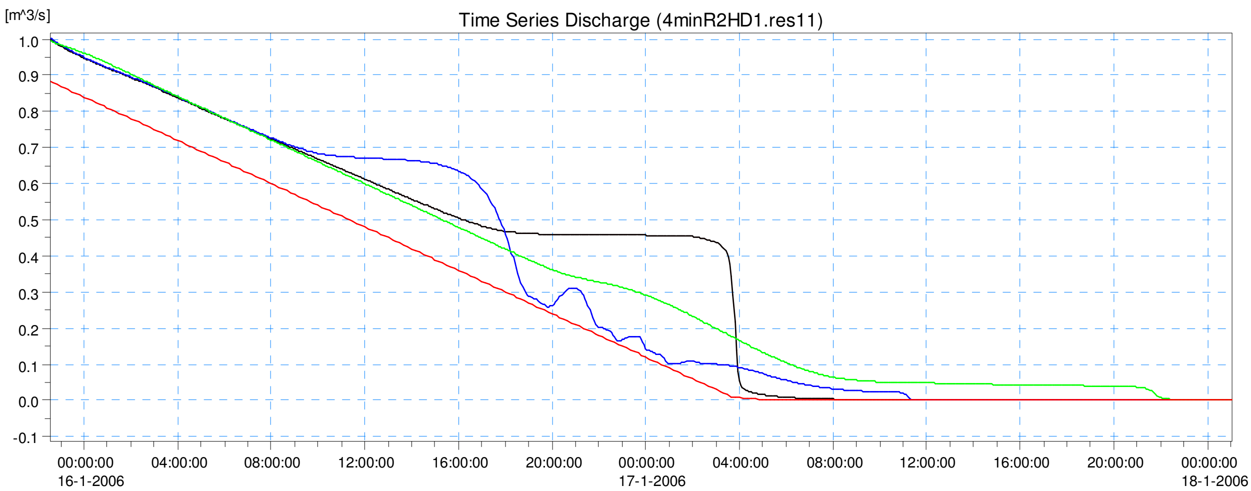



**Figure B-4: View of Flow Rate reducing to 1 L/s with a 4 min Computational Time Step**

Figures B-5 to B-7 shows how each of the Assumed Channel Sections is able to model a rapid increase in flow rate from 0.001 $m^3/s$ to 45 $m^3/s$ in 10 min. This condition exists during the opening of the OVIS Tunnel Outlet and has been experienced in the past. Experience with both the Mike 11 and InfoWorks found that this rapid increase in flow rate from a low initial rate to be the most destabilising flow event for the computational model.

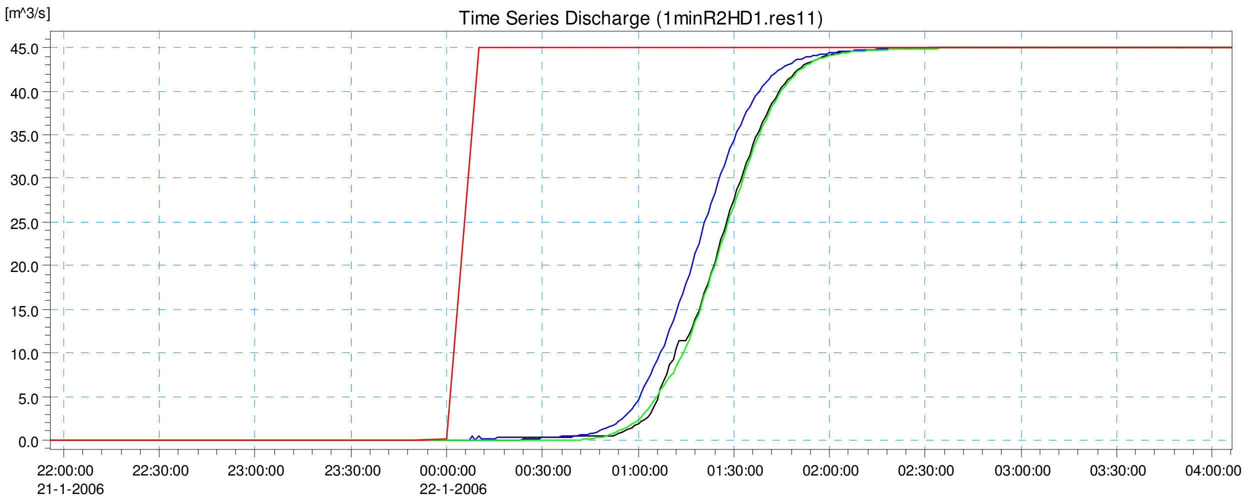



**Figure B-5: View of Flow Rate being increased rapidly from 0.001 $m^3/s$ to 45 $m^3/s$ in 10 min with a 1 min Computational Time Step**

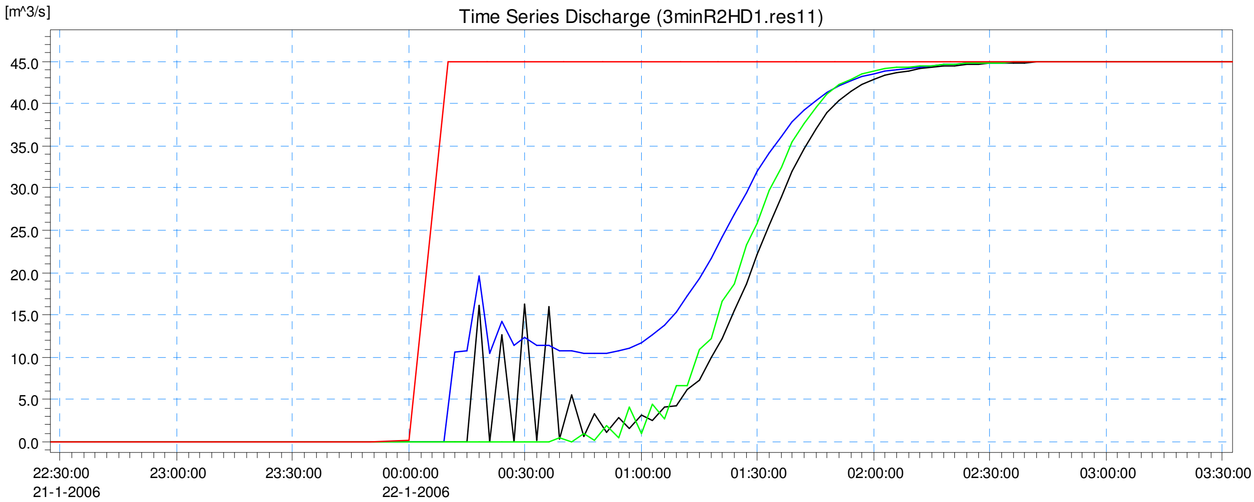



**Figure B-6: View of Flow Rate being increased rapidly from 0.001 $m^3/s$ to 45 $m^3/s$ in 10 min with a 3 min Computational Time Step**

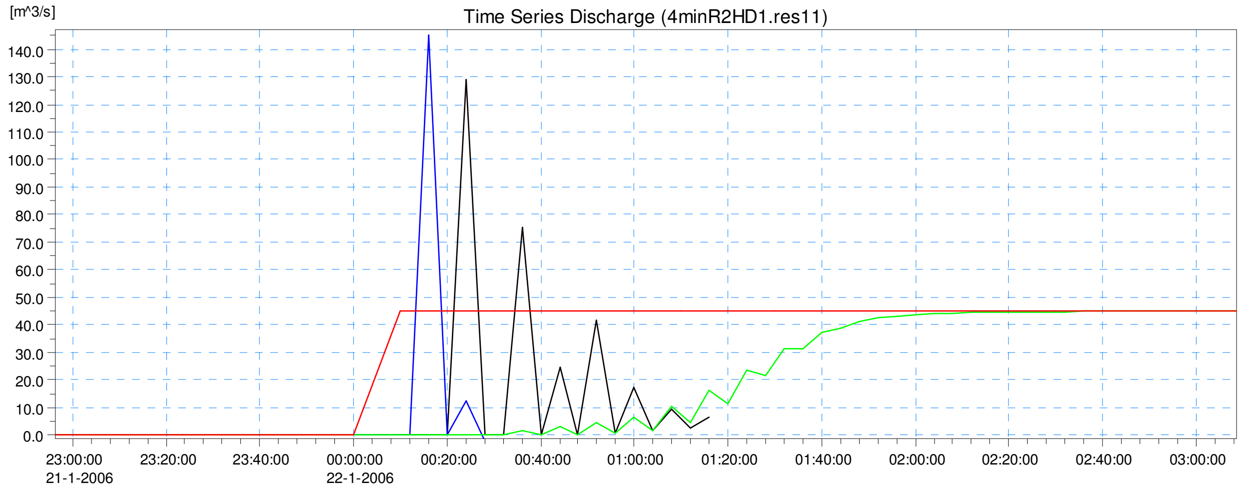



**Figure B-7: View of Flow Rate being increased rapidly from 0.001 $m^3/s$ to 45 $m^3/s$ in 10 min with a 4 min Computational Time Step**

As the computational time step increases, the stability of the modelled results decreases. This is displayed by the fluctuations observed in the results. It is clear that the un-stable fluctuations of Assumptions 1 and 2 are far greater than those observed for Assumption 3.

Figure B-8 to B-10 show the variation in the Velocity for each of the Assumed Cross Sections as the Computational Time Step is increased.

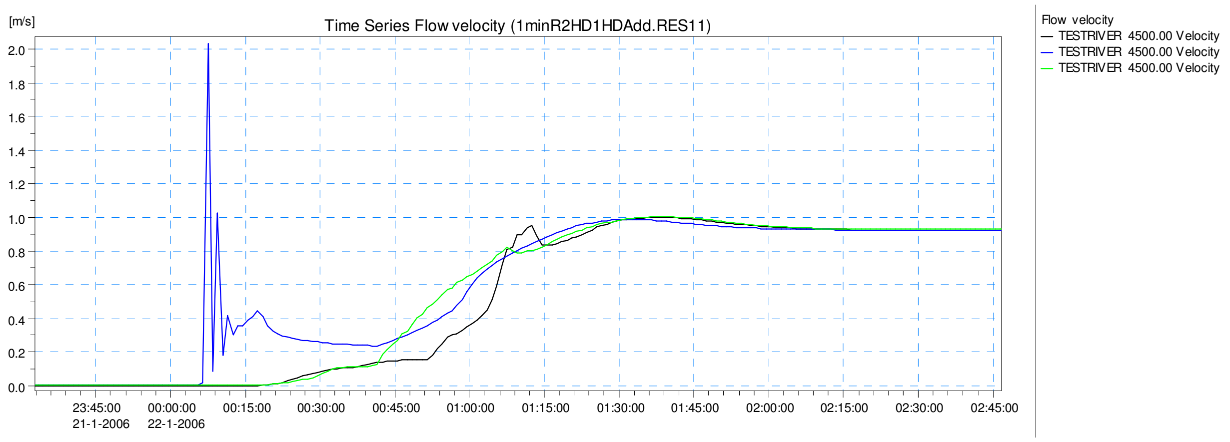



**Figure B-8: View of the change in Velocity as the Flow Rate increases from 0.001 $m^3/s$ to 45 $m^3/s$ in 10 min for a Computational Time Step of 1 min**

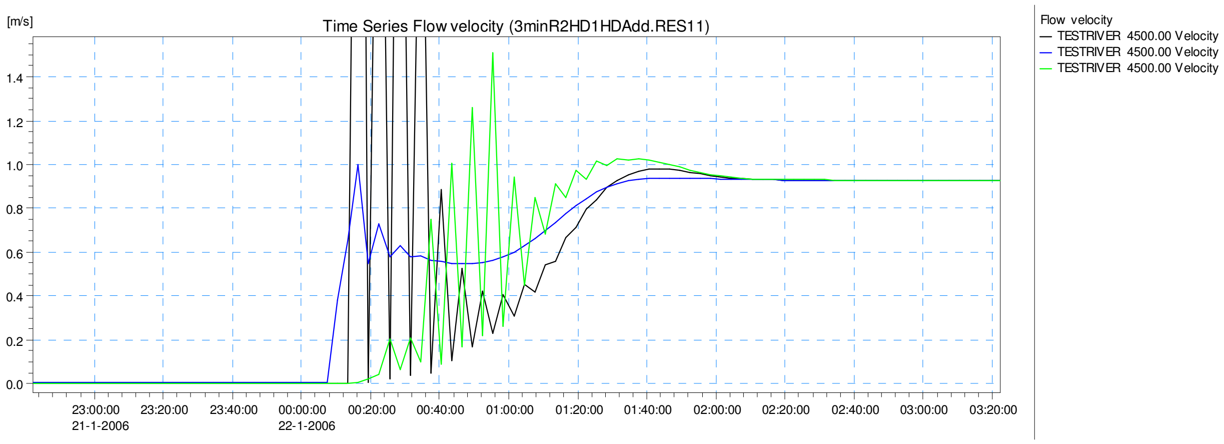



**Figure B-9: View of the change in Velocity as the Flow Rate increases from 0.001 m3/s to 45 m3/s in 10 min for a Computational Time Step of 3 min**

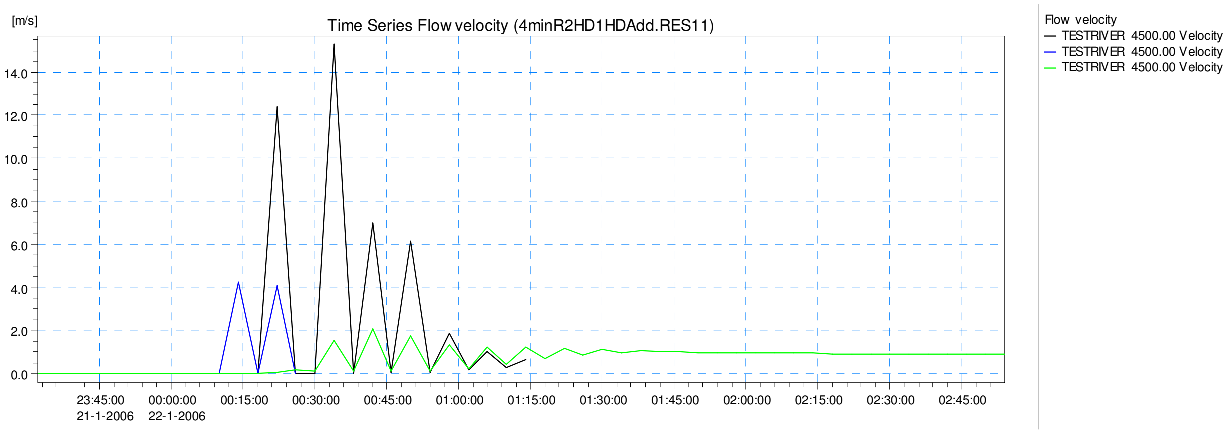



**Figure B-10: View of the change in Velocity as the Flow Rate increases from 0.001 m3/s to 45 m3/s in 10 min for a Computational Time Step of 4 min**

Figures B-8 to B-10 confirms the improved computational performance attained with Assumption 3 compared with that of Assumption 1 and 2.

The final set of plots (Figures B-11 to B-13) shows the change in the Hydraulic Radius for each of the three Assumed Channel Cross Sections as the flow rate is increased rapidly as before. One will notice clearly that Assumption 3 is the only simulation that is able to complete the simulation at the 4 min computational time step. Figure B-13 shows that even at this large time step the variation in the hydraulic radius is still smooth and regular. According to these results the skewed trapezium is found to be a more stable and reliable assumption for a river channel than the more common low flow slot.

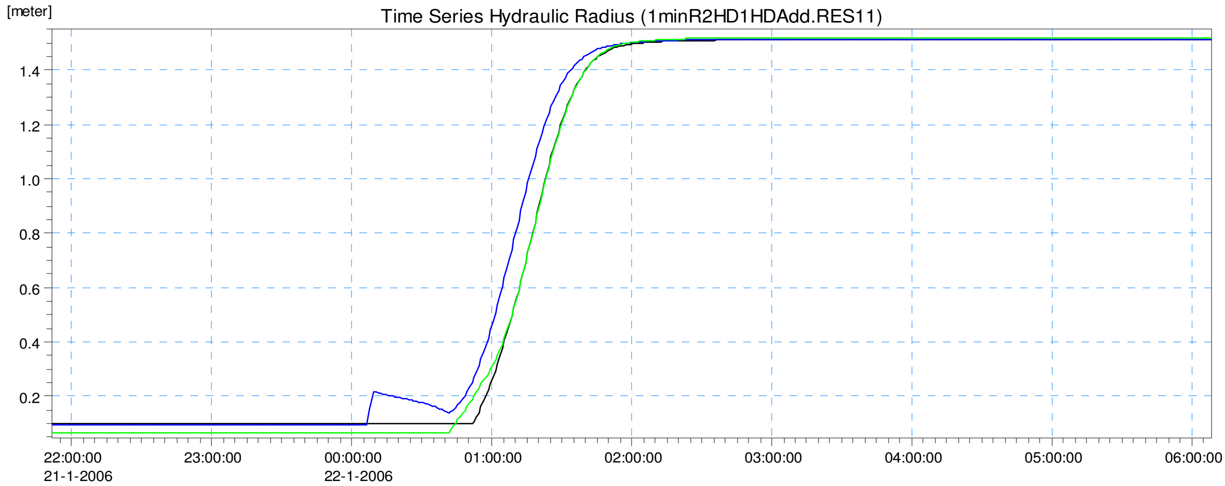



**Figure B-11: View of the change in the Hydraulic Radius as the Flow Rate increases from 0.001 m3/s to 45 m3/s in 10 min for a Computational Time Step of 1 min**

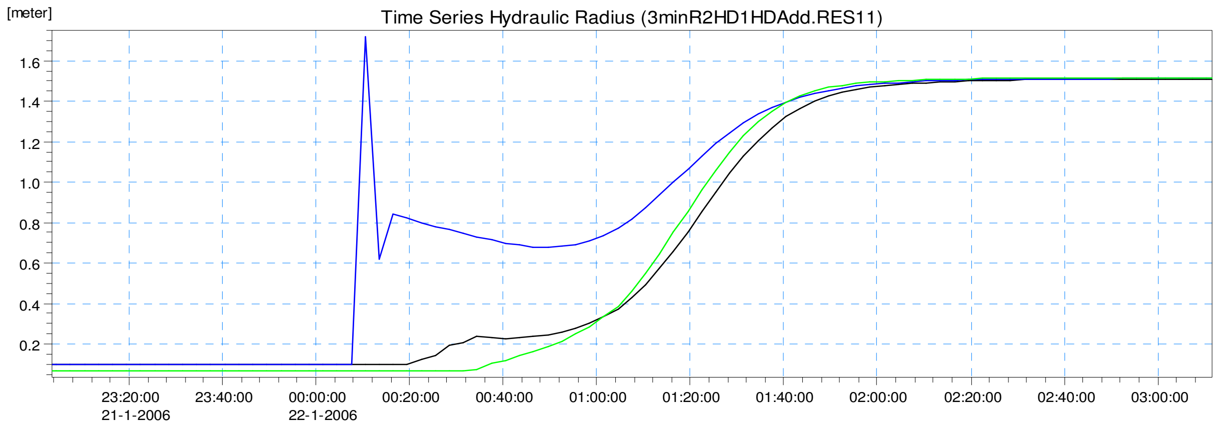



**Figure B-12: View of the change in the Hydraulic Radius as the Flow Rate increases from 0.001 m3/s to 45 m3/s in 10 min for a Computational Time Step of 3 min**

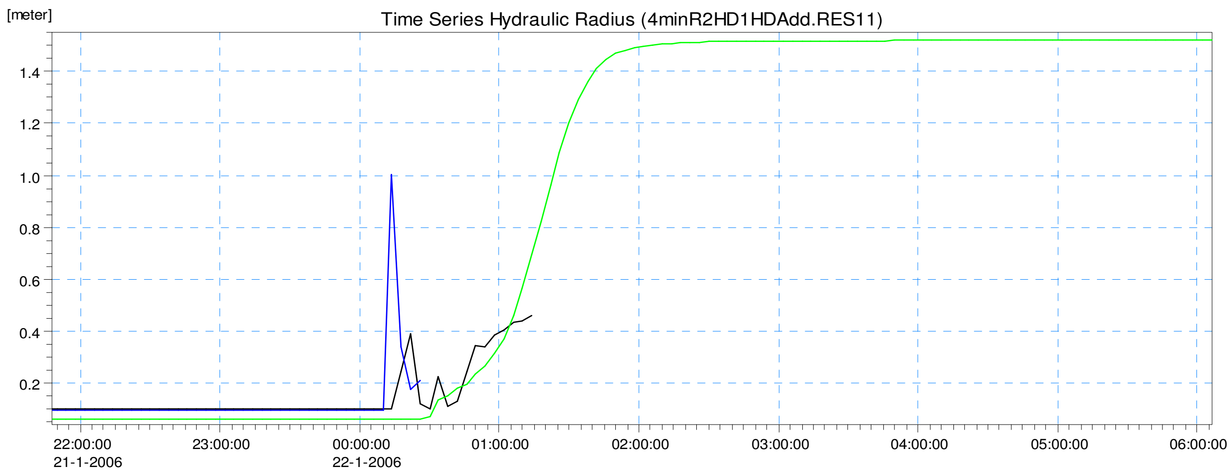



**Figure B-13: View of the change in the Hydraulic Radius as the Flow Rate increases from 0.001 m3/s to 45 m3/s in 10 min for a Computational Time Step of 4 min**

# Appendix C: Details of the Structural Limits of the Orange-Fish-Sundays Scheme

The Structural limitations of each of the seven control structures are presented below:

## OVIS Tunnel Outlet

The discharge capacity through the OVIS Tunnel is determined by the water level in Gariep Dam. The stage – flow relationship is given in Table C-1 and Figure C-1.

**Table C-1: Stage - Flow Relationship for the OVIS Tunnel**

| Gariep Water Level | Flow | Comment |
|---|---|---|
| masl | $m^3/s$ | |
| 1258.69 | 57.4 | FSL of Gariep Reservoir |
| 1253.69 | 55.2 | |
| 1248.69 | 52.8 | |
| 1243.69 | 50.3 | |
| 1238.69 | 47.7 | |
| 1236.13 | 46.4 | Lowest Gariep water level to get water into tunnel |

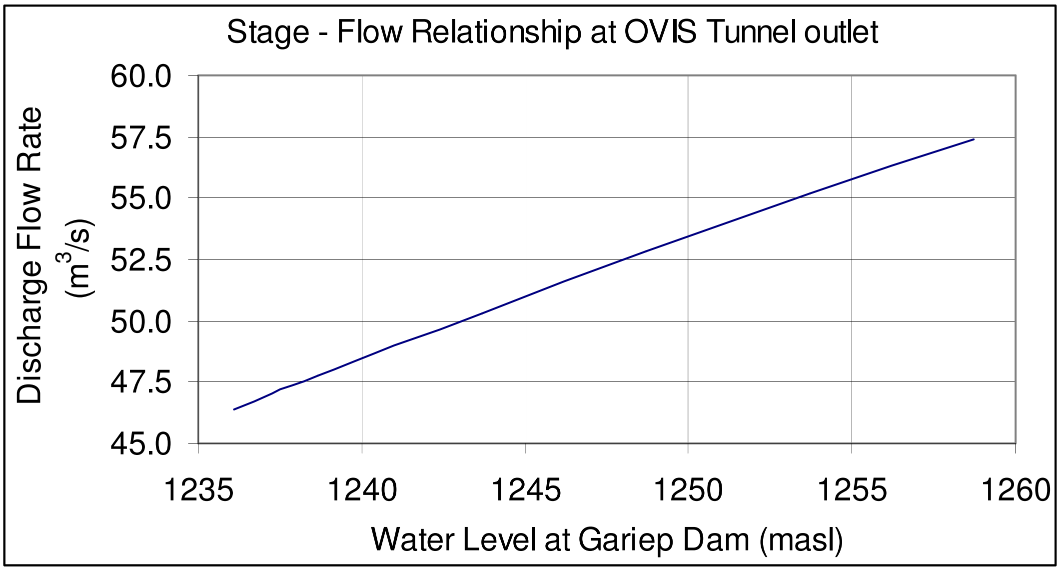



**Figure C-1: Stage - Flow Relationship for Discharge from the OVIS Tunnel**

## Grassridge Dam

No stage – flow relationship was available for the diversion gates at Grassridge Dam. From the stage – flow relationship for the flow gauging station directly below the diversion gates, the maximum calibrated flow rate is 82 $m^3$/s. From historic records the largest flow passed through the diversion gates of Grassridge Dam is 69.69 $m^3$/s. Figure C-2 shows the stage – capacity curve for Grassridge Dam and Table C-2 gives the data as a table.

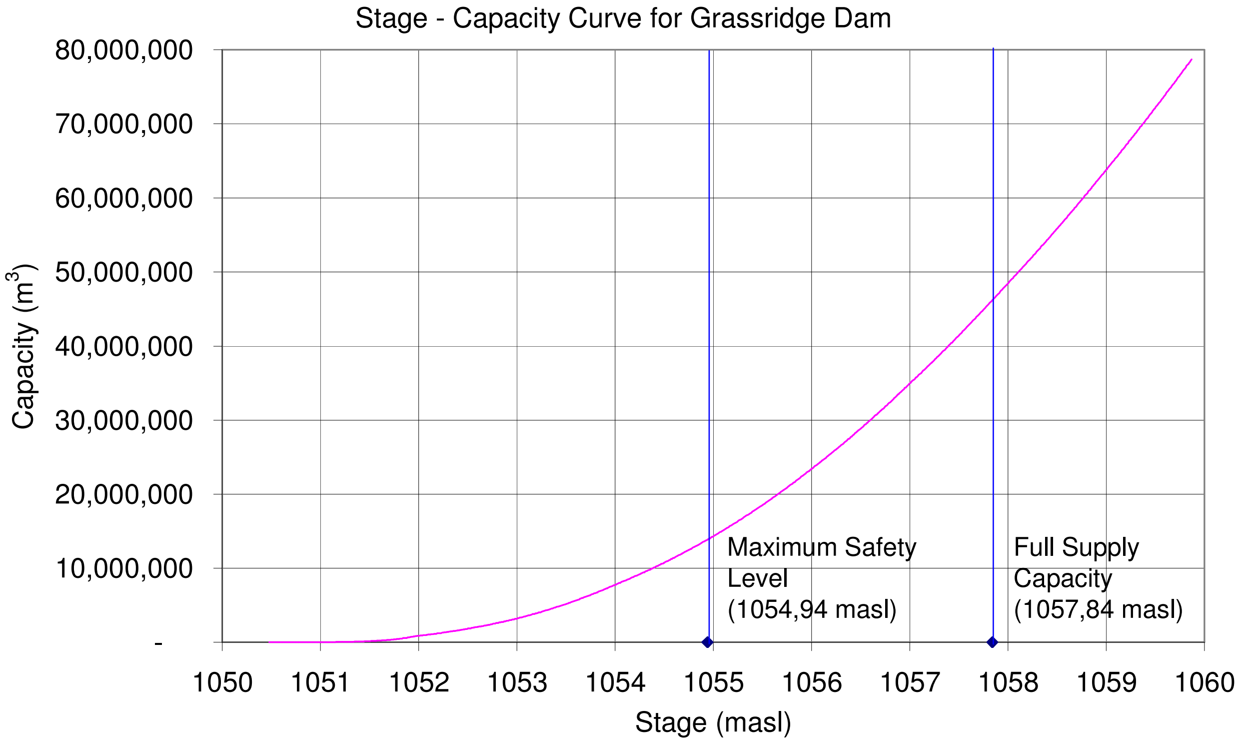



**Figure C-2: Stage - Capacity Curve for Grassridge Dam**

**Table C-2: Elevation - Capacity Table for Grassridge Dam**

| Elevation (masl) | % of FSC | Capacity ($m^3$) |
|---|---|---|
| 1050.88 | 0 | 0 |
| 1051.88 | 1.3 | 600,463 |
| 1052.88 | 6.1 | 817,559 |
| 1053.88 | 15.3 | 7,066,993 |
| 1054.88 | 29 | 13,394,955 |
| 1055.88 | 48 | 22,170,960 |
| 1056.88 | 72.4 | 33,441,198 |
| 1057.88 | 101.2 | 46,743,774 |

Grassridge Dam has an auxiliary spillway at the Full Supply Capacity level of 1057.84 masl. The stage discharge curve for this spillway is given in Figure C-3. This data was received from the DWAF Hydro Office. Table C-3 gives this data as a table.

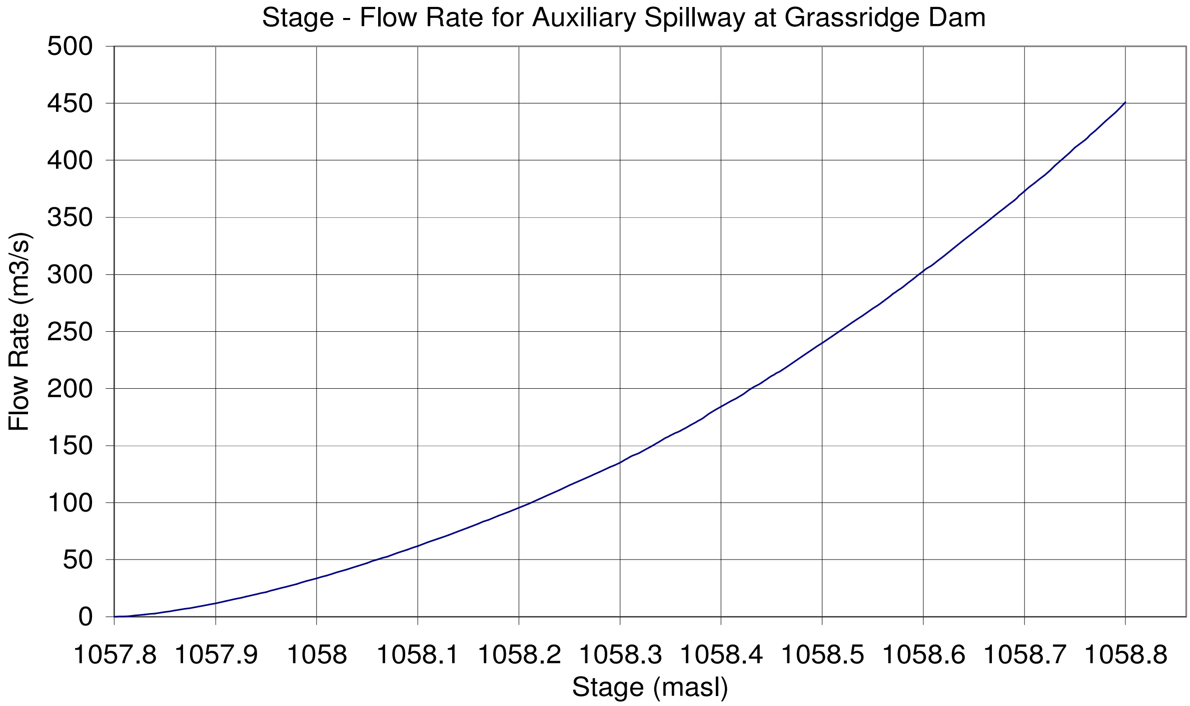



**Figure C-3: Stage - Flow Rate for the Auxiliary Spillway at Grassridge Dam**

**Table C-3: Elevation - Discharge Table for Grassridge Dam Auxiliary Spillway**

| Elevation (masl) | Auxiliary Spillway Discharge ($m^{3/s}$) |
|---|---|
| 1057.84 | 0 |
| 1057.94 | 11.9 |
| 1058.04 | 33.7 |
| 1058.14 | 62 |
| 1058.24 | 95.6 |
| 1058.34 | 135 |
| 1058.44 | 184 |
| 1058.54 | 240 |
| 1058.64 | 303 |
| 1058.74 | 373 |
| 1058.84 | 451 |

## Elandsdrift Dam

Elandsdrift Dam acts as a diversion weir to divert flow from the Groot Vis River to the Little Fish Canal. The maximum capacity of the canal is 20 $m^3/s$. Flow tests were conducted on the two 1.2 m Diameter sleeve valves that divert flow from the dam to the Groot Vis River. The results show the following stage – flow relationship given in Table C-4. There is an additional 0.9 m diameter sleeve valve that was under going maintenance and could not be tested.

**Table C-4: Stage - Flow Rate Relationship for the 1.2 m Diameter Sleve Valves at Elandsdrift Dam**

| Water Level | Flow Rate |
|---|---|
| **masl** | **$m^3/s$** |
| 731.83 | 1.430 |
| 732.83 | 2.022 |
| 733.83 | 2.477 |
| 734.83 | 2.860 |
| 735.83 | 3.198 |
| 736.83 | 3.503 |
| 737.83 | 3.783 |
| 738.83 | 4.045 |
| 739.83 | 4.290 |
| 740.83 | 4.522 |
| 741.83 | 4.743 |
| 742.83 | 4.954 |
| 743.83 | 5.156 |
| 744.83 | 5.350 |
| 745.83 | 5.538 |

Elandsdrift Dam has a stage – capacity curve as given in Figure C-4 and shown in Table C-5.

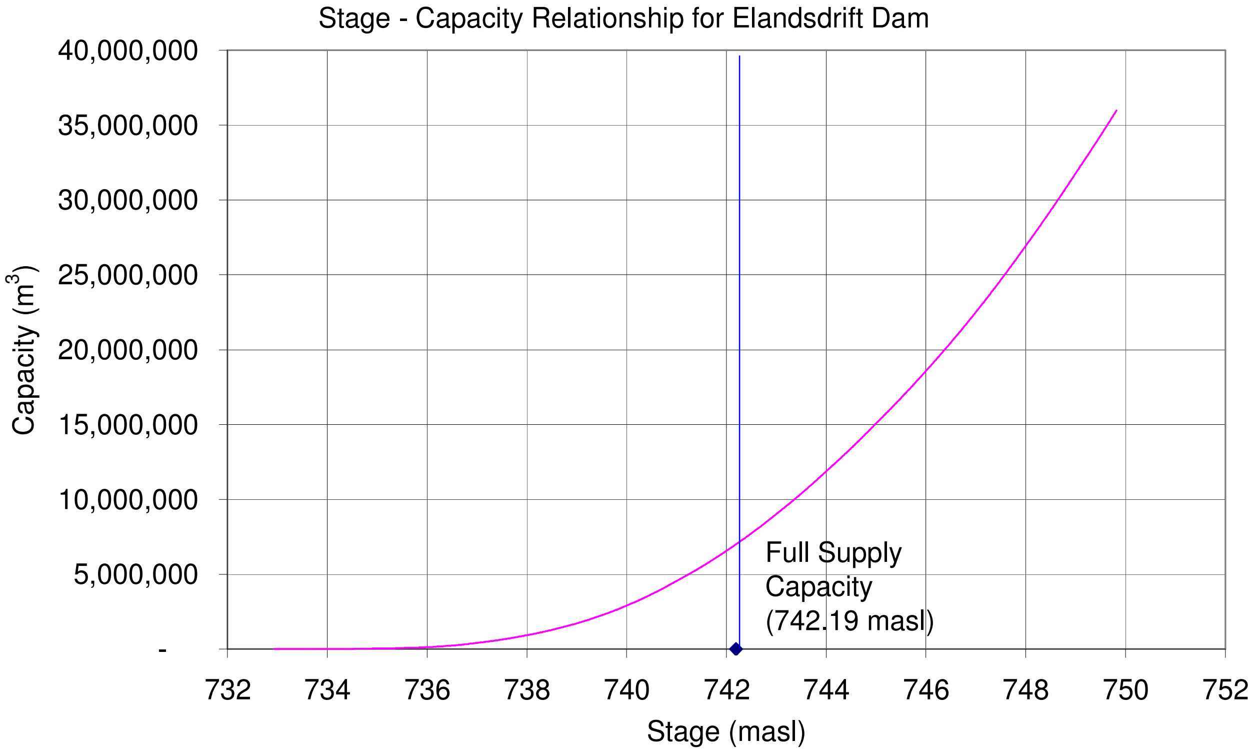



**Figure C-4: Stage - Capacity Curve for Elandsdrift Dam**

**Table C-5: Elevation - Capacity Table for Elandsdrift Dam**

| Elevation (masl) | % of FSC | Capacity ($m^3$) |
|---|---|---|
| 730.83 | 0 | 0 |
| 733.83 | 0.1 | 6,391 |
| 734.83 | 0.5 | 31,962 |
| 735.83 | 1.5 | 105,144 |
| 736.83 | 5 | 349,000 |
| 737.83 | 11.8 | 823,220 |
| 738.83 | 22.4 | 1,565,189 |
| 739.83 | 38.2 | 2,672,689 |
| 740.83 | 60.6 | 4,237,932 |
| 741.83 | 88.4 | 6,187,882 |
| 742.83 | 122.5 | 8,571,402 |
| 743.83 | 162.3 | 11,359,845 |
| 744.83 | 207.2 | 14,501,892 |

There are 4 Radial Flood Gates with the following stage – discharge relationship as given in Table C-6 (Data collected from the Water Control Officer at Elandsdrift Dam). A more comprehensive plot of the different discharge flow rates for different water levels and gate openings for one gate is shown in Figure C-5.

**Table C-6: Stage - Flow Relationship for the Radial Flood Gates at Elandsdrift Dam**

| Maximum Discharge for given Water Level | | |
|---|---|---|
| H (masl) | Q (one gate) | Q (All Gates) |
| 735.83 | 224.30 | 897.20 |
| 736.83 | 247.80 | 991.20 |
| 737.83 | 353.50 | 1414.00 |
| 738.83 | 469.30 | 1877.20 |
| 739.83 | 594.50 | 2378.00 |
| 740.83 | 630.00 | 2520.00 |
| 741.83 | 767.70 | 3070.80 |
| 742.83 | 913.40 | 3653.60 |
| 743.83 | 954.80 | 3819.20 |

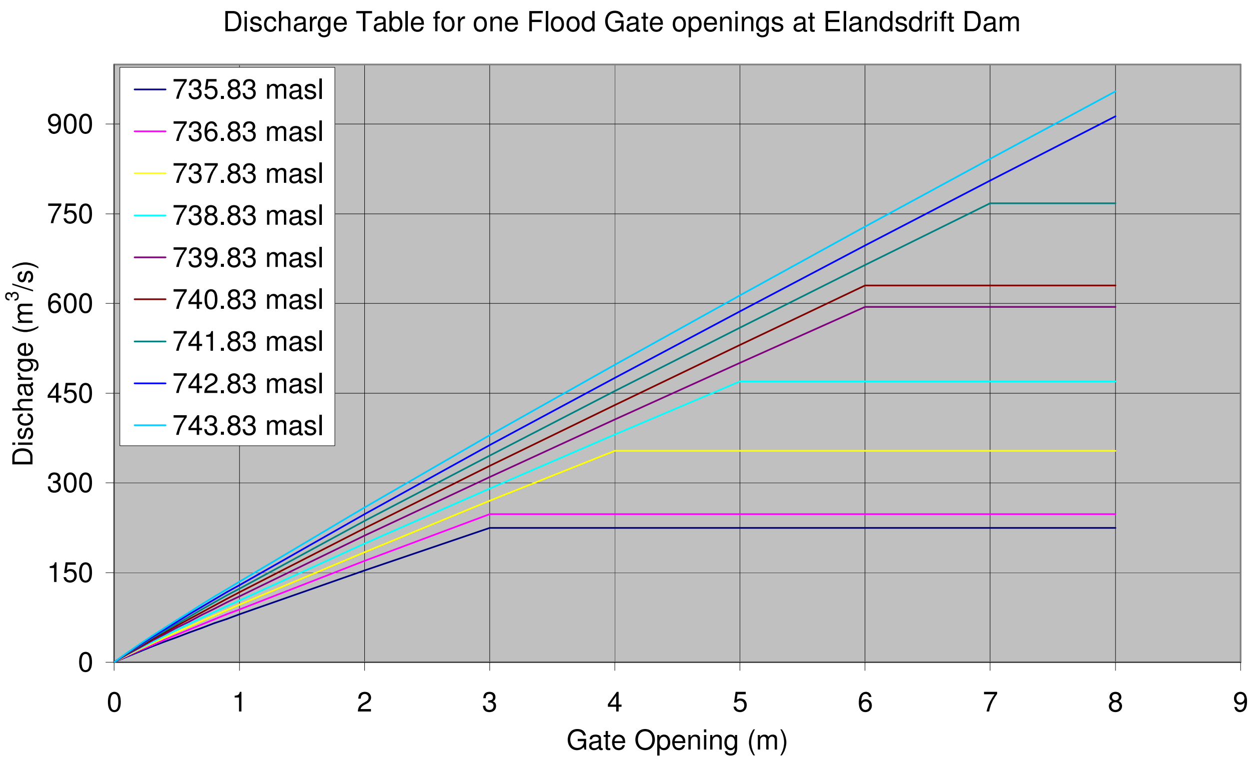



**Figure C-5: Discharge Flow Rate for one Radial Flood Gate at Elandsdrift Dam for Various Water Levels and Gate Openings**

## De Mistkraal Dam

De Mistkraal Dam diverts flow from the Little Fish River to the Skoenmakers Canal. The maximum possible diversion to the canal is 19 $m^3/s$. Flow can be passed to the Little Fish River via two sleeve valves. These sleeve valves have not been able to be calibrated and as such an assumed maximum discharge of 10 $m^3/s$ has been made.

De Mistkraal Dam is a concrete gravity dam with a stepped spillway in the main channel. Figure C-6 shows the stage – flow relationship for the free overflow spillway at De Mistkraal Dam (DWAF Hydrology Office) and Table C-7 gives the data in tabular form.

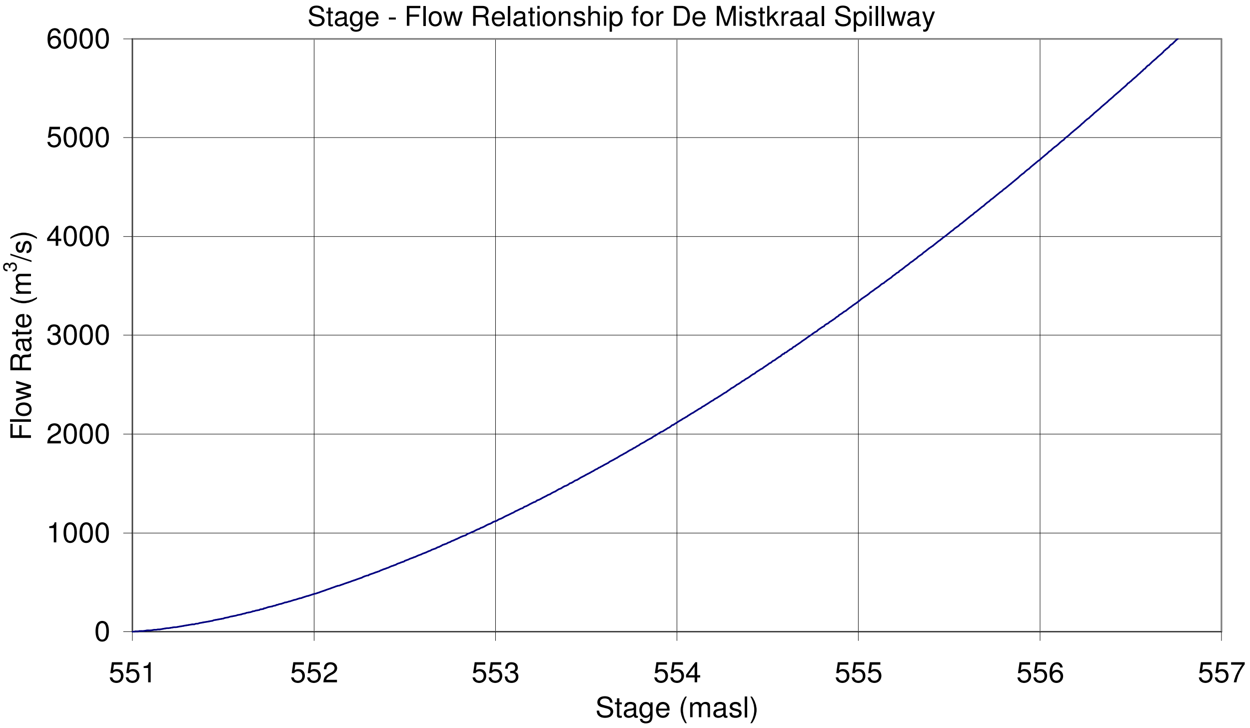



**Figure C-6: Stage - Flow Rate Relationship for the Free Overflow Spillway at De Mistkraal Dam**

**Table C-7: Elevation - Discharge Table for De Mistkraal Dam Spillway**

| Elevation (masl) | Spillway Discharge ($m^3/s$) |
|---|---|
| 550.96 | 0 |
| 551 | 1.9 |
| 551.2 | 37 |
| 551.4 | 97.5 |
| 551.6 | 177 |
| 551.8 | 273 |
| 552 | 383 |
| 553 | 1119 |
| 554 | 2117 |
| 555 | 3343 |
| 556 | 4779 |

De Mistkraal Dam has a small storage capacity as shown in Figure C-7. This is given in Table C-8 in tabular form.

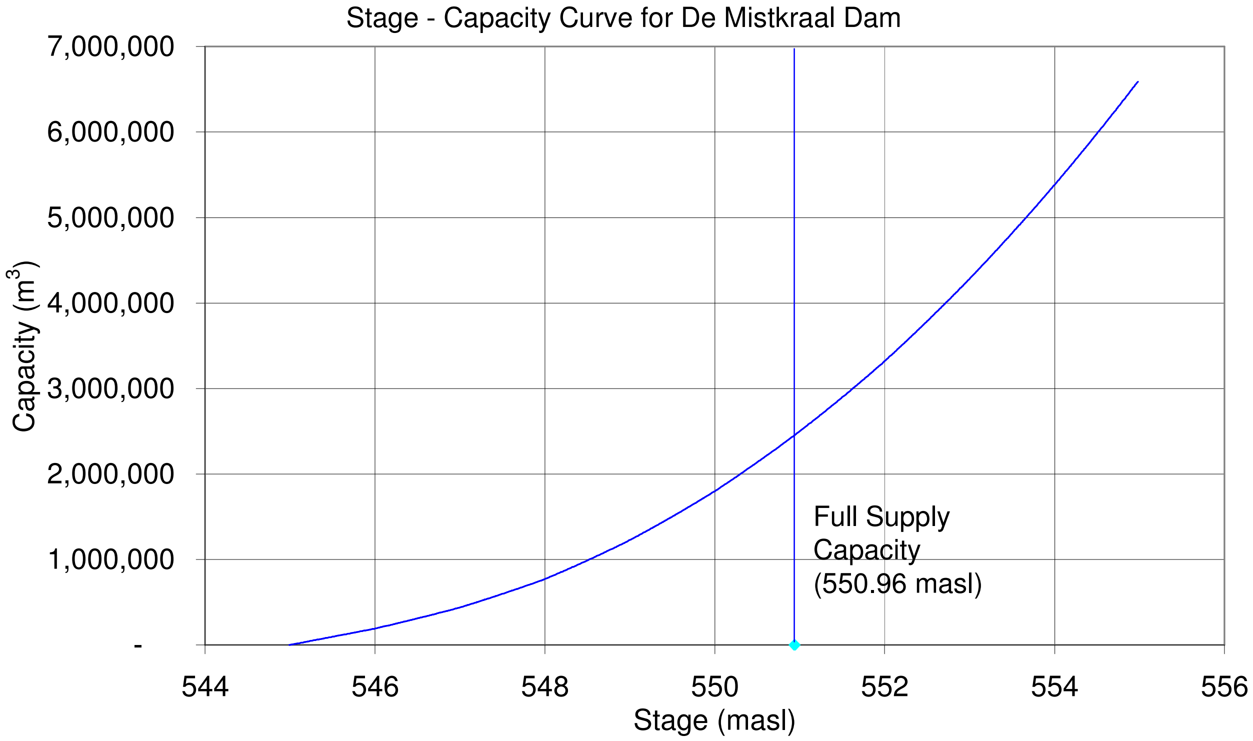



**Figure C-7: Stage - Capacity Relationship for De Mistkraal Dam**

**Table C-8: Elevation - Capacity Table for De Mistkraal Dam**

| Elevation (masl) | % FSC | Capacity ($m^3$) |
|---|---|---|
| 545 | 0.2 | 1,903 |
| 546 | 7.9 | 192,956 |
| 547 | 17.9 | 439,574 |
| 548 | 31.4 | 771,777 |
| 549 | 50 | 1,227,699 |
| 550 | 73.3 | 1,799,955 |
| 551 | 101.8 | 2,500,506 |
| 552 | 135.2 | 3,320,476 |
| 553 | 174.5 | 4,284,029 |
| 554 | 219.2 | 5,382,387 |

## Darlington Dam

Darlington Dam has a main spillway and an auxiliary spillway on the right embankment and 6 sleeve valves on the left embankment of the dam. The stage – discharge relationships for each of the spillways are shown in Figure C-8 and in tabular form in Table C-9. The stage – discharge relationships for one of the sleeve valves is shown in Figure C-9 and in tabular form in Table C-10.

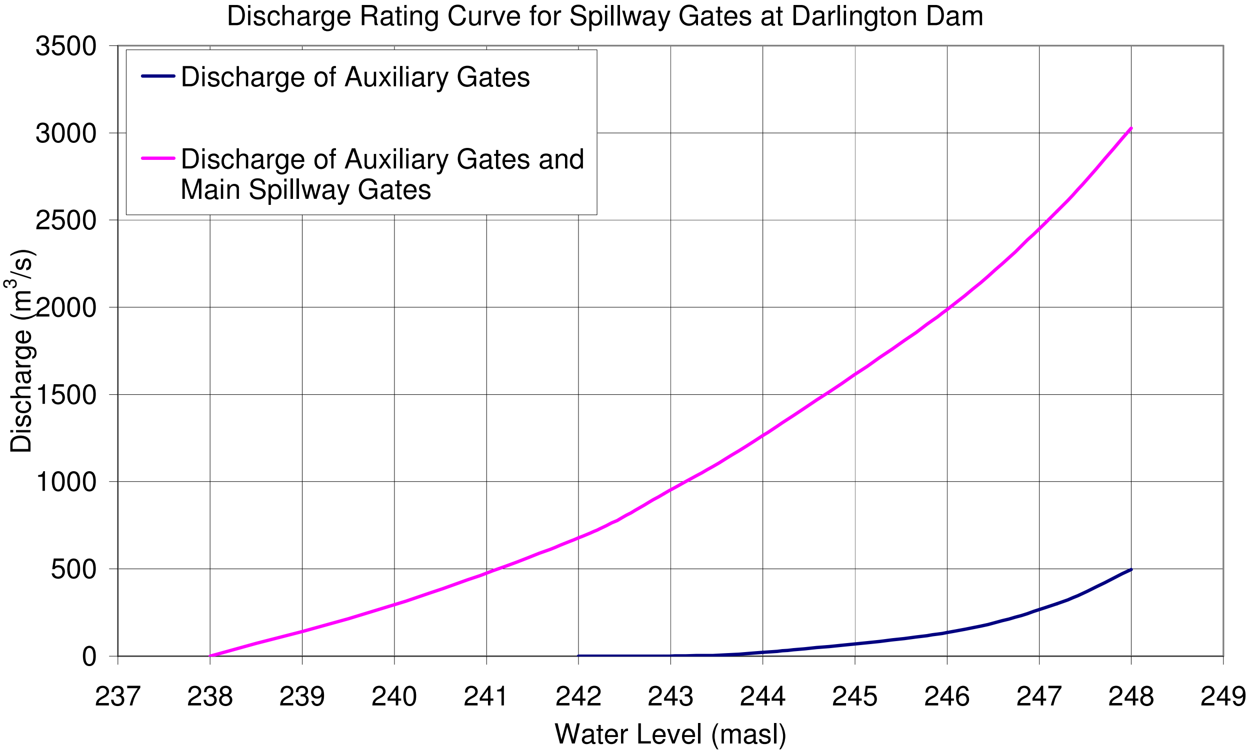



**Figure C-8: Stage - Discharge Relationships for the Spillways at Darlington Dam**

**Table C-9: Stage - Discharge Table for Darlington Dam's Spillways**

| Water Level in Dam (masl) | Discharge of Auxiliary Gates ($m^3/s$) | Discharge of Auxiliary Gates and Main Spillway Gates ($m^3/s$) |
|---|---|---|
| 238 | 0.0 | 0.0 |
| 240 | 0.0 | 293.5 |
| 242 | 0.0 | 677.4 |
| 242.925 | 0.0 | 930.0 |
| 244 | 20.0 | 1264.5 |
| 246 | 135.5 | 1987.1 |
| 247.19 | 300.0 | 2550.0 |
| 248 | 496.8 | 3025.8 |

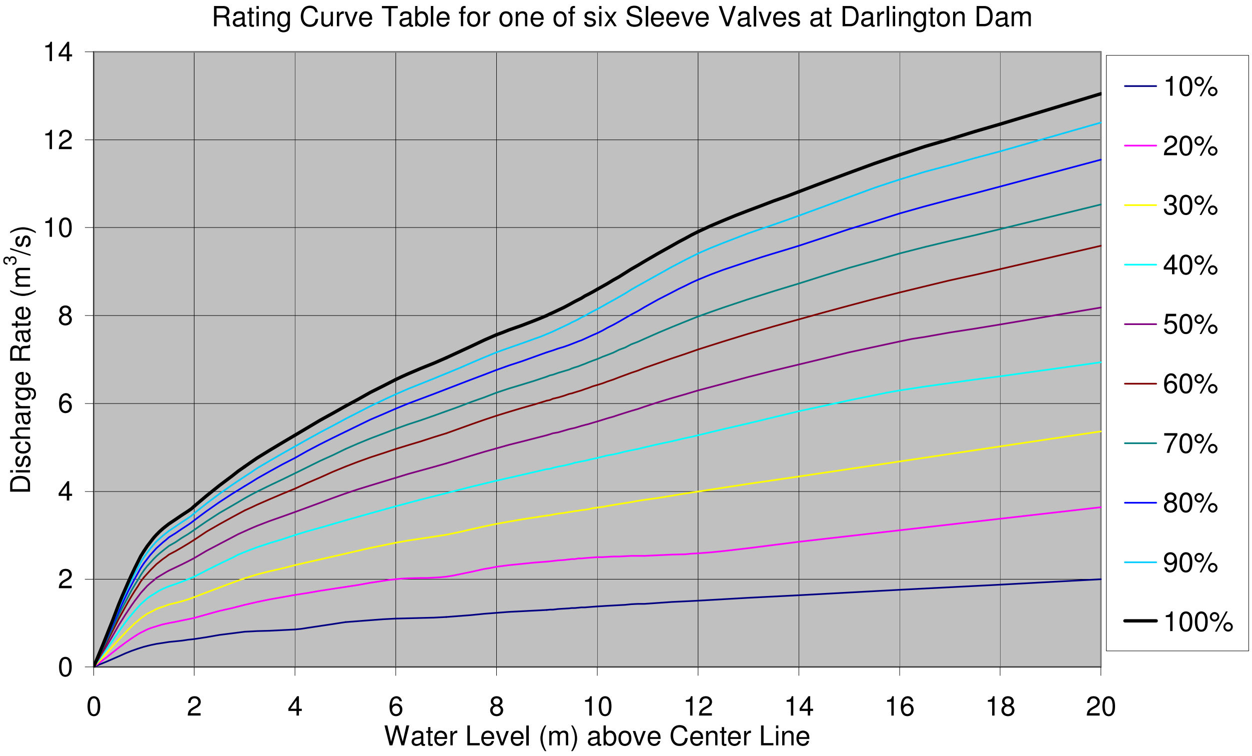



**Figure C-9: Stage - Discharge Relationship for one Sleeve Valve at Darlington Dam**

**Table C-10: Stage - Discharge Table for Darlington Dam's Sleeve Valves**

| Head (m) | Water Level (masl) | Percentage open | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 10% | 20% | 30% | 40% | 50% | 60% | 70% | 80% | 90% | 100% |
| 0 | 236.475 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 1 | 237.475 | 0.460 | 0.820 | 1.160 | 1.500 | 1.770 | 2.040 | 2.200 | 2.360 | 2.500 | 2.640 |
| 2 | 238.475 | 0.640 | 1.120 | 1.590 | 2.060 | 2.480 | 2.900 | 3.120 | 3.340 | 3.500 | 3.660 |
| 3 | 239.475 | 0.800 | 1.420 | 2.020 | 2.620 | 3.090 | 3.560 | 3.840 | 4.120 | 4.340 | 4.560 |
| 4 | 240.475 | 0.860 | 1.640 | 2.320 | 3.000 | 3.530 | 4.060 | 4.410 | 4.760 | 5.020 | 5.280 |
| 5 | 241.475 | 1.020 | 1.820 | 2.580 | 3.340 | 3.950 | 4.560 | 4.960 | 5.360 | 5.650 | 5.940 |
| 6 | 242.475 | 1.100 | 2.000 | 2.830 | 3.660 | 4.310 | 4.960 | 5.420 | 5.880 | 6.210 | 6.540 |
| 7 | 243.475 | 1.140 | 2.060 | 3.010 | 3.960 | 4.640 | 5.320 | 5.820 | 6.320 | 6.680 | 7.040 |
| 8 | 244.475 | 1.240 | 2.280 | 3.260 | 4.240 | 4.980 | 5.720 | 6.240 | 6.760 | 7.160 | 7.560 |
| 9 | 245.475 | 1.300 | 2.400 | 3.450 | 4.500 | 5.280 | 6.060 | 6.610 | 7.160 | 7.580 | 8.000 |
| 10 | 246.475 | 1.380 | 2.500 | 3.630 | 4.760 | 5.590 | 6.420 | 7.010 | 7.600 | 8.150 | 8.600 |
| 12 | 248.475 | 1.515 | 2.590 | 4.000 | 5.273 | 6.295 | 7.227 | 7.977 | 8.818 | 9.409 | 9.909 |
| 14 | 250.475 | 1.636 | 2.852 | 4.341 | 5.818 | 6.886 | 7.909 | 8.727 | 9.591 | 10.273 | 10.818 |
| 16 | 252.475 | 1.758 | 3.113 | 4.682 | 6.295 | 7.409 | 8.523 | 9.409 | 10.318 | 11.091 | 11.659 |
| 18 | 254.475 | 1.879 | 3.375 | 5.023 | 6.614 | 7.795 | 9.057 | 9.966 | 10.932 | 11.739 | 12.352 |
| 20 | 256.475 | 2.000 | 3.636 | 5.364 | 6.932 | 8.182 | 9.591 | 10.523 | 11.545 | 12.386 | 13.045 |

Darlington Dam is restrained by dam safety restrictions such that the auxiliary gates are to be kept open at all times in case of their failure during an emergency operation. The stage – capacity curve for Darlington Dam is shown in Figure C-10 and in tabular form in Table C-11.

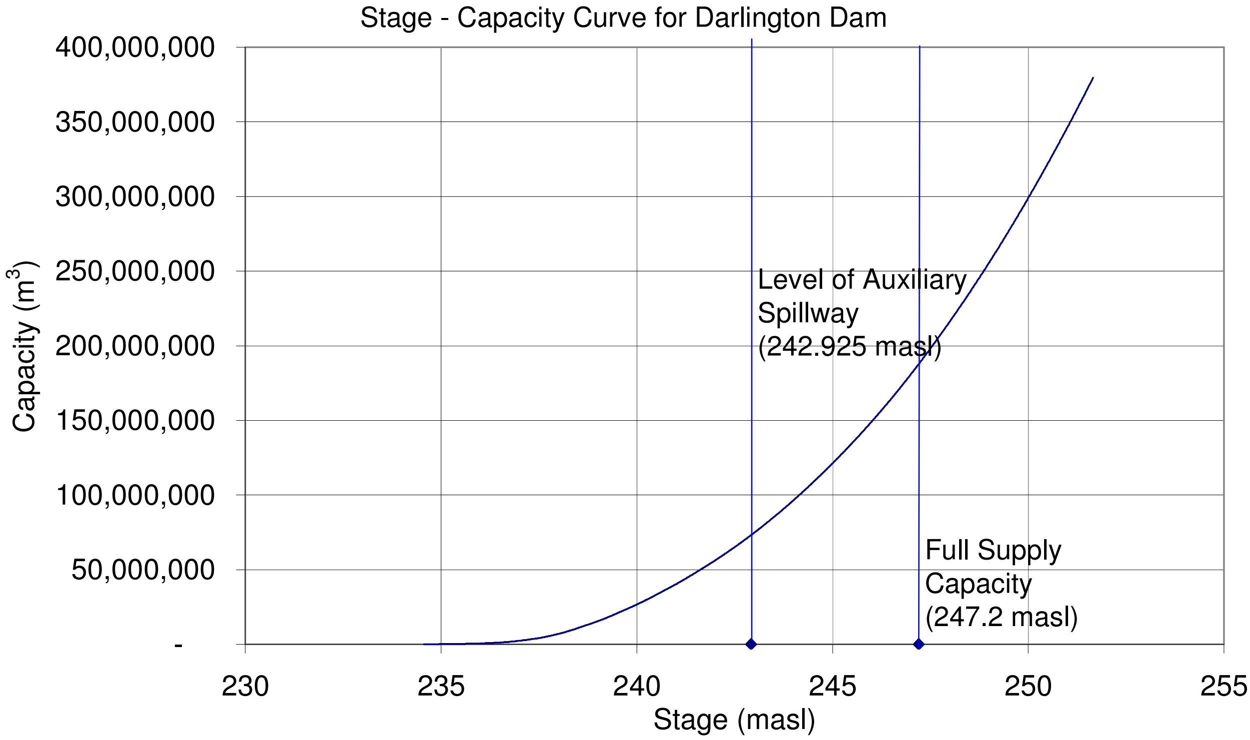



**Figure C-10: Stage - Capacity Curve for Darlington Dam**

**Table C-11: Stage - Capacity Table for Darlington Dam**

| Elevation (masl) | % FSC | Capacity ($m^3$) |
|---|---|---|
| 234.56 | 0 | 0 |
| 235 | 0 | 78,582 |
| 236 | 0.3 | 494,520 |
| 237 | 1.2 | 2,310,444 |
| 238 | 3.7 | 6,944,563 |
| 239 | 8.2 | 15,408,683 |
| 240 | 14.2 | 26,625,771 |
| 241 | 21.4 | 40,233,391 |
| 242 | 29.9 | 56,182,439 |
| 243 | 39.8 | 74,767,921 |
| 244 | 51.4 | 96,432,691 |
| 245 | 64.6 | 121,233,720 |
| 246 | 79.5 | 149,229,200 |
| 247 | 96.4 | 180,885,280 |
| 248 | 115.3 | 216,438,740 |
| 249 | 136.3 | 255,758,630 |
| 250 | 159.3 | 299,050,350 |
| 251 | 184.5 | 346,378,340 |

# Appendix D: Full Calibration Results

The final full calibration results are presented for each of the calibration locations in this Appendix.

