THE IMPLEMENTATION OF THE WATER RELEASE MODULE OF THE ***WAS*** PROGRAM AT THE VAALHARTS WATER USERS' ASSOCIATION

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BLOEMFONTEIN
APRIL 2008

DECLARATION OF INDEPENDENT WORK

I, **ARNO JANSEN VAN VUUREN**, Identity Number [redacted] and Student Number 9704957, do hereby declare that this research project, submitted to the Central University of Technology, Free State for the degree **MAGISTER TECHNOLOGIAE: ENGINEERING: CIVIL**, is my own independent work.

This work has not been submitted before to any institution by me or, to the best of my knowledge, any other person in fulfilment of requirements for the attainment of any qualification.

	15 April 2008
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SIGNATURE OF CANDIDATE	**DATE**

OPSOMMING

Kos en water is twee basiese menslike behoeftes. Internationale vooruitskattings dui aan dat watertekort sal voortduur onder armer gemeenskappe, waar bronne beperk is en populasiegroei spoedig is, byvoorbeeld die Midde Ooste, dele van Asië en Afrika. Voorlopige beramings dui aan dat Suid-Afrika uit oorskot, bruikbare water sal loop teen 2025 of kort daarna. Stedelike en semi-stedelike gebiede sal gevolglik nuwe infrastruktuur en opvangsgebied water oordrag vereis om veilige water en sanitasie te lewer. As gevolg van die hoë koste van hierdie ontwikkelinge, word laasgenoemde water gesien as gebruik vir industriële en publieke behoeftes en nie vir besproeiings doeleindes nie. Huidiglik gebruik besproeiingswater verbruikers die meerderheid van die totale verbruik. In oordenking met die bogenoemde, is dit ‘n realiteit dat die besproeiingsektor in die toekoms water sal moet afstaan aan publiek en industriële gebruik. Hierdie dui op groeiende konflik tussen verskillende water gebruikers en landbouwater gebruikers. In ‘n poging deur die Departement van Waterwese en Bosbou (DWAF) om hierdie konflik aan te spreek is loots projekte geimplimenteer om die stappe te bepaal wat die Water Gebruikers Vereenigings (WGV) kan volg om meer effektiewe water gebruik deur die landbousektor te verseker. Hierdie stappe sluit ‘n toename in besproeiings doeltreffentheid volgens die standaarde van gewas besproeiingsbehoeftes asook meer effiektiewe dam – en kanaal bestuur in.

Die Water Administrasie Stelsel (WAS) is ontwikkel om in hierdie spesifieke behoefte te voldoen deurdat dit optimale lewering van aangevraagde besproeiingswater verseker. Die program is ontwerp as ‘n bestuurs hulpmiddel vir besproeiingskemas, WGVs en water bestuurskantore om hul rekeninge te bestuur, asook om water lewering aan kliente meer effektief deur kanaal netwerke, pype en riviere te lewer. Die WAS program bestaan uit vier modules wat geintegreer is in ‘n enkele program. Drie van die modules van die WAS program is alreeds geimplimenteer by die Vaalharts besproeiingskema. Hierdie skema is oorgeskakel van ‘n regerings beheerde skema na ‘n privaat beheerde skema en staan nou bekend as die Vaalharts Water Gebruikers Vereeniging (VHWGV). Die hoofdoel van hierdie studie is om die uitstaande module van die WAS program te implimenteer by die VHWGV deurdat slegs die volkome funksionaliteit van die WAS program in geheel, effektiewe water gebruik by die skema kan verseker.

Die vierde module bereken die volume water wat losgelaat moet word in alle kanale (hoofkanaal en alle sytakke) inaggenome looptyd, water verliese, en opeenhopings om verlies te minimaliseer met gevolglike water besparing. Die metodologie wat gevolg is in die studie was eerstens om ‘n verstandhouding te vorm van waterverdelingsiklus en die huidige berekeningsprosedure by die VHWGV. Die vierdie module is daarna toegepas op ‘n tipiese voerkanaal, gebruik om die loslatingsvolume te bereken en gevolglike die resultate met huidige

waardes te vergelyk. Die volgende stap was toe om alle data, uitgetrek uit die databasis en gebruik deur die WAS program om die loslatingsvolume te bereken, te verifieer. Die databasis bestaan uit inligting soos dwarssnit eienskappe, posisies van sluise, kanaal- helling, asook kanaal- kapasiteit. Die verifieering van data was gedoen deur veldwerk, bestudering van bestaande ingenieurstekeninge, vergaderings en konsultasies met alle belanghebbende partye betrokke by die VHWGV, te belê en wiskundige berekeninge. Nasorg en verifieering, soos nodig, was op alle bogenoemde data gedoen. Nà die verifieeringsproses is die databasis opgedateer en ‘n finale siklus van berekeninge uitgevoer om die finale kalibrasie te verkry. Akkurate kalibrasie was gedoen aan deursyfering en looptyd. Geringe aanpassings is ook gedoen aan die kanaal geometrie in die databasis. Hierdie was ‘n belangrike gedeelte van die studie aangesien slegs ‘n vertroude en geverifieerde databasis korrekte resultate sal lewer, onhafhanklik van die sagteware program wat gebruik word.

Nà kalibrasie van die databasis was die vierdie module weer toegepas, maar hierdie keer is water verliese ingesluit in die berekeninge en die resultate het betroubare en akkurate ware loslating volumes gelewer. Die studie het dus geslaag in die implimenteering van die vierde module op ‘n tipiese voerkanaal by die VHWGV. Die studie was afgesluit met die opstel van ‘n merklys wat die VHWGV kan gebruik om die laaste module op die hele skema te implimenteer. Dit sal die VHWGV in staat stel om die volle WAS program, wat die voordele en antwoorde van enige behoeftes van ‘n water bestuurs kantoor bied, te implimenteer en aan te wend. Volhoubare waterhulpbron benutting kan slegs bereik word deur voldoende bestuur. Deur die toepassing van hierdie effektiewe bestuursprogram kan koste- effektiewe en ‘n geoptimiseerde proses by die VHWGV verseker word.

SUMMARY

Food and water are two basic human needs. International projections indicate that water shortages will be prevalent among poorer countries where resources are limited and population growth is rapid, such as the Middle East, parts of Asia and Africa. Provisional estimates are that South Africa will run out of surplus usable water by 2025, or soon thereafter. Urban and peri-urban areas will therefore require new infrastructure and inter-basin transfers to provide safe water and adequate sanitation. Due to the high cost of these developments, such water is seen as being used for industrial and public needs only and not for irrigation. Currently, the agricultural water users consume the majority of the water used by humans. Taking cognisance of the before mentioned it is a reality that in the future the irrigation sector will have to sacrifice some of its water for public and industrial usage. This suggests growing conflict between the different water users and the agricultural water users. An attempt by the Department of Water Affairs and Forestry (DWAF) to address this conflict has been the implementation of pilot studies to determine the steps Water User Associations (WUAs) could take to ensure more effective water use in the future by the agricultural sector. These steps include an increase in irrigation efficiency according to the benchmarks of crop irrigation requirements and more efficient dam and canal management.

The Water Administration System (WAS) has been developed to fulfill this exact requirement as it ensures optimal delivery of irrigation water on demand. The program is designed as a management tool for irrigation schemes, WUAs and water management offices to manage their accounts, and also to manage water supply to clients more efficiently through canal networks, pipelines and rivers. The WAS program consists of four modules that are integrated into a single program. Three modules of the WAS program have already been implemented at the Vaalharts irrigation scheme. This scheme has been transformed from a government controlled scheme to a privately owned scheme, and is now known as the Vaalharts Water User's Association (VHWUA). The main purpose of this study was to implement the fourth module of the WAS program at the VHWUA as only full functionality of the complete program will ensure effective water use at the scheme.

The fourth module calculates the volume of water to be released for all the canals (main canal and all its branches), allowing for lag times, water losses and accruals in order to minimise waste and thus save water. The methodology followed in this study was to first of all develop an understanding of the distribution cycle and the current calculation procedure of the VHWUA. The fourth module was then applied on a typical feeder canal and used to calculate the release volumes in order to compare these results with the current values. The next step was then to verify all data abstracted from the database used by the WAS program to calculate the release volumes. The database consists of information like

cross-sectional properties, positioning of the sluices, canal slope, as well as canal capacities. The verification of data was done by field work, by studying existing engineering design drawings, through meetings and consultations with all parties involved in the VHWUA as well as by mathematical calculations. Cross-checking and verification, if necessary, of all above mentioned data were done. After the verification process, the database was updated and another cycle of calculations were run to do the final calibrations. Accurate calibrations were done to the seepage and the lag time coefficient. Some final adjustments were also made to the canal geometry in the database. This was an important part of the study as only a trusted and verified database will deliver correct results, irrespective of the software program used.

After calibration of the database, the fourth module was again applied, but this time water losses were included in the calculations and the results revealed trustworthy and accurate real-time release volumes. The study therefore succeeded in the implementation of the fourth module on a typical feeder canal at the VHWUA. The study was concluded by the compilation of a checklist, which the VHWUA can use to implement the module on the whole scheme. This would enable the VHWUA to implement and apply the complete WAS program, which offers all the benefits and answers in every need of any water management office. Sustainable water resource utilisation can only be achieved through proper management. Applying this most effective management program will ensure a cost effective and optimised process at the VHWUA.

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Introduction

CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

The philosophy of sustainable management of water resources in South Africa is neatly encapsulated in the slogan of the Department of Water Affairs and Forestry (DWAF): *"Some, for all, forever"*. Projected global water scarcity is shown in Figure 1.1 and South Africa is one of the areas indicated to experience physical water scarcity in 2025 (FAO, 2006). Therefore, to make this philosophy a reality and ensure water availability to everyone even after this date, we will need to use our water resources much more efficiently and pay careful attention to the Israeli Water Directorate's slogan, which states that *"...no one may waste a single drop of water that another man may turn into bread..."*.

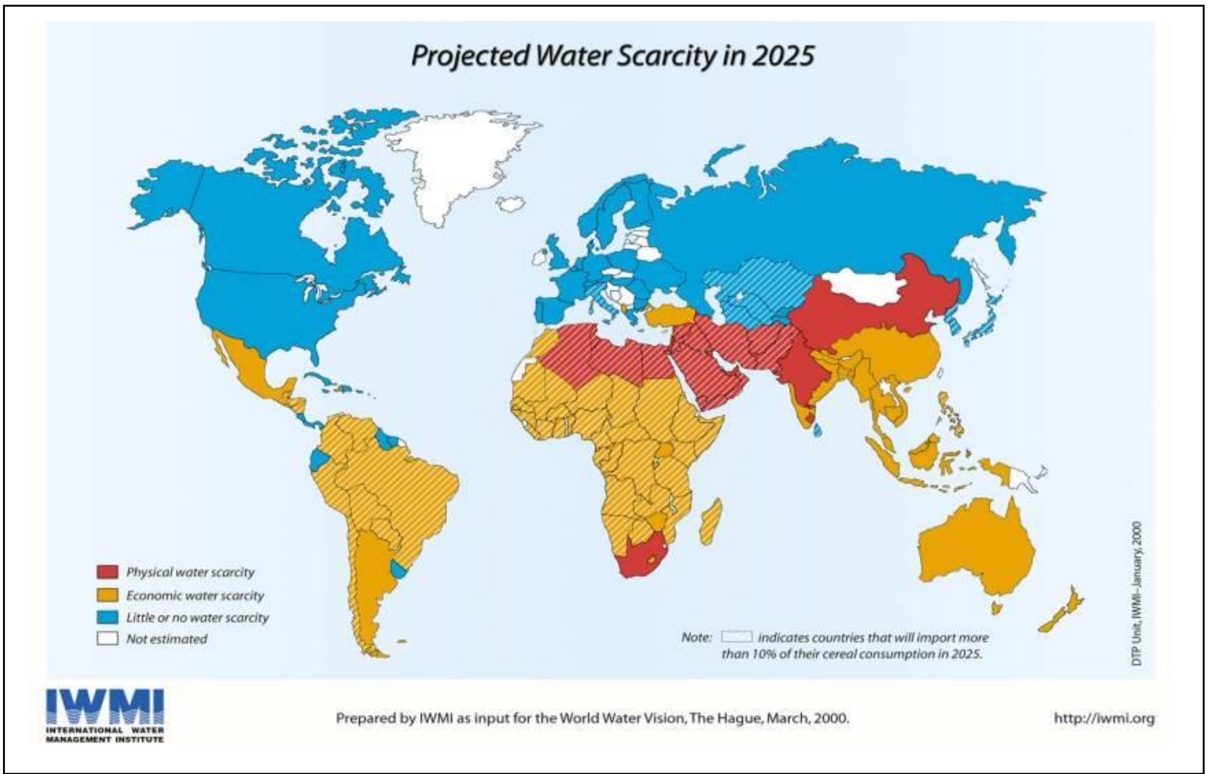


Figure 1.1: Projected Water Scarcity in 2025

(from http://www.iwmi.cgiar.org)

The total annual surface runoff of South Africa is estimated at 48.2 km^3, or approximately 9 percent of annual rainfall (FAO, 2006). However, much of the total runoff volume is lost through flood spillage and evaporation, so that in 2000 the available yield was estimated at 3.9 km^3/year only. About 4.8 km^3 of groundwater is produced per year, of which an estimated 3 km^3 is in turn drained by the rivers. Available yields from these resources are estimated at 1 km^3/year in 2000. Taking into consideration the overlap between surface water and groundwater, total internal renewable water resources are estimated at 44.8 km^3/year out of the total actual renewable water resources, which are estimated at 50.0 km^3/year, including incoming water from other countries (Aquastat, 2006). Provisional estimates, which confirms the international projections, are that South Africa will run out above mentioned surplus usable water by 2025, or soon thereafter. Inter-basin transfers are in place and more are planned, but due to the high cost of this development, such water is seen as being used for industrial and public needs only and not for irrigation. In 2000 the total water withdrawal from the different economical sectors was estimated at 12.5 km^3, with irrigation accounting for 62 percent (Figure 1.2; Aquastat, 2006). Apart from the fact that the irrigation sector will in future have to sacrifice some of its water for public and industrial usage, it can also play an important role in saving water by increasing irrigation efficiency through more accurate calculations of crop irrigation requirements and also through more efficient dam and canal management at irrigation schemes.

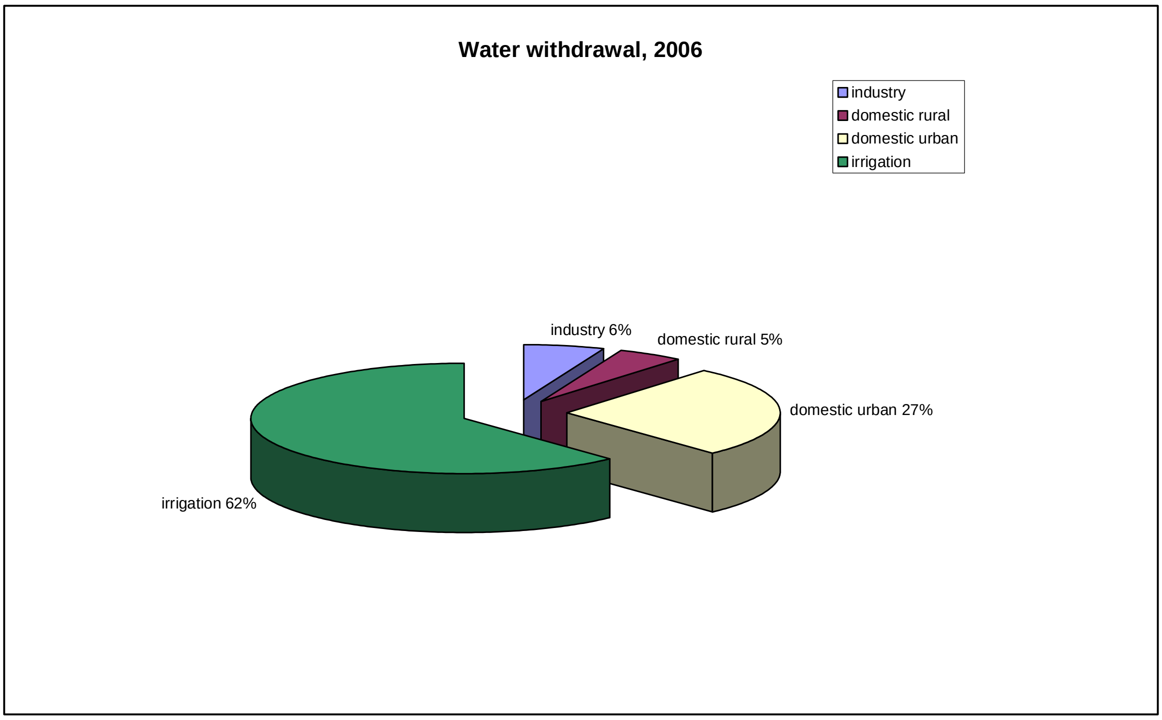


Figure 1.2: Water withdrawal of different economical sectors (from http://www.iwmi.cgiar.org)

1.2 IRRIGATION DEVELOPMENT AND WATER MANAGEMENT

From time immemorial man has scooped water from a pit or relayed water from a river to irrigate his crops, often producing just enough to feed his family for a certain period of time. However, modern day irrigation farmers focus on crop production, which has become an income-generating business. Irrigating the land on a regular basis increases the crop production that in turn results in an increase in financial gain (De Lange, 2000). Based on the water availability and land suitability, the potential for full or partial control irrigation development, is estimated at 1.5 million ha (FAO, 2006). The most important methods of irrigation currently used are flood irrigation (Plate A3 and Plate A4, Appendix A), sprinkler irrigation (Plate A5 and Plate A6, Appendix A) and micro-irrigation on 28,5%, 53% and 18,5% respectively of the total area irrigated (WRC, 2000a). Water is applied for the production of a wide range of field, industrial, horticultural, pasture and fruit crops. Under normal conditions, irrigated agriculture makes a substantial contribution of between 25 to 30% towards gross agricultural production (WRC, 2000a; SA Water bulletin, 2001).

To ensure the effective distribution between the different economical sectors, including irrigation, the resource is needed to be managed by well organised bodies. According to Görgens, Pegram, Uys, Grobicki, Loots, Tanner and Bengu (1998) water resource management in South Africa has been affected and therefore transformed significantly by the following two events:

- The democratisation of the Republic of South Africa, and
- The need for new approaches to water resource management due to misuse and mismanagement of available water resources.

The ministries involved in irrigation development and water management, which are thus responsible to identify and to implement these new approaches, are (Aquastat, 2006):

- The Ministry of Water Affairs and Forestry, through the Department of Water Affairs and Forestry (DWAF), monitors surface water and groundwater resources, formulates the national water strategy and is responsible for the implementation of the National Water Act;
- The Ministry of Agriculture, through the National and Provincial Departments of Agriculture (NDA and PDA), promotes irrigation

engineering concepts and is responsible for agricultural extension with the aim of improving irrigation efficiency;

- The Ministry of Land Affairs is responsible for the settlement of new farmers.

The development of new irrigation schemes and the upgrading of existing schemes for commercial agriculture are coordinated between the relevant departments by provincial liaison committees, known as the Irrigation Action Committees (IACs). Development and upgrading of irrigation schemes for non-commercial agriculture are coordinated by the Coordinating Committee on Small-Scale Irrigation Support (CCSIS).

The National Water Act, Act 36 of 1998 (South African Government, 1998) stipulates water management by Water User Associations (WUA) at the local level. Each WUA will have all water users in an area as members, and the local management should eventually be the total responsibility of the WUA. Each WUA will have an elected management body, with all sectors of water users represented on the committee. Several WUAs will fall under an umbrella organization, the Catchment Management Agency (CMA). Each CMA will have an area of responsibility, which could be a catchment, a portion of a large catchment or a combination of small catchments. Nineteen such water management areas have been identified (South African Government, 2002). The administrative procedures for converting irrigation board areas, private schemes and government water schemes into WUAs are currently in progress. Until the CMAs are in place, the DWAF will carry out their function. The conditions for the use of water that the CMAs and WUAs must oversee include:

- Safeguards against water wastage and low efficiencies of water use;
- Equal access to water by all interested parties; and
- Safeguards against pollution.

1.3 PROBLEM STATEMENT AND PURPOSE OF STUDY

The formulation of the problem statement resulted from the previous mentioned discussions and conditions and can be stated as follows:

“Sustainable water resource utilisation cannot be achieved without proper management. To ensure maximum efficiency of water use and minimisation of water wastage an effective irrigation management program, which is cost effective and which will produce an optimised process must be applied at the study area.”

The study area is the Vaalharts irrigation scheme. This scheme has been transformed from a government controlled scheme to a privately controlled scheme, and is now known as the Vaalharts Water User’s Association (VHWUA). The Water Administration System (WAS) has been developed to ensure optimal delivery of irrigation water on demand. The program is designed as a management tool for irrigation schemes, WUAs and water management offices to manage their accounts, and also to manage water supply to clients more efficiently through canal networks, pipelines and rivers (Benade, 2001a). The WAS program consists of four modules that are integrated into a single program. Three modules of the WAS program have already been implemented at the Vaalharts irrigation scheme. The main purpose of this study was therefore to implement the fourth module of the WAS program at the VHWUA as only full functionality of the complete program will ensure effective water use at the scheme.

1.4 FRAMEWORK OF THE STUDY

To address the problem statement and fulfil the purpose and objective of the study, the first step was to compile a structure / framework for the study (Mokwena and Furumele, 2001). The different tasks identified to be undertaken are described below.

The WAS program was identified as the management tool to by fully implemented at the VHWUA in order to manage water resources effectively. The four modules of which the WAS program consists of can be integrated into a single program that can be used on a single PC, a PC network system or a multi-user environment. The PC network system is currently in use at the VHWUA. The four modules are:

- An administration module;
- A water request module;
- A water accounts module; and
- A water release module.

The first three modules are already in use at the VHWUA, while the water release module has been partially implemented. This module links with the water administration and water request modules and calculates the volume of water to be released for the main canal and all its branches, while allowing for lag times, water losses and accruals. As indicated in the previous section, the purpose of this study is to maximise irrigation efficiency and minimise waste. To realise this it is necessary that the complete WAS program, which offers all the benefits and answers in every need of any water management office, must be implemented at the VHWUA. This implies the calibration and implementation of the final module (water release module) to improve on the current means of calculating the volume of water, as requested by the farmers, to be released.

First it was necessary to develop a detailed understanding of the water distribution process in South Africa and the different methods currently in use to calculate water release volumes. The WAS program was studied in detail, with focus on the implementation and application of the fourth module at an irrigation scheme. The next step was to determine and understand the current calculation procedure used at VHWUA. A typical feeder canal was then identified for implementation of the fourth module and test runs. Water release volumes were determined and the results obtained were then compared with the current values. Following this, it was necessary to verify all data abstracted from the database used by the WAS program to calculate the release volumes. The information captured in the database is detail about the cross-sectional properties of the canal, positioning of the sluices, canal slope, as well as canal capacities. The verification of data was done by field work, by studying existing engineering design drawings, through meetings and consultations with all parties involved in the VHWUA as well as by mathematical calculations. Cross-checking and verification of all above mentioned data were done. After the verification process, the database was updated and another cycle of calculations were run to do the final calibrations. Accurate calibrations were done to the seepage and the lag time coefficient. Some final adjustments were also made to the canal geometry. This was an important part of the study as only a trusted and verified database will deliver correct results, irrespective of the software program used.

After calibration of the database, the fourth module was again applied, but this time water losses were included in the calculations and the results revealed trustworthy and accurate real-time release volumes. The final step was to design a checklist to be used by the VHWUA, to verify and calibrate all canals which would result in minimisation of water loss of the whole canal network. Implementing the fourth module will enable the VHWUA to implement and apply the complete WAS program and therefore move towards a more sustainable management process and improved utilisation of the water resources. Succeeding in the latter, the VHWUA will also succeed in working towards saving water for the next generations to come and fulfilling the slogan of the DWAF as mentioned in the first paragraph of this chapter.

A detailed methodology of the research project is given in Chapter 4. The theoretical concepts on which the research is based are discussed and explained in the next chapter.

Chapter 2

Literature review

CHAPTER 2 LITERATURE REVIEW

2.1 INTRODUCTION

The main aim of the study was to implement the WAS program at the VHWUA. The management tool to be used in this implementation was a computer program that does the same work as what normally would have been done manually, but the benefits and advantages of information technologies have been harnessed to ease the process. This implies that the same methods will be applied on computer, but it is necessary to understand the basics of the calculation procedures and processes in hydraulics. This chapter will discuss the relevant engineering hydraulics, considerations for flow calculations, water distribution principles, and some basics of irrigation management.

2.2 WATER DISTRIBUTION

2.2.1 Introduction to water distribution

When authorities are responsible for distributing water to consumers, various factors must be taken into account. There is a need for a governing body to oversee the fair distribution of the water. Research into the National Water Act, Act 36 of 1998 (South African Government, 1998) reveals that the core idea of the Act is to conserve, control and rightfully distribute water in South Africa. The water control officer of any irrigation scheme should understand this thoroughly (DWAF, 1986). Of all the issues that exist in the distribution of water, the most important aspect is to distribute the available water equitably between all the interested parties without any water being wasted. The water control officer needs to balance the interests of the irrigators with the relevant legislation in order to distribute water in the most efficient manner. Through this optimal resource utilization can be achieved while complying with the National Water Act. A full understanding of water distribution dynamics will ensure that the intentions of the Act are met.

2.2.2 The aim of water distribution

The purpose of water distribution is to make water available at a specific time, for a predefined period at an established flow rate at a certain point for the best benefit of the irrigation community. In normal circumstances municipalities supply water to users (towns or densely populated areas) at a certain tariff which constitute income for the municipality (Pretorius, 1996; Turner, Fowler, Manson and Stephenson, 1997). Water will therefore be available when required in the desired volume simply by opening a tap. Unfortunately the same does not apply to irrigation systems. The cost of pipes and pumps is far higher than that of canals, and the distances over which the water must be transported is much greater in relation to the number of users. In order to deal with these problems, engineers must design the most appropriate types of canals for the water needs of the crops and soil types in the region. This will supply users with the required amount of water as long as their needs remain within the limitations of the system. The flow rate of the water and the time required for the water to move through the canal depends on the canal geometry, slope and roughness.

In pressure pipe systems, water is available immediately at the specified abstraction point upon the request of the user. With gravitation systems, however, the availability of water must be calculated or determined by experience and can only be available to the user after a time traveled in the canal. This is referred to as the *lag time* of the water. All needs must, however, be within the limitations of the irrigation scheme.

2.2.3 Scheduling and water rights

Supplying limited water resources to areas where water requirements vary from one crop type to another and one region to another, has led to the demarcation of irrigation districts, Water Users Associations and Government water schemes (IAHS, 2001). Government water schemes will benefit the community where perennial rivers feed weirs, or where a series of weirs or dams collects flood water. To determine the extent of the irrigation it is necessary to consider the available water that can be collected annually, the water requirements of the crops, irrigation methods and the available land that is suitable for irrigation and types of irrigation.

Irrigation schemes are formed when a number of interested users, each with a right to the water from a public stream, construct dams and canals in order to fulfill their water needs. DWAF will in turn schedule the area according to the scope and the size of the land. The National Water Act (South African Government, 1998) gives authority to scheduling councils, which deal with the scheduling of individual premises and farms. A list is then compiled which indicates the land, the title deeds, the registered owner and the scope of the scheduling area. This list is reviewed annually or as required. All the parties eligible for water are thus indicated on this list, which is signed by the Minister. A user now has water rights from the source or canal enabling them to utilise available water for irrigation purposes.

2.2.4 Water quota

As mentioned above, the National Water Act (South African Government, 1998) requires that the amount of available irrigation water be publicised in the South African gazette annually. This refers to the water quota of an irrigation scheme and is expressed in terms of cubic meters per hectare (m^3/ha). Quotas are calculated annually according to the available water in the source and the scheduled area per irrigator. This needs to be done annually due to variations in climate and rainfall, expected outflow, evaporation, distribution losses, reserves for fish life and domestic use. In the same calculation the quota per user per hectare is also determined. The full quota of an irrigator in m^3 (in a given water year) is calculated by multiplying the quota (m^3/ha) by the scheduled area. The quota system is applicable in areas where water supply for the area is reasonably assured (DWAF, 1996).

2.2.5 Balancing dams, feeder dams and weirs

For the distribution and control of water in irrigation schemes, water control offices may use balancing dams. These dams are specifically designed according to the needs of the users and could include any of the following:

- Weirs to act as balancing dams. After the water has been released from the source, water can be stored in the weir and then let into the canal system.
- If the topography and the geographic position of the site are inadequate for irrigation/holding dams, then communal irrigation dams are built, acting as balancing dams, from which a group of users can then abstract water.
- It is impossible for the last user on a community canal or on the end of feeder and main canals to abstract all the remaining water. Balancing dams are required in order to catch the excess water to use for redistribution.
- In long canals there are often fluctuations in the flow of water in the canal due to poor opening/closing adjustments of sluices. Balancing dams will make water available when needed, thus balancing out the fluctuations.
- The dams make provision for water to be available at remote turnouts of the scheme on time and with minimum losses. Such a dam acts as a smaller source lower down the irrigation canal.

A balancing dam therefore serves the purpose of collecting excess water or supplementing shortages that may develop in the canal below the balancing dam. Water that previously may have been lost to the irrigation scheme can now be reused in the form of a supplementary source. Determining this gain or loss in a balancing dam can make daily calculation easier as a more accurate volume can be released from the original source. It is therefore necessary to install a measuring plate at the dam in order to determine the amount of water in the dam, inflow and outflow of water.