**7th Full Calibration Run - OFS-RT Model from 29th July to 20 May 2006.**

| Reach Name: | Return Flow Values (m3/s): | Manning's Number: | Next Values: | Next n: |
|---|---|---|---|---|
| Teebus | 0.2 | 0.037 | | |
| Groot Brak1 | 0.7 | 0.040 | | |
| Groot Brak 2 | 0.70 | 0.040 | | |
| Groot Vis 1 | 1.0 | 0.033 | | |
| Groot Vis 2 | 2 | 0.045 | | |
| Groot Vis 3 | 1 | 0.070 | | |
| Groot Vis 4 | 0 | 0.050 | | |
| Little Fish 1 | 0.25 | 0.055 | | |
| Little Fish Canal | 0 | 0.020 | | |
| Little Fish 2 | 0.5 | 0.055 | | |
| Skoenmakers Canal | 0 | 0.025 | | |
| Skoenmakers | -0.5 | 0.055 | | |
| Sundays2 | 0 | 0.075 | | |
| Darlington Leak | 0 | Taken as included in spill | | |
| Darlington Spillway | Yes | From DT in Maintenance manual | | |
| Elandsdrift Spillway (assumed) | Yes | Assumed | | |

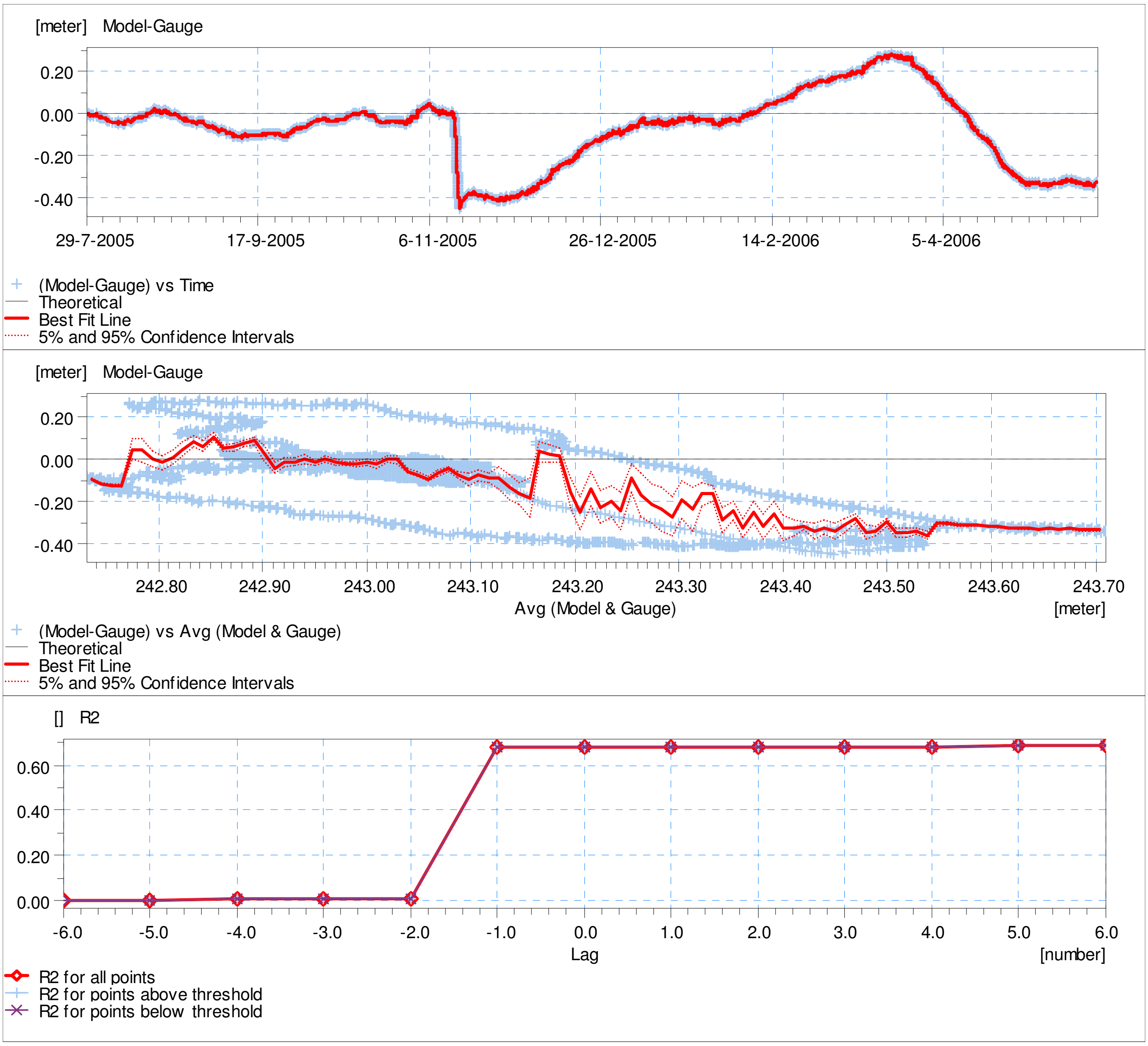
| Gauge Factors: | |
|---|---|
| Teebus | 1 |
| Grassridge Dam | 0.9 |
| Elandsdrift River | 1 |
| Elandsdrift Canal | 1.12 |
| De Mistkraal River | 1 |
| De Mistkraal Canal | 0.95 |
| Darlington Dam | 1 |

| Tributary Factors: | | Base Flow: |
|---|---|---|
| Klein Brak | 1 | 0.01 |
| Groot Vis (new DT) | 1.7 | 0.01 |
| Pauls | 1.25 | 0.01 |
| Tarka | 1.1 | 0.01 |
| Baviaans | 1.1 | 0.01 |
| Little Fish | 2.8 | 0.01 |
| Sundays | 2.5 | 0.01 |

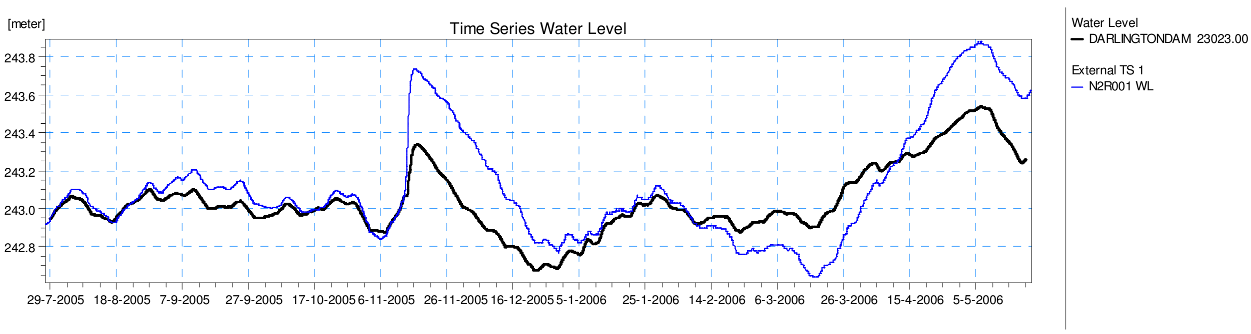
| Rainfall factors: | |
|---|---|
| Darlington Dam | 1 |
| Grassridge Dam | 1 |
| Elandsdrift Dam | 1 |
| De Mistkraal Dam | 1 |

# Calibration Results for Darlington Dam Water Level

DARLINGTONDAM 23023.00



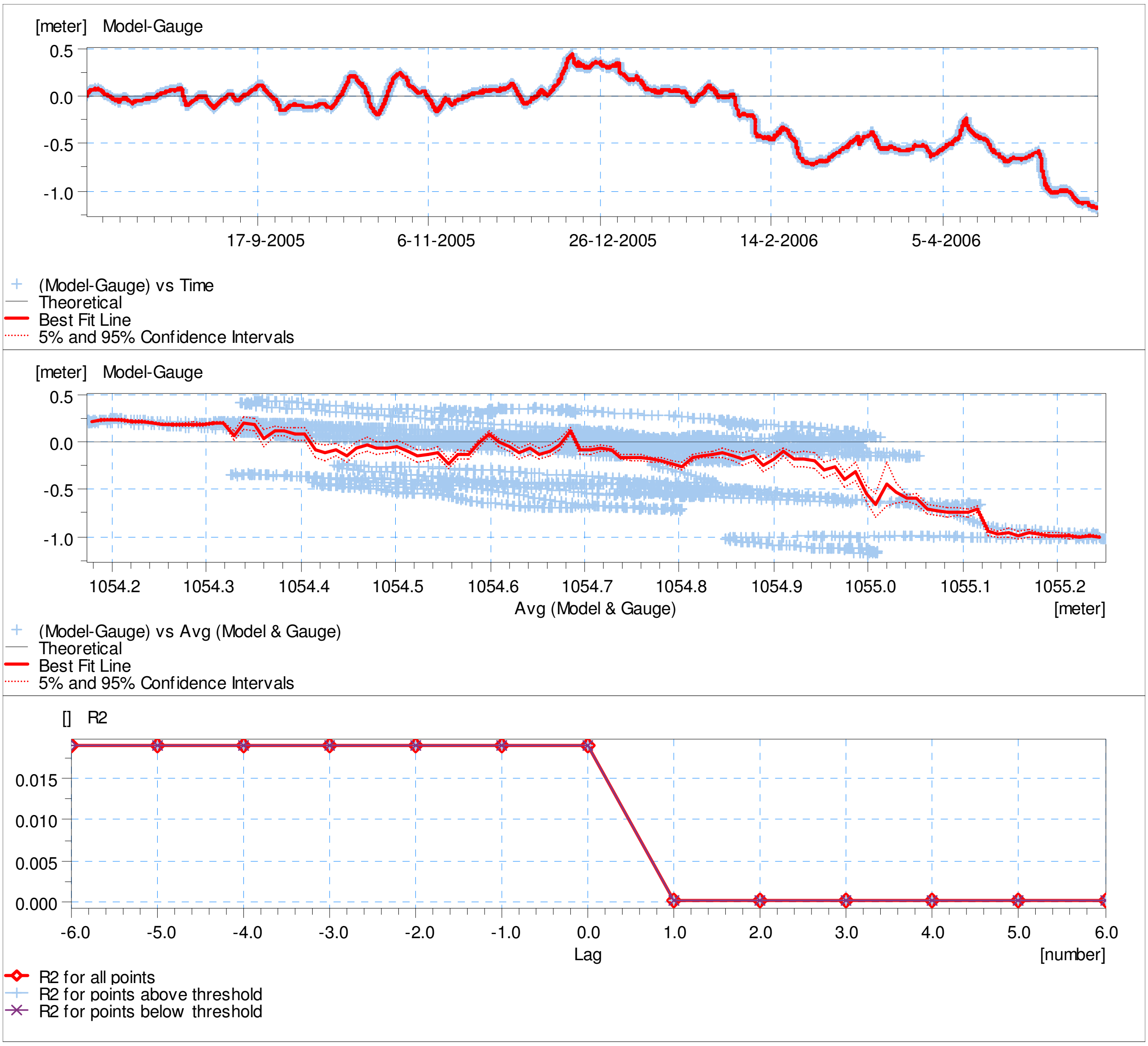

R2 = 0.682



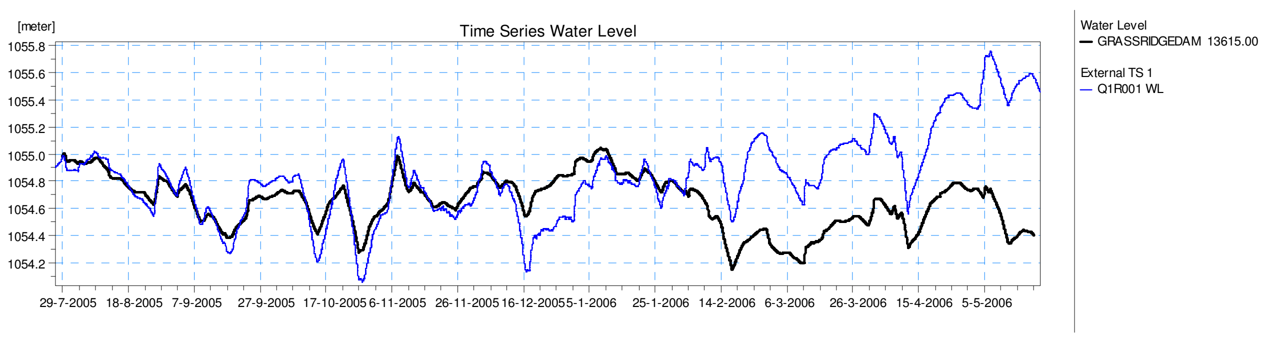
| N2R001 WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.682 | |
| Max. positive difference | 0.28 | meter |
| Max. negative difference | -0.449 | meter |
| Volume observed | 6193899602 | |
| Volume modelled | 6192123542 | |
| Volume error | -0.029 | % |
| Peak observed value | 243.88 | meter |
| Peak modelled value | 243.538 | meter |
| Peak error | -0.14 | % |

# Calibration Results for Grassridge Dam Water Level

GRASSRIDGEDAM 13615.00



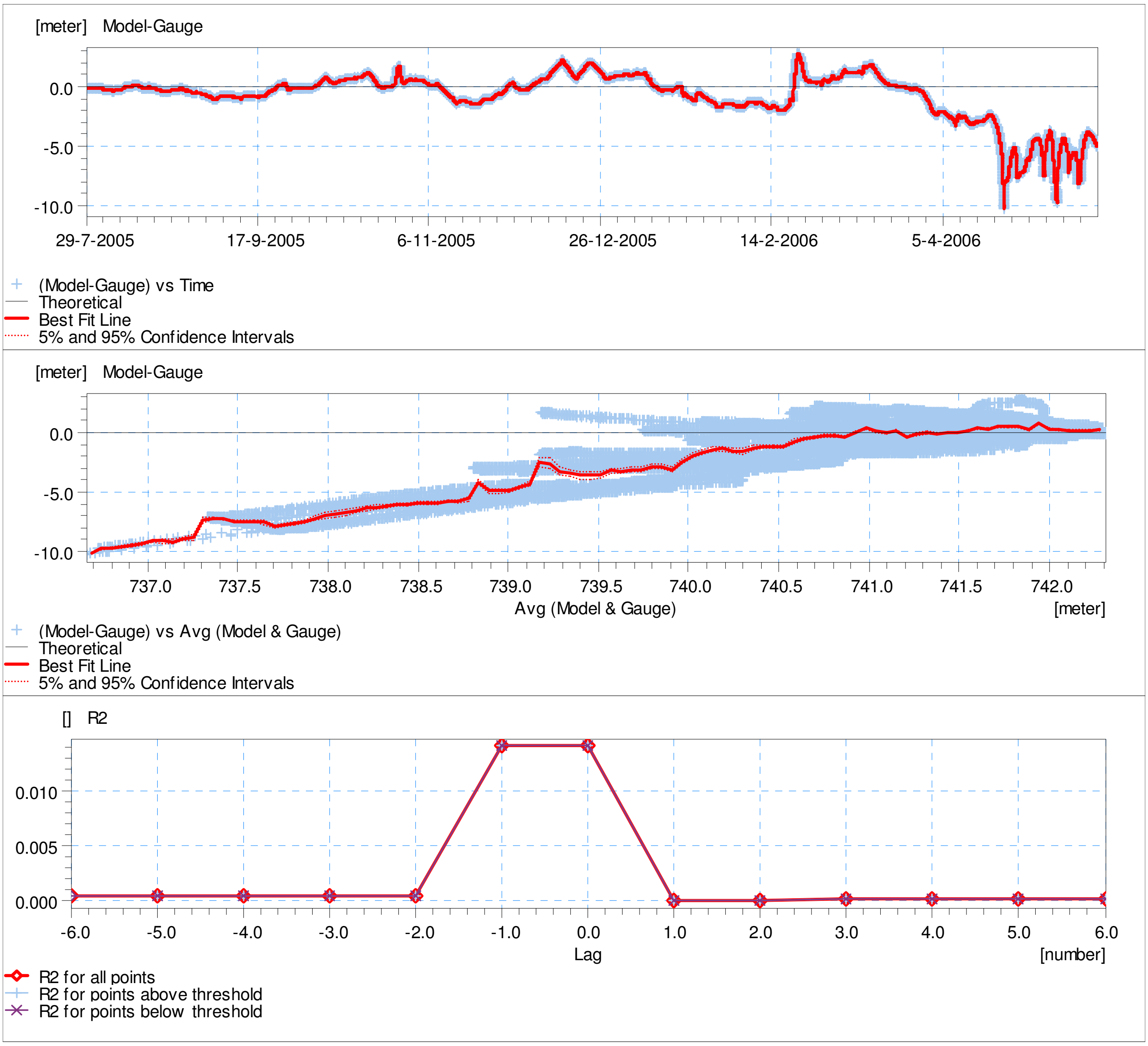

R2 = 0.019



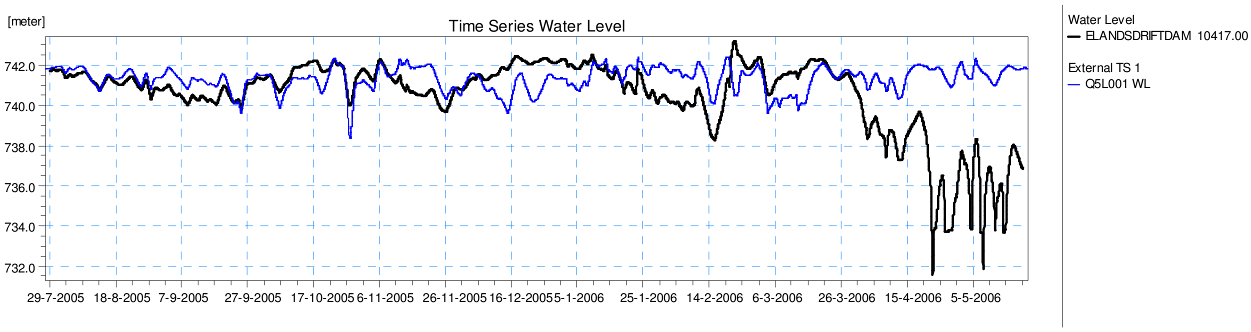
| Q1R001 WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.019 | |
| Max. positive difference | 0.438 | meter |
| Max. negative difference | -1.18 | meter |
| Volume observed | 26884821780 | |
| Volume modelled | 26880076475 | |
| Volume error | -0.018 | % |
| Peak observed value | 1055.76 | meter |
| Peak modelled value | 1055.043 | meter |
| Peak error | -0.068 | % |

# Calibration Results for Elandsdrift Dam Water Level

ELANDSDRIFTDAM 10417.00



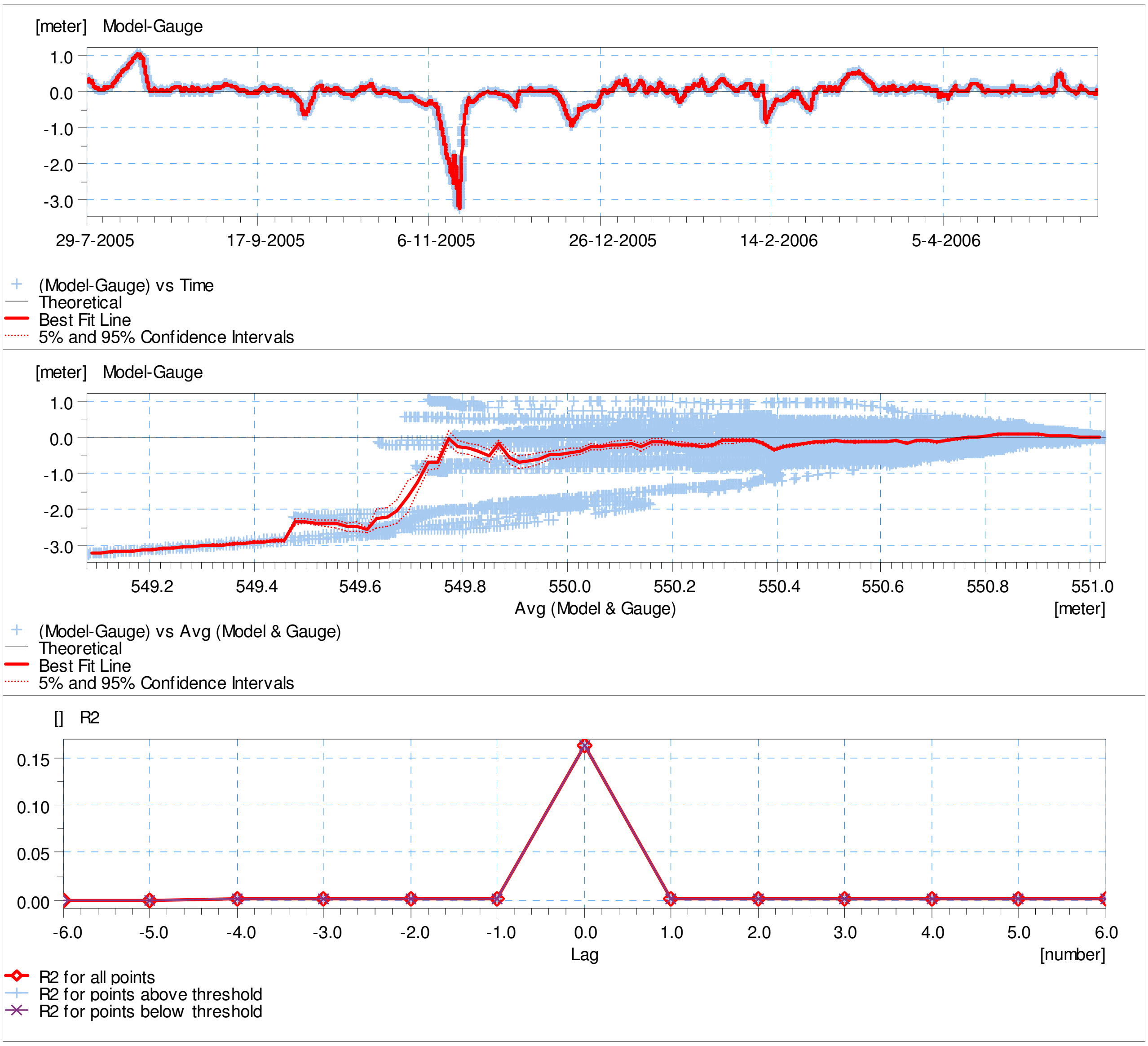

R2 = 0.014



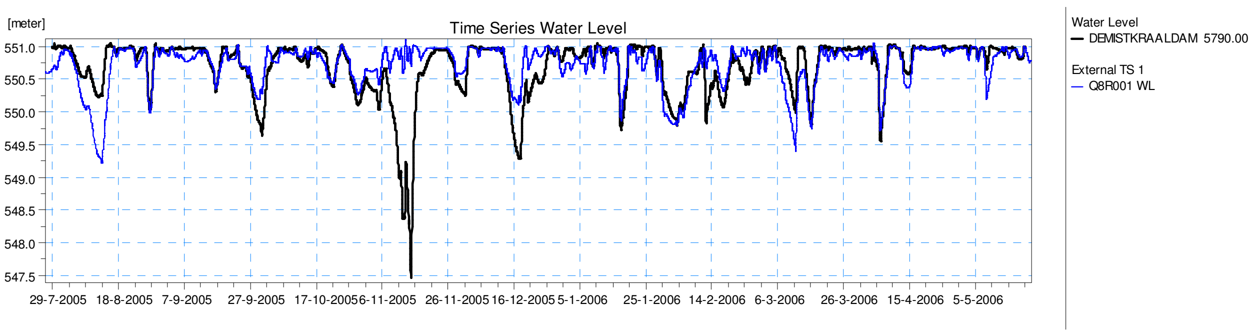
| Q5R001 WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.014 | |
| Max. positive difference | 2.689 | meter |
| Max. negative difference | -10.199 | meter |
| Volume observed | 18894533126 | |
| Volume modelled | 18874465176 | |
| Volume error | -0.106 | % |
| Peak observed value | 742.41 | meter |
| Peak modelled value | 743.199 | meter |
| Peak error | 0.106 | % |

# Calibration Results for De Mistkraal Dam Water Level

DEMISTKRAALDAM 5790.00



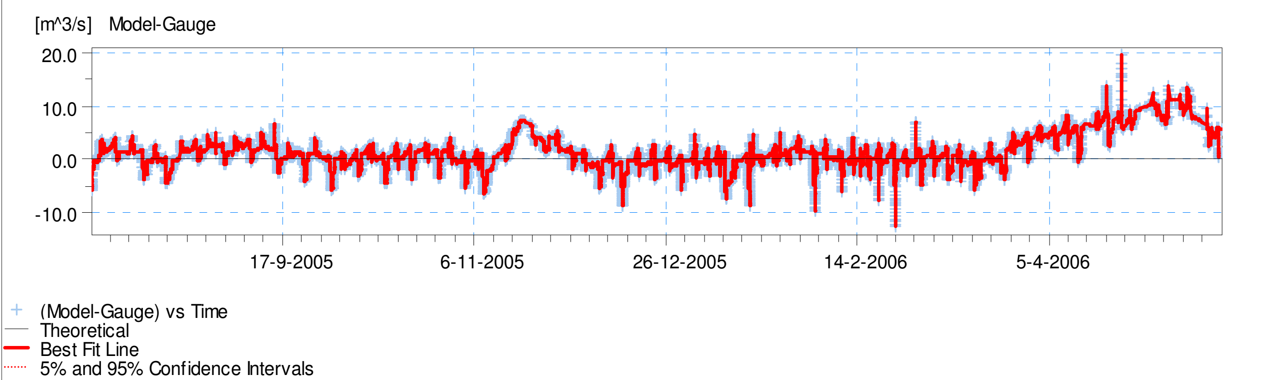

R2 = 0.162

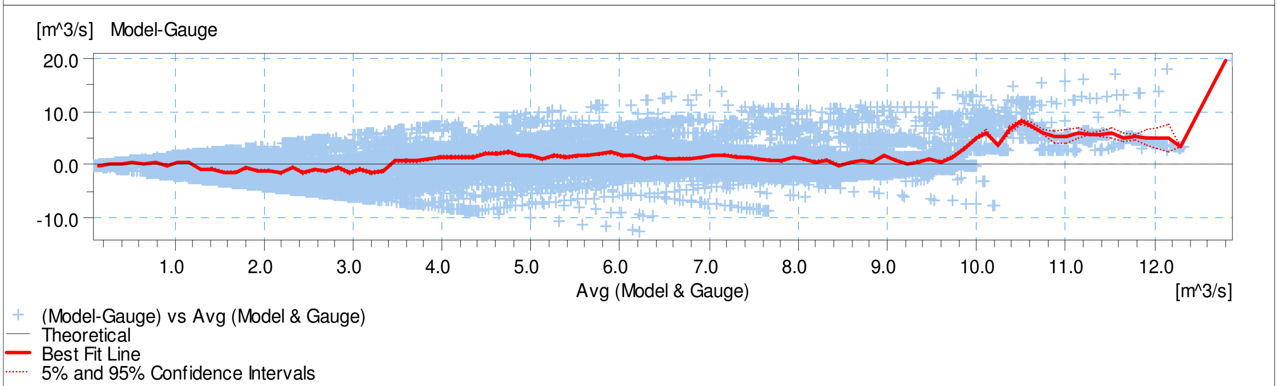


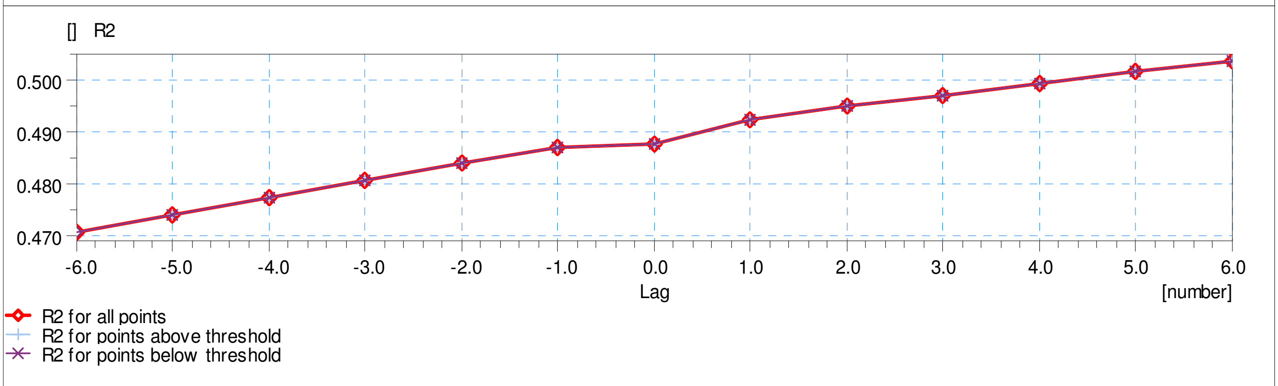
| Q8R001 WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.162 | |
| Max. positive difference | 1.029 | meter |
| Max. negative difference | -3.244 | meter |
| Volume observed | 14036846911 | |
| Volume modelled | 14036002738 | |
| Volume error | -0.006 | % |
| Peak observed value | 551.114 | meter |
| Peak modelled value | 551.048 | meter |
| Peak error | -0.012 | % |

# Calibration Results for Flow Rate at Korhaansdrift Weir

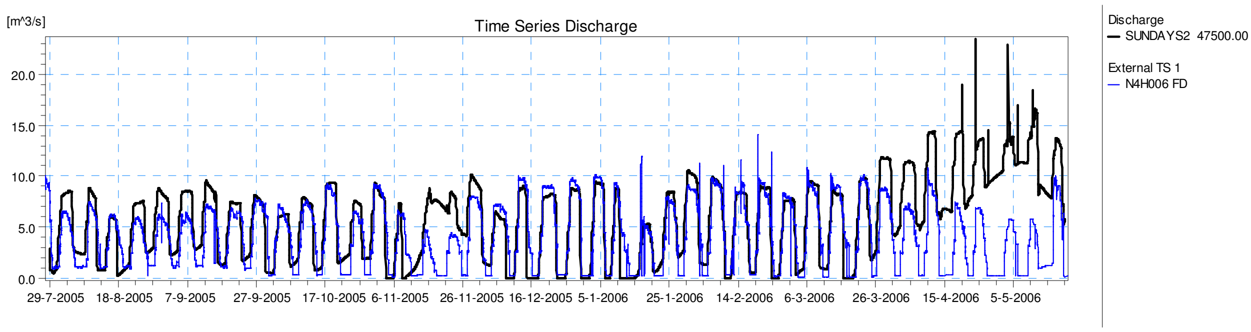
SUNDAYS2 47500.00



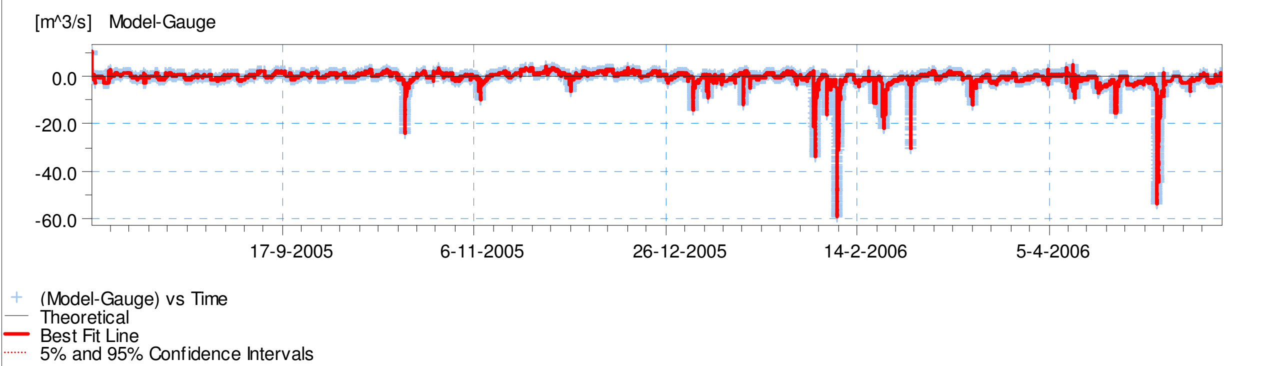

R2 = 0.488

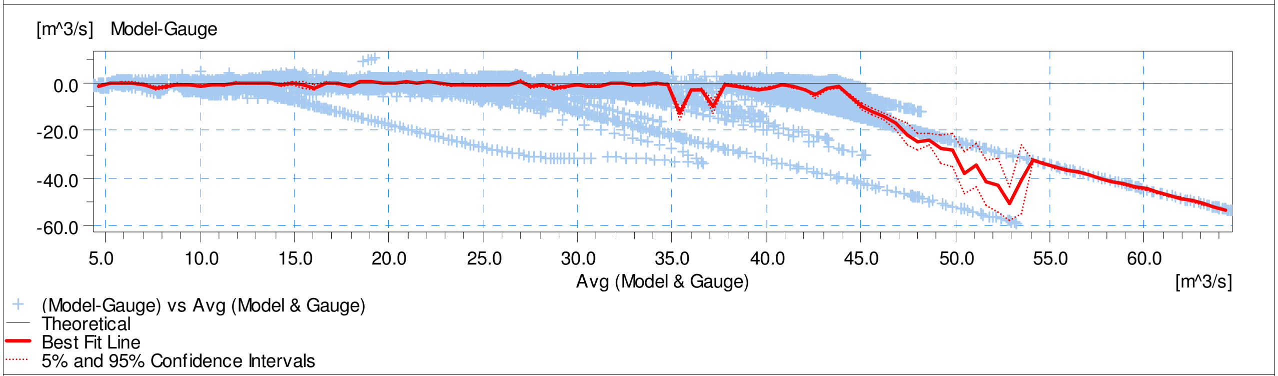


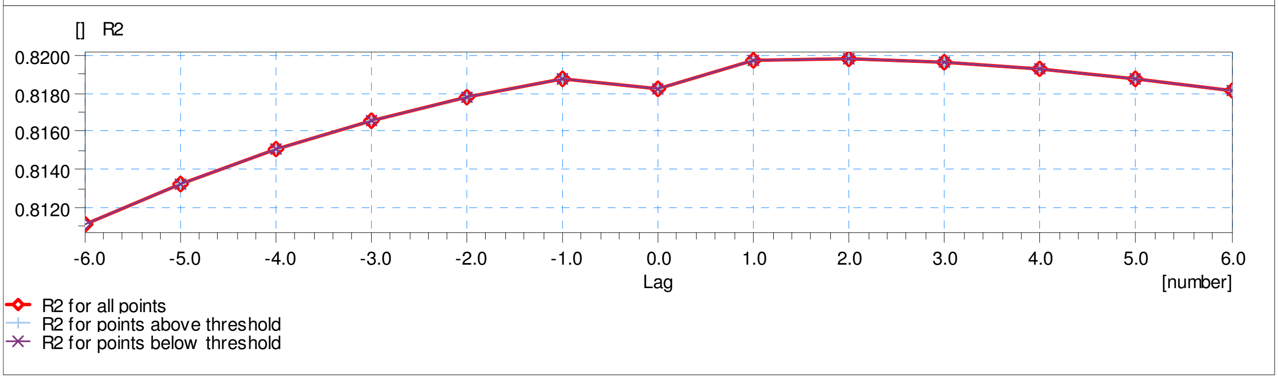
| N4H006 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.488 | |
| Max. positive difference | 19.467 | $m^3/s$ |
| Max. negative difference | -12.362 | $m^3/s$ |
| Volume observed | 104158564.7 | $m^3$ |
| Volume modelled | 147272678.4 | $m^3$ |
| Volume error | 41.393 | % |
| Peak observed value | 14.095 | $m^3/s$ |
| Peak modelled value | 22.597 | $m^3/s$ |
| Peak error | 60.318 | % |

# Calibration of Flow Rate at Teebus at Jan Blaauwskop

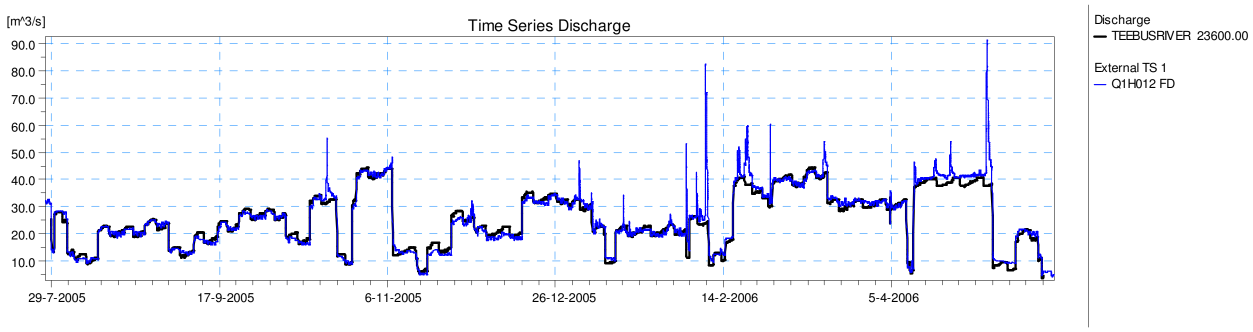
TEEBUSRIVER 23600.00



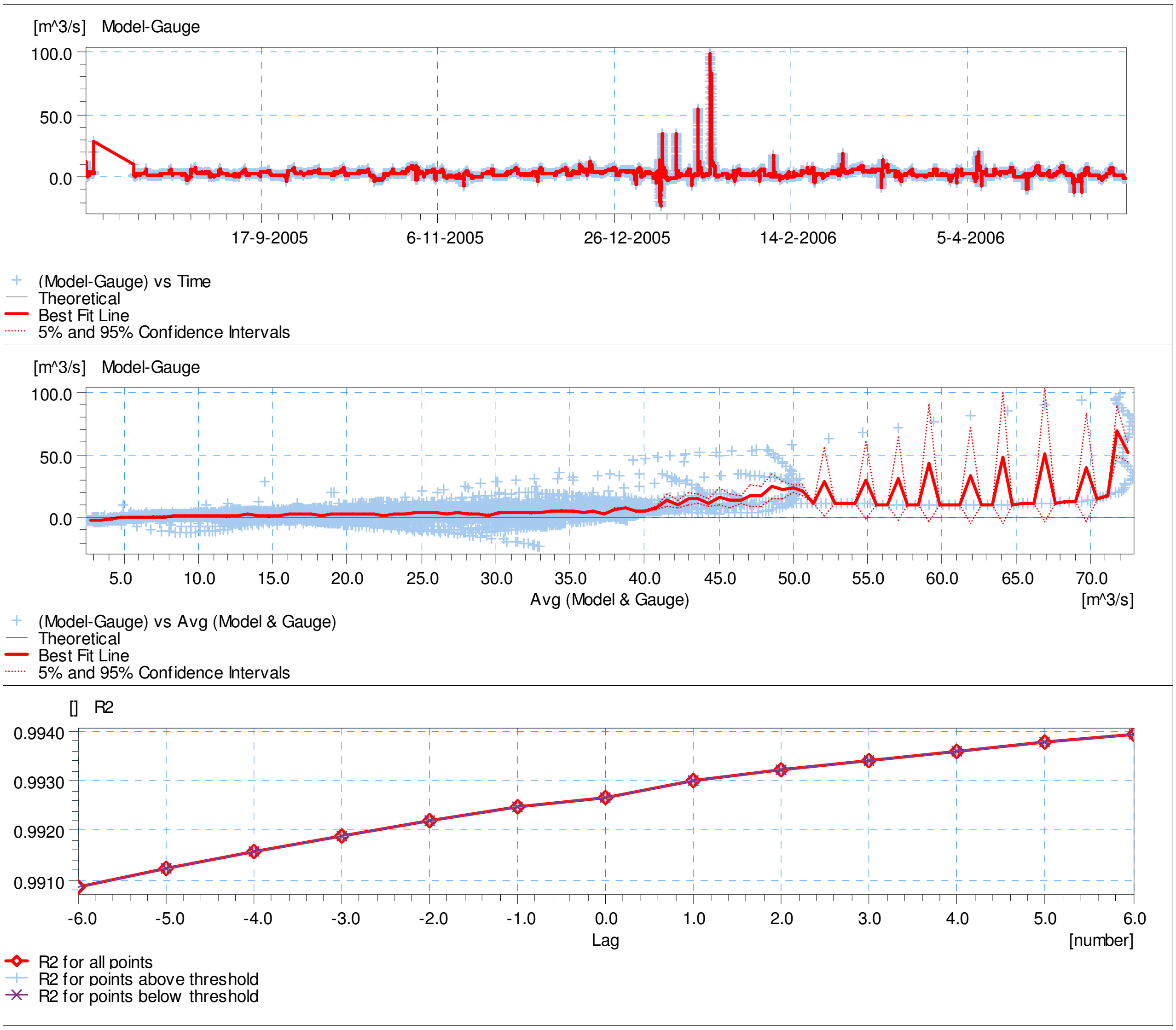

R2 = 0.818



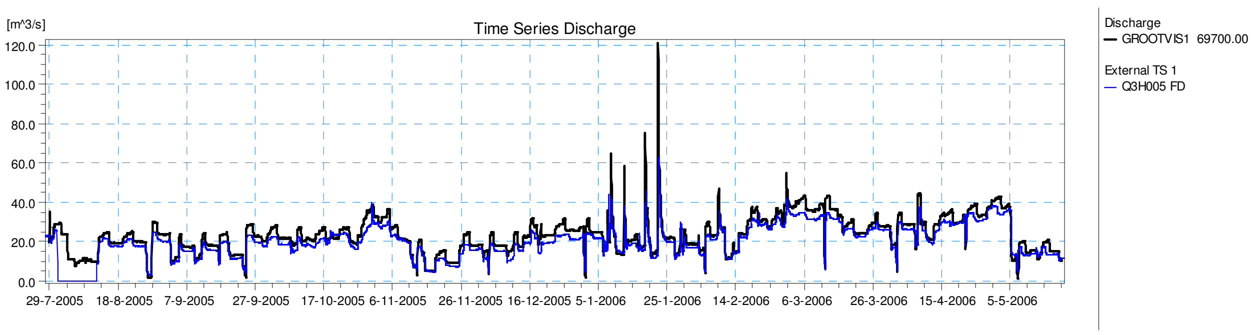
| Q1H012 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.818 | |
| Max. positive difference | 10.315 | $m^3/s$ |
| Max. negative difference | -58.823 | $m^3/s$ |
| Volume observed | 662662292.8 | $m^3$ |
| Volume modelled | 648795404.4 | $m^3$ |
| Volume error | -2.093 | % |
| Peak observed value | 91.522 | $m^3/s$ |
| Peak modelled value | 44.328 | $m^3/s$ |
| Peak error | -51.565 | % |

# Calibration of Flow Rate at Waaikraal Weir

GROOTVIS1 69700.00



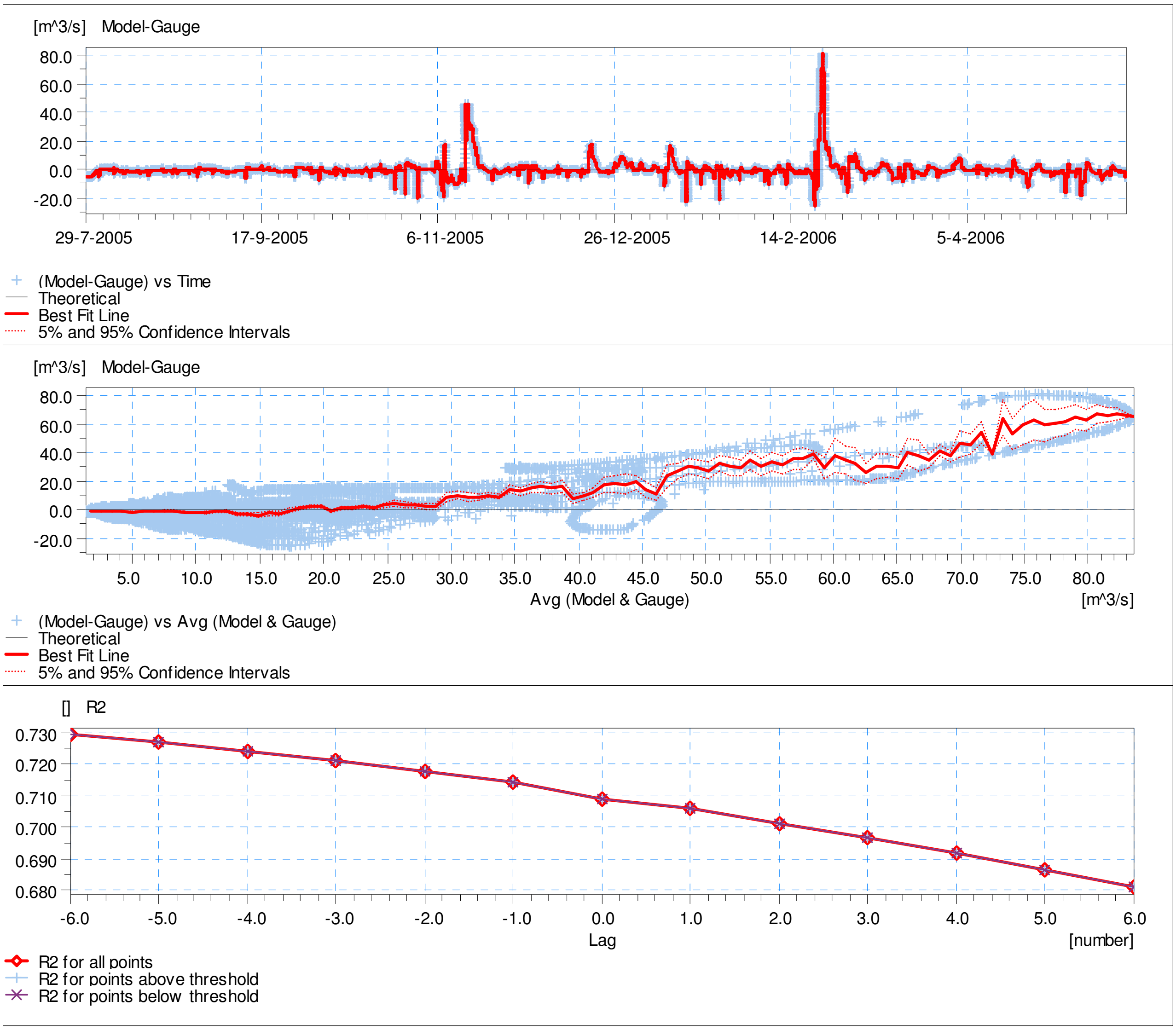

R2 = 0.993



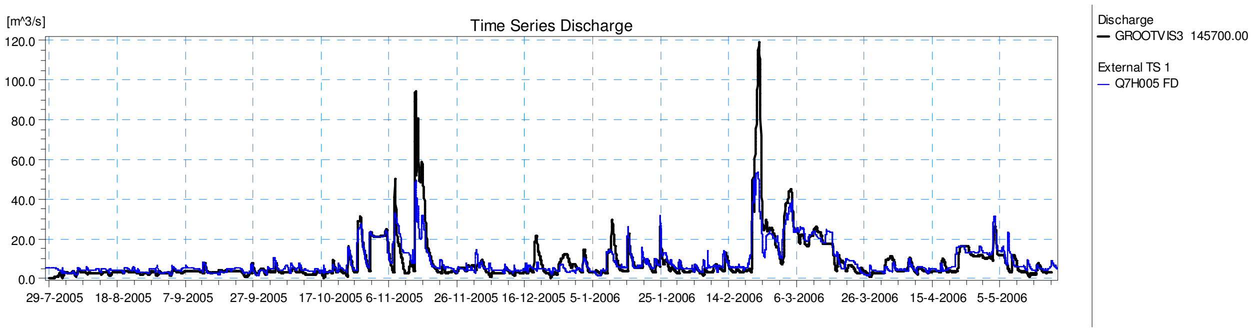
| Q3H005 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.993 | |
| Max. positive difference | 98.14 | $m^3/s$ |
| Max. negative difference | -22.566 | $m^3/s$ |
| Volume observed | 511746177.6 | $m^3$ |
| Volume modelled | 602984798.2 | $m^3$ |
| Volume error | 17.829 | % |
| Peak observed value | 63.1 | $m^3/s$ |
| Peak modelled value | 121.044 | $m^3/s$ |
| Peak error | 91.829 | % |

# Calibration of Flow Rate at Sheldon Weir

GROOTVIS3 145700.00



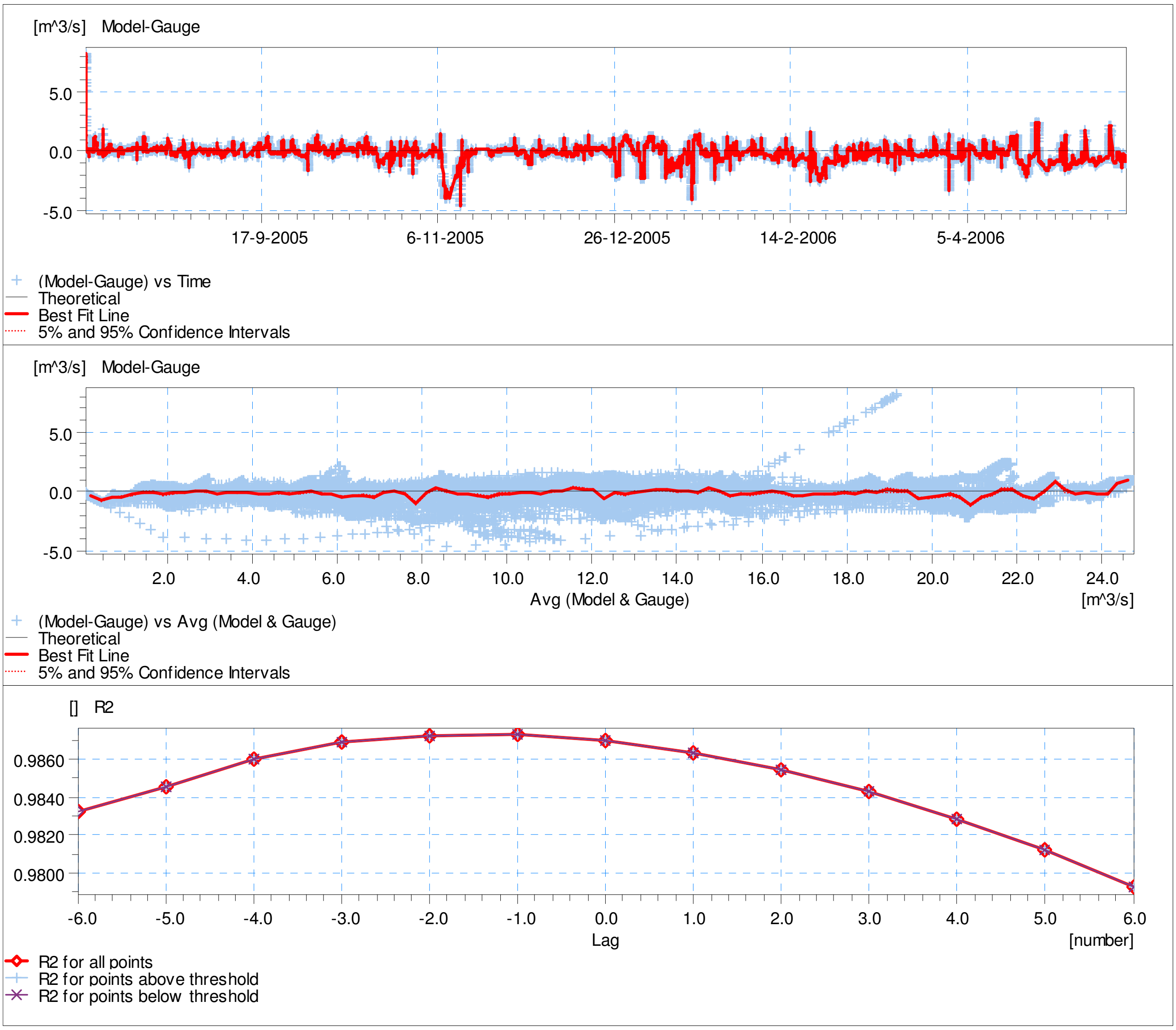

R2 = 0.709



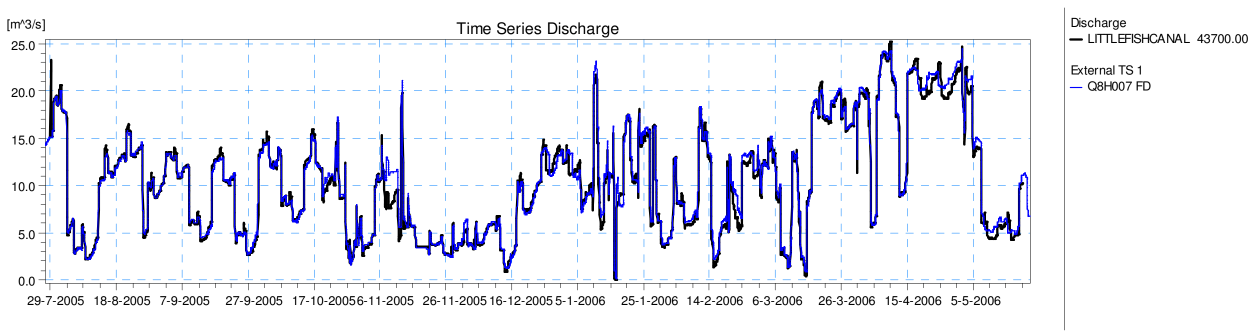
| Q7H005 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.709 | |
| Max. positive difference | 80.569 | $m^3/s$ |
| Max. negative difference | -25.181 | $m^3/s$ |
| Volume observed | 213991296 | $m^3$ |
| Volume modelled | 207665533.9 | $m^3$ |
| Volume error | -2.956 | % |
| Peak observed value | 53.569 | $m^3/s$ |
| Peak modelled value | 119.258 | $m^3/s$ |
| Peak error | 122.625 | % |

# Calibration of Flow Rate at Parshall 2 on the Little Fish Canal

LITTLEFISHCANAL 43700.00



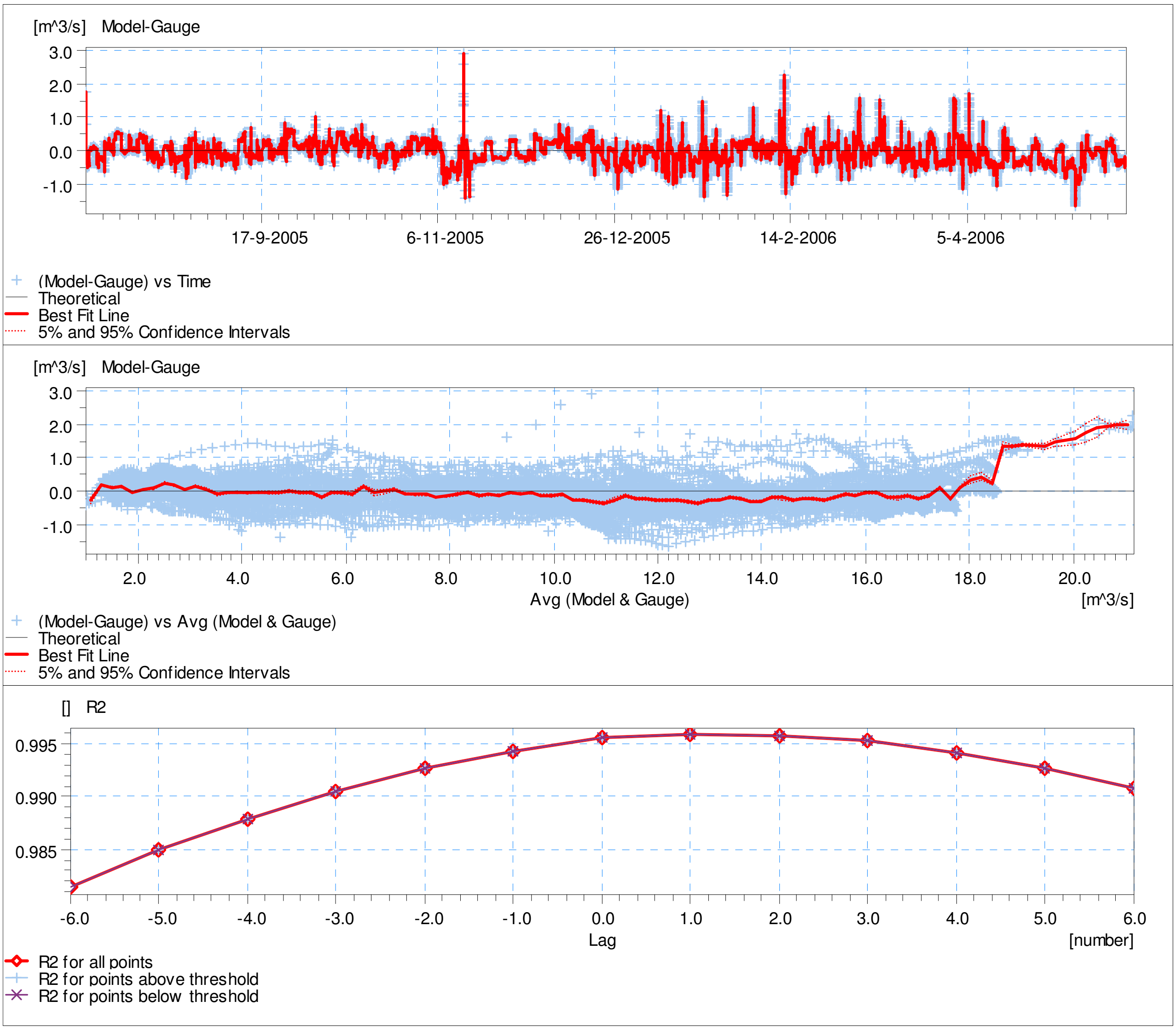

R2 = 0.987



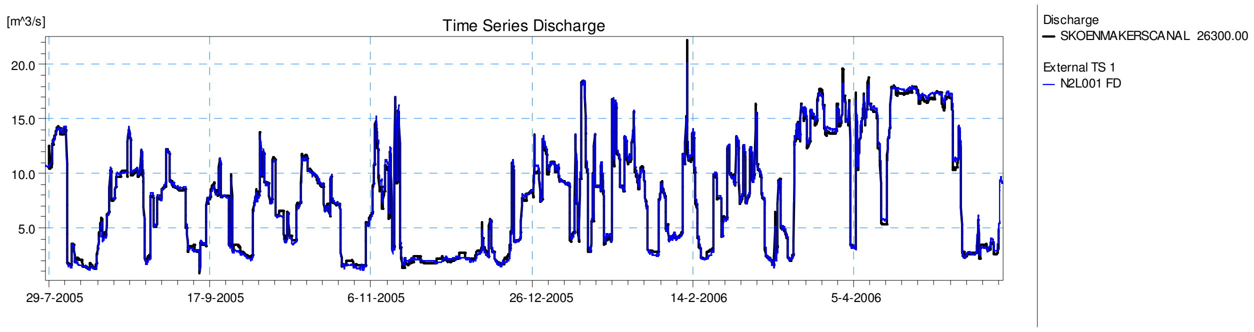
| Q8H007 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.987 | |
| Max. positive difference | 8.144 | $m^3/s$ |
| Max. negative difference | -4.562 | $m^3/s$ |
| Volume observed | 267387847.3 | $m^3$ |
| Volume modelled | 262488679.2 | $m^3$ |
| Volume error | -1.832 | % |
| Peak observed value | 24.469 | $m^3/s$ |
| Peak modelled value | 25.227 | $m^3/s$ |
| Peak error | 3.099 | % |

# Calibration of Flow Rate at Parshall 3 on the Skoenmakers Canal

SKOENMAKERSCANAL 26300.00



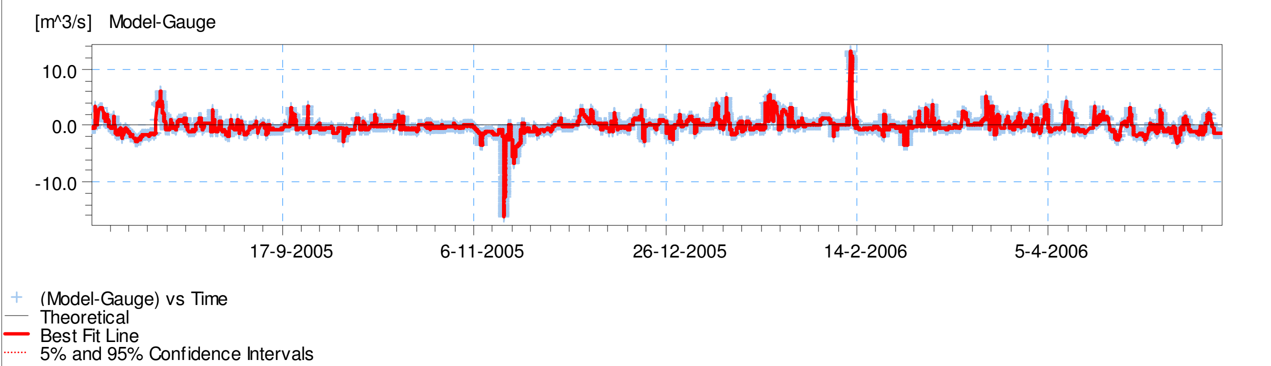

R2 = 0.996

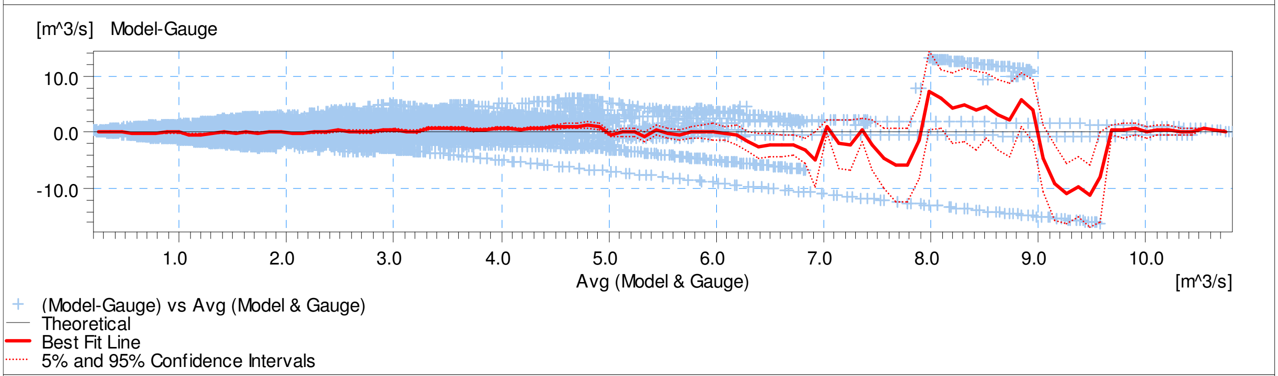


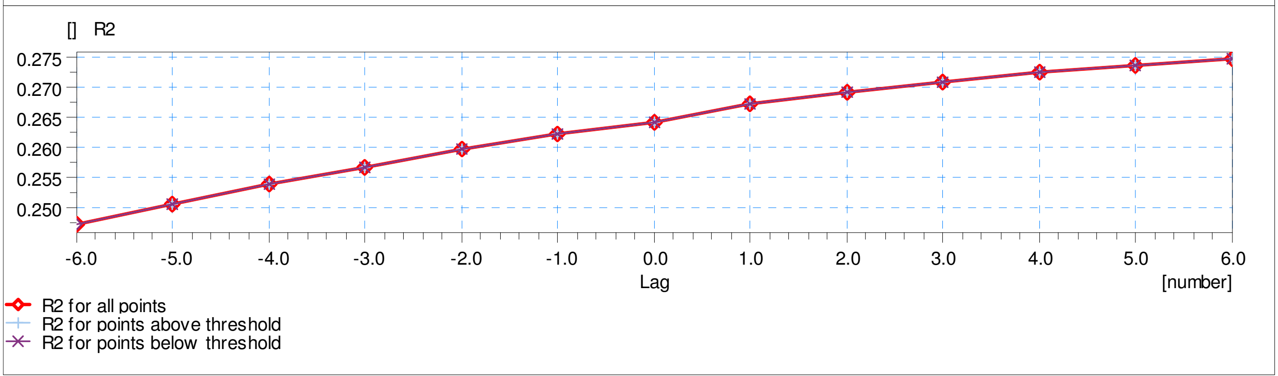
| N2L001 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.996 | |
| Max. positive difference | 2.874 | $m^3/s$ |
| Max. negative difference | -1.608 | $m^3/s$ |
| Volume observed | 200514374.7 | $m^3$ |
| Volume modelled | 199469227.1 | $m^3$ |
| Volume error | -0.521 | % |
| Peak observed value | 20.168 | $m^3/s$ |
| Peak modelled value | 22.255 | $m^3/s$ |
| Peak error | 10.349 | % |