Another form of a balancing dam is private irrigation- or feeder dams (Plate A9, Appendix A). Farmers use these dams to collect water to schedule their own irrigation program for the farm. They do this because a single stream of water delivered by a sluice may not be enough to satisfy a certain irrigation need. Irrigation water can now be abstracted from the dam as the need arises, however the loss of water due to evaporation is the loss of the irrigator.

Weirs in a canal system can serve to keep the water level at an outlet sluice constant, so that a uniform flow rate through the sluice may be achieved. Long weirs in the canal system may also be regarded as balancing dams although they may contribute to water loss due to water collected by the weirs and evaporation. The losses are calculated in the same manner as for normal balancing dams. The capacity of the canal can be determined by measuring the time it takes to fill the weir of which the volume is known. This is done by installing a measuring plate at the deepest end of the weir and reading of the height of water versus the time it takes to fill the weir. Volume thus equals flow rate multiplied by time. This will produce a graph that will be used to determine the lag time in the weir and other sluices.

2.2.6 Water Loss

In a realistic approach one must realise that in any canal system there will always be water loss. It depends on the controlling body and on how it will be handled and controlled. The best that can be done is to try to minimise loss as far as possible. Controlling and limiting these losses will reflect on the efficiency of the irrigation scheme. Losses in a canal system are defined as follows:

- Leakage loss: Water with its density and viscosity has the ability to move through very small holes and cracks and can even filter through densely compacted soil (Plate A7, Appendix A). Mechanical distribution systems are also subject to leakages, depending on the type of construction material used. It is obvious that at any given time there will be leakage problems in the canal due to natural weaknesses in sluices and faults in the concrete lining. The density of the material of the canal wall and the friction slope of the canal floor largely determines how much water will seep away through the walls. Seepage is therefore defined as water that is lost due to water moving through the sidewalls of the conduit.
- Spilling loss (also operational loss): Water is an incompressible fluid and will move from a point of high energy to a point of low energy (Chapter 2), in other words from an upstream point to a downstream point. The cross-sectional surface, friction coefficient and the slope of

the canal control the flow rate and flow speed of the water in the canal. Weirs in the canal will cause fluctuations in the flow profile due to changing backwater profiles. The purpose of the weir is to dam the water up in order to distribute it to the various sluices of the irrigators. If the calculation of the release time of the water is incorrect or the specific sluice is closed, the result will be water flowing past the point of abstraction. Water is also lost if the canal overflows. This wastage of water due to poor administration or calculation is referred to as spilling loss. It can therefore be seen as unaccounted for water.

- Transit loss: In some instances water is transported through a river to the users, giving rise to the possibility of seepage through the riverbed and damming at the weir. Transit loss refers to the loss of water as a result of the transportation of water through the canal to the end user.
- Compensation loss: Losses in a canal will occur, however, the loss needs to be minimised as far as possible. The volume of water that needs to be added to the original demand to obtain the least actual loss is referred to as compensation loss. The volume of water cannot be calculated from an equation or a table, but rather from local knowledge, personal experience and statistics on the behavior of the water in the specific field of application. In order to increase the efficiency of the scheme, compensation loss (consisting of leakage and spilling and transit losses) can be reduced by continuous maintenance and improvements to the canal system. It is therefore necessary to determine the losses on each and every main, feeder and community

canal. An accurate indication of losses for a given canal will be obtained by consistently recording the following:

- volume of water that was supplied to consumers but does not appear on the feeder charts;
- volume of water that was collected in the balancing dams;
- volume of water that has run off at the end-point of all the canals;
- actual flow rate at each measuring device and distribution sluice where water is released;
- actual time when each plot sluice is opened and closed;
- amount of water flowing into the canal from balancing dams, small tributaries, etc.;
- actual time when changes were made and the corresponding flow;
- actual time when the dam-sluices were opened and the actual flow rate.

Losses are included at the VHWUA only as a percentage of the total volume. Calculation of loss plays an important role as the efficiency of the scheme and/or canal can be determined. If a certain canal proves to be lacking the ability to perform efficiently it is evident that severe losses will take place in the canal. Applying a defined calculation procedure can then be used to determine the loss, thus enabling the irrigation official to rectify and optimise both the canal and the scheme by eliminating the losses.

2.2.7 Feeder charts, feeder streams and maximum abstraction right

One of the administrative tools required for an irrigation scheme is feeder charts. By using feeder charts all the delineated areas of the scheme can be accommodated in the accurate daily water requirement determination. Users will indicate on feeder charts exactly when, where and how much water should be delivered. Depending on the needs of various schemes a variety of charts can be used. On some charts water request, water cancellation and additional water application can be made, while other schemes uses separate forms for the various requirements. The use of feeder charts will be discussed in Chapters 2 (Figures 2.10 – 2.12).

Users can only receive the requested feeder stream at a turnout on a community canal. Feeder streams refer to the flow rate of a requested volume of water that can be delivered at the mentioned point. The flow rate of a feeder stream is determined by the carrying capacity of the canal, crop type and the scheduled area. The flow rate also determines the time needed for a stream of water to flow between any two given points. The efficiency of the water distribution system is influenced by the accurate determination and effective application of the time a stream of water will take to flow from one point to the other. The incorrect calculation of this time will result in water passing the intended abstraction point (loss) or arriving late for abstraction. Figure 2.1 shows a canal transporting 1000 m^3/h of water. The water travels for four hours from the source (weir/dam) to reach the first sluice, and thereafter one hour to the next sluice. This has the effect that if the full 1000 m^3/h is released from the source, the first sluice can only be opened four hours later, the second five hours later, and so forth. Water is wasted if sluice #1 is opened an hour too late, as the water that should have been abstracted there has already progressed to sluice #2. If sluice #7 is opened before the scheduled time, the water intended for sluices #8, #9 and #10 will be withdrawn at sluice #7 (if the capacity of the sluice does not allow), with the result that there will be a shortage of water at the lower points for two hours, after which it will return to normal.

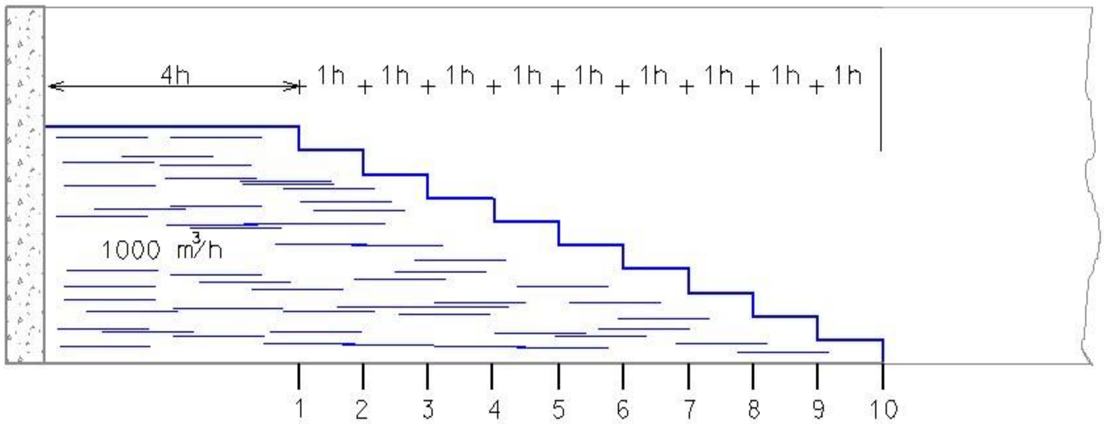


Figure 2.1: Lag time of flow rate

Figure 2.1 also shows the effect that the flow time of water can have on an irrigation scheme. For this purpose flow-time charts are compiled for each canal in order to be able to make adequate predictions (also see chapter 4). Depending on the intended use of the requested volume of water, the feeder stream cannot always meet the request. It is thus suggested and encouraged that irrigators build feeder dams with a minimum capacity of 24 hours to store water for their required use. Water scheduling needs of the irrigator can then be fulfilled by this feeder dam and the irrigator scheduling need not depend on the supplied feeder stream only. Use of the feeder stream and application to the irrigation land will therefore be optimised.

By applying the formula V = t x Q, where

V = volume (m^3)

T = time (sec)

Q = flow rate (feeder stream) (m^3/s)

the volume of water that can be delivered to a piece of scheduled plot during a given time under a given flow rate can be determined. The maximum abstraction right (m.a.r.) of any canal system refers to the maximum volume of water that may be delivered at full request per scheduled hectare during the irrigation period, in other words,

m.a.r. = (t/a) x Q where:

m.a.r. = maximum abstraction right (m^3/ha)

t = irrigation period (h)

a = scheduled area (ha)

Q = flow rate (feeder stream) (m^3/s)

The m.a.r. shows the volume of water that an irrigator can claim per irrigation year. In the calculation of the m.a.r. all canals in the scheme need to be calculated using values that incorporate the actual carrying capacity and losses in the canal. No theoretical values may be used. If all other users along the canal request water below the m.a.r. and only one, or perhaps a few, request water more than the m.a.r., then provision might be made for the additional requests. With water conservation in mind the m.a.r. ensures that the canal will never be overloaded, and water loss will be kept to a minimum.

2.2.8 Water distribution systems

The main purpose of water distribution is the availability of a requested volume of water at a specific time and specific period. Over the years several systems have been developed to achieve this purpose, namely:

- Irrigation turn system: The irrigation turn system is the oldest system and works on the basis that in a given period a single user will only receive water for a requested time. This system is still used in areas where the water stream fluctuates, depending upon the season of the year. Types of irrigation turn systems are:
  - single-stream distribution;
  - multiple-stream distribution;
  - adjusted irrigation turn system; and
  - periodic irrigation turns.
- Request system: During the rainy season reserve water is collected that makes it possible to use the request system. With the constant supply of water interested parties, each with a share of the water, can request any given amount at any given time in advance as long as it falls within the regulations of the irrigation scheme. The basis is thus distribution of the available water among the interested parties in accordance with a time-stipulation per unit area while taking the maximum abstraction right into account. There are two types of request systems:
  - in the individual request system each user will request water only for his own use, according to his irrigation needs;

- in the communal request system, several users will request water on one request form. The volume is released and each user is entitled to his requested volume and share.

- Self distribution system: In this age of automation new distribution systems exist where irrigators can withdraw water on their own predefined time using self-registering maximum flow-control valves. These systems are known as self-distribution systems.
- Mechanical system: Various forms of mechanical equipment to withdraw water from public streams have been in use since the previous century. In the system of mechanical withdrawal; the river may be regarded as the main canal and the pumps as the sluices or turnouts (DWAF, 1975).

2.2.9 Summary of requests and distribution orders

Once all the forms for the application of water are collected and feeder-charts have been compiled, a summary of all the requested water for a given week is compiled. This will determine how much water needs to be ordered for each receiving point so that it will be available on time for distribution. This summary is completed on a daily basis. Distribution orders can now be made out to all the applicable water control officers, instructing them which sluice to open when, and to what volume. As the total volume of water requested will be released from the source, water control officers should ensure that sluices are opened at the correct time in order not to waste any water. Any changes in the flow pattern, or any other irregularities, should be noted and reported to the main office.

A proper understanding of the working of an irrigation scheme forms the basis of any distribution of water on an irrigation scheme. One should understand the procedures according to which water is handled, requested, conserved and distributed to all interested parties. From this basis, various methods can be applied to streamline the process of water distribution and to make it more acceptable and applicable for current and future applications (de Lange, 2000). The VHWUA uses the water request system.

2.3 OPEN CANAL FLOW

2.3.1 General flow characteristics and hydraulic principles

The science of hydraulics is not new, as proven by the extensive hydraulic works constructed by ancient civilisations. Since then many principles and laws relating to the flow of fluids have been developed. Many of these laws, among them, Bernoulli's theorem of the conservation of energy, are still valid today and form an important basis on which modern and future science and hydraulics can be built (Kanen, 1986; Brater, King, Lindell and Wei, 1996). In studying hydraulics one cannot gain a full understanding of the field without studying the general terminology and governing principles of hydraulics. A better understanding of these fields will aid canal network design.

Water travels from a point of high energy to a point of low energy until it reaches the point of equilibrium. This natural occurrence is evident in examples of natural conveyance canals such as brooks, streams and rivers and the ocean (Haested Publication, 2003). All water flows from dams and catchment areas, situated inlands, through the conveyance systems until it reaches the ocean. Open canal flow is defined by Chow (1959) as the transportation of water along a purpose-made longitudinal opening where it is subjected to a free surface i.e. exposed to atmospheric pressure.

The aforementioned hydraulic concepts can be applied equally to both man-made structures and natural features. Open canal flow is the general term used to refer to types of free surface flow. Full flow, non atmospheric conditions can only exist in pipe flow and the therefore canal systems remain as free surface flow although the canal can flow totally full (Simon, 1986 and Ayoda, 1988). Unlike rivers, concrete canals can be called prismatic, parabolic, rectangular, etc. canals because of the consistent cross-sectional shape, slope and roughness coefficient. The cross sectional area of flow in any conveyance system can now be determined by using the specific cross sectional shape and depth of water in the conveyance system (Figure 2.2). For this specific area only a certain part of the conveyance section is covered with or in contact with water and is called the wetted perimeter. As shown in Figure 2.2, contact between the canal surface and the fluid is only on the bottom and sides of the canal in each canal reach under consideration. The hydraulic radius (1) is defined by the two above mentioned properties, cross sectional area of flow and wetted perimeter. The equation that defines the hydraulic radius is given as:

$$R = \frac{A}{P_w} \qquad \text{...(1)}$$

where:

A = cross-sectional area (m^2)

R = hydraulic radius (m)

P_w = wetted perimeter (m).

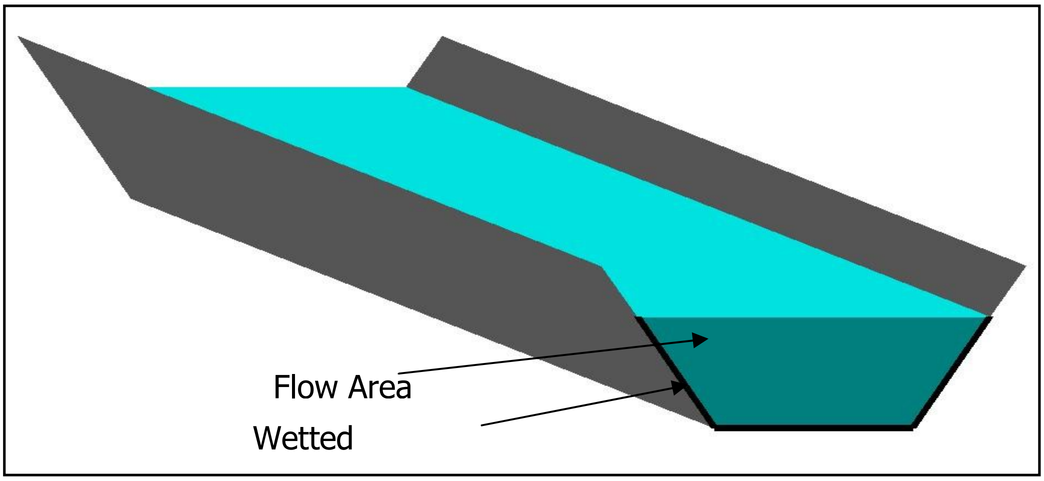
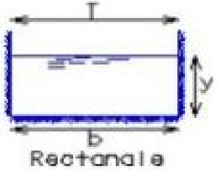
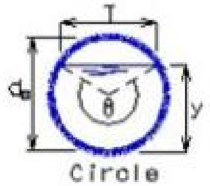
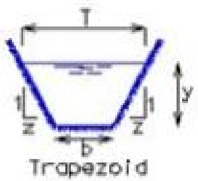
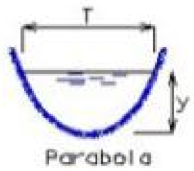
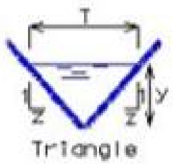


Figure 2.2: Cross-sectional area and wetted perimeter

A canal profile is the cross-sectional geometrical shape of a canal taken normally in the direction of flow. There are two types of canal sections: natural and artificial canals (Mays, 1996). In natural canal sections (river systems) the shape can take any form and it will remain fairly irregular. Engineers and hydrologists design artificial canals (e.g. concrete-lined canals) for certain purposes, so that the geometric shape will remain regular until intended otherwise. Table 2.1 shows the generally used types of canal sections. The area, wetted perimeter and hydraulic radius are all derived from the water flow depth, y (hydraulic depth).

Table 2.1: Canal section types (Chow, 1988)

Section	Area A	Wetted Perimeter P	Hydraulic radius R	Top width T	Hydraulic depth D
	by	$b+2y$	$\frac{by}{b+2y}$	b	y
	$\frac{1}{8}(\theta-\sin\theta)d_o^{\ 2}$	$\frac{1}{2}\theta\cdot d_0$	$\frac{1}{4}\left(1-\frac{\sin\theta}{\theta}\right)d_0$	$(\sin\frac{1}{2}\theta)d_0$ or $2\sqrt{y(d_0-y)}$	$\frac{1}{8}\left(\frac{\theta-\sin\theta}{\sin\frac{1}{2}\theta}\right)d_0$
	$(b+zy)y$	$b+2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b+2zy$	$\frac{(b+zy)y}{b+2zy}$
	$\frac{2}{3}Ty$	$T+\frac{8}{3}\frac{y^2}{T}$	$\frac{2T^2y}{3T^3+8y^2}$	$\frac{3}{2}\frac{A}{y}$	$\frac{2}{3}y$
	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$	$\frac{1}{2}y$

The hydraulic radius of the conveyance system is important as it is used to calculate the flow velocity of water in the canal by means of flow equations. All flow equations are compiled in terms of average velocity and hydraulic radius. The average velocity (in units of length per time) can thus be used to define the flowrate (2) of water passing through the given area and is given by the following equation:

$$\boxed{Q = A \cdot V} \quad \text{...} (2)$$

Where:

Q = flowrate (m^3/s)

A = cross-sectional area (m^2)

V = mean velocity (m/s).

Classifying open canal flow can be done either by the time criterion or by the space criterion. Two classifying categories of the time criterion exist namely steady- and unsteady flow. Steady flow can be defined as flow in a canal where neither the flow depth nor the discharge changes with time during the period under consideration. The flow is unsteady if the water depth or discharge changes over time (Chow, 1959).

In steady flow the water depth and discharge remain the same, but has space as the criterion, which means that the flow is the same in any section of the canal (Hwang and Houghtalen, 1996). Varied flow in an open canal may be either steady or unsteady, because the water depth and/or the discharge could change along the length of the canal.

Figure 2.3 shows the classifications of open canal flow: (a) steady, uniform flow; (b) unsteady, uniform flow; (c) steady, varied flow; (d) unsteady, varied flow; and (e) unsteady, varied flow (Hwang and Houghtalen, 1996).

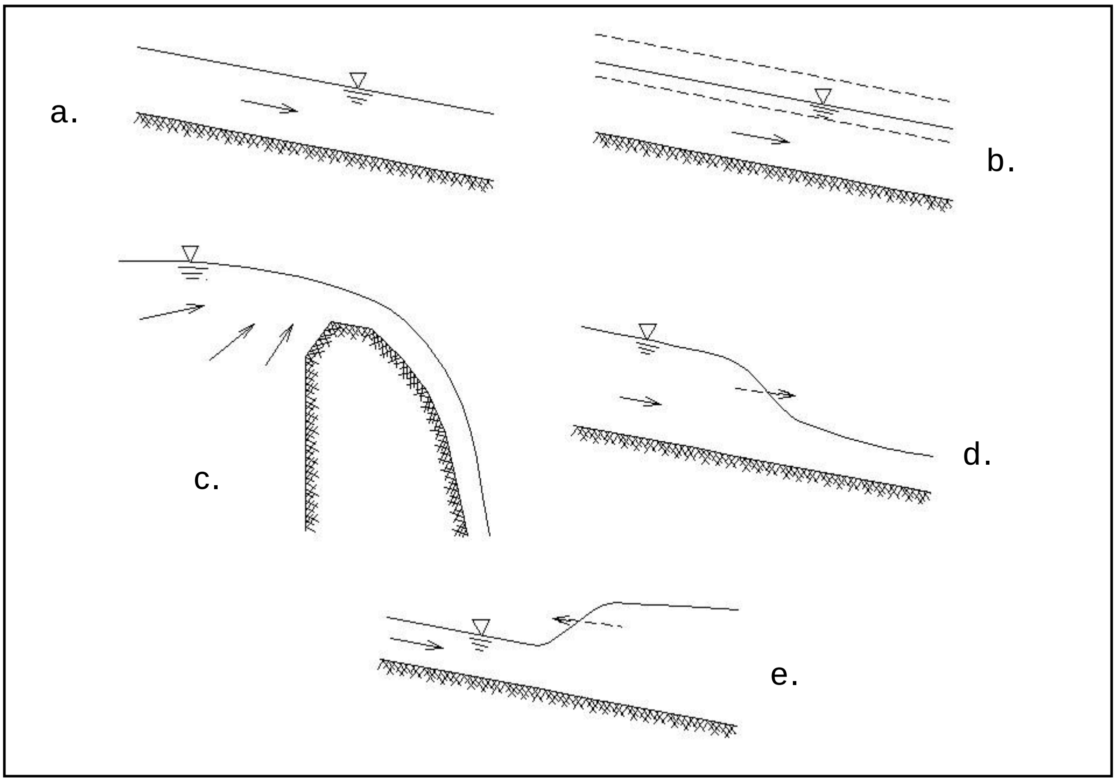


Figure 2.3: Classifications of types of open canal flow

Further classification of varied flow includes rapidly or gradually varied flow. In summary, the classification fields are as follows (Chow, 1959):

A. Steady flow
    1. Uniform flow
    2. Varied flow
        a. Gradually varied flow
        b. Rapidly varied flow

B. Unsteady flow

1. Unsteady uniform flow

2. Unsteady flow

    a. Gradually varied unsteady flow

    b. Rapidly varied unsteady flow

Another factor that also influences the flow rate is viscosity. Viscosity is a measure of the internal friction of a fluid, or its resistance to flow. Figure 2.4 shows a space between two plates which, for demonstration purposes, could be assumed to be filled with a fluid. If the upper plate is in motion at velocity *V*, the particles of fluid that are in contact with that surface will move at the same velocity as the moving surface. The velocity will decrease to zero at the stationary plate. It has been shown in laboratory tests that for a particular fluid at a given temperature the ratio of the shear stress, τ to the rate of deformation (*dv/dy*) is constant. Thus it is called coefficient of viscosity and could be defined in terms of dynamic viscosity, absolute viscosity, or simple viscosity (3) and can be calculated as:

$$\boxed{\mu = \tau / (dv/dy)} \qquad \text{...(3)}$$

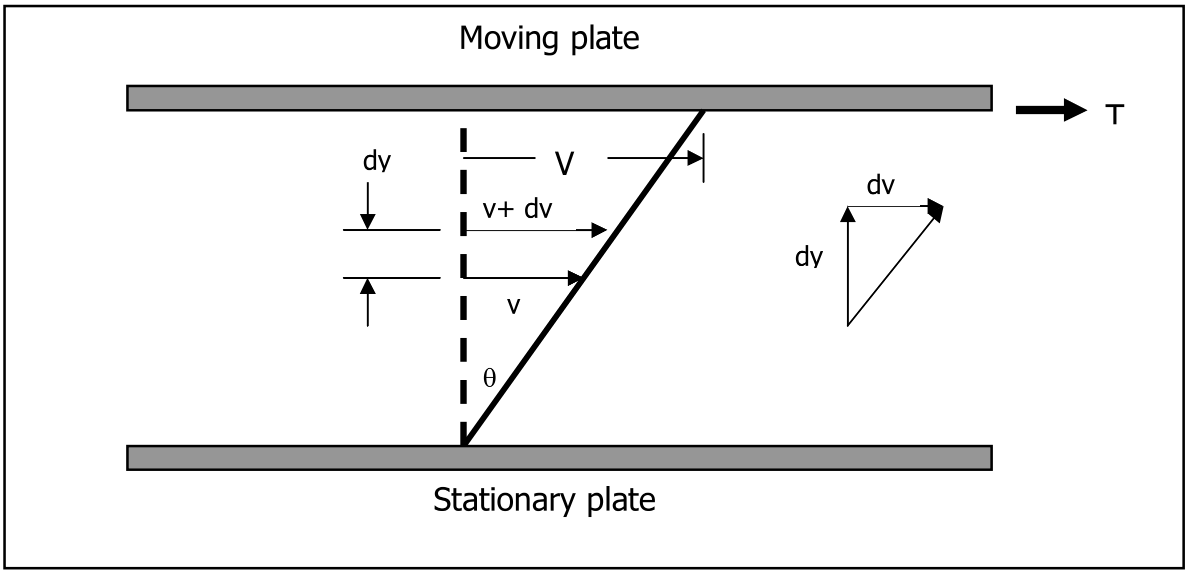


Figure 2.4: Velocity distribution in Newtonian fluids

The velocity of the fluid in a particular cross-sectional area is therefore not constant and is sometimes zero (Figure 2.4) where it is in contact with the canal surface (Henderson, 1966). Figure 2.5 shows how the velocity of a typical fluid in motion can be zero where it is in contact with a conduit surface (Meyer, 1995). Viscosity of the fluid makes it difficult to perform hydraulic analysis because in the variations in flow dynamics. The engineer is therefore forced to compute and use the mean (average) velocity of the section under consideration.

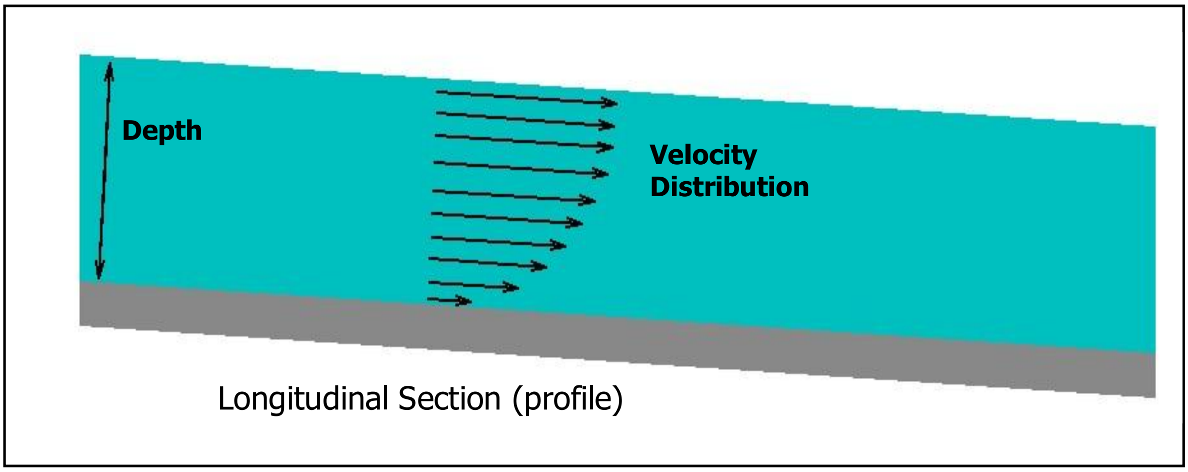


Figure 2.5: Typical velocity distribution

The effect of the viscosity relative to the inertia results in flow which could be laminar, turbulent or transitional (Figure 2.6). Laminar flow is characterised by fluid particles appearing to move in smooth, predictable streamlines or thin layers sliding over adjacent layers. In turbulent flow the viscous forces are weak relative to the inertial forces, resulting in erratic, unpredictable flow lines. The general flow movement is still in the forward direction. It is furthermore characterised by the formation of eddies within the flow. Eddies continuously cause mixing of the fluid within the section. To classify whether flow is laminar or turbulent, there is an index called the Reynolds number (Chow, 1988), as indicated below:

$$\boxed{\mathrm{Re} = \frac{4VR}{\nu}} \qquad \text{...(4)}$$

where:

Re = Reynolds number (unitless)

V = mean velocity (m/s)

R = hydraulic radius (m)

ν = kinematic viscosity (m^2/s)

For laminar flow the Reynolds number (4) is below 2000, while it is above 4000 for turbulent flow. Between 2000 and 4000 the flow will be transitional.