# Calibration of Flow Rate at Junctiondrift Weir

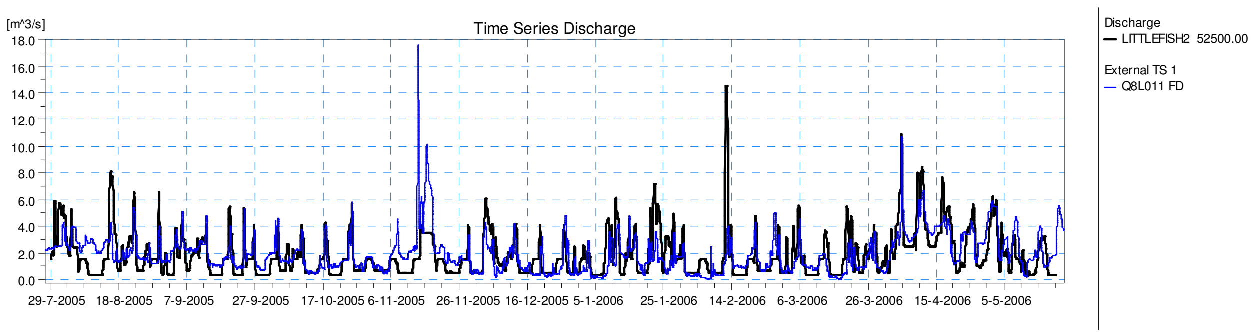
LITTLEFISH2 52500.00



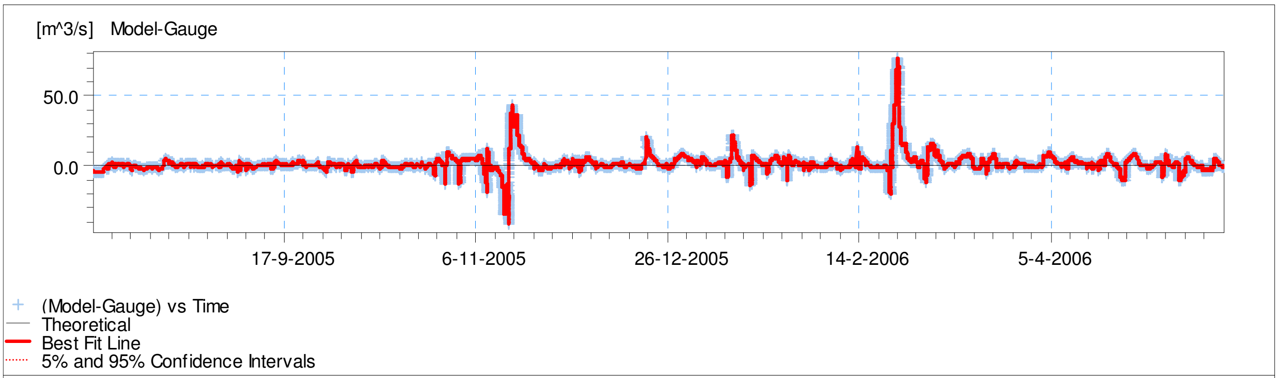

$R2 = 0.264$

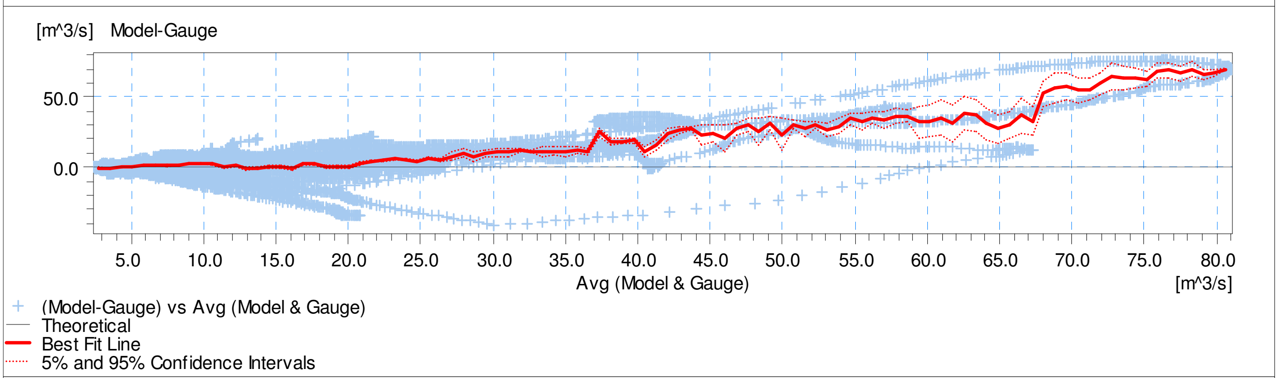


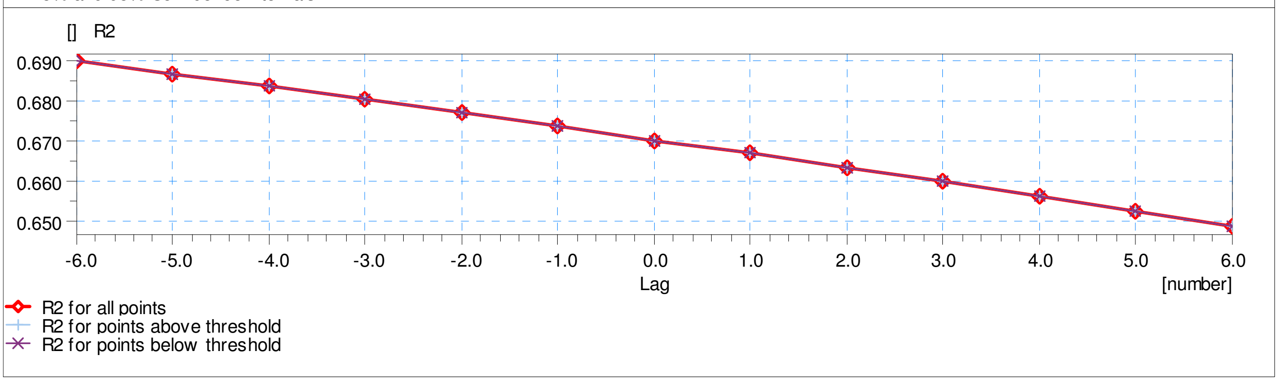
| Q8L011 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.264 | |
| Max. positive difference | 13.096 | $m^3/s$ |
| Max. negative difference | -16.072 | $m^3/s$ |
| Volume observed | 48806441.41 | $m^3$ |
| Volume modelled | 47479575.93 | $m^3$ |
| Volume error | -2.719 | % |
| Peak observed value | 17.611 | $m^3/s$ |
| Peak modelled value | 14.54 | $m^3/s$ |
| Peak error | -17.439 | % |

# Calibration of Flow Rate at Piggott’s Bridge Weir

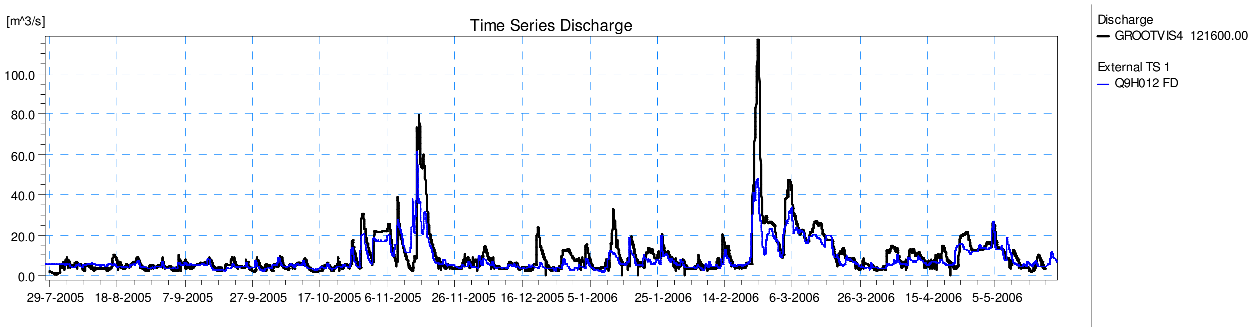
GROOTVIS4 121600.00




R2 = 0.670



| Q9H012 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.67 | |
| Max. positive difference | 76.337 | $m^3/s$ |
| Max. negative difference | -41.188 | $m^3/s$ |
| Volume observed | 208478242.2 | $m^3$ |
| Volume modelled | 253430349.8 | $m^3$ |
| Volume error | 21.562 | % |
| Peak observed value | 61.587 | $m^3/s$ |
| Peak modelled value | 116.877 | $m^3/s$ |
| Peak error | 89.775 | % |

# Appendix E: Full Verification Results

The final full verification results are presented for each of the calibration locations in this Appendix.

**<u>3rd Verification Run - OFS-RT Model from 5th May to 29th October 2006.</u>**

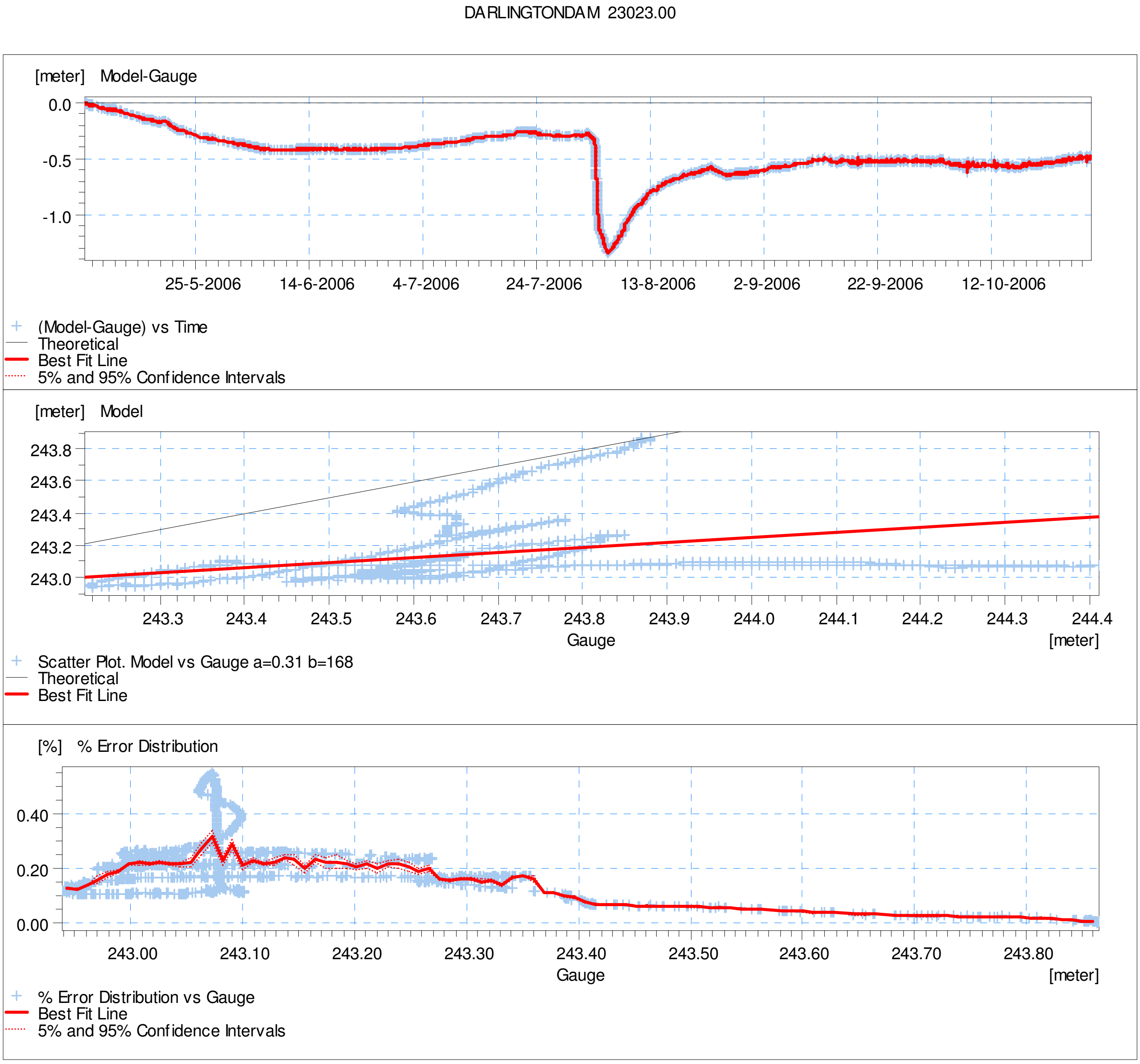
| Reach Name: | Return Flow Values (m3/s): | Manning's Number: | Next Values: | Next n: |
|---|---|---|---|---|
| Teebus | 0.2 | 0.037 | | |
| Groot Brak1 | 0.7 | 0.040 | | |
| Groot Brak 2 | 0.70 | 0.040 | | |
| Groot Vis 1 | 1.0 | 0.033 | | |
| Groot Vis 2 | 2 | 0.045 | | |
| Groot Vis 3 | 1 | 0.070 | | |
| Groot Vis 4 | 0 | 0.050 | | |
| Little Fish 1 | 0.25 | 0.055 | | |
| Little Fish Canal | 0 | 0.020 | | |
| Little Fish 2 | 0.5 | 0.055 | | |
| Skoenmakers Canal | 0 | 0.025 | | |
| Skoenmakers | -0.5 | 0.055 | | |
| Sundays2 | 0 | 0.075 | | |
| Darlington Leak | 0 | Taken as included in spill | | |
| Darlington Spillway | Yes | From DT in Maintenance manual | | |
| Elandsdrift Spillway (assumed) | Yes | Assumed | | |

| Gauge Factors: | |
|---|---|
| Teebus | 1 |
| Grassridge Dam | 0.9 |
| Elandsdrift River | 1 |
| Elandsdrift Canal | 1.12 |
| De Mistkraal River | 1 |
| De Mistkraal Canal | 0.95 |
| Darlington Dam | 1 |

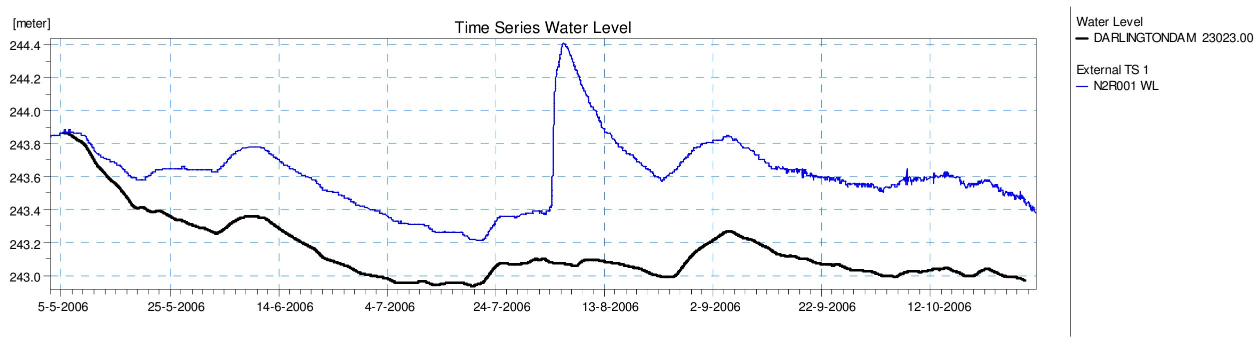
| Tributary Factors: | | Base Flow: |
|---|---|---|
| Klein Brak | 1 | 0.01 |
| Groot Vis (new DT) | 1.7 | 0.01 |
| Pauls | 1.25 | 0.01 |
| Tarka | 1.1 | 0.01 |
| Baviaans | 1.1 | 0.01 |
| Little Fish | 2.8 | 0.01 |
| Sundays | 2.5 | 0.01 |

| Rainfall factors: | |
|---|---|
| Darlington Dam | 1 |
| Grassridge Dam | 1 |
| Elandsdrift Dam | 1 |
| De Mistkraal Dam | 1 |

# Verification Results for Darlington Dam Water Level



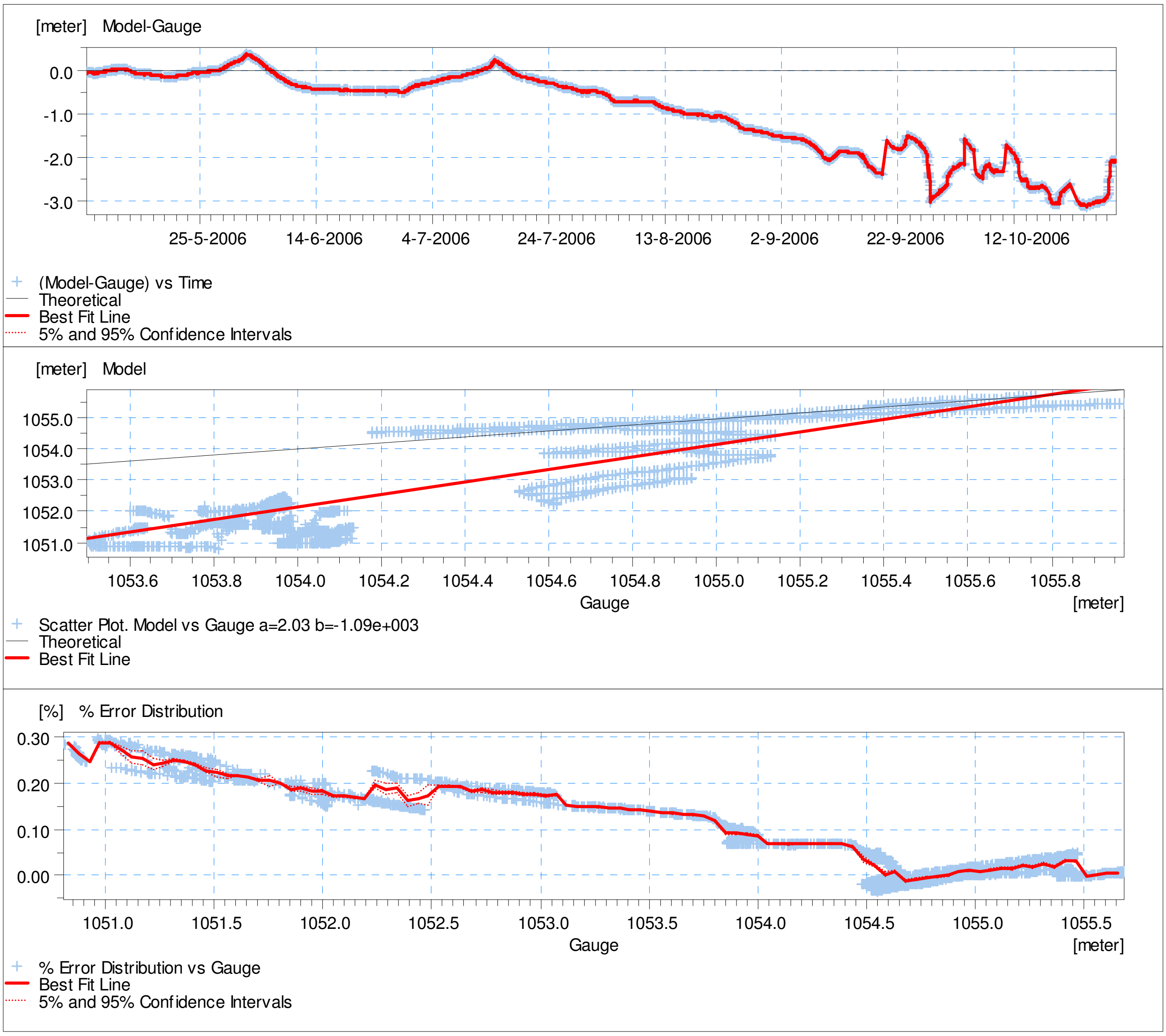

R2 = 0.142



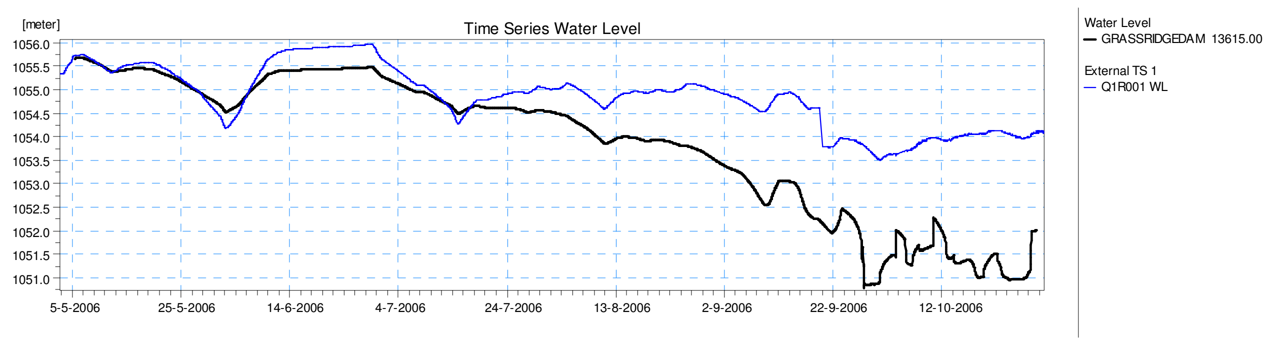
| N2R001 WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.142 | |
| Max. positive difference | -0.006 | meter |
| Max. negative difference | -1.337 | meter |
| Volume observed | 3722321318 | |
| Volume modelled | 3715253426 | |
| Volume error | -0.19 | % |
| Peak observed value | 244.41 | meter |
| Peak modelled value | 243.864 | meter |
| Peak error | -0.223 | % |

# Verification Results for Grassridge Dam Water Level

GRASSRIDGEDAM 13615.00



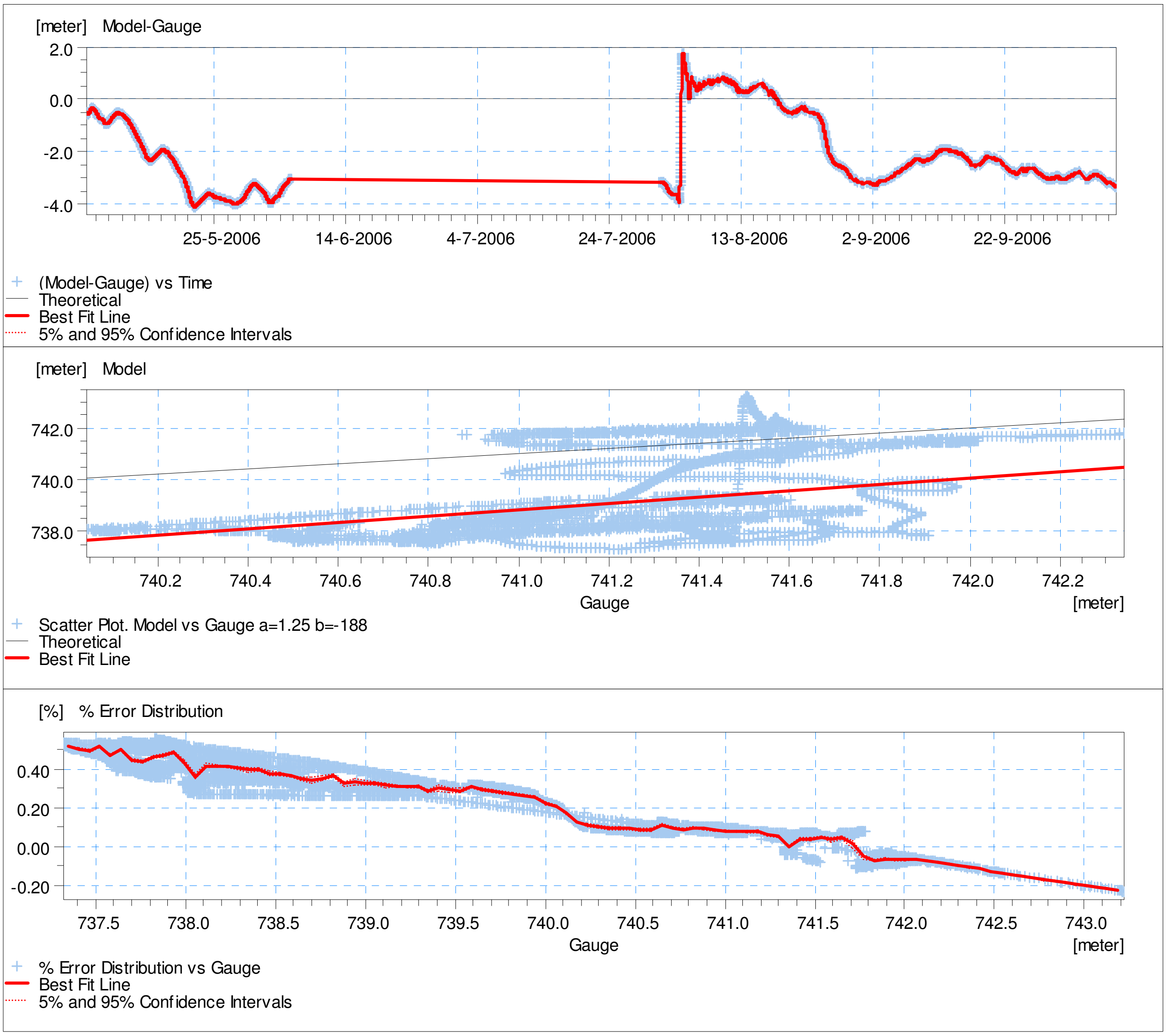

R2 = 0.747



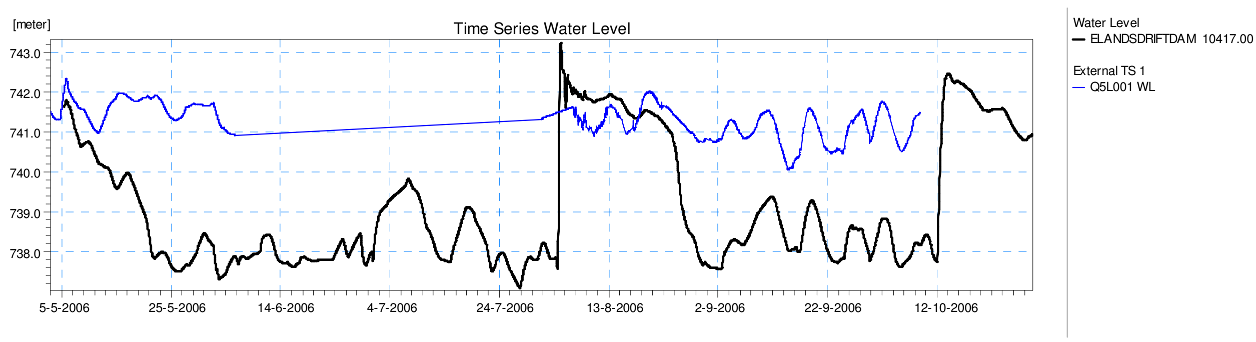
| Q1R001 WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.747 | |
| Max. positive difference | 0.36 | meter |
| Max. negative difference | -3.113 | meter |
| Volume observed | 16124868058 | |
| Volume modelled | 16109691950 | |
| Volume error | -0.094 | % |
| Peak observed value | 1055.97 | meter |
| Peak modelled value | 1055.685 | meter |
| Peak error | -0.027 | % |

# Verification Results for Elandsdrift Dam Water Level

ELANDSDRIFTDAM 10417.00



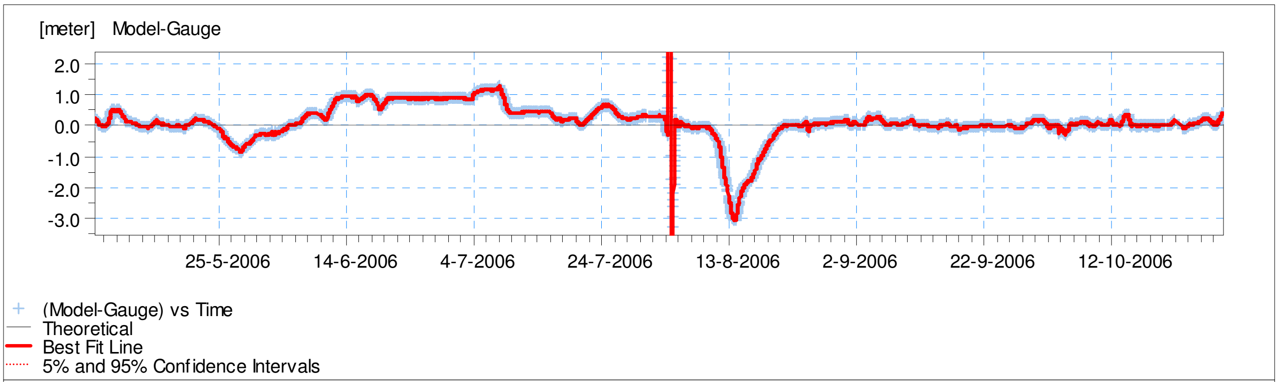

R2 = 0.114

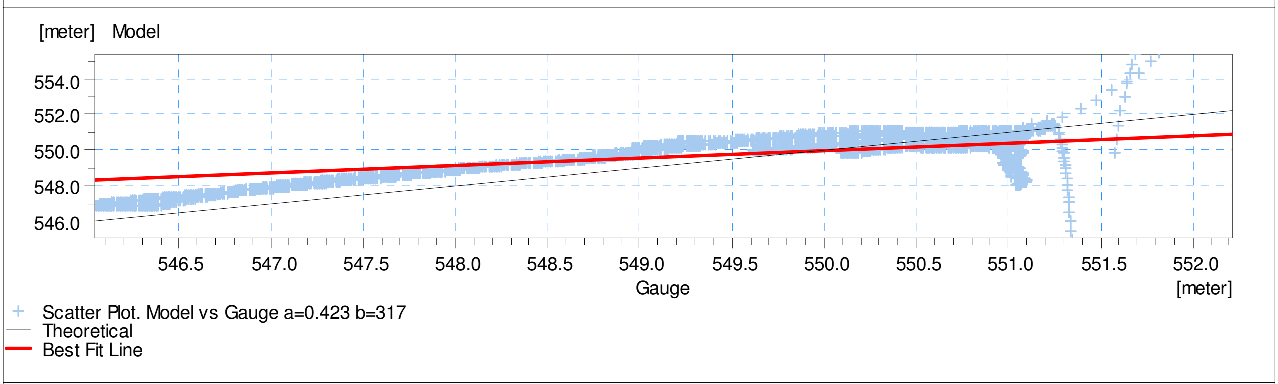


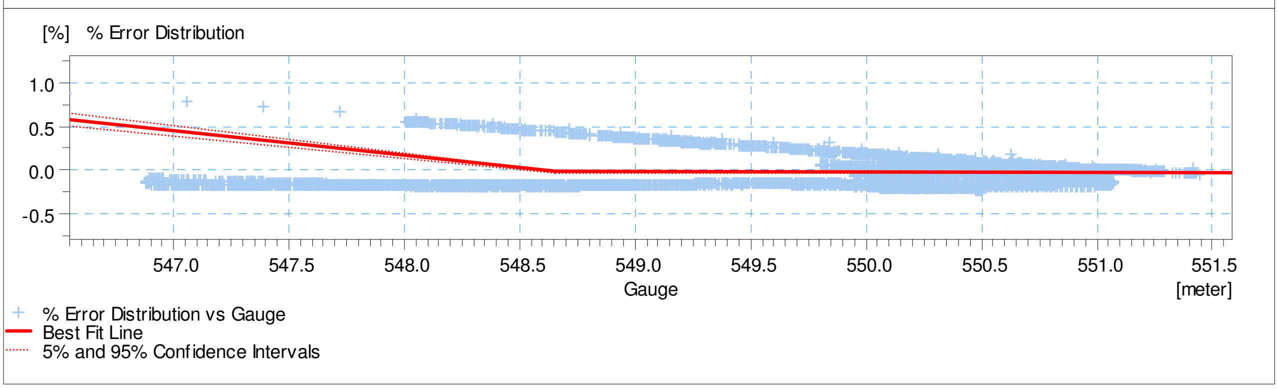
| Q5R001 WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.114 | |
| Max. positive difference | 1.712 | meter |
| Max. negative difference | -4.074 | meter |
| Volume observed | 10019200267 | |
| Volume modelled | 9986555300 | |
| Volume error | -0.326 | % |
| Peak observed value | 742.34 | meter |
| Peak modelled value | 743.217 | meter |
| Peak error | 0.118 | % |

# Verification Results for De Mistkraal Dam Water Level

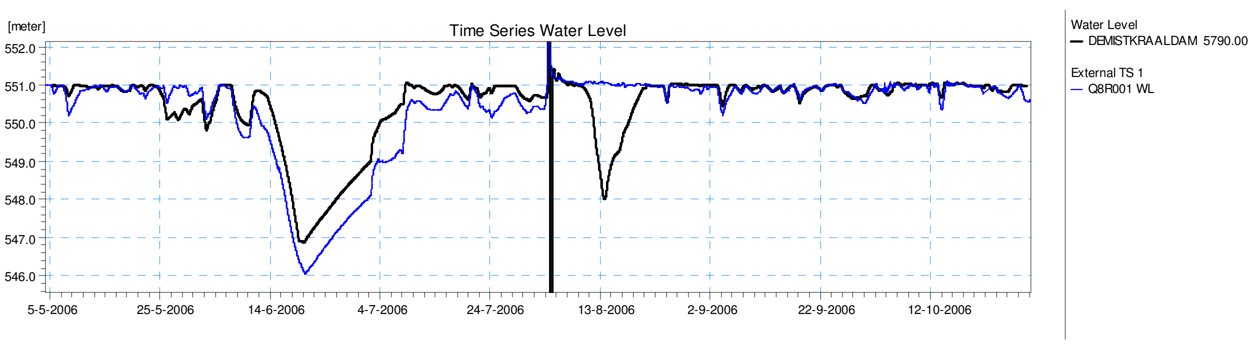
DEMISTKRAALDAM 5790.00



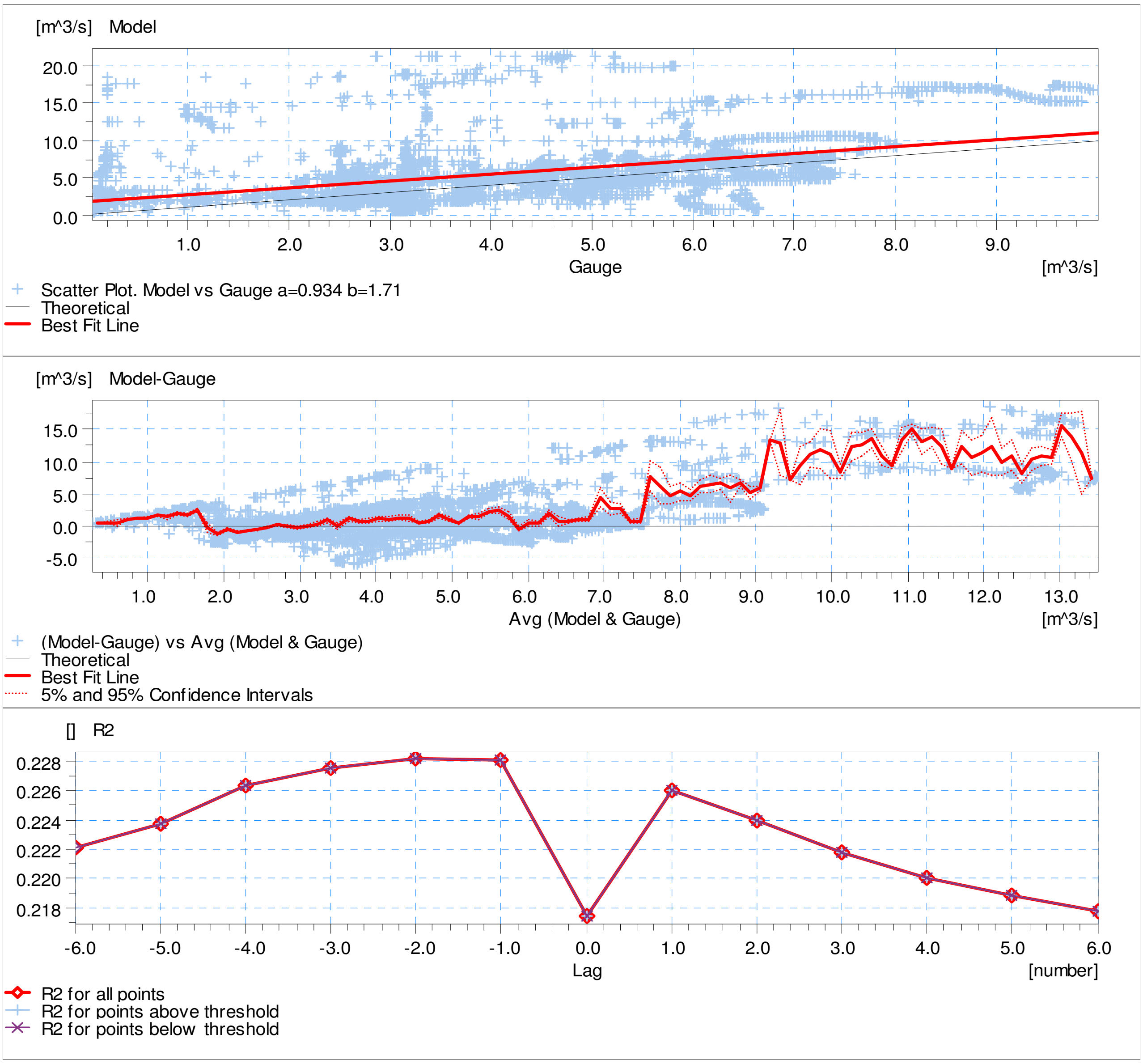

R2 = 0.003



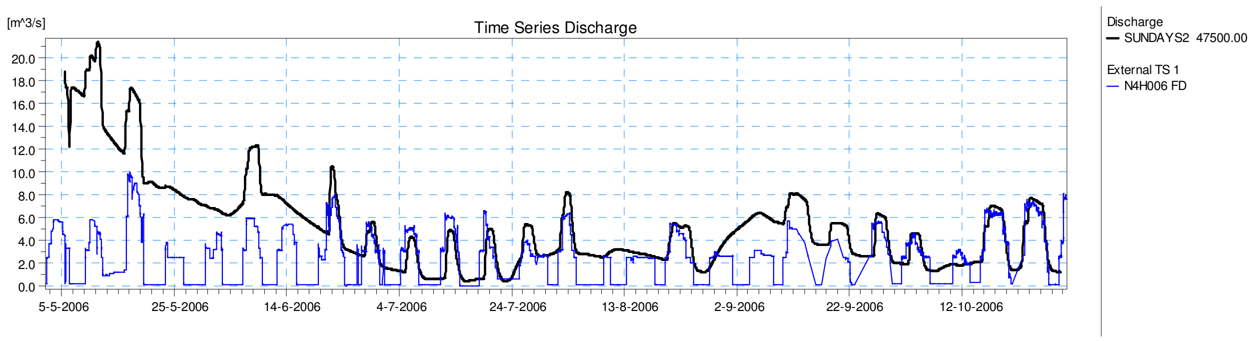
| Q8R001 WL | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.003 | |
| Max. positive difference | 10.044 | meter |
| Max. negative difference | -377.69 | meter |
| Volume observed | 8408691600 | |
| Volume modelled | 8407059572 | |
| Volume error | -0.019 | % |
| Peak observed value | 552.21 | meter |
| Peak modelled value | 562.247 | meter |
| Peak error | 1.818 | % |

# Verification Results for the Flow Rate at Korhaansdrift

SUNDAYS2 47500.00



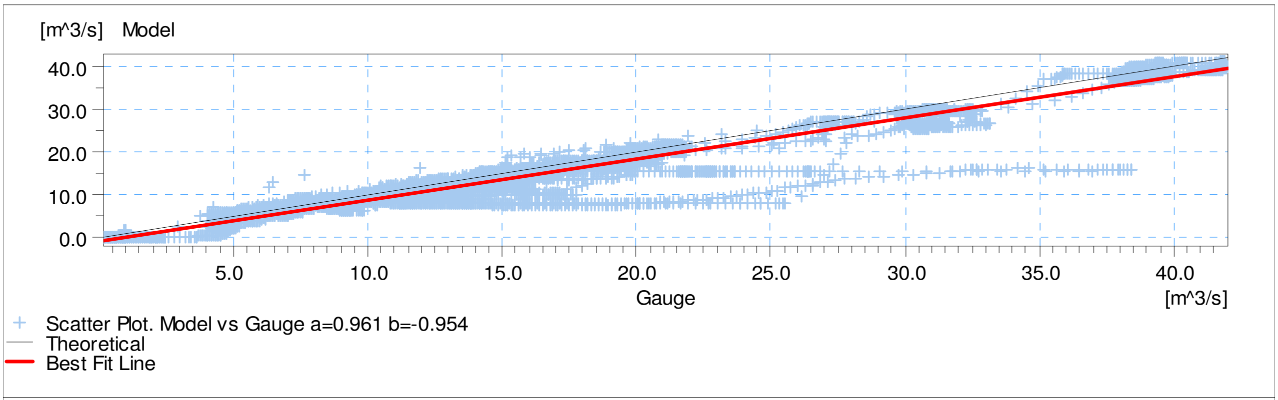

R2 = 0.217

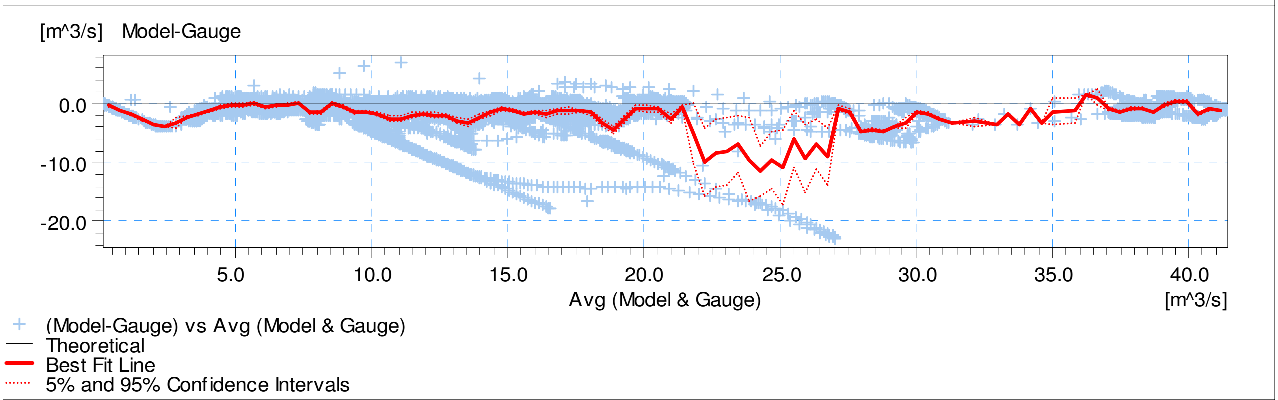


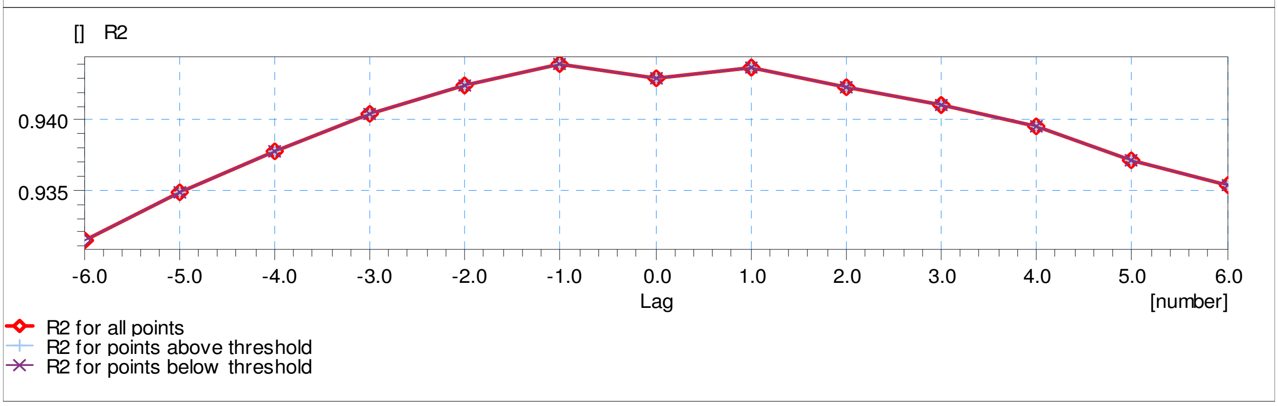
| N4H006 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.217 | |
| Max. positive difference | 18.422 | $m^3/s$ |
| Max. negative difference | -5.826 | $m^3/s$ |
| Volume observed | 35668501.92 | $m^3$ |
| Volume modelled | 80969412.7 | $m^3$ |
| Volume error | 127.005 | % |
| Peak observed value | 9.999 | $m^3/s$ |
| Peak modelled value | 21.394 | $m^3/s$ |
| Peak error | 113.958 | % |

# Verification Results for the Flow Rate at Teebus at Jan Blaauwskop

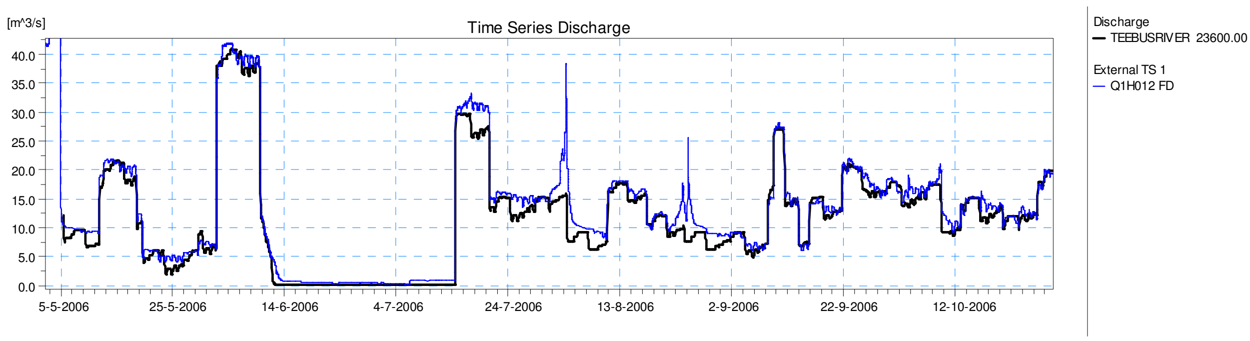
TEEBUSRIVER 23600.00



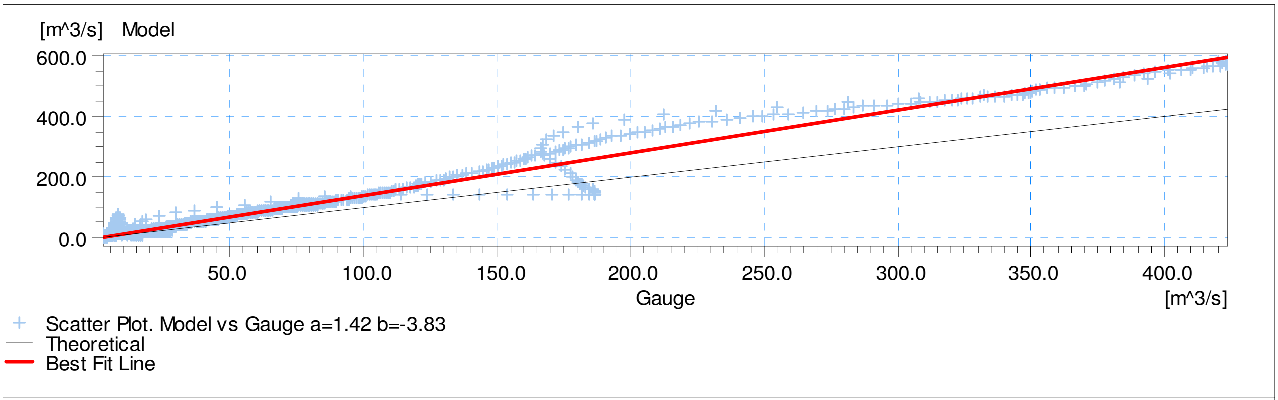

R2 = 0.943

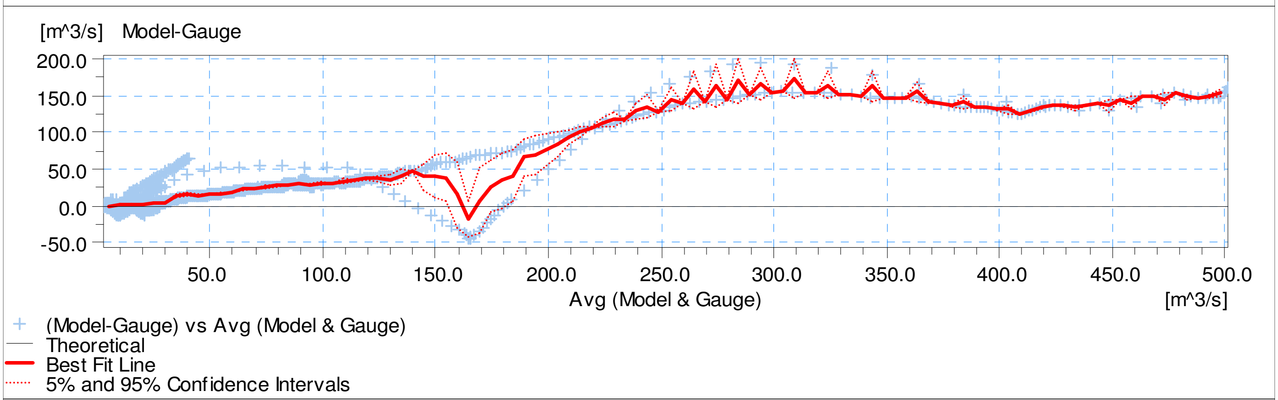


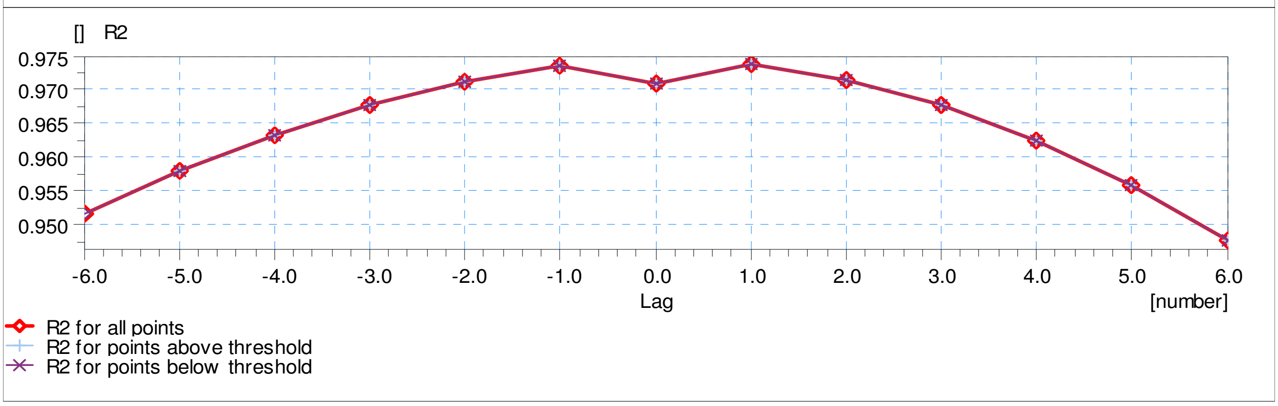
| Q1H012 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.943 | |
| Max. positive difference | 6.848 | $m^3/s$ |
| Max. negative difference | -22.748 | $m^3/s$ |
| Volume observed | 194024504.2 | $m^3$ |
| Volume modelled | 176931039.1 | $m^3$ |
| Volume error | -8.81 | % |
| Peak observed value | 41.986 | $m^3/s$ |
| Peak modelled value | 40.888 | $m^3/s$ |
| Peak error | -2.616 | % |

# Verification Results for the Flow Rate at Waaikraal Weir

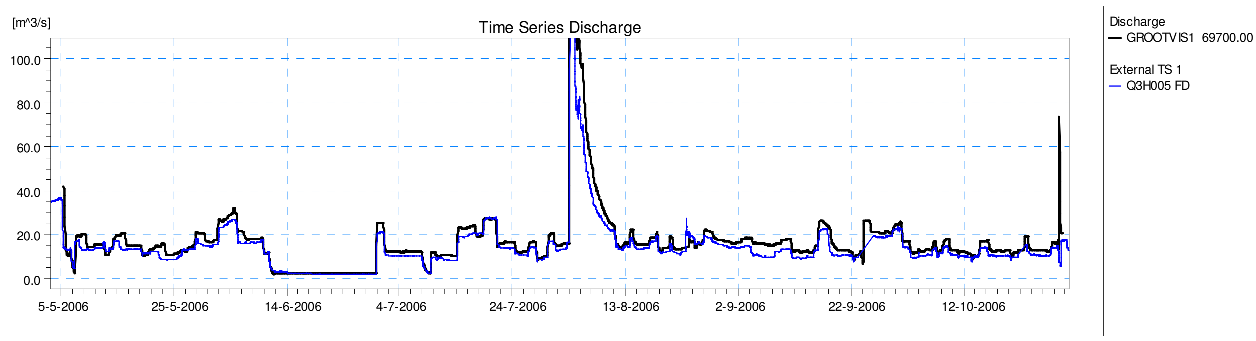
GROOTVIS1 69700.00



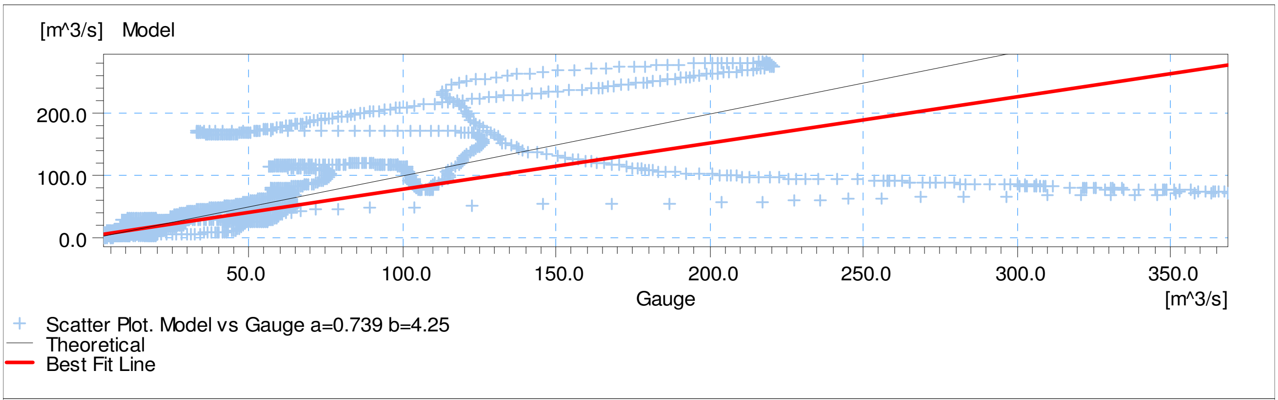

R2 = 0.971

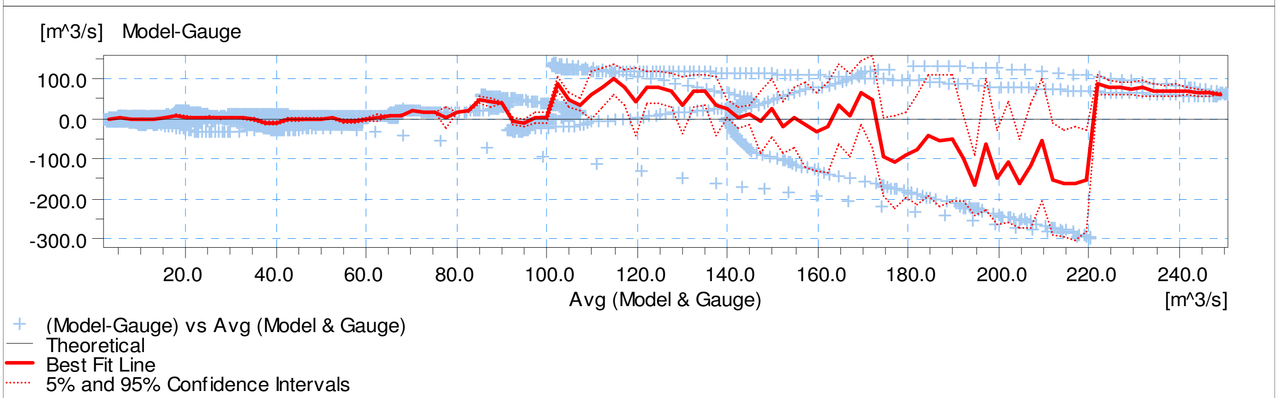


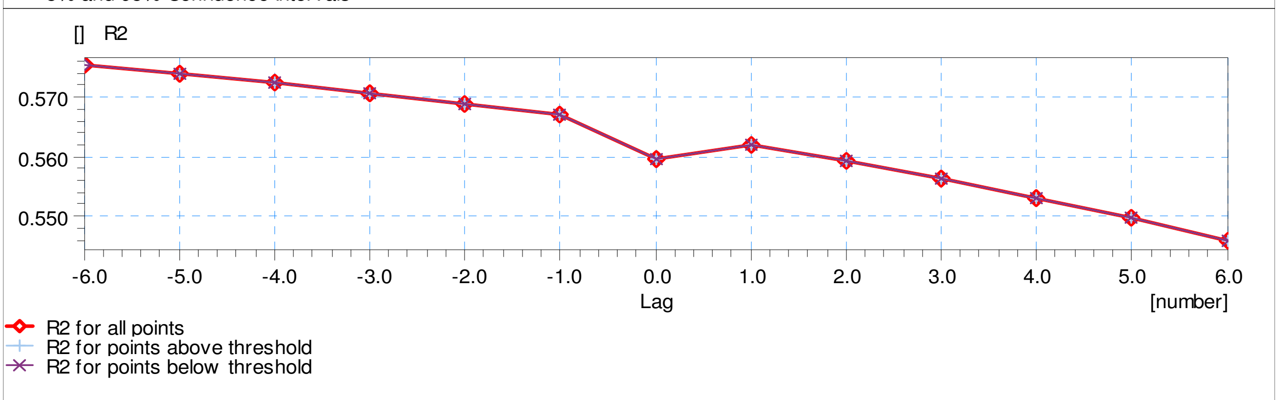
| Q3H005 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.971 | |
| Max. positive difference | 194.108 | $m^3/s$ |
| Max. negative difference | -43.683 | $m^3/s$ |
| Volume observed | 224745301.8 | $m^3$ |
| Volume modelled | 270801813.2 | $m^3$ |
| Volume error | 20.493 | % |
| Peak observed value | 423.471 | $m^3/s$ |
| Peak modelled value | 581.73 | $m^3/s$ |
| Peak error | 37.372 | % |