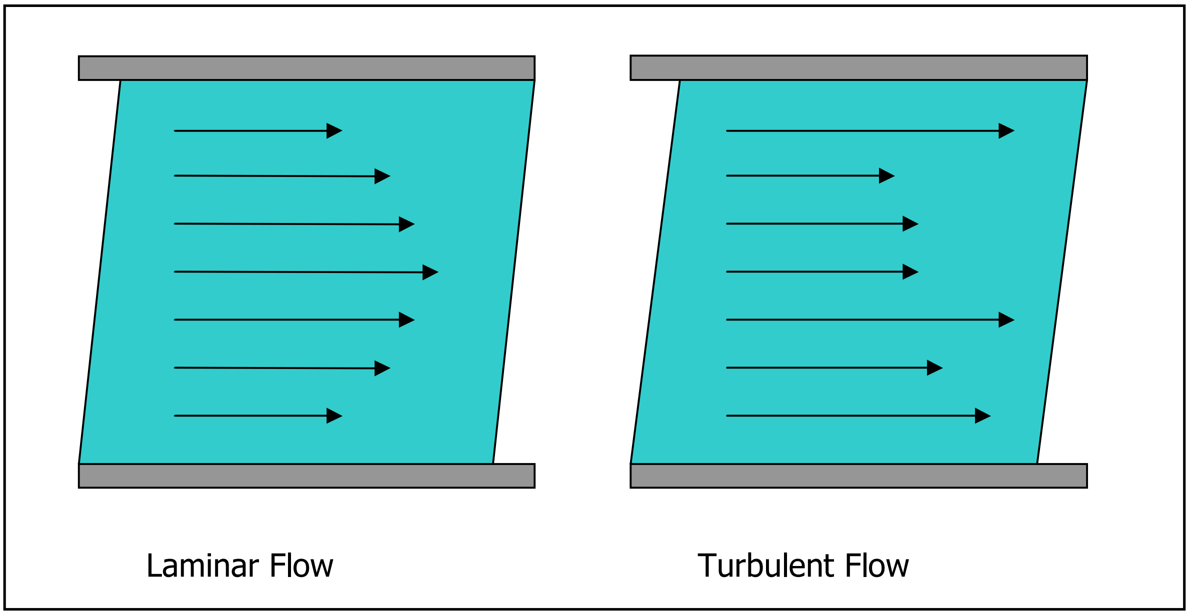


Figure 2.6: Instantaneous velocity distribution for laminar and turbulent flow

2.3.2 Energy

According to the Energy Principle (Figure 2.7) the first law of thermodynamics states that for a given system, the difference between the heat transferred to the system (*Q*) and the work done by the system in its surroundings (*W*) during a given time interval, will be the change in energy (ΔE). The energy mentioned is the total sum of energy in the system, i.e. potential energy, kinetic energy, and internal (molecular) energy such as electrical and chemical energy. By converting energy values into units of energy per unit weight, units of length will be derived and could be used more appropriately and with better understanding in hydraulic systems. The energy in a hydraulic system is thus expressed in terms of "head" and has three parts:

- pressure head ρ/γ
- elevation head Z
- velocity head $V^2/2g$

where:

p = pressure (N/m^2)

γ = specific weight (N/m^3)

Z = elevation (m)

V = velocity (m/s)

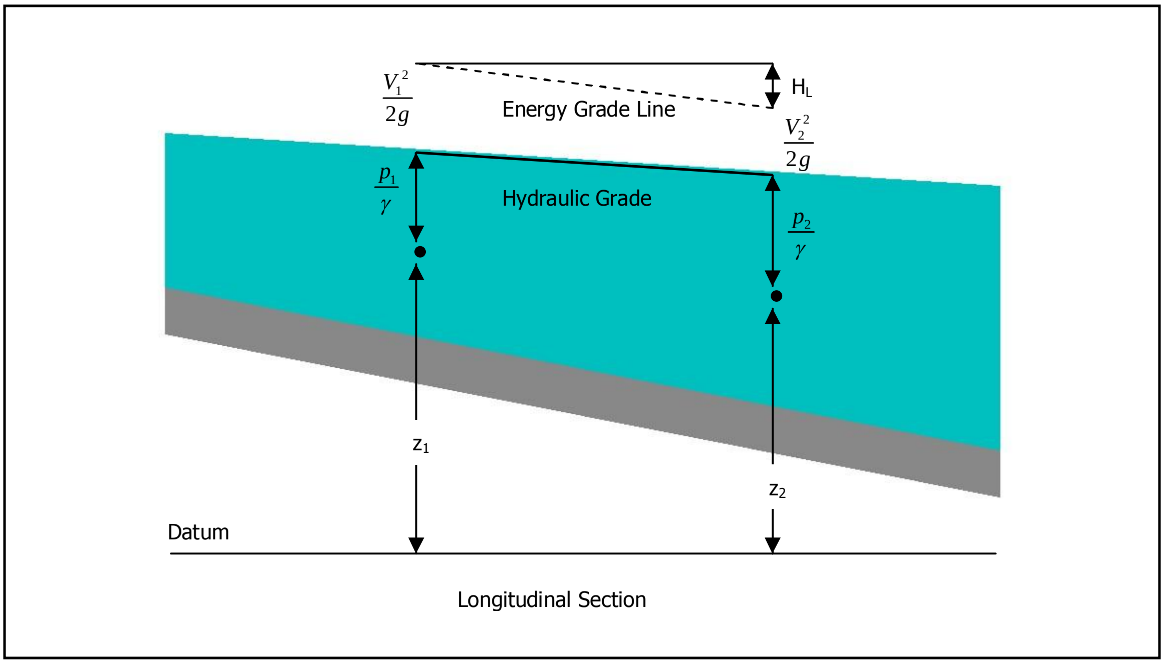


Figure 2.7: The Energy Principle

Pressure-, elevation-, and velocity head are ways to add energy to a hydraulic system. Energy is removed from the system through friction and other disturbances. The primary reason for energy losses (H_L) in a system is usually internal friction between fluid particles. Secondary means of losses could be due to changes in the shape of the canal or obstacles in the canal. Losses in a canal can be expressed in terms of length per length, in other words, losing 0,5 m of head for every 100 m lost in length. All these removals and additions of energy may also be referred to as head losses and head gains. To keep the energy in equilibrium the energy in the system can be balanced across any two points in the system, thus forming the energy equation:

$$\boxed{\frac{p_1}{\gamma} + z_1 + \frac{V_1^2}{2g} + H_G = \frac{p_2}{\gamma} + z_2 + \frac{V_2^2}{2g} + H_L} \qquad \text{.....................(5)}$$

where:

p = pressure (N/m^2)

γ = specific weight (N/m^3)

z = elevation at the cancroids (m)

V = fluid velocity (m/s)

g = gravitational acceleration (m/s^2)

H_G = head gain, such as from a pump (m)

H_L = combined head loss (m)

The hydraulic grade and energy grade lines are shown in Figure 2.7. Since the canal is open to atmospheric pressure, the hydraulic grade is the water surface elevation and is the sum of the pressure head (p/γ) and the elevation head, *z.* This is the case for a typical canal section but could change if a hydraulic jump is present. The line at the top of the water surface in Figure 2.7 may be referred to as the hydraulic grade line (HGL). The energy grade in turn is the sum of the hydraulic grade and the velocity head ($V^2/2g$) and is referred to as the energy grade line (EGL) as shown in Figure 2.7. The EGL will be equal to the HGL where the flow velocity is zero, such as in a dam or a reservoir.

2.3.3 Orifices, weirs and canal profiles

Since the energy equation can be used as the basis for the calculation of flow over hydraulic structures, V_2 can be calculated by taking the difference in energy on both sides of the structure, size of the opening of the structure and substituting these values into equation (5). By adding a coefficient for different hydraulic and physical factors, the flow rate can be calculated.

An orifice (Figure 2.8) is a measuring structure which is situated at a certain distance below the surface of the water and is usually rectangular or circular in shape. The flow rate is calculated by taking the energy difference between the upstream and downstream sides of the opening. The water exiting the orifice is called the jet. Due to the upstream pressure the jet tends to contract to a point from where the flow lines remain relatively constant and parallel. This shape is called the Vena Contracta.

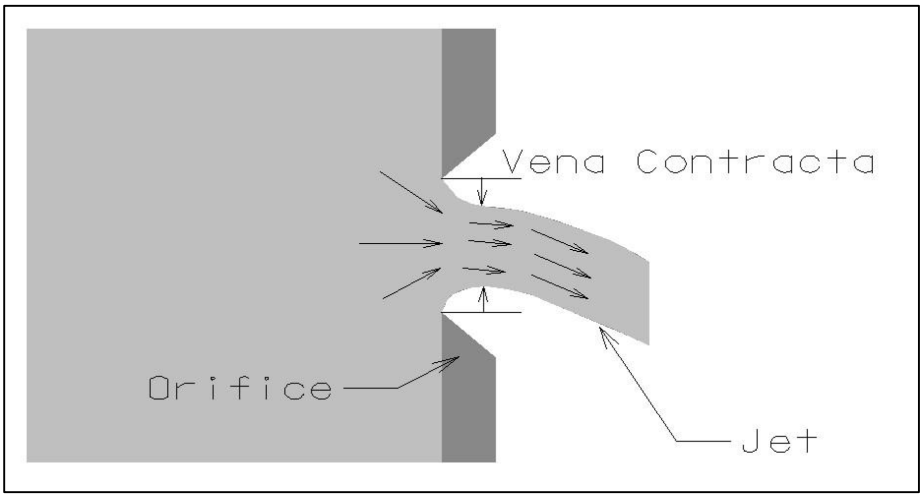


Figure 2.8: The Vena Contracta type Orifice

Orifices can be used to:

- regulate the flow out of detention ponds through the controlled outflow;
- regulate the flow through canals;
- approximate the flow allowed through a submerged culvert operating under inlet control; and
- measure flow.

Weirs on the other hand, are notches and gaps over which fluids can flow. As shown in Figure 2.9 the bottom of a weir is called the crest and surface of water overflowing the top of the weir is called the nappe. Again it is possible, as with orifices, for the water to contract as it overflows the different types of weirs. The stream of water over flowing the structure is also called the Vena Contracta.

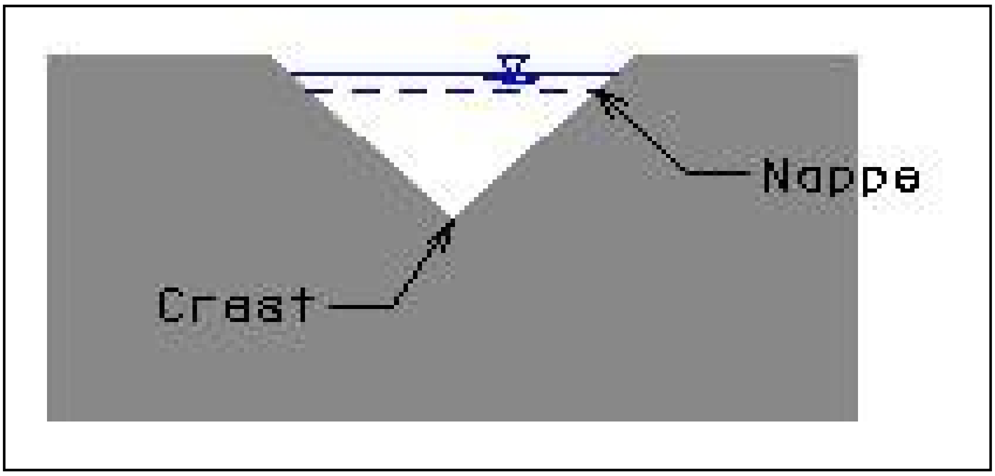
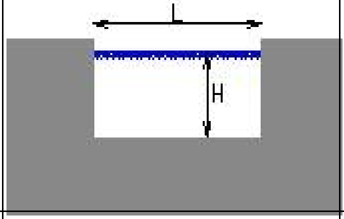
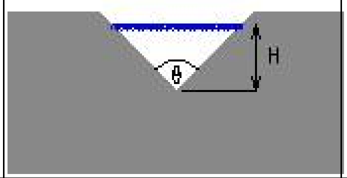
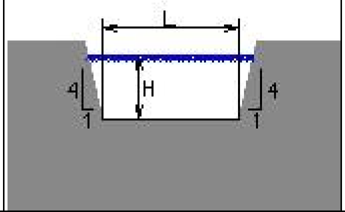
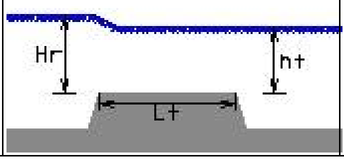


Figure 2.9: V-notch weir

Table 2.2 shows the most general types of weirs. All the different shapes and geometric differences are accounted for in both the equation and coefficient. Depending on the size and use of the canal the user can opt for any one or a combination of any of these weirs in the network. In a long weir water overflows the weir parallel to the side of the canal and not normal to the flow direction. This allows for more water to overflow. The function of the long weir is not only to measure flow but also primarily, to keep the pressure constant on pressure controlled sluices throughout the canal system. This is a common practice in South Africa. All weirs have one or more of the following functions:

- serve as emergency spillways for regulating high return event flows overtopping dams and retention ponds;
- measure flow;
- determine the flow over roadways which act as broad-crested weirs when a flow exceeds a culvert's capacity;
- determine the interception capacity of un-submerged drainage inlets and swales;
- determine the flow allowed through an un-submerged culvert operating under inlet control; and
- determine flow through un-submerged orifices.

Table 2.2: Weir types (Haested publication, 2003)

	Weir type	Figure	Equation	Coefficients
Sharp crested	Rectangular		$Q = CLH^{3/2}$	$C = 3{,}33$
	V-Notch		$Q = C\left(\frac{8}{15}\right)\sqrt{2g}\cdot\tan\left(\frac{\theta}{2}\right)H^{\frac{5}{2}}$	C varies between 0,611 and 0,570 depending on H and θ.
	Trapezoidal		$Q = CLH^{3/2}$	$C = 3{,}33$
Not Sharp Crested	Broad-Crested (side view)		$Q = C_d L H_r^{3/2}$	C_d is a function of H_r, h_t and L_t ranging between 1,25 and 3,1

2.3.4 Friction losses

To approximate friction losses of a moving liquid over a given area or section, equations have been derived from certain hydraulic principles mentioned previously. For any situation and/or problem one of these commonly used equations can be applied. These are:

- Darcy-Weisbach equation;
- Chezy equation;
- Hazen-Williams equation; and
- Manning equation.

All of the above mentioned equations can be described by the generalised uniform flow equation:

$$\boxed{V = kCR^{x}S^{y}} \qquad \text{..}(6)$$

where:

V = mean velocity

k = factor to account for empirical constants, unit conversions, etc.

C = flow resistance factor

R = hydraulic radius

S = friction slope

x,y = exponents

Fluids are conveyed in a canal or by means of a conveyance structure. Depending on the engineering method, various materials could be used to construct the lining of the proposed canal. This will give rise to varying friction losses, depending on the material used. Table 2.3 provides a guideline for the use of friction coefficients for various lining materials. However, the ultimate value of the C component may be a function of the canal shape, depth and fluid velocity, all of which are subject to at the discretion of the designing engineer.

Table 2.3: Friction coefficients (Haested publication, 2003)

Material	Manning's Coefficient *n*	Hazen- Williams *C*	Darcy-Weisbach Roughness Height *k* (mm)
Asbestos cement	0.011	140	0.0015
Brass	0.011	135	0.0015
Brick	0.015	100	0.6
Cast-iron, new.	0.012	130	0.26
Concrete:			
Steel forms	0.011	140	0.18
Wooden forms	0.015	120	0.6
Centrifugal spun	0.013	135	0.36
Copper	0.011	135	0.0015
Corrugated metal	0.022	---	45
Galvanised iron	0.016	120	0.15
Glass	0.011	140	0.0015
Lead	0.011	135	0.0015
Plastic	0.009	150	0.0015
Steel:			
Coal-tar enamel	0.01	148	0.0048
New unlined	0.011	145	0.045
Riveted	0.019	110	0.9
Wood stave	0.012	120	0.18

2.3.4.1 *Darcy-Weisbach (Colebrook-White) equation*

The Darcy-Weisbach equation (7) can be applied to both pressure pipe- and open canal systems. A disadvantage in the use of this equation is that desktop calculations are rather complex (Hamill, 1995). With modern-day developments and the use of computers, these calculations can be eased. As the velocity and hydraulic radius varies with different flows, so will the Reynolds number change likewise. Both the Reynolds number and the material used for the construction of the canal have an impact on the roughness component in the Darcy-Weisbach equation. Darcy-Weisbach uses the Colebrook-White *f* friction factor in the calculation. Darcy-Weisbach was developed for uniform flow and is calculated as follows:

$$\boxed{V = \sqrt{\frac{8g}{f} RS}} \qquad \text{...(7)}$$

where:

V = mean velocity (m/s

g = gravitational acceleration (m/s^2)

f = Darcy-Weisbach friction factor (unitless)

R = hydraulic radius (m)

S = friction slope (m/m)

2.3.4.2 The Chezy equation

The equation of Chezy and Kutter (8) can be applied to sanitary sewer design and analysis as well as open canal flow (Chow, 1988). Factors like hydraulic radius, friction slope and lining material are all important in the selection of the roughness factor *C*. The roughness factor for Chezy can be calculated by the equations below or may be read off from a table. The Manning *n* values are used in these equations and not the values from Kutter's coefficient. Chezy's equation is calculated as follows:

$$\boxed{V = C\sqrt{RS}} \qquad \text{...(8)}$$

where:

V = mean velocity (m/s)

C = roughness coefficients

R = hydraulic radius (m)

S = slope of energy line

The roughness coefficient C is calculated by the Kutter friction equations:

English Units (Kutter)

$$\boxed{C = \frac{\left(23 + \frac{0.00155}{S} + \frac{1}{n}\right)}{\left[1 + \frac{\left(23 + \frac{0.00155}{S}\right)n}{\sqrt{R}}\right]}}$$

SI units (Kutter)

$$\boxed{C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + \frac{\left(41.65 + \frac{0.00281}{S}\right)n}{\sqrt{R}}}} \qquad \text{.........(9)}$$

where:

C = roughness coefficients

S = friction slope (m/m)

n = Manning's roughness value

R = hydraulic radius (m)

2.3.4.3 The Hazen-Williams equation

The Hazen-Williams equation is calculated by:

$$\boxed{V = kCR^{0.63}S^{0.54}} \quad \text{...(10)}$$

where:

V = mean velocity (m/s)

k = constant of 0.85

C = Hazen-Williams roughness coefficient

R = hydraulic radius (m)

S = friction slope (m/m)

For the design and analysis of pressure pipe systems the Hazen-Williams equation is the most commonly used. It is only intended for water and not for other fluids (Hauser, 1996).

2.3.4.4 The Manning equation

The most commonly used equation for open canal flow, also the equation used as flow equation for this study, is the Manning equation. This equation is intended for water and not for other fluids in motion. The roughness component in an entire canal is assumed to be constant over the full length of the canal while the Manning roughness value *n*, could vary from reach to reach. This value is experimentally determined for various types of lining materials. The equation is calculated by:

$$\boxed{V = \frac{k}{n} \cdot R^{\frac{2}{3}} \cdot \sqrt{S}} \qquad \text{...(11)}$$

where:

V = mean velocity (m/s)

k = 1.49 for English units of 1.00 for SI units

n = Manning's roughness

R = hydraulic radius (m)

S = slope of energy line

Manning is the equation used in the WAS program: the normal flow depth can be calculated from the equation. Factors like velocity, cross sectional flow area and wetted perimeter must be known and can be calculated mathematically. The majority of all the calculations are based on the calculated flow depth. With the flow depth known, calculations regarding seepage and evaporation can be done. The WAS program uses the wetted perimeter of a specific cross section and calculates the seepage loss in m^3 for a given seepage factor in ℓ/s per 1000 m^2. For evaporation losses the WAS program takes the evaporation rate (mm/day) multiplied by the reach length multiplied by the water surface width and expresses the loss in m^3. The water surface width is derived from the cross-sectional properties and the calculated normal flow depth. Transpiration losses are ignored in the calculations as this is a concrete lined canal with no riparian width.

As the Manning equation is used by the WAS program (WAS as designed by Dr. N. Benade), it is important to understand the working of it. Additional flow equations were also studied in order to have a full understanding of the working of other calculation methods. With the proper knowledge and understanding of the Manning equation, the implementation of the water release module can be executed effectively.

2.4 IRRIGATION MANAGEMENT

Irrigation schemes supply water to the users on a specific time, for a predefined duration, at a fixed flow rate to a given abstraction point. This study will emphasize the water request system in order to have a full understanding of the water distribution of the scheme and therefore to supply irrigation water effectively. The water request system can be operated in two different methods:

- A manual request system; and
- A computerised request system.

The optimal supply of irrigation water on request by means of canal systems can be achieved through the computerisation of the hand water distribution system (Benadé, 1993). With the limited water resources available irrigation schemes have no other choice than to fully optimise the available resources. The computerised distribution system makes use of a computer program to substitute the hand means of calculation and in turn optimise the distribution of water resources.

Although the manual (hand) calculation system is a tested system that has been in use for a long time it has its shortcomings which constitute a lot to water losses. The most important shortcomings are:

- A lot of personnel are involved in the calculation process and therefore errors can easily occur,
- Losses are an unknown value and are approximated most of the time. A computer system can approximate losses more accurate,
- Lag time under various flow and weather conditions can not be simulated and therefore can not be very accurate,
- The system could be slow and ineffective if re-calculation of volumes have to be executed,
- If personnel are re- allocated the expertise in knowledge are lost and personnel have to be re- trained,

- Reports are insufficient.

Computerised systems attend to these shortcomings and improve the optimisation in the following ways:

- Depending on the size of the scheme, at least two personnel members are required to enter and check new water requests,
- Calculation errors are eliminated as the computer will perform all calculations,
- Losses are measured and calculated according to assumptions,
- Lag times are calculated,
- If the request change, a new release volume calculation can be done quickly and accurately,
- The system is not affected if personnel are transferred,
- Reports are generated by computer and are therefore immediately available and in the proper format,
- Productivity is increased, and
- Water is conserved.

It is therefore necessary for any irrigation scheme to apply a required methodology for overall management on the scheme. Which ever distribution system is chosen must have the aim to conserve water. For the VHWUA the water request system of applying for water must be optimised in the most effective way in order to ease and improve irrigation scheme management.

An optimised computerised system will enable the manager responsible for the scheme and the personnel responsible for the water distribution to minimise operation losses in order to enhance the overall productivity of the irrigation scheme (Benadé, Engelbrecht, Annandale, 1990).

2.5 EXPLANATION OF CALCULATION PROCEDURES

In understanding the reasons for collecting and verifying data, it is important to understand the full calculation procedure that the WAS program and Excel calculation procedure follow to calculate the water release volume (Benade, Annandale and van Zijl, 1997).

2.5.1 The Hand (manual/Excel) water distribution system

Currently the conventional method for requesting water is used at the VHWUA (manual- hand request system). The VHWUA has already changed this system to a computer format, but it is still the hand method of calculation. Microsoft Excel spread sheets are used to execute the calculations: The following steps are followed to derive the release volume:

2.5.1.1 (Step 1): Application for water

As water can not be abstracted from the canal at random, water must be applied for. Each user at the VHWUA has a water quota for one water year as mentioned in previously. In order to request for water the application document (Figure 2.10) must be handed in. Here the volume of water, the date, time, abstraction point and duration is mentioned. In the event of something unforeseen happening on the farm, users should also be able to request additional water or to cancel already requested water. All transactions are reflected in the water quota of each user. This is done by means of the documents indicated in figures 2.11 and 2.12. In these forms the volume, date, time and sluice number are also mentioned.

The appropriate forms (figures 2.10 – 2.12) are placed in specific post boxes before 14:00 every Thursday. Water control officers collect the sheets and they are handed in at the irrigation office. The irrigation office is now able to compile a distribution chart, summary of requests and canal guard instruction sheets. All calculations are done on a weekly basis.

SIEN KEERSY VIR INSTRUKSIES/SEE REVERSE SIDE FOR INSTRUCTIONS

DEPARTEMENT VAN WATERWESE EN BOSBOU
DEPARTMENT OF WATER AFFAIRS AND FORESTRY

AANSOEK OM WATER • APPLICATION FOR WATER

Voerkanaal/Feeder canal .... Periode/Period..

Sluis No. Sluice No.	Vloeitempo Rate of flow	Ure Hours	Son. Sun.		Maan. Mon.		Dins. Tues.		Woens. Wed.		Don. Thurs.		Vry. Fri.		Sat. Sat.		Son. Sun.		Handtekening Signature
			D	N	D	N	D	N	D	N	D	N	D	N	D	N	D	N	

Ek/Ons doen hiermee aansoek vir water soos hierbo uiteengesit en onderneem om die Departementele toekening van die leibeurt te aanvaar.
I/We hereby apply for water as set out above and agree to accept the Departmental allocation of leading turns.

Besproeier/Segsman—Irrigator/Spokesman ....

Figure 2.10: Application for water

DEPARTEMENT VAN WATERWESE EN BOSBOU
DEPARTMENT OF WATER AFFAIRS AND FORESTRY

SPESIALE AANSOEK OM WATER • SPECIAL APPLICATION FOR WATER

Ek, die ondergetekende, doen hiermee aansoek om water soos aangedui; redes:
I, the undersigned, hereby apply for water as set out below; reasons:

....
....
....

	Sluis No. Sluice No.	Vloeitempo Rate of flow	Ure Hours	M		D/T		W		D/T		V/F		S		S	
				D	N	D	N	D	N	D	N	D	N	D	N	D	N
Aanvraag Application																	
Goedgekeur Approved																	

Besproeier Irrigator.................... Datum Date.................... Toevoer voor Feeder.................... Periode Period....................

Aansoek is op ontvang en word toegestaan/nie toegestaan soos aangedui.
The application was received on.................... and may/may not be allocated as indicated.

Handtekening Signature.................... Rang Rank.................... Datum Date....................

Ek aanvaar hiermee die bostaande toekenning van water deur die Departement.
I hereby accept the above-mentioned allocation of water by the Department.

Besproeier Irrigator.................... Datum Date....................

Opmerkings/Remarks ....
....

Figure 2.11: Special application for water

Pretoria Printers DW 246C

DEPARTEMENT VAN WATERWESE EN BOSBOU
DEPARTMENT OF WATER AFFAIRS AND FORESTRY
KANSELLASIE VAN WATER • CANCELLATION OF WATER

Alle aansoeke om water te kanselleer moet by die Wykswaterbeheerbeampte gedoen word.	All applications for the cancellation of water must be made to the Section Water Control Officer.
Hierdie vorm moet volledig ingevul en onderteken word voordate die kansellasie plaasvind.	This form must be fully completed and signed before cancellation is effected.
HIERDIE VORM SAL NIE AANVAAR WORD AS DIT NIE VOLLEDIG INGEVUL EN ONDERTEKEN IS NIE.	THIS FORM WILL NOT BE ACCEPTED UNLESS COMPLETED PROPERLY.

Voerkanaal
Feeder Canal .... Periode No.
Period No. ....

Sluis No. Sluice No.	Vloeitempo Rate of flow	Ure Hours	So. Su.		Ma. Mo.		Di. Tu.		Wo. We.		Do. Th.		Vr. Fr.		Sa. Sa.		So. Su.		Handtekening • Signature Besproeier • Irrigator
			D	N	D	N	D	N	D	N	D	N	D	N	D	N	D	N	

Bogemelde aansoek is op .... ontvang en kan soos aangedui toegestaan word.

The above-mentioned application was received on............ .... and may be granted as shown.

Handtekening
Signature............ Rang
Rank .... Datum
Date ....

Ek/Ons bevestig en aanvaar hiermee die bostaande aansoek om kansellasie van besproeiingswater.

I/We hereby confirm and accept the above-mentioned application for the cancellation of irrigation water.

Besproeier • Segsman............ Datum • Date ....

Figure 2.12: Application for cancellation of water

2.5.1.2 (Step 2): Lag time of water

The lag time of water is the time water will take to move from one point in the canal to the next point in the canal. If a sluice has to be opened at a specific time, the lag time for the water to flow from the sluice to the abstraction point must be known so that the water can be released from the source at the correct time. The efficiency of the distribution system depends on the accurate determination and application of the time the water will take to flow from one point to the next in various conditions (Benadé, 1993).

2.5.1.3 (Step 3): Compilation of feeder charts

At this stage of the calculation process the gross water demand for each feeder canal for the given week will be known (Figure 2.13). The yellow columns show day requests and the white shows night requests. The summary of the canal demands, as given by Figure 2.15 can be calculated once all the water requests are consolidated. It indicates the period of use, day or night flow and volumes per canal. The daily feeder chart can be compiled from the gross demand summary. The feeder chart (Figure 2.14) shows the day of the water week, volume to be released for each community canal and estimation of losses. Losses are based on a percentage of the flow volume in the canal. The feeder charts (Figure 2.14) are used for the planning of water distribution by graphically indicating the distribution of water used in the given period (Benadé, 1994).