# Verification Results for the Flow Rate at Sheldon Weir

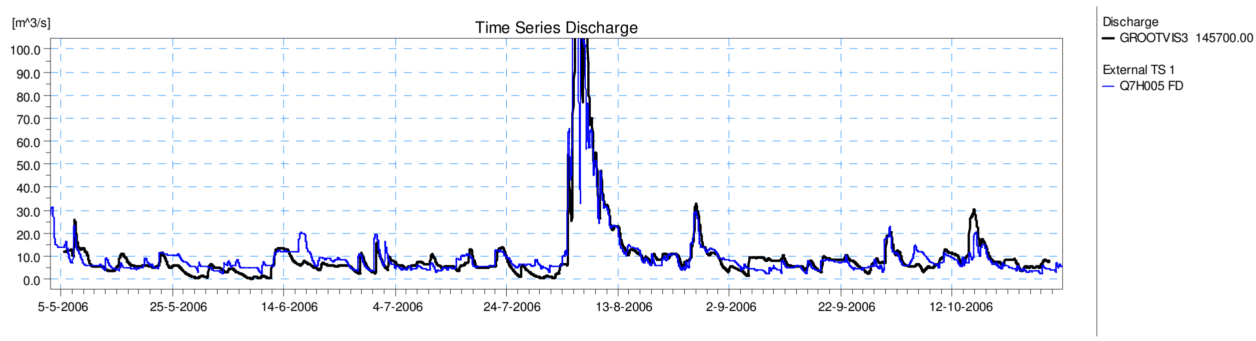
GROOTVIS3 145700.00



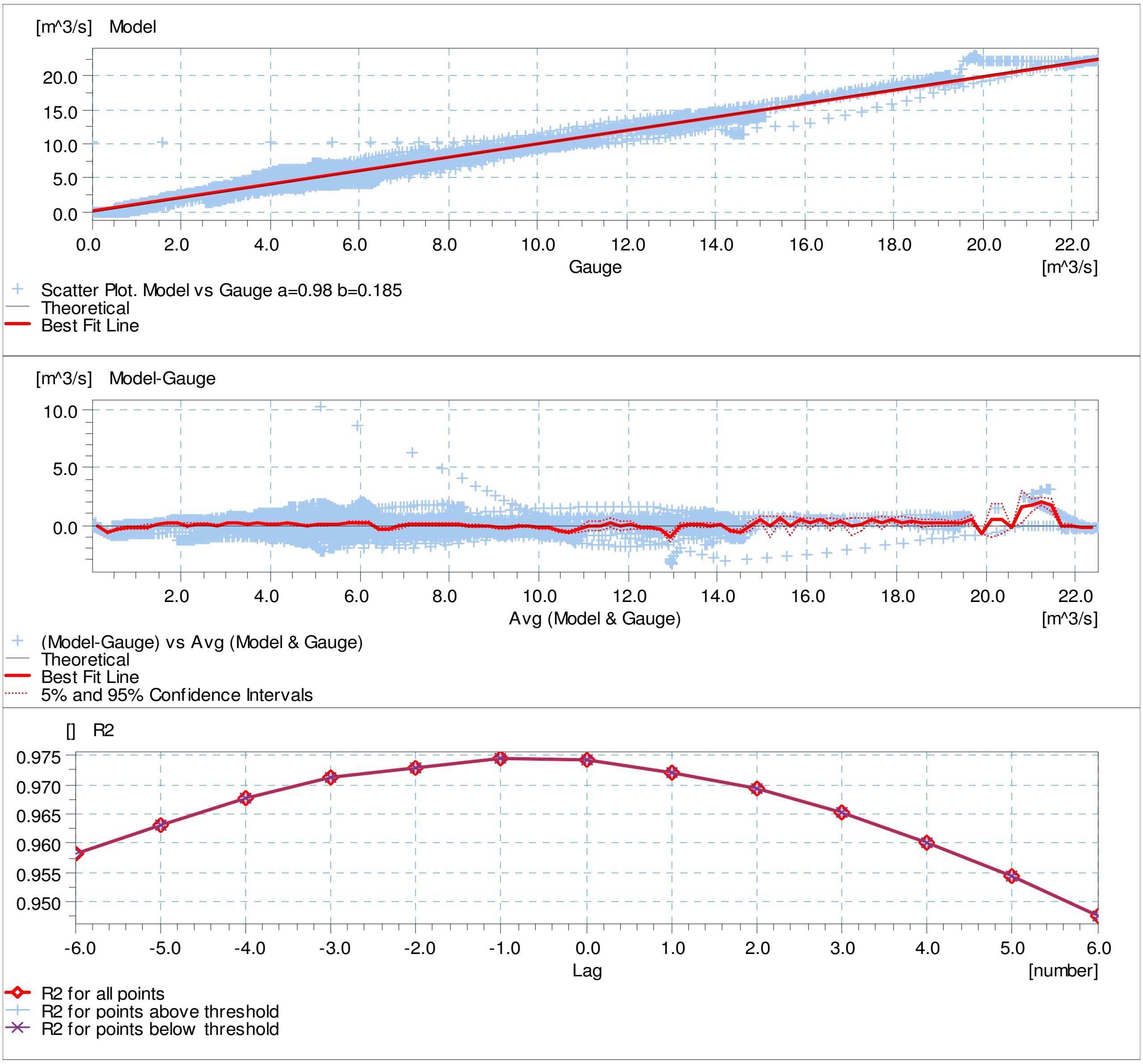

R2 = 0.560



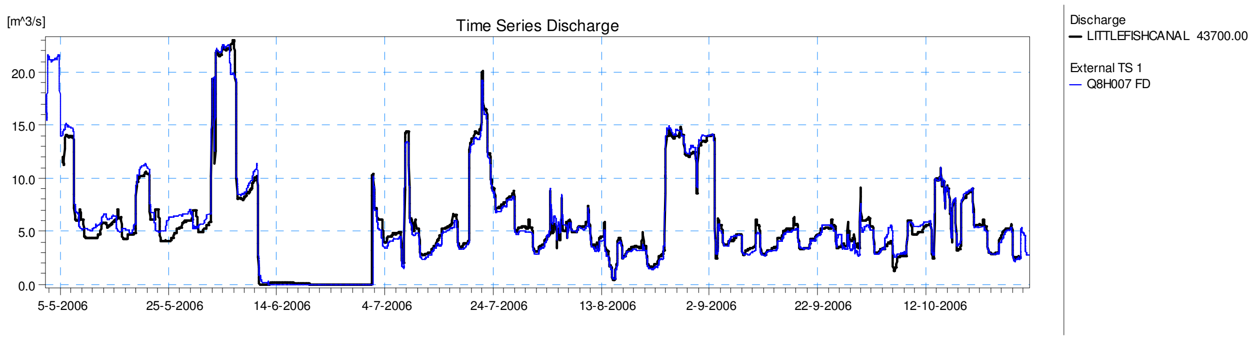
| Q7H005 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.56 | |
| Max. positive difference | 137.572 | $m^3/s$ |
| Max. negative difference | -296.493 | $m^3/s$ |
| Volume observed | 164286497.6 | $m^3$ |
| Volume modelled | 161462431.7 | $m^3$ |
| Volume error | -1.719 | % |
| Peak observed value | 368.515 | $m^3/s$ |
| Peak modelled value | 282.428 | $m^3/s$ |
| Peak error | -23.36 | % |

# Verification Results for the Flow Rate at Parshall 2 on the Little Fish Canal

LITTLEFISHCANAL 43700.00



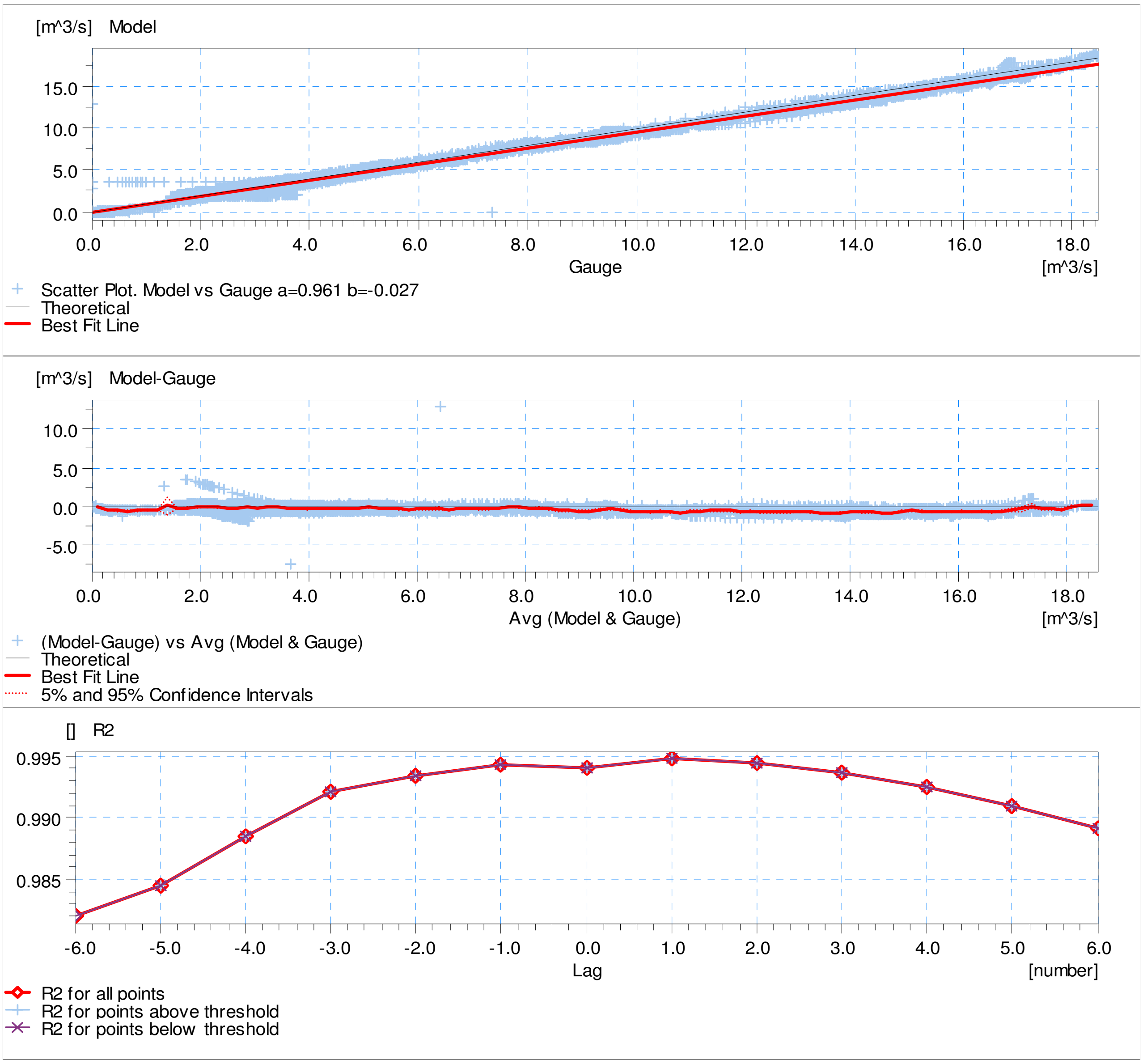

R2 = 0.974



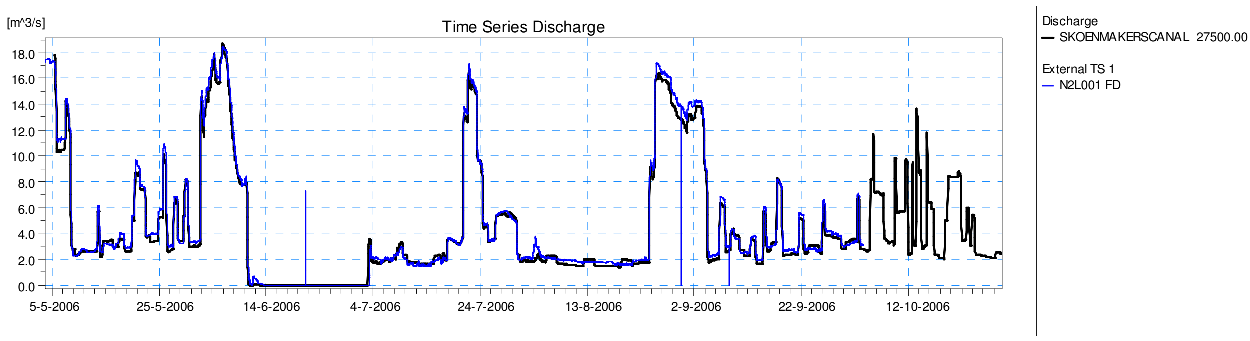
| Q8H007 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.974 | |
| Max. positive difference | 10.269 | $m^3/s$ |
| Max. negative difference | -3.315 | $m^3/s$ |
| Volume observed | 85979466.96 | $m^3$ |
| Volume modelled | 86333606.7 | $m^3$ |
| Volume error | 0.412 | % |
| Peak observed value | 22.579 | $m^3/s$ |
| Peak modelled value | 23.007 | $m^3/s$ |
| Peak error | 1.896 | % |

# Verification Results for the Flow Rate at Parshall 3 on the Skoenmakers Canal

SKOENMAKERSCANAL 27500.00



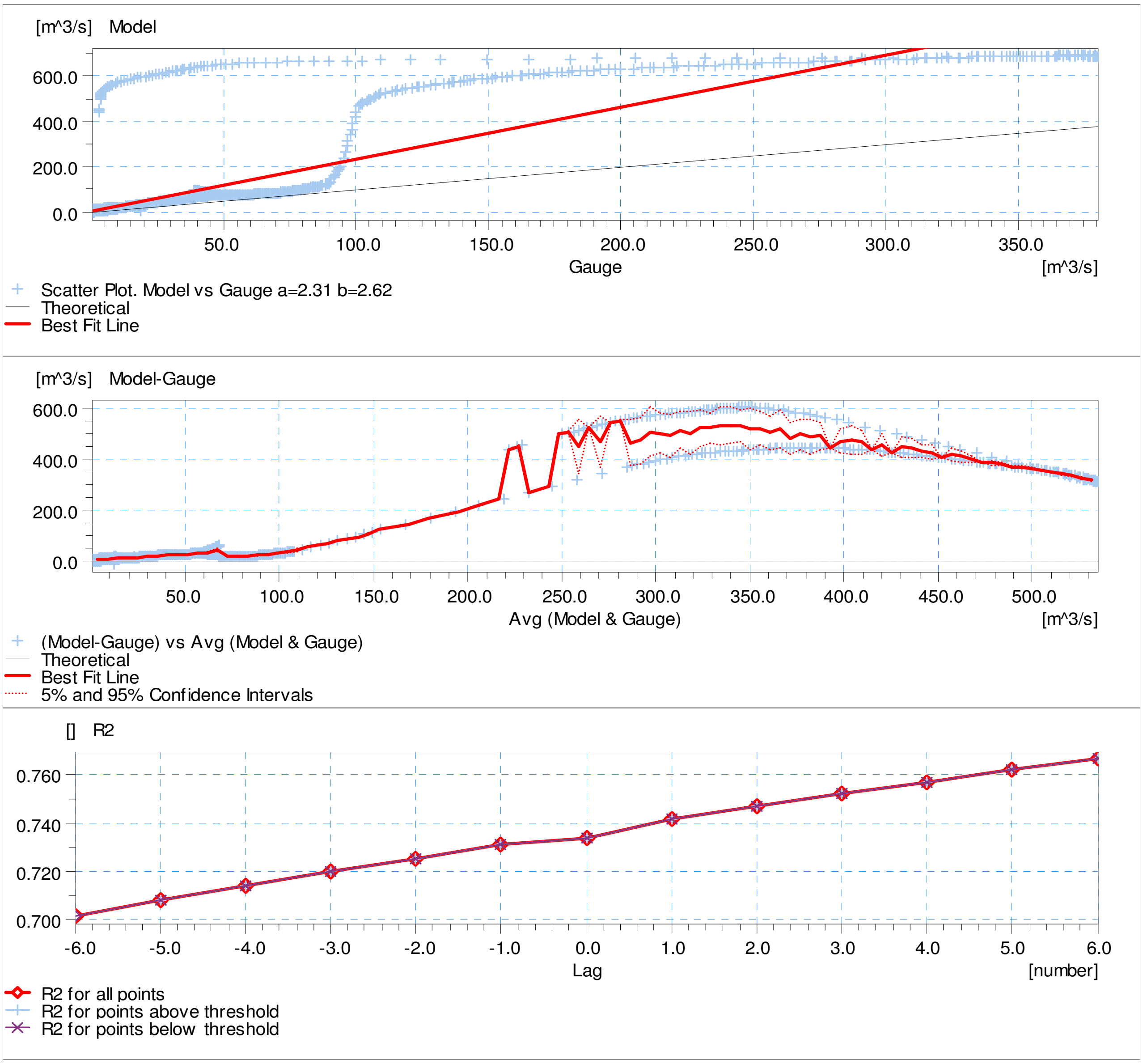

R2 = 0.994



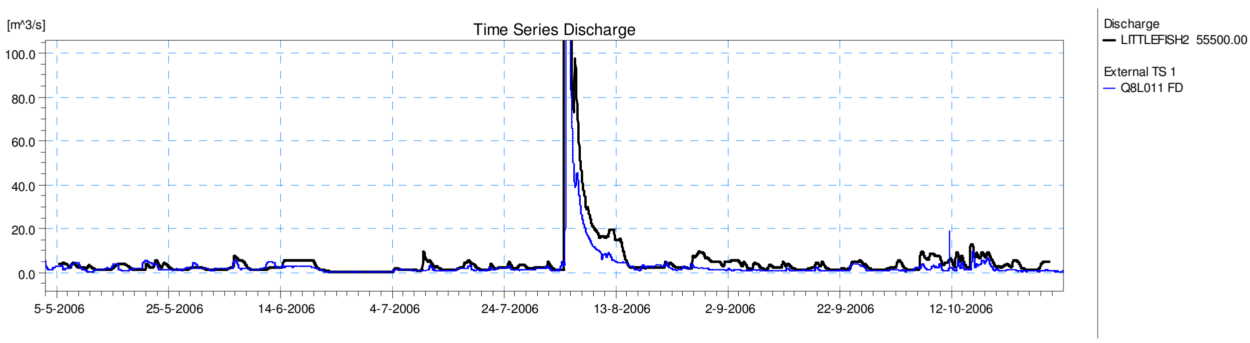
| **N2L001 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.994 | |
| Max. positive difference | 12.859 | $m^3/s$ |
| Max. negative difference | -7.338 | $m^3/s$ |
| Volume observed | 58993335.51 | $m^3$ |
| Volume modelled | 56474607.08 | $m^3$ |
| Volume error | -4.27 | % |
| Peak observed value | 18.461 | $m^3/s$ |
| Peak modelled value | 18.691 | $m^3/s$ |
| Peak error | 1.247 | % |

# Verification Results for the Flow Rate at Junctiondrift Weir

LITTLEFISH2 55500.00



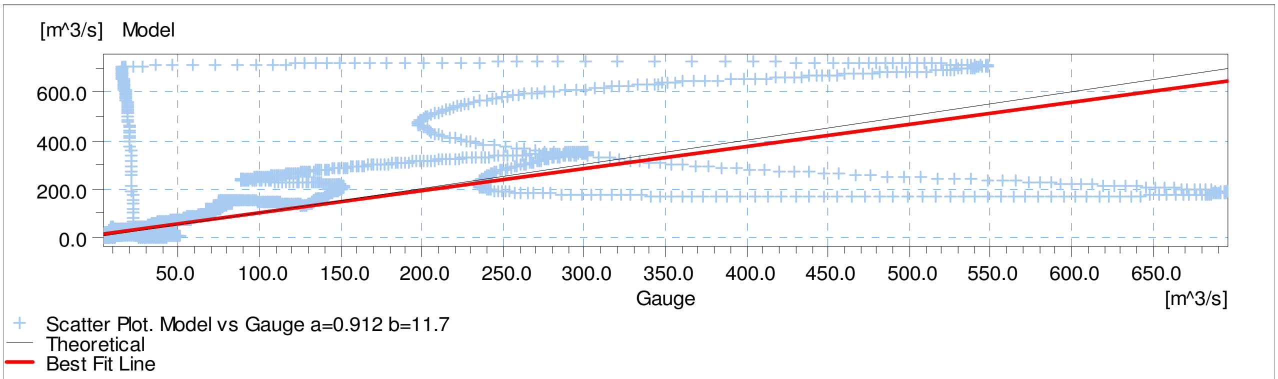

R2 = 0.734

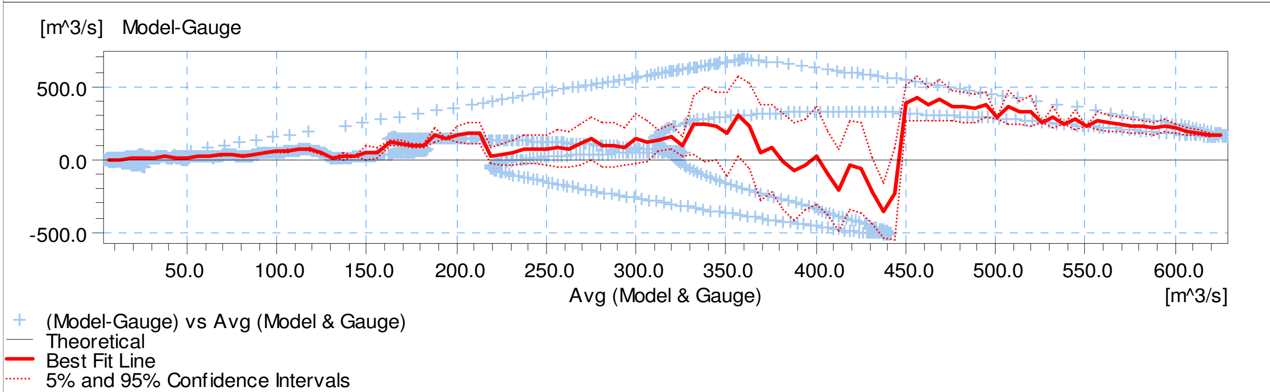


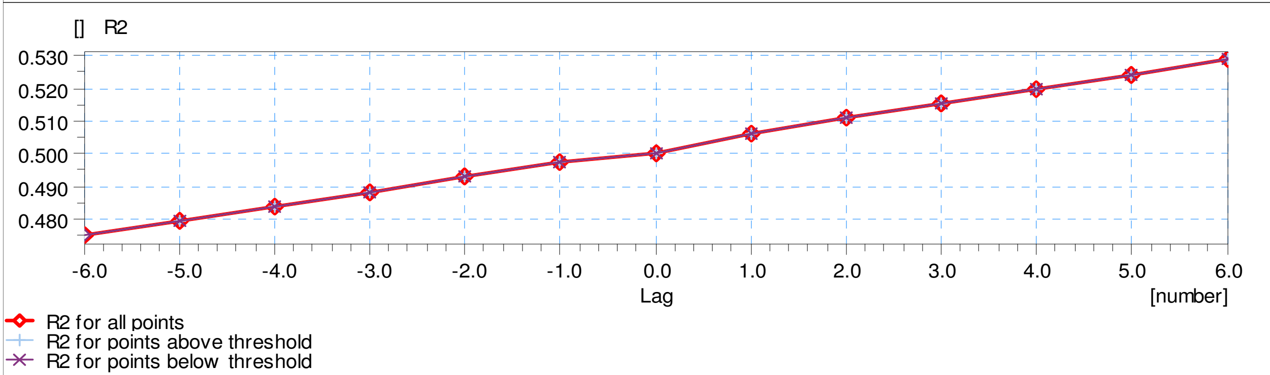
| Q8L011 FD | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.734 | |
| Max. positive difference | 600.795 | $m^3/s$ |
| Max. negative difference | -12.809 | $m^3/s$ |
| Volume observed | 56089937.34 | $m^3$ |
| Volume modelled | 127702364.3 | $m^3$ |
| Volume error | 127.674 | % |
| Peak observed value | 380.14 | $m^3/s$ |
| Peak modelled value | 690.473 | $m^3/s$ |
| Peak error | 81.636 | % |

# Verification Results for the Flow Rate at Piggott's Bridge Weir

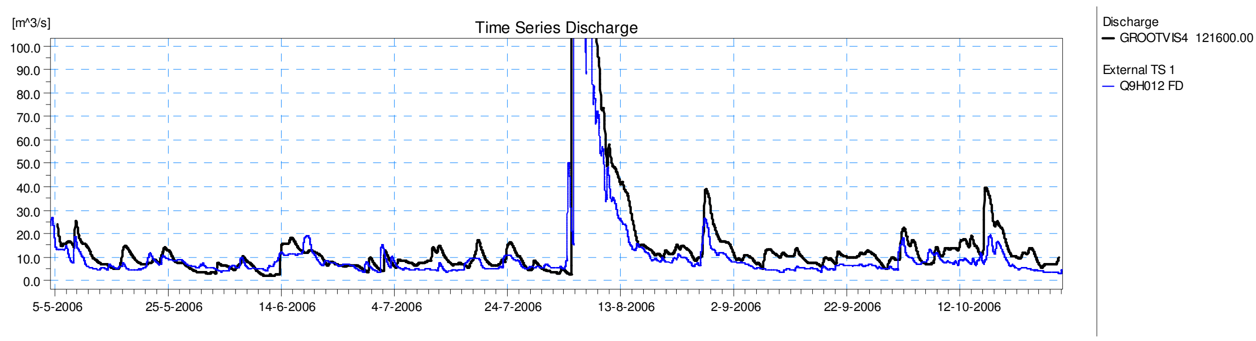
GROOTVIS4 121600.00




R2 = 0.500



| **Q9H012 FD** | | |
|---|---|---|
| Result | Value | |
| Correlation coefficient $R^2$ | 0.5 | |
| Max. positive difference | 687.784 | $m^3/s$ |
| Max. negative difference | -509.004 | $m^3/s$ |
| Volume observed | 207423773.7 | $m^3$ |
| Volume modelled | 290440953.1 | $m^3$ |
| Volume error | 40.023 | % |
| Peak observed value | 695.605 | $m^3/s$ |
| Peak modelled value | 724.588 | $m^3/s$ |
| Peak error | 4.167 | % |

# Appendix F: Tributary Recession Curves and Calibration of the K value

The K value defines the general recession curve for a tributary catchment based on historic flood events. This value is based on a pulse response function and has the form given in Equations F-1 to F-3.

$$If\left(q_1 > T_f\right) \rightarrow then \rightarrow q_2 = q_1.e^{(-(t_2-t).24/K_U)}$$ **Equation F-1**

$$If\left(q_1 > B_f\right) \rightarrow then \rightarrow q_2 = q_1.e^{(-(t_2-t).24/K_L)}$$ **Equation F-2**

$$Else \rightarrow q_2 = B_f$$ **Equation F-3**

The error was calculated by a simple difference divided by the measured value as an absolute value scheme. Thus small differences for small flows were more heavily penalised than small differences for larger flows. This was chosen as the majority of the flows encountered would be small flows, thus greater accuracy was required for these flows.

# Calibration of K value for Sundays River at De Draai N2H007

## Data

**N2H007**

Mode 4.702
Mean 50.399
Min 0.842
Max 368.798

## Forecast

$B_f$ 0.001 Tf 1.00
$K_U$ 11.00 $K_L$ 110

Basic Forecast Equation is defined as:

$$q_2 = q_1 \times \exp^{(-dt/k)}$$

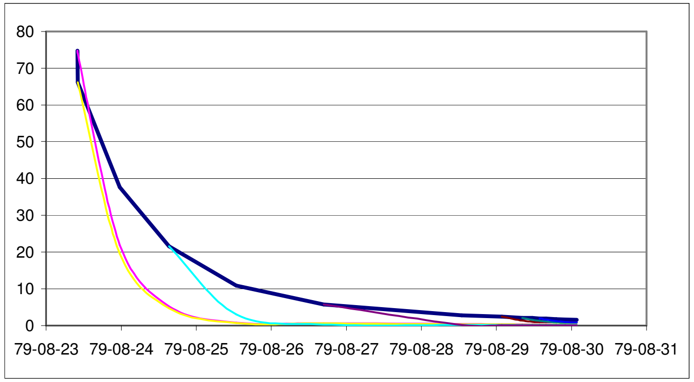
## Error Calculation

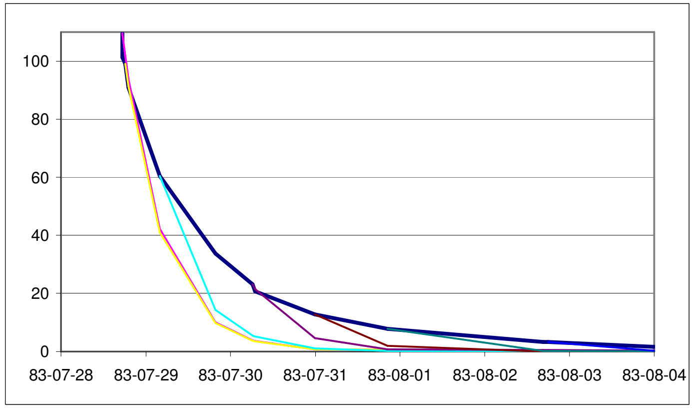
**Total AVG** **#REF!**

Error = (ABS(measured data - forecast data) / measured date) as a percentage

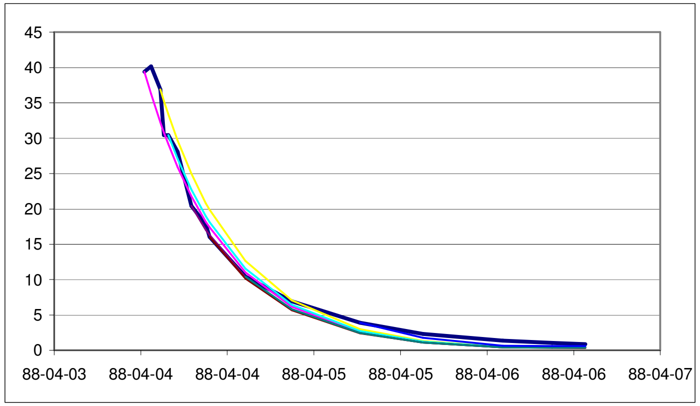
## Plots of Data vs Forecast

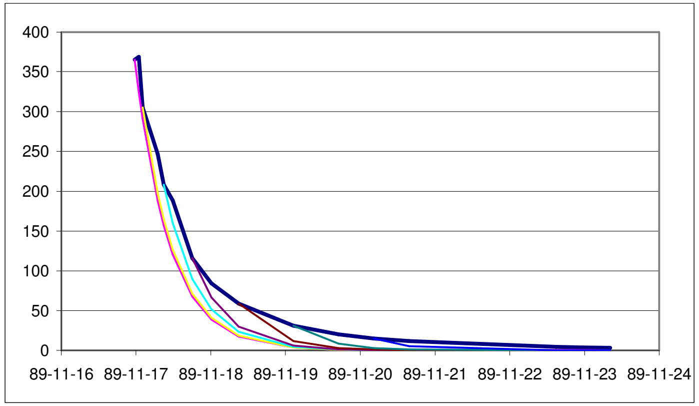
| DATE | Discharge | Forecast | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1979-08-21 22:18 | 72.4 | TOF | | | | | | | Error | | | | | Total | 4244% |
| 1979-08-23 10:00 | 74.774 | 74.774 | | | | | | | 0% | | | | | Avg | 69% |
| 1979-08-23 10:06 | 66.111 | 74.1 | 66.11 | | | | | | 12% | 0% | | | | | |
| 1979-08-23 23:30 | 37.588 | 21.9 | 19.6 | | | | | | 42% | 48% | | | | | |
| 1979-08-24 15:18 | 21.494 | 5.2 | 4.6 | 21.5 | | | | | 76% | 78% | 0% | | | | |
| 1979-08-25 12:48 | 10.878 | 0.7 | 0.7 | 3.0 | | | | | 93% | 94% | 72% | | | | |
| 1979-08-26 16:42 | 5.716 | 0.6 | 0.5 | 0.2 | 5.72 | | | | 90% | 91% | 96% | 0% | | | |
| 1979-08-28 12:54 | 2.736 | 0.4 | 0.3 | 0.2 | 0.1 | | | | 86% | 88% | 94% | 96% | | | |
| 1979-08-29 01:42 | 2.352 | 0.3 | 0.3 | 0.1 | 0.1 | 2.35 | | | 85% | 87% | 94% | 96% | 0% | | |
| 1979-08-29 08:18 | 2.026 | 0.3 | 0.3 | 0.1 | 0.1 | 1.3 | 2.03 | | 84% | 86% | 93% | 96% | 36% | | |
| 1979-08-29 11:24 | 2.014 | 0.3 | 0.3 | 0.1 | 0.1 | 1.0 | 1.5 | | 84% | 86% | 93% | 96% | 52% | 24% | |
| 1979-08-29 13:48 | 1.912 | 0.3 | 0.3 | 0.1 | 0.1 | 1.0 | 1.2 | 1.91 | 84% | 86% | 93% | 96% | 50% | 36% | |
| 1979-08-29 15:48 | 1.835 | 0.3 | 0.3 | 0.1 | 0.1 | 0.9 | 1.0 | 1.6 | 84% | 85% | 93% | 96% | 49% | 44% | 13% |
| 1979-08-29 19:06 | 1.707 | 0.3 | 0.3 | 0.1 | 0.1 | 0.9 | 0.8 | 1.2 | 83% | 85% | 93% | 95% | 47% | 56% | 31% |
| 1979-08-29 22:00 | 1.605 | 0.3 | 0.3 | 0.1 | 0.1 | 0.9 | 0.7 | 0.9 | 82% | 84% | 93% | 95% | 45% | 54% | 43% |
| 1979-08-30 01:42 | 1.516 | 0.3 | 0.2 | 0.1 | 0.1 | 0.9 | 0.7 | 0.9 | 82% | 84% | 92% | 95% | 44% | 53% | 42% |
| 1983-07-28 16:24 | 109.773 | TOF | | | | | | | Error | | | | | Total | 4279% |
| 1983-07-28 17:24 | 109.587 | 109.587 | | | | | | | 0% | | | | | Avg | 72% |
| 1983-07-28 17:25 | 101.233 | 109.4 | | | | | | | 8% | 100% | | | | | |
| 1983-07-28 18:12 | 99.071 | 101.9 | 99.07 | | | | | | 3% | 0% | | | | | |
| 1983-07-28 19:06 | 90.895 | 93.9 | 91.3 | | | | | | 3% | 0% | 100% | | | | |
| 1983-07-29 03:54 | 60.437 | 42.2 | 41.0 | 60.4 | | | | | 30% | 32% | 0% | | | | |
| 1983-07-29 19:42 | 33.683 | 10.0 | 9.8 | 14.4 | | | | | 70% | 71% | 57% | 100% | | | |
| 1983-07-30 06:12 | 23.075 | 3.9 | 3.8 | 5.5 | 23.1 | | | | 83% | 84% | 76% | 0% | | | |
| 1983-07-30 07:00 | 20.734 | 3.6 | 3.5 | 5.1 | 21.5 | | | | 83% | 83% | 75% | 3% | 100% | | |
| 1983-07-30 23:54 | 12.789 | 0.8 | 0.8 | 1.1 | 4.6 | 12.8 | | | 94% | 94% | 91% | 64% | 0% | | |
| 1983-07-31 20:24 | 7.804 | 0.6 | 0.6 | 0.2 | 0.7 | 2.0 | 7.8 | | 92% | 92% | 98% | 91% | 75% | 0% | |
| 1983-08-02 16:30 | 3.231 | 0.4 | 0.4 | 0.1 | 0.5 | 0.0 | 0.1 | | 87% | 87% | 96% | 85% | 99% | 96% | |
| 1983-08-02 17:54 | 3.247 | 0.4 | 0.4 | 0.1 | 0.5 | 0.0 | 0.1 | 3.25 | 87% | 87% | 97% | 85% | 99% | 96% | 0% |
| 1983-08-03 23:36 | 1.635 | 0.3 | 0.3 | 0.1 | 0.4 | 0.0 | 0.1 | 0.2 | 80% | 81% | 95% | 78% | 98% | 93% | 87% |
| 1983-08-04 09:06 | 1.516 | 0.3 | 0.3 | 0.1 | 0.3 | 0.0 | 0.1 | 0.2 | 80% | 81% | 95% | 78% | 98% | 94% | 87% |
| 1983-08-04 23:36 | 1.008 | 0.3 | 0.3 | 0.1 | 0.3 | 0.0 | 0.1 | 0.2 | 74% | 75% | 93% | 71% | 98% | 91% | 83% |
| 1983-08-05 02:24 | 0.992 | 0.3 | 0.2 | 0.1 | 0.3 | 0.0 | 0.1 | 0.2 | 74% | 75% | 93% | 71% | 98% | 92% | 83% |

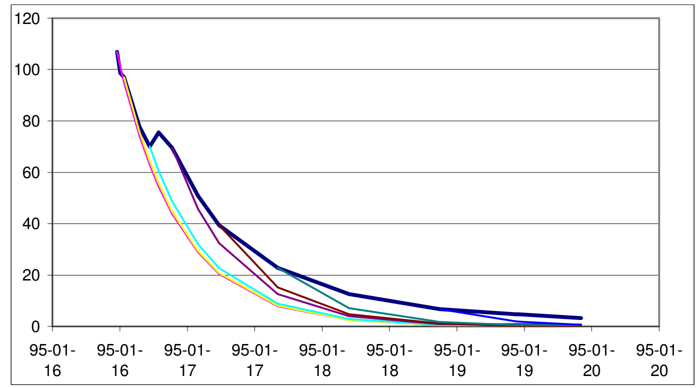


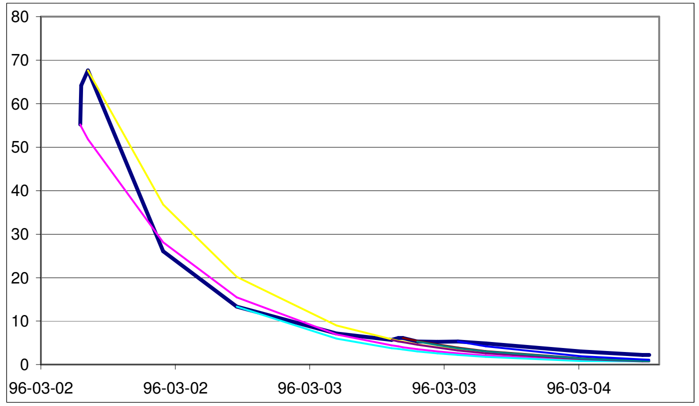
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1988-04-04 00:06 | 35.669 | TOF | | | | | | | Error | | | | | Total | 1686% |
| 1988-04-04 00:30 | 39.375 | 39.375 | | | | | | | 0% | | | | | Avg | 32% |
| 1988-04-04 01:24 | 40.184 | 36.3 | | | | | | | 10% | 100% | | | | | |
| 1988-04-04 02:42 | 36.913 | 32.2 | 36.91 | | | | | | 13% | 0% | | | | | |
| 1988-04-04 03:12 | 30.461 | 30.8 | 35.3 | | | | | | 1% | 16% | 100% | | | | |
| 1988-04-04 03:48 | 30.461 | 29.2 | 33.4 | 30.5 | | | | | 4% | 10% | 0% | | | | |
| 1988-04-04 05:06 | 28.089 | 25.9 | 29.7 | 27.1 | | | | | 8% | 6% | 4% | 100% | | | |
| 1988-04-04 07:00 | 20.432 | 21.8 | 25.0 | 22.8 | 20.4 | | | | 7% | 22% | 11% | 0% | | | |
| 1988-04-04 09:00 | 17.798 | 18.2 | 20.8 | 19.0 | 17.0 | | | | 2% | 17% | 7% | 4% | 100% | | |
| 1988-04-04 09:30 | 16.025 | 17.4 | 19.9 | 18.1 | 16.3 | 16 | | | 8% | 24% | 13% | 2% | 0% | | |
| 1988-04-04 14:30 | 10.397 | 11.0 | 12.6 | 11.5 | 10.3 | 10.2 | 10.4 | | 6% | 21% | 11% | 1% | 2% | 0% | |
| 1988-04-04 20:54 | 6.938 | 6.2 | 7.1 | 6.4 | 5.8 | 5.7 | 5.8 | | 11% | 2% | 7% | 17% | 18% | 16% | |
| 1988-04-05 06:24 | 3.897 | 2.6 | 3.0 | 2.7 | 2.4 | 2.4 | 2.4 | 3.9 | 33% | 24% | 30% | 38% | 38% | 37% | 0% |
| 1988-04-05 15:06 | 2.339 | 1.2 | 1.3 | 1.2 | 1.1 | 1.1 | 1.1 | 1.8 | 50% | 42% | 47% | 53% | 54% | 53% | 24% |
| 1988-04-06 02:12 | 1.357 | 0.4 | 0.5 | 0.4 | 0.4 | 0.4 | 0.4 | 0.6 | 68% | 64% | 67% | 70% | 71% | 70% | 53% |
| 1988-04-06 09:00 | 1.023 | 0.4 | 0.5 | 0.4 | 0.4 | 0.4 | 0.4 | 0.6 | 61% | 55% | 59% | 63% | 64% | 63% | 41% |
| 1988-04-06 13:36 | 0.842 | 0.4 | 0.4 | 0.4 | 0.4 | 0.4 | 0.4 | 0.6 | 54% | 47% | 52% | 57% | 58% | 57% | 31% |
| 1989-11-16 21:18 | 352.523 | TOF | | | | | | | Error | | | | | Total | 3876% |
| 1989-11-16 23:36 | 365.037 | 365.037 | | | | | | | 0% | | | | | Avg | 68% |
| 1989-11-17 00:54 | 368.798 | 324.3 | | | | | | | 12% | 100% | | | | | |
| 1989-11-17 02:06 | 305.355 | 290.8 | 305.4 | | | | | | 5% | 0% | | | | | |
| 1989-11-17 06:54 | 247.007 | 188.0 | 197.4 | | | | | | 24% | 20% | 100% | | | | |
| 1989-11-17 08:54 | 208.091 | 156.7 | 164.6 | 208 | | | | | 25% | 21% | 0% | | | | |
| 1989-11-17 11:48 | 188.56 | 120.4 | 126.4 | #### | | | | | 36% | 33% | 15% | 100% | | | |
| 1989-11-17 18:06 | 115.776 | 67.9 | 71.3 | 90.2 | 116 | | | | 41% | 38% | 22% | 0% | | | |
| 1989-11-18 00:12 | 84.28 | 39.0 | 41.0 | 51.8 | 66.5 | | | | 54% | 51% | 39% | 21% | 100% | | |
| 1989-11-18 08:54 | 58.618 | 17.7 | 18.6 | 23.5 | 30.2 | 58.6 | | | 70% | 68% | 60% | 49% | 0% | | |
| 1989-11-19 02:36 | 31.143 | 3.5 | 3.7 | 4.7 | 6.0 | 11.7 | 31.1 | | 89% | 88% | 85% | 81% | 62% | 0% | |
| 1989-11-19 17:00 | 20.432 | 1.0 | 1.0 | 1.3 | 1.6 | 3.2 | 8.4 | | 95% | 95% | 94% | 92% | 84% | 59% | |
| 1989-11-20 04:18 | 14.852 | 0.9 | 0.4 | 0.5 | 0.6 | 1.1 | 3.0 | 14.9 | 94% | 98% | 97% | 96% | 92% | 80% | 0% |
| 1989-11-20 15:42 | 11.885 | 0.8 | 0.3 | 0.4 | 0.5 | 0.4 | 1.1 | 5.3 | 93% | 97% | 97% | 96% | 97% | 91% | 56% |
| 1989-11-22 14:54 | 4.702 | 0.5 | 0.2 | 0.3 | 0.3 | 0.3 | 0.0 | 0.1 | 89% | 96% | 94% | 93% | 94% | 100% | 98% |
| 1989-11-22 23:36 | 3.644 | 0.5 | 0.2 | 0.2 | 0.3 | 0.2 | 0.0 | 0.1 | 87% | 95% | 93% | 91% | 93% | 100% | 98% |
| 1989-11-23 08:12 | 3.201 | 0.4 | 0.2 | 0.2 | 0.3 | 0.2 | 0.0 | 0.1 | 86% | 94% | 93% | 91% | 93% | 100% | 98% |
| 1995-01-16 10:30 | 97.292 | TOF | | | | | | | Error | | | | | Total | 2872% |
| 1995-01-16 11:30 | 106.942 | 106.942 | | | | | | | 0% | | | | | Avg | 53% |
| 1995-01-16 12:06 | 98.711 | 101.3 | | | | | | | 3% | | | | | | |
| 1995-01-16 12:48 | 96.948 | 95.0 | 96.95 | | | | | | 2% | 0% | | | | | |
| 1995-01-16 15:30 | 77.814 | 74.3 | 75.8 | | | | | | 4% | 3% | | | | | |
| 1995-01-16 17:18 | 70.074 | 63.1 | 64.4 | 70.1 | | | | | 10% | 8% | 0% | | | | |
| 1995-01-16 18:54 | 75.528 | 54.6 | 55.7 | 60.6 | | | | | 28% | 26% | 20% | | | | |
| 1995-01-16 21:18 | 69.495 | 43.9 | 44.8 | 48.7 | 69.5 | | | | 37% | 36% | 30% | 0% | | | |
| 1995-01-17 01:54 | 50.845 | 28.9 | 29.5 | 32.1 | 45.7 | | | | 43% | 42% | 37% | 10% | | | |
| 1995-01-17 05:42 | 39.375 | 20.4 | 20.9 | 22.7 | 32.4 | 39.4 | | | 48% | 47% | 42% | 18% | 0% | | |
| 1995-01-17 16:06 | 22.831 | 7.9 | 8.1 | 8.8 | 12.6 | 15.3 | 22.8 | | 65% | 65% | 61% | 45% | 33% | 0% | |
| 1995-01-18 04:48 | 12.627 | 2.5 | 2.6 | 2.8 | 4.0 | 4.8 | 7.2 | | 80% | 80% | 78% | 69% | 62% | 43% | |
| 1995-01-18 20:54 | 6.758 | 0.6 | 0.6 | 0.6 | 0.9 | 1.1 | 1.7 | 6.76 | 91% | 91% | 90% | 86% | 83% | 75% | 0% |
| 1995-01-19 10:24 | 4.702 | 0.5 | 0.5 | 0.6 | 0.8 | 0.3 | 0.5 | 2.0 | 89% | 89% | 88% | 83% | 93% | 90% | 58% |
| 1995-01-19 10:54 | 4.702 | 0.5 | 0.5 | 0.6 | 0.8 | 0.3 | 0.5 | 1.9 | 89% | 89% | 88% | 83% | 93% | 90% | 60% |
| 1995-01-19 22:06 | 3.293 | 0.5 | 0.5 | 0.5 | 0.7 | 0.3 | 0.4 | 0.7 | 86% | 86% | 84% | 78% | 91% | 87% | 79% |

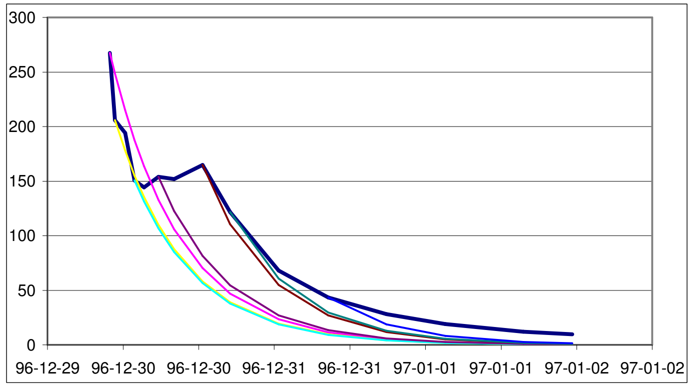


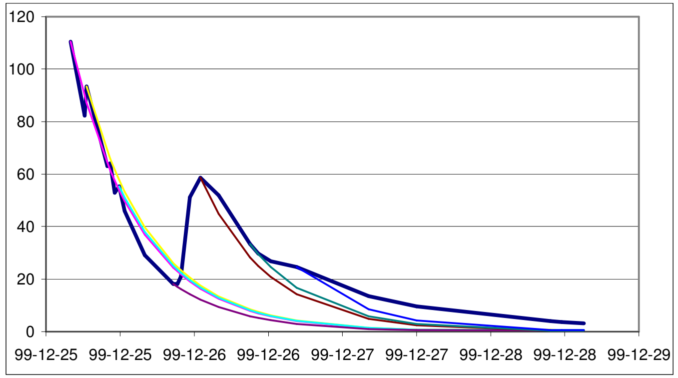



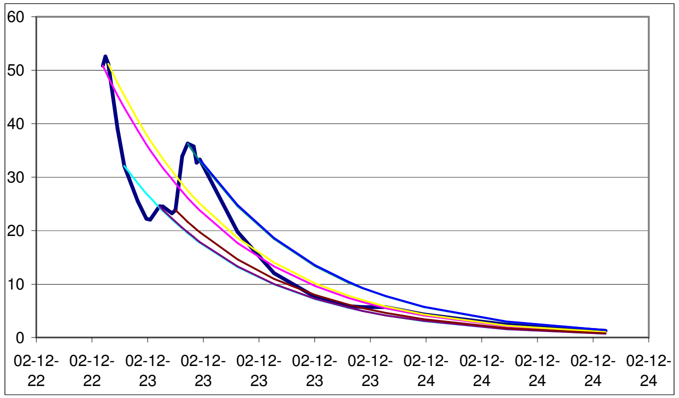
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1996-03-02 06:42 | 17.327 | TOF | | | | | | | Error | | | | | Total | 1661% |
| 1996-03-02 08:18 | 55.197 | 55.197 | | | | | | | 0% | | | | | Avg | 35% |
| 1996-03-02 08:24 | 64.187 | 54.7 | | | | | | | 15% | | | | | | |
| 1996-03-02 09:00 | 67.648 | 51.8 | 67.65 | | | | | | 23% | 0% | | | | | |
| 1996-03-02 15:42 | 26.02 | 28.2 | 36.8 | | | | | | 8% | 41% | | | | | |
| 1996-03-02 22:18 | 13.342 | 15.5 | 20.2 | 13.3 | | | | | 16% | 51% | 0% | | | | |
| 1996-03-03 07:12 | 7.158 | 6.9 | 9.0 | 5.9 | | | | | 4% | 26% | 17% | | | | |
| 1996-03-03 12:06 | 5.653 | 4.4 | 5.8 | 3.8 | 5.65 | | | | 22% | 2% | 33% | 0% | | | |
| 1996-03-03 12:42 | 6.106 | 4.2 | 5.5 | 3.6 | 5.4 | | | | 32% | 11% | 41% | 12% | | | |
| 1996-03-03 13:06 | 6.106 | 4.0 | 5.3 | 3.5 | 5.2 | 6.11 | | | 34% | 14% | 43% | 15% | 0% | | |
| 1996-03-03 14:24 | 5.283 | 3.6 | 4.7 | 3.1 | 4.6 | 5.4 | 5.28 | | 32% | 12% | 42% | 13% | 3% | 0% | |
| 1996-03-03 16:12 | 5.222 | 3.0 | 4.0 | 2.6 | 3.9 | 4.6 | 4.5 | | 42% | 24% | 50% | 25% | 12% | 14% | |
| 1996-03-03 18:00 | 5.283 | 2.6 | 3.4 | 2.2 | 3.3 | 3.9 | 3.8 | 5.28 | 51% | 36% | 58% | 37% | 26% | 28% | 0% |
| 1996-03-03 20:30 | 4.843 | 2.1 | 2.7 | 1.8 | 2.6 | 3.1 | 3.0 | 4.2 | 58% | 45% | 63% | 46% | 36% | 37% | 13% |
| 1996-03-04 05:00 | 2.992 | 0.9 | 1.2 | 0.8 | 1.2 | 1.4 | 1.4 | 1.9 | 68% | 59% | 73% | 59% | 52% | 53% | 35% |
| 1996-03-04 10:30 | 2.24 | 0.9 | 0.8 | 0.8 | 0.7 | 0.9 | 0.8 | 1.2 | 60% | 66% | 65% | 67% | 61% | 62% | 47% |
| 1996-03-04 11:06 | 2.252 | 0.9 | 0.7 | 0.8 | 0.7 | 0.9 | 0.8 | 1.1 | 60% | 67% | 66% | 67% | 61% | 62% | 50% |
| 1996-12-29 20:30 | 194.49 | TOF | | | | | | | Error | | | | | Total | 3145% |
| 1996-12-29 21:54 | 267.376 | 267.376 | | | | | | | 0% | | | | | Avg | 55% |
| 1996-12-29 22:42 | 205.847 | 248.6 | 205.8 | | | | | | 21% | 0% | | | | | |
| 1996-12-30 00:18 | 194.208 | 215.0 | 178.0 | | | | | | 11% | 8% | | | | | |
| 1996-12-30 01:48 | 150.692 | 187.6 | 155.3 | 151 | | | | | 24% | 3% | 0% | | | | |
| 1996-12-30 03:18 | 144.068 | 163.7 | 135.5 | 131 | | | | | 14% | 6% | 9% | | | | |
| 1996-12-30 05:36 | 153.964 | 132.8 | 109.9 | 107 | 154 | | | | 14% | 29% | 31% | 0% | | | |
| 1996-12-30 08:06 | 151.86 | 105.8 | 87.6 | 85.0 | 123 | | | | 30% | 42% | 44% | 19% | | | |
| 1996-12-30 12:36 | 164.772 | 70.3 | 58.2 | 56.5 | 81.5 | 165 | | | 57% | 65% | 66% | 51% | 0% | | |
| 1996-12-30 17:00 | 121.781 | 47.1 | 39.0 | 37.8 | 54.6 | 110 | 122 | | 61% | 68% | 69% | 55% | 9% | | |
| 1996-12-31 00:42 | 67.932 | 23.4 | 19.4 | 18.8 | 27.1 | 54.8 | 60.5 | | 66% | 71% | 72% | 60% | 19% | 11% | |
| 1996-12-31 08:36 | 43.532 | 11.4 | 9.4 | 9.2 | 13.2 | 26.7 | 29.5 | 43.5 | 74% | 78% | 79% | 70% | 39% | 32% | |
| 1996-12-31 17:48 | 28.179 | 4.9 | 4.1 | 4.0 | 5.7 | 11.6 | 12.8 | 18.9 | 82% | 85% | 86% | 80% | 59% | 55% | 33% |
| 1997-01-01 03:06 | 18.979 | 2.1 | 1.8 | 1.7 | 2.5 | 5.0 | 5.5 | 8.1 | 89% | 91% | 91% | 87% | 74% | 71% | 57% |
| 1997-01-01 15:30 | 12.041 | 0.7 | 0.6 | 0.6 | 0.8 | 1.6 | 1.8 | 2.6 | 94% | 95% | 95% | 93% | 87% | 85% | 78% |
| 1997-01-01 23:18 | 9.521 | 0.6 | 0.5 | 0.5 | 0.7 | 0.8 | 0.9 | 1.3 | 93% | 94% | 95% | 92% | 92% | 91% | 86% |



| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1999-12-25 02:48 | 89.383 | TOF | | | | | | | Error | | | | | Total | 1009% |
| 1999-12-25 03:57 | 110.541 | 110.541 | | | | | | | 0% | | | | | Avg | 54% |
| 1999-12-25 06:12 | 82.196 | 90.1 | | | | | | | 10% | | | | | | |
| 1999-12-25 06:34 | 93.459 | 87.1 | 93.46 | | | | | | 7% | 0% | | | | | |
| 1999-12-25 09:53 | 62.962 | 64.5 | 69.1 | | | | | | 2% | 10% | | | | | |
| 1999-12-25 10:25 | 64.046 | 61.4 | 65.9 | | | | | | 4% | 3% | | | | | |
| 1999-12-25 11:08 | 52.99 | 57.5 | 61.7 | | | | | | 9% | 16% | | | | | |
| 1999-12-25 11:52 | 55.326 | 53.8 | 57.7 | 55.3 | | | | | 3% | 4% | 0% | | | | |
| 1999-12-25 12:45 | 45.926 | 49.7 | 53.3 | 51.1 | | | | | 8% | 16% | 11% | | | | |
| 1999-12-25 16:00 | 29.115 | 37.0 | 39.6 | 38.0 | | | | | 27% | 36% | 31% | | | | |
| 1999-12-25 20:35 | 18.071 | 24.4 | 26.1 | 25.0 | 18.1 | | | | 35% | 45% | 39% | 0% | | | |
| 1999-12-25 21:15 | 18.071 | 22.9 | 24.6 | 23.6 | 17.0 | | | | 27% | 36% | 30% | 6% | | | |
| 1999-12-25 21:49 | 20.96 | 21.8 | 23.4 | 22.4 | 16.2 | | | | 4% | 11% | 7% | 23% | | | |
| 1999-12-25 23:16 | 51.199 | 19.1 | 20.5 | 19.6 | 14.2 | | | | 63% | 60% | 62% | 72% | | | |
| 1999-12-26 00:58 | 58.618 | 16.4 | 17.5 | 16.8 | 12.1 | 58.6 | | | 72% | 70% | 71% | 79% | 0% | | |
| 1999-12-26 03:55 | 51.911 | 12.5 | 13.4 | 12.9 | 9.3 | 44.8 | | | 76% | 74% | 75% | 82% | 14% | | |
| 1999-12-26 09:00 | 33.372 | 7.9 | 8.5 | 8.1 | 5.8 | 28.2 | 33.4 | | 76% | 75% | 76% | 82% | 15% | 0% | |
| 1999-12-26 10:22 | 29.689 | 7.0 | 7.5 | 7.2 | 5.2 | 24.9 | 29.5 | | 77% | 75% | 76% | 83% | 16% | 1% | |
| 1999-12-26 12:23 | 26.727 | 5.8 | 6.2 | 6.0 | 4.3 | 20.8 | 24.5 | | 78% | 77% | 78% | 84% | 22% | 8% | |
| 1999-12-26 16:35 | 24.559 | 4.0 | 4.2 | 4.1 | 2.9 | 14.2 | 16.7 | 24.6 | 84% | 83% | 83% | 88% | 42% | 32% | 0% |
| 1999-12-27 04:14 | 13.567 | 1.4 | 1.5 | 1.4 | 1.0 | 4.9 | 5.8 | 8.5 | 90% | 89% | 90% | 93% | 64% | 57% | 37% |
| 1999-12-27 12:00 | 9.566 | 0.7 | 0.7 | 0.7 | 0.5 | 2.4 | 2.9 | 4.2 | 93% | 92% | 93% | 95% | 75% | 70% | 56% |
| 1999-12-28 09:41 | 3.931 | 0.6 | 0.6 | 0.6 | 0.4 | 0.3 | 0.4 | 0.6 | 86% | 85% | 85% | 90% | 91% | 90% | 85% |
| 1999-12-28 11:29 | 3.611 | 0.5 | 0.6 | 0.6 | 0.4 | 0.3 | 0.4 | 0.6 | 85% | 84% | 84% | 89% | 91% | 89% | 84% |
| 1999-12-28 15:05 | 3.155 | 0.5 | 0.6 | 0.5 | 0.4 | 0.3 | 0.4 | 0.6 | 83% | 82% | 83% | 88% | 90% | 88% | 82% |
| 2002-12-22 20:03 | 28.004 | TOF | | | | | | | Error | | | | | Total | 1161% |
| 2002-12-22 20:08 | 50.831 | 50.831 | | | | | | | 0% | | | | | Avg | 26% |
| 2002-12-22 20:21 | 52.592 | 49.9 | | | | | | | 5% | | | | | | |
| 2002-12-22 20:37 | 51.228 | 48.7 | 51.23 | | | | | | 5% | 0% | | | | | |
| 2002-12-22 21:24 | 39.038 | 45.3 | 47.6 | | | | | | 16% | 22% | | | | | |
| 2002-12-22 21:58 | 32.1 | 43.0 | 45.3 | 32.1 | | | | | 34% | 41% | 0% | | | | |
| 2002-12-22 23:08 | 25.558 | 38.7 | 40.7 | 28.9 | | | | | 51% | 59% | 13% | | | | |
| 2002-12-22 23:54 | 22.198 | 36.1 | 38.0 | 26.9 | | | | | 63% | 71% | 21% | | | | |
| 2002-12-23 00:13 | 22.047 | 35.0 | 36.9 | 26.2 | | | | | 59% | 67% | 19% | | | | |
| 2002-12-23 01:01 | 24.546 | 32.6 | 34.3 | 24.3 | 24.5 | | | | 33% | 40% | 1% | 0% | | | |
| 2002-12-23 01:20 | 24.518 | 31.7 | 33.4 | 23.7 | 23.9 | | | | 29% | 36% | 4% | 3% | | | |
| 2002-12-23 02:05 | 23.216 | 29.6 | 31.2 | 22.1 | 22.3 | | | | 28% | 34% | 5% | 4% | | | |
| 2002-12-23 02:23 | 23.839 | 28.8 | 30.3 | 21.5 | 21.7 | 23.8 | | | 21% | 27% | 10% | 9% | 0% | | |
| 2002-12-23 02:58 | 33.917 | 27.3 | 28.8 | 20.4 | 20.6 | 22.6 | | | 19% | 15% | 40% | 39% | 33% | | |
| 2002-12-23 03:27 | 36.273 | 26.1 | 27.5 | 19.5 | 19.7 | 21.6 | 36.3 | | 28% | 24% | 46% | 46% | 40% | 0% | |
| 2002-12-23 03:58 | 35.75 | 24.9 | 26.3 | 18.6 | 18.8 | 20.7 | 34.6 | | 30% | 27% | 48% | 47% | 42% | 3% | |
| 2002-12-23 04:13 | 32.682 | 24.4 | 25.7 | 18.2 | 18.4 | 20.2 | 33.8 | | 25% | 21% | 44% | 44% | 38% | 4% | |
| 2002-12-23 04:30 | 33.321 | 23.8 | 25.0 | 17.7 | 17.9 | 19.7 | 33.0 | 33.3 | 29% | 25% | 47% | 46% | 41% | 1% | 0% |
| 2002-12-23 07:45 | 19.778 | 17.7 | 18.6 | 13.2 | 13.3 | 14.6 | 24.6 | 24.8 | 11% | 6% | 33% | 33% | 26% | 24% | 25% |
| 2002-12-23 10:53 | 12.041 | 13.3 | 14.0 | 9.9 | 10.0 | 11.0 | 18.4 | 18.6 | 10% | 16% | 18% | 17% | 9% | 53% | 55% |
| 2002-12-23 14:24 | 7.642 | 9.7 | 10.2 | 7.2 | 7.3 | 8.0 | 13.4 | 13.5 | 26% | 33% | 6% | 5% | 5% | 75% | 77% |
| 2002-12-23 17:15 | 5.909 | 7.5 | 7.9 | 5.6 | 5.6 | 6.2 | 10.4 | 10.5 | 26% | 33% | 6% | 5% | 5% | 75% | 77% |
| 2002-12-23 18:32 | 5.691 | 6.6 | 7.0 | 5.0 | 5.0 | 5.5 | 9.2 | 9.3 | 17% | 23% | 13% | 12% | 3% | 62% | 64% |
| 2002-12-23 20:29 | 5.717 | 5.6 | 5.8 | 4.1 | 4.2 | 4.6 | 7.7 | 7.8 | 3% | 2% | 27% | 27% | 19% | 35% | 36% |
| 2002-12-23 23:48 | 4.364 | 4.1 | 4.3 | 3.1 | 3.1 | 3.4 | 5.7 | 5.8 | 6% | 1% | 30% | 29% | 22% | 31% | 32% |
| 2002-12-24 06:56 | 2.411 | 2.1 | 2.3 | 1.6 | 1.6 | 1.8 | 3.0 | 3.0 | 11% | 6% | 33% | 33% | 26% | 24% | 25% |
| 2002-12-24 15:29 | 1.291 | 1.0 | 1.0 | 0.7 | 0.7 | 0.8 | 1.4 | 1.4 | 23% | 19% | 43% | 42% | 37% | 6% | 7% |

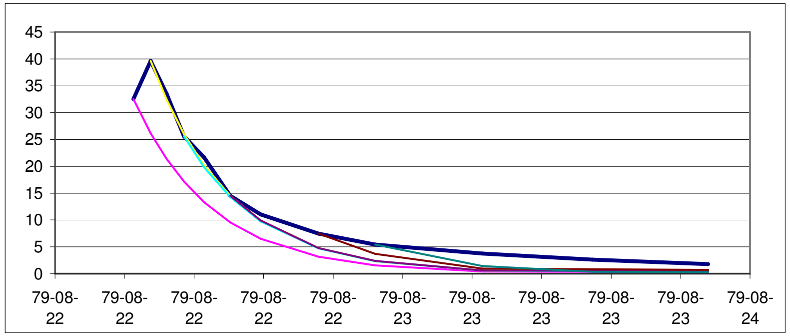


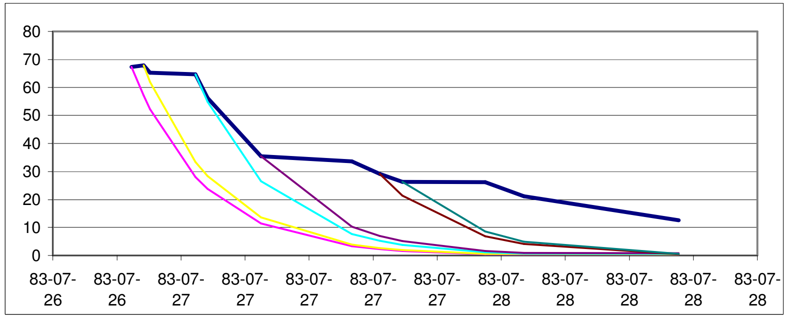



# Calibration of K value for Riet River at Groen Leegte N2H008

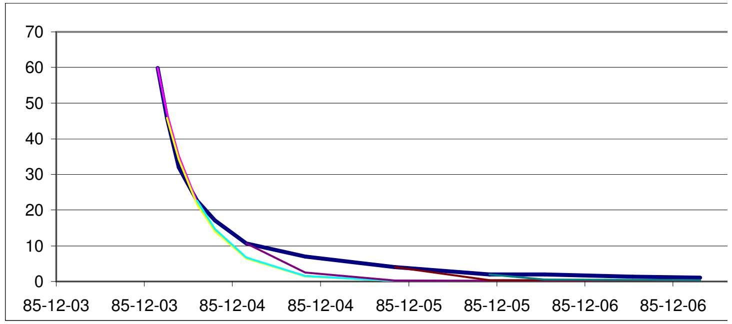
| **Data** | | **Forecast** | | | | **Error Calculation** | | |
|---|---|---|---|---|---|---|---|---|
| **N2H008** | | $B_f$ | 0.001 | Tf | 1.00 | | **Total AVG** | 736% |
| Mode | 1.939 | $K_U$ | 5.48 | $K_L$ | 55 | | | |
| Mean | 16.171 | Basic Forecast Equation is defined as: | | | | | | |
| Min | 0.001 | | | | | | | |
| Max | 71.801 | $q_2 = q_1 \times \exp^{(-dt/k)}$ | | | | | | |

| Date | Discharge | Forecast | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1979-08-22 03:36 | 26.314 | TOF | | | | | | Error | | | | | Total | 2122% |
| 1979-08-22 05:24 | 32.516 | 32.516 | | | | | | 0% | | | | | Avg | 45% |
| 1979-08-22 06:36 | 39.624 | 26.1 | 39.62 | | | | | 34% | 0% | | | | | |
| 1979-08-22 07:42 | 33.472 | 21.4 | 32.4 | | | | | 36% | 3% | | | | | |
| 1979-08-22 08:54 | 25.58 | 17.2 | 26.0 | 25.6 | | | | 33% | 2% | 0% | | | | |
| 1979-08-22 10:18 | 21.676 | 13.3 | 20.2 | 19.8 | | | | 39% | 7% | 9% | | | | |
| 1979-08-22 12:06 | 14.542 | 9.6 | 14.5 | 14.3 | 14.5 | | | 34% | 0% | 2% | 0% | | | |
| 1979-08-22 14:12 | 11.046 | 6.5 | 9.9 | 9.7 | 9.9 | | | 41% | 10% | 12% | 10% | | | |
| 1979-08-22 18:12 | 7.48 | 3.1 | 4.8 | 4.7 | 4.8 | 7.48 | | 58% | 36% | 37% | 36% | 0% | | |
| 1979-08-22 22:06 | 5.429 | 1.5 | 2.3 | 2.3 | 2.3 | 3.7 | 5.43 | 72% | 57% | 58% | 57% | 32% | | |
| 1979-08-23 05:30 | 3.793 | 0.4 | 0.6 | 0.6 | 0.6 | 1.0 | 1.4 | 89% | 84% | 84% | 84% | 75% | | 63% |
| 1979-08-23 13:06 | 2.579 | 0.3 | 0.5 | 0.5 | 0.5 | 0.8 | 0.4 | 86% | 79% | 80% | 79% | 68% | | 86% |
| 1979-08-23 21:06 | 1.769 | 0.3 | 0.5 | 0.4 | 0.5 | 0.7 | 0.3 | 83% | 74% | 75% | 74% | 60% | | 83% |
| | | Forecast | | | | | | | | | | | | |
| 1983-07-26 20:00 | 71.801 | TOF | | | | | | Error | | | | | Total | 3090% |
| 1983-07-26 20:18 | 67.372 | 67.372 | | | | | | 0% | | | | | Avg | 64% |
| 1983-07-26 21:12 | 67.916 | 57.2 | 67.92 | | | | | 16% | 0% | | | | | |
| 1983-07-26 21:42 | 65.232 | 52.2 | 62.0 | | | | | 20% | 5% | | | | | |
| 1983-07-27 01:06 | 64.701 | 28.1 | 33.3 | 64.7 | | | | 57% | 48% | 0% | | | | |
| 1983-07-27 02:00 | 56.22 | 23.8 | 28.3 | 54.9 | | | | 58% | 50% | 2% | | | | |
| 1983-07-27 06:00 | 35.497 | 11.5 | 13.6 | 26.5 | 35.5 | | | 68% | 62% | 25% | 0% | | | |
| 1983-07-27 12:48 | 33.62 | 3.3 | 3.9 | 7.7 | 10.3 | | | 90% | 88% | 77% | 69% | | | |
| 1983-07-27 14:54 | 29.084 | 2.3 | 2.7 | 5.2 | 7.0 | 29.1 | | 92% | 91% | 82% | 76% | 0% | | |
| 1983-07-27 16:36 | 26.356 | 1.7 | 2.0 | 3.8 | 5.1 | 21.3 | 26.4 | 94% | 93% | 85% | 81% | 19% | | 0% |
| 1983-07-27 22:48 | 26.109 | 0.5 | 0.6 | 1.2 | 1.7 | 6.9 | 8.5 | 98% | 98% | 95% | 94% | 74% | | 67% |
| 1983-07-28 01:42 | 21.139 | 0.5 | 0.6 | 0.7 | 1.0 | 4.1 | 5.0 | 98% | 97% | 97% | 95% | 81% | | 76% |
| 1983-07-28 13:18 | 12.675 | 0.4 | 0.5 | 0.6 | 0.8 | 0.5 | 0.6 | 97% | 96% | 95% | 94% | 96% | | 95% |

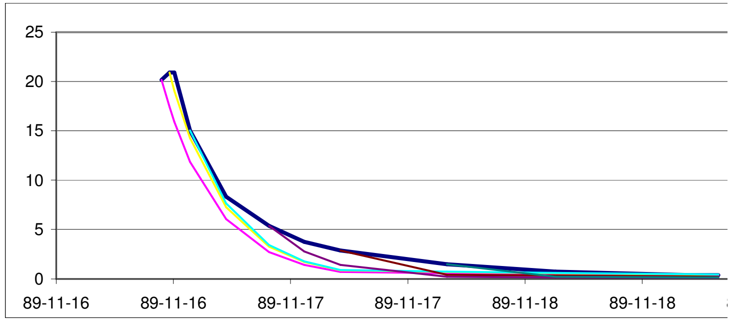


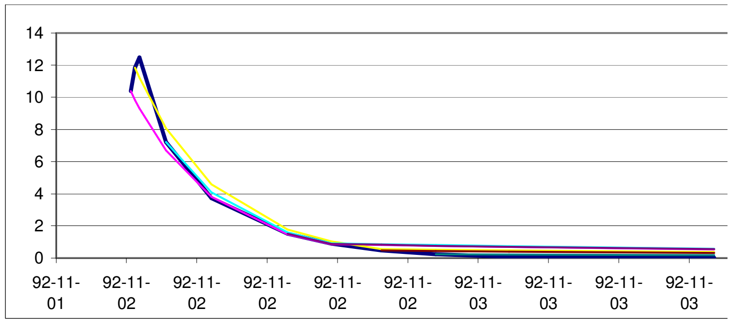
| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Forecast | | | | | | | | | | | | |
| 1985-12-03 10:06 | 23.042 | TOF | | | | | | Error | | | | | Total | 2880% |
| 1985-12-03 13:48 | 59.861 | 59.861 | | | | | | 0% | | | | | Avg | 61% |
| 1985-12-03 15:06 | 45.833 | 47.2 | 45.83 | | | | | 3% | 0% | | | | | |
| 1985-12-03 16:42 | 31.948 | 35.3 | 34.2 | | | | | 10% | 7% | | | | | |
| 1985-12-03 19:12 | 22.667 | 22.4 | 21.7 | 22.7 | | | | 1% | 4% | 0% | | | | |
| 1985-12-03 21:36 | 17.082 | 14.4 | 14.0 | 14.6 | | | | 16% | 18% | 14% | | | | |
| 1985-12-04 01:54 | 10.673 | 6.6 | 6.4 | 6.7 | 10.7 | | | 38% | 40% | 37% | 0% | | | |
| 1985-12-04 09:54 | 6.961 | 1.5 | 1.5 | 1.6 | 2.5 | | | 78% | 79% | 78% | 64% | | | |
| 1985-12-04 22:06 | 4.034 | 0.2 | 0.2 | 0.2 | 0.3 | 4.03 | | 96% | 96% | 96% | 93% | 0% | | |
| 1985-12-05 11:00 | 1.939 | 0.1 | 0.1 | 0.1 | 0.2 | 0.4 | 1.94 | 93% | 93% | 93% | 89% | 80% | | |
| 1985-12-05 18:30 | 1.939 | 0.1 | 0.1 | 0.1 | 0.2 | 0.3 | 0.5 | 94% | 94% | 94% | 90% | 83% | | 75% |
| 1985-12-06 06:30 | 1.342 | 0.1 | 0.1 | 0.1 | 0.1 | 0.3 | 0.4 | 93% | 93% | 93% | 89% | 80% | | 70% |
| 1985-12-06 15:42 | 1.032 | 0.1 | 0.1 | 0.1 | 0.1 | 0.2 | 0.3 | 93% | 93% | 92% | 88% | 78% | | 68% |



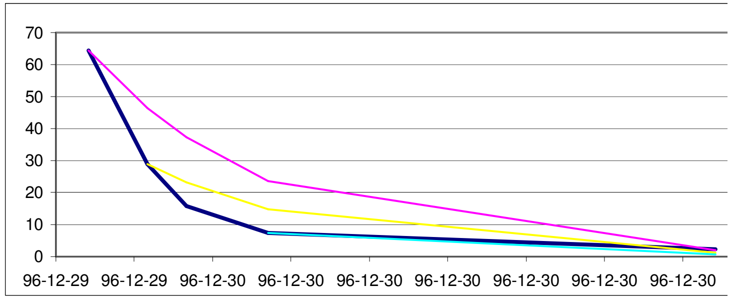

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Forecast | | | | | | | | | | | | |
| 1989-11-16 08:36 | 13.365 | TOF | | | | | | Error | | | | | Total | 1729% |
| 1989-11-16 10:48 | 20.159 | 20.159 | | | | | | 0% | | | | | Avg | 37% |
| 1989-11-16 11:36 | 20.891 | 17.4 | 20.89 | | | | | 17% | 0% | | | | | |
| 1989-11-16 12:06 | 20.891 | 15.9 | 19.1 | | | | | 24% | 9% | | | | | |
| 1989-11-16 13:42 | 15.016 | 11.9 | 14.2 | 15 | | | | 21% | 5% | 0% | | | | |
| 1989-11-16 17:24 | 8.341 | 6.0 | 7.3 | 7.6 | | | | 28% | 13% | 8% | | | | |
| 1989-11-16 21:48 | 5.352 | 2.7 | 3.2 | 3.4 | 5.35 | | | 49% | 39% | 36% | 0% | | | |
| 1989-11-17 01:24 | 3.772 | 1.4 | 1.7 | 1.8 | 2.8 | | | 63% | 55% | 53% | 26% | | | |
| 1989-11-17 05:06 | 2.872 | 0.7 | 0.9 | 0.9 | 1.4 | 2.87 | | 75% | 70% | 69% | 51% | 0% | | |
| 1989-11-17 16:00 | 1.478 | 0.6 | 0.7 | 0.7 | 0.2 | 0.4 | 1.48 | 60% | 52% | 50% | 87% | 73% | | |
| 1989-11-18 03:06 | 0.74 | 0.5 | 0.6 | 0.6 | 0.2 | 0.3 | 0.2 | 35% | 22% | 18% | 79% | 57% | | 74% |
| 1989-11-18 16:54 | 0.378 | 0.4 | 0.4 | 0.5 | 0.1 | 0.2 | 0.2 | 2% | 18% | 25% | 68% | 34% | | 60% |
| 1989-11-18 19:48 | 0.386 | 0.4 | 0.4 | 0.4 | 0.1 | 0.2 | 0.1 | 9% | 10% | 16% | 70% | 39% | | 63% |



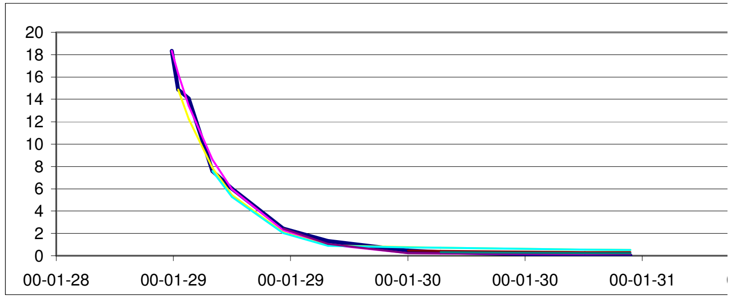

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Forecast | | | | | | | | | | | | |
| 1992-11-01 23:24 | 9.912 | TOF | | | | | | Error | | | | | Total | 15221% |
| 1992-11-02 00:18 | 10.38 | 10.380 | | | | | | 0% | | | | | Avg | 371% |
| 1992-11-02 00:36 | 11.847 | 9.8 | 11.85 | | | | | 17% | 0% | | | | | |
| 1992-11-02 00:54 | 12.475 | 9.3 | 11.2 | | | | | 25% | 10% | | | | | |
| 1992-11-02 02:42 | 7.202 | 6.7 | 8.1 | 7.2 | | | | 7% | 12% | 0% | | | | |
| 1992-11-02 05:48 | 3.73 | 3.8 | 4.6 | 4.1 | | | | 2% | 23% | 10% | | | | |
| 1992-11-02 11:00 | 1.506 | 1.5 | 1.8 | 1.6 | 1.51 | | | 2% | 18% | 5% | 0% | | | |
| 1992-11-02 14:00 | 0.881 | 0.9 | 1.0 | 0.9 | 0.9 | | | 3% | 17% | 4% | 1% | | | |
| 1992-11-02 17:24 | 0.464 | 0.8 | 0.6 | 0.9 | 0.8 | 0.46 | | 73% | 19% | 86% | 76% | 0% | | |
| 1992-11-02 21:06 | 0.235 | 0.7 | 0.5 | 0.8 | 0.8 | 0.4 | 0.24 | 219% | 120% | 243% | 226% | 85% | | |
| 1992-11-03 02:06 | 0.083 | 0.7 | 0.5 | 0.7 | 0.7 | 0.4 | 0.2 | 724% | 468% | 785% | 742% | 377% | | 158% |
| 1992-11-03 16:06 | 0.022 | 0.5 | 0.4 | 0.6 | 0.5 | 0.3 | 0.2 | 2307% | 1560% | 2488% | 2360% | 1294% | | 655% |



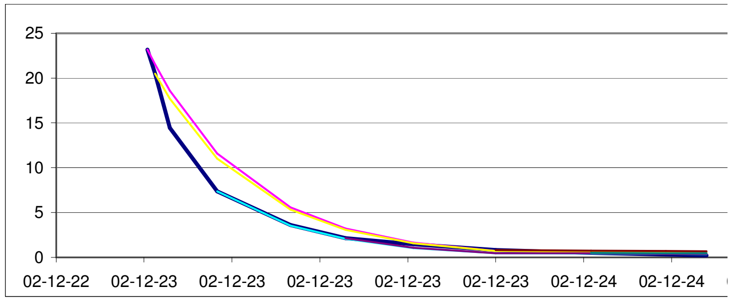
| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Forecast | | | | | | | | | | | |
| 1996-12-29 19:06 | 43.603 | TOF | | | | | | Error | | | | Total | 695% |
| 1996-12-29 20:12 | 64.399 | 64.399 | | | | | | 0% | | | | Avg | 63% |
| 1996-12-29 22:00 | 28.864 | 46.4 | 28.86 | | | | | 61% | 0% | | | | |
| 1996-12-29 23:12 | 15.809 | 37.3 | 23.2 | | | | | 136% | 47% | | | | |
| 1996-12-30 01:42 | 7.418 | 23.6 | 14.7 | 7.42 | | | | 218% | 98% | 0% | | | |
| 1996-12-30 15:24 | 2.282 | 1.9 | 1.2 | 0.6 | | | | 15% | 47% | 73% | | | |




| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Forecast | | | | | | | | | | | |
| 2000-01-28 23:02 | 6.697 | TOF | | | | | | Error | | | | Total | 24742% |
| 2000-01-28 23:51 | 18.355 | 18.355 | | | | | | 0% | | | | Avg | 603% |
| 2000-01-29 00:30 | 14.886 | 16.3 | 14.89 | | | | | 10% | 0% | | | | |
| 2000-01-29 01:34 | 14.073 | 13.4 | 12.3 | | | | | 5% | 13% | | | | |
| 2000-01-29 04:00 | 7.604 | 8.6 | 7.9 | 7.6 | | | | 13% | 3% | 0% | | | |
| 2000-01-29 06:01 | 6.007 | 6.0 | 5.4 | 5.3 | | | | 1% | 9% | 12% | | | |
| 2000-01-29 11:14 | 2.455 | 2.3 | 2.1 | 2.0 | 2.46 | | | 6% | 14% | 17% | 0% | | |
| 2000-01-29 15:48 | 1.328 | 1.0 | 0.9 | 0.9 | 1.1 | | | 25% | 31% | 33% | 20% | | |
| 2000-01-30 00:02 | 0.464 | 0.2 | 0.8 | 0.8 | 0.2 | 0.46 | | 52% | 69% | 64% | 49% | 0% | |
| 2000-01-30 03:20 | 0.321 | 0.2 | 0.7 | 0.7 | 0.2 | 0.4 | 0.32 | 35% | 130% | 123% | 30% | 36% | |
| 2000-01-30 18:09 | 0.017 | 0.2 | 0.6 | 0.5 | 0.2 | 0.3 | 0.2 | 841% | 3221% | 3113% | 904% | 1861% | 1341% |
| 2000-01-30 22:47 | 0.014 | 0.1 | 0.5 | 0.5 | 0.2 | 0.3 | 0.2 | 950% | 3606% | 3485% | 1021% | 2088% | 1508% |




| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Forecast | | | | | | | | | | | |
| 2002-12-23 00:00 | 22.967 | TOF | | | | | | Error | | | | Total | 1882% |
| 2002-12-23 00:12 | 23.154 | 23.154 | | | | | | 0% | | | | Avg | 46% |
| 2002-12-23 00:36 | 20.498 | 21.5 | 20.5 | | | | | 5% | 0% | | | | |
| 2002-12-23 01:24 | 14.444 | 18.6 | 17.7 | | | | | 29% | 23% | | | | |
| 2002-12-23 04:00 | 7.351 | 11.6 | 11.0 | 7.35 | | | | 57% | 50% | 0% | | | |
| 2002-12-23 08:00 | 3.585 | 5.6 | 5.3 | 3.5 | | | | 56% | 48% | 1% | | | |
| 2002-12-23 11:00 | 2.134 | 3.2 | 3.1 | 2.0 | 2.13 | | | 51% | 44% | 4% | 0% | | |
| 2002-12-23 14:42 | 1.442 | 1.6 | 1.6 | 1.0 | 1.1 | | | 14% | 9% | 28% | 25% | | |
| 2002-12-23 19:12 | 0.823 | 0.7 | 0.7 | 0.5 | 0.5 | 0.82 | | 12% | 16% | 44% | 42% | 0% | |
| 2002-12-24 00:24 | 0.488 | 0.7 | 0.6 | 0.4 | 0.4 | 0.7 | 0.49 | 35% | 28% | 14% | 11% | 53% | |
| 2002-12-24 04:30 | 0.32 | 0.6 | 0.6 | 0.4 | 0.4 | 0.7 | 0.5 | 91% | 82% | 21% | 26% | 117% | 42% |
| 2002-12-24 06:42 | 0.214 | 0.6 | 0.6 | 0.4 | 0.4 | 0.7 | 0.4 | 174% | 161% | 74% | 81% | 212% | 103% |



# Calibration of K value for Klein Brak River at Zevenfontein Q1H013

## Data

**Q1H013**

| | |
|---|---|
| Mode | 0.003 |
| Mean | 12.947 |
| Min | 0.000 |
| Max | 130.586 |

## Forecast

| | | | |
|---|---|---|---|
| $B_f$ | 0.001 | Tf | 1.00 |
| $K_U$ | 4.65 | $K_L$ | 47 |

Basic Forecast Equation is defined as:

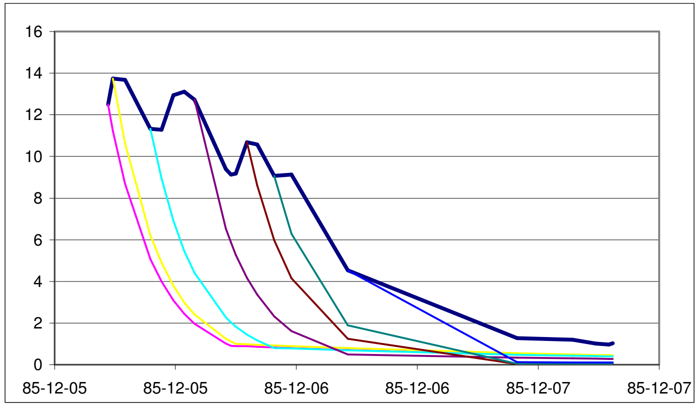
$$q_2 = q_1 \times \exp^{(-dt/k)}$$

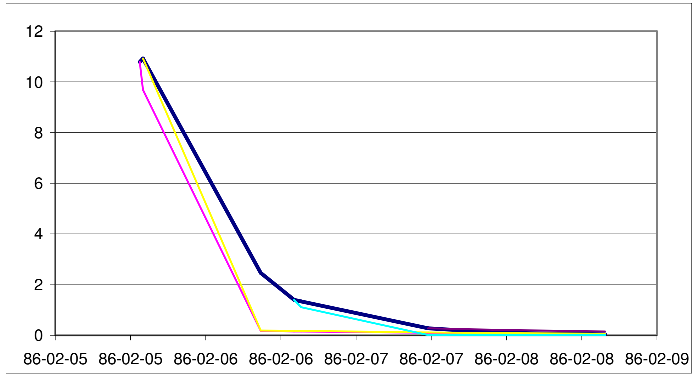
## Error Calculation

**Total AVG** 94%

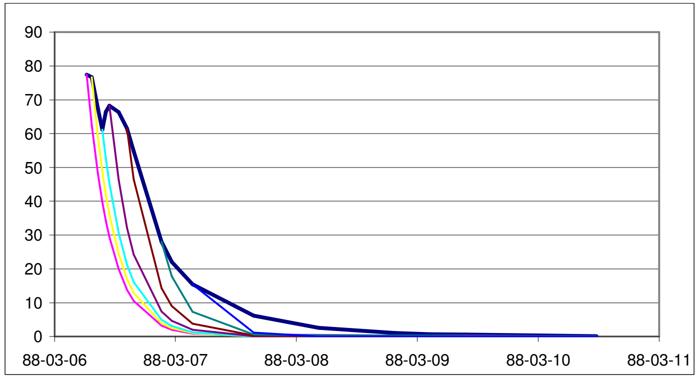
## Plots of Data vs Forecast

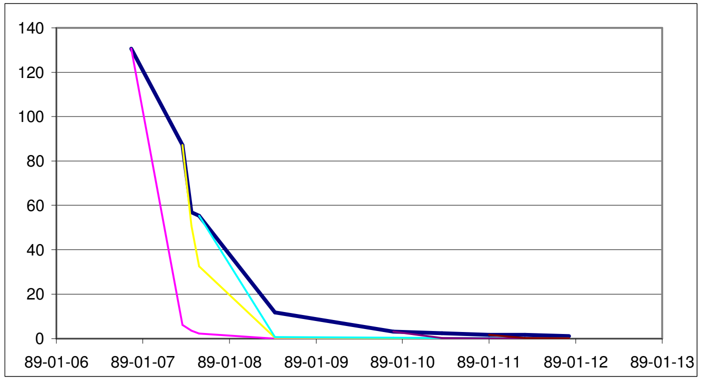
| Date | Discharge | Forecast | | | | | | | Error | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1985-12-05 04:12 | 4.869 | TOF | | | | | | | Error | | | | | Total | 6491% |
| 1985-12-05 05:18 | 12.504 | 12.504 | | | | | | | 0% | | | | | Avg | 67% |
| 1985-12-05 05:48 | 13.75 | 11.2 | 13.75 | | | | | | 18% | 0% | | | | | |
| 1985-12-05 07:00 | 13.692 | 8.7 | 10.6 | | | | | | 37% | 22% | | | | | |
| 1985-12-05 09:30 | 11.331 | 5.1 | 6.2 | 11.3 | | | | | 55% | 45% | 0% | | | | |
| 1985-12-05 10:36 | 11.279 | 4.0 | 4.9 | 8.9 | | | | | 65% | 57% | 21% | | | | |
| 1985-12-05 11:48 | 12.948 | 3.1 | 3.8 | 6.9 | | | | | 76% | 71% | 47% | | | | |
| 1985-12-05 12:54 | 13.117 | 2.4 | 3.0 | 5.5 | | | | | 81% | 77% | 58% | | | | |
| 1985-12-05 13:54 | 12.725 | 2.0 | 2.4 | 4.4 | 12.7 | | | | 85% | 81% | 65% | 0% | | | |
| 1985-12-05 17:00 | 9.388 | 1.0 | 1.2 | 2.3 | 6.5 | | | | 89% | 87% | 76% | 30% | | | |
| 1985-12-05 17:30 | 9.138 | 0.9 | 1.1 | 2.0 | 5.9 | | | | 90% | 88% | 78% | 36% | | | |
| 1985-12-05 18:00 | 9.179 | 0.9 | 1.0 | 1.8 | 5.3 | | | | 90% | 89% | 80% | 43% | | | |
| 1985-12-05 19:06 | 10.676 | 0.9 | 1.0 | 1.4 | 4.2 | 10.7 | | | 92% | 91% | 87% | 61% | 0% | | |
| 1985-12-05 20:06 | 10.579 | 0.9 | 1.0 | 1.2 | 3.4 | 8.6 | | | 92% | 91% | 89% | 68% | 19% | | |
| 1985-12-05 21:48 | 9.059 | 0.8 | 0.9 | 0.8 | 2.3 | 6.0 | 9.06 | | 91% | 90% | 91% | 74% | 34% | 0% | |
| 1985-12-05 23:30 | 9.138 | 0.8 | 0.9 | 0.8 | 1.6 | 4.1 | 6.3 | | 91% | 90% | 92% | 82% | 55% | 31% | |
| 1985-12-06 05:06 | 4.528 | 0.7 | 0.8 | 0.7 | 0.5 | 1.2 | 1.9 | 4.53 | 84% | 83% | 85% | 89% | 73% | 58% | 0% |
| 1985-12-06 21:54 | 1.276 | 0.5 | 0.5 | 0.5 | 0.3 | 0.0 | 0.1 | 0.1 | 61% | 57% | 62% | 74% | 97% | 96% | 90% |
| 1985-12-07 03:24 | 1.199 | 0.4 | 0.5 | 0.4 | 0.3 | 0.0 | 0.0 | 0.1 | 63% | 59% | 64% | 75% | 98% | 96% | 91% |
| 1985-12-07 05:42 | 1.007 | 0.4 | 0.5 | 0.4 | 0.3 | 0.0 | 0.0 | 0.1 | 59% | 54% | 60% | 72% | 97% | 96% | 90% |
| 1985-12-07 07:00 | 0.969 | 0.4 | 0.5 | 0.4 | 0.3 | 0.0 | 0.0 | 0.1 | 58% | 53% | 59% | 71% | 97% | 96% | 90% |
| 1985-12-07 07:24 | 1.022 | 0.4 | 0.4 | 0.4 | 0.3 | 0.0 | 0.0 | 0.1 | 61% | 56% | 62% | 73% | 97% | 96% | 90% |
| 1986-02-04 07:42 | 0.01 | | | | | | | | | | | | | | |
| 1986-02-05 12:12 | 0.007 | | | | | | | | | | | | | | |
| 1986-02-05 13:06 | 9.179 | TOF | | | | | | | Error | | | | | Total | 1479% |
| 1986-02-05 13:30 | 10.776 | 10.776 | | | | | | | 0% | | | | | Avg | 64% |
| 1986-02-05 14:00 | 10.925 | 9.7 | 10.93 | | | | | | 11% | 0% | | | | | |
| 1986-02-06 08:48 | 2.453 | 0.2 | 0.2 | | | | | | 93% | 92% | | | | | |
| 1986-02-06 14:06 | 1.402 | 0.2 | 0.2 | 1.4 | | | | | 89% | 88% | 0% | | | | |
| 1986-02-06 15:12 | 1.338 | 0.1 | 0.2 | 1.1 | | | | | 89% | 87% | 17% | | | | |
| 1986-02-07 11:30 | 0.273 | 0.1 | 0.1 | 0.0 | 0.27 | | | | 65% | 60% | 95% | 0% | | | |
| 1986-02-07 22:48 | 0.098 | 0.1 | 0.1 | 0.0 | 0.2 | | | | 23% | 13% | 89% | 118% | | | |
| 1986-02-08 15:42 | 0.039 | 0.1 | 0.1 | 0.0 | 0.1 | | | | 34% | 51% | 80% | 282% | | | |
| 1986-02-10 08:30 | 0.009 | | | | | | | | | | | | | | |
| 1986-02-12 11:42 | 0.004 | | | | | | | | | | | | | | |
| 1986-02-15 22:00 | 0.003 | | | | | | | | | | | | | | |
| 1986-02-19 11:42 | 0.003 | | | | | | | | | | | | | | |
| 1986-02-23 10:18 | 0.002 | | | | | | | | | | | | | | |
| 1986-02-23 12:30 | 0.003 | | | | | | | | | | | | | | |
| 1988-03-06 02:18 | 0.024 | | | | | | | | | | | | | | |



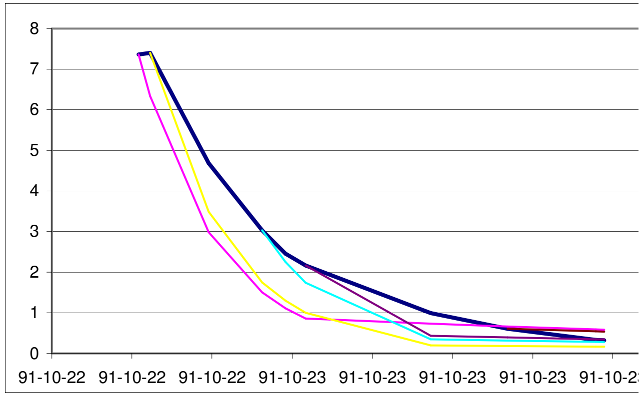



| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1988-03-06 04:36 | 0.021 | TOF | | | | | | | Error | | | | | Total | 3731% |
| 1988-03-06 06:24 | 77.447 | 77.447 | | | | | | | 0% | | | | | Avg | 66% |
| 1988-03-06 07:24 | 76.604 | 62.5 | 76.6 | | | | | | 18% | 0% | | | | | |
| 1988-03-06 08:36 | 67.479 | 48.3 | 59.2 | | | | | | 28% | 12% | | | | | |
| 1988-03-06 09:30 | 61.197 | 39.8 | 48.8 | 61.2 | | | | | 35% | 20% | 0% | | | | |
| 1988-03-06 10:12 | 66.451 | 34.2 | 42.0 | 52.6 | | | | | 49% | 37% | 21% | | | | |
| 1988-03-06 10:54 | 68.283 | 29.4 | 36.1 | 45.3 | 68.3 | | | | 57% | 47% | 34% | 0% | | | |
| 1988-03-06 12:42 | 66.34 | 20.0 | 24.5 | 30.8 | 46.4 | | | | 70% | 63% | 54% | 30% | | | |
| 1988-03-06 14:24 | 61.533 | 13.9 | 17.0 | 21.3 | 32.2 | 61.5 | | | 77% | 72% | 65% | 48% | 0% | | |
| 1988-03-06 15:42 | 54.84 | 10.5 | 12.9 | 16.1 | 24.3 | 46.5 | | | 81% | 77% | 71% | 56% | 15% | | |
| 1988-03-06 21:12 | 28.02 | 3.2 | 3.9 | 4.9 | 7.5 | 14.3 | 28 | | 89% | 86% | 82% | 73% | 49% | 0% | |
| 1988-03-06 23:18 | 22.075 | 2.0 | 2.5 | 3.2 | 4.8 | 9.1 | 17.8 | | 91% | 89% | 86% | 78% | 59% | 19% | |
| 1988-03-07 03:24 | 15.495 | 0.8 | 1.0 | 1.3 | 2.0 | 3.8 | 7.4 | 15.5 | 95% | 93% | 92% | 87% | 76% | 52% | 0% |
| 1988-03-07 15:30 | 6.229 | 0.7 | 0.1 | 0.1 | 0.1 | 0.3 | 0.5 | 1.1 | 89% | 99% | 98% | 98% | 96% | 91% | 82% |
| 1988-03-08 04:24 | 2.679 | 0.5 | 0.1 | 0.1 | 0.1 | 0.2 | 0.4 | 0.1 | 81% | 98% | 97% | 96% | 92% | 84% | 97% |
| 1988-03-08 19:30 | 1.108 | 0.4 | 0.0 | 0.1 | 0.1 | 0.2 | 0.3 | 0.1 | 68% | 96% | 95% | 93% | 86% | 73% | 95% |
| 1988-03-09 02:30 | 0.7 | 0.3 | 0.0 | 0.0 | 0.1 | 0.1 | 0.3 | 0.0 | 56% | 95% | 93% | 90% | 81% | 63% | 94% |
| 1988-03-09 09:42 | 0.601 | 0.3 | 0.0 | 0.0 | 0.1 | 0.1 | 0.2 | 0.0 | 56% | 95% | 93% | 90% | 81% | 63% | 94% |
| 1988-03-10 11:30 | 0.167 | 0.2 | 0.0 | 0.0 | 0.0 | 0.1 | 0.1 | 0.0 | 9% | 89% | 87% | 80% | 61% | 24% | 87% |
| 1989-01-05 23:18 | 0.04 | | | | | | | | | | | | | | |
| 1989-01-05 23:54 | 0.573 | TOF | | | | | | | Error | | | | | Total | 2595% |
| 1989-01-06 20:48 | 130.586 | 130.586 | | | | | | | 0% | | | | | Avg | 76% |
| 1989-01-07 11:00 | 87.153 | 6.2 | 87.15 | | | | | | 93% | 0% | | | | | |
| 1989-01-07 13:30 | 56.858 | 3.6 | 50.9 | | | | | | 94% | 10% | | | | | |
| 1989-01-07 15:36 | 55.368 | 2.3 | 32.4 | 55.4 | | | | | 96% | 41% | 0% | | | | |
| 1989-01-08 12:36 | 11.748 | 0.0 | 0.4 | 0.6 | | | | | 100% | 97% | 95% | | | | |
| 1989-01-09 21:24 | 3.138 | 0.0 | 0.2 | 0.3 | 3.14 | | | | 100% | 94% | 90% | 0% | | | |
| 1989-01-10 11:00 | 2.376 | 0.0 | 0.1 | 0.2 | 0.2 | | | | 100% | 94% | 91% | 93% | | | |
| 1989-01-11 00:00 | 1.676 | 0.0 | 0.1 | 0.2 | 0.1 | 1.68 | | | 100% | 94% | 90% | 92% | 0% | | |
| 1989-01-11 09:54 | 1.605 | 0.0 | 0.1 | 0.1 | 0.1 | 0.2 | | | 100% | 95% | 91% | 94% | 88% | | |
| 1989-01-11 22:06 | 1.108 | 0.0 | 0.1 | 0.1 | 0.1 | 0.2 | | | 100% | 94% | 91% | 93% | 86% | | |
| 1989-01-13 19:36 | 0.634 | | | | | | | | | | | | | | |
| 1989-01-14 22:36 | 0.402 | | | | | | | | | | | | | | |
| 1989-01-16 11:06 | 0.36 | | | | | | | | | | | | | | |
| 1991-10-22 13:42 | 0.039 | | | | | | | | | | | | | | |
| 1991-10-22 14:06 | 4.446 | | | | | | | | | | | | | | |

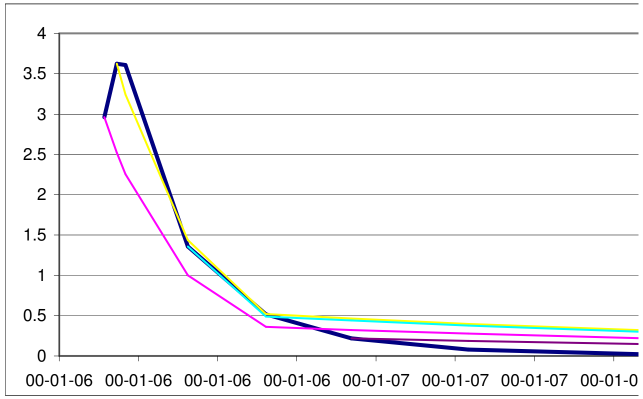


| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1991-10-22 14:30 | 6.878 | TOF | | | | | Error | | | | | Total | 1238% |
| 1991-10-22 14:48 | 7.358 | 7.358 | | | | | 0% | | | | | Avg | 36% |
| 1991-10-22 15:30 | 7.393 | 6.3 | 7.393 | | | | 14% | 0% | | | | | |
| 1991-10-22 19:00 | 4.679 | 3.0 | 3.5 | | | | 36% | 26% | | | | | |
| 1991-10-22 22:12 | 3.034 | 1.5 | 1.8 | 3.03 | | | 51% | 42% | 0% | | | | |
| 1991-10-22 23:36 | 2.453 | 1.1 | 1.3 | 2.2 | | | 55% | 47% | 8% | | | | |
| 1991-10-23 00:48 | 2.166 | 0.9 | 1.0 | 1.7 | 2.17 | | 60% | 54% | 20% | 0% | | | |
| 1991-10-23 08:18 | 0.992 | 0.7 | 0.2 | 0.3 | 0.4 | | 26% | 80% | 65% | 56% | | | |
| 1991-10-23 12:54 | 0.606 | 0.7 | 0.2 | 0.3 | 0.4 | 0.61 | 9% | 70% | 48% | 35% | 0% | | |
| 1991-10-23 18:12 | 0.328 | 0.6 | 0.2 | 0.3 | 0.3 | 0.5 | 80% | 51% | 15% | 6% | 65% | | |
| 1991-10-23 18:42 | 0.324 | 0.6 | 0.2 | 0.3 | 0.3 | 0.5 | 80% | 51% | 15% | 7% | 65% | | |
| 1991-10-23 19:54 | 0.675 | | | | | | | | | | | | |
| 1991-10-23 20:30 | 0.788 | | | | | | | | | | | | |
| 1991-10-23 21:06 | 0.828 | | | | | | | | | | | | |
| 1991-10-23 22:12 | 0.828 | | | | | | | | | | | | |
| 1991-10-24 01:00 | 0.675 | | | | | | | | | | | | |




| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2000-01-06 06:56 | 0 | TOF | | | | Error | | | | Total | 5909% |
| 2000-01-06 07:33 | 2.961 | 2.961 | | | | 0% | | | | Avg | 257% |
| 2000-01-06 08:18 | 3.624 | 2.5 | 3.624 | | | 30% | 0% | | | | |
| 2000-01-06 08:49 | 3.608 | 2.3 | 3.2 | | | 37% | 10% | | | | |
| 2000-01-06 12:36 | 1.356 | 1.0 | 1.4 | 1.36 | | 26% | 6% | 0% | | | |
| 2000-01-06 17:20 | 0.515 | 0.4 | 0.5 | 0.5 | | 30% | 1% | 5% | | | |
| 2000-01-06 22:31 | 0.218 | 0.3 | 0.5 | 0.4 | 0.22 | 48% | 113% | 101% | 0% | | |
| 2000-01-07 05:35 | 0.077 | 0.3 | 0.4 | 0.4 | 0.2 | 261% | 419% | 389% | 143% | | |
| 2000-01-07 16:22 | 0.021 | 0.2 | 0.3 | 0.3 | 0.1 | 949% | 1409% | 1323% | 607% | | |

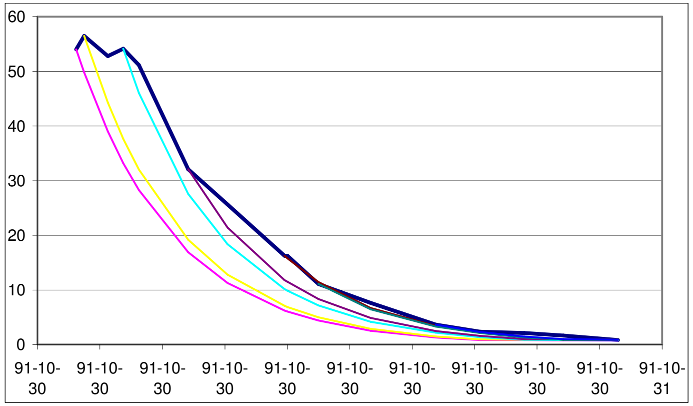


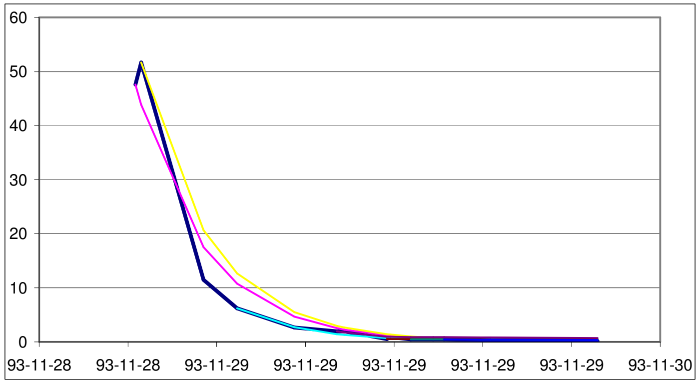
# Calibration of K value for Groot Vis River at Soutpansdrift Q2H002

| Data | | Forecast | | | | Error Calculation | | |
|---|---|---|---|---|---|---|---|---|
| **Q2H002** | | $B_f$ | 0.100 | Tf | 1.00 | | **Total AVG** | 54% |
| Mode | 45.79 | $K_U$ | 3.70 | $K_L$ | 37 | | | |
| Mean | 29.990 | Basic Forecast Equation is defined as: | | | | | | |
| Min | 0.053 | | | | | | | |
| Max | 185.785 | $q_2 = q_1 \times \exp^{(-dt/k)}$ | | | | | | |

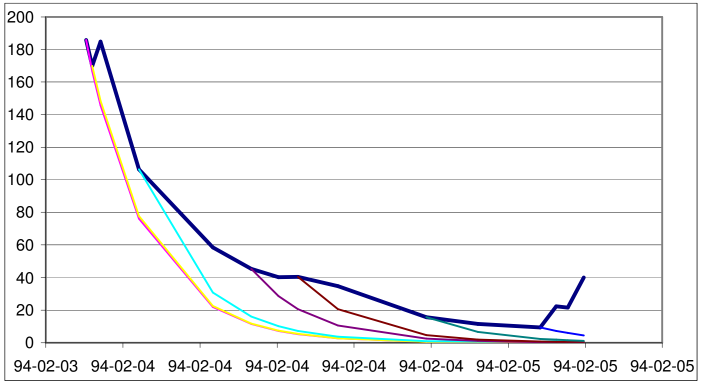
| DATE | Discharge | Forecast | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1991-10-30 00:54 | 55.168 | TOF | | | | | | | Error | | | | | | Total | 6882% |
| 1991-10-30 01:30 | 53.996 | 53.996 | | | | | | | 0% | | | | | | Avg | 32% |
| 1991-10-30 01:48 | 56.493 | 49.8 | 56.49 | | | | | | 12% | 0% | | | | | | |
| 1991-10-30 02:42 | 52.835 | 39.0 | 44.3 | | | | | | 26% | 16% | | | | | | |
| 1991-10-30 03:18 | 54.138 | 33.2 | 37.7 | 54.1 | | | | | 39% | 30% | 0% | | | | | |
| 1991-10-30 03:54 | 51.112 | 28.2 | 32.0 | 46.0 | | | | | 45% | 37% | 10% | | | | | |
| 1991-10-30 05:48 | 32.054 | 16.9 | 19.2 | 27.5 | 32.1 | | | | 47% | 40% | 14% | 0% | | | | |
| 1991-10-30 07:18 | 25.637 | 11.3 | 12.8 | 18.4 | 21.4 | | | | 56% | 50% | 28% | 17% | | | | |
| 1991-10-30 09:30 | 16.236 | 6.2 | 7.1 | 10.1 | 11.8 | 16.2 | | | 62% | 57% | 38% | 27% | 0% | | | |
| 1991-10-30 09:36 | 16.236 | 6.1 | 6.9 | 9.9 | 11.5 | 15.8 | | | 63% | 58% | 39% | 29% | 3% | | | |
| 1991-10-30 10:48 | 11.043 | 4.4 | 5.0 | 7.1 | 8.3 | 11.4 | 11 | | 60% | 55% | 35% | 25% | 3% | 0% | | |
| 1991-10-30 12:48 | 7.63 | 2.5 | 2.9 | 4.2 | 4.8 | 6.7 | 6.4 | | 67% | 62% | 46% | 37% | 13% | 16% | | |
| 1991-10-30 15:18 | 3.595 | 1.3 | 1.5 | 2.1 | 2.5 | 3.4 | 3.3 | 3.6 | 64% | 59% | 41% | 32% | 6% | 9% | | 0% |
| 1991-10-30 17:00 | 2.353 | 0.8 | 0.9 | 1.3 | 1.6 | 2.1 | 2.1 | 2.3 | 65% | 61% | 43% | 34% | 9% | 12% | | 3% |
| 1991-10-30 18:42 | 2.083 | 0.8 | 0.9 | 0.8 | 1.0 | 1.4 | 1.3 | 1.4 | 62% | 57% | 59% | 53% | 35% | 37% | | 31% |
| 1991-10-30 20:12 | 1.605 | 0.8 | 0.9 | 0.8 | 0.9 | 0.9 | 0.9 | 1.0 | 53% | 47% | 50% | 41% | 44% | 46% | | 40% |
| 1991-10-30 22:18 | 0.787 | 0.7 | 0.8 | 0.8 | 0.9 | 0.9 | 0.8 | 0.9 | 10% | 2% | 3% | 13% | 8% | 5% | | 15% |
| 1993-11-28 19:24 | 0.053 | TOF | | | | | | | Error | | | | | | Total | 4260% |
| 1993-11-28 19:36 | 47.57 | 47.570 | | | | | | | 0% | | | | | | Avg | 62% |
| 1993-11-28 19:54 | 51.686 | 43.9 | 51.69 | | | | | | 15% | 0% | | | | | | |
| 1993-11-28 23:18 | 11.436 | 17.5 | 20.6 | | | | | | 53% | 80% | | | | | | |
| 1993-11-29 01:06 | 6.16 | 10.8 | 12.7 | 6.16 | | | | | 75% | 106% | 0% | | | | | |
| 1993-11-29 04:12 | 2.657 | 4.7 | 5.5 | 2.7 | | | | | 75% | 107% | 0% | | | | | |
| 1993-11-29 06:30 | 1.93 | 2.5 | 2.9 | 1.4 | 1.93 | | | | 30% | 53% | 26% | 0% | | | | |
| 1993-11-29 06:54 | 1.93 | 2.2 | 2.6 | 1.3 | 1.7 | | | | 16% | 37% | 33% | 10% | | | | |
| 1993-11-29 09:12 | 0.543 | 1.2 | 1.4 | 0.7 | 0.9 | 0.54 | | | 122% | 162% | 27% | 71% | 0% | | | |
| 1993-11-29 09:24 | 0.865 | 1.1 | 1.3 | 0.7 | 0.9 | 0.5 | | | 32% | 56% | 21% | 7% | 38% | | | |
| 1993-11-29 10:30 | 0.454 | 0.8 | 1.0 | 0.7 | 0.9 | 0.5 | 0.45 | | 87% | 120% | 47% | 98% | 15% | 0% | | |
| 1993-11-29 11:42 | 0.429 | 0.8 | 0.7 | 0.6 | 0.9 | 0.5 | 0.4 | | 92% | 69% | 50% | 103% | 18% | 2% | | |
| 1993-11-29 12:18 | 0.429 | 0.8 | 0.7 | 0.6 | 0.9 | 0.5 | 0.4 | 0.43 | 88% | 66% | 48% | 99% | 16% | 1% | | 0% |
| 1993-11-29 14:18 | 0.354 | 0.8 | 0.7 | 0.6 | 0.8 | 0.5 | 0.4 | 0.4 | 116% | 90% | 70% | 129% | 34% | 16% | | 15% |
| 1993-11-29 14:48 | 0.354 | 0.8 | 0.7 | 0.6 | 0.8 | 0.5 | 0.4 | 0.4 | 113% | 88% | 68% | 126% | 32% | 14% | | 13% |
| 1993-11-29 20:36 | 0.198 | 0.6 | 0.6 | 0.5 | 0.7 | 0.4 | 0.3 | 0.3 | 226% | 187% | 156% | 245% | 102% | 75% | | 73% |

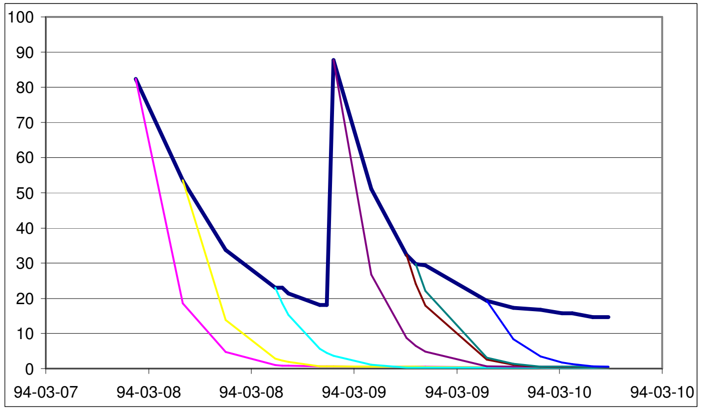
## Plots of Data vs Forecast



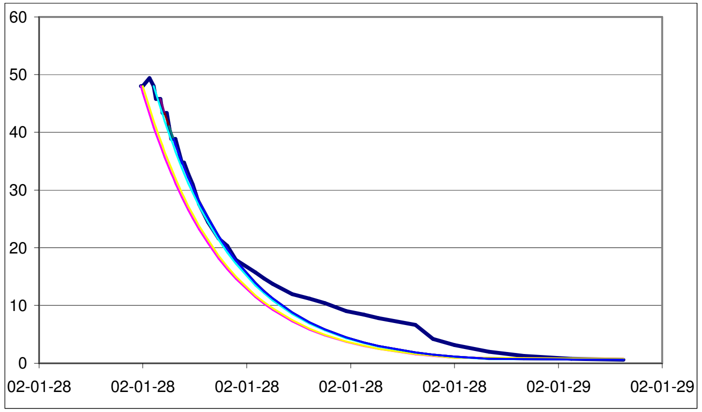



| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1994-02-03 21:18 | 36.74 | TOF | | | | | | | Error | | | | | Total | ###### |
| 1994-02-03 21:42 | 185.785 | 185.785 | | | | | | | 0% | | | | | Avg | 71% |
| 1994-02-03 22:06 | 169.899 | 166.8 | 169.9 | | | | | | 2% | 0% | | | | | |
| 1994-02-03 22:36 | 184.852 | 145.7 | 148.4 | | | | | | 21% | 20% | | | | | |
| 1994-02-04 01:00 | 106.467 | 76.2 | 77.6 | 106 | | | | | 28% | 27% | 0% | | | | |
| 1994-02-04 05:36 | 58.439 | 22.0 | 22.4 | 30.7 | | | | | 62% | 62% | 47% | | | | |
| 1994-02-04 08:00 | 45.295 | 11.5 | 11.7 | 16.1 | 45.3 | | | | 75% | 74% | 65% | 0% | | | |
| 1994-02-04 09:42 | 40.17 | 7.3 | 7.4 | 10.1 | 28.6 | | | | 82% | 82% | 75% | 29% | | | |
| 1994-02-04 10:54 | 40.387 | 5.2 | 5.3 | 7.3 | 20.7 | 40.4 | | | 87% | 87% | 82% | 49% | 0% | | |
| 1994-02-04 13:24 | 34.529 | 2.7 | 2.7 | 3.7 | 10.5 | 20.6 | | | 92% | 92% | 89% | 70% | 40% | | |
| 1994-02-04 18:54 | 15.609 | 0.6 | 0.6 | 0.8 | 2.4 | 4.6 | 15.6 | | 96% | 96% | 95% | 85% | 70% | 0% | |
| 1994-02-04 22:06 | 11.594 | 0.6 | 0.6 | 0.8 | 1.0 | 2.0 | 6.6 | | 95% | 95% | 93% | 91% | 83% | 43% | |
| 1994-02-05 02:00 | 9.375 | 0.5 | 0.5 | 0.7 | 0.3 | 0.7 | 2.3 | 9.38 | 95% | 95% | 93% | 96% | 93% | 76% | 0% |
| 1994-02-05 03:00 | 22.291 | 0.5 | 0.5 | 0.7 | 0.3 | 0.7 | 1.7 | 7.2 | 98% | 98% | 97% | 98% | 97% | 92% | 68% |
| 1994-02-05 03:42 | 21.478 | 0.5 | 0.5 | 0.7 | 0.3 | 0.7 | 1.4 | 5.9 | 98% | 98% | 97% | 98% | 97% | 93% | 72% |
| 1994-02-05 04:42 | 39.947 | 0.5 | 0.5 | 0.6 | 0.3 | 0.6 | 1.1 | 4.5 | 99% | 99% | 98% | 99% | 98% | 97% | 89% |
| 1994-03-07 21:42 | 72.691 | TOF | | | | | | | Error | | | | | Total | 8499% |
| 1994-03-07 22:30 | 82.401 | 82.401 | | | | | | | 0% | | | | | Avg | 82% |
| 1994-03-08 04:00 | 53.561 | 18.6 | 53.56 | | | | | | 65% | 0% | | | | | |
| 1994-03-08 09:00 | 33.752 | 4.8 | 13.9 | | | | | | 86% | 59% | | | | | |
| 1994-03-08 14:48 | 22.987 | 1.0 | 2.9 | 23 | | | | | 96% | 87% | 0% | | | | |
| 1994-03-08 15:36 | 22.987 | 0.8 | 2.3 | 18.5 | | | | | 96% | 90% | 19% | | | | |
| 1994-03-08 16:18 | 21.348 | 0.8 | 1.9 | 15.3 | | | | | 96% | 91% | 28% | | | | |
| 1994-03-08 20:00 | 18.122 | 0.7 | 0.7 | 5.6 | | | | | 96% | 96% | 69% | | | | |
| 1994-03-08 20:48 | 18.122 | 0.7 | 0.7 | 4.5 | | | | | 96% | 96% | 75% | | | | |
| 1994-03-08 21:36 | 87.716 | 0.7 | 0.7 | 3.7 | 87.7 | | | | 99% | 99% | 96% | 0% | | | |
| 1994-03-09 02:00 | 51.112 | 0.6 | 0.6 | 1.1 | 26.7 | | | | 99% | 99% | 98% | 48% | | | |
| 1994-03-09 06:06 | 32.422 | 0.5 | 0.5 | 0.4 | 8.8 | 32.4 | | | 98% | 98% | 99% | 73% | 0% | | |
| 1994-03-09 07:12 | 29.726 | 0.5 | 0.5 | 0.4 | 6.6 | 24.1 | 29.7 | | 98% | 98% | 99% | 78% | 19% | 0% | |
| 1994-03-09 08:18 | 29.378 | 0.5 | 0.5 | 0.3 | 4.9 | 17.9 | 22.1 | | 98% | 98% | 99% | 83% | 39% | 25% | |
| 1994-03-09 15:30 | 19.31 | 0.4 | 0.4 | 0.3 | 0.7 | 2.6 | 3.2 | 19.3 | 98% | 98% | 99% | 96% | 87% | 84% | 0% |
| 1994-03-09 18:36 | 17.323 | 0.4 | 0.4 | 0.3 | 0.6 | 1.1 | 1.4 | 8.4 | 98% | 98% | 98% | 96% | 94% | 92% | 52% |
| 1994-03-09 21:48 | 16.771 | 0.4 | 0.4 | 0.2 | 0.6 | 0.5 | 0.6 | 3.5 | 98% | 98% | 99% | 97% | 97% | 97% | 79% |
| 1994-03-10 00:18 | 15.713 | 0.3 | 0.3 | 0.2 | 0.5 | 0.4 | 0.5 | 1.8 | 98% | 98% | 99% | 97% | 97% | 97% | 89% |
| 1994-03-10 01:30 | 15.817 | 0.3 | 0.3 | 0.2 | 0.5 | 0.4 | 0.5 | 1.3 | 98% | 98% | 99% | 97% | 97% | 97% | 92% |
| 1994-03-10 03:54 | 14.708 | 0.3 | 0.3 | 0.2 | 0.5 | 0.4 | 0.5 | 0.7 | 98% | 98% | 99% | 97% | 97% | 97% | 95% |
| 1994-03-10 05:42 | 14.708 | 0.3 | 0.3 | 0.2 | 0.5 | 0.4 | 0.5 | 0.6 | 98% | 98% | 99% | 97% | 97% | 97% | 96% |



| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2002-01-28 04:36 | 45.79 | TOF | | | | | | | Error | | | | | Total | 751% |
| 2002-01-28 04:42 | 48.03 | 48.030 | | | | | | | 0% | | | | | Avg | 24% |
| 2002-01-28 04:48 | 48.03 | 46.7 | 48.03 | | | | | | 3% | 0% | | | | | |
| 2002-01-28 05:06 | 49.42 | 43.1 | 44.3 | | | | | | 13% | 10% | | | | | |
| 2002-01-28 05:18 | 48.03 | 40.8 | 42.0 | 48 | | | | | 15% | 13% | 0% | | | | |
| 2002-01-28 05:24 | 45.79 | 39.8 | 40.8 | 46.7 | | | | | 13% | 11% | 2% | | | | |
| 2002-01-28 05:36 | 45.79 | 37.7 | 38.7 | 44.3 | 45.8 | | | | 18% | 15% | 3% | 0% | | | |
| 2002-01-28 05:42 | 43.37 | 36.7 | 37.7 | 43.1 | 44.6 | | | | 15% | 13% | 1% | 3% | | | |
| 2002-01-28 05:48 | 43.37 | 35.7 | 36.7 | 42.0 | 43.4 | 43.4 | | | 18% | 15% | 3% | 0% | 0% | | |
| 2002-01-28 05:54 | 43.37 | 34.7 | 35.7 | 40.8 | 42.2 | 42.2 | | | 20% | 18% | 6% | 3% | 3% | | |
| 2002-01-28 06:00 | 41.06 | 33.8 | 34.7 | 39.8 | 41.1 | 41.1 | 41.1 | | 18% | 15% | 3% | 0% | 0% | 0% | |
| 2002-01-28 06:06 | 38.85 | 32.9 | 33.8 | 38.7 | 40.0 | 40.0 | 40.0 | | 15% | 13% | 0% | 3% | 3% | 3% | |
| 2002-01-28 06:12 | 38.85 | 32.0 | 32.9 | 37.7 | 38.9 | 38.9 | 38.9 | 38.9 | 18% | 15% | 3% | 0% | 0% | 0% | 0% |
| 2002-01-28 06:18 | 38.85 | 31.2 | 32.0 | 36.7 | 37.9 | 37.9 | 37.9 | 37.8 | 20% | 18% | 6% | 2% | 2% | 3% | 3% |
| 2002-01-28 06:36 | 34.73 | 28.7 | 29.5 | 33.8 | 34.9 | 34.9 | 34.9 | 34.9 | 17% | 15% | 3% | 1% | 1% | 1% | 0% |
| 2002-01-28 06:42 | 34.73 | 28.0 | 28.7 | 32.9 | 34.0 | 34.0 | 34.0 | 33.9 | 19% | 17% | 5% | 2% | 2% | 2% | 2% |
| 2002-01-28 06:54 | 32.8 | 26.5 | 27.2 | 31.2 | 32.2 | 32.2 | 32.2 | 32.2 | 19% | 17% | 5% | 2% | 2% | 2% | 2% |
| 2002-01-28 07:06 | 30.96 | 25.1 | 25.8 | 29.5 | 30.5 | 30.5 | 30.5 | 30.5 | 19% | 17% | 5% | 1% | 1% | 1% | 2% |
| 2002-01-28 07:24 | 27.54 | 23.2 | 23.8 | 27.2 | 28.2 | 28.1 | 28.1 | 28.1 | 16% | 14% | 1% | 2% | 2% | 2% | 2% |
| 2002-01-28 07:48 | 24.43 | 20.8 | 21.4 | 24.4 | 25.3 | 25.3 | 25.2 | 25.2 | 15% | 13% | 0% | 3% | 3% | 3% | 3% |
| 2002-01-28 08:18 | 21.61 | 18.2 | 18.7 | 21.4 | 22.1 | 22.1 | 22.1 | 22.0 | 16% | 14% | 1% | 2% | 2% | 2% | 2% |
| 2002-01-28 08:42 | 20.31 | 16.3 | 16.7 | 19.2 | 19.8 | 19.8 | 19.8 | 19.8 | 20% | 18% | 6% | 2% | 2% | 3% | 3% |
| 2002-01-28 09:06 | 17.89 | 14.6 | 15.0 | 17.2 | 17.8 | 17.8 | 17.8 | 17.7 | 18% | 16% | 4% | 1% | 1% | 1% | 1% |
| 2002-01-28 10:00 | 15.71 | 11.5 | 11.8 | 13.5 | 13.9 | 13.9 | 13.9 | 13.9 | 27% | 25% | 14% | 11% | 11% | 11% | 11% |
| 2002-01-28 10:24 | 14.71 | 10.3 | 10.6 | 12.1 | 12.5 | 12.5 | 12.5 | 12.5 | 30% | 28% | 18% | 15% | 15% | 15% | 15% |
| 2002-01-28 10:48 | 13.76 | 9.2 | 9.5 | 10.9 | 11.2 | 11.2 | 11.2 | 11.2 | 33% | 31% | 21% | 18% | 18% | 18% | 19% |
| 2002-01-28 11:42 | 12 | 7.2 | 7.4 | 8.5 | 8.8 | 8.8 | 8.8 | 8.8 | 40% | 38% | 29% | 27% | 27% | 27% | 27% |
| 2002-01-28 12:30 | 11.2 | 5.8 | 6.0 | 6.9 | 7.1 | 7.1 | 7.1 | 7.1 | 48% | 46% | 39% | 37% | 37% | 37% | 37% |
| 2002-01-28 13:12 | 10.44 | 4.8 | 5.0 | 5.7 | 5.9 | 5.9 | 5.9 | 5.9 | 54% | 52% | 46% | 44% | 44% | 44% | 44% |
| 2002-01-28 14:12 | 9.042 | 3.7 | 3.8 | 4.3 | 4.5 | 4.5 | 4.5 | 4.5 | 59% | 58% | 52% | 50% | 50% | 50% | 51% |
| 2002-01-28 15:00 | 8.403 | 3.0 | 3.1 | 3.5 | 3.6 | 3.6 | 3.6 | 3.6 | 65% | 64% | 58% | 57% | 57% | 57% | 57% |
| 2002-01-28 15:42 | 7.802 | 2.5 | 2.5 | 2.9 | 3.0 | 3.0 | 3.0 | 3.0 | 68% | 68% | 63% | 62% | 62% | 62% | 62% |
| 2002-01-28 17:24 | 6.633 | 1.6 | 1.6 | 1.8 | 1.9 | 1.9 | 1.9 | 1.9 | 77% | 76% | 72% | 72% | 72% | 72% | 72% |
| 2002-01-28 18:12 | 4.241 | 1.3 | 1.3 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 71% | 70% | 65% | 64% | 64% | 64% | 64% |
| 2002-01-28 19:12 | 3.138 | 1.0 | 1.0 | 1.1 | 1.2 | 1.2 | 1.2 | 1.2 | 70% | 69% | 64% | 63% | 63% | 63% | 63% |
| 2002-01-28 20:48 | 1.99 | 0.9 | 0.9 | 0.7 | 0.8 | 0.8 | 0.8 | 0.8 | 54% | 53% | 63% | 62% | 62% | 62% | 62% |
| 2002-01-28 22:24 | 1.255 | 0.9 | 0.9 | 0.7 | 0.7 | 0.7 | 0.7 | 0.7 | 30% | 28% | 44% | 43% | 43% | 43% | 43% |
| 2002-01-29 00:36 | 0.787 | 0.8 | 0.8 | 0.7 | 0.7 | 0.7 | 0.7 | 0.7 | 5% | 8% | 16% | 14% | 14% | 14% | 14% |
| 2002-01-29 03:00 | 0.575 | 0.8 | 0.8 | 0.6 | 0.6 | 0.6 | 0.6 | 0.6 | 34% | 38% | 7% | 11% | 11% | 11% | 11% |