2004.01.26 Periode: 44.04

	MAANDAG		DINSDAG		WOENSDAG		DONDERDAG		VRYDAG		SATERDAG		SONDAG	
Tw 1	3744	3110	5232	4656	4790	2688	4848	4627	5232	4435	1690			
Tw 2	576	384	1536	1536	4186	1920	2688	1728	2688	1152	0			
Tw 4	1516	1309	1848	1848	2195	2079	3108	2688	3242	2688	1309	455		
Tw 5	1742	1363	2625	2625	2500	2731	3008	2905	3237	2554	2295	782		
Tw 5F	894	894	1044	1044	1459	1459	1044	1044	1315	1315	527	0		
Tw 6	1341	927	3244	3244	3475	2929	3891	3475	4201	3475	988	455		
Tw 7	783	783	2747	2405	3880	3749	3749	3450	3553	2767	455	325		
Tw 8	2008	1835	5091	5087	5441	5087	5647	5541	5647	4918	1823	209		
Tw 9	3898	3485	4986	4394	5229	4986	4978	4574	5163	5163	3328	834		
Tw 10	3998	3694	5623	4734	6248	5883	6053	5479	6463	5455	3767	2404		
Tw 11	4585	4585	5994	5994	6348	6170	6348	5994	6348	5649	3034			
Jx	722	541	541	709	1070	902	721	889	902	541	245			
Tw 12	1812	1812	4637	4637	5689	5689	4013	3841	4455	3841	2132			
Tw 13	2415	2415	3824	3824	4629	4629	4629	4629	4629	4629	1207			
Tw 14	2554	2554	4164	3983	4949	4949	5170	4587	5552	3602	205			
Tw 15	1610	1610	3623	3422	4831	4596	4428	4428	4227	3422	0			
Tw 16	2611	2611	4476	4289	5039	4849	4849	4365	4103	2984	308			
Tw 17	450	450	900	900	750	750	900	900	900	900	450			
Taung	3800	3700	4000	3700	3800	4300	3800	3800	3600	3600	3000	3000		
Dam	1.0		2.8		2.8		2.5		2.5		2.4		1.0	
Spoel	1.000		0.080		0.120		0.120		0.120		0.150		1.000	
Taung	1.000		0.340		0.300		0.410		0.400		0.300		1.000	
K.B.	5271	5271	5271	5271	5271	5271	5271	5271	5271	5271	5271	5271	5271	5271
Proefplaas	300	150	300	150	300	150	300	150	300	150	300	150	300	150
Tw 24/25	5403	4684	6824	6566	7004	6475	6707	5796	7205	5068	2897	0		
Tw 26/27	6141	6141	2955	6300	7550	6953	7550	7300	7464	6955	5827	3699		
Groot bruto	11544	10824	14031	13079	14879	13711	14550	12337	15011	12337	9170	3911		
Dam	2.0		2.0		2.0		2.0		1.8		1.0		1.0	
Meetplaat	0.625	0.600	0.700	0.670	0.720	0.690	0.720	0.650	0.735	0.650	0.560	0.000	0.000	0.000

NETTO	58174	54307	85737	81531	96958	89477	93993	86702	96039	80848	41504	17796	5571	5421
VERLIESE	4931	5067	5105	5013	4978	5007	4936	4863	5143	4986	3264	2618	1400	1400
BRUTO	63105	59374	90842	86544	101936	94484	98929	91565	101182	85834	44768	20414	6971	6821

Figure 2.13: Gross water canal demands

DATUM : 26-Jan	PERIODE : 44/04	DAG : MAANDAG

KB	TAUNG	17	16	15	14	13	12	11	10	5F	9	8	7	JX	26-27	6	5	4	2	1	24-25	PP
5271	3800	450	2611	1610	2554	2415	1812	4585	3998	894	3898	2008	783	722	11544	1341	1742	1516	576	3744	0	300

VERLIESE : 7.814

TYD		TOTAAL	VERLIESE	BRUTO	VERSKIL
	KB+ VRYBURG	5271	401	5672	
13:00	TAUNG,17	9521	1000	10521	4849
15:00	16,15,14,13	18711	1000	19711	9190
19:00	12,11,10	29106	1500	30606	10895
21:00	9,8,7,5F,26/27,JX	48955	3823	52778	22172
23:00	6,5,4,2,1,24/25,PP	58174	4931	63105	10327

Dam:	1
Spoel:	1.000
Taung:	1.000

		Mp.		Mp.	
Wes Dam:	2	0.625		0.600	
M.J. Groot bruto:		11544		10824	
Wes Sonder Verlies		11544		10824	

KB	TAUNG	17	16	15	14	13	12	11	10	5F	9	8	7	JX	26-27	6	5	4	2	1	24-25	PP
5271	3700	450	2611	1610	2554	2415	1812	4585	3694	894	3485	1835	783	541	10824	927	1363	1309	384	3110	0	150

VERLIESE : 8.534

TYD		TOTAAL	VERLIESE	BRUTO	VERSKIL
	KB+ VRYBURG	5271	401	5672	
01:00	TAUNG,17	9421	1000	10421	4749
03:00	16,15,14,13	18611	1000	19611	9190
07:00	12,11,10	28702	1500	30202	10591
09:00	9,8,7,5F,26/27,JX	47064	3813	50877	20675
11:00	6,5,4,2,1,24/25,PP	54307	5067	59374	8497

Figure 2.14: Daily water requests

2.5.1.4 (Step 4): Summary of weekly requests

Figure 2.15 shows the calculation of the total gross water requests for each feeder canal for the specific period (eg. Period 44/2004). Information is taken from the previous pages to calculate the weekly gross release per feeder. With the daily gross requests known, a summary of all the requests as well as the total weekly request can be compiled. The calculated release volume also includes losses (figures 2.13 and 2.14), which are estimated in terms of a percentages. Losses to be accounted for include evaporation, seepage, spilling and filling losses.

2.5.1.5 (Step 5): Distribution orders

In completing the DWAF form (DW 607) it is necessary to specify when the required volume of water for the week should be released (Figure 2.16). The required volume of water is released if the reading on the measuring plate at each weir corresponds with the specified reading on form DW 607. Form DW 607 will also be used to give instruction to the canal guards on the various opening sequences of the sluices. Measuring plate readings indicated should correspond to a volume of water to be released at specific time.

DATUM	TYD	Q	DATUM	TYD	Q	DATUM	TYD	Q	DATUM	TYD	Q
2004/01/26	13:00		38012	15:00	118748	2004/01/27	15:00	173088	2004/01/28	15:00	188968
	19:00	63126		19:00	237496		19:00	170246		19:00	372252
	21:00	39422		21:00	119646		21:00	173590		21:00	188268
Sondag	23:00	61212		23:00	133944		23:00	180946		23:00	188374
38012	01:00	105556	2004/01/27	01:00	145870	2004/01/28	01:00	203872	2004/01/29	01:00	185718
Maandag	03:00	126210	Dinsdag	03:00	184526	Woensdag	03:00	203872	Donderdag	03:00	200700
	07:00	252420		07:00	369052		07:00	413428		07:00	395716
	09:00	126010		09:00	181084		09:00	204872		09:00	195724
	11:00	125402		11:00	179946		11:00	204872		11:00	195724
	13:00	125402		13:00	178168		13:00	202936		13:00	183130
		1024760			1848480			2131722			2294574

DATUM	TYD	Q	DATUM	TYD	Q	DATUM	TYD	Q
2004/01/29	15:00	183130	2004/01/30	16:00	257502	2004/01/31	15:00	40828
	19:00	360576		19:00	257502		19:00	81656
	21:00	182730		21:00	169568		21:00	34828
	23:00	183734		23:00	143734		23:00	34828
2004/01/30	01:00	187638	2004/01/31	01:00	131710	2004/02/01	01:00	30020
Vrydag	03:00	202364	Saterdag	03:00	108744	Sondag	03:00	21508
	07:00	404728		07:00	313376		07:00	27884
	09:00	202364		09:00	88636		09:00	13942
	11:00	194616		11:00	85196		11:00	13942
	13:00	189974		13:00	72138		13:00	13942
		2291854			1628106			313378

Week Totaal	11532874

Figure 2.15: Summary of weekly water request

Periode 44/04.

	Maandag				Dinsdag				Woensdag				Donderdag						
DAG		NAG		DAG		NAG		DAG		NAG		DAG		NAG					
5672	xxxxx	5672	xxxxx	5672	xxxxx	7093	xxxxx	5672	xxxxx	7093	xxxxx	5672	xxxxx	7093					
	4849	xxxxx	4749	xxxxx	5198	xxxxx	3477	xxxxx	5149	xxxxx	4228	xxxxx	5299	xxxxx	3878				
		9190	xxxxx	9190	xxxxx	16339	xxxxx	15770	xxxxx	19448	xxxxx	19023	xxxxx	19076	xxxxx	18009			
			10895	xxxxx	10591	xxxxx	16554	xxxxx	15665	xxxxx	18785	xxxxx	18242	xxxxx	16914	xxxxx	15814		
				22172	xxxxx	20675	xxxxx	30404	xxxxx	28682	xxxxx	34271	xxxxx	32303	xxxxx	33012	xxxxx	30148	
					10327	xxxxx	8497	xxxxx	16675	xxxxx	15857	xxxxx	18611	xxxxx	13595	xxxxx	18956	xxxxx	16623

	Donderdag				Vrydag				Saterdag				Sondag						
DAG		NAG		DAG		NAG		DAG		NAG		DAG		NAG					
5672	xxxxx	7093	xxxxx	5672	xxxxx	5672	xxxxx	5672	xxxxx	5672	xxxxx	5672		5672					
	5299	xxxxx	3878	xxxxx	5099	xxxxx	5099	xxxxx	4049	xxxxx	3599	xxxxx	599	xxxxx	599				
		19076	xxxxx	18009	xxxxx	18511	xxxxx	14637	xxxxx	1720	xxxxx	0	xxxxx	0	xxxxx	0			
			16914	xxxxx	15814	xxxxx	17768	xxxxx	15445	xxxxx	9433	xxxxx	2904	xxxxx	500	xxxxx	500		
				33012	xxxxx	30148	xxxxx	33914	xxxxx	29354	xxxxx	17871	xxxxx	6579	xxxxx	2323	xxxxx	2323	
					18956	xxxxx	16623	xxxxx	20220	xxxxx	15627	xxxxx	6023	xxxxx	1660		-2123	xxxxx	-2273

DATUM	TYD	Q	M P	DATUM	TYD	Q	M P	DATUM	TYD	Q	M P	DATUM	TYD	Q	M P
25-Jan	13:00	10521	0.360	26-Jan	15:00	59374	0.750	27-Jan	15:00	85123	0.955	28-Jan	15:00	93063	1.010
	19:00	19711	0.550		19:00	59823	0.750		19:00	86795	0.955		19:00	94134	1.010
	21:00	30606	0.690		21:00	66972	0.860		21:00	90473	1.060		21:00	94187	1.040
Sondag	23:00	52778	0.780		23:00	72935	0.985		23:00	93593	1.060		23:00	92859	1.040
26-Jan	01:00	63105	0.780	27-Jan	01:00	92263	0.985	28-Jan	01:00	101936	1.060	29-Jan	01:00	100350	1.040
Maand.	03:00	63105	0.780	Dinsd.	03:00	92263	0.985	Woensd.	03:00	103357	1.060	Donderd.	03:00	98929	1.040
	07:00	63005	0.780		07:00	90542	0.985		07:00	102436	1.060		07:00	97862	1.040
	09:00	62701	0.780		09:00	89973	0.985		09:00	102436	1.060		09:00	97862	1.040
	11:00	62701	0.780		11:00	89084	0.985		11:00	101468	1.060		11:00	96762	1.040
	13:00	59374	0.750		13:00	86544	0.955		13:00	94484	1.010		13:00	91565	0.390

DATUM	TYD	Q	M P	DATUM	TYD	Q	M P	DATUM	TYD	Q	M P
29-Jan	13:00	90144	0.990	30-Jan	15:00	85834	0.930	31-Jan	15:00	20414	0.375
	19:00	91365	0.990		19:00	84784	0.930		19:00	17414	0.330
	21:00	91867	1.055		21:00	71867	0.850		21:00	17414	0.330
	23:00	93819	1.055		23:00	65855	0.800		23:00	15010	3.000
30-Jan	01:00	101182	1.055	31-Jan	01:00	54372	0.720	1-Feb	01:00	10754	0.240
Vryd.	03:00	101182	1.055	Saterd.	03:00	44768	0.625	Sondag	03:00	6971	0.200
	07:00	101182	1.055		07:00	44318	0.625		07:00	6971	0.200
	09:00	97308	1.055		09:00	42598	0.625		09:00	6971	0.200
	11:00	94987	1.030		11:00	36069	0.550		11:00	6971	0.200
	13:00	85834	0.950		13:00	20414	0.375		13:00	6771	0.200

Figure 2.16: DWAF DW 607

2.5.2 The computerized (WAS) water distribution system

The purpose of the water request module of the WAS program is to make water available at a specific time, for a predefined period at an established flow rate at a certain point for the best benefit of the users of the VHWUA. With the Manning equation the normal flow depth in every reach can be calculated as the value of the discharge (Q) is known. The discharge (Q) will be derived from the water request module where all the users have entered their water requests. The reach length and flow depth are then used to calculate the wetted area and water surface area. The seepage loss and evaporation in each reach can be calculated from the wetted area and water surface area respectively. The critical point for the WAS is to accurately calculate the flow depth from the given Q value for each reach in the canal system. All the other required values are derived from the flow depth. By applying the calculation procedure as shown by Figure 2.20, all volumes will be accounted for chronologically as losses are calculated for the mentioned flow depth. The final water release volume can be derived by following the next steps:

2.5.2.1 (Step 1): Water applications

Water is requested by users by filling in the computer form (online) indicated in Figure 2.17. This from serves the same purpose as the form used in the hand calculation method, but are combined in a single page. By selecting the type of request water can be applied for, additionally requested or cancelled. The color at the bottom of the page indicates the type of request. Blue is an additional request, red indicates cancellation and white is for original requests. Request are therefore entered directly into the water request module of the WAS program. The computer will automatically calculate requests entered to the database. The quota balance is indicated and ensure that users do not exceed their given water quotas.

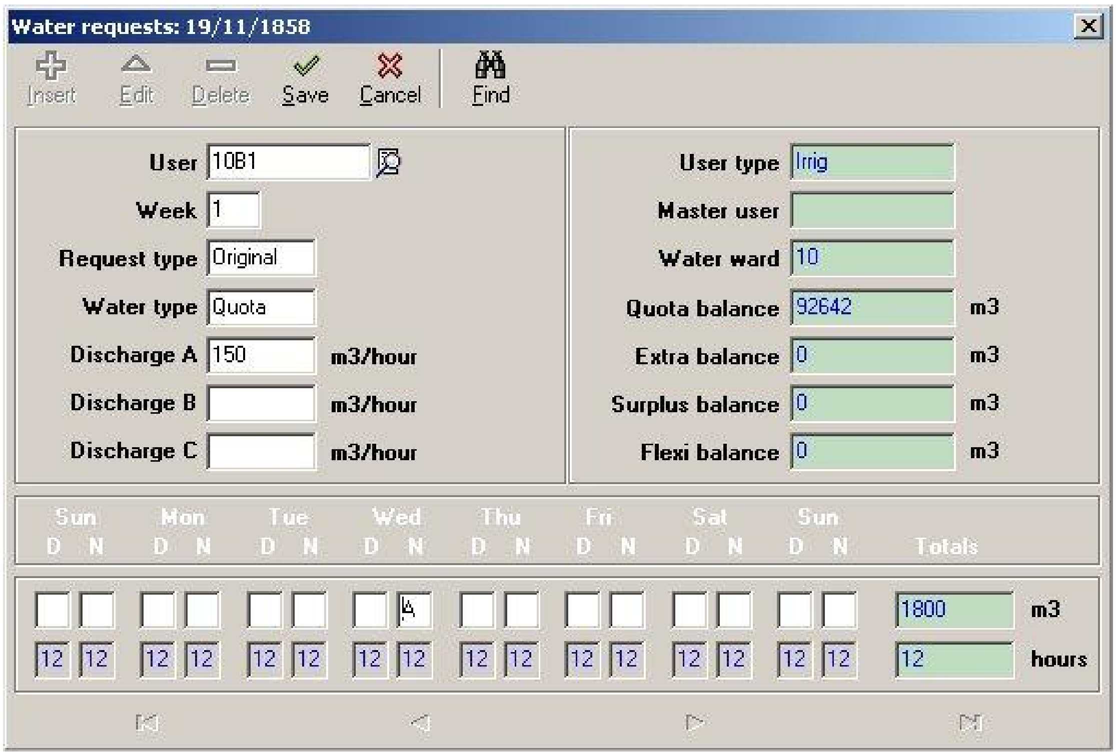
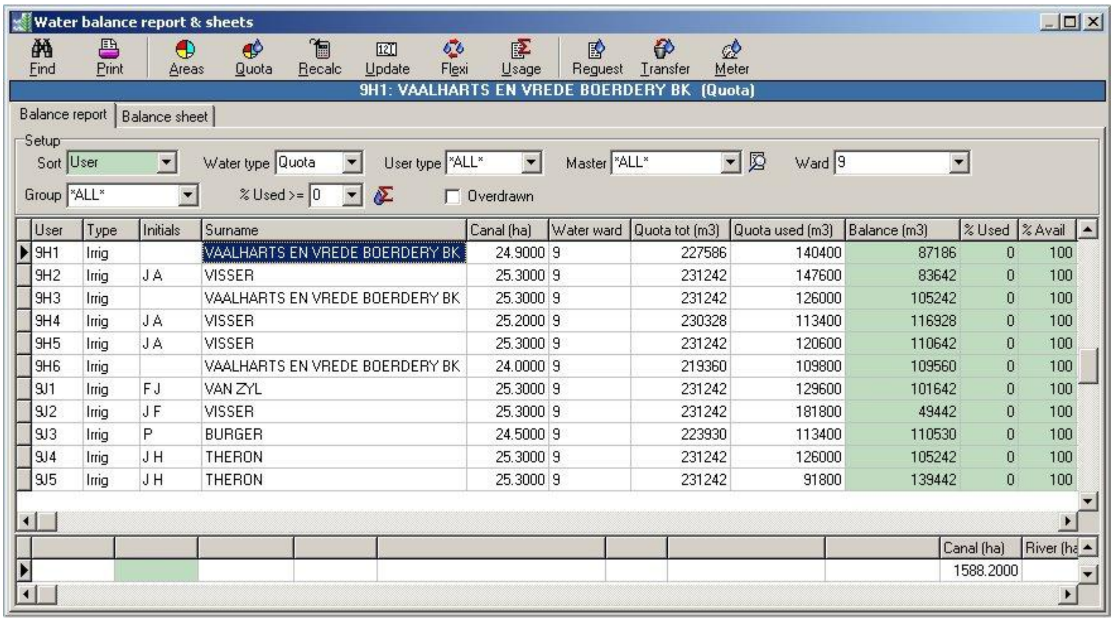


Figure 2.17: Water request sheet

2.5.2.2 (Step 2): Balance sheets

Once all the information as previously mentioned is entered, documentation such as balance sheets and reports is calculated automatically and is immediately available. Figure 2.18 indicates the request made by each user. This is the balance report sheet and resembles all water transactions for a given sluice number.

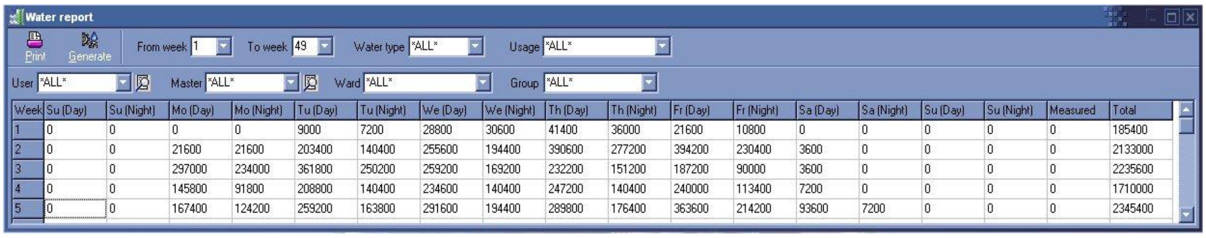


User	Type	Initials	Surname	Canal (ha)	Water ward	Quota tot (m3)	Quota used (m3)	Balance (m3)	% Used	% Avail
9H1	Irrig		VAALHARTS EN VREDE BOERDERY BK	24.9000	9	227586	140400	87186	0	100
9H2	Irrig	J A	VISSER	25.3000	9	231242	147600	83642	0	100
9H3	Irrig		VAALHARTS EN VREDE BOERDERY BK	25.3000	9	231242	126000	105242	0	100
9H4	Irrig	J A	VISSER	25.2000	9	230328	113400	116928	0	100
9H5	Irrig	J A	VISSER	25.3000	9	231242	120600	110642	0	100
9H6	Irrig		VAALHARTS EN VREDE BOERDERY BK	24.0000	9	219360	109800	109560	0	100
9J1	Irrig	F J	VAN ZYL	25.3000	9	231242	129600	101642	0	100
9J2	Irrig	J F	VISSER	25.3000	9	231242	181800	49442	0	100
9J3	Irrig	P	BURGER	24.5000	9	223930	113400	110530	0	100
9J4	Irrig	J H	THERON	25.3000	9	231242	126000	105242	0	100
9J5	Irrig	J H	THERON	25.3000	9	231242	91800	139442	0	100

Figure 2.18: Water balance report

2.5.2.3 (Step 3): Water report

The water report (Figure 2.19) displays a summary of all the water requests on a daily- and weekly basis. The total volume of water demands for the entire scheme will therefore be available immediately after entering all request, additional water and cancellations. At this stage it is only the requested volume of water and losses and lag times still need to be accounted for.



Week	Su (Day)	Su (Night)	Mo (Day)	Mo (Night)	Tu (Day)	Tu (Night)	We (Day)	We (Night)	Th (Day)	Th (Night)	Fr (Day)	Fr (Night)	Sa (Day)	Sa (Night)	Su (Day)	Su (Night)	Measured	Total
1	0	0	0	0	9000	7200	28800	30600	41400	36000	21600	10800	0	0	0	0	0	185400
2	0	0	21600	21600	203400	140400	255600	194400	390600	277200	394200	230400	3600	0	0	0	0	2133000
3	0	0	297000	234000	361800	250200	259200	169200	232200	151200	187200	90000	3600	0	0	0	0	2235600
4	0	0	145800	91800	208800	140400	234600	140400	247200	140400	240000	113400	7200	0	0	0	0	1710000
5	0	0	167400	124200	259200	163800	291600	194400	289800	176400	363600	214200	93600	7200	0	0	0	2345400

Figure 2.19: Water report

2.5.2.4 (Step 4): Water transfers, distribution sheets and water summary

Other reports and sheets that are automatically derived are immediately available and include the following:

- *Water transfers:* When the water quota of a given plot is exhausted or when a plot not listed requires water, water can be transferred from other plots to the mentioned plot. In this sheet all water transfers from one plot to another are indicated and consolidated.
- *Distribution sheets:* The distribution sheet for a water ward can be generated once all water requests are entered. The distribution sheets show all originally requested water, cancellations and additional requested water. The total volume as well as the day and night flow will also be indicated. Possible errors can easily be detected and rectified.

- *Water summary:* This report shows the summary of the available amount of water from the original water quota for a given irrigator.

2.5.2.5 (Step 5): Water release calculations

The process as indicated in Figure 2.20 is followed to determine the final release volume. The release calculation procedure used by the WAS starts at the source (Warrenton Weir) (Benadé, 2001a) and moves down to the end of the last abstraction points on the north and west canals, respectively, and last abstraction point on the community canal of each of these. The calculation will then move up to the inlet of the community canal by determining the release hydrograph.

The release hydrograph is calculated in order to make the volume of water and time of flow known. Water demands are taken from the water request module. If it encounters a branch, the result up to that point is temporarily stored. As the program would realise that this was not the source (Warrenton Weir), this procedure would continue until the entire network was accounted for. At each primary branch (canal or feeder) the computer temporarily stores the result and then adds the release hydrograph, as well as all the losses and the lag time from the previous canal or feeder.

To calculate the seepage and evaporation losses, a typical resistance equation for the design of flow in an open canal (Manning Equation) is used (Nalluri and Featherstone, 1995). The Manning equation is used to calculate the normal flow depth (y_n). From the y_n the seepage and evaporation can be calculated. A slide bar built into the program may also be used manually to change evaporation and seepage (Chapter 5, Figure 5.7) by the user. When the procedure reaches the Warrenton Weir, a release volume is available that will reflect the total release volume, including all losses.

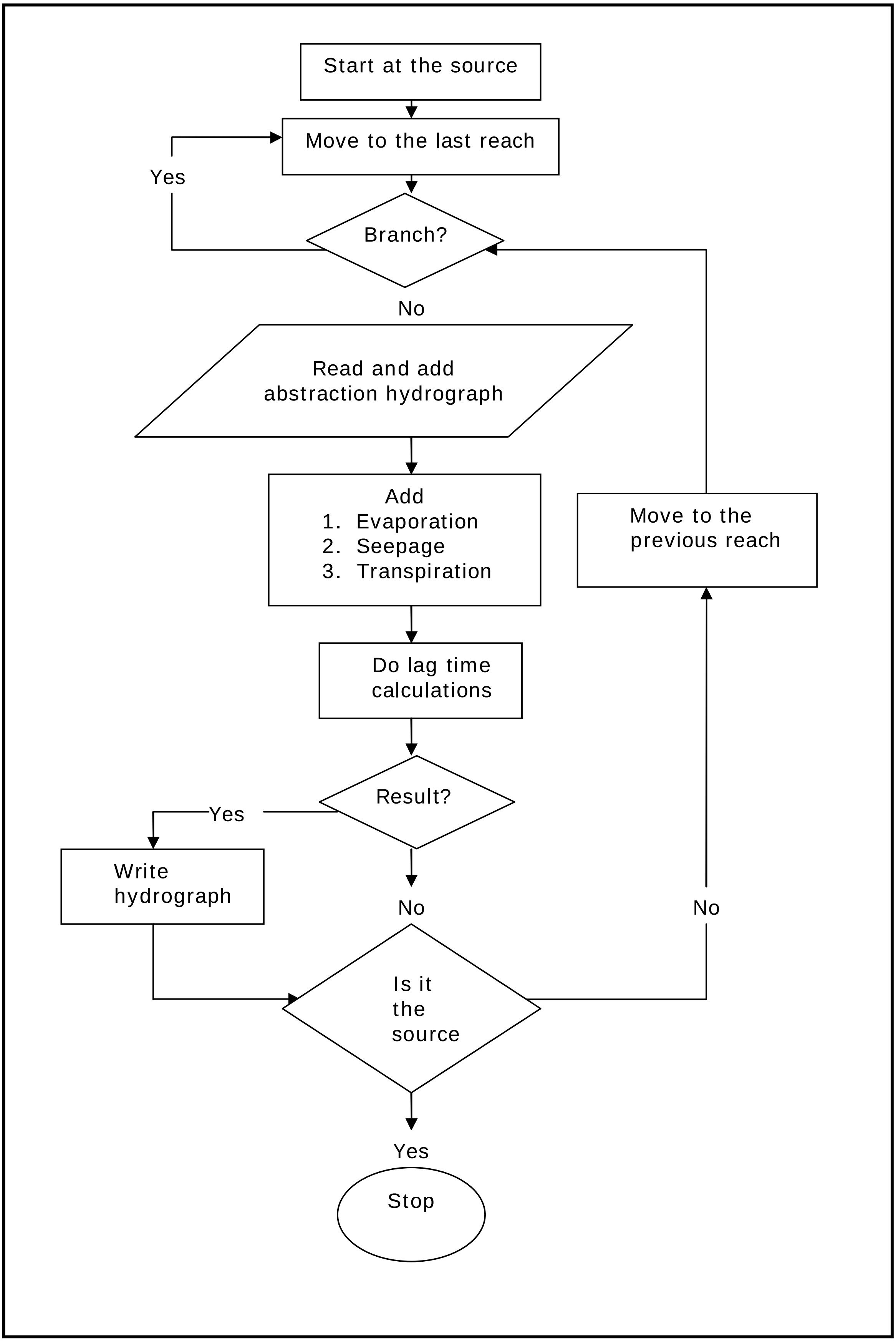


Figure 2.20: WAS calculation procedure (Benadé, 2001a).

The information/data needed by the WAS to calculate the water release is as follows:

- Canal ID: This is the identification string of the canal. The ID for the fourth feeder canal on the north canal is F4, while for the fourth community canal branching from this feeder canal, the ID is 4A (Figure 3.3).
- Turnout: This is to identify a point on the canal (e.g. branch or abstraction point).
- Turnout type: Specifies whether the turnout is a branching canal or an outflow. A branch is a secondary canal, while outflows are captured as water requests or meter readings.
- Reach: The distance between turnouts/abstraction points, slope changes or canal geometry shape changes on a canal.
- Chainage (m): This is accumulated reach distances.
- Q Cap: Refers to the maximum discharge capacity of the canal of each of the canals.
- Seep: Factor indicating seepage loss in l/s per 1000 m^2 wetted area. It is normally taken as a constant for a specific canal reach and needs to be calibrated with measured inflows and outflows.
- Time: Specifies the opening and closing times of the outflow for every reach. The user can set it to a default or it can be specified in the water request form.
- Cv: Coefficient to calibrate the lag time for every reach.

- Section: Specifies a certain type of canal profile (e.g. rectangle, parabola etc).
- Top b: Top width of the canal.
- h Max: Maximum height of the canal (including free board).
- Manning's –n: Roughness coefficient of the reach.
- Canal Slope: Longitudinal slope of the reach (1/horizontal distance).

2.6 SUMMARY

It is essential in the designing of a computer model and/or management proposal that the design of these depends on the knowledge of the basics of hydraulics. With this knowledge, specialisation in the mentioned field can be attained and implemented at a specific study area. To understand water distribution a sound knowledge of water engineering and hydraulics is essential. Knowledge of the theory and basics of water distribution and hydraulics opens up a number of developing possibilities. With this knowledge known various management and mathematical models can be derived and developed for further use.

Chapter 3

Study Area

CHAPTER 3 STUDY AREA

3.1 WAS IMPLEMENTED IN SOUTH AFRICAN SCHEMES

The WAS has been successfully installed in various irrigation boards and Water User's Associations through out South Africa. At each scheme WAS is installed completely or partially as required by the situation. The following are a list of the current users and contact details of the WAS:

- Loskop Irrigation Board (013 2622078)
- Hartbeespoort Irrigation Board (012 2522027)
- Sundays River Water User Association (042 2340038)
- Korente Vette River Irrigation Board (028 7133433)
- Groenland Irrigation Board (021 848 9755)
- Gamtoos Irrigation Board (iib@iafrica.com)
- Impala Irrigation Board (iib@iafrica.com)
- Lower Olifants River Water User Association (unita@xsinet.co.za)
- Orange Riet Water User Association (unita@xsinet.co.za)
- Sandvet Government Water Scheme (057 3527375)
- Mooiriver Government Water Scheme (018 2973867)
- Vaalharts Government Water Scheme (053 4560508)
- Kalkfontein Irrigation Board (053 20504950)
- Department of Water Affairs: Worcester (023 3471625)
- Department of Water Affairs: Nelspruit
- Department of Water Affairs: Hartbeespoort
- Komati Basin Water Authority (KOBWA)
- Groot Marico Government Water Scheme

3.2 VHWUA COMPARED TO OTHER SCHEMES

WAS were compared to three other irrigation schemes/ boards in order to compare the effect of the implementation of the Water Release module as applied to various situations. The schemes were chosen on the grounds of similarities with the VHWUA. The three other schemes were:

- The Oranje Riet Water User Association;
- The Loskop Irrigation Board;
- The Hartbeespoort Irrigation Board.

All of these were compared to the VHWUA in terms of the degree of implementation of the various modules (Table 3.1). During the investigation and contact with the other schemes the effect of the implementation of the WAS were enquired about. In all instances the modules installed/ used, ease of calculating losses, water savings and advantages were compared to that obtained from the VHWUA.