# Calibration of K value for Pauls River at Coutzenburg Q3H004

**Data**

**Q3H004**

| | |
|---|---|
| Mode | 11.574 |
| Mean | 13.561 |
| Min | 0.114 |
| Max | 102.100 |

**Forecast**

$B_f$ 0.150 Tf 1.50

$K_U$ 7.20 $K_L$ 72

Basic Forecast Equation is defined as:

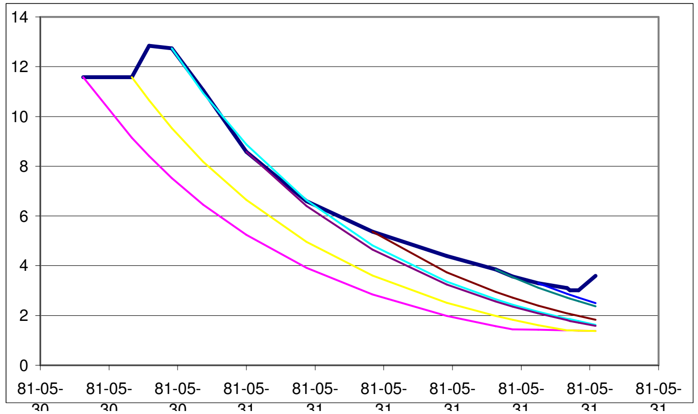
$$q_2 = q_1 \times \exp^{(-dt/k)}$$

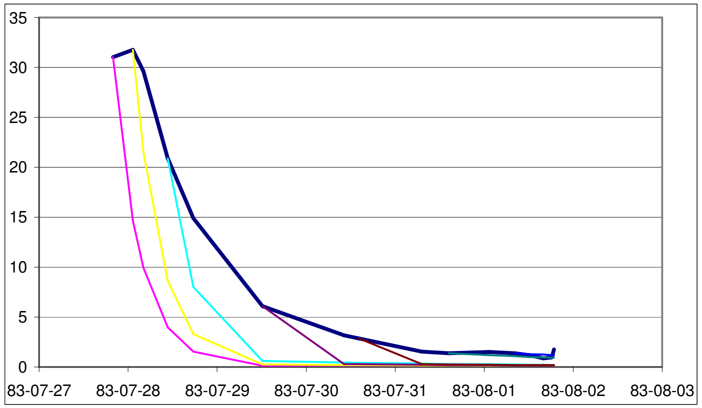
**Error Calculation**

**Total AVG** 78%

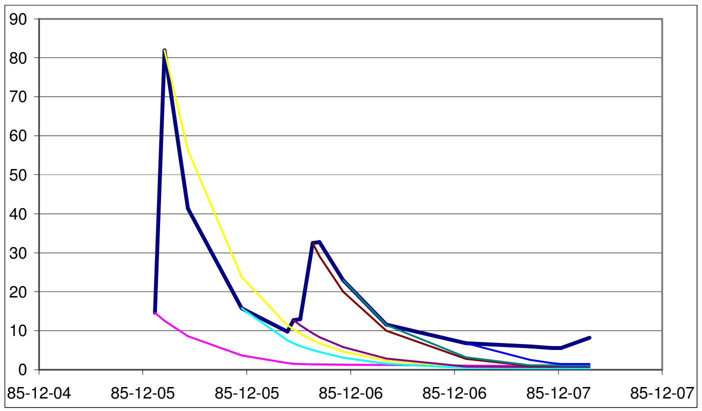
**Plots of Data vs Forecast**

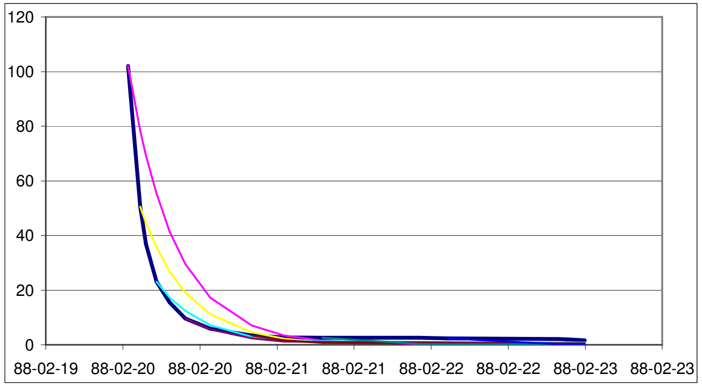
| Date | Discharge | Forecast | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1981-05-30 18:00 | 10.867 | TOF | | | | | | | Error | | | | | Total | 1691% |
| 1981-05-30 18:18 | 11.574 | 11.574 | | | | | | | 0% | | | | | Avg | 30% |
| 1981-05-30 20:00 | 11.574 | 9.1 | 11.57 | | | | | | 21% | 0% | | | | | |
| 1981-05-30 20:36 | 12.839 | 8.4 | 10.6 | | | | | | 35% | 17% | | | | | |
| 1981-05-30 21:24 | 12.739 | 7.5 | 9.5 | 12.7 | | | | | 41% | 25% | 0% | | | | |
| 1981-05-30 22:30 | 11.05 | 6.5 | 8.2 | 10.9 | | | | | 42% | 26% | 1% | | | | |
| 1981-05-31 00:00 | 8.577 | 5.2 | 6.6 | 8.9 | 8.6 | | | | 39% | 23% | 3% | 0% | | | |
| 1981-05-31 02:06 | 6.593 | 3.9 | 5.0 | 6.6 | 6.4 | | | | 41% | 25% | 1% | 3% | | | |
| 1981-05-31 04:24 | 5.369 | 2.8 | 3.6 | 4.8 | 4.7 | 5.4 | | | 47% | 33% | 10% | 13% | 0% | | |
| 1981-05-31 07:00 | 4.388 | 2.0 | 2.5 | 3.4 | 3.2 | 3.7 | | | 55% | 43% | 24% | 26% | 15% | | |
| 1981-05-31 08:42 | 3.841 | 1.6 | 2.0 | 2.7 | 2.6 | 3.0 | 3.8 | | 59% | 48% | 31% | 33% | 23% | 0% | |
| 1981-05-31 09:18 | 3.559 | 1.4 | 1.8 | 2.4 | 2.4 | 2.7 | 3.5 | | 60% | 49% | 31% | 34% | 24% | 1% | |
| 1981-05-31 10:12 | 3.305 | 1.4 | 1.6 | 2.2 | 2.1 | 2.4 | 3.1 | 3.3 | 57% | 51% | 35% | 37% | 27% | 6% | 0% |
| 1981-05-31 11:12 | 3.108 | 1.4 | 1.4 | 1.9 | 1.8 | 2.1 | 2.7 | 2.9 | 55% | 55% | 40% | 42% | 33% | 13% | 7% |
| 1981-05-31 11:18 | 3.004 | 1.4 | 1.4 | 1.8 | 1.8 | 2.1 | 2.7 | 2.8 | 53% | 53% | 39% | 41% | 31% | 11% | 6% |
| 1981-05-31 11:36 | 3.019 | 1.4 | 1.4 | 1.8 | 1.7 | 2.0 | 2.6 | 2.7 | 54% | 54% | 41% | 43% | 35% | 15% | 10% |
| 1981-05-31 12:12 | 3.591 | 1.4 | 1.4 | 1.6 | 1.6 | 1.8 | 2.4 | 2.5 | 61% | 62% | 55% | 56% | 49% | 34% | 30% |
| 1983-07-27 15:42 | 28.918 | TOF | | | | | | | Error | | | | | Total | 4191% |
| 1983-07-27 19:54 | 31.05 | 31.050 | | | | | | | 0% | | | | | Avg | 68% |
| 1983-07-28 01:18 | 31.771 | 14.7 | 31.8 | | | | | | 54% | 0% | | | | | |
| 1983-07-28 04:06 | 29.619 | 9.9 | 21.5 | | | | | | 66% | 27% | | | | | |
| 1983-07-28 10:42 | 20.887 | 4.0 | 8.6 | 20.9 | | | | | 81% | 59% | 0% | | | | |
| 1983-07-28 17:36 | 14.904 | 1.5 | 3.3 | 8.0 | | | | | 90% | 78% | 46% | | | | |
| 1983-07-29 12:12 | 6.095 | 0.1 | 0.2 | 0.6 | 6.1 | | | | 98% | 96% | 90% | 0% | | | |
| 1983-07-30 10:12 | 3.168 | 0.2 | 0.2 | 0.4 | 0.3 | | | | 95% | 94% | 86% | 91% | | | |
| 1983-07-30 14:18 | 2.86 | 0.2 | 0.2 | 0.4 | 0.3 | 2.9 | | | 95% | 94% | 85% | 91% | 0% | | |
| 1983-07-31 07:12 | 1.525 | 0.2 | 0.1 | 0.3 | 0.2 | 0.3 | | | 90% | 91% | 78% | 86% | 82% | | |
| 1983-07-31 14:24 | 1.356 | 0.2 | 0.2 | 0.3 | 0.2 | 0.2 | 1.4 | | 89% | 89% | 78% | 86% | 82% | 0% | |
| 1983-08-01 01:24 | 1.496 | 0.2 | 0.2 | 0.3 | 0.2 | 0.2 | 1.2 | | 90% | 90% | 83% | 89% | 86% | 22% | |
| 1983-08-01 08:12 | 1.365 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 1.1 | 1.4 | 89% | 89% | 83% | 89% | 86% | 22% | 0% |
| 1983-08-01 12:06 | 1.199 | 0.2 | 0.2 | 0.2 | 0.1 | 0.2 | 1.0 | 1.3 | 87% | 87% | 81% | 88% | 85% | 16% | 8% |
| 1983-08-01 16:00 | 0.876 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 1.0 | 1.2 | 83% | 83% | 76% | 83% | 80% | 8% | 40% |
| 1983-08-01 18:12 | 0.955 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.9 | 1.2 | 84% | 84% | 79% | 84% | 82% | 4% | 24% |
| 1983-08-01 18:48 | 1.749 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.9 | 1.2 | 91% | 91% | 88% | 91% | 90% | 48% | 33% |

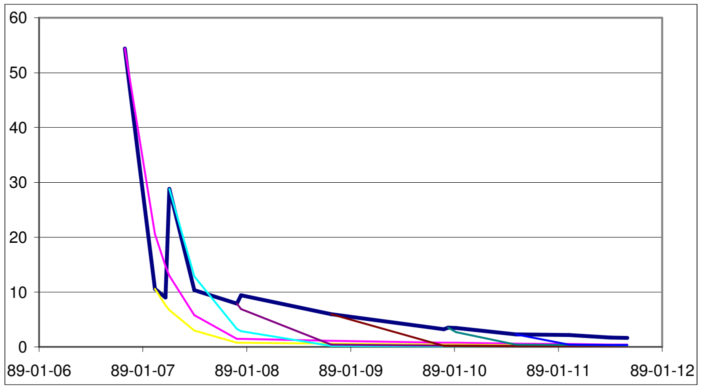


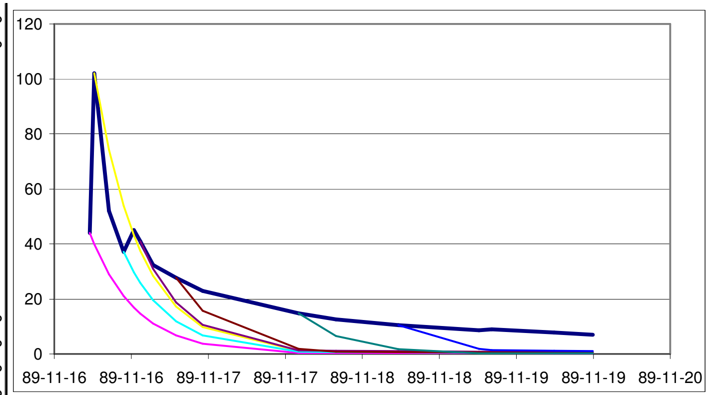
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1985-12-05 01:12 | 14.428 | TOF | | | | | | | Error | | | | | Total | 2659% |
| 1985-12-05 01:24 | 14.586 | 14.586 | | | | | | | | | | | | Avg | 62% |
| 1985-12-05 02:30 | 81.933 | 12.5 | 81.9 | | | | | | | 0% | | | | | |
| 1985-12-05 05:12 | 41.33 | 8.6 | 56.3 | | | | | | | 36% | | | | | |
| 1985-12-05 11:24 | 15.716 | 3.6 | 23.8 | 15.7 | | | | | | 51% | 0% | | | | |
| 1985-12-05 16:42 | 9.751 | 1.7 | 11.4 | 7.5 | | | | | | 17% | 23% | | | | |
| 1985-12-05 17:24 | 12.739 | 1.6 | 10.3 | 6.8 | 12.7 | | | | | 19% | 46% | 0% | | | |
| 1985-12-05 18:12 | 12.938 | 1.4 | 9.2 | 6.1 | 11.4 | | | | | 29% | 53% | 12% | | | |
| 1985-12-05 19:36 | 32.502 | 1.4 | 7.6 | 5.0 | 9.4 | 32.5 | | | | 77% | 85% | 71% | 0% | | |
| 1985-12-05 20:24 | 32.719 | 1.4 | 6.8 | 4.5 | 8.4 | 29.1 | | | | 79% | 86% | 74% | 11% | | |
| 1985-12-05 23:06 | 22.954 | 1.3 | 4.7 | 3.1 | 5.8 | 20.0 | 23.0 | | | 80% | 87% | 75% | 13% | 0% | |
| 1985-12-06 04:06 | 11.525 | 1.2 | 2.3 | 1.5 | 2.9 | 10.0 | 11.5 | | | 80% | 87% | 75% | 13% | 1% | |
| 1985-12-06 13:18 | 6.858 | 1.1 | 0.7 | 0.4 | 0.8 | 2.8 | 3.2 | 6.9 | | 91% | 94% | 88% | 59% | 53% | 0% |
| 1985-12-06 20:42 | 5.949 | 1.0 | 0.6 | 0.4 | 0.7 | 1.0 | 1.1 | 2.5 | | 90% | 93% | 88% | 83% | 81% | 59% |
| 1985-12-06 23:12 | 5.5 | 0.9 | 0.6 | 0.4 | 0.7 | 1.0 | 1.1 | 1.7 | | 90% | 93% | 87% | 83% | 80% | 68% |
| 1985-12-07 00:18 | 5.582 | 0.9 | 0.6 | 0.4 | 0.7 | 0.9 | 1.1 | 1.5 | | 90% | 93% | 88% | 83% | 81% | 73% |
| 1985-12-07 03:36 | 8.179 | 0.9 | 0.5 | 0.4 | 0.7 | 0.9 | 1.0 | 1.4 | | 93% | 96% | 92% | 89% | 87% | 83% |
| 1988-02-19 22:36 | 12.15 | TOF | | | | | | | Error | | | | | Total | 3910% |
| 1988-02-20 00:48 | 102.1 | 102.100 | | | | | | | 0% | | | | | Avg | 64% |
| 1988-02-20 02:42 | 50.593 | 78.4 | 50.6 | | | | | | 55% | 0% | | | | | |
| 1988-02-20 03:36 | 36.922 | 69.2 | 44.6 | | | | | | 87% | 21% | | | | | |
| 1988-02-20 05:12 | 23.147 | 55.4 | 35.7 | 23.1 | | | | | 139% | 54% | 0% | | | | |
| 1988-02-20 07:18 | 15.499 | 41.4 | 26.7 | 17.3 | | | | | 167% | 72% | 12% | | | | |
| 1988-02-20 09:42 | 9.621 | 29.6 | 19.1 | 12.4 | 9.6 | | | | 208% | 99% | 29% | 0% | | | |
| 1988-02-20 13:36 | 5.861 | 17.2 | 11.1 | 7.2 | 5.6 | | | | 194% | 90% | 23% | 5% | | | |
| 1988-02-20 20:06 | 3.415 | 7.0 | 4.5 | 2.9 | 2.3 | 3.4 | | | 105% | 32% | 14% | 34% | 0% | | |
| 1988-02-21 01:06 | 2.721 | 3.5 | 2.3 | 1.5 | 1.1 | 1.7 | | | 28% | 17% | 46% | 58% | 37% | | |
| 1988-02-21 07:06 | 2.612 | 1.5 | 1.0 | 1.3 | 1.0 | 0.7 | 2.6 | | 42% | 63% | 49% | 60% | 72% | 0% | |
| 1988-02-21 22:06 | 2.585 | 0.2 | 0.8 | 1.1 | 0.8 | 0.6 | 0.3 | | 93% | 69% | 58% | 67% | 77% | 87% | |
| 1988-02-22 02:36 | 2.288 | 0.2 | 0.7 | 1.0 | 0.8 | 0.6 | 0.3 | 2.3 | 92% | 67% | 55% | 65% | 75% | 87% | 0% |
| 1988-02-22 19:48 | 2.095 | 0.1 | 0.6 | 0.8 | 0.6 | 0.4 | 0.2 | 0.2 | 93% | 72% | 62% | 70% | 79% | 89% | 90% |
| 1988-02-22 21:12 | 2.002 | 0.2 | 0.6 | 0.8 | 0.6 | 0.4 | 0.2 | 0.2 | 93% | 71% | 61% | 69% | 78% | 88% | 90% |
| 1988-02-22 23:54 | 1.686 | 0.2 | 0.6 | 0.8 | 0.6 | 0.4 | 0.2 | 0.2 | 91% | 67% | 55% | 65% | 75% | 87% | 88% |
| 1989-01-06 18:54 | 6.246 | TOF | | | | | | | Error | | | | | Total | 4406% |
| 1989-01-06 19:48 | 54.371 | 54.371 | | | | | | | 0% | | | | | Avg | 72% |
| 1989-01-07 02:48 | 10.593 | 20.6 | 10.6 | | | | | | 94% | 0% | | | | | |
| 1989-01-07 05:12 | 8.986 | 14.7 | 7.6 | | | | | | 64% | 16% | | | | | |
| 1989-01-07 06:06 | 28.781 | 13.0 | 6.7 | 28.8 | | | | | 55% | 77% | 0% | | | | |
| 1989-01-07 11:54 | 10.37 | 5.8 | 3.0 | 12.9 | | | | | 44% | 71% | 24% | | | | |
| 1989-01-07 21:42 | 7.908 | 1.5 | 0.8 | 3.3 | 7.9 | | | | 81% | 90% | 58% | 0% | | | |
| 1989-01-07 22:42 | 9.449 | 1.5 | 0.8 | 2.9 | 6.9 | | | | 84% | 92% | 70% | 27% | | | |
| 1989-01-08 19:30 | 5.977 | 1.1 | 0.6 | 0.2 | 0.4 | 6.0 | | | 82% | 91% | 97% | 94% | 0% | | |
| 1989-01-09 21:30 | 3.229 | 0.8 | 0.4 | 0.1 | 0.3 | 0.2 | | | 76% | 88% | 97% | 92% | 95% | | |
| 1989-01-09 22:30 | 3.51 | 0.8 | 0.4 | 0.2 | 0.3 | 0.2 | 3.5 | | 78% | 89% | 96% | 93% | 95% | 0% | |
| 1989-01-10 00:24 | 3.479 | 0.7 | 0.4 | 0.2 | 0.3 | 0.2 | 2.7 | | 79% | 89% | 96% | 93% | 96% | 23% | |
| 1989-01-10 14:12 | 2.301 | 0.6 | 0.3 | 0.2 | 0.2 | 0.1 | 0.4 | 2.3 | 74% | 86% | 93% | 91% | 94% | 83% | 0% |
| 1989-01-11 02:24 | 2.142 | 0.5 | 0.3 | 0.2 | 0.2 | 0.2 | 0.3 | 0.4 | 76% | 88% | 93% | 92% | 93% | 84% | 80% |
| 1989-01-11 12:06 | 1.707 | 0.4 | 0.2 | 0.2 | 0.2 | 0.2 | 0.3 | 0.4 | 74% | 86% | 91% | 91% | 91% | 83% | 78% |
| 1989-01-11 15:54 | 1.656 | 0.4 | 0.2 | 0.2 | 0.1 | 0.2 | 0.3 | 0.4 | 74% | 87% | 91% | 91% | 91% | 83% | 79% |

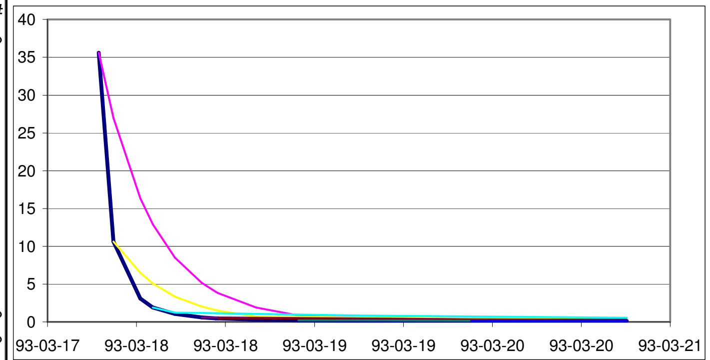


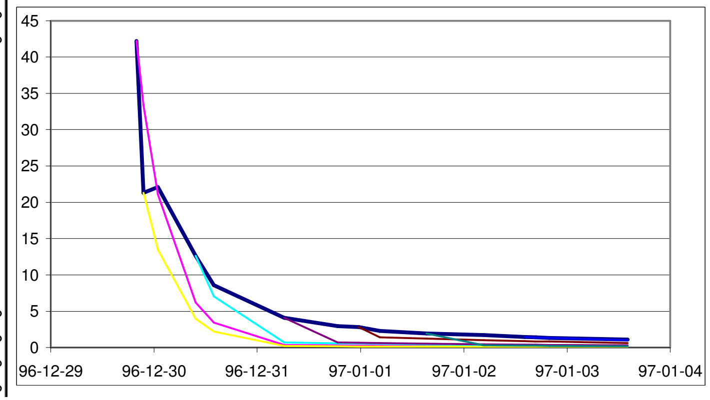



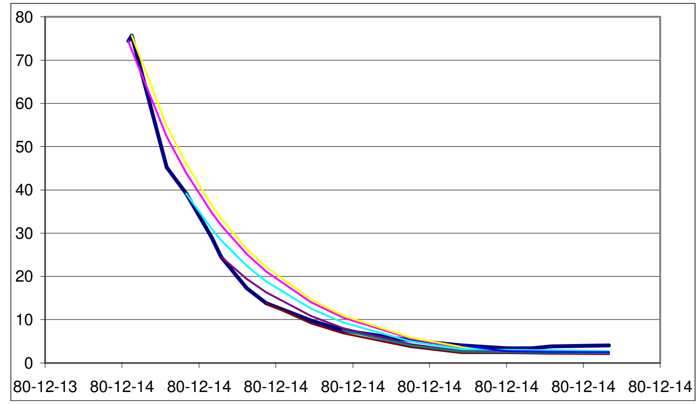
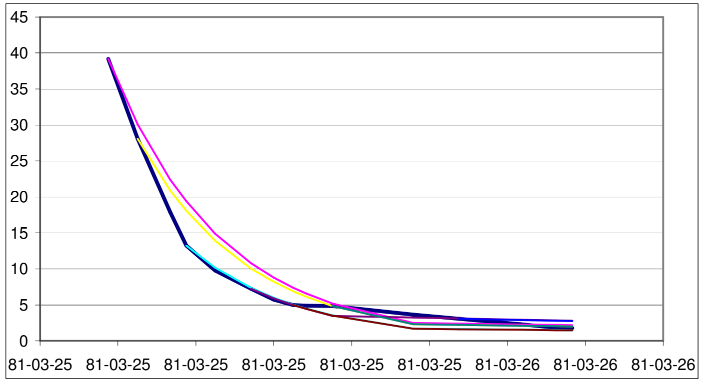
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1989-11-16 04:24 | 30.904 | TOF | | | | | | | Error | | | | | Total | 2889% |
| 1989-11-16 05:30 | 44.003 | 44.0 | | | | | | | | | | | | Avg | 66% |
| 1989-11-16 06:12 | 102.1 | 39.9 | 102.1 | | | | | | | 0% | | | | | |
| 1989-11-16 08:30 | 52.074 | 29.0 | 74.2 | | | | | | | 42% | | | | | |
| 1989-11-16 10:48 | 37.153 | 21.1 | 53.9 | 37.2 | | | | | | 45% | 0% | | | | |
| 1989-11-16 12:24 | 45 | 16.9 | 43.1 | 29.7 | | | | | | 4% | 34% | | | | |
| 1989-11-16 13:24 | 40.931 | 14.7 | 37.5 | 25.9 | 40.9 | | | | | 8% | 37% | 0% | | | |
| 1989-11-16 15:24 | 32.354 | 11.1 | 28.4 | 19.6 | 31.0 | | | | | 12% | 39% | 4% | | | |
| 1989-11-16 19:00 | 27.804 | 6.7 | 17.2 | 11.9 | 18.8 | 27.8 | | | | 38% | 57% | 32% | 0% | | |
| 1989-11-16 23:06 | 22.954 | 3.8 | 9.8 | 6.7 | 10.6 | 15.7 | | | | 57% | 71% | 54% | 31% | | |
| 1989-11-17 14:06 | 14.797 | 0.5 | 1.2 | 0.8 | 1.3 | 2.0 | 14.8 | | | 92% | 94% | 91% | 87% | 0% | |
| 1989-11-17 19:54 | 12.591 | 0.4 | 1.1 | 0.8 | 1.2 | 0.9 | 6.6 | | | 91% | 94% | 90% | 93% | 48% | |
| 1989-11-18 05:36 | 10.503 | 0.4 | 1.0 | 0.7 | 1.1 | 0.8 | 1.7 | 10.5 | | 91% | 94% | 90% | 93% | 84% | 0% |
| 1989-11-18 18:12 | 8.738 | 0.3 | 0.8 | 0.6 | 0.9 | 0.6 | 0.3 | 1.8 | | 91% | 94% | 90% | 93% | 97% | 79% |
| 1989-11-18 20:12 | 8.944 | 0.3 | 0.8 | 0.6 | 0.9 | 0.6 | 0.3 | 1.4 | | 91% | 94% | 90% | 93% | 97% | 85% |
| 1989-11-19 05:54 | 7.832 | 0.3 | 0.7 | 0.5 | 0.8 | 0.5 | 0.3 | 1.2 | | 91% | 94% | 90% | 93% | 97% | 85% |
| 1989-11-19 11:54 | 6.995 | 0.3 | 0.6 | 0.4 | 0.7 | 0.5 | 0.2 | 1.1 | | 91% | 94% | 90% | 93% | 97% | 84% |
| 1993-03-17 18:36 | 0.114 | TOF | | | | | | | Error | | | | | Total | ###### |
| 1993-03-17 18:54 | 35.632 | 35.632 | | | | | | | 0% | | | | | Avg | 199% |
| 1993-03-17 20:54 | 10.639 | 27.0 | 10.6 | | | | | | 154% | 0% | | | | | |
| 1993-03-18 00:30 | 3.108 | 16.4 | 6.5 | | | | | | 427% | 108% | | | | | |
| 1993-03-18 02:12 | 1.878 | 12.9 | 5.1 | 1.9 | | | | | 588% | 171% | 0% | | | | |
| 1993-03-18 05:12 | 1.053 | 8.5 | 3.4 | 1.2 | | | | | 709% | 219% | 18% | | | | |
| 1993-03-18 08:48 | 0.606 | 5.2 | 2.0 | 1.2 | 0.6 | | | | 752% | 236% | 94% | 0% | | | |
| 1993-03-18 10:42 | 0.475 | 4.0 | 1.6 | 1.1 | 0.6 | | | | 735% | 229% | 141% | 24% | | | |
| 1993-03-18 11:00 | 0.475 | 3.8 | 1.5 | 1.1 | 0.6 | 0.5 | | | 701% | 216% | 140% | 24% | 0% | | |
| 1993-03-18 16:06 | 0.291 | 1.9 | 0.7 | 1.1 | 0.5 | 0.4 | | | 544% | 154% | 266% | 88% | 52% | | |
| 1993-03-18 21:48 | 0.206 | 0.8 | 0.7 | 1.0 | 0.5 | 0.4 | 0.2 | | 312% | 231% | 377% | 146% | 98% | 0% | |
| 1993-03-19 05:54 | 0.159 | 0.8 | 0.6 | 0.9 | 0.5 | 0.4 | 0.2 | | 377% | 283% | 452% | 184% | 130% | 16% | |
| 1993-03-19 21:06 | 0.14 | 0.6 | 0.5 | 0.7 | 0.4 | 0.3 | 0.1 | 0.1 | 338% | 253% | 408% | 161% | 111% | 6% | 0% |
| 1993-03-20 18:06 | 0.131 | 0.5 | 0.4 | 0.5 | 0.3 | 0.2 | 0.2 | 0.2 | 250% | 181% | 305% | 109% | 69% | 15% | 15% |
| 1996-12-29 19:18 | 9.279 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.2 | 0.2 | 100% | 100% | 100% | 100% | 100% | 98% | 98% |
| 1996-12-29 19:36 | 8.537 | TOF | | | | | | | Error | | | | | Total | 3858% |
| 1996-12-29 19:54 | 42.215 | 42.215 | | | | | | | 0% | | | | | Avg | 63% |
| 1996-12-29 21:36 | 21.318 | 33.3 | 21.3 | | | | | | 56% | 0% | | | | | |
| 1996-12-30 00:54 | 22.132 | 21.1 | 13.5 | | | | | | 5% | 39% | | | | | |
| 1996-12-30 09:42 | 12.639 | 6.2 | 4.0 | 12.6 | | | | | 51% | 69% | 0% | | | | |
| 1996-12-30 13:54 | 8.577 | 3.5 | 2.2 | 7.1 | | | | | 60% | 74% | 18% | | | | |
| 1996-12-31 06:18 | 4.082 | 0.4 | 0.2 | 0.7 | 4.1 | | | | 91% | 94% | 82% | 0% | | | |
| 1996-12-31 18:42 | 2.975 | 0.3 | 0.2 | 0.6 | 0.7 | | | | 90% | 94% | 80% | 76% | | | |
| 1996-12-31 23:36 | 2.818 | 0.3 | 0.2 | 0.6 | 0.7 | 2.8 | | | 90% | 94% | 80% | 76% | 0% | | |
| 1997-01-01 04:30 | 2.301 | 0.3 | 0.2 | 0.5 | 0.6 | 1.4 | | | 89% | 93% | 77% | 72% | 38% | | |
| 1997-01-01 15:18 | 1.934 | 0.2 | 0.1 | 0.5 | 0.5 | 1.2 | 1.9 | | 88% | 93% | 76% | 72% | 37% | 0% | |
| 1997-01-02 04:36 | 1.728 | 0.2 | 0.2 | 0.4 | 0.5 | 1.0 | 0.3 | | 89% | 91% | 78% | 74% | 41% | 82% | |
| 1997-01-02 14:12 | 1.458 | 0.2 | 0.2 | 0.3 | 0.4 | 0.9 | 0.3 | 1.5 | 89% | 90% | 77% | 73% | 39% | 82% | 0% |
| 1997-01-02 16:42 | 1.439 | 0.2 | 0.2 | 0.3 | 0.4 | 0.9 | 0.3 | 1.4 | 89% | 90% | 78% | 73% | 40% | 82% | 2% |
| 1997-01-02 19:54 | 1.338 | 0.2 | 0.2 | 0.3 | 0.4 | 0.8 | 0.2 | 1.3 | 89% | 89% | 77% | 73% | 38% | 82% | 1% |
| 1997-01-03 14:06 | 1.109 | 0.1 | 0.2 | 0.2 | 0.3 | 0.6 | 0.2 | 1.0 | 89% | 86% | 79% | 74% | 42% | 83% | 6% |



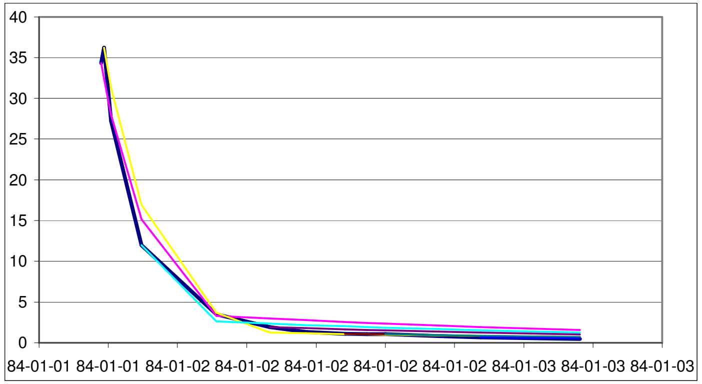



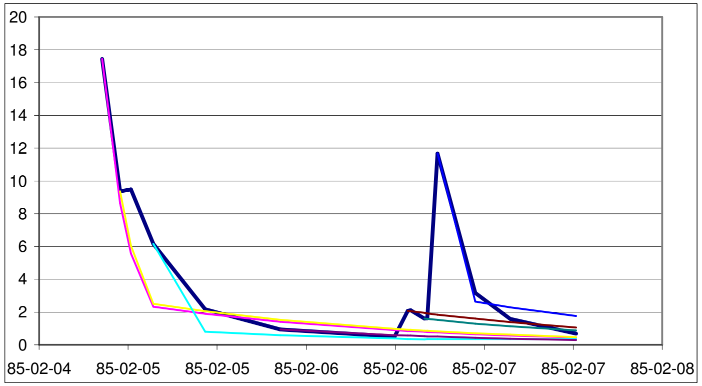



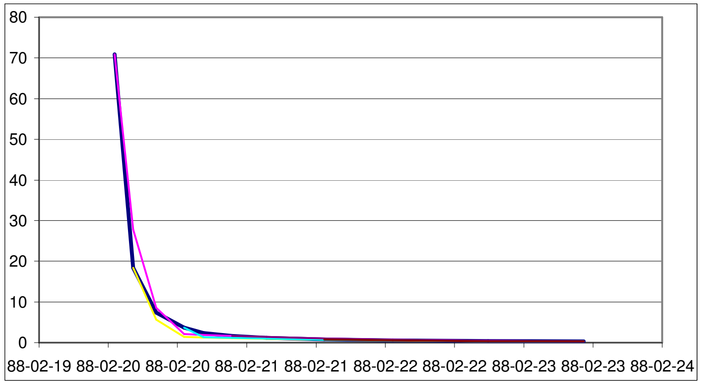
# Calibration of K value for Tarka River at Bridge Farm Q4H013

| **Data** | | **Forecast** | | | | | | | **Error Calculation** | | | | | | | **Plots of Data vs Forecast** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Q4H013** | | $B_f$ | 0.350 | Tf | 3.50 | | | | | | | | **Total AVG** | | 389% | |
| Mode | 106.91 | $K_U$ | 3.42 | $K_L$ | 34 | | | | | | | | | | | |
| Mean | 13.570 | Basic Forecast Equation is defined as: | | | | | | | | | | | | | | |
| Min | 0.087 | | | | | | | | | | | | | | | |
| Max | 106.910 | $q_2 = q_1 \times \exp^{(-dt/k)}$ | | | | | | | | | | | | | | |
| DATE | Discharge | Forecast | | | | | | | | | | | | | | |
| 1980-12-13 22:48 | 74.394 | TOF | | | | | | | Error | | | | | Total | 1387% |  |
| 1980-12-14 00:12 | 74.394 | 74.394 | | | | | | | 0% | | | | | Avg | 25% | |
| 1980-12-14 00:18 | 75.671 | 72.2 | 75.67 | | | | | | 5% | 0% | | | | | | |
| 1980-12-14 01:24 | 45.253 | 52.4 | 54.8 | | | | | | 16% | 21% | | | | | | |
| 1980-12-14 02:00 | 39.148 | 43.9 | 46.0 | 39.1 | | | | | 12% | 18% | 0% | | | | | |
| 1980-12-14 02:48 | 28.915 | 34.8 | 36.4 | 31.0 | | | | | 20% | 26% | 7% | | | | | |
| 1980-12-14 03:06 | 24.541 | 31.8 | 33.3 | 28.4 | 24.5 | | | | 30% | 36% | 16% | 0% | | | | |
| 1980-12-14 03:54 | 17.24 | 25.2 | 26.4 | 22.5 | 19.4 | | | | 46% | 53% | 30% | 13% | | | | |
| 1980-12-14 04:30 | 13.78 | 21.1 | 22.1 | 18.8 | 16.3 | 13.8 | | | 53% | 61% | 37% | 18% | 0% | | | |
| 1980-12-14 05:54 | 9.756 | 14.0 | 14.7 | 12.5 | 10.8 | 9.1 | | | 44% | 51% | 28% | 11% | 6% | | | |
| 1980-12-14 06:54 | 7.565 | 10.5 | 11.0 | 9.3 | 8.1 | 6.8 | 7.57 | | 38% | 45% | 23% | 7% | 10% | 0% | | |
| 1980-12-14 09:00 | 5.226 | 5.7 | 5.9 | 5.0 | 4.4 | 3.7 | 4.1 | | 8% | 14% | 3% | 16% | 29% | 22% | | |
| 1980-12-14 10:36 | 4.069 | 3.5 | 3.7 | 3.2 | 2.7 | 2.3 | 2.6 | 4.07 | 13% | 9% | 22% | 33% | 43% | 37% | 0% | |
| 1980-12-14 12:00 | 3.372 | 2.4 | 2.5 | 3.0 | 2.6 | 2.2 | 2.5 | 2.7 | 30% | 27% | 10% | 22% | 34% | 27% | 20% | |
| 1980-12-14 12:48 | 3.403 | 2.3 | 2.4 | 3.0 | 2.6 | 2.2 | 2.4 | 2.6 | 32% | 29% | 13% | 25% | 36% | 29% | 22% | |
| 1980-12-14 13:24 | 3.777 | 2.3 | 2.4 | 2.9 | 2.5 | 2.1 | 2.4 | 2.6 | 40% | 37% | 23% | 33% | 44% | 38% | 31% | |
| 1980-12-14 15:12 | 4.051 | 2.1 | 2.2 | 2.8 | 2.4 | 2.0 | 2.2 | 2.5 | 47% | 45% | 32% | 41% | 50% | 45% | 39% | |
| 1981-03-25 10:12 | 1.229 | TOF | | | | | | | Error | | | | | Total | 1407% |  |
| 1981-03-25 11:42 | 39.148 | 39.148 | | | | | | | 0% | | | | | Avg | 23% | |
| 1981-03-25 12:36 | 28.055 | 30.1 | 28.06 | | | | | | 7% | 0% | | | | | | |
| 1981-03-25 13:36 | 17.938 | 22.5 | 20.9 | | | | | | 25% | 17% | | | | | | |
| 1981-03-25 14:06 | 13.234 | 19.4 | 18.1 | 13.2 | | | | | 47% | 37% | 0% | | | | | |
| 1981-03-25 15:00 | 9.756 | 14.9 | 13.9 | 10.2 | | | | | 53% | 42% | 4% | | | | | |
| 1981-03-25 16:06 | 7.259 | 10.8 | 10.1 | 7.4 | 7.26 | | | | 49% | 39% | 2% | 0% | | | | |
| 1981-03-25 16:48 | 5.725 | 8.8 | 8.2 | 6.0 | 5.9 | | | | 54% | 43% | 5% | 3% | | | | |
| 1981-03-25 17:24 | 4.992 | 7.4 | 6.9 | 5.0 | 5.0 | 4.99 | | | 48% | 38% | 1% | 1% | 0% | | | |
| 1981-03-25 17:48 | 4.892 | 6.6 | 6.1 | 4.5 | 4.4 | 4.4 | | | 34% | 25% | 8% | 10% | 9% | | | |
| 1981-03-25 18:36 | 4.819 | 5.2 | 4.8 | 3.5 | 3.5 | 3.5 | 4.82 | | 8% | 1% | 26% | 28% | 27% | 0% | | |
| 1981-03-25 21:06 | 3.661 | 2.5 | 2.3 | 1.7 | 3.2 | 1.7 | 2.3 | | 32% | 36% | 53% | 11% | 54% | 37% | | |
| 1981-03-25 22:30 | 3.082 | 2.4 | 2.2 | 1.6 | 3.1 | 1.6 | 2.2 | 3.08 | 22% | 27% | 47% | 1% | 47% | 28% | 0% | |
| 1981-03-26 00:24 | 2.305 | 2.3 | 2.1 | 1.5 | 2.9 | 1.5 | 2.1 | 2.9 | 1% | 8% | 33% | 28% | 33% | 9% | 26% | |
| 1981-03-26 01:30 | 1.861 | 2.2 | 2.1 | 1.5 | 2.9 | 1.5 | 2.0 | 2.8 | 18% | 10% | 19% | 53% | 20% | 10% | 52% | |
| 1981-03-26 02:00 | 1.775 | 2.2 | 2.0 | 1.5 | 2.8 | 1.5 | 2.0 | 2.8 | 22% | 14% | 17% | 58% | 17% | 13% | 57% | |

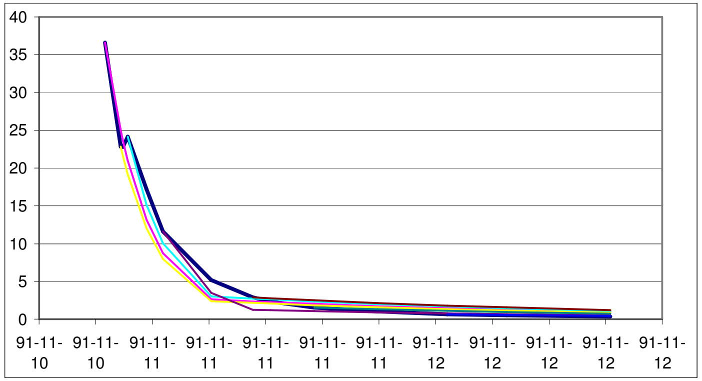
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1984-01-01 18:36 | 0.343 | TOF | | | | | | | Error | | | | | Total | 2760% |
| 1984-01-01 18:42 | 34.336 | 34.336 | | | | | | | 0% | | | | | Avg | 50% |
| 1984-01-01 18:54 | 36.23 | 32.4 | 36.23 | | | | | | 11% | 0% | | | | | |
| 1984-01-01 19:24 | 27.2 | 28.0 | 31.3 | | | | | | 3% | 15% | | | | | |
| 1984-01-01 21:30 | 11.953 | 15.1 | 16.9 | 12 | | | | | 27% | 42% | 0% | | | | |
| 1984-01-02 02:42 | 3.515 | 3.3 | 3.7 | 2.6 | | | | | 6% | 5% | 26% | | | | |
| 1984-01-02 06:24 | 1.916 | 3.0 | 1.3 | 2.3 | 1.92 | | | | 55% | 35% | 22% | 0% | | | |
| 1984-01-02 08:54 | 1.325 | 2.8 | 1.2 | 2.2 | 1.8 | | | | 108% | 12% | 64% | 34% | | | |
| 1984-01-02 11:36 | 1.113 | 2.5 | 1.1 | 2.0 | 1.6 | 1.11 | | | 129% | 3% | 81% | 48% | 0% | | |
| 1984-01-02 13:06 | 1.065 | 2.4 | 1.0 | 1.9 | 1.6 | 1.1 | | | 129% | 3% | 81% | 48% | 0% | | |
| 1984-01-02 14:24 | 1.034 | 2.3 | 1.0 | 1.9 | 1.5 | 1.0 | 1.03 | | 127% | 4% | 79% | 47% | 1% | 0% | |
| 1984-01-02 18:06 | 0.812 | 2.1 | 0.9 | 1.7 | 1.4 | 0.9 | 0.9 | | 159% | 9% | 105% | 68% | 13% | 14% | |
| 1984-01-02 21:00 | 0.655 | 1.9 | 0.8 | 1.5 | 1.2 | 0.8 | 0.9 | 0.66 | 195% | 25% | 133% | 91% | 29% | 30% | 0% |
| 1984-01-03 03:54 | 0.479 | 1.6 | 0.7 | 1.2 | 1.0 | 0.7 | 0.7 | 0.5 | 230% | 39% | 161% | 113% | 44% | 45% | 12% |
| 1984-01-03 17:54 | 0.323 | | | | | | | | | | | | | | |
| 1984-01-04 08:30 | 0.283 | | | | | | | | | | | | | | |
| 1985-02-04 19:48 | 0.375 | TOF | | | | | | | Error | | | | | Total | 3104% |
| 1985-02-04 20:30 | 17.449 | 17.449 | | | | | | | 0% | | | | | Avg | 51% |
| 1985-02-04 22:54 | 9.379 | 8.6 | 9.379 | | | | | | 8% | 0% | | | | | |
| 1985-02-05 00:24 | 9.471 | 5.6 | 6.0 | | | | | | 41% | 36% | | | | | |
| 1985-02-05 03:24 | 6.169 | 2.3 | 2.5 | 6.17 | | | | | 62% | 59% | 0% | | | | |
| 1985-02-05 10:24 | 2.171 | 1.9 | 2.0 | 0.8 | | | | | 13% | 6% | 63% | | | | |
| 1985-02-05 20:30 | 0.93 | 1.4 | 1.5 | 0.6 | 0.93 | | | | 51% | 64% | 36% | 0% | | | |
| 1985-02-06 11:54 | 0.504 | 0.9 | 1.0 | 0.4 | 0.6 | | | | 78% | 93% | 25% | 18% | | | |
| 1985-02-06 13:42 | 2.076 | 0.8 | 0.9 | 0.4 | 0.6 | 2.08 | | | 59% | 56% | 83% | 73% | 0% | | |
| 1985-02-06 14:06 | 2.111 | 0.8 | 0.9 | 0.4 | 0.6 | 2.1 | | | 60% | 57% | 83% | 74% | 3% | | |
| 1985-02-06 15:54 | 1.599 | 0.8 | 0.9 | 0.3 | 0.5 | 1.9 | 1.6 | | 50% | 46% | 79% | 67% | 22% | 0% | |
| 1985-02-06 16:18 | 1.619 | 0.8 | 0.9 | 0.4 | 0.5 | 1.9 | 1.6 | | 51% | 47% | 78% | 68% | 19% | 2% | |
| 1985-02-06 17:42 | 11.674 | 0.8 | 0.8 | 0.4 | 0.5 | 1.8 | 1.5 | 11.7 | 94% | 93% | 97% | 96% | 84% | 87% | 0% |
| 1985-02-06 22:48 | 3.172 | 0.7 | 0.7 | 0.4 | 0.4 | 1.6 | 1.3 | 2.6 | 79% | 78% | 89% | 86% | 50% | 59% | 17% |
| 1985-02-07 03:30 | 1.589 | 0.6 | 0.6 | 0.4 | 0.4 | 1.4 | 1.1 | 2.3 | 64% | 61% | 78% | 76% | 13% | 28% | 44% |
| 1985-02-07 12:24 | 0.691 | 0.4 | 0.5 | 0.4 | 0.3 | 1.1 | 0.9 | 1.8 | 37% | 31% | 49% | 58% | 55% | 27% | 155% |
| 1988-02-19 23:25 | 17.309 | TOF | | | | | | | Error | | | | | Total | 768% |
| 1988-02-20 01:06 | 70.842 | 70.842 | | | | | | | 0% | | | | | Avg | 20% |
| 1988-02-20 04:18 | 18.433 | 27.8 | 18.43 | | | | | | 51% | 0% | | | | | |
| 1988-02-20 08:18 | 7.428 | 8.6 | 5.7 | | | | | | 16% | 23% | | | | | |
| 1988-02-20 13:06 | 3.661 | 2.1 | 1.4 | 3.66 | | | | | 42% | 62% | 0% | | | | |
| 1988-02-20 16:30 | 2.368 | 1.9 | 1.3 | 1.4 | | | | | 19% | 46% | 43% | | | | |
| 1988-02-20 21:24 | 1.67 | 1.7 | 1.1 | 1.2 | 1.67 | | | | 1% | 34% | 30% | 0% | | | |
| 1988-02-21 03:24 | 1.195 | 1.4 | 0.9 | 1.0 | 1.4 | | | | 16% | 23% | 18% | 17% | | | |
| 1988-02-21 13:24 | 0.766 | 1.0 | 0.7 | 0.7 | 1.0 | 0.77 | | | 36% | 10% | 4% | 37% | 0% | | |
| 1988-02-22 01:18 | 0.545 | 0.7 | 0.5 | 0.5 | 0.7 | 0.5 | | | 35% | 11% | 5% | 35% | 1% | | |
| 1988-02-22 17:48 | 0.414 | 0.5 | 0.3 | 0.3 | 0.5 | 0.3 | | | 9% | 27% | 23% | 10% | 19% | | |
| 1988-02-23 10:24 | 0.371 | 0.3 | 0.4 | 0.4 | 0.3 | 0.4 | | | 25% | 6% | 6% | 24% | 6% | | |
| 1988-02-24 08:06 | 0.331 | | | | | | | | | | | | | | |
| 1988-02-24 08:54 | 0.339 | | | | | | | | | | | | | | |
| 1988-02-24 11:06 | 0.514 | | | | | | | | | | | | | | |
| 1988-02-24 11:54 | 0.529 | | | | | | | | | | | | | | |

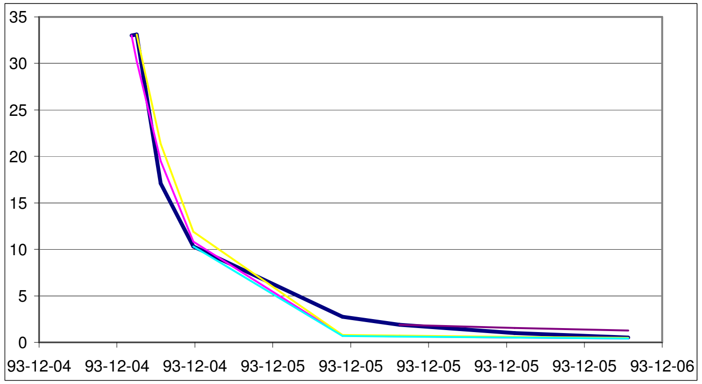


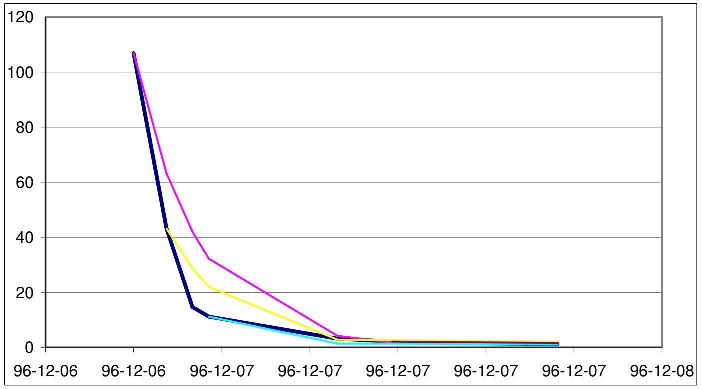



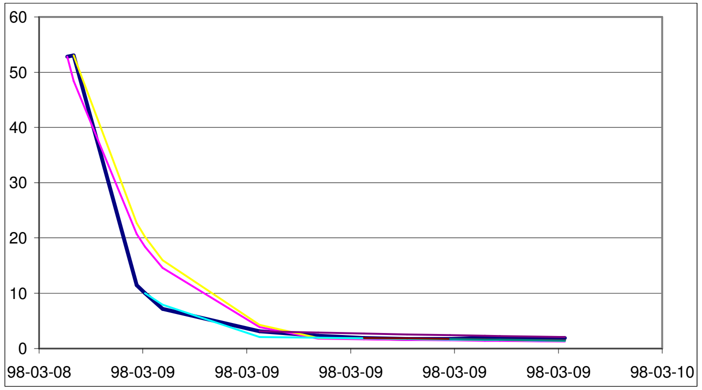
| | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1991-11-10 19:18 | 17.728 | TOF | | | | | | | Error | | | | | | | Total | 3471% |
| 1991-11-10 20:00 | 36.615 | 36.615 | | | | | | | 0% | | | | | | | Avg | 56% |
| 1991-11-10 21:18 | 22.863 | 25.0 | 22.86 | | | | | | 9% | 0% | | | | | | | |
| 1991-11-10 21:30 | 22.783 | 23.6 | 21.6 | | | | | | 4% | 5% | | | | | | | |
| 1991-11-10 21:54 | 24.137 | 21.0 | 19.2 | 24.1 | | | | | 13% | 21% | 0% | | | | | | |
| 1991-11-10 23:30 | 17.173 | 13.1 | 12.0 | 15.1 | | | | | 23% | 30% | 12% | | | | | | |
| 1991-11-11 00:54 | 11.618 | 8.7 | 8.0 | 10.0 | 11.6 | | | | 25% | 31% | 14% | 0% | | | | | |
| 1991-11-11 05:00 | 5.199 | 2.6 | 2.4 | 3.0 | 3.5 | | | | 49% | 54% | 42% | 33% | | | | | |
| 1991-11-11 08:30 | 2.922 | 2.4 | 2.2 | 2.7 | 1.3 | 2.92 | | | 19% | 26% | 7% | 57% | 0% | | | | |
| 1991-11-11 10:00 | 2.381 | 2.3 | 2.1 | 2.6 | 1.2 | 2.8 | | | 5% | 13% | 10% | 49% | 17% | | | | |
| 1991-11-11 13:42 | 1.55 | 2.0 | 1.9 | 2.3 | 1.1 | 2.5 | 1.55 | | 32% | 20% | 51% | 30% | 62% | 0% | | | |
| 1991-11-11 19:06 | 1.019 | 1.7 | 1.6 | 2.0 | 0.9 | 2.1 | 1.3 | | 71% | 56% | 96% | 10% | 110% | 30% | | | |
| 1991-11-12 01:00 | 0.685 | 1.5 | 1.3 | 1.7 | 0.8 | 1.8 | 1.1 | 0.69 | 114% | 95% | 146% | 13% | 163% | 63% | 0% | | |
| 1991-11-12 06:00 | 0.519 | 1.3 | 1.2 | 1.5 | 0.7 | 1.6 | 1.0 | 0.6 | 144% | 123% | 180% | 29% | 200% | 85% | 14% | | |
| 1991-11-12 14:48 | 0.355 | 1.0 | 0.9 | 1.1 | 0.5 | 1.2 | 0.7 | 0.5 | 175% | 152% | 217% | 46% | 239% | 109% | 29% | | |
| 1991-11-13 03:06 | 0.265 | | | | | | | | | | | | | | | | |
| 1993-12-04 14:24 | 15.038 | TOF | | | | | | | Error | | | | | | | Total | 856% |
| 1993-12-04 15:18 | 33.038 | 33.038 | | | | | | | 0% | | | | | | | Avg | 37% |
| 1993-12-04 15:36 | 33.128 | 30.3 | 33.13 | | | | | | 9% | 0% | | | | | | | |
| 1993-12-04 17:06 | 17.103 | 19.5 | 21.4 | | | | | | 14% | 25% | | | | | | | |
| 1993-12-04 19:06 | 10.347 | 10.9 | 11.9 | 10.3 | | | | | 5% | 15% | 0% | | | | | | |
| 1993-12-05 04:18 | 2.767 | 0.7 | 0.8 | 0.7 | | | | | 73% | 71% | 75% | | | | | | |
| 1993-12-05 07:48 | 1.905 | 0.7 | 0.7 | 0.6 | 1.91 | | | | 65% | 62% | 67% | 0% | | | | | |
| 1993-12-05 14:54 | 1.019 | 0.5 | 0.6 | 0.5 | 1.5 | | | | 47% | 42% | 50% | 52% | | | | | |
| 1993-12-05 21:54 | 0.55 | 0.4 | 0.5 | 0.4 | 1.3 | | | | 20% | 12% | 24% | 129% | | | | | |
| 1993-12-06 05:06 | 0.319 | | | | | | | | | | | | | | | | |
| 1993-12-06 14:42 | 0.226 | | | | | | | | | | | | | | | | |
| 1993-12-07 16:24 | 0.129 | | | | | | | | | | | | | | | | |
| 1993-12-08 18:30 | 0.098 | | | | | | | | | | | | | | | | |
| 1993-12-09 03:54 | 0.102 | | | | | | | | | | | | | | | | |
| 1993-12-09 09:00 | 0.096 | | | | | | | | | | | | | | | | |
| 1993-12-09 15:48 | 0.087 | | | | | | | | | | | | | | | | |
| 1996-12-06 16:12 | 0.852 | TOF | | | | | | | Error | | | | | | | Total | 1046% |
| 1996-12-06 19:12 | 106.91 | 106.910 | | | | | | | 0% | | | | | | | Avg | 45% |
| 1996-12-06 21:00 | 43.076 | 63.1 | 43.08 | | | | | | 47% | 0% | | | | | | | |
| 1996-12-06 22:24 | 14.717 | 41.9 | 28.6 | | | | | | 185% | 94% | | | | | | | |
| 1996-12-06 23:18 | 11.182 | 32.2 | 22.0 | 11.2 | | | | | 188% | 97% | 0% | | | | | | |
| 1996-12-07 06:18 | 3.082 | 4.2 | 2.8 | 1.4 | | | | | 35% | 8% | 53% | | | | | | |
| 1996-12-07 09:18 | 2.099 | 1.7 | 2.6 | 1.3 | 2.1 | | | | 18% | 24% | 37% | 0% | | | | | |
| 1996-12-07 13:12 | 1.472 | 1.5 | 2.3 | 1.2 | 1.9 | | | | 5% | 57% | 20% | 27% | | | | | |
| 1996-12-07 18:18 | 1.113 | 1.3 | 2.0 | 1.0 | 1.6 | | | | 19% | 79% | 9% | 45% | | | | | |
| 1996-12-08 04:06 | 0.798 | | | | | | | | | | | | | | | | |
| 1996-12-08 16:18 | 0.621 | | | | | | | | | | | | | | | | |
| 1996-12-08 22:42 | 0.571 | | | | | | | | | | | | | | | | |
| 1996-12-09 02:24 | 0.484 | | | | | | | | | | | | | | | | |
| 1996-12-09 07:30 | 0.427 | | | | | | | | | | | | | | | | |
| 1996-12-09 08:54 | 0.423 | | | | | | | | | | | | | | | | |
| 1996-12-09 09:48 | 0.446 | | | | | | | | | | | | | | | | |