The VHUWA is unique in its situation as it is at an interesting phase of the schemes life. The timing to implement the WAS is correct as the scheme is currently transferred from an Irrigation Scheme to a Water User Association. It is therefore ideal to implement a modern day water release application. The geographic location and layout canal network of the VHWUA makes a further reason to study the implementation of the WAS at the VHWUA.

Table 3.1: Comparison of VHWUA with other irrigation schemes/boards

Orange Riet Water User Association			
Contact: Nick Knoetze/ Danie de Wet **Tel:** 053 591 9200		**Installed:**	1993
Modules implemented/ used:	1	**Administration module:**	In use
	2	**Water Accounts module:**	In use
	3	**Water Request module:**	In use
	4	**Water Release module:**	Partially implemented
Ease of calculating losses:		Not calculating losses with WAS system at the moment.	
Water savings:		To the extent so far yes, but could increase if fully operation al WAS is	
Advantages:		Ease of use and proper administration.	

Loskop Irrigation board			
Contact: **Tel:** 013 262 3992		**Installed:**	1992
Modules implemented/ used:	1	**Administration module:**	In use
	2	**Water Accounts module:**	In use
	3	**Water Request module:**	In use
	4	**Water Release module:**	Conventional method (excel)
Ease of calculating losses:		Not calculating losses with WAS system at the moment.	
Water savings:		To the extent so far yes, but could increase if fully operation al WAS is	
Advantages:		Administration module assist the scheme a lot.	

Hartbeespoort Irrigation Board			
Contact: Vaun Thomas **Tel:** 012 252 2027		**Installed:**	+- 1999
Modules implemented/ used:	1	**Administration module:**	In use
	2	**Water Accounts module:**	In use
	3	**Water Request module:**	In use
	4	**Water Release module:**	Under implementation
Ease of calculating losses:		Very complete and accurate.	
Water savings:		Accurate calculation of losses proves to save water with current implementation of WAS.	
Advantages:		Accurate.	

Vaalharts Water User's Association			
Contact: Kobus Harbron **Tel:** 053 456 0131		**Installed:**	1993
Modules implemented/ used:	1	**Administration module:**	In use
	2	**Water Accounts module:**	In use
	3	**Water Request module:**	In use
	4	**Water Release module:**	Partially implemented
Ease of calculating losses:		Easy to calculate and operate.	
Water savings:		Real saving of water.	
Advantages:			

3.3 DESCRIPTION OF STUDY AREA

The study was conducted in the VHWUA, which is located in the Warrenton/Jan Kempdorp/Hartswater district in the Northern Cape, South Africa. The head office, in Jan Kempdorp, administers the VHWUA (Plate A2, Appendix A). The office is 60 km from Kimberley and is also known as the Lower-Vaal area office. This office comprises of the following sections: Vaalharts; Bloemhof Dam; Middle-Vaal Abstraction Control Area; Lower Vaal; Barkly West: KB Canal System; Taung Irrigation Area and Taung Dam; Harts River: Spitzkop; Harts River: Wentzel Dam and central workshop of the Department of Water Affairs and Forestry (DWAF). The office manages most of the water used in the Lower Vaal Management Area which incorporates the Northern Cape, North West and some of the Free State areas. The duties performed by this office include operation and management of the scheme for:

- approximately 1200 km of concrete-lined irrigation canals;
- 300 km of concrete-lined storm water drainage canals;
- Bloemhof Dam, Vaalharts Weir (Plate A1, Appendix A), Spitzkop Dam and Taung Dam; and
- approximately 250 houses and a number of office blocks, workshops, stores and other buildings.

Table 3.2 gives a brief summary of canal lengths in the entire network. The area office has 574 personnel excluding the personnel doing improvement and upgrading work on the canal. Existing water rights for irrigation amount to approximately 51 654 ha which represents a total volume of about 446 780 000 m^3 of water per annum. Additional to irrigation requirements existing industrial and household usage requires some 63 400 000 m^3 more water per annum.

Table 3.2: Summary of canal lengths

Main Canals	100 km
Secondary Canals	180 km
Tertiary Canals	540 km
KB Canals	320 km
Storm water drains	300 km

Figure 3.1 depicts the geographical location of the study area and the location of the VHWUA in relation to the rest of South Africa. As shown, the scheme stretches over three provinces namely; Free State, North West and Northern Cape. Several of the main cities of South Africa, as well as the important towns in the study area, are indicated on this map. The scheme is used mainly for irrigation of a wide range of crops, ranging from maize and peanuts to sunflower and lucerne. The Warrenton weir acts as the source for water and feeds the network. Figure 3.2 depicts the study area and the general positioning of the canals in relation to the irrigation district. The main source of water for the canals is the Warrenton weir which is fed by the Vaal River. The Vaal River flows into the Harts River and then continues on as the Vaal River with the Modder River joining near Douglas. The major dams in the area are also shown.

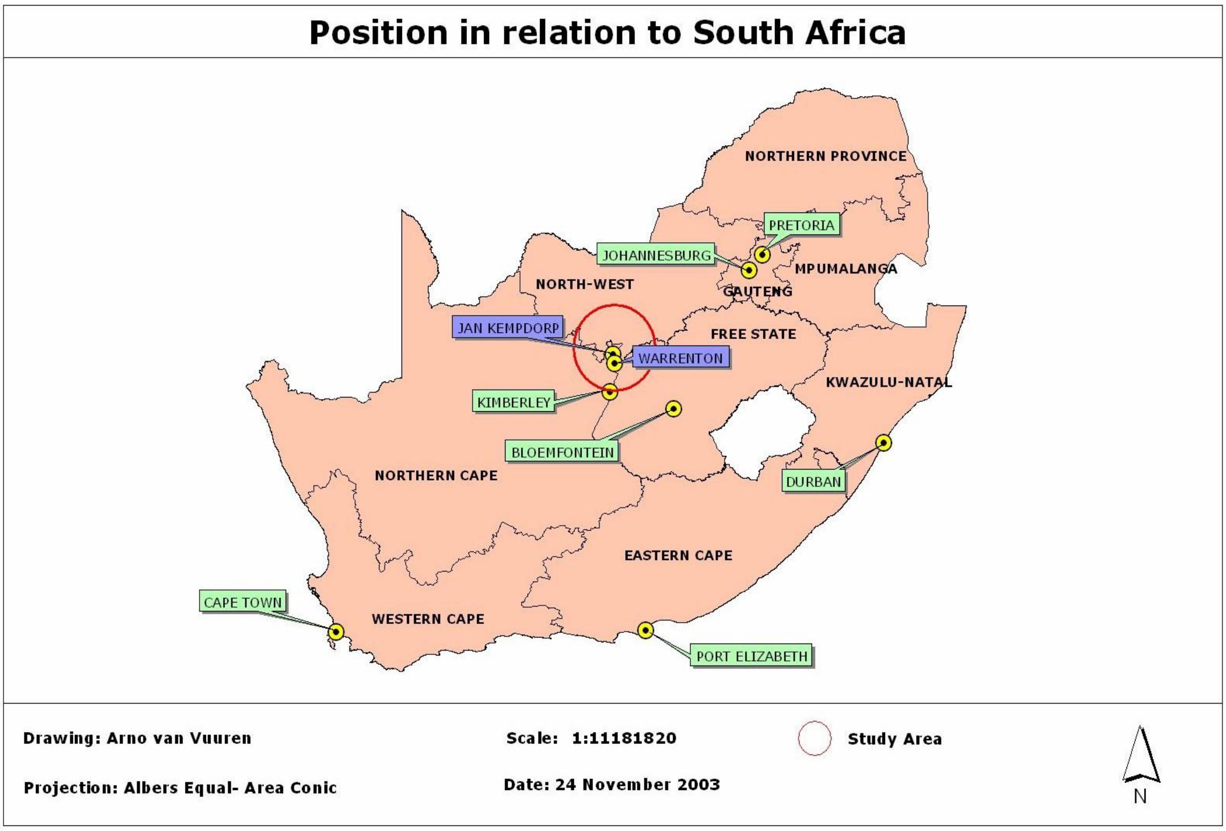


Figure 3.1: Position of VHWUA in relation to South Africa

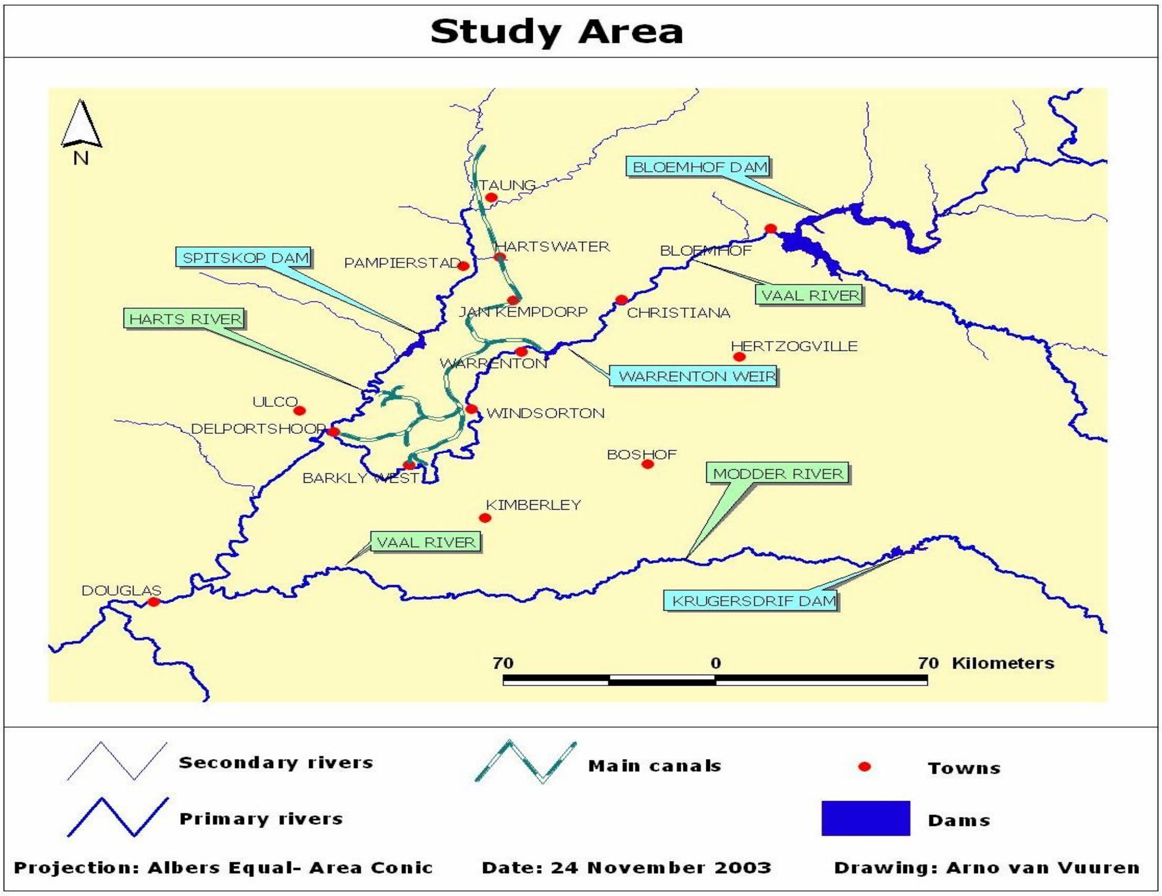


Figure 3.2: Study area of the VHWUA

As from the release calculation procedure (WAS), the required amount of water is released into the canal for the given week of the year. The water is then transported in the transfer canal to where it is diverted into two main canals, the north and west canals (Figure 3.3). The results of the WAS program as implemented at the VHWUA are illustrated by using the community canal 4A on one of the feeder canals, Feeder Four (F4) as an example. From each of these two main canals there are several feeder canals (see F4, Figure 3.3) transporting the water to listed users in the irrigation district (Plate A8, Appendix A). The mentioned district is topographically lower than the feeder canals. The water control officers then release a given amount of water from the feeder canal into the smaller community canals. Each user can then subtract water from these community canals (see 4A, Figure 3.3) as required. Another nine community canals branch from F4, namely 4A to 4H and 4J. The canal data for 4A is given in Figure 3.3. This canal has six turnouts/sluices and they are numbered 4A1 to 4A4.

Figure 3.4 indicates the type of each turnout as well as the chain distance from the source (e.g. turnout 4A2 is an outflow and the distance is 410 m from the source (community canal 4A). The source of canal 4A is a 2 ft Parshall weir structure measuring the amount of water let into the canal. The program will calculate the evaporation and seepage for the entire community canal. The capacity of the canal indicated in Figure 3.4 is 391 m^3/h.

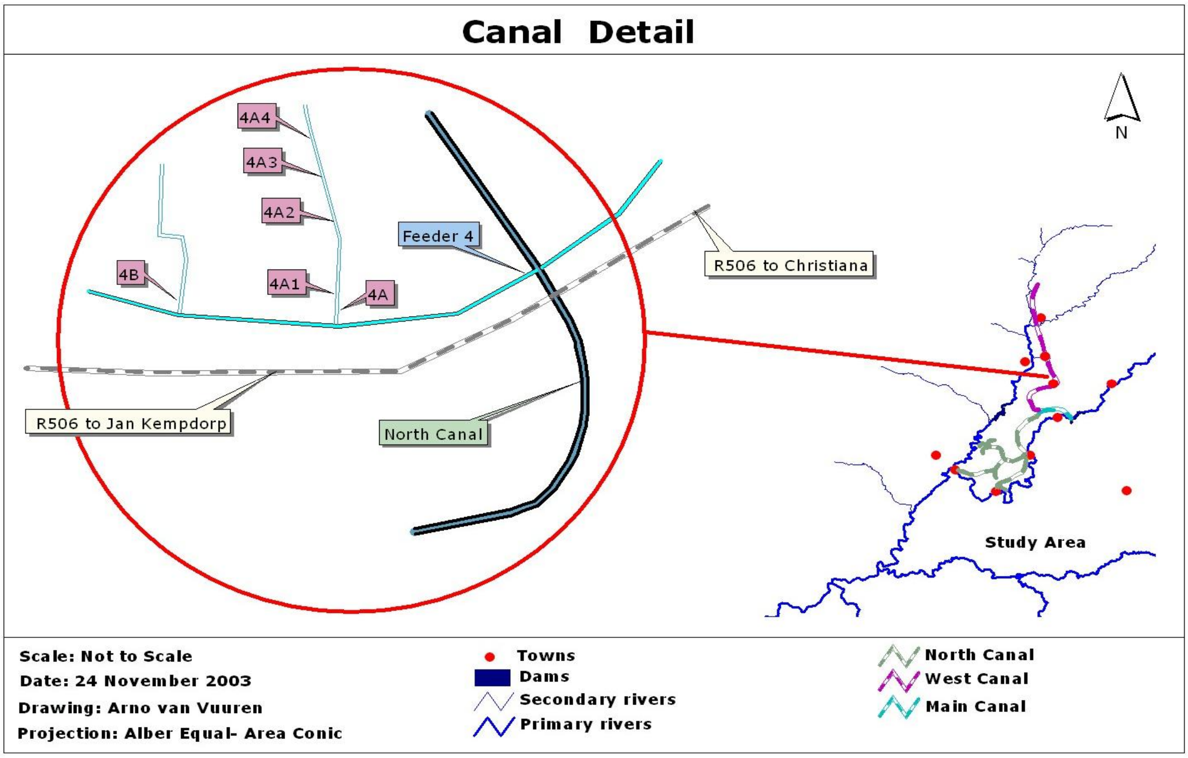


Figure 3.3: Canal detail

The maximum abstraction right (m.a.r.) of the users therefore cannot exceed 391 m^3/h. All the canals are concrete-lined and differ only in geometric dimensions (see Table 3.2). Table 3.3 indicates the average canal dimensions for each of the two general types of canals and gives the reader an idea of the general size of a canal. The two main canals and the feeder canals are all trapezoidal, while most of the community canals are parabolic sections. The canals are only rectangular where they are used for transitional areas for flow measurement and are thus dependent upon the size of the measuring structure, such as parshall flumes and crump weirs.

Table 3.3: Average canal dimension

Geometric shape	Top width (m)	Canal depth (m)
Trapezoidal (main)	8.699	2.704
Parabolic (feeder)	2.06	1.283

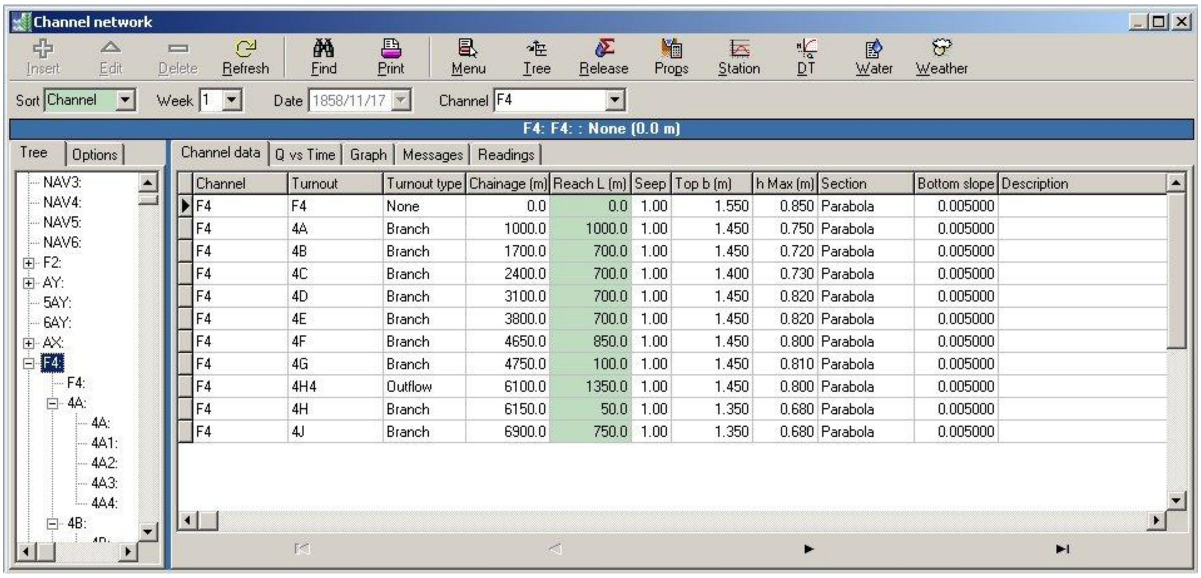


Figure 3.4: Detail on F4 (Feeder Four)

Chapter 4

Data Collection and Calculation Procedures

CHAPTER 4 DATA COLLECTION AND CALCULATION PROCEDURES

4.1 INTRODUCTION

A proper research methodology not only comprises the physical method of conducting research, but also includes the methodology to understand the specific research to be done. Therefore complementary knowledge such as theory of hydraulics and distribution systems is essential in order to conduct the research effectively. Once this background is in place, data could be collected in the correct way for analyses and result purposes.

The field work was done at the VHWUA and Feeder Four were once again selected as the typical feeder to prove the hypothesis on.

4.2 DATA COLLECTION AND VERIFICATION

The WAS program, as designed by Dr. Benadé, bases its flow calculations on the Manning Equation (Benadé, 2001b: Online). It is evident thus far that a variety of geometrical information regarding the canal is needed to calculate the release of water. Since the main purpose of the project described in this study is to implement the release module of the WAS program, it was crucial to verify all information and data for flow calculation purposes, as all the calculations were based on this database. Four general methods to verify information and existing data, namely:

- Meetings/consultations with farmers, technicians, etc;
- Engineering design drawings;

- Mathematical calculations and comparisons; and
- Collection of field data.

A fifth method should also be applied. In this method calculated data should be compared to one another in order to interpret the results and findings of calculations. Volumes calculated with the manual method of volume calculation should be compared to the volumes delivered by the corrected WAS database. This method is the most important as it will indicate the effectiveness and validity of the WAS calculation procedure. Further details and results are discussed in Chapter 5.

4.2.1 Meetings and consultations

Conducting meetings and having consultations with the various water control officers can prove to be a very useful tool. Each one of the 10 water control officers in the VHWUA is responsible for a certain number of the 27 water wards. On the north canal there are 23 wards, and there are four wards on the west canal. Each officer is responsible for his/her given wards and is therefore well acquainted with their assigned wards. Consulting and meeting with them will be valuable in terms of the knowledge they can offer for the ward and given length of canal under their control. These water control officers can supply data like canal shape, approximate size, measuring structure, turnouts and sluices, approximate reach between community canals, canal capacity, estimated lag times, etc. Other useful information to the scheme is in terms of improvement and upgrading the canal. Here they could supply information on leakages in the canal, faulty sluices and theft of water from the canal. Data such as this will improve the overall management of the irrigation scheme and this optimises the irrigation scheme.

4.2.2 Engineering design drawings

Further data collection and verification should be done by studying and interpreting engineering as-built drawings. These drawings are the original technical drawings used in the design and construction of the canal network. One of the most important data sets that could be collected from the technical drawings is the length of the canals and the friction slopes in the various canals. Necessary data can be read off from the corresponding canal section in the drawings. Minor details on canal geometry can also be checked and verified.

4.2.3 Mathematical calculations and comparisons

In the early nineties (1991–1993), both the north and west canals were enlarged by converting the shape to a combined trapezoidal canal. A type of collar was added to the top edges of the canal (Plate 4.1). This collar changed the geometrical shape of the canal and does not correspond to the geometrical details as in the WAS database. Several advantages of using this shape exist, although the main purpose is for improved canal utilisation. The combined section yields a greater discharge than the original normal trapezoidal canal and in times of low flow evaporation is reduced due to the smaller surface area exposed to the sun. When the flow rises and exceeds the original depth of the canal, the canal discharge is subject to higher levels of evaporation and seepage. The problem arises in the water release calculations. The program will either calculate the discharge in a given section of the canal by looking at it as a normal trapezoidal section or it might calculate it as a combined trapezoidal canal. The data according to which the program calculates depends on the settings the user specifies to the computer. The data as used and entered into the WAS program are for a normal trapezoidal canal section, while in reality a combined canal section is used.

The water release is calculated regarding the current data available but mathematical calculations could be done to investigate the difference in discharge between these two types of sections. Current calculations could have the effect that either too much or too little water is calculated for the canal. The correct canal type will yield the correct required volume for the canal, but corrective changes could be made to the database resulting from these comparatives. All parameters such as friction slope and Manning coefficient stay the same, but the area of flow and hydraulic radius will change for the two different section types. Once a result is obtained, further assumptions and recommendations can be made. The results of the calculations are discussed in Chapter 5.

Plate 4.1: Combined trapezoidal canal (north canal)

4.2.4 Collection of field data

A more time-consuming method of data verification would be making field trips to investigate the canals. However pre-defined spot checks on the canal are very reliable as data can be collected in the actual field situation. Canal sections with lacking information can be visited in person. In a spot check visit, various data were collected, including:

- type of canal;
- canal geometry;
- measuring structure in place;
- information for the calculation of canal capacity;
- number of turnouts, etc.

The best way would be to visit every canal on the entire network, but due to time constraints pre-defined canals should be visited in order to collect information for calculation and verification purposes. Table B1, Appendix B shows an extract from the data collected for F4. The method to be followed should be to visit Feeder Four in detail and the scheme on a representative basis. Once the data is collected a comparison should be made between field data and data currently on the canal layout in the WAS program. From the comparisons various alterations could be made to the data set of the irrigation scheme. Plate 4.2 and Plate 4.3 indicate methods to collect the mentioned data.

Plate 4.2: Structures to be verified for correctness

Plate 4.3: Measurements for data verification of canal geometry

4.3 SUMMARY

Following the correct methodology, satisfactory results can be obtained through calculations. A full understanding of the methodology to be followed will therefore enable the researcher to correctly discuss and interpret calculations and findings. The application of the correct methodology will yield consistent and reliable results. It is for the same reason that it is important to apply the correct methodology in order to obtain trustworthy results. The correct methodology can also form the basis for other related studies.

Chapter 5

Results and Discussion

CHAPTER 5 RESULTS AND DISCUSSION

5.1 INTRODUCTION

As described in Chapter 4, the following four methods of data collection and verification were used:

- Meetings/consultations;
- Engineering design drawings;
- Mathematical calculations and comparisons; and
- Collection of field data.

The program verification and comparison results will make up the additional fifth verification method. Results described in this chapter constitute the basis of the recommendations. Data collection and verification were done on the entire irrigation scheme, but focus was placed on F4. If all verification can be done successfully on F4 in terms of delivering satisfactory results and proving the calculation procedure correct, the same procedure can be applied to all the other canals in the network. It is therefore stated that if the WAS program can be implemented on F4; the same can be done successfully on a scheme-wide basis.

5.2 MEETINGS AND CONSULTATIONS

Modern day engineers tend to focus less of their time on the physical designing component of a project and more time on planning, marketing, management and community involvement (Ossin, 1999). It is becoming more important to incorporate these aspects into a project as this will ensure sustainability and success of the project. One of the most important aspects in this regard is surely community involvement. With positive community involvement key role players in any project are instrumental in collecting required data. For this reason it is important to involve the community in the project.

On 27 August 2002 meetings and consultations were initiated with some of the water control officers and other key role players of the community. A general planning meeting was held where various aspects of the WAS program were explained to them as well as the reasons why various implementations needed to take place. It was important to inform them, to enable them to delegate instructions to subordinates, and for them to have a better understanding of the motivations of the project. At this meeting some important information were collected. From this information decisive decisions can be made for further assumptions. The information included:

- the attitude of the community towards the use of the model;
- community's knowledge of the WAS program;
- areas where data should be collected;
- type of data to be collected;
- methods of data collection; and
- the best approach to be used with regard to the community.

Seven more consultations were held to obtain the necessary information from the water control officers. Relevant and trustworthy information was obtained through the results. This information was used to compare data on the system and collected data. Information was disseminated to the community. A further method of keeping the community involved is to be available on a regular basis if they want to contribute or enquire about information (Plate A10, Appendix A). It is possible to continuously calibrate the WAS program through keeping the community involved in the calibration process. Refer to section 5.6.1 in Chapter 5 for detail on the calibration process of the WAS program.

5.3 ENGINEERING DESIGN DRAWINGS

In Chapter 4 it was mentioned that one of the data sets to be collected is the friction coefficient and slope. The friction coefficient (Chow, 1959) is directly influenced by the slope of the canal. It is therefore important to verify the slope at various intervals. The gradient of the energy line could also be derived from this. There are two basic methods of checking the adequacy of the friction slope. In the one method the change in height over the change in length were calculated using a leveling instrument and tape measure (Plate 4.3, Chapter 4). These measurements were taken in the field, calculated in the office and then compared to the known data on the computer.

The other method was to use the as-built drawings from the consulting engineer. The data was compared with the corresponding data on the canal layout on the computer. The latter method is the most applicable as initial design intentions are stipulated on the drawings. Specifically for this type of data set, engineering drawings proved to be a satisfactory means of data verification. Data in the WAS program verified against existing records, using this method proved to be correct and valid. Data that were previously unavailable were updated using this method.

5.4 MATHEMATICAL CALCULATIONS AND COMPARISONS

The presence of two different trapezoidal canals made it difficult for the user to choose which type to use. The combined trapezoidal canal is in use on the scheme, but the normal trapezoidal shape is used by the WAS program in calculations. Depending on several factors the section could stay in use as long as the depth does not increase beyond a certain depth. Figure 5.1 shows the geometry detail for the two mentioned canals. Values compared are true when the normal flow depth is lower than 2.135 m. Indicated in Figure 5.1 section number 2 is in use in the canal network, while section number 1 is described in the WAS database.

The object of this exercise was to compare the flow discharge from the two canals and then to make decisions regarding the change in volume delivered by the WAS. Using the Manning equation, cumulative values for discharge were calculated for both canals in each instance. The flow rate was calculated for increases in depth of 100 mm for each increment up to the full depth of 3.128 m. The user is now able to compare the results in volume delivered and change the database accordingly. Table B2 and Table B3 in Appendix B, show the normal and combined trapezoidal canals respectively. Indicated in red at the bottom of each table is the discharge at a depth of 3.128 m. It can clearly be seen that for a combined trapezoidal canal the discharge is 19.13 m^3/s less than for the normal trapezoidal canal. If the combined canal is in operation at the scheme, but the normal canal shape is used as the basis for calculation, problems might occur. The exercise proves that the wrong flow rate of water is calculated. The entire exercise regarding the mathematical comparison of the two canal sections proved to be a valuable input, particularly when it came to optimisation of the canal calculation procedure. One of the benefits of the WAS program is to improve water utilisation (Benadé, 2001a). Mathematical comparisons like this procedure add to the refinement of the program and ensure optimised implementation into the irrigation scheme. The WAS offers the user the option of choosing between predefined canal section types or user-defined types. The user can now switch to a user-defined canal section type as it will yield correct volumes.

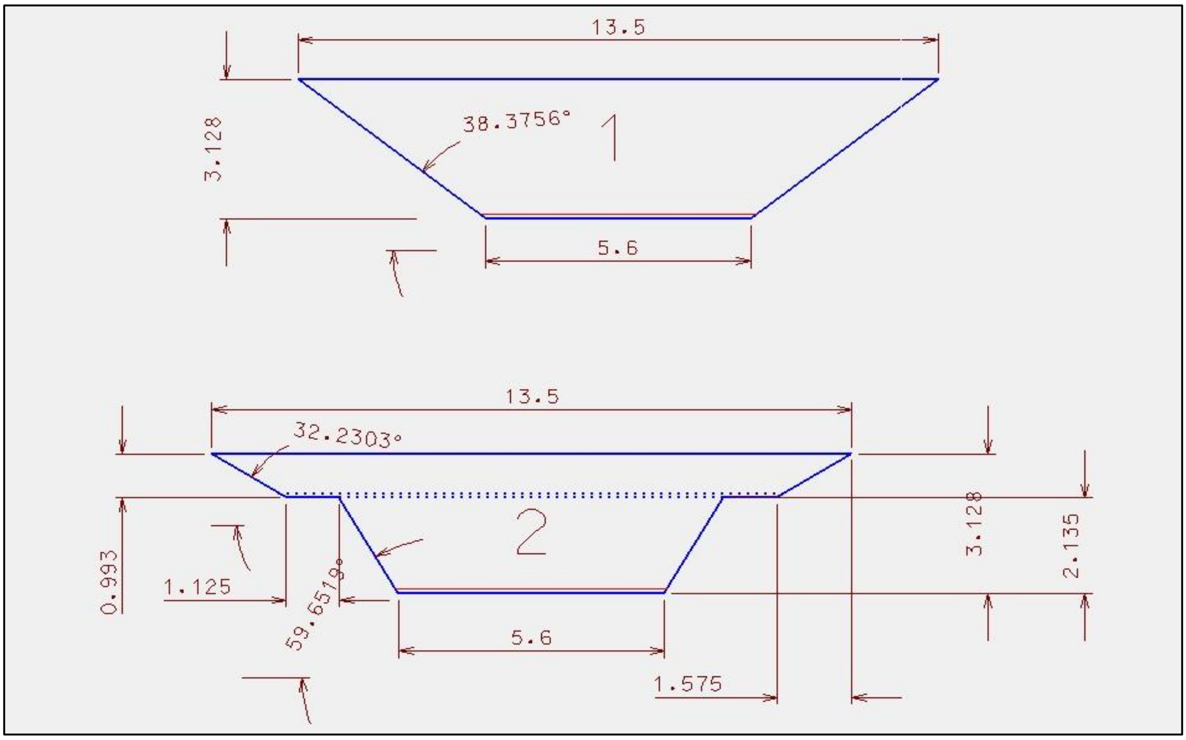


Figure 5.1: Comparing trapezoidal canals

Figure 5.2 - Figure 5.4 show comparisons done between the two canal types. In Figure 5.2 the depth was compared with the discharges delivered. The normal canal type constantly calculates a greater discharge (m^3/h) for the same depth than the combined canal type. In Figure 5.3 the depth was compared to the wetted perimeter and it can clearly be seen how the wetted perimeter increases as the water starts flowing in the enlarged section of the canal. Although the wetted perimeter for the combined canal is bigger than the normal canal type, the normal canal type still delivers more water at the same depth (3.128 m). The kink in the curve is due to the change in the hydraulic a radius. In the comparison between the depth and the cross sectional area (Figure 5.4) the area of the normal canal type is still larger and will therefore still deliver a greater discharge than for the combined canal type.

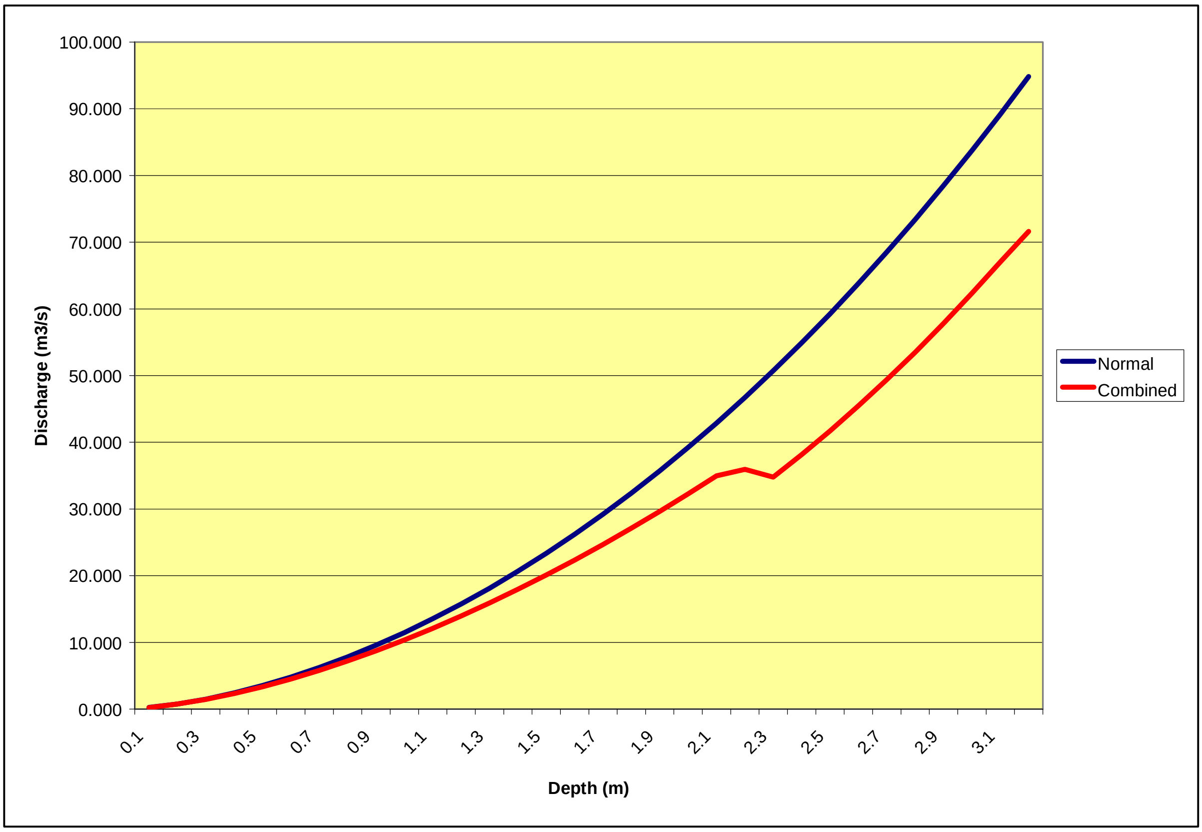


Figure 5.2: Mathematical calculations: d versus Q

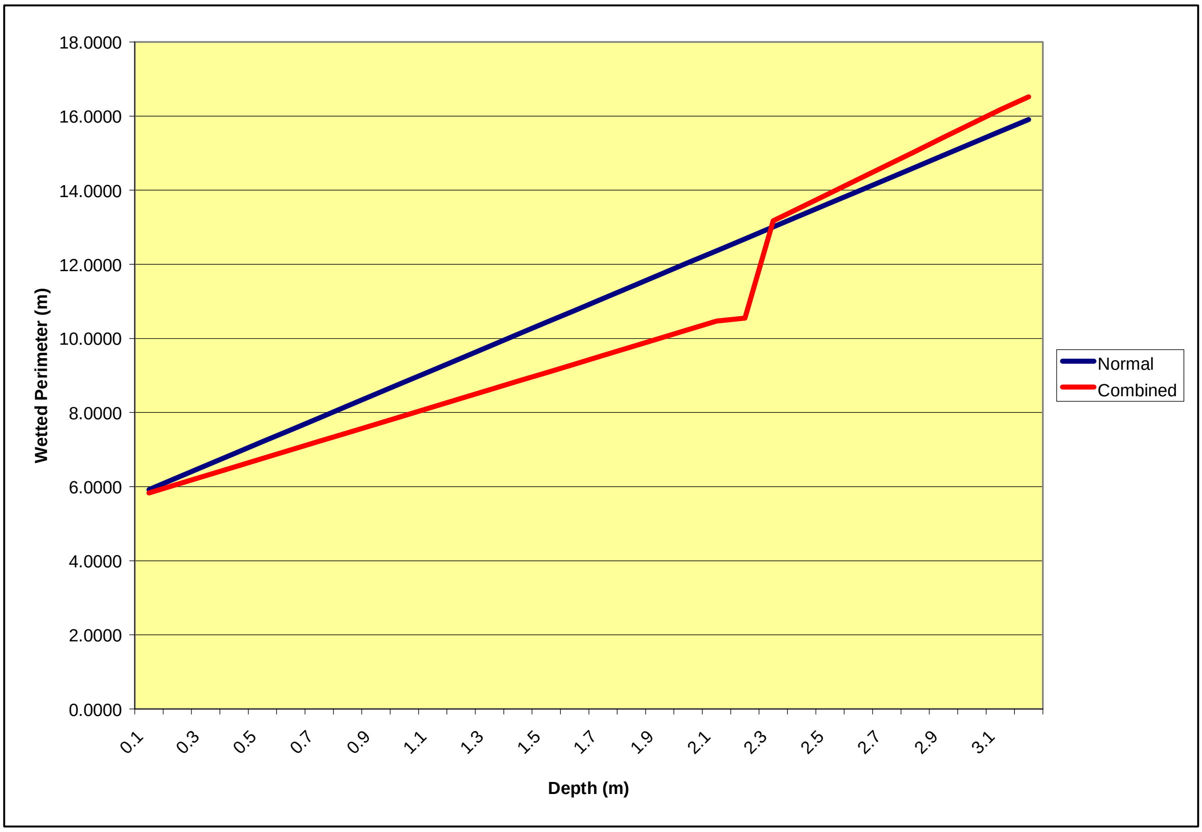


Figure 5.3: Mathematical calculations: d versus P

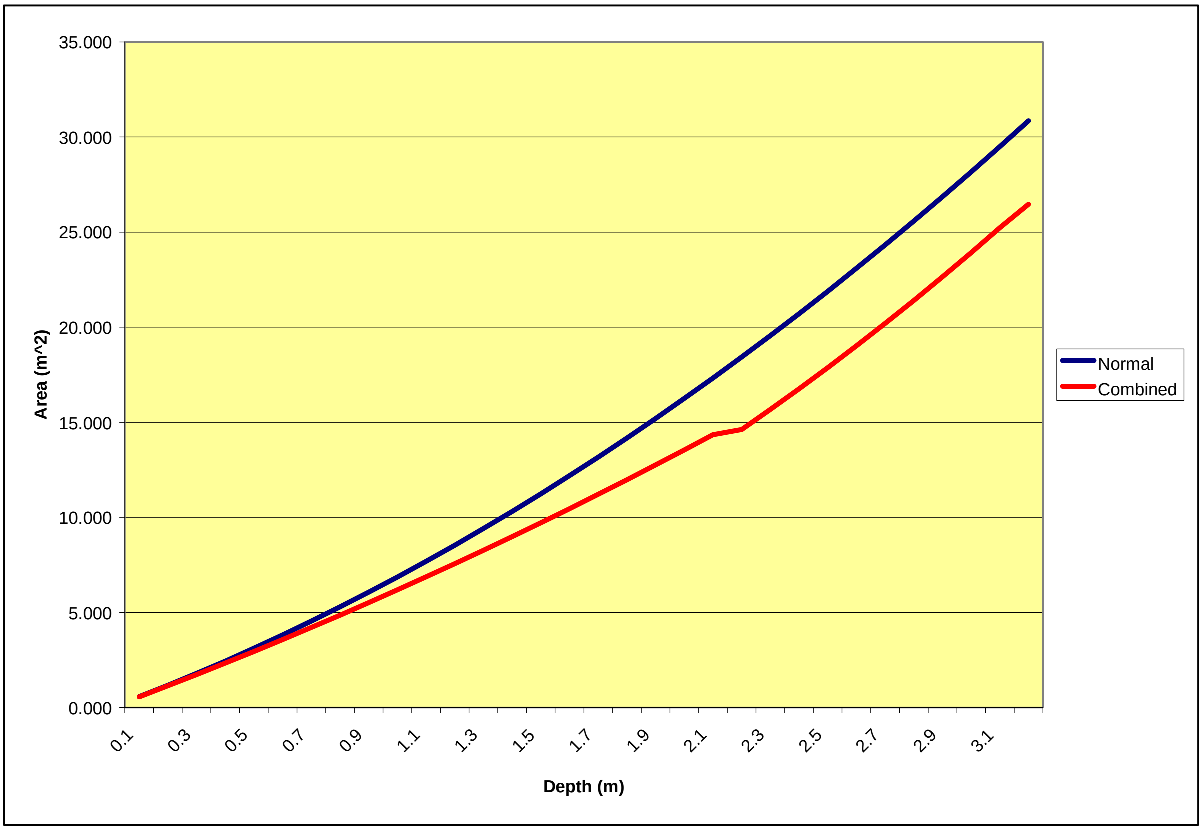


Figure 5.4: Mathematical calculations: d versus A

5.5 COLLECTION OF FIELD DATA

Satisfactory results were obtained by collecting data, as described in Chapter 4 on site in the field and comparing it to existing data on the canal layout in the WAS program. Table B4 in Appendix B contains both the data set resulting from field work and the canal layout as it is on the WAS program. Once again community canal 4A was used to illustrate the desired results. As discussed before, data from the canal layout was used to base the water release calculations on. It can clearly be seen that data collected in the field should correspond with the actual computer data. Field work thus proved that the data for community canal 4A were different from that of the actual canal. This implies that data on the computer should be updated regularly so that only the correct and verified data are used. Applying this method randomly to other sections of the canal will result in a general idea of all the correct and incorrect data. The most desired scenario would be to verify each and every turnout on the scheme using this method. Due to time and other constraints, however, it was not possible in this study.

Another result from the field data collection was the verification of the size of measuring structures. This is an important aspect as incorrect measurement of water will result in users receiving too much or too little water. Users will react negatively if they receive too little water, and allowing them more water than requested reflects an improper resource utilisation and mismanagement of available water. It was therefore important to check the adequacy of the measuring structures letting water into various canals. One example was to check the width of a crump weir, as the width is used in the formula to calculate the flow rate. The Parshall flume was verified by checking the geometrical properties of the measuring structure (Plate 5.1 and 5.2). The correct geometrical properties should be entered to the WAS database. It can clearly be seen how measuring the geometry of the structure would correspond to the known data on the canal database and therefore prove it correct or not. Together with the width, the depth is also necessary to calculate the discharge at such a point. The measuring plate can thus be checked by comparing the actual measured depth with the reading on the measuring plate. This measuring plate was found to be working properly. Water utilisation and general scheme management could thus be optimised by reducing minor errors in distribution losses.

Plate 5.1: Parshall flume in Feeder 4

Plate 5.2: Measurements in Parshall flume in Feeder 4

With all the data to be verified, the four methods of data verification proved to be sufficiently adequate to obtain all the information needed. Each type of dataset to be checked was represented in at least one of the methods. Therefore, all data to be checked and verified were collected and analysed using one or a combination of the four methods. For each type of data an appropriate method could be applied to collect and verify the information. In the water release calculation procedure and process, the WAS program calculated water releases for the main canal and all its branches, allowing for lag times and any water losses and accruals (Chapter 1). The water release volume can therefore be calculated correctly.

The calculation message are summarised Figure 5.5. The message page is the first official sign that results have been updated on the database, and this can be seen when calculations use the applicable data. The moment data are changed the effect can be seen on the message page after the water release volume has been calculated. The message page shows the settings used in the calculation as well as the time to execute the calculation. Shown on the left hand side of the message page is the tree diagram of the canal layout. On the right hand side the checked information from the release calculation is shown. This reflects the release volume of water for a specific section of the canal and during a certain week of the water year. Errors in the calculation of the release volume are shown here.

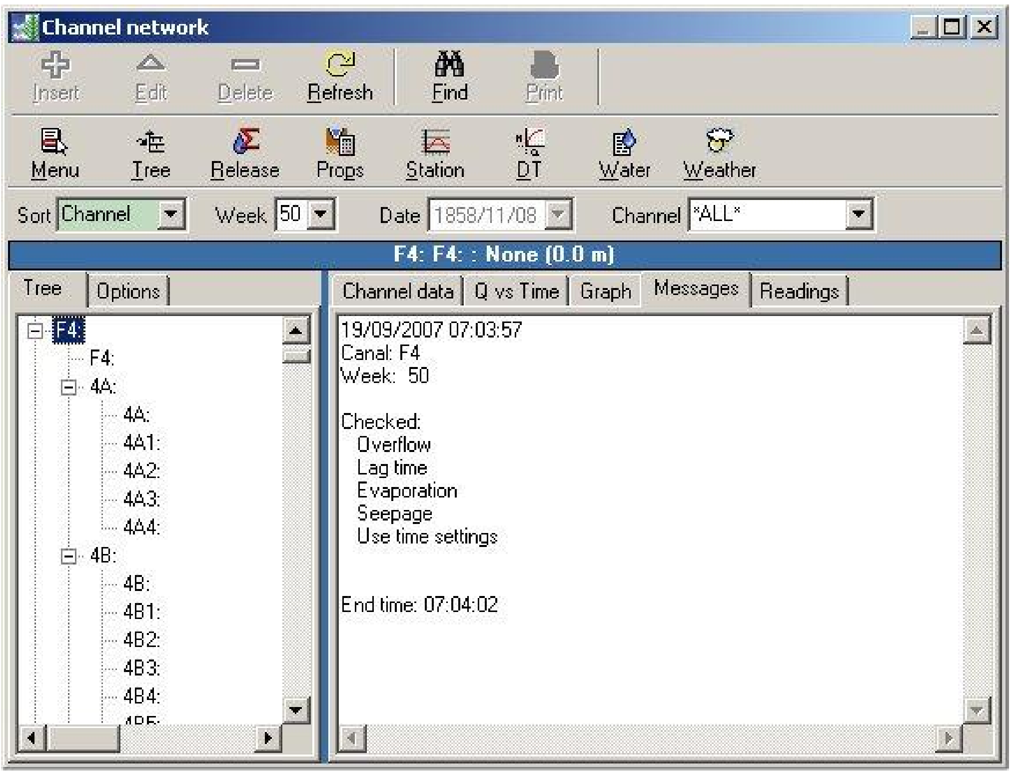


Figure 5.5: Message page

5.6 EXCEL VERSUS WAS COMPARISON/EVALUATION

A fifth method of data verification was also used. As mentioned and discussed in previous chapters, the VHWUA uses its own process of calculating the water release volume (Microsoft Excel) and the WAS program can accurately calculate the same release volume using its own process. Since all the relevant data, for calculation purposes, are updated and verified, a process of calculation comparisons and evaluations need to take place. The aim of the exercise was to calculate the release volume for a given irrigation period as well as each feeder canal separately, by using both programs. A cycle of analysis and evaluation was done to show the similarities and differences between the two calculation methods. Various assumptions can then be made from the data.

5.6.1 Comparison methodology

Water control officers can calculate the total volume of water needed by a single feeder canal from all the application forms handed in by the users. This refers to the requested volume of water that will be needed by each and every user only. The losses, as derived by the manual (Excel) method of calculation, are only accounted for when Excel does the canal volume calculation. The WAS in its turn will use all available data and perform the same calculation, also producing a release volume (Figure 5.6). Figure 5.6 indicates the details supplied by the WAS program of the time when a certain volume of water should be released, the flow rate that would be delivered, the expected reading on the measuring plate as well as the total of the cumulative volume that would be released.

Channel data | Q vs Time | Graph | Messages | Readings

Day	Time	Q (m3/hour)	Reading (mm)	Vol (m3)
Monday	00:00	2036	266	170
Tuesday	00:00	2636	314	49075
Tuesday	12:00	2470	301	80694
Wednesday	00:00	2746	322	110363
Wednesday	12:00	2605	312	143304
Thursday	00:00	2692	318	174574
Thursday	12:00	2516	305	206862
Friday	00:00	2899	334	237089
Friday	12:00	2736	322	271865
Saturday	00:00	2153	276	304643
Saturday	12:00	852	151	330368
Sunday	00:00	504	107	340562
Sunday	07:00	0	0	344049

Figure 5.6: Q versus time calculation results in WAS

The dataset used was for the 2003/2004 water year and the irrigation periods were 14, 30 and 46. These sets were chosen at random. In the first cycle of analysis the release volume for all the major feeder canals is compiled in a table (Table B5, Appendix B). The table shows the WAS and Excel release volumes respectively (m^3) and then the volume difference (m^3) between the two methods. The difference in volume is by how much WAS calculates more or less in relation to Excel. The last column shows the percentage difference between the various calculation methods. The difference is calculated with Excel in relation to WAS.

In the second cycle the same values were used for the Excel calculation while the values were refined for the WAS calculation (Figure 5.7). Accurate weather data were obtained to determine the most appropriate evaporation value to use (Table B7, Appendix B). A monthly average of 6 mm/day for evaporation was obtained. This value was used for calibration purposes only, but is noted that the real time evaporation values for the VHWUA should be used when the fully operational WAS program in use at the VHWUA. Transpiration was not included since the canal in question was a concrete lined canal and it may be assumed that little or no vegetation would be present. The coefficient for lag time (C_V) was adjusted to represent the correct lag time of water in the canal.

Various sections of the canal network suffer from seepage due to defects. Seepage in WAS are corrected according to the water control office operator to resemble the true field conditions. A slide bar in the release calculator (Figure 5.7) can be used for finer alterations. The various options to be included in the calculations can also be marked or unmarked. Indicated in Figurer 5.7 the transpiration was unmarked as we are working with a concrete lined canal where no vegetation is present. The canal storage is only used when the canal are empty and needs to be filled. If a release volume for the given feeder canal is now calculated, different but more accurate values are obtained. The first cycle of calculations was done as a starting block, with no alteration to the calculation data. The second cycle of calculations was done on the same periods of data but with adjustments to suit the actual scheme conditions.

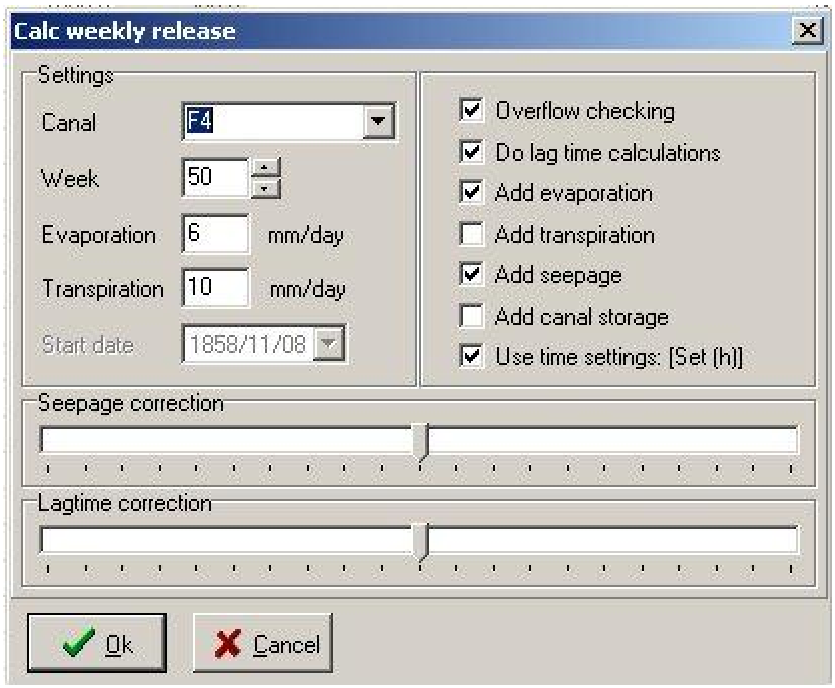


Figure 5.7: Release calculator

The final calibration was done in the third cycle of calculations. This calibration is of utmost importance to the results. Accurate calibrations were done regarding the seepage and the lag time coefficient by adjusting some final canal geometry inputs. Seven calibration sets were applied to the data after one another and calibration set #7 proved to deliver the most accurate data. In each set of calculations the WAS values were compared to the Excel values. The two methods were compared by calculating values without including the losses. Both methods should deliver the same volume. By proving this it was ensured that the correct calculations are done (excluding losses) and that the losses can now be added to calculate the true release volume. No errors were shown on the message page (Figure 5.5) and the final results could therefore be used as recommendations.

5.6.2 Comparison of results

It is evident from tables B5 and B6, Appendix B, as well as Figure 5.8, that the WAS over-calculates the majority of the data in the first cycle of calculations. On the other hand the WAS under-calculates the release volume in the second cycle but with a smaller value than what the difference was in the first cycle. In both of these cycles under-calculated values are obtained for Feeder 1, 2, 17 and 24 to 27 (The Excel calculation combines the values for Feeders 24 to Feeder 27.) The figure for Period 30 (Figure 5.9) shows that Feeder 12 (F12) over-calculates the values by a large amount due to various incorrect data regarding the canal. All other volumes are calculated more accurately in relation to the Excel calculation. An even smaller difference can be observed in the second cycle of calculations. With F12 removed from the graph the same type of layout will be obtained as for Period 14, i.e. under-calculated values are obtained for feeders at the beginning and at the end of the canal network. The same pattern was repeated for Period 14 (Figure 5.8) and Period 30. F12 also calculates a percentage difference in the range of 2000% but the values were reduced in the second cycle of calculations (tables B5 and B6, Appendix B). Period 46 (Figure 5.10), calculates a more evenly distributed diagram with no exaggerated miscalculated values. It can be seen that the moment the refined second cycle calculations are compared with first cycle calculations, the WAS generally under-calculates by a smaller margin that it over-calculates with unrefined values. Therefore refining the calculation procedure to make it more applicable to the scheme will ensure that correct values will be obtained.

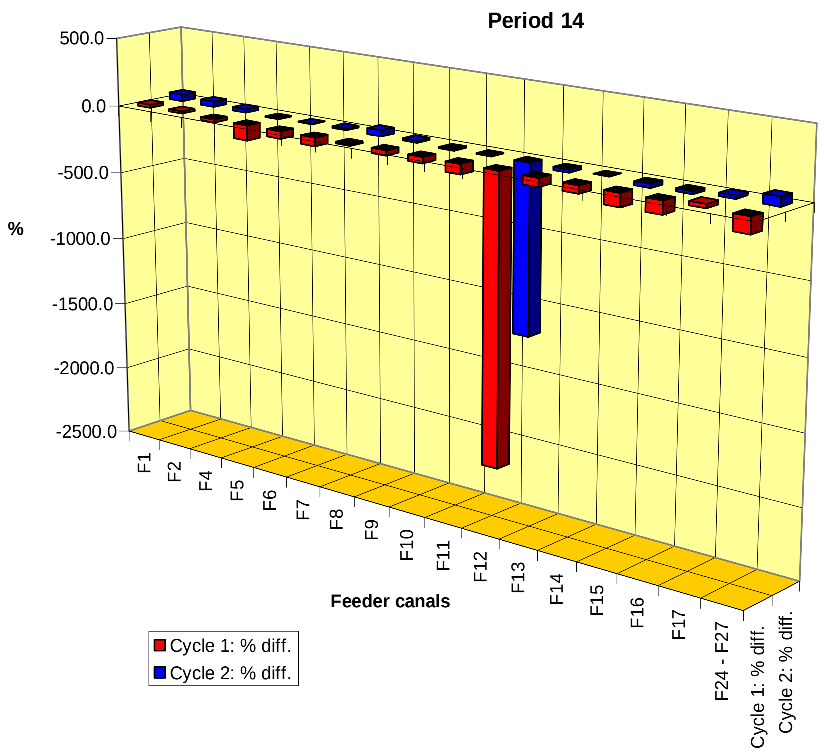


Figure 5.8: Calculation comparison for Period 14

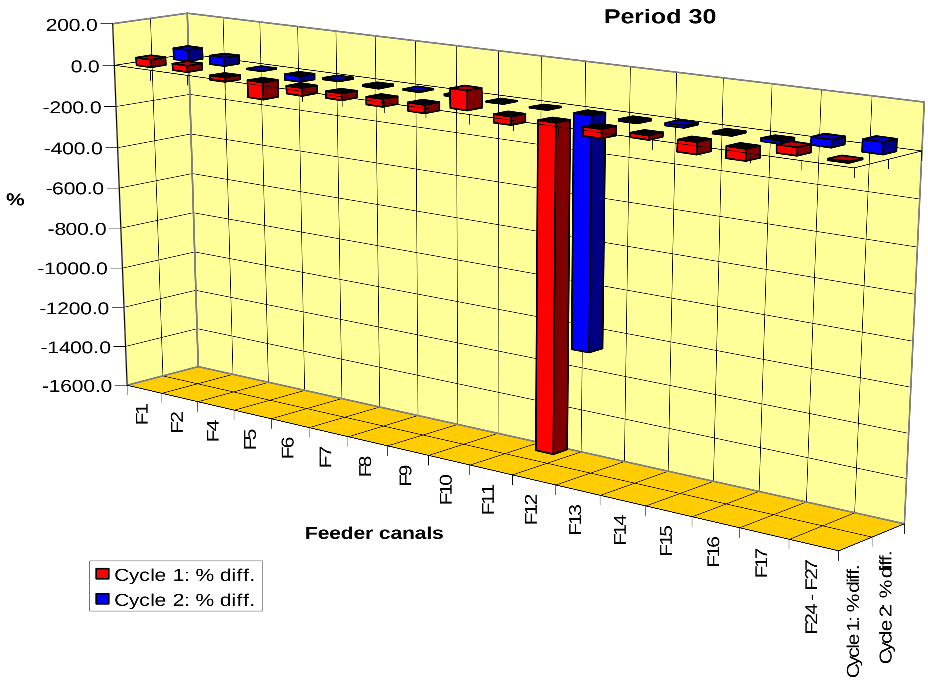


Figure 5.9: Calculation comparison for Period 30

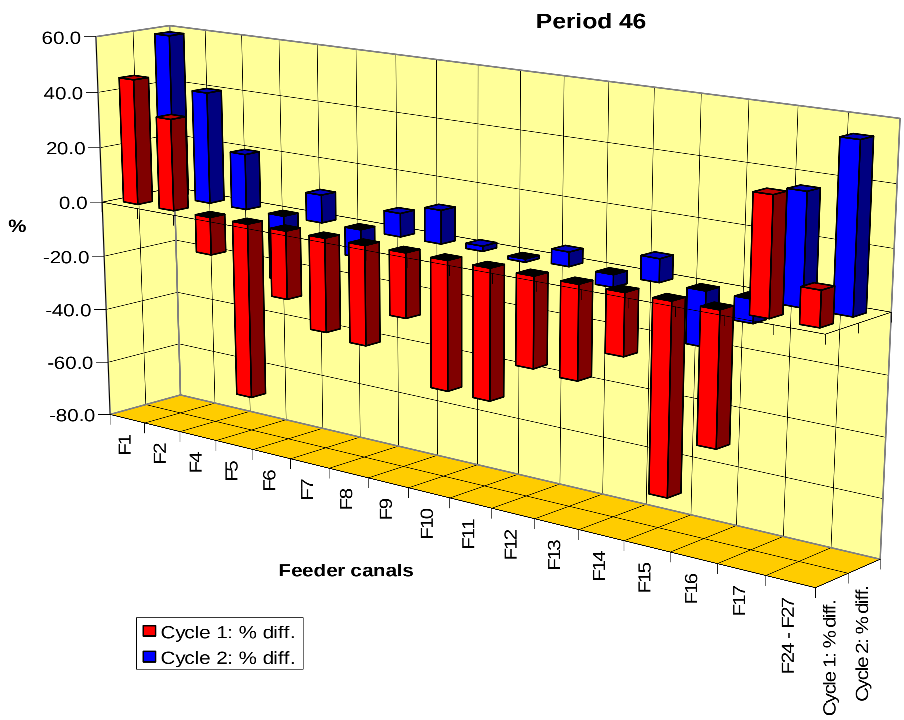


Figure 5.10: Calculation comparison for Period 46

The hypothesis has stated at the beginning of this chapter that if all verification could be done successfully on a single feeder, delivering satisfactory results and proving the calculation procedure correct, then the same procedure could be applied to all the other canals in the network. For this reason F4 was singled out and the third cycle of calibrations and calculations was executed. Table 5.1 shows the calculation results after the third cycle of calibrations. The percentage difference (as indicated in red) for the volumes without losses is 0% (calculation comparison test). As the same calibration values were applied for all the periods, the values shown in green are the most accurate. The test has already shown that the calculation procedure of the WAS is correct as the values indicated reflect only the requested amount of water without any losses. This corresponds with the Excel values, thus proving them correct. Any differences still encountered indicate the difference in calculation of water losses between the two programs. Finer calibration to the WAS can now be done at scheme level by adjusting the slide bars for seepage and lag time.