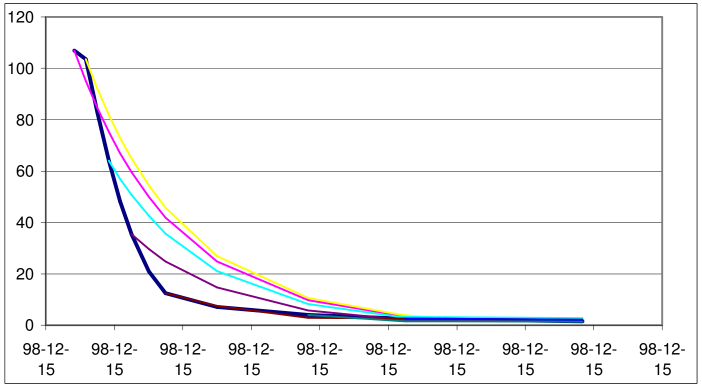


| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1998-03-08 20:12 | 50.169 | TOF | | | | | | | Error | | | | | Total | 1336% |
| 1998-03-08 20:30 | 52.823 | 52.823 | | | | | | | 0% | | | | | Avg | 25% |
| 1998-03-08 20:48 | 53.041 | 48.4 | 53.04 | | | | | | 9% | 0% | | | | | |
| 1998-03-08 23:42 | 11.453 | 20.7 | 22.7 | | | | | | 81% | 98% | | | | | |
| 1998-03-09 00:06 | 9.996 | 18.4 | 20.2 | 10 | | | | | 84% | 102% | 0% | | | | |
| 1998-03-09 00:54 | 7.192 | 14.6 | 16.0 | 7.9 | | | | | 103% | 122% | 10% | | | | |
| 1998-03-09 05:24 | 3.112 | 3.9 | 4.3 | 2.1 | 3.11 | | | | 26% | 38% | 32% | 0% | | | |
| 1998-03-09 08:06 | 2.305 | 1.8 | 1.9 | 2.0 | 2.9 | | | | 23% | 16% | 15% | 25% | | | |
| 1998-03-09 10:12 | 1.939 | 1.7 | 1.8 | 1.8 | 2.7 | 1.94 | | | 14% | 6% | 5% | 39% | 0% | | |
| 1998-03-09 12:12 | 1.711 | 1.6 | 1.7 | 1.7 | 2.6 | 1.8 | | | 8% | 1% | 2% | 49% | 7% | | |
| 1998-03-09 14:12 | 1.711 | 1.5 | 1.6 | 1.6 | 2.4 | 1.7 | 1.71 | | 13% | 5% | 4% | 41% | 1% | 0% | |
| 1998-03-09 15:48 | 1.872 | 1.4 | 1.6 | 1.6 | 2.3 | 1.6 | 1.6 | | 24% | 17% | 16% | 23% | 12% | 13% | |
| 1998-03-09 16:30 | 1.894 | 1.4 | 1.5 | 1.5 | 2.2 | 1.6 | 1.6 | | 27% | 20% | 19% | 19% | 15% | 16% | |
| 1998-03-09 19:30 | 1.894 | 1.3 | 1.4 | 1.4 | 2.1 | 1.5 | 1.5 | | 33% | 27% | 26% | 9% | 22% | 23% | |
| 1998-03-09 23:36 | 1.796 | | | | | | | | | | | | | | |
| 1998-03-10 02:42 | 1.619 | | | | | | | | | | | | | | |
| 1998-12-14 23:12 | 106.91 | TOF | | | | | | | Error | | | | | Total | 3542% |
| 1998-12-15 01:00 | 106.91 | 106.910 | | | | | | | 0% | | | | | Avg | 56% |
| 1998-12-15 01:24 | 103.596 | 95.1 | 103.6 | | | | | | 8% | 0% | | | | | |
| 1998-12-15 01:48 | 82.9 | 84.6 | 92.2 | | | | | | 2% | 11% | | | | | |
| 1998-12-15 02:12 | 64.077 | 75.3 | 82.0 | 64.1 | | | | | 17% | 28% | 0% | | | | |
| 1998-12-15 02:36 | 48.219 | 66.9 | 72.9 | 57.0 | | | | | 39% | 51% | 18% | | | | |
| 1998-12-15 03:00 | 35.376 | 59.5 | 64.9 | 50.7 | 35.4 | | | | 68% | 83% | 43% | 0% | | | |
| 1998-12-15 03:36 | 20.931 | 50.0 | 54.4 | 42.5 | 29.7 | | | | 139% | 160% | 103% | 42% | | | |
| 1998-12-15 04:12 | 12.524 | 41.9 | 45.7 | 35.7 | 24.9 | 12.5 | | | 235% | 265% | 185% | 99% | 0% | | |
| 1998-12-15 06:00 | 7.059 | 24.8 | 27.0 | 21.1 | 14.7 | 7.4 | | | 251% | 282% | 199% | 108% | 5% | | |
| 1998-12-15 09:12 | 3.965 | 9.7 | 10.6 | 8.3 | 5.8 | 2.9 | 3.97 | | 145% | 167% | 108% | 45% | 27% | 0% | |
| 1998-12-15 12:24 | 2.616 | 3.8 | 4.1 | 3.2 | 2.3 | 2.6 | 1.6 | | 45% | 58% | 24% | 14% | 1% | 41% | |
| 1998-12-15 12:36 | 2.589 | 3.6 | 3.9 | 3.2 | 2.2 | 2.6 | 1.5 | 2.59 | 39% | 51% | 24% | 13% | 1% | 40% | 0% |
| 1998-12-15 13:48 | 2.281 | 2.5 | 2.8 | 3.1 | 2.2 | 2.5 | 1.5 | 2.5 | 11% | 21% | 36% | 5% | 11% | 35% | 10% |
| 1998-12-15 14:00 | 2.256 | 2.5 | 2.7 | 3.1 | 2.2 | 2.5 | 1.5 | 2.5 | 11% | 21% | 37% | 4% | 12% | 34% | 10% |
| 1998-12-15 18:48 | 1.501 | 2.2 | 2.4 | 2.7 | 1.9 | 2.2 | 1.3 | 2.2 | 45% | 58% | 79% | 25% | 46% | 14% | 44% |



# Calibration of K value for Baviaans River at de Klerksdal Q6H003

**Data**

**Q6H003**

Mode 2.388
Mean 22.565
Min 0.006
Max 241.260

**Forecast**

$B_f$ 0.050 Tf 0.50
$K_U$ 1.77 $K_L$ 18

Basic Forecast Equation is defined as:

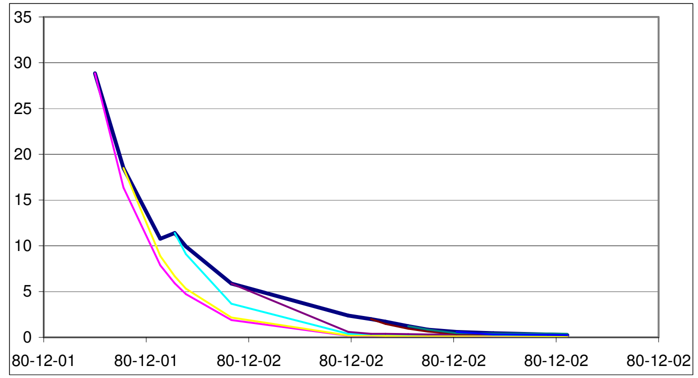
$$q_2 = q_1 \times \exp^{(-dt/k)}$$

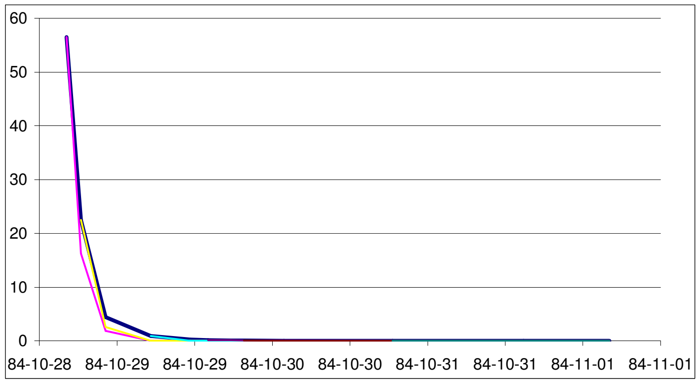
**Error Calculation**

**Total AVG** 536%

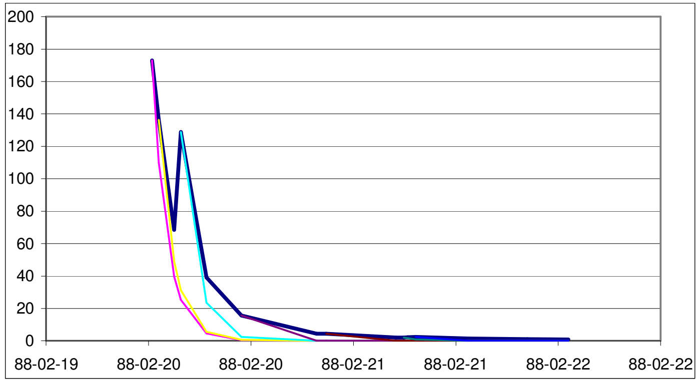
| Date | Discharge | Forecast | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1980-12-01 19:24 | 7.478 | TOF | | | | | | | Error | | | | | Total | 2873% |
| 1980-12-01 19:48 | 28.873 | 28.873 | | | | | | | 0% | | | | | Avg | 44% |
| 1980-12-01 20:48 | 18.511 | 16.4 | 18.51 | | | | | | 11% | 0% | | | | | |
| 1980-12-01 22:06 | 10.778 | 7.8 | 8.9 | | | | | | 27% | 18% | | | | | |
| 1980-12-01 22:36 | 11.393 | 5.9 | 6.7 | 11.4 | | | | | 48% | 41% | 0% | | | | |
| 1980-12-01 23:00 | 9.925 | 4.7 | 5.3 | 9.1 | | | | | 53% | 46% | 8% | | | | |
| 1980-12-02 00:36 | 5.869 | 1.9 | 2.2 | 3.7 | 5.87 | | | | 68% | 63% | 37% | 0% | | | |
| 1980-12-02 04:42 | 2.363 | 0.2 | 0.2 | 0.4 | 0.6 | | | | 92% | 91% | 85% | 76% | | | |
| 1980-12-02 05:30 | 1.969 | 0.2 | 0.2 | 0.3 | 0.4 | 1.97 | | | 91% | 90% | 83% | 81% | 0% | | |
| 1980-12-02 06:00 | 1.719 | 0.2 | 0.2 | 0.3 | 0.4 | 1.5 | | | 90% | 89% | 81% | 79% | 14% | | |
| 1980-12-02 06:48 | 1.203 | 0.2 | 0.2 | 0.3 | 0.3 | 0.9 | 1.2 | | 86% | 84% | 73% | 72% | 22% | 0% | |
| 1980-12-02 07:30 | 0.839 | 0.2 | 0.2 | 0.3 | 0.3 | 0.6 | 0.8 | | 81% | 79% | 63% | 61% | 24% | 4% | |
| 1980-12-02 08:30 | 0.589 | 0.2 | 0.2 | 0.3 | 0.3 | 0.4 | 0.5 | 0.59 | 74% | 71% | 51% | 48% | 39% | 22% | 0% |
| 1980-12-02 09:48 | 0.447 | 0.1 | 0.2 | 0.3 | 0.3 | 0.3 | 0.4 | 0.3 | 69% | 65% | 40% | 36% | 25% | 5% | 37% |
| 1980-12-02 11:42 | 0.284 | 0.1 | 0.1 | 0.2 | 0.3 | 0.3 | 0.4 | 0.3 | 56% | 50% | 15% | 9% | 6% | 35% | 11% |
| 1980-12-02 12:24 | 0.247 | 0.1 | 0.1 | 0.2 | 0.2 | 0.3 | 0.4 | 0.2 | 51% | 45% | 6% | 0% | 17% | 49% | 1% |
| 1984-10-28 16:00 | 0.014 | TOF | | | | | | | Error | | | | | Total | 4894% |
| 1984-10-28 16:12 | 56.521 | 56.521 | | | | | | | 0% | | | | | Avg | 104% |
| 1984-10-28 18:24 | 22.511 | 16.3 | 22.51 | | | | | | 28% | 0% | | | | | |
| 1984-10-28 22:12 | 4.414 | 1.9 | 2.6 | | | | | | 57% | 41% | | | | | |
| 1984-10-29 05:12 | 0.952 | 0.0 | 0.0 | 0.95 | | | | | 96% | 95% | 0% | | | | |
| 1984-10-29 11:18 | 0.26 | 0.1 | 0.1 | 0.0 | | | | | 81% | 81% | 88% | | | | |
| 1984-10-29 14:06 | 0.151 | 0.1 | 0.1 | 0.1 | 0.15 | | | | 67% | 67% | 67% | 0% | | | |
| 1984-10-29 17:36 | 0.097 | 0.1 | 0.1 | 0.1 | 0.1 | | | | 48% | 48% | 48% | 28% | | | |
| 1984-10-29 19:36 | 0.087 | 0.1 | 0.1 | 0.1 | 0.1 | 0.09 | | | 43% | 43% | 43% | 27% | 0% | | |
| 1984-10-30 01:48 | 0.05 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | | | 0% | 0% | 0% | 56% | 22% | | |
| 1984-10-30 18:36 | 0.021 | 0.1 | 0.1 | 0.1 | 0.0 | 0.0 | 0.02 | | 138% | 138% | 138% | 43% | 13% | 0% | |
| 1984-10-31 14:48 | 0.013 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | | 285% | 285% | 285% | 285% | 285% | 285% | |
| 1984-11-01 04:06 | 0.014 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | | 257% | 257% | 257% | 257% | 257% | 257% | |
| 1984-11-01 04:30 | 0.016 | | | | | | | | | | | | | | |
| 1984-11-01 04:42 | 0.181 | | | | | | | | | | | | | | |
| 1984-11-01 04:48 | 0.204 | | | | | | | | | | | | | | |

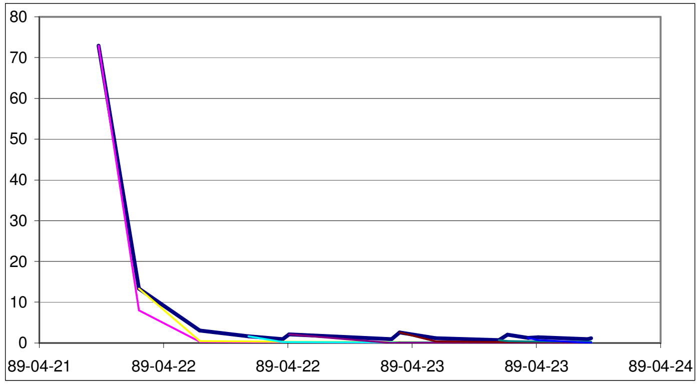
**Plots of Data vs Forecast**

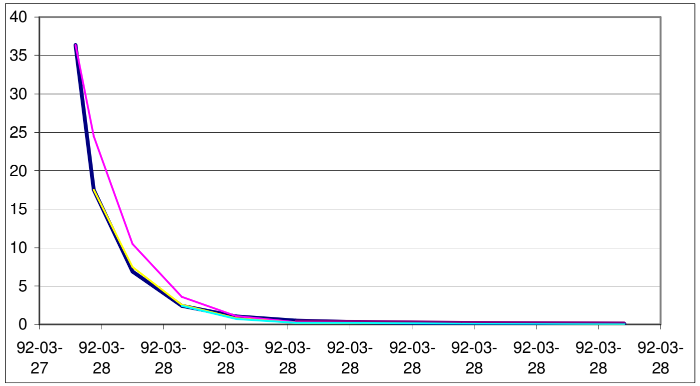


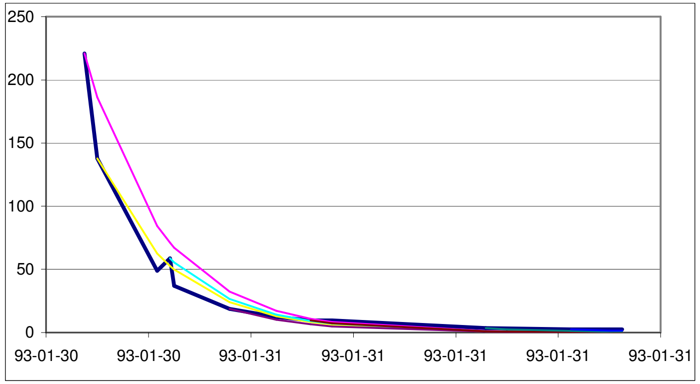
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1988-02-19 23:48 | 136.125 | TOF | | | | | | | Error | | | | | Total | 4884% |
| 1988-02-20 00:24 | 173.085 | 173.085 | | | | | | | 0% | | | | | Avg | 80% |
| 1988-02-20 01:12 | 136.287 | 110.0 | 136.3 | | | | | | 19% | 0% | | | | | |
| 1988-02-20 03:00 | 68.568 | 39.7 | 49.2 | | | | | | 42% | 28% | | | | | |
| 1988-02-20 03:48 | 128.908 | 25.2 | 31.3 | 129 | | | | | 80% | 76% | 0% | | | | |
| 1988-02-20 06:48 | 39.073 | 4.6 | 5.7 | 23.6 | | | | | 88% | 85% | 40% | | | | |
| 1988-02-20 10:54 | 15.594 | 0.5 | 0.6 | 2.3 | 15.6 | | | | 97% | 96% | 85% | 0% | | | |
| 1988-02-20 19:42 | 4.435 | 0.3 | 0.0 | 0.0 | 0.1 | | | | 94% | 100% | 100% | 98% | | | |
| 1988-02-20 20:48 | 4.435 | 0.3 | 0.1 | 0.1 | 0.1 | 4.44 | | | 94% | 99% | 99% | 98% | 0% | | |
| 1988-02-21 05:00 | 2.131 | 0.2 | 0.1 | 0.1 | 0.1 | 0.0 | | | 92% | 98% | 98% | 97% | 98% | | |
| 1988-02-21 05:54 | 2.131 | 0.2 | 0.1 | 0.1 | 0.1 | 0.1 | 2.13 | | 93% | 98% | 98% | 97% | 98% | 0% | |
| 1988-02-21 06:24 | 2.3 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 1.6 | | 93% | 98% | 98% | 97% | 98% | 30% | |
| 1988-02-21 07:18 | 2.313 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 1.0 | 2.31 | 94% | 98% | 98% | 98% | 98% | 58% | 0% |
| 1988-02-21 13:30 | 1.538 | 0.1 | 0.1 | 0.1 | 0.0 | 0.1 | 0.0 | 0.1 | 93% | 97% | 97% | 97% | 97% | 98% | 96% |
| 1988-02-21 20:24 | 1.105 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.0 | 94% | 95% | 95% | 95% | 95% | 95% | 96% |
| 1988-02-22 01:12 | 0.923 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 94% | 95% | 95% | 95% | 95% | 95% | 95% |
| 1989-04-21 17:30 | 15.306 | TOF | | | | | | | Error | | | | | Total | 4755% |
| 1989-04-21 17:42 | 72.878 | 72.878 | | | | | | | 0% | | | | | Avg | 78% |
| 1989-04-21 21:36 | 13.372 | 8.0 | 13.37 | | | | | | 40% | 0% | | | | | |
| 1989-04-22 03:30 | 3.076 | 0.3 | 0.5 | | | | | | 91% | 85% | | | | | |
| 1989-04-22 08:12 | 1.658 | 0.2 | 0.4 | 1.66 | | | | | 87% | 78% | 0% | | | | |
| 1989-04-22 11:30 | 0.867 | 0.2 | 0.3 | 0.3 | | | | | 79% | 65% | 71% | | | | |
| 1989-04-22 12:06 | 2.06 | 0.2 | 0.3 | 0.2 | 2.06 | | | | 92% | 86% | 88% | 0% | | | |
| 1989-04-22 22:00 | 0.909 | 0.1 | 0.2 | 0.1 | 0.0 | | | | 89% | 82% | 84% | 99% | | | |
| 1989-04-22 22:48 | 2.53 | 0.1 | 0.2 | 0.1 | 0.1 | 2.53 | | | 96% | 94% | 95% | 98% | 0% | | |
| 1989-04-23 02:18 | 1.121 | 0.1 | 0.1 | 0.1 | 0.1 | 0.3 | | | 93% | 88% | 90% | 96% | 69% | | |
| 1989-04-23 08:24 | 0.674 | 0.1 | 0.1 | 0.1 | 0.1 | 0.2 | 0.67 | | 92% | 86% | 88% | 93% | 63% | 0% | |
| 1989-04-23 09:12 | 2.003 | 0.1 | 0.1 | 0.1 | 0.1 | 0.2 | 0.4 | | 97% | 96% | 96% | 98% | 88% | 79% | |
| 1989-04-23 11:12 | 1.263 | 0.0 | 0.1 | 0.1 | 0.1 | 0.2 | 0.4 | 1.26 | 96% | 94% | 95% | 96% | 83% | 70% | 0% |
| 1989-04-23 12:12 | 1.333 | 0.1 | 0.1 | 0.1 | 0.1 | 0.2 | 0.4 | 0.7 | 96% | 94% | 95% | 96% | 85% | 73% | 46% |
| 1989-04-23 17:00 | 0.902 | 0.1 | 0.1 | 0.0 | 0.1 | 0.2 | 0.3 | 0.0 | 94% | 94% | 95% | 94% | 83% | 69% | 95% |
| 1989-04-23 17:18 | 1.145 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.3 | 0.1 | 96% | 95% | 96% | 96% | 87% | 76% | 96% |
| 1992-03-27 22:24 | 16.088 | TOF | | | | | | | Error | | | | | Total | 1379% |
| 1992-03-27 23:00 | 36.36 | 36.360 | | | | | | | 0% | | | | | Avg | 51% |
| 1992-03-27 23:42 | 17.509 | 24.5 | 17.51 | | | | | | 40% | 0% | | | | | |
| 1992-03-28 01:12 | 6.863 | 10.5 | 7.5 | | | | | | 52% | 9% | | | | | |
| 1992-03-28 03:06 | 2.414 | 3.6 | 2.6 | 2.41 | | | | | 48% | 6% | 0% | | | | |
| 1992-03-28 05:12 | 1.05 | 1.1 | 0.8 | 0.7 | | | | | 3% | 26% | 30% | | | | |
| 1992-03-28 07:30 | 0.495 | 0.3 | 0.2 | 0.2 | 0.5 | | | | 40% | 57% | 60% | 0% | | | |
| 1992-03-28 10:00 | 0.277 | 0.3 | 0.2 | 0.2 | 0.4 | | | | 8% | 34% | 37% | 55% | | | |
| 1992-03-28 13:54 | 0.137 | 0.2 | 0.1 | 0.1 | 0.3 | | | | 50% | 7% | 1% | 151% | | | |
| 1992-03-28 20:12 | 0.055 | 0.1 | 0.1 | 0.1 | 0.2 | | | | 161% | 87% | 77% | 338% | | | |
| 1992-03-29 03:42 | 0.028 | | | | | | | | | | | | | | |
| 1992-03-29 16:48 | 0.011 | | | | | | | | | | | | | | |
| 1992-03-30 12:00 | 0.01 | | | | | | | | | | | | | | |
| 1992-03-31 06:18 | 0.009 | | | | | | | | | | | | | | |
| 1992-03-31 10:12 | 0.007 | | | | | | | | | | | | | | |
| 1992-03-31 15:12 | 0.006 | | | | | | | | | | | | | | |

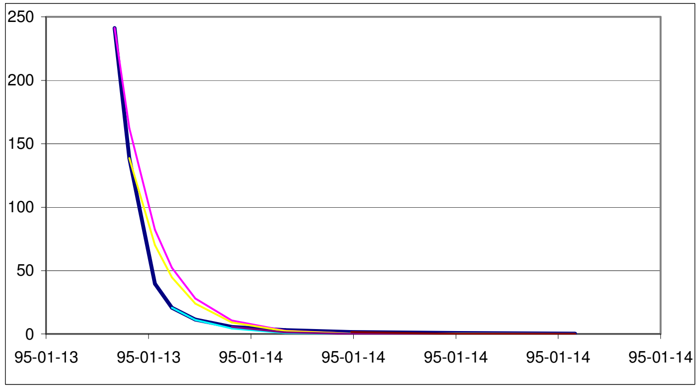


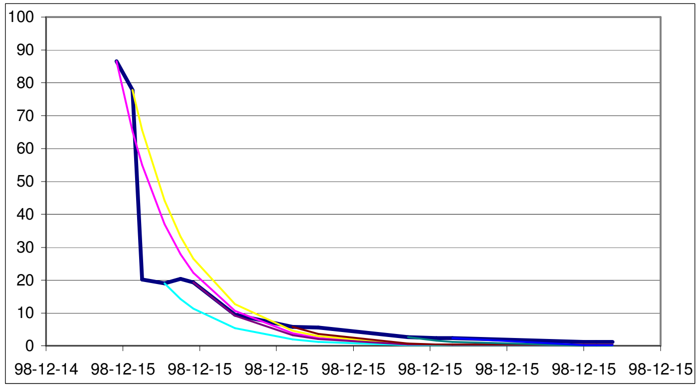



| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1993-01-30 19:12 | 137.847 | TOF | | | | | | | Error | | | | | Total | 3137% |
| 1993-01-30 20:06 | 220.741 | 220.741 | | | | | | | 0% | | | | | Avg | 54% |
| 1993-01-30 20:24 | 137.847 | 186.2 | 137.8 | | | | | | 35% | 0% | | | | | |
| 1993-01-30 21:48 | 48.87 | 84.3 | 62.4 | | | | | | 72% | 28% | | | | | |
| 1993-01-30 22:06 | 58.532 | 71.1 | 52.6 | 58.5 | | | | | 21% | 10% | 0% | | | | |
| 1993-01-30 22:12 | 36.963 | 67.2 | 49.7 | 55.3 | | | | | 82% | 35% | 50% | | | | |
| 1993-01-30 23:30 | 18.766 | 32.2 | 23.8 | 26.5 | 18.8 | | | | 71% | 27% | 41% | 0% | | | |
| 1993-01-31 00:36 | 12.484 | 17.3 | 12.8 | 14.2 | 10.1 | | | | 38% | 2% | 14% | 19% | | | |
| 1993-01-31 01:24 | 9.626 | 11.0 | 8.1 | 9.0 | 6.4 | 9.63 | | | 14% | 16% | 6% | 34% | 0% | | |
| 1993-01-31 01:54 | 9.584 | 8.3 | 6.1 | 6.8 | 4.8 | 7.3 | | | 14% | 36% | 29% | 50% | 24% | | |
| 1993-01-31 05:30 | 3.398 | 1.1 | 0.8 | 0.9 | 0.6 | 0.9 | 3.4 | | 68% | 77% | 74% | 82% | 72% | 0% | |
| 1993-01-31 06:00 | 3.242 | 0.8 | 0.6 | 0.7 | 0.5 | 0.7 | 2.6 | | 75% | 82% | 79% | 85% | 78% | 21% | |
| 1993-01-31 07:30 | 2.517 | 0.3 | 0.3 | 0.3 | 0.4 | 0.3 | 1.1 | 2.52 | 86% | 90% | 89% | 83% | 88% | 57% | 0% |
| 1993-01-31 08:12 | 2.491 | 0.3 | 0.2 | 0.3 | 0.4 | 0.3 | 0.7 | 1.7 | 87% | 90% | 89% | 83% | 88% | 70% | 32% |
| 1993-01-31 08:24 | 2.388 | 0.3 | 0.2 | 0.3 | 0.4 | 0.3 | 0.7 | 1.5 | 86% | 90% | 89% | 83% | 88% | 72% | 37% |
| 1993-01-31 08:42 | 2.388 | 0.3 | 0.2 | 0.3 | 0.4 | 0.3 | 0.6 | 1.3 | 86% | 90% | 89% | 83% | 88% | 77% | 47% |
| 1995-01-13 17:24 | 138.627 | TOF | | | | | | | Error | | | | | Total | 2068% |
| 1995-01-13 17:36 | 241.26 | 241.260 | | | | | | | 0% | | | | | Avg | 61% |
| 1995-01-13 18:18 | 138.627 | 162.3 | 138.6 | | | | | | 17% | 0% | | | | | |
| 1995-01-13 19:30 | 39.61 | 82.2 | 70.3 | | | | | | 108% | 77% | | | | | |
| 1995-01-13 20:18 | 20.868 | 52.3 | 44.7 | 20.9 | | | | | 151% | 114% | 0% | | | | |
| 1995-01-13 21:24 | 11.44 | 28.0 | 23.9 | 11.2 | | | | | 145% | 109% | 2% | | | | |
| 1995-01-13 23:06 | 5.784 | 10.7 | 9.1 | 4.3 | 5.78 | | | | 85% | 58% | 26% | 0% | | | |
| 1995-01-14 01:36 | 2.973 | 2.6 | 2.2 | 1.0 | 1.4 | | | | 13% | 25% | 65% | 53% | | | |
| 1995-01-14 04:42 | 1.688 | 0.4 | 0.4 | 0.2 | 0.2 | 1.69 | | | 73% | 77% | 89% | 86% | 0% | | |
| 1995-01-14 09:42 | 0.78 | 0.3 | 0.3 | 0.1 | 0.2 | 0.1 | | | 57% | 63% | 83% | 77% | 87% | | |
| 1995-01-14 15:12 | 0.443 | 0.2 | 0.2 | 0.1 | 0.1 | 0.1 | | | 44% | 52% | 78% | 70% | 84% | | |
| 1995-01-14 15:18 | 36.444 | | | | | | | | | | | | | | |
| 1995-01-14 16:36 | 13.213 | | | | | | | | | | | | | | |
| 1995-01-14 16:42 | 37.521 | | | | | | | | | | | | | | |
| 1995-01-14 18:30 | 18.133 | | | | | | | | | | | | | | |
| 1995-01-14 19:06 | 22.932 | | | | | | | | | | | | | | |
| 1998-12-14 23:42 | 65.983 | TOF | | | | | | | Error | | | | | Total | 3811% |
| 1998-12-14 23:48 | 86.587 | 86.587 | | | | | | | 0% | | | | | Avg | 63% |
| 1998-12-15 00:18 | 77.779 | 65.2 | 77.78 | | | | | | 16% | 0% | | | | | |
| 1998-12-15 00:36 | 20.199 | 55.0 | 65.6 | | | | | | 172% | 225% | | | | | |
| 1998-12-15 01:18 | 18.96 | 37.0 | 44.1 | 19 | | | | | 95% | 133% | 0% | | | | |
| 1998-12-15 01:48 | 20.333 | 27.9 | 33.3 | 14.3 | | | | | 37% | 64% | 30% | | | | |
| 1998-12-15 02:12 | 19.282 | 22.2 | 26.5 | 11.4 | 19.3 | | | | 15% | 38% | 41% | 0% | | | |
| 1998-12-15 03:30 | 9.46 | 10.6 | 12.7 | 5.5 | 9.2 | | | | 13% | 34% | 42% | 2% | | | |
| 1998-12-15 05:18 | 5.756 | 3.8 | 4.6 | 2.0 | 3.3 | 5.76 | | | 33% | 20% | 66% | 42% | 0% | | |
| 1998-12-15 06:06 | 5.534 | 2.4 | 2.9 | 1.3 | 2.1 | 3.7 | | | 56% | 47% | 77% | 62% | 34% | | |
| 1998-12-15 08:54 | 2.711 | 0.5 | 0.6 | 0.3 | 0.4 | 0.7 | 2.71 | | 82% | 78% | 91% | 84% | 72% | 0% | |
| 1998-12-15 09:48 | 2.426 | 0.5 | 0.4 | 0.2 | 0.4 | 0.4 | 1.6 | | 80% | 85% | 90% | 83% | 81% | 33% | |
| 1998-12-15 10:18 | 2.388 | 0.5 | 0.3 | 0.2 | 0.4 | 0.4 | 1.2 | 2.39 | 81% | 85% | 90% | 83% | 82% | 49% | 0% |
| 1998-12-15 14:24 | 1.17 | 0.4 | 0.3 | 0.2 | 0.3 | 0.3 | 0.1 | 0.2 | 69% | 76% | 84% | 73% | 70% | 90% | 80% |
| 1998-12-15 14:48 | 1.161 | 0.4 | 0.3 | 0.2 | 0.3 | 0.3 | 0.1 | 0.2 | 69% | 77% | 84% | 73% | 71% | 90% | 80% |
| 1998-12-15 15:18 | 1.254 | 0.3 | 0.3 | 0.2 | 0.3 | 0.3 | 0.1 | 0.2 | 72% | 79% | 86% | 76% | 74% | 91% | 82% |



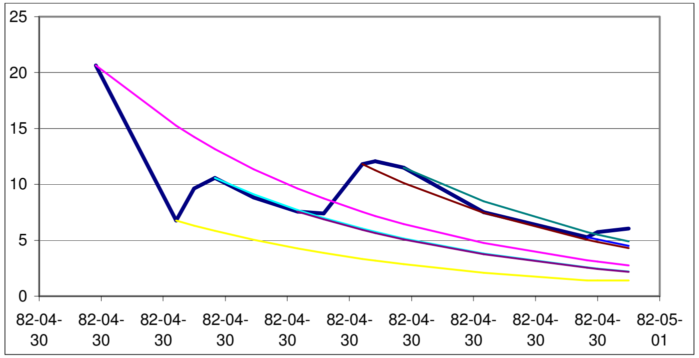



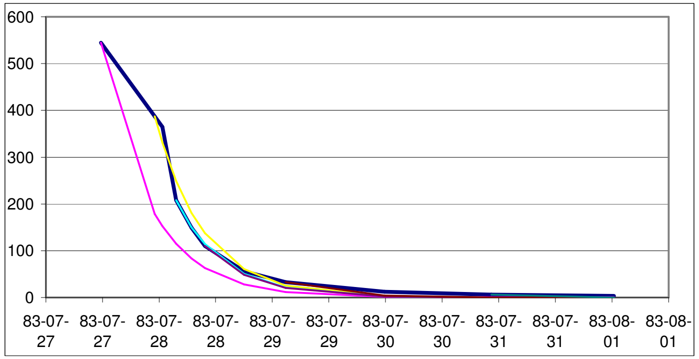



# Calibration of K value for Little Fish River at Doornkraal Q8H008

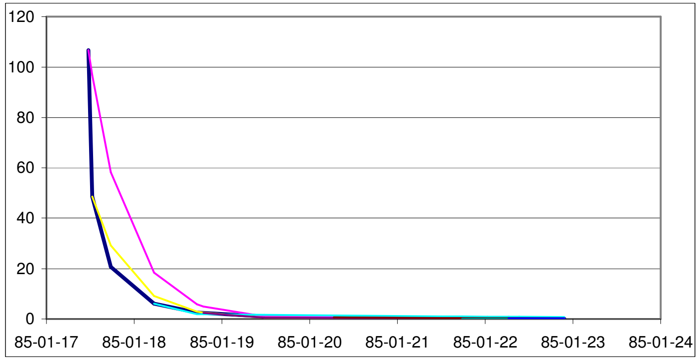
| **Data** | | **Forecast** | | | | **Error Calculation** | | **Plots of Data vs Forecast** |
|---|---|---|---|---|---|---|---|---|
| **Q8H008** | | $B_f$ | 0.200 | Tf | 2.00 | **Total AVG** | 41% | |
| Mode | 6.069 | $K_U$ | 10.23 | $K_L$ | 102 | | | |
| Mean | 80.897 | Basic Forecast Equation is defined as: | | | | | | |
| Min | 0.094 | | | | | | | |
| Max | 545.121 | $q_2 = q_1 \times \exp^{(-dt/k)}$ | | | | | | |

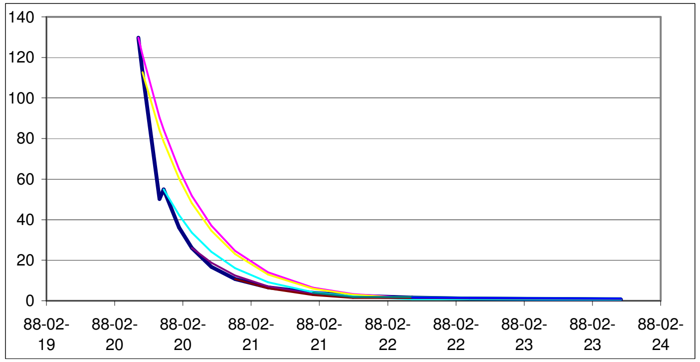
| Date | Discharge | Forecast | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1982-04-30 01:48 | 20.348 | TOF | | | | | | | Error | | | | | | Total | 2224% |
| 1982-04-30 02:12 | 20.645 | 20.645 | | | | | | | 0% | | | | | | Avg | 36% |
| 1982-04-30 05:18 | 6.779 | 15.2 | 6.779 | | | | | | 125% | 0% | | | | | | |
| 1982-04-30 06:00 | 9.642 | 14.2 | 6.3 | | | | | | 48% | 34% | | | | | | |
| 1982-04-30 06:48 | 10.564 | 13.2 | 5.9 | 10.6 | | | | | 25% | 45% | 0% | | | | | |
| 1982-04-30 08:18 | 8.852 | 11.4 | 5.1 | 9.1 | | | | | 28% | 43% | 3% | | | | | |
| 1982-04-30 10:00 | 7.578 | 9.6 | 4.3 | 7.7 | 7.58 | | | | 27% | 43% | 2% | 0% | | | | |
| 1982-04-30 11:00 | 7.407 | 8.7 | 3.9 | 7.0 | 6.9 | | | | 18% | 48% | 5% | 7% | | | | |
| 1982-04-30 12:30 | 11.821 | 7.5 | 3.4 | 6.1 | 5.9 | 11.8 | | | 36% | 72% | 49% | 50% | 0% | | | |
| 1982-04-30 13:00 | 12.056 | 7.2 | 3.2 | 5.8 | 5.7 | 11.3 | | | 40% | 74% | 52% | 53% | 7% | | | |
| 1982-04-30 14:06 | 11.498 | 6.5 | 2.9 | 5.2 | 5.1 | 10.1 | 11.5 | | 44% | 75% | 55% | 56% | 12% | 0% | | |
| 1982-04-30 17:12 | 7.509 | 4.8 | 2.1 | 3.8 | 3.7 | 7.5 | 8.5 | | 37% | 72% | 49% | 50% | 1% | 13% | | |
| 1982-04-30 21:12 | 5.275 | 3.2 | 1.4 | 2.6 | 2.5 | 5.1 | 5.7 | 5.28 | 39% | 73% | 51% | 52% | 4% | 9% | | 0% |
| 1982-04-30 21:36 | 5.748 | 3.1 | 1.4 | 2.5 | 2.4 | 4.9 | 5.5 | 5.1 | 46% | 75% | 57% | 58% | 16% | 4% | | 12% |
| 1982-04-30 22:48 | 6.069 | 2.8 | 1.4 | 2.2 | 2.2 | 4.3 | 4.9 | 4.5 | 55% | 77% | 64% | 64% | 29% | 19% | | 26% |
| 1983-07-27 05:48 | 526.286 | TOF | | | | | | | Error | | | | | | Total | 1937% |
| 1983-07-27 11:42 | 545.121 | 545.121 | | | | | | | 0% | | | | | | Avg | 47% |
| 1983-07-27 23:06 | 387.319 | 178.9 | 387.3 | | | | | | 54% | 0% | | | | | | |
| 1983-07-28 00:42 | 364.805 | 153.0 | 331.2 | | | | | | 58% | 9% | | | | | | |
| 1983-07-28 03:36 | 207.896 | 115.2 | 249.5 | 208 | | | | | 45% | 20% | 0% | | | | | |
| 1983-07-28 06:54 | 149.115 | 83.4 | 180.7 | #### | | | | | 44% | 21% | 1% | | | | | |
| 1983-07-28 09:42 | 110.389 | 63.5 | 137.4 | #### | 110 | | | | 43% | 24% | 4% | 0% | | | | |
| 1983-07-28 18:00 | 56.355 | 28.2 | 61.1 | 50.9 | 49.0 | | | | 50% | 8% | 10% | 13% | | | | |
| 1983-07-29 02:48 | 33.094 | 11.9 | 25.8 | 21.5 | 20.7 | 33.1 | | | 64% | 22% | 35% | 37% | 0% | | | |
| 1983-07-29 23:48 | 13.025 | 1.5 | 3.3 | 2.8 | 2.7 | 4.2 | | | 88% | 75% | 79% | 80% | 67% | | | |
| 1983-07-30 22:24 | 6.069 | 1.2 | 0.4 | 0.3 | 0.3 | 0.5 | 6.07 | | 80% | 94% | 95% | 95% | 92% | 0% | | |
| 1983-08-01 00:24 | 3.632 | 1.0 | 0.3 | 0.2 | 0.2 | 0.4 | 0.5 | | 74% | 92% | 94% | 94% | 90% | 87% | | |
| 1983-08-02 10:48 | 2.124 | | | | | | | | | | | | | | | |
| 1983-08-03 06:24 | 1.388 | | | | | | | | | | | | | | | |
| 1983-08-04 11:18 | 0.793 | | | | | | | | | | | | | | | |

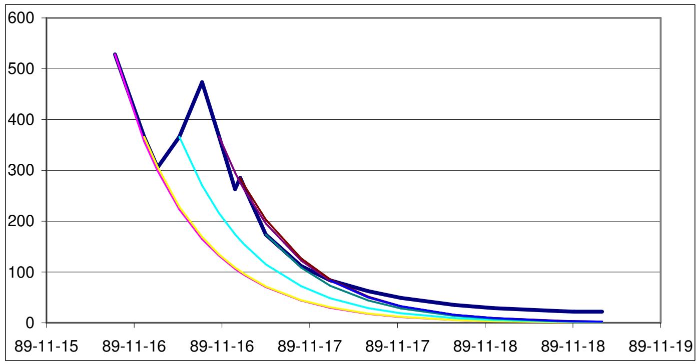


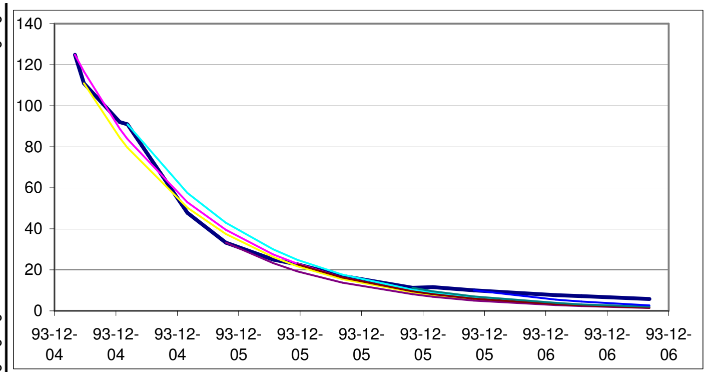
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1985-01-17 11:06 | 0.127 | TOF | | | | | | | Error | | | | | Total | 1735% |
| 1985-01-17 11:30 | 106.691 | 106.691 | | | | | | | 0% | | | | | Avg | 60% |
| 1985-01-17 12:30 | 48.559 | 96.8 | 48.56 | | | | | | 99% | 0% | | | | | |
| 1985-01-17 17:42 | 20.705 | 58.2 | 29.2 | | | | | | 181% | 41% | | | | | |
| 1985-01-18 05:30 | 6.129 | 18.4 | 9.2 | 6.13 | | | | | 200% | 50% | 0% | | | | |
| 1985-01-18 17:12 | 2.44 | 5.9 | 2.9 | 2.0 | | | | | 140% | 20% | 20% | | | | |
| 1985-01-18 18:54 | 2.424 | 5.0 | 2.5 | 1.9 | 2.42 | | | | 104% | 3% | 21% | 0% | | | |
| 1985-01-19 11:36 | 0.969 | 1.0 | 0.5 | 1.6 | 0.5 | | | | 0% | 50% | 68% | 51% | | | |
| 1985-01-20 06:36 | 0.554 | 0.8 | 0.4 | 1.4 | 0.4 | 0.55 | | | 45% | 27% | 145% | 29% | 0% | | |
| 1985-01-21 12:00 | 0.28 | 0.6 | 0.3 | 1.0 | 0.3 | 0.4 | | | 116% | 8% | 263% | 5% | 48% | | |
| 1985-01-21 17:42 | 0.114 | 0.6 | 0.3 | 1.0 | 0.3 | 0.4 | 0.11 | | | | | | | | |
| 1985-01-22 00:36 | 0.096 | 0.5 | 0.3 | 0.9 | 0.3 | 0.4 | 0.2 | | | | | | | | |
| 1985-01-22 06:24 | 0.212 | 0.5 | 0.3 | 0.8 | 0.2 | 0.3 | 0.2 | 0.21 | | | | | | | |
| 1985-01-22 09:18 | 0.209 | 0.5 | 0.2 | 0.8 | 0.2 | 0.3 | 0.2 | 0.2 | | | | | | | |
| 1985-01-22 21:36 | 0.094 | 0.4 | 0.2 | 0.7 | 0.2 | 0.3 | 0.2 | 0.2 | | | | | | | |
| 1988-02-20 03:48 | 112.521 | TOF | | | | | | | Error | | | | | Total | 2605% |
| 1988-02-20 04:12 | 129.753 | 129.753 | | | | | | | 0% | | | | | Avg | 42% |
| 1988-02-20 04:54 | 112.73 | 121.2 | 112.7 | | | | | | 7% | 0% | | | | | |
| 1988-02-20 07:54 | 50.164 | 90.4 | 84.1 | | | | | | 80% | 68% | | | | | |
| 1988-02-20 08:36 | 54.933 | 84.4 | 78.5 | 54.9 | | | | | 54% | 43% | 0% | | | | |
| 1988-02-20 11:18 | 36.149 | 64.8 | 60.3 | 42.2 | | | | | 79% | 67% | 17% | | | | |
| 1988-02-20 13:36 | 26.099 | 51.8 | 48.2 | 33.7 | 26.1 | | | | 98% | 85% | 29% | 0% | | | |
| 1988-02-20 17:00 | 16.659 | 37.1 | 34.5 | 24.2 | 18.7 | | | | 123% | 107% | 45% | 12% | | | |
| 1988-02-20 21:12 | 10.694 | 24.6 | 22.9 | 16.0 | 12.4 | 10.7 | | | 130% | 114% | 50% | 16% | 0% | | |
| 1988-02-21 03:00 | 6.558 | 14.0 | 13.0 | 9.1 | 7.0 | 6.1 | | | 113% | 98% | 39% | 7% | 7% | | |
| 1988-02-21 10:54 | 3.798 | 6.5 | 6.0 | 4.2 | 3.3 | 2.8 | 3.8 | | 70% | 58% | 11% | 14% | 26% | 0% | |
| 1988-02-21 17:54 | 2.519 | 3.3 | 3.0 | 2.1 | 1.6 | 1.4 | 1.9 | | 29% | 20% | 16% | 35% | 44% | 24% | |
| 1988-02-22 04:18 | 1.551 | 1.2 | 1.1 | 0.8 | 1.5 | 1.3 | 1.7 | 1.55 | 24% | 29% | 51% | 4% | 18% | 12% | 0% |
| 1988-02-22 14:24 | 1.085 | 1.1 | 1.0 | 0.7 | 1.3 | 1.2 | 1.6 | 1.4 | 2% | 8% | 36% | 24% | 7% | 45% | 30% |
| 1988-02-23 17:00 | 0.538 | 0.8 | 0.8 | 0.5 | 1.0 | 0.9 | 1.2 | 1.1 | 53% | 42% | 0% | 93% | 66% | 125% | 101% |
| 1989-11-15 18:42 | 472.656 | TOF | | | | | | | Error | | | | | Total | 2123% |
| 1989-11-15 21:24 | 528.235 | 528.235 | | | | | | | 0% | | | | | Avg | 52% |
| 1989-11-16 01:24 | 365.559 | 357.3 | 365.6 | | | | | | 2% | 0% | | | | | |
| 1989-11-16 03:12 | 305.898 | 299.6 | 306.6 | | | | | | 2% | 0% | | | | | |
| 1989-11-16 06:12 | 365.943 | 223.5 | 228.7 | 366 | | | | | 39% | 38% | 0% | | | | |
| 1989-11-16 09:18 | 473.881 | 165.1 | 168.9 | 270 | | | | | 65% | 64% | 43% | | | | |
| 1989-11-16 11:36 | 366.574 | 131.8 | 134.9 | 216 | 367 | | | | 64% | 63% | 41% | 0% | | | |
| 1989-11-16 13:48 | 263.012 | 106.3 | 108.8 | 174 | 296 | | | | 60% | 59% | 34% | 12% | | | |
| 1989-11-16 14:30 | 285.613 | 99.3 | 101.6 | 163 | 276 | 286 | | | 65% | 64% | 43% | 3% | 0% | | |
| 1989-11-16 15:06 | 265.454 | 93.6 | 95.8 | 153 | 260 | 269 | | | 65% | 64% | 42% | 2% | 1% | | |
| 1989-11-16 18:00 | 173.841 | 70.5 | 72.2 | 115 | 196 | 203 | 174 | | 59% | 58% | 34% | 13% | 17% | 0% | |
| 1989-11-16 22:48 | 112.312 | 44.1 | 45.1 | 72.2 | 123 | 127 | 109 | | 61% | 60% | 36% | 9% | 13% | 3% | |
| 1989-11-17 02:48 | 83.884 | 29.8 | 30.5 | 48.9 | 83.0 | 85.8 | 73.5 | 83.9 | 64% | 64% | 42% | 1% | 2% | 12% | 0% |
| 1989-11-17 08:00 | 62.531 | 17.9 | 18.4 | 29.4 | 49.9 | 51.6 | 44.2 | 50.5 | 71% | 71% | 53% | 20% | 17% | 29% | 19% |
| 1989-11-17 12:30 | 49.199 | 11.6 | 11.8 | 18.9 | 32.1 | 33.3 | 28.5 | 32.5 | 77% | 76% | 62% | 35% | 32% | 42% | 34% |
| 1989-11-17 19:48 | 35.574 | 5.7 | 5.8 | 9.3 | 15.7 | 16.3 | 14.0 | 15.9 | 84% | 84% | 74% | 56% | 54% | 61% | 55% |
| 1989-11-18 01:12 | 29.32 | 3.3 | 3.4 | 5.5 | 9.3 | 9.6 | 8.2 | 9.4 | 89% | 88% | 81% | 68% | 67% | 72% | 68% |
| 1989-11-18 09:54 | 23.129 | 1.4 | 1.5 | 2.3 | 4.0 | 4.1 | 3.5 | 4.0 | 94% | 94% | 90% | 83% | 82% | 85% | 83% |
| 1989-11-18 12:12 | 22.33 | 1.4 | 1.4 | 1.9 | 3.2 | 3.3 | 2.8 | 3.2 | 94% | 94% | 92% | 86% | 85% | 87% | 86% |
| 1989-11-18 16:00 | 22.025 | 1.3 | 1.4 | 1.8 | 2.2 | 2.3 | 1.9 | 2.2 | 94% | 94% | 92% | 90% | 90% | 91% | 90% |

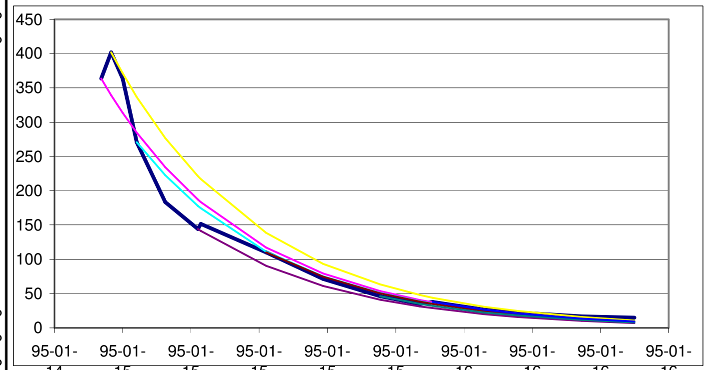


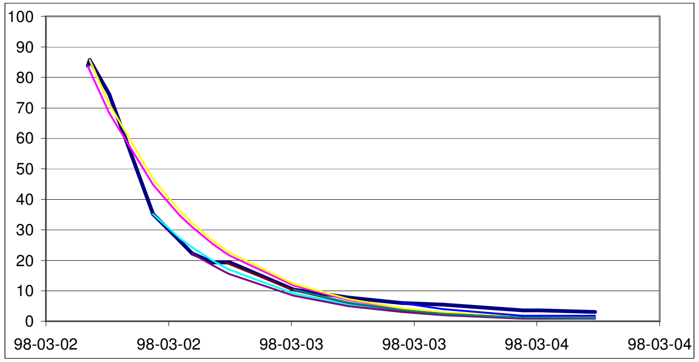



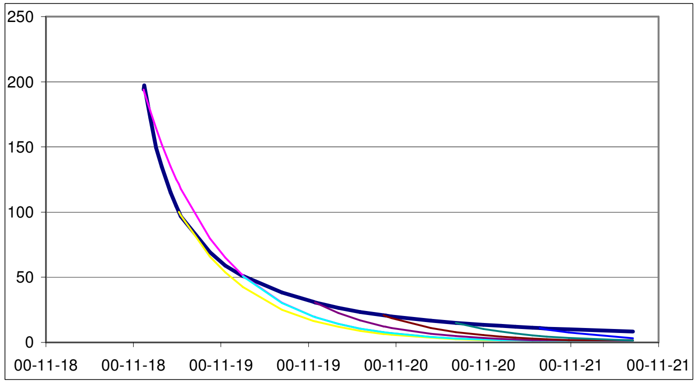
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1993-12-04 13:24 | 110.919 | TOF | | | | | | | Error | | | | | Total | 1546% |
| 1993-12-04 13:36 | 124.956 | 124.956 | | | | | | | 0% | | | | | Avg | 30% |
| 1993-12-04 14:18 | 110.919 | 116.7 | 110.9 | | | | | | 5% | 0% | | | | | |
| 1993-12-04 17:06 | 91.935 | 88.8 | 84.4 | | | | | | 3% | 8% | | | | | |
| 1993-12-04 17:42 | 91.042 | 83.7 | 79.6 | 91 | | | | | 8% | 13% | 0% | | | | |
| 1993-12-04 22:24 | 47.838 | 52.9 | 50.3 | 57.5 | | | | | 11% | 5% | 20% | | | | |
| 1993-12-05 01:24 | 33.231 | 39.4 | 37.5 | 42.9 | 33.2 | | | | 19% | 13% | 29% | 0% | | | |
| 1993-12-05 05:06 | 24.947 | 27.5 | 26.1 | 29.9 | 23.1 | | | | 10% | 5% | 20% | 7% | | | |
| 1993-12-05 07:00 | 22.636 | 22.8 | 21.7 | 24.8 | 19.2 | 22.6 | | | 1% | 4% | 10% | 15% | 0% | | |
| 1993-12-05 10:30 | 16.955 | 16.2 | 15.4 | 17.6 | 13.7 | 16.1 | | | 4% | 9% | 4% | 19% | 5% | | |
| 1993-12-05 16:00 | 11.18 | 9.5 | 9.0 | 10.3 | 8.0 | 9.4 | 11.2 | | 15% | 20% | 8% | 29% | 16% | 0% | |
| 1993-12-05 17:36 | 11.545 | 8.1 | 7.7 | 8.8 | 6.8 | 8.0 | 9.6 | | 30% | 33% | 24% | 41% | 30% | 17% | |
| 1993-12-05 20:48 | 10.054 | 5.9 | 5.6 | 6.4 | 5.0 | 5.9 | 7.0 | 10.1 | 41% | 44% | 36% | 50% | 42% | 30% | 0% |
| 1993-12-06 03:12 | 7.543 | 3.2 | 3.0 | 3.4 | 2.7 | 3.1 | 3.7 | 5.4 | 58% | 60% | 54% | 65% | 58% | 50% | 29% |
| 1993-12-06 05:12 | 7.138 | 2.6 | 2.5 | 2.8 | 2.2 | 2.6 | 3.1 | 4.4 | 64% | 65% | 60% | 69% | 64% | 57% | 38% |
| 1993-12-06 10:30 | 5.806 | 1.6 | 1.5 | 1.7 | 1.3 | 1.5 | 1.8 | 2.6 | 73% | 75% | 71% | 77% | 73% | 68% | 55% |
| 1995-01-14 20:30 | 110.175 | TOF | | | | | | | Error | | | | | Total | 882% |
| 1995-01-14 22:30 | 363.666 | 363.666 | | | | | | | 0% | | | | | Avg | 18% |
| 1995-01-14 23:12 | 401.782 | 339.6 | 401.8 | | | | | | 15% | 0% | | | | | |
| 1995-01-15 00:00 | 363.928 | 314.1 | 371.6 | | | | | | 14% | 2% | | | | | |
| 1995-01-15 01:00 | 270.666 | 284.8 | 337.0 | 271 | | | | | 5% | 24% | 0% | | | | |
| 1995-01-15 03:00 | 183.402 | 234.2 | 277.1 | 223 | | | | | 28% | 51% | 21% | | | | |
| 1995-01-15 05:18 | 144.184 | 187.1 | 221.3 | 178 | 144 | | | | 30% | 54% | 23% | 0% | | | |
| 1995-01-15 05:30 | 151.598 | 183.5 | 217.0 | 174 | 141 | | | | 21% | 43% | 15% | 7% | | | |
| 1995-01-15 10:06 | 110.175 | 117.0 | 138.4 | 111 | 90.2 | 110 | | | 6% | 26% | 1% | 18% | 0% | | |
| 1995-01-15 14:06 | 71.192 | 79.2 | 93.6 | 75.2 | 61.0 | 74.5 | | | 11% | 32% | 6% | 14% | 5% | | |
| 1995-01-15 18:06 | 45.944 | 53.5 | 63.3 | 50.9 | 41.3 | 50.4 | 45.9 | | 17% | 38% | 11% | 10% | 10% | 0% | |
| 1995-01-15 20:54 | 34.644 | 40.7 | 48.2 | 38.7 | 31.4 | 38.3 | 34.9 | | 18% | 39% | 12% | 9% | 11% | 1% | |
| 1995-01-15 21:48 | 37.456 | 37.3 | 44.1 | 35.4 | 28.7 | 35.1 | 32.0 | 37.5 | 0% | 18% | 5% | 23% | 6% | 15% | 0% |
| 1995-01-16 01:24 | 26.874 | 26.2 | 31.0 | 24.9 | 20.2 | 24.7 | 22.5 | 26.3 | 2% | 15% | 7% | 25% | 8% | 16% | 2% |
| 1995-01-16 03:48 | 22.087 | 20.7 | 24.5 | 19.7 | 16.0 | 19.5 | 17.8 | 20.8 | 6% | 11% | 11% | 28% | 12% | 19% | 6% |
| 1995-01-16 08:18 | 17.013 | 13.4 | 15.8 | 12.7 | 10.3 | 12.6 | 11.5 | 13.4 | 21% | 7% | 25% | 39% | 26% | 33% | 21% |
| 1995-01-16 12:00 | 14.687 | 9.3 | 11.0 | 8.8 | 7.2 | 8.8 | 8.0 | 9.3 | 37% | 25% | 40% | 51% | 40% | 46% | 36% |



| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1998-03-02 03:42 | 21.603 | TOF | | | | | | | Error | | | | | Total | 1679% |
| 1998-03-02 04:06 | 83.789 | 83.789 | | | | | | | 0% | | | | | Avg | 34% |
| 1998-03-02 04:18 | 85.726 | 82.2 | 85.73 | | | | | | 4% | 0% | | | | | |
| 1998-03-02 06:12 | 74.506 | 68.2 | 71.2 | | | | | | 8% | 4% | | | | | |
| 1998-03-02 10:30 | 35.145 | 44.8 | 46.8 | 35.1 | | | | | 28% | 33% | 0% | | | | |
| 1998-03-02 13:06 | 26.422 | 34.8 | 36.3 | 27.3 | | | | | 32% | 37% | 3% | | | | |
| 1998-03-02 14:18 | 22.271 | 30.9 | 32.3 | 24.2 | 22.3 | | | | 39% | 45% | 9% | 0% | | | |
| 1998-03-02 16:18 | 19.254 | 25.4 | 26.5 | 19.9 | 18.3 | | | | 32% | 38% | 4% | 5% | | | |
| 1998-03-02 17:36 | 19.381 | 22.4 | 23.4 | 17.6 | 16.1 | 19.4 | | | 16% | 21% | 9% | 17% | 0% | | |
| 1998-03-02 18:00 | 19.381 | 21.5 | 22.5 | 16.9 | 15.5 | 18.6 | | | 11% | 16% | 13% | 20% | 4% | | |
| 1998-03-03 00:12 | 10.435 | 11.7 | 12.3 | 9.2 | 8.5 | 10.2 | 10.4 | | 13% | 17% | 12% | 19% | 3% | 0% | |
| 1998-03-03 05:36 | 7.751 | 6.9 | 7.2 | 5.4 | 5.0 | 6.0 | 6.2 | | 11% | 7% | 30% | 36% | 23% | 21% | |
| 1998-03-03 10:48 | 6.01 | 4.2 | 4.3 | 3.3 | 3.0 | 3.6 | 3.7 | 6.01 | 31% | 28% | 46% | 50% | 40% | 38% | 0% |
| 1998-03-03 14:48 | 5.466 | 2.8 | 2.9 | 2.2 | 2.0 | 2.4 | 2.5 | 4.1 | 48% | 46% | 60% | 63% | 55% | 54% | 26% |
| 1998-03-03 22:36 | 3.673 | 1.3 | 1.4 | 1.0 | 0.9 | 1.1 | 1.2 | 1.9 | 64% | 63% | 72% | 74% | 69% | 68% | 48% |
| 1998-03-04 00:18 | 3.571 | 1.3 | 1.3 | 1.0 | 0.9 | 1.1 | 1.1 | 1.9 | 64% | 62% | 72% | 74% | 69% | 68% | 48% |
| 1998-03-04 05:42 | 3.015 | 1.2 | 1.3 | 1.0 | 0.9 | 1.1 | 1.1 | 1.8 | 59% | 58% | 68% | 71% | 65% | 64% | 41% |
| 2000-11-18 13:18 | 193.379 | TOF | | | | | | | Error | | | | | Total | 3714% |
| 2000-11-18 13:24 | 194.029 | 194.029 | | | | | | | 0% | | | | | Avg | 51% |
| 2000-11-18 13:30 | 197.271 | 192.1 | | | | | | | 3% | | | | | | |
| 2000-11-18 14:18 | 172.114 | 177.7 | | | | | | | 3% | | | | | | |
| 2000-11-18 15:06 | 149.36 | 164.3 | | | | | | | 10% | | | | | | |
| 2000-11-18 15:54 | 134.504 | 152.0 | | | | | | | 13% | | | | | | |
| 2000-11-18 17:06 | 115.319 | 135.1 | | | | | | | 17% | | | | | | |
| 2000-11-18 17:54 | 104.594 | 125.0 | | | | | | | 19% | | | | | | |
| 2000-11-18 18:12 | 100.348 | 121.4 | 100.3 | | | | | | 21% | 0% | | | | | |
| 2000-11-18 18:30 | 96.567 | 117.9 | 97.4 | | | | | | 22% | 1% | | | | | |
| 2000-11-18 22:30 | 69.373 | 79.7 | 65.9 | | | | | | 15% | 5% | | | | | |
| 2000-11-19 00:36 | 58.814 | 64.9 | 53.7 | | | | | | 10% | 9% | | | | | |
| 2000-11-19 02:54 | 51.139 | 51.9 | 42.9 | | | | | | 1% | 16% | | | | | |
| 2000-11-19 03:00 | 50.896 | 51.3 | 42.5 | 50.9 | | | | | 1% | 17% | 0% | | | | |
| 2000-11-19 03:06 | 50.733 | 50.8 | 42.0 | 50.4 | | | | | 0% | 17% | 1% | | | | |
| 2000-11-19 08:24 | 38.262 | 30.3 | 25.0 | 30.0 | | | | | 21% | 35% | 22% | | | | |
| 2000-11-19 12:42 | 31.016 | 19.9 | 16.4 | 19.7 | | | | | 36% | 47% | 36% | | | | |
| 2000-11-19 12:54 | 30.674 | 19.5 | 16.1 | 19.3 | 30.7 | | | | 36% | 47% | 37% | 0% | | | |
| 2000-11-19 13:06 | 30.333 | 19.1 | 15.8 | 19.0 | 30.1 | | | | 37% | 48% | 37% | 1% | | | |
| 2000-11-19 16:06 | 26.422 | 14.3 | 11.8 | 14.1 | 22.4 | | | | 46% | 55% | 46% | 15% | | | |
| 2000-11-19 19:06 | 23.316 | 10.6 | 8.8 | 10.5 | 16.7 | | | | 54% | 62% | 55% | 28% | | | |
| 2000-11-19 22:18 | 20.705 | 7.8 | 6.4 | 7.7 | 12.2 | 20.7 | | | 62% | 69% | 63% | 41% | 0% | | |
| 2000-11-19 22:36 | 20.526 | 7.6 | 6.2 | 7.5 | 11.9 | 20.1 | | | 63% | 70% | 63% | 42% | 2% | | |
| 2000-11-19 23:24 | 19.874 | 7.0 | 5.8 | 6.9 | 11.0 | 18.6 | | | 65% | 71% | 65% | 45% | 6% | | |
| 2000-11-20 04:48 | 16.541 | 4.1 | 3.4 | 4.1 | 6.5 | 11.0 | | | 75% | 79% | 75% | 61% | 34% | | |
| 2000-11-20 08:12 | 14.904 | 3.0 | 2.4 | 2.9 | 4.6 | 7.9 | 14.9 | | 80% | 84% | 80% | 69% | 47% | 0% | |
| 2000-11-20 12:00 | 13.43 | 2.0 | 1.7 | 2.0 | 3.2 | 5.4 | 10.3 | | 85% | 87% | 85% | 76% | 60% | 23% | |
| 2000-11-20 14:24 | 12.583 | 1.6 | 1.6 | 1.6 | 2.5 | 4.3 | 8.1 | | 87% | 87% | 87% | 80% | 66% | 35% | |
| 2000-11-20 16:24 | 11.915 | 1.6 | 1.6 | 1.6 | 2.1 | 3.5 | 6.7 | | 87% | 86% | 87% | 82% | 70% | 44% | |
| 2000-11-20 18:42 | 11.271 | 1.5 | 1.6 | 1.5 | 1.7 | 2.8 | 5.3 | | 86% | 86% | 86% | 85% | 75% | 53% | |
| 2000-11-20 19:42 | 10.958 | 1.5 | 1.6 | 1.5 | 1.6 | 2.6 | 4.8 | 11 | 86% | 86% | 86% | 85% | 77% | 56% | 0% |
| 2000-11-20 20:42 | 10.65 | 1.5 | 1.5 | 1.5 | 1.6 | 2.3 | 4.4 | 9.9 | 86% | 85% | 86% | 85% | 78% | 59% | 7% |
| 2000-11-20 23:42 | 9.971 | 1.5 | 1.5 | 1.5 | 1.6 | 1.7 | 3.3 | 7.4 | 85% | 85% | 85% | 84% | 83% | 67% | 26% |
| 2000-11-21 08:30 | 8.253 | 1.4 | 1.4 | 1.3 | 1.5 | 1.6 | 1.4 | 3.1 | 84% | 83% | 84% | 82% | 81% | 83% | 62% |



# Appendix G: Rainfall, Evaporation and Tributary Flow Hydrographs

Plots for the Rainfall and Evaporation at each Dam along with the Flow Hydrographs for each of the tributary flow stations are included in the Appendix.

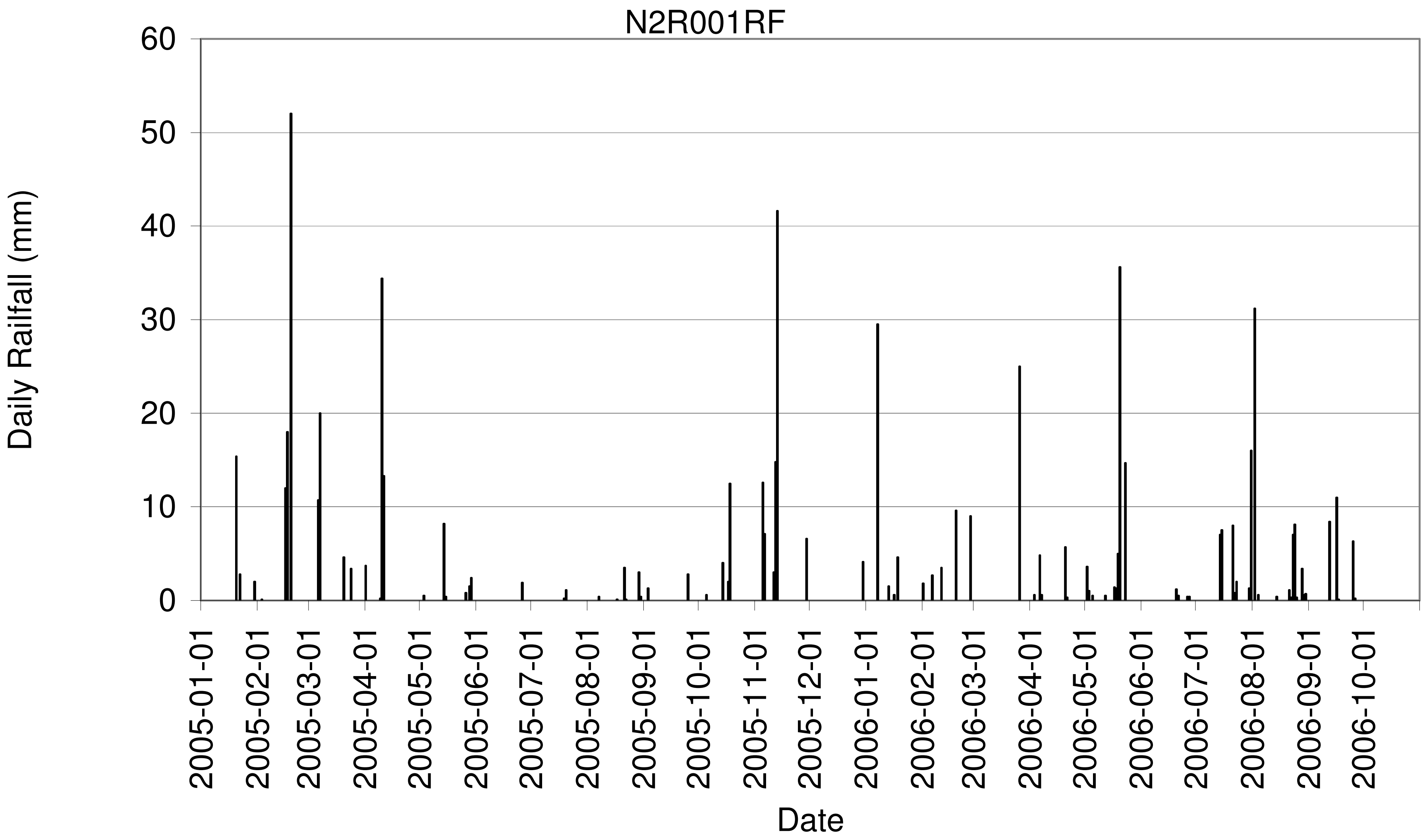



**Figure G-1: Daily Rainfall at Darlington Dam**

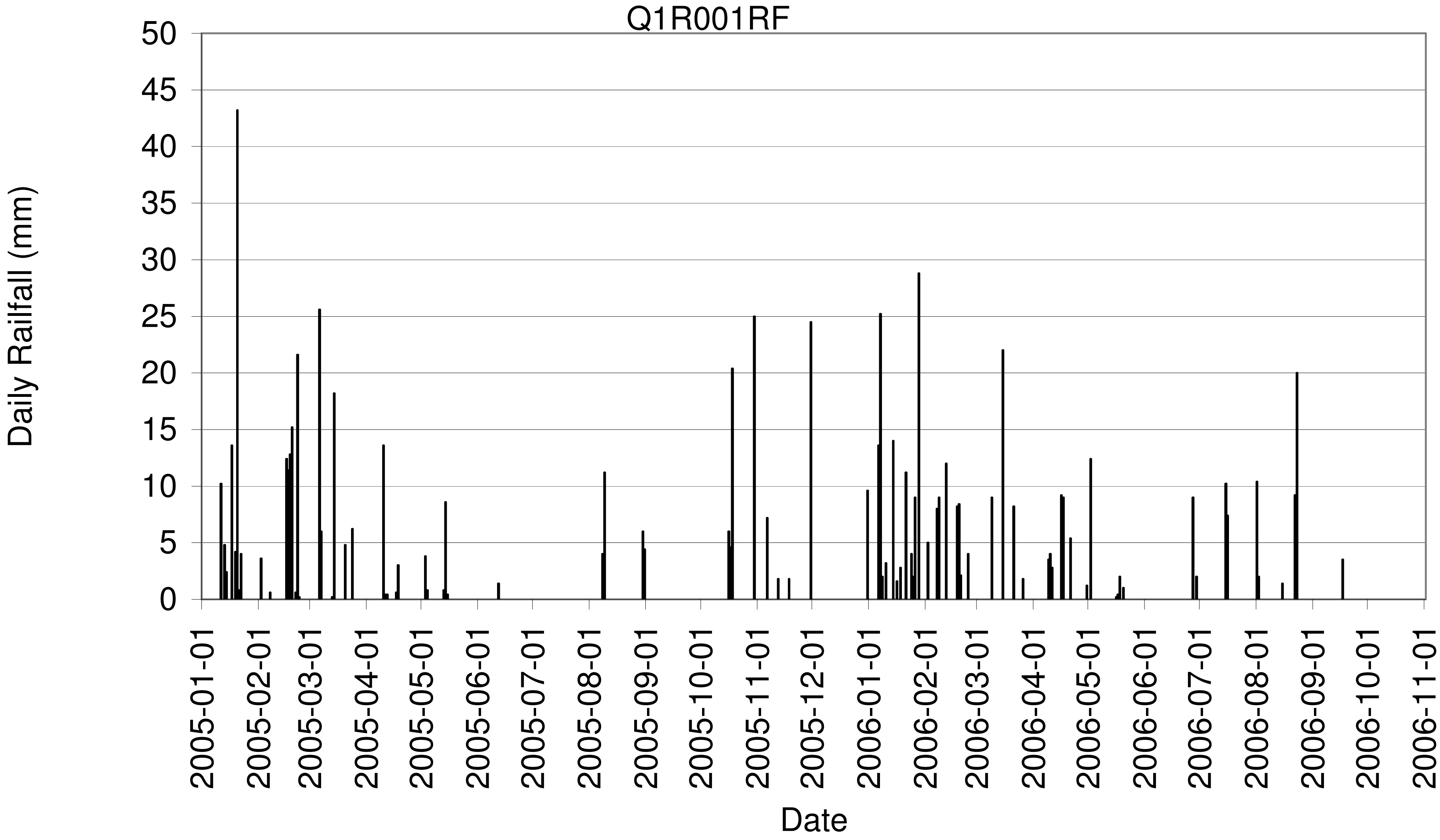



**Figure G-2: Daily Rainfall at Grassridge Dam**

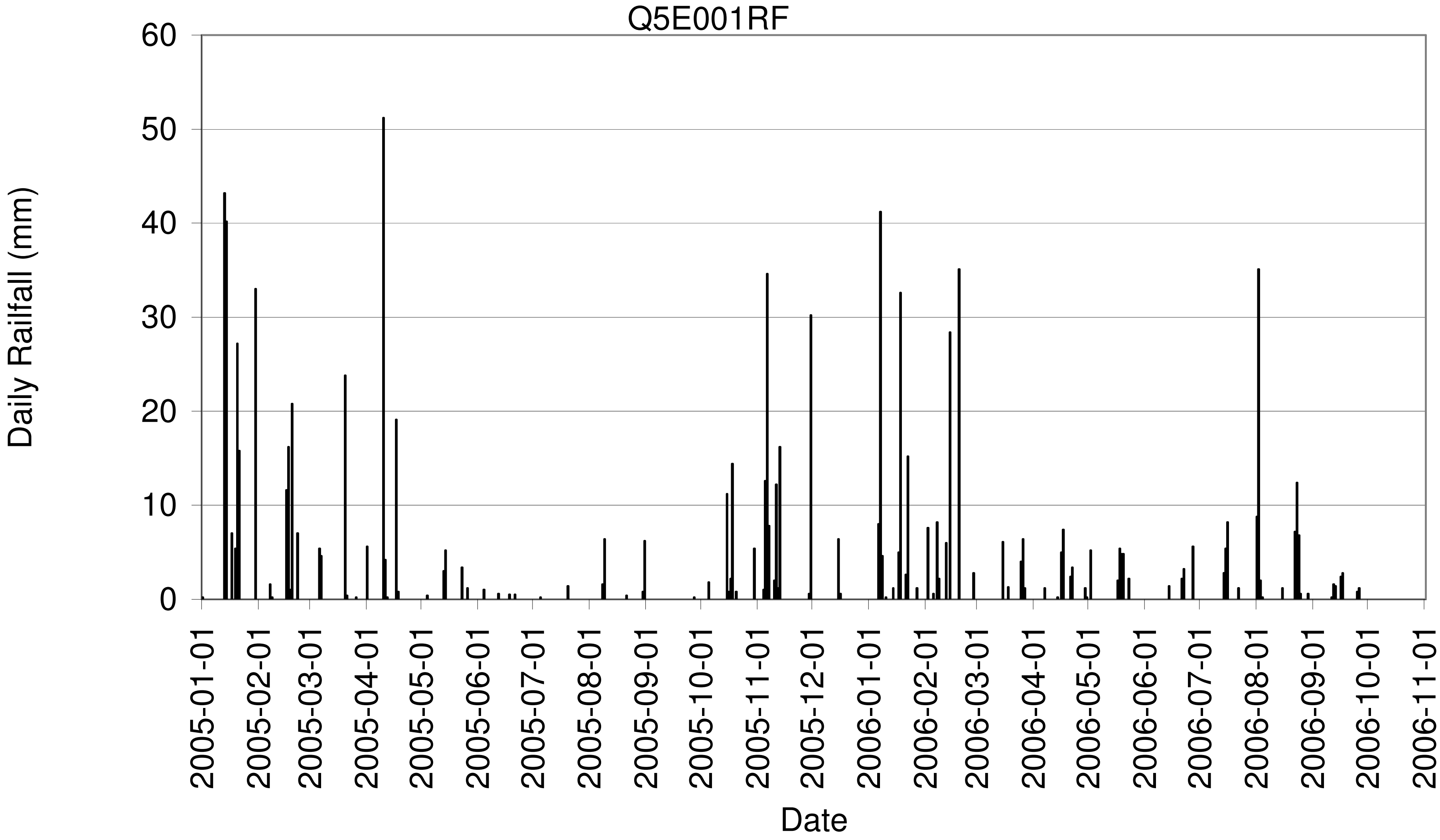



**Figure G-3: Daily Rainfall at Elandsdrift Dam**

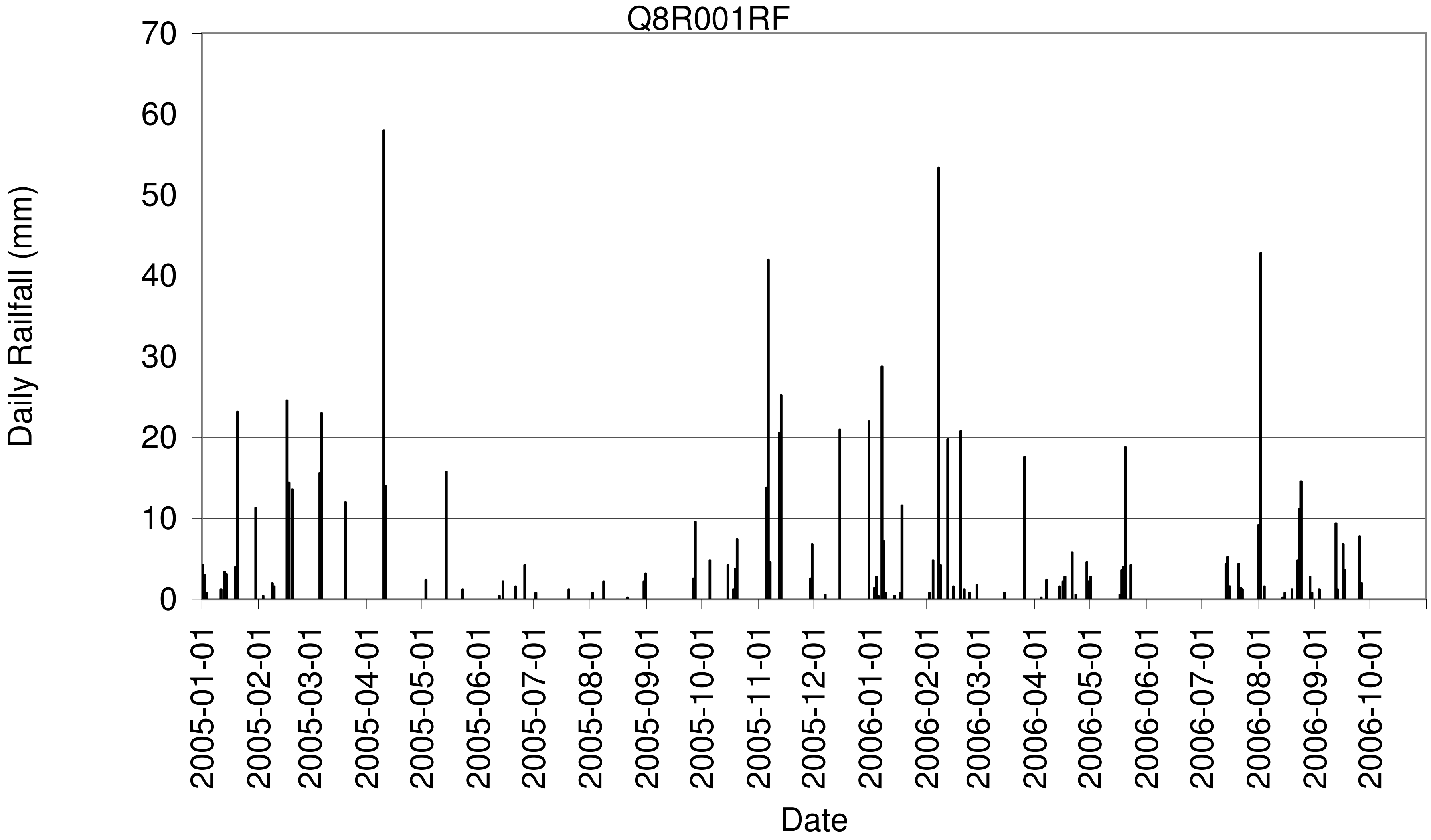



**Figure G-4: Daily Rainfall at De Mistkraal Dam**

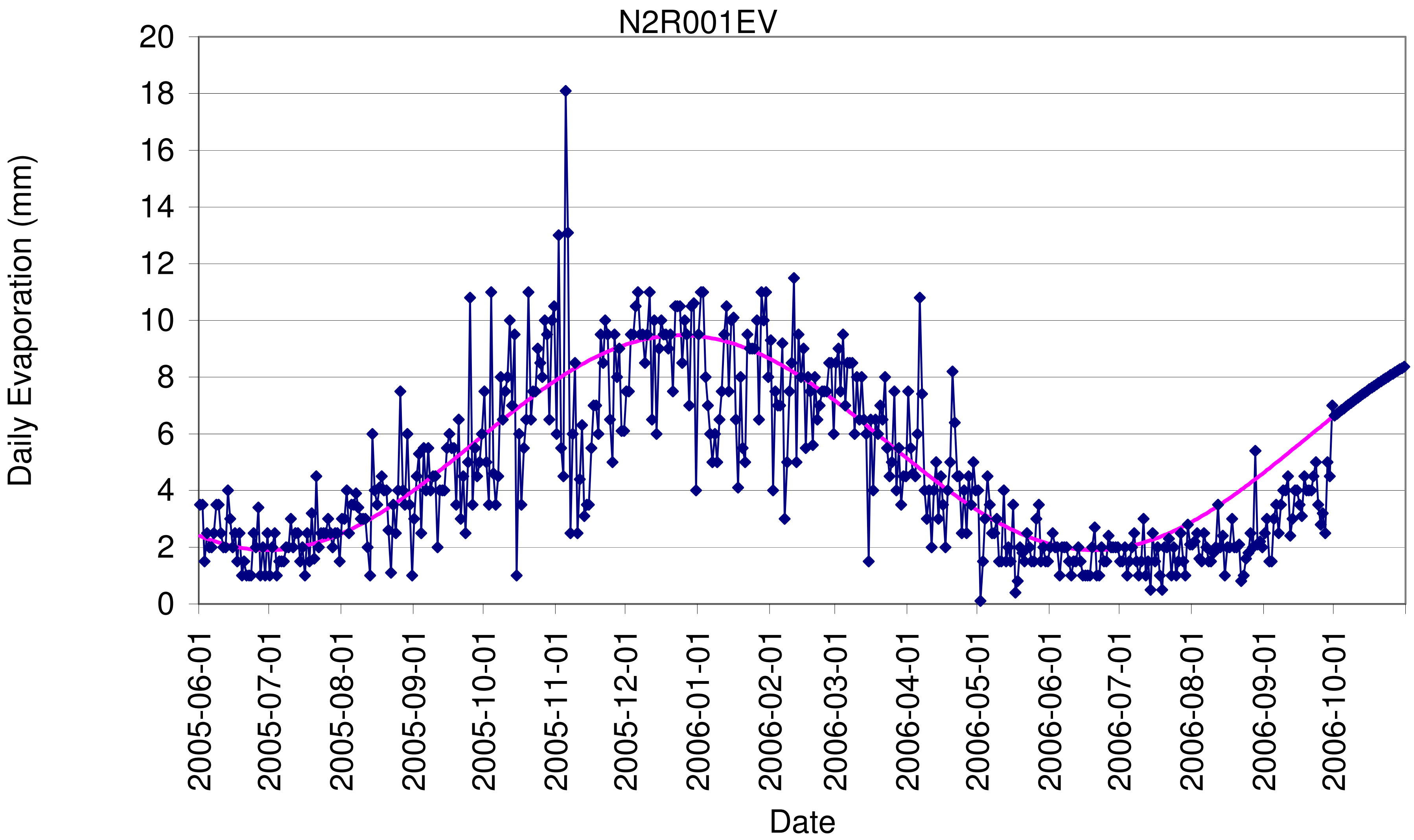



**Figure G-5: Evaporation Rate at Darlington Dam**

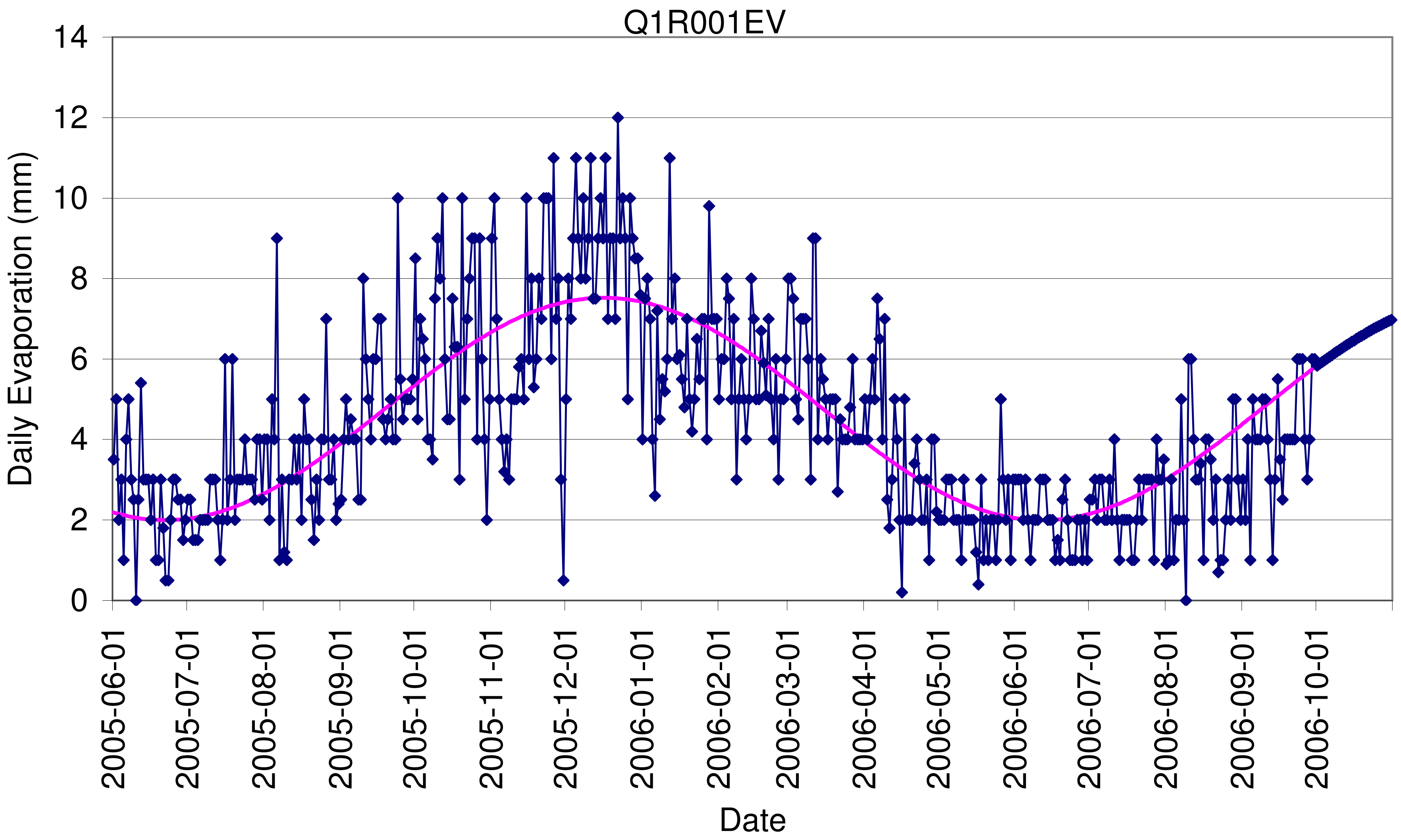



**Figure G-6: Evaporation Rate at Grassridge Dam**

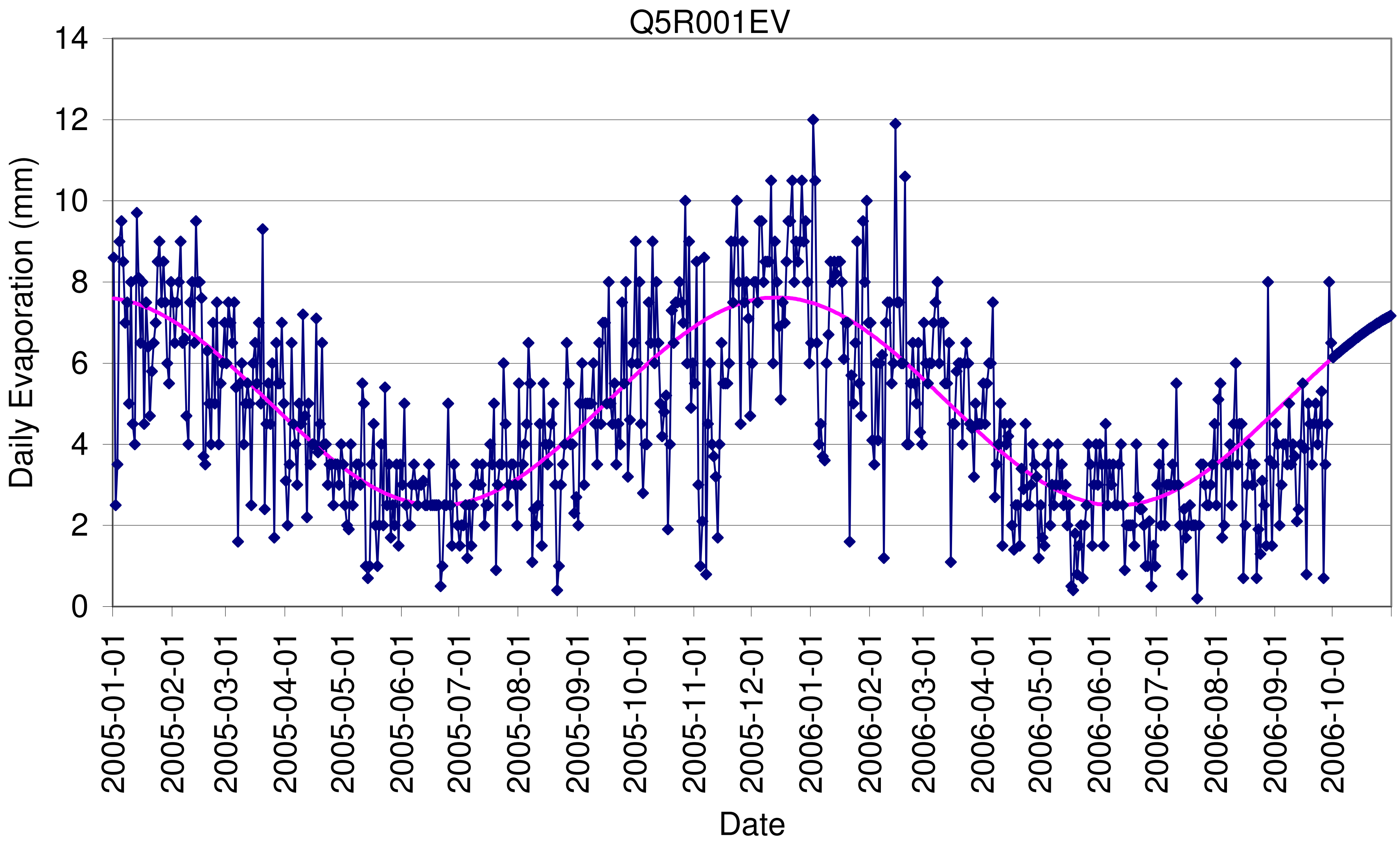



**Figure G-7: Evaporation Rate at Elandsdrift Dam**

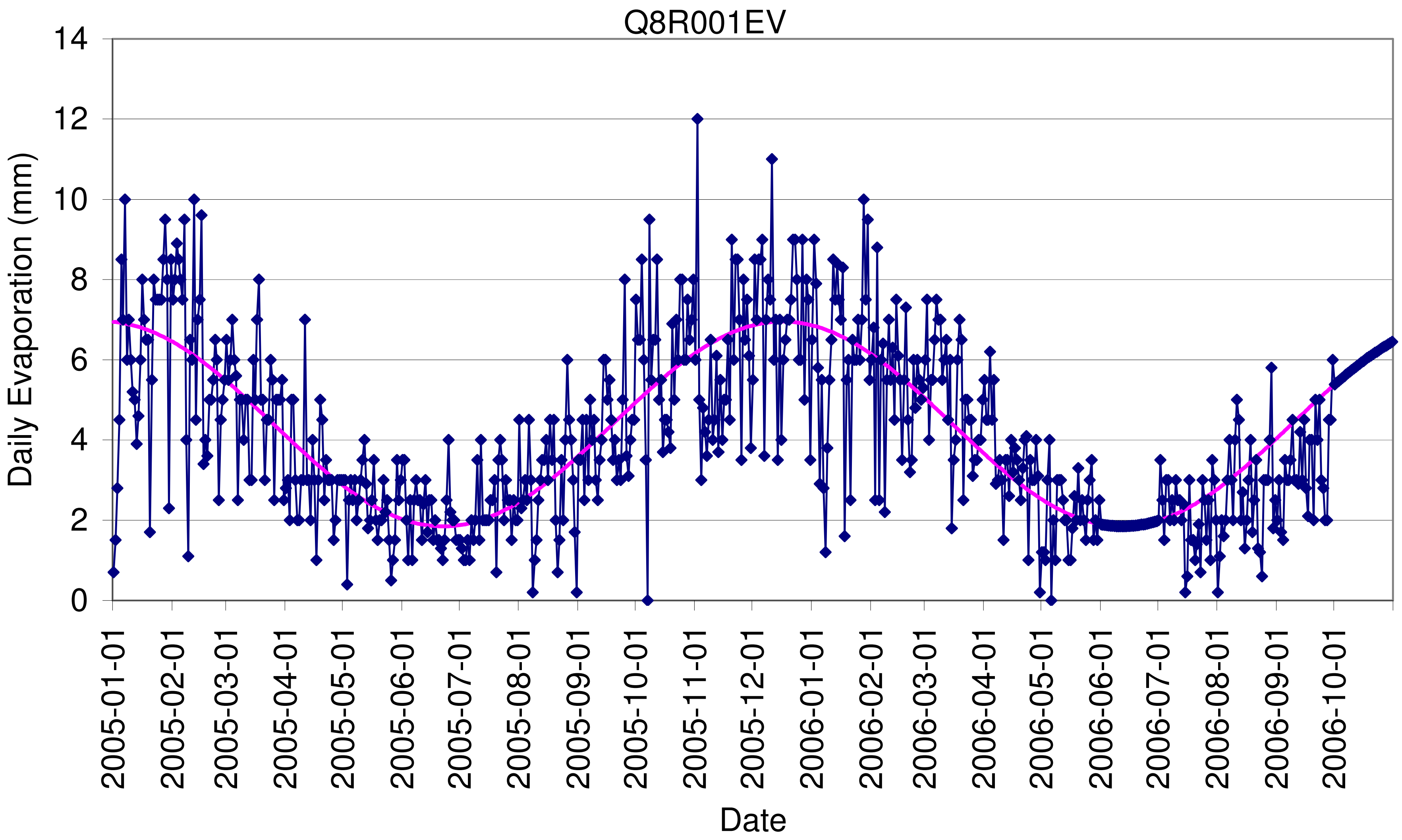



**Figure G-8: Evaporation Rate at De Mistkraal Dam**

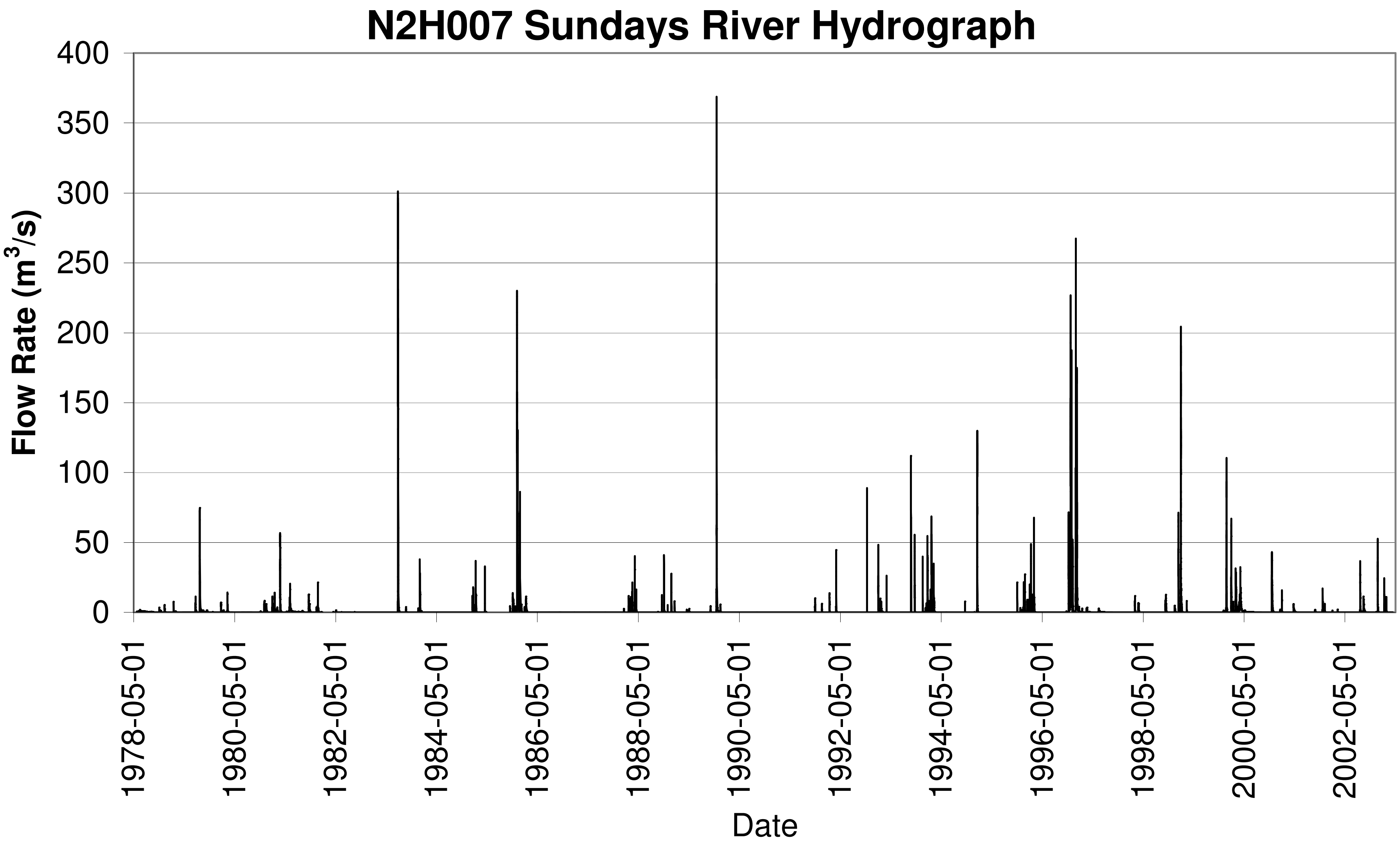



**Figure G-9: Flow Hydrograph for the Sundays River at De Draai**

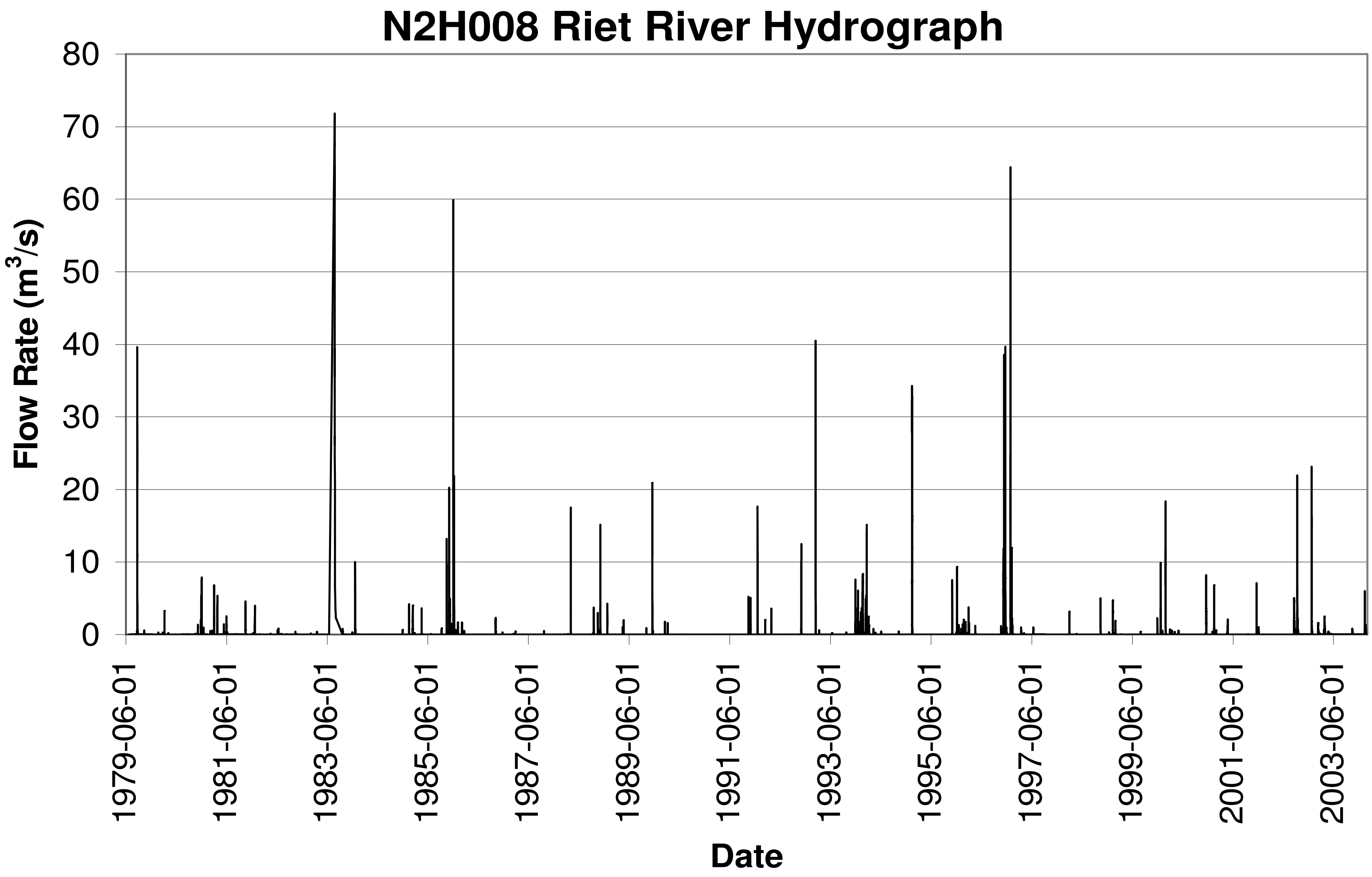



**Figure G-10: Flow Hydrograph for the Riet River at Groen Leegte**

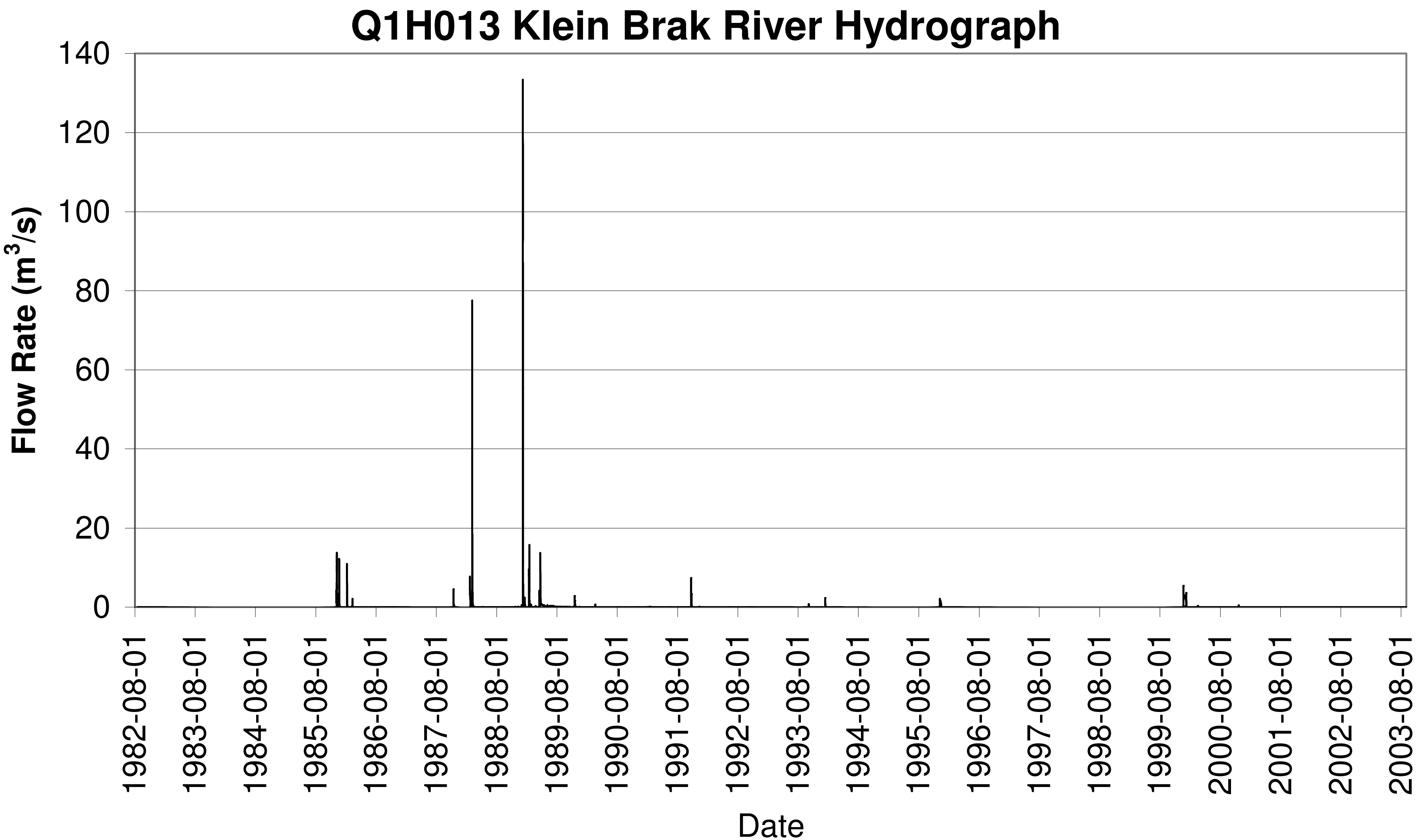



**Figure G-11: Flow Hydrograph for the Klein Brak River at Zevenfontein**

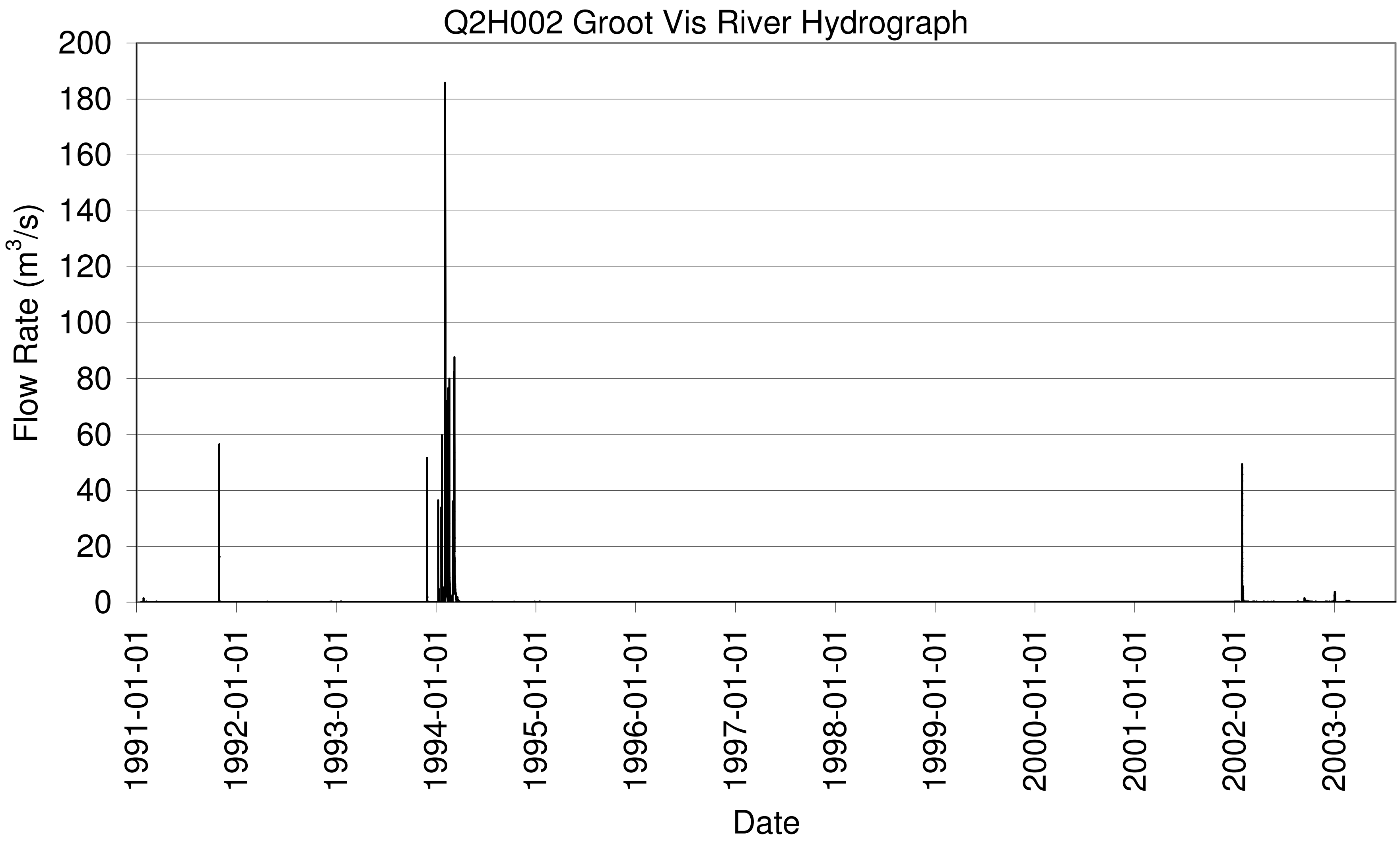



**Figure G-12: Flow Hydrograph for the Groot Vis River at Soutpansdrift**

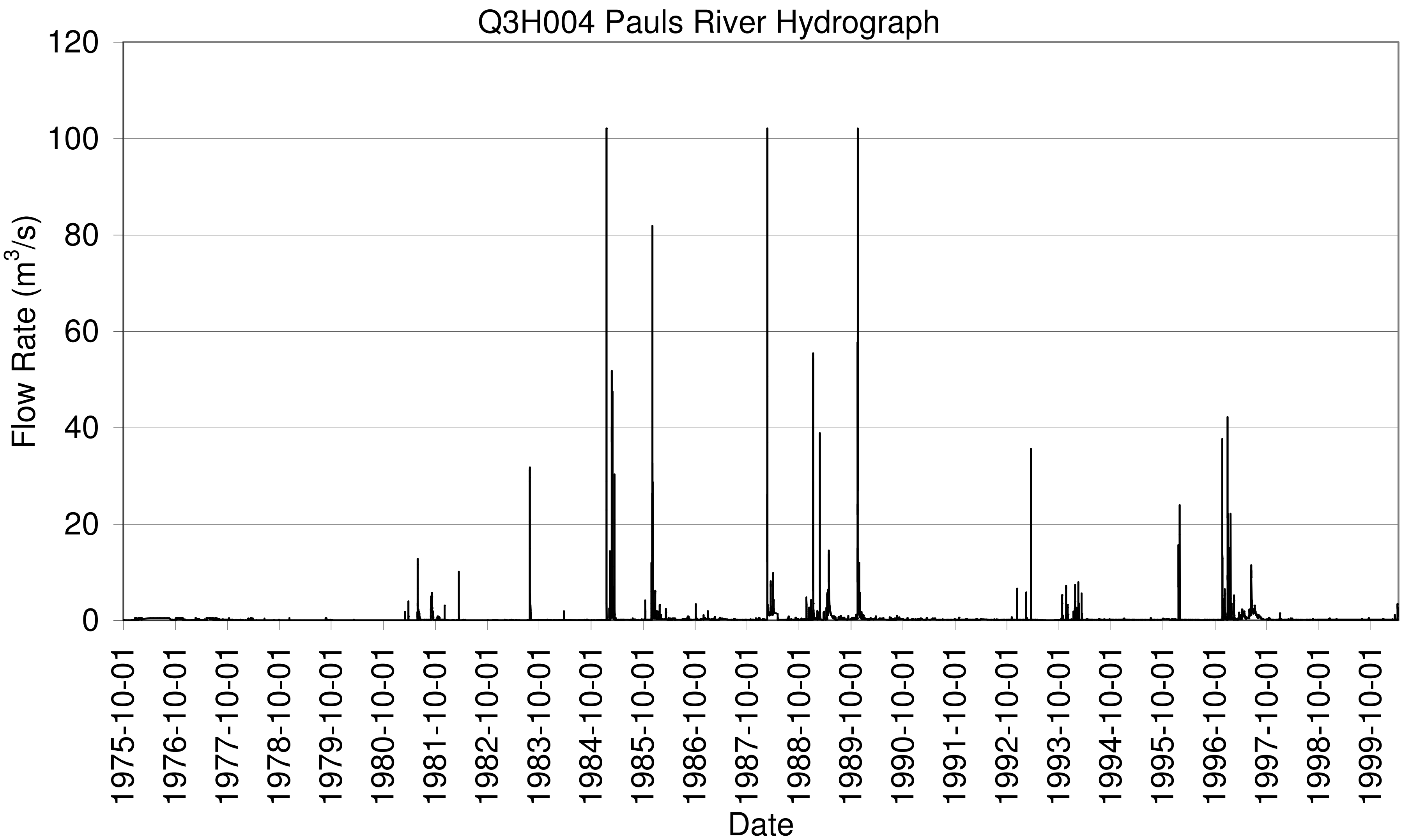



**Figure G-13: Flow Hydrograph for the Pauls River at Coutzenburg**

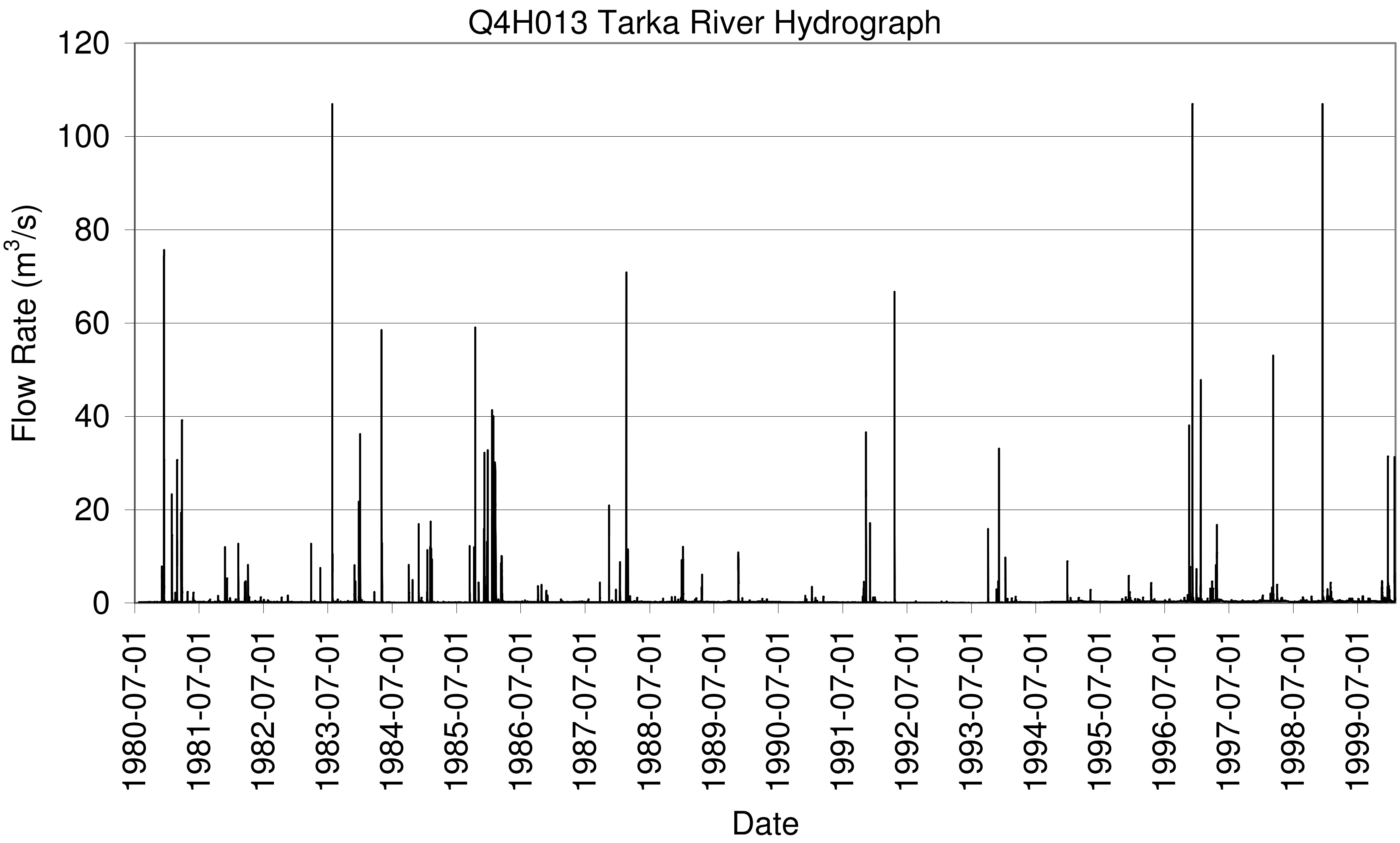



**Figure G-14: Flow Hydrograph for the Tarka River at Bridge Farm**

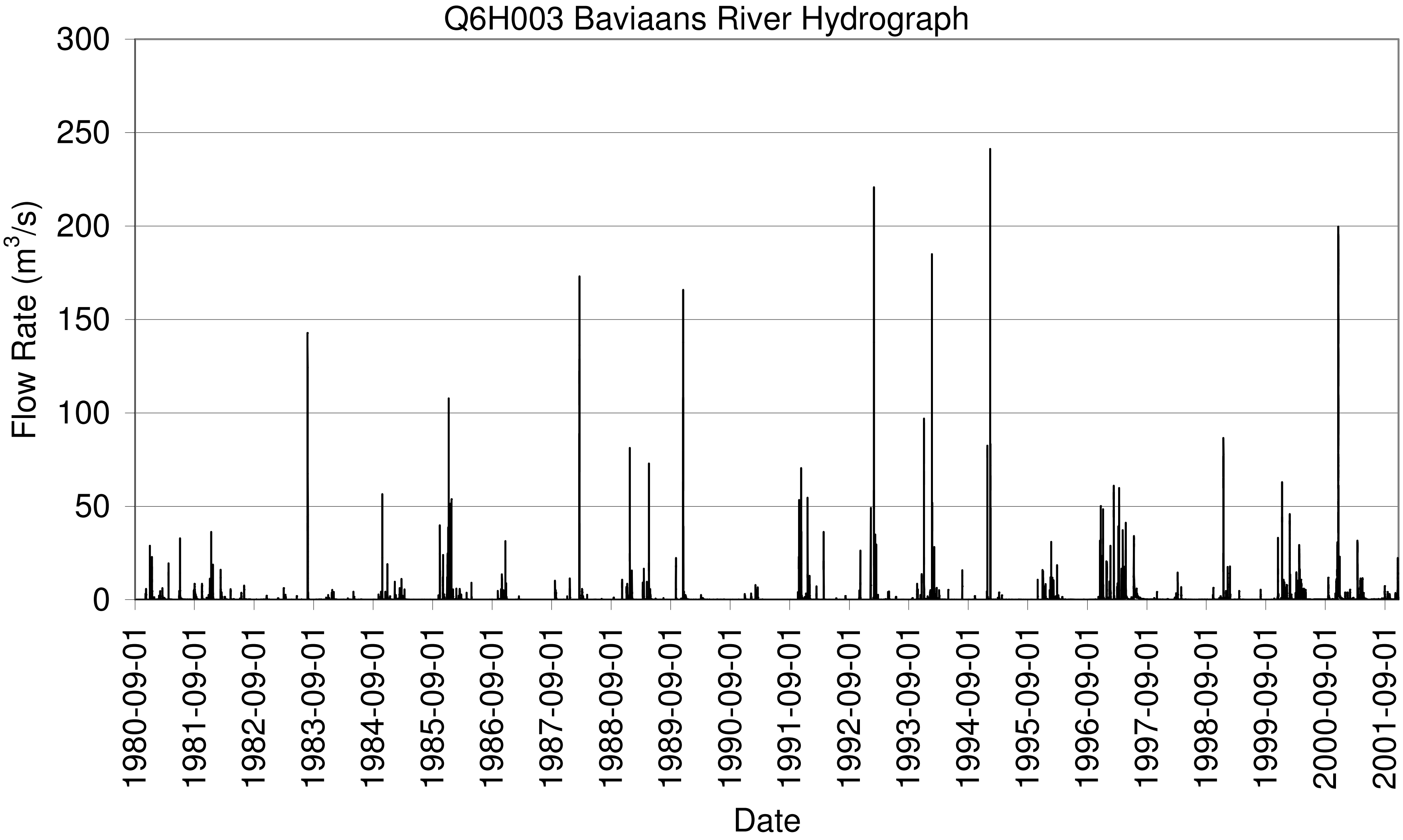



**Figure G-15: Flow Hydrograph for the Baviaans River at de Klerksdal**

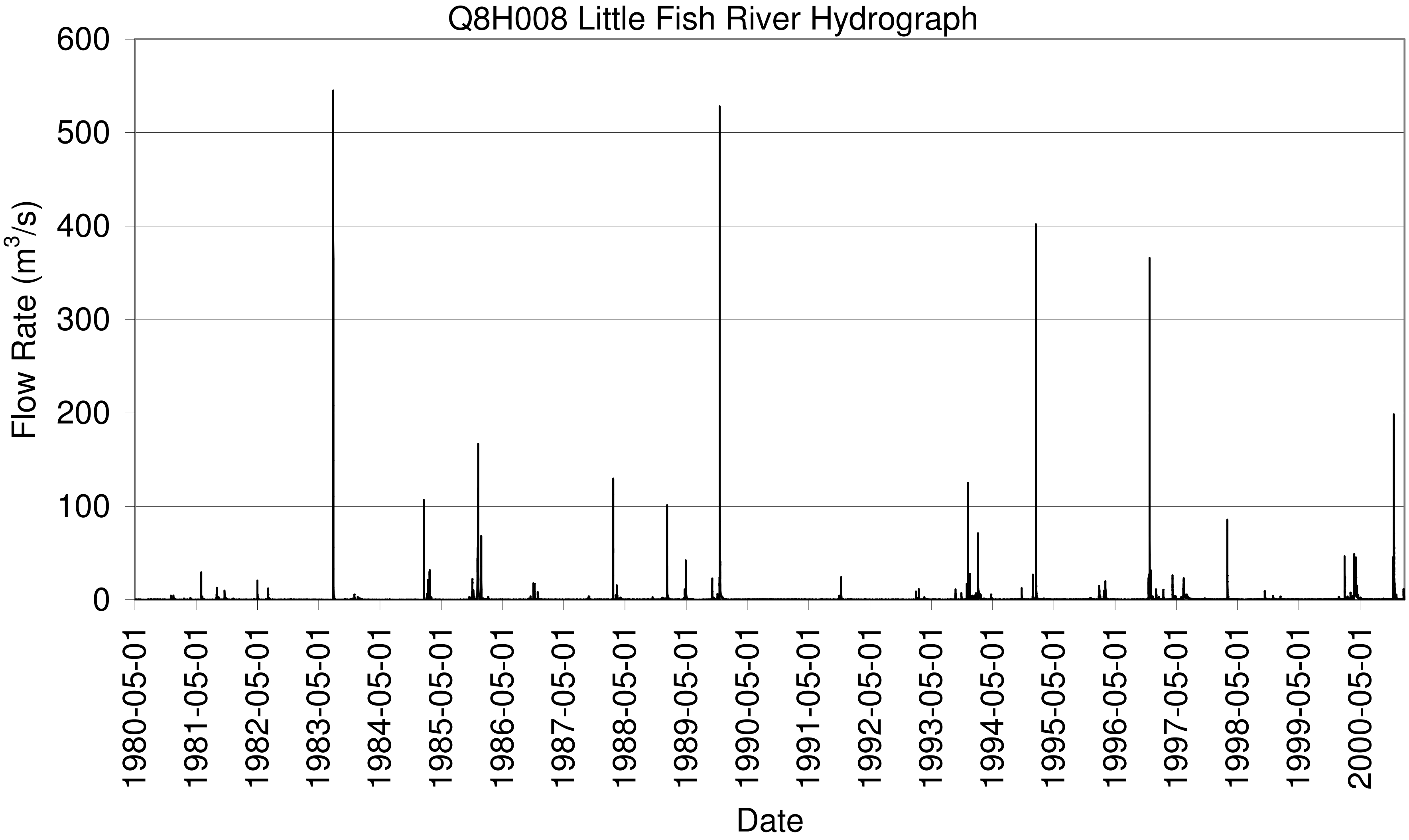



**Figure G-16: Flow Hydrograph for the Little Fish River at Doornkraal**