Table 5.1: Calculation results for Cycle 3

	Period 14		Period 30		Period 46	
	Description	Volume	Description	Volume	Description	Volume
VHWUA	Without losses	138600	Without losses	241200	Without losses	279000
	With losses (Excel)	223692	With losses (Excel)	359412	With losses (Excel)	418803
WAS	Without losses	138600	Without losses	241200	Without losses	279000
	With losses (WAS)	232968	With losses (WAS)	355899	With losses (WAS)	397506
	% diff- without losses	0.0	% diff- without losses	0.0	% diff- without losses	0.0
	% diff- with losses	4.1	% diff- with losses	-1.0	% diff- with losses	-5.1

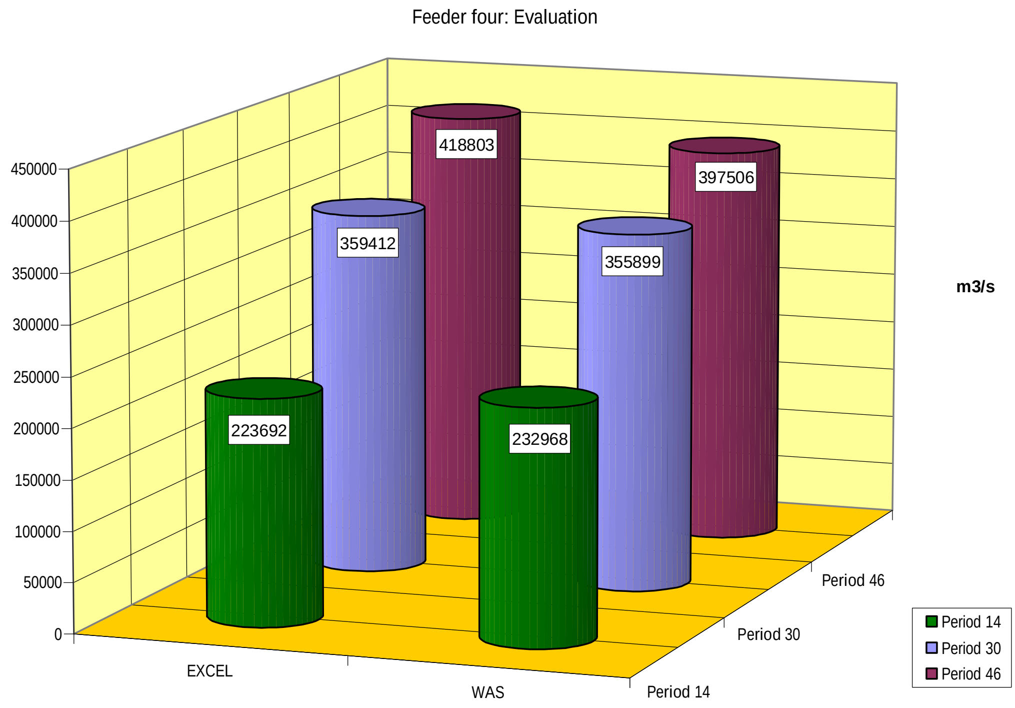


Figure 5.11: WAS versus Excel: Final calibration results

Figure 5.11 shows the difference between the WAS calculation results and the Excel calculation results. It may therefore be concluded that all necessary data were updated and verified correctly as final calculations proved to correspond. The verified dataset can now be applied to the VHWUA. Since F4 was successfully validated, it may be assumed that the same method to validate and calibrate data can be applied to any of the other feeder canals in the VHWUA. Testing each canal after the validations and calibrations is very important. The rest of the canal network could therefore be built into this basis of data verification. By completing this exercise, trustworthy results can be obtained and the WAS program can be implemented fully.

5.7 MANAGEMENT IMPROVEMENT

As the WAS is a management tool, all four of the modules should contribute to the overall management of the scheme. Depending on the requirements of the irrigation scheme, only some of the modules will be required for use; the VHWUA, however, requires all four modules to be implemented. Therefore with the implementation of the final module, better management can be achieved with the use of all the benefits of the WAS program (Chapter 4). There are also positive managerial implications other than the mentioned benefits. One of these is a streamlined, well explained and open route of communication between the irrigation office and the irrigation community. This can be achieved by a tool (WAS) to facilitate the communication process.

Some of the more important community role players are the water users (farmers) and the water control officers. With a well-informed water control office, all water enquires, requests and information can be available to any water user at any time (Plate A10, Appendix A). The water users also understand the method and procedure of the WAS program with regard to the operation and management of the irrigation scheme. Better management procedures and the use of proper tools will ensure the sustainability of the water resource.

Attempts should therefore be made to utilise the WAS program as far as possible, as a powerful tool like this can only ease the management of a WUA. Future upgraded versions and applications of the WAS should be considered to complement the existing use of the WAS program as a management tool.

Various methods may be used to verify and check the data of the canal network but the following should be kept in mind:

- All data is in one way or another related (data are shared by the 4 modules) and the link between all data should be kept.
- Data can be simulated, evaluated and tested in the office, but it is the personnel at the irrigation scheme who will have the correct knowledge to refine and manipulate data most appropriately.
- Data should be updated to function correctly in field conditions and not only in office conditions.
- It should also be kept in mind that various small technical and debatable influences, like growth of unwanted vegetation and theft, could have an effect on the refining of data to be used. It depends on the researcher to eliminate all unnecessary data and to use only the data that are the most appropriate for verification processes.
- Various methods exist to verify the different data components. The most appropriate method for each type of data verification process should be identified and applied.

5.8 SUMMARY

If all data are verified and checked and satisfactory results are obtained, certain assumptions and recommendations can be made. It is these assumptions and recommendations, derived from the results, which should be implemented in the scheme in order to have a positive and lasting effect on the management of the irrigation office and of the water resources.

Chapter 6

Conclusions and Recommendations

CHAPTER 6 CONCLUSION AND RECOMMENDATIONS

6.1 CONCLUSION

As the study progressed it was realised that the major problem was to verify the data to be used by the WAS rather than to convince the community of the benefits to be gained by the use of the program. If the program can yield trustworthy results, convincing the community will be far easier.

Knowledge gained was mainly in the field of hydraulics and management of water and water resources. In studying hydraulics, gradually varied flow in an open canal was investigated. The Manning equation was also studied, together with how it could be specifically applied to open canal flow. The obvious result of working with and compiling a management tool like the WAS is that more streamlined managerial concepts could be addressed and utilised. The type of management tool that was derived is in line with what is needed by a WUA. The WUA requires appropriate assistance as it must deal with the water users who need a program that is user-friendly, punctual, powerful and accurate. The WAS program should be implemented in such a way that it will fulfill the needs of the users and the WUA for years to come. The WAS program will fulfill all the water utilisation needs of a WUA.

At the beginning of the project the first three modules were implemented in the scheme while the fourth, the water release module, was still not implemented. Presently, however, the VHWUA in Jan Kempdorp administers the Vaalharts irrigation scheme without all four modules of the Water Administration System, but it is anticipated that the final module can be fully implemented in the near future. The release module is implemented and in use at the moment. The fourth module has been implemented recently by conducting ongoing calibrations and validations. It is anticipated that all data on the program can be verified by the method proved in Chapter 4 and that the correct water release volume can be calculated on a continuous basis. As with all other similar computer-based managerial programs, a cycle of calibration is needed to evaluate and verify the performance of the system in operation.

6.2 RECOMMENDATIONS

The WAS has already been implemented on a number of other irrigation schemes in South Africa with satisfactory results (Chapter 3). Some schemes use the WAS only for accounts, while others use it for the administrational benefits. The intention is to implement the WAS fully at the VHWUA, making it the water management tool of the scheme. The VHWUA also lends itself to future developments of the WAS and water management. With the current water agreements between South Africa and Lesotho, and with the next phase in the Lesotho Highlands Water Project about to start, South Africa could have more water available for irrigation. This will also be in line with new proposed community projects where agriculture needs to be improved (WRC, 2000b). An adequate water supply is a much-needed commodity for any upcoming farmer (Cousens, 1998). Water management tools such as the WAS program will always be in demand to deal with and manage new irrigation schemes. Other possibilities include the scheduling of water usage (Benadé, Annandale and Jovanovic, 2002). There are certainly application possibilities for new or already established irrigation schemes that need a computer-based management tool. This document might thus be used as guide or as assistance in applying the program to other irrigation schemes that are also actively implementing water release modules, or similar applications. The checklist in Annexure B (Table B8) could be used as assistance.

Recommendations for future refinements of the WAS model at the VHWUA are as follows:

- Ongoing calibration as done in the study of the canal data and calculation procedure should take place until the users and developer are satisfied;
- Ongoing communication regarding WAS with the community and water control officers should take place (Crosby, Da Lange, Stimie and van der Stoep, 2000).
- As the VHWUA is a scheme owned by the users, regular meetings with the users should be initiated as the scheme can only improve with suggestions.
- Shown in Plate A7, Appendix A, various water losses exist on the scheme due to seepage. If it is necessary to reduce losses and improve the sustainable resource management (WRC, 2000a), a program should be launched to investigate, repair and maintain areas

where losses of this type occur. Proper management also includes combating the losses themselves.

- Implementing or incorporating a Global Positioning System phase using Geographic Information Systems would help considerably in locating sections on the canal, reading and using canal network data and pin-pointing certain sections on the canal, among other things. This is strongly recommended for new irrigation schemes (van Riet, Jansen van Rensburg, Dreyer and Slabbert, 1994);
- A proposed future water scheduling program could assist users and ensure the sustainability of the water resource, which would also lead to better planning and management of their water quota (Crosby, 1998);
- Records should be kept and/or indicated on a map regarding what section has already been verified on the canal layout. Fully updated files on all the verifications should be logged.
- Additional records should be kept and/or indicated on a map stating which area has received maintenance to the canal, as this might indicate certain tendencies on the canal network like leakages, vegetation growth and theft.
- The five methods applied to extract data from the scheme should still be used to verify suspicious data. More refinements such as easier means of data collection could also be added to the methods to ensure valid and easy-to-use data verification methods.
- Evaporation levels in the WAS calculation were set as they were used at the VHWUA, which were 6mm/day. The evaporation data obtained

from the weather bureau (Table B7, Appendix B) shows how the evaporation differs from month to month and year to year. It is suggested that the current method of determining a 5-day running average for evaporation level should still be used.

- Another study that could be conducted as a result of this work is the automation of the WAS. Possibilities exist to use the WAS to open and close sluices telemetrically after the release collation has been completed (Wolff-Piggott, 1995). Telemetric technologies can also be used to indicate to the user the moment severe losses are encountered at any point on the canal network.

6.3 APPLICATION TO OTHER IRRIGATION SCHEMES

Although the WAS program is widely used and the data given in this document could assist in the application of the same type of study on other schemes, it is recommended that the following be kept in mind:

- Not one irrigation scheme is the same as another.
- Methods applied might not be applicable to other situations, e.g. rivers.
- Management forms the basis for various types of studies and proper management should be applied to any attempted hydraulic study (Huges, Sami,and Murdock, 1993) as management principles will be derived from the work researched.
- The initial purpose of this type of tool is water release calculation, thus valid data should always be used. This can only be done if ongoing data verification and calibration is assured in any study.

6.4 SUMMARY

The WAS is most certainly a real-time possibility at the VHWUA as values can be calculated on a day to day basis or as required. It offers all the benefits and provides in the requirements of the management office. Once the program is in full use, it should be implemented on a continuous basis, i.e. the scheme should not revert back to the old method of calculation. The more the program can be run on a continuous basis, the more satisfactory and trustworthy results can be obtained. The WUA must use correct and updated data in their weekly calculations. The validity of information and results can only be seen with time, thus verification and error checking will provide results that are correct and that can be used for years to come.

Sustainable water resource utilisation can only be achieved through proper management (Pretorius, 2002). Applying the most effective management procedure will ensure a cost effective and optimised process at the VHWUA. As the WAS can definitely calculate the correct release volume of water into a canal or river system, it is the ideal package for water utilization management to implement in any irrigation board or water users' association.

References

References

REFERENCES

AYOADA, J.O. 1988. Tropical hydraulics and water resources (1st ed). London. Macmillan Publishers. pp 189–239

BENADÉ, N. 1993. Die ontwikkeling van 'n gerekenaliseerde waterverdelingstelsel vir die optimale bestuur van besproeiingskanaalstelsels. WRC Report No. 367/1/93.

BENADÉ, N. 1994. Geoptimiseerde lewering van besproeiingswater op aanvraag deur kanaalstelsels. Thesis (PhD). Stellenbosch, University of Stellenbosch.

BENADÉ, N. 2001a. WAS user's guide. Pretoria. NB Systems.

BENADÉ, N. 2001b. NB Systems [online] Pretoria, South Africa. Available from: < http://www.nbsystems.co.za/was/index.html> Accessed: August 2003

BENADÉ, N., ENGELBRECHT, R.J. & ANNANDALE, G.W. 1990. Die optimalisering van die bedryf van besproeiingskanaalstelsels. WRC Report No. 176/1/90. Water Research Commission, Pretoria.

BENADÉ, N., ANNANDALE, G.W. & JOVANOVIC, N.Z. 2002. The development of an integrated information system for irrigation water management systems for irrigation water management using WAS, SWB and RISKMAN computer models. WRC Report No. 946/1/02. Water Research Commission, Pretoria.

BENADÉ, N., ANNANDALE, G.W. & VAN ZIJL, H. 1997. The development of a computerized management system for irrigation schemes. WRC Report No. 513/1/97 pp 4, 7–9. Water Research Commission, Pretoria.

BRATER, E.F., KING, H.W., LINDELL, J.E. & WEI, C.Y. 1996. Handbook of hydraulics for the solution of hydraulic engineering problems (7th ed). New York. McGraw-Hill Book Company. p 8.12.

CHOW, V.T. 1959. Open-channel Hydraulics. Auckland. McGraw-Hill Book Company. pp 19–55, 222–226.

CHOW, V.T. 1988. Applied Hydrology (1st ed). New York. McGraw-Hill Book Company. pp 20–42.

COUSENS, D.W.H. 1998. Surface water resources of South Africa: Research needs. WRC Report No. AV 1/88. Water Research Commission, Pretoria.

CROSBY, C. T. 1998. SAPWAT 1.0—A Computer Program for Estimating Irrigation Requirements in South Africa. WRC Report No. 379/1/96. Water Research Commission, Pretoria.

CROSBY, C.T., DE LANGE, M., STIMIE, C.M. & VAN DER STOEP, I. 2000. A review of planning and design procedures applicable to small-scale farmer irrigation projects. WRC Report No. 578/2/00 pp 3–1 to 3–14. Water Research Commission, Pretoria.

DE LANGE, M. 1994. Small Scale Irrigation in South Africa. WRC Report No. 578/1/94. Water Research Commission, Pretoria.

DE LANGE, M. 2000. Improving Agricultural Water Management. 2000. WRC Report No. TT 127/00. Water Research Commission, Pretoria.

DEPARTMENT OF WATER AFFAIRS AND FORESTRY (DWAF). 1986. Management of the Water Resources of the Republic of South Africa. Pretoria: Government Printers.

DEPARTMENT OF WATER AFFAIRS AND FORESTRY (DWAF). 1975. Manual for Water Control. Pretoria: Government Printers.

DEPARTMENT OF WATER AFFAIRS AND FORESTRY (DWAF). 1996. Water Law Review Process. WRC Report No. TT 81/96.

FOOD AND AGRICULTURE ORGANISATIONS FOR THE UNITED NATIONS, 2006. Aquastat, [online] Rome, Italy. Available from: <http://www.fao.org/nr/water/aquastat/countries/south_africa/index.stm> Accessed: July 2006 and Nov 2007

GöRGENS, A., PEGRAM, G., UYS, M., GROBICKI, A., LOOTS, L., TANNER, A. & BENGU, R. 1998. Guidelines for Catchment management to achieve integrated water resource management in South Africa. WRC Report No. KV 108/98. Water Research Commission, Pretoria.

HAMILL, L. 1995. Understanding Hydraulics: A guide to the basic principles of hydraulics with an explanation of the essential theory. New York. St. Martin's Press. pp 109–136.

HAESTED PUBLICATION. 2003. Overview: Basic Hydraulic Principles. Current Methods, 1(1) pp 47–59.

HAUSER, B.A. 1996. Practical Hydraulics Handbook (2nd ed). Florida. Lewis Publishers. pp 81–85.

HENDERSON, F.M. 1966. Open Channel Flow. New York. Collier Macmillan Publishers. pp 66–75, 88–104.

HUGES, D.A., SAMI, K. & MURDOCK, K.A. 1993. Hydrological models–development and application. WRC Report No. 2/93. Water Research Commission, Pretoria.

HWANG, N.H.C., HOUGHTALEN, R.C. 1996. Fundamentals of hydraulic engineering systems. (3rd ed). Englewood Cliffs. Prentice Hall. pp 159–169.

INTERNATIONAL ASSOCIATION OF HYDROLOGICAL SCIENCES, IAHS. 2001. Integrated Water Resource Management. Oxfordshire. IAHS Press.

INTERNATIONAL WATER MANAGEMENT INSTITUTE [online] Battarmulla, Sri Lanka. Available from: < http://www.iwmi.cgiar.org/ > Accessed: July 2006

KANEN, J.D. 1986. Applied hydraulics for technology. New York. CBS Publishing Japan Ltd. pp 347–359.

MAYS, L.W. 1996. Water Resource Handbook (1st ed). New York. McGraw-Hill Book Company. pp 22.1–22.29.

MEYER, C.F. 1995. Water Engineering (1st ed). Pretoria. CFM Publications. pp 1–8, 206–209.

MOKWENA, M. E., FURUMELE, M. S. 2001. Water conservation and demand management strategy for the agricultural sector. Civil Engineer 9(5). pp 28–34.

NALLURI, C., FEATHERSTONE, R.E. 1995. Civil Engineering Hydraulics (3rd ed). Oxford. Blackwell Science, UK.

OSSIN, D. 1999. Water in the millennium, S.A. Irrigation, 21 (2), p17.

PRETORIUS, E. 1996. An investigation into the effect of various levels of sanitation on surface water quality in a typical developing community. Thesis (M.Tech). Bloemfontein, Technikon Free State.

PRETORIUS, E. 2002. The impact of socio-economic and human behavioral factors on the water of the fontein spruit catchment – A water management model study in a developing community. Thesis (PhD). Bloemfontein, University of the Free State.

SOUTH AFRICAN GOVERNMENT ONLINE. Government Communications, 1998. White Paper on a National Water Policy for South Africa. Pretoria, South Africa. Available from: <http://www.gov.za/whitepaper/1997/water.pdf> Accessed: October 2003

SOUTH AFRICAN GOVERNMENT, 2002. Government Gazette: Proposed national water resources strategy. Department of Water Affairs and Forestry.

SOUTH AFRICAN GOVERNMENT. 1998. National Water Act, Act 36 of 1998. Pretoria: Government Printers.

SA WATERBULLETIN. 2001. Agricultural management. SA Water bulletin, 27 (4), pp 4–5.

SIMON, A.L. 1986. Hydraulics (3rd ed). New York. John Wiley & Sons. pp 279–304.

TURNER, K.N., FOWLER, T.G., MANSON, N.J. & STEPHENSON, D. 1997. Optimization of Rand Water's distribution system. WRC Report No. 488/1/97. Water Research Commission, Pretoria.

VAN RIET, W.F., JANSEN VAN RENSBURG, J.D., DREYER, R. & SLABBERT, S. (1994) Geographic Information Systems (GIS) and the Integrated Environmental Management Procedure in the Planning and

Management of Water Resources. WRC Report No. 300/2/94. Water Research Commission, Pretoria.

WATER RESEARCH COMMISSION (WRC). 2000a. Improving agricultural water management. WRC Report No TT127/00. Pretoria. p 13. Water Research Commission, Pretoria.

WATER RESEARCH COMMISSION (WRC). 2000b. 2000 Technical Report. pp 97–101. Water Research Commission, Pretoria.

WOLFF-PIGGOTT, B. (1995) Demonstrating the potential of GIS Technology in hydro salinity Modeling through interfacing the DISA model and a GIS. WRC Report No. 588/1/95. Water Research Commission, Pretoria.

Appendix A

Appendix A

APPENDIX A

Picture A1: Warrenton Weir.

Picture A2: Jan Kempdorp Water Control Office.

Picture A3: Flood irrigation from community canal 5 F.

Picture A4: Distribution of flood irrigation water.

Picture A5: Sprinkler irrigation from community canal 5 G.

Picture A6: Centre pivot irrigation system.

Picture A7: Water losses due to seepage in community canal 5G.

Picture A8: Typical Feeder canal (Feeder 4).

Picture A9: Feerder dam on Feerder 14.

Picture A10: The water contol office available for the community.

Appendix B

Appendix B

APPENDIX B

Table B1: Extract from field data.

CANAL LAYOUT			Measure structure	Section	top	Top b	h Max
F4			crump	parabolic		2000	
	F4						
	4A		2 ft par	parabolic		1.55	0.85
		4A					
		4A1					
		4A2					
		4A3					
		4A4					
	4B		2ft par	parabolic		1.201	0.676
		4B					
		4B1					
		4B2					
		4B3					
		4B4					
		4B5					
		4B6					
	4C		2 ft par	parabolic		1.15	0.45
		4C					
		4C1					
		4C1 A:					
		4C1 B:					
		4C1 C:					
		4C4					
		4C5					
		4C6					
	4D		2 ft par	parabolic		1.201	0.676
		4D					
		4D1					
		4D2					
		4D3					
		4D4					
		4D5					
		4D6					

CANAL LAYOUT		Measure structure	Section	top	Top b	h Max
4E		2 ft par	parabolic		1.201	0.676
	4E					
	4E1					
	4E2					
	4E3					
	4E4					
	4E5					
	4E6					
4F		2 ft par	parabolic		1.201	0.676
	4F					
	4F1					
	4F2					
	4F3					
4G						
	4G					
	4G2					
	4G3					
	4G5					
	4G6					
4H4		2ft par	parabolic		1.05	0.9
4H						
	4H					
	4H5	2ft par	parabolic		0.9	0.4
	4H6					
4J		2ft par	parabolic		1.05	0.9
	4J					
	4J4					
	4J5					
	4J6					
	4J7					

Table B2: Normal Trapezoidal Canal.

Depth (m)	Bottom W (m)	Top W (m)	Area (m^2)	Wet perimeter (m)	Hydraulic R	Slope (1:x)	n	Q (m^3/s)	V (m/s)
0.1	5.6	5.853	0.573	5.9222	0.097	0.001	0.016	0.238	0.416
0.2	5.6	6.105	1.171	6.2443	0.187	0.001	0.016	0.758	0.647
0.3	5.6	6.358	1.794	6.5665	0.273	0.001	0.016	1.492	0.832
0.4	5.6	6.610	2.442	6.8886	0.355	0.001	0.016	2.418	0.990
0.5	5.6	6.863	3.116	7.2108	0.432	0.001	0.016	3.520	1.130
0.6	5.6	7.115	3.815	7.5330	0.506	0.001	0.016	4.790	1.256
0.7	5.6	7.368	4.539	7.8551	0.578	0.001	0.016	6.223	1.371
0.8	5.6	7.620	5.288	8.1773	0.647	0.001	0.016	7.816	1.478
0.9	5.6	7.873	6.063	8.4994	0.713	0.001	0.016	9.566	1.578
1.0	5.6	8.126	6.863	8.8216	0.778	0.001	0.016	11.473	1.672
1.1	5.6	8.378	7.688	9.1438	0.841	0.001	0.016	13.536	1.761
1.2	5.6	8.631	8.538	9.4659	0.902	0.001	0.016	15.754	1.845
1.3	5.6	8.883	9.414	9.7881	0.962	0.001	0.016	18.129	1.926
1.4	5.6	9.136	10.315	10.1102	1.020	0.001	0.016	20.661	2.003
1.5	5.6	9.388	11.241	10.4324	1.078	0.001	0.016	23.352	2.077
1.6	5.6	9.641	12.193	10.7546	1.134	0.001	0.016	26.201	2.149
1.7	5.6	9.894	13.169	11.0767	1.189	0.001	0.016	29.212	2.218
1.8	5.6	10.146	14.171	11.3989	1.243	0.001	0.016	32.384	2.285
1.9	5.6	10.399	15.199	11.7210	1.297	0.001	0.016	35.720	2.350
2.0	5.6	10.651	16.251	12.0432	1.349	0.001	0.016	39.222	2.413
2.1	5.6	10.904	17.329	12.3654	1.401	0.001	0.016	42.891	2.475
2.2	5.6	11.156	18.432	12.6875	1.453	0.001	0.016	46.728	2.535
2.3	5.6	11.409	19.560	13.0097	1.504	0.001	0.016	50.737	2.594
2.4	5.6	11.661	20.714	13.3318	1.554	0.001	0.016	54.918	2.651
2.5	5.6	11.914	21.893	13.6540	1.603	0.001	0.016	59.274	2.708
2.6	5.6	12.167	23.097	13.9762	1.653	0.001	0.016	63.807	2.763
2.7	5.6	12.419	24.326	14.2983	1.701	0.001	0.016	68.518	2.817
2.8	5.6	12.672	25.580	14.6205	1.750	0.001	0.016	73.409	2.870
2.9	5.6	12.924	26.860	14.9426	1.798	0.001	0.016	78.483	2.922
3.0	5.6	13.177	28.165	15.2648	1.845	0.001	0.016	83.741	2.973
3.1	5.6	13.429	29.496	15.5870	1.892	0.001	0.016	89.186	3.024
3.200	5.6	13.682	30.851	15.9091	1.939	0.001	0.016	94.820	3.073

Table B3: Combined Trapezoidal Canal.

Depth (m)	Bottom W (m)	Top W (m)	Area (m^2)	Wet perimeter (m)	Hydraulic R	Slope (1:x)	n	Q (m^3/s)	V (m/s)
0.1	5.6	5.717	0.566	5.8318	0.097	0.001	0.016	0.236	0.417
0.2	5.6	5.834	1.143	6.0635	0.189	0.001	0.016	0.743	0.650
0.3	5.6	5.951	1.733	6.2953	0.275	0.001	0.016	1.449	0.836
0.4	5.6	6.068	2.334	6.5270	0.358	0.001	0.016	2.323	0.996
0.5	5.6	6.186	2.946	6.7588	0.436	0.001	0.016	3.348	1.136
0.6	5.6	6.303	3.571	6.9906	0.511	0.001	0.016	4.510	1.263
0.7	5.6	6.420	4.207	7.2223	0.582	0.001	0.016	5.799	1.378
0.8	5.6	6.537	4.855	7.4541	0.651	0.001	0.016	7.209	1.485
0.9	5.6	6.654	5.514	7.6858	0.717	0.001	0.016	8.734	1.584
1.0	5.6	6.771	6.186	7.9176	0.781	0.001	0.016	10.370	1.676
1.1	5.6	6.888	6.868	8.1494	0.843	0.001	0.016	12.112	1.763
1.2	5.6	7.005	7.563	8.3811	0.902	0.001	0.016	13.959	1.846
1.3	5.6	7.122	8.269	8.6129	0.960	0.001	0.016	15.907	1.924
1.4	5.6	7.239	8.988	8.8446	1.016	0.001	0.016	17.954	1.998
1.5	5.6	7.357	9.717	9.0764	1.071	0.001	0.016	20.100	2.068
1.6	5.6	7.474	10.459	9.3082	1.124	0.001	0.016	22.342	2.136
1.7	5.6	7.591	11.212	9.5399	1.175	0.001	0.016	24.679	2.201
1.8	5.6	7.708	11.977	9.7717	1.226	0.001	0.016	27.111	2.264
1.9	5.6	7.825	12.754	10.0034	1.275	0.001	0.016	29.637	2.324
2.0	5.6	7.942	13.542	10.2352	1.323	0.001	0.016	32.257	2.382
2.1	5.6	8.059	14.342	10.4670	1.370	0.001	0.016	34.969	2.438
2.135	5.6	8.100	14.625	10.5481	1.386	0.001	0.016	35.940	2.457
0.1	10.35	10.667	15.6759	13.1731	1.190	0.001	0.016	34.792	2.219
0.2	10.35	10.984	16.7584	13.5481	1.237	0.001	0.016	38.167	2.277
0.3	10.35	11.302	17.8727	13.9231	1.284	0.001	0.016	41.723	2.334
0.4	10.35	11.619	19.0188	14.2981	1.330	0.001	0.016	45.464	2.390
0.5	10.35	11.936	20.1965	14.6731	1.376	0.001	0.016	49.392	2.446
0.6	10.35	12.253	21.4060	15.0481	1.423	0.001	0.016	53.512	2.500
0.7	10.35	12.571	22.6472	15.4231	1.468	0.001	0.016	57.826	2.553
0.8	10.35	12.888	23.9201	15.7981	1.514	0.001	0.016	62.337	2.606
0.9	10.35	13.205	25.2247	16.1731	1.560	0.001	0.016	67.050	2.658
0.993	10.35	13.500	26.4665	16.5219	1.602	0.001	0.016	**71.615**	2.706

Table B4: Data comparison.

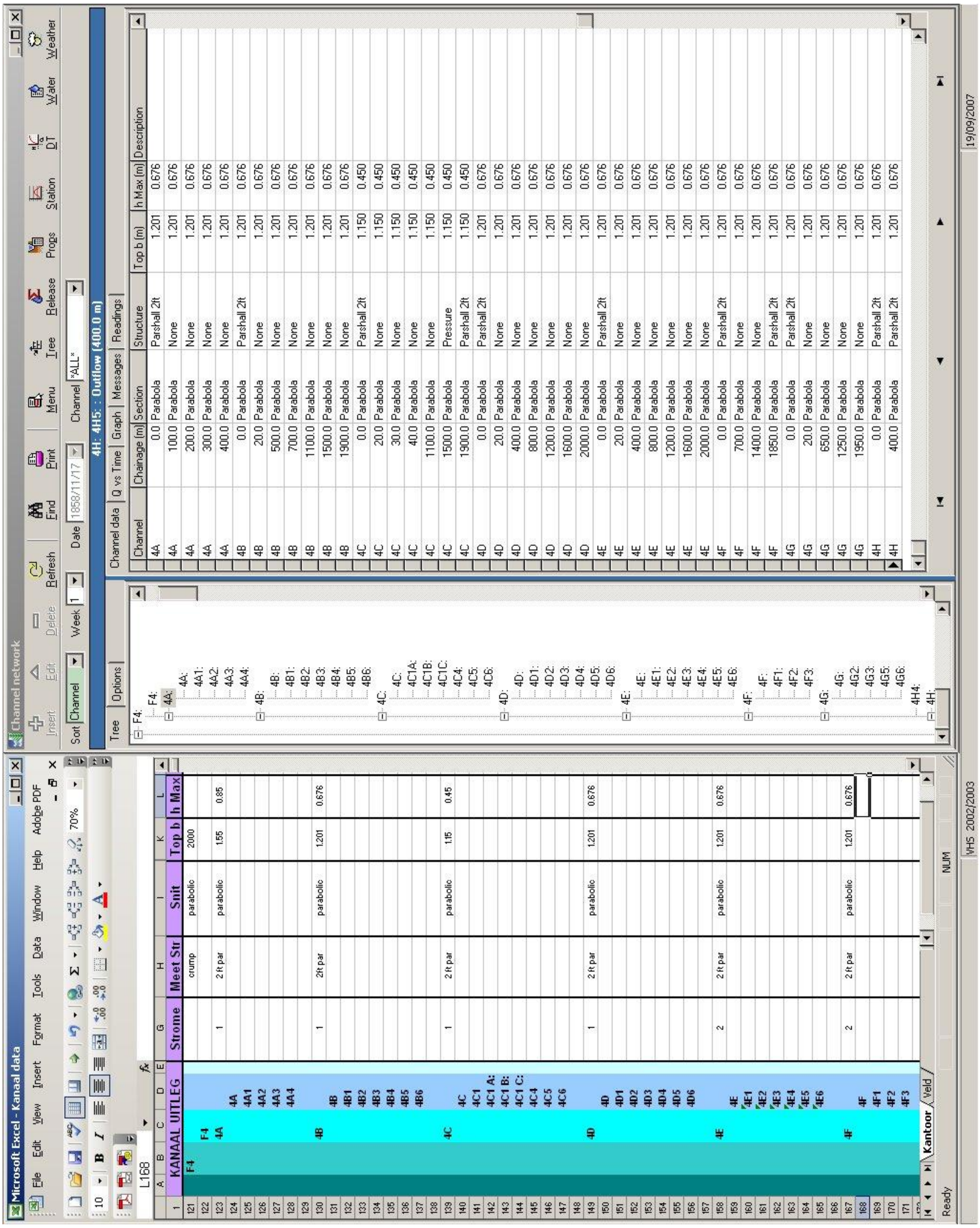


Table B5: Excel vs. WAS: Cycle 1 calculation comparison.

	30 2003/2004				14 2003/2004				46 2003/2004			
	EXCEL (m^3/s)	WAS (m^3/s)	diff	%diff	EXCEL (m^3/s)	WAS (m^3/s)	diff	%diff	EXCEL (m^3/s)	WAS (m^3/s)	diff	%diff
F1	558828	254404	-304424	54.5	191280	92075	-99205	51.9	657853	270739	-387114	58.8
F2	281280	165083	-116197	41.3	141452	86961	-54491	38.5	304646	182550	-122096	40.1
F4	359412	349588	-9824	2.7	223692	170397	-53295	23.8	418803	335289	-83514	19.9
F5	404620	510767	106147	-26.2	94536	100876	6340	-6.7	448999	554218	105219	-23.4
F6	391236	364331	-26905	6.9	145896	140148	-5748	3.9	447130	402806	-44324	9.9
F7	410604	442614	32010	-7.8	169704	147996	-21708	12.8	448150	492608	44458	-9.9
F8	641400	607396	-34004	5.3	261252	147570	-113682	43.5	724310	665143	-59167	8.2
F9	755112	698807	-56305	7.5	235164	201171	-33993	14.5	759980	671312	-88668	11.7
F10	819096	839579	20483	-2.5	404376	443422	39046	-9.7	861578	845788	-15790	1.8
F11	791088	816182	25094	-3.2	254280	266845	12565	-4.9	772465	780883	8418	-1.1
F12	65513	764581	699068	-1067.1	19494	253414	233920	-1200.0	762022	724525	-37497	4.9
F13	553092	601517	48425	-8.8	210192	255629	45437	-21.6	638838	666274	27436	-4.3
F14	748692	672686	-76006	10.2	284172	286107	1935	-0.7	792255	729864	-62391	7.9
F15	615096	661154	46058	-7.5	239724	306642	66918	-27.9	574816	680273	105457	-18.3
F16	600240	679255	79015	-13.2	236688	281763	45075	-19.0	682788	735984	53196	-7.8
F17	141540	94752	-46788	33.1	65652	51608	-14044	21.4	122400	77598	-44802	36.6
F24 - F27	1929408	950753	-978655	50.7	726876	259620	-467256	64.3	2119093	960855	-1158238	54.7
F24 - F25	887736		-887736	100.0	0		0	0.0	0		0	0.0
F24		279977				137572				361525		
F25		381897				110876				357454		
F26 - F27	1041672	950753	-90919	8.7	726876	259620	-467256	64.3	2119093	960855	-1158238	54.7
F26		295837				79741				296661		
F27		654916				179879				664194		

Table B6: Excel vs. WAS: Cycle 2 calculation comparison.

	30 2003/2004				14 2003/2004				46 2003/2004			
	EXCEL (m^3/s)	WAS (m^3/s)	diff	% diff	EXCEL (m^3/s)	WAS (m^3/s)	diff	% diff	EXCEL (m^3/s)	WAS (m^3/s)	diff	% diff
F1	558828	354204	-204624	36.6	191280	143825	-47455	24.8	657853	360847	-297006	45.1
F2	281280	193503	-87777	31.2	141452	117470	-23982	17.0	304646	204236	-100410	33.0
F4	359412	434887	75475	-21.0	223692	274094	50402	-22.5	418803	475495	56692	-13.5
F5	404620	726528	321908	-79.6	94536	197696	103160	-109.1	448999	731275	282276	-62.9
F6	391236	534709	143473	-36.7	145896	222683	76787	-52.6	447130	555486	108356	-24.2
F7	410604	542987	132383	-32.2	169704	272371	102667	-60.5	448150	597164	149014	-33.3
F8	641400	865920	224520	-35.0	261252	297713	36461	-14.0	724310	978322	254012	-35.1
F9	755112	1045221	290109	-38.4	235164	316965	81801	-34.8	759980	931656	171676	-22.6
F10	819096	93081	-726015	88.6	404376	578278	173902	-43.0	861578	1248081	386503	-44.9
F11	791088	1085828	294740	-37.3	254280	439482	185202	-72.8	772465	1121185	348720	-45.1
F12	65513	1039805	974292	-1487.2	19494	419088	399594	-2049.8	762022	998355	236333	-31.0
F13	553092	778826	225734	-40.8	210192	336470	126278	-60.1	638838	843567	204729	-32.0
F14	748692	889506	140814	-18.8	284172	441761	157589	-55.5	792255	958368	166113	-21.0
F15	615096	934788	319692	-52.0	239724	466582	226858	-94.6	574816	944995	370179	-64.4
F16	600240	902720	302480	-50.4	236688	454729	218041	-92.1	682788	988658	305870	-44.8
F17	141540	92777	-48763	34.5	65652	47207	-18445	28.1	122400	75477	-46923	38.3
F24 - F27	1929408	1811374	-118034	6.1	726876	1552914	826038	-113.6	2119093	1872715	-246378	11.6
F24 - F25	887736	765932	-121804	13.7	0	1257570	1257570	0.0	0	826761	826761	0.0
F24		338261				79602				402188		
F25		427671				1177968				424573		
F26 - F27	1041672	1045442	3770	-0.4	726876	295344	-431532	59.4	2119093	1045954	-1073139	50.6
F26		330185				109509				342732		
F27		715257				185835				703222		

Table B7: Evaporation data.

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DES
1968	348.3	285.9	155.7	125.8	107.6	91.9	126.9	171.9	225.1	273.9	283.2	304.6
1969	318.8	224.2	165.4	137.3	94.1	80.8	109.7	158.6	213.8	232.1	271.8	311.7
1970	322.9	263.5	246.0	184.9	128.3	93.9	108.1	155.5	207.4	251.3	276.7	258.0
1971	228.0	185.4	201.6	135.9	88.8	89.7	117.0	143.6	207.1	212.1	265.3	277.2
1972	199.3	207.9	124.0	117.2	103.3	97.4	122.3	145.9	203.1	265.8	275.5	361.9
1973	320.5	179.6	192.6	111.8	118.6	106.0	121.5	139.7	181.3	217.8	258.3	227.7
1974	192.5	156.3	145.5	97.1	104.6	90.9	124.9	122.3	197.4	224.9	218.7	269.7
1975	246.2	153.4	114.5	103.8	114.0	90.8	106.2	149.3	208.0	260.6	273.3	210.3
1976	161.4	192.1	137.6	122.7	**	97.5	114.1	153.2	179.7	204.6	295.3	292.7
1977	232.2	172.4	137.8	118.9	112.1	97.3	108.0	144.0	173.7	239.9	277.9	300.8
1978	242.7	189.3	168.0	115.8	116.7	92.2	115.1	148.8	161.0	248.7	298.0	300.5
1979	282.0	185.5	240.5	145.3	118.3	102.2	109.8	116.9	186.2	215.9	235.0	291.9
1980	290.5	208.6	168.6	155.8	148.0	104.2	129.0	164.9	178.7	274.5	252.5	279.2
1981	243.4	161.5	156.8	**	117.4	92.8	114.1	144.5	175.8	211.1	251.3	238.2
1982	290.8	**	183.6	**	**	87.6	92.6	159.9	206.6	227.5	266.2	277.5
1983	301.1	266.8	206.5	162.0	125.1	87.5	96.7	143.3	210.7	227.9	230.6	258.2
1984	306.6	281.5	194.3	127.6	106.9	82.3	108.0	137.2	194.0	188.3	248.4	300.2
1985	278.7	185.2	143.8	127.3	105.7	80.2	95.4	168.1	183.0	240.5	299.0	263.1
1986	269.9	210.2	188.0	158.5	144.1	109.2	116.6	158.5	186.1	220.0	244.6	339.0
1987	312.1	197.5	206.0	156.4	136.6	97.3	105.	148.5	137.5	238.6	258.7	286.7
AVE	269.4	205.6	173.8	133.6	116.1	93.6	112.0	147.2	190.8	233.8	264.0	282.5

Table B8: Generic Checklist.

CHECKLIST: Verifying canal details for water release calculation purposes

Irrigation Scheme/Board:			
Date:		Sheet	of
Complete by:			
Defects noted:			

Canal type (Main, Feeder, Community):			check #
Measuring Structure:		Reach	to
Type of Canal	Top width	Bottom width	Depth
	m	m	m
Slope:	Canal Material:		
Abstraction sluices:	Flow through Sluice:		

Canal type (Main, Feeder, Community):			check #
Measuring Structure:		Reach	to
Type of Canal	Top width	Bottom width	Depth
	m	m	m
Slope:	Canal Material:		
Abstraction sluices:	Flow through Sluice:		

Canal type (Main, Feeder, Community):			check #
Measuring Structure:		Reach	to
Type of Canal	Top width	Bottom width	Depth
	m	m	m
Slope:	Canal Material:		
Abstraction sluices:	Flow through Sluice:		

Canal type (Main, Feeder, Community):			check #
Measuring Structure:		Reach	to
Type of Canal	Top width	Bottom width	Depth
	m	m	m
Slope:	Canal Material:		
Abstraction sluices:	Flow through Sluice:		

Canal type (Main, Feeder, Community):			check #
Measuring Structure:		Reach	to
Type of Canal	Top width	Bottom width	Depth
	m	m	m
Slope:	Canal Material:		
Abstraction sluices:	Flow through Sluice:		

Canal type (Main, Feeder, Community):			check #
Measuring Structure:		Reach	to
Type of Canal	Top width	Bottom width	Depth
	m	m	m
Slope:	Canal Material:		
Abstraction sluices:	Flow through Sluice:		

Canal type (Main, Feeder, Community):			check #
Measuring Structure:		Reach	to
Type of Canal	Top width	Bottom width	Depth
	m	m	m
Slope:	Canal Material:		
Abstraction sluices:	Flow through Sluice:		

Uploaded to database:
Verified:

Appendix C

Appendix C

APPENDIX C

Verifying data for the implementation of the water release module of the WAS program

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Abstract

The Water Administration System (WAS) is designed to be a management tool for irrigation schemes and water control offices that want to manage their water accounts and supply water to clients through canal networks, pipelines and rivers. The ultimate aim of WAS is to optimise irrigation water management and minimise management-related distribution losses in irrigation canals. This research projects focused on the implementation of the water release module of the WAS program at the Vaalharts irrigation scheme. The WAS consists of four modules that are integrated into a single program that can be used on a single PC or a multi-user environment. The four modules are an administration module, a water release module, water accounts module and a water request module. The first three modules are already implemented at Vaalharts, while module four is implemented only partially. This module links with the water request module and calculated water releases for the main canal and all its branches allowing for lag times and any water losses and accruals. To precisely calculate this water release, accurate data is needed to ensure that the correct volume of water is released into the canal network. This can be done by verifying existing data with field data. To optimise the management of the irrigation scheme the fully implemented WAS program need to be installed and running at the scheme. A series of data and calculation verifications need to be executed. The exercise will show the adequacy and correctness of the available database WAS uses to do the release calculation from. This will ensure improved management of the irrigation scheme, catchment and water resource sustainability. It is planned that the information generated from this project will be used in the compilation of integrated catchment management information system, currently underway at the Central university of Technology, Free State, South Africa. It is for this reason that all data should be verified, as trustworthy results and service through management can then be offered to the community and irrigation area.

Keywords: water administration, irrigation, water distribution, canal network, water utilisation, agricultural, reach distances, lag time, discharge, water, demand management

Introduction

The slogan of the Israeli Water Directorate which states that '*...no man may waste a single drop of water that another man may turn into bread...*' could be applied to the current situation in South Africa. In general, South Africa is considered a water-scarce country (Ayoda, 1988). Water restrictions and the scarcity of water in South Africa have taught us to use water sparingly from an early age. The National Water Act, Act 36 of 1998 (South Africa, 1998a) also emphasises this issue of water usage and constantly refers to conservation control and equitable distribution of water (South Africa, 1998b). Conferences, symposiums and workshops therefore play a vital role in defining and planning sustainable resource management (Cousens, 1998). Water resources need to be controlled by a well-organised managerial body, which will form the basis for effective distribution of the resource.

According to Görgens et al. (1998) water resource management in South Africa has been transformed significantly as a result of the following two events:

- The democratisation of the Republic of South Africa
- The need for new approaches to water resource management due to misuse and mismanagement of available water resources.

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Simplifying the management process is important, as it could be a lengthy and difficult program. Supplying the controlling body with a management tool will increase the degree of conservation, control, distribution and efficiency of water resource use. The Water Administration System (WAS) has been implemented at the Vaalharts Waters User's Association (VHWUA) in order to manage water resources effectively. The WAS is designed as a management tool for irrigation schemes and water management offices to manage their accounts and to supply water to clients through canal networks, pipelines and rivers (Benadé, 2001). The WAS consists of four modules that are integrated into a single program that can be used on a single PC or a multi-user environment. The PC network system is currently in use at the VHUWA. The four modules can be implemented partially or as a whole, depending on the requirements of the specific scheme or office. The four modules are:

- An administration module
- A water request module
- A water accounts module
- A water release module.

The first three modules have been fully implemented at Vaalharts, while the water release module has been partially implemented. This module links with the water administration and water request modules and calculates water releases for the main canal and all its branches, allowing for lag times, water losses and accruals. The four modules can be implemented partially or as a whole, depending on the requirements of the specific

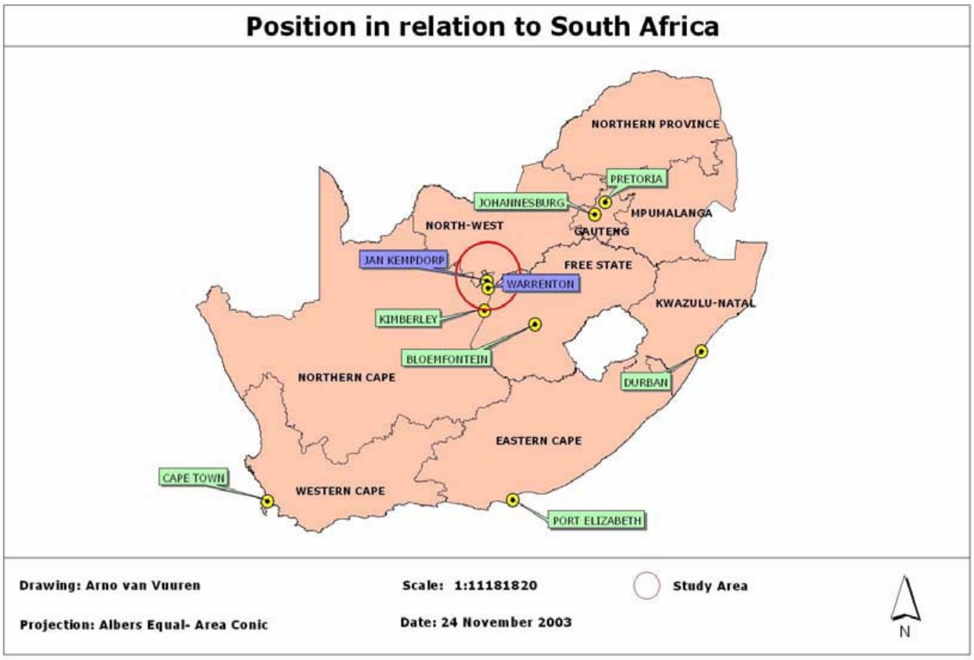

Figure 1
Vaalharts Irrigation Scheme

scheme or office (Benadé, 2001), although the Vaalharts Irrigation Scheme requires the full operation of the WAS program. This implies that the final module (water release module) must be fully implemented and calibrated to take over from the current means of water release calculation.

Methodology

The study was conducted in the VHWUA, which is located in the Warrenton/Jan Kempodorp/Hartswater district in the Northern Cape, South Africa. The head office in Jan Kempdorp, administers the VHWUA (Fig. 1). The main purpose of the study is to fully implement the WAS program and more specifically the water release module. In order to achieve the cycle of data verification, exercises need to be completed. In the process of capturing data, detail such as the cross-sectional properties, positioning of sluices and pumps, canal slopes, as well as canal capacities must be presented in a layout of the canal network. Once this has been done it is possible to realise the aims of the study. Irrigation scheme management will thus be optimised and the benefits can be applied from the managerial properties? Not clear what is meant of the WAS program.

All four of the WAS modules will contribute towards the successful completion. The WAS Program, as designed by N Benadé, bases all its calculations on the Manning flow equation (Benadé, 2001). It is evident thus far that a variety of geometrical information regarding the canal is needed to calculate the release. Since the main purpose of the project described in thus study is to implement the release module of the WAS program, it was crucial to verify all information and data, as all the calculations were based on this database. Four general methods to verify information and data were used, namely:

- Meetings/ consultations with farmers, technicians, etc.
- Engineering design drawings
- Mathematical calculations
- Collection of field data.

A 5th method was also applied. In this method calculated results were compared to one another in order to interpret the results and findings of calculations. This method is the most important as it will indicate the effectiveness and validity of the WAS calculation procedure.

Results and discussion

Meetings and consultations

On 27 August 2002 meetings and consultations were initiated with some of the water control officers and other key role-players of the community. A general planning meeting was held where various aspects of the WAS program were explained as well as the reasons why various implementations needed to take place. It was important to inform these key role-players, to give them a better understanding of the motivation for the project and to enable them to delegate instructions to subordinates.. At this meeting some important information was gathered. From this information decisive decisions could be made for further assumptions. The information included:

- The attitude of the community towards the use of the model
- Community's knowledge of the WAS program
- Areas where data should be collected
- Type of data to be collected
- Methods of data collection
- The best approach to be used with regard to the community.

Engineering design drawings

An alternative approach is to use the design drawings of the structural engineer. The data are compared with corresponding data on the canal layout on the computer. The latter method is the most applicable as initial design intentions are stipulated in the drawings. Specifically for this type of data set, engineering drawings proved to be a satisfactory means of data verifications. Data verified using this method proved to be correct and valid. Data that were previously unavailable were updated using this method.

Mathematical calculations

The objective of this exercise was to compare the flow discharge from the two canals and then to make assumptions regarding

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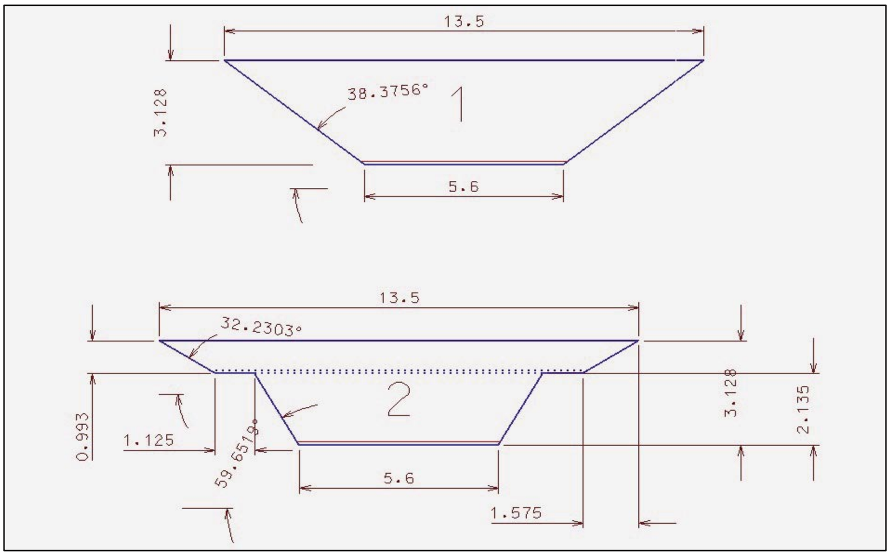

Figure 2
Comparing trapezoidal canals

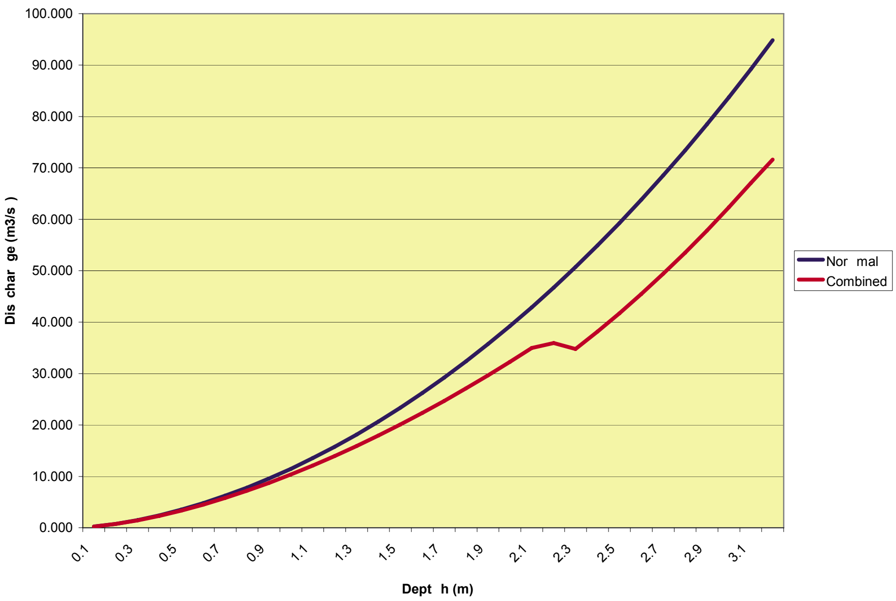

Figure 3
Mathematical comparison: d vs. Q

which canal section type to use in the WAS (Fig. 2). Using the Manning formula, cumulative values for discharge were calculated for both canals in each instance. The flow rate was calculated for increases in depth of 100 mm for each increment up to the full depth of 3.128 m. The user is now able to choose between the two canals according to scheme demands and other assumptions.

The combined trapezoidal canal delivers a discharge of 19.13 m^3/s less than the normal trapezoidal canal. If the combined canal is in operation at the scheme, but the normal canal shape is used as the basis for calculation, problems might occur. This exercise proves that the wrong volume of water is calculated. Figure 3 shows the comparison when calculating the discharge in relation to depth and comparing it to each other. The entire exercise regarding the mathematical comparison of the two canal sections proved to be a valuable input, particularly with regard to optimisations of the canal calculation procedure. One of the benefits of the WAS program is to improve water utilisation (Benadé, 2001).

Collection of field data

Satisfactory results were obtained by collecting data on site in the field and comparing these to existing data on the canal layout in the WAS program. The majority of data were collected using this method. Once again Community Canal 4A was used to illustrate the desired results. As discussed before, data from the canal layout was used to base the water release calculations on. It can clearly be seen that data collected in the field should correspond with the actual computer data.

Field work thus proved that the data for Community Canal 4A were different from those of the actual canal layout. This implies that data on the computer should be updated regularly so that only the correct and verified data are used. Applying this method randomly to other sections of the canal will result in a general idea of all the correct and incorrect data. The most desired scenario would be to verify each and every turnout on the scheme using this method. Due to time and other constraints, however, it was not possible in this study. Picture 1 shows some of the methods used to verify canal geometry and some of the

Picture 1
Structures to be verified for correctness

TABLE 1
Calculation results for Cycle 3

	Period 14		Period 30		Period 46	
	Description	**Volume**	**Description**	**Volume**	**Description**	**Volume**
VHWUA	Without losses	138 600	Without losses	241 200	Without losses	279 000
	With losses (Excel)	223 692	With losses (Excel)	359 412	With losses (Excel)	418 803
WAS	Without losses	138 600	Without losses	241 200	Without losses	279 000
	With losses (WAS)	232 968	With losses (WAS)	355 899	With losses (WAS)	397 506
	% diff- without losses	0.0	% diff- without losses	0.0	% diff- without losses	0.0
	% diff- with losses	4.1	% diff- with losses	-1.0	% diff- with losses	-5.1

structures to be verified. This proved to be valuable for further verification methods.

Excel vs. WAS comparison/evaluation

At the beginning of this chapter the hypothesis was stated that if all verification could be done successfully on a singe feeder, delivering satisfactory results and proving the calculation procedure correct, the same procedure could then be applied to all the other canals in the network. For this reason F4 was singled out and the 3rd cycle of calibrations and calculations was executed. Table 1 shows the calculation results after the 3rd cycle of calibrations. The percentage difference (as indicated in red) for the volumes without losses is 0% (calculation comparison test) as the same calibration values indicated reflect only the requested amount of water without any losses. This corresponds with the Excel values, thus proving them correct. Any differences still encountered indicate the difference.

Figure 4 graphically shows the difference between the WAS calculation results and the EXCEL calculation results. It may

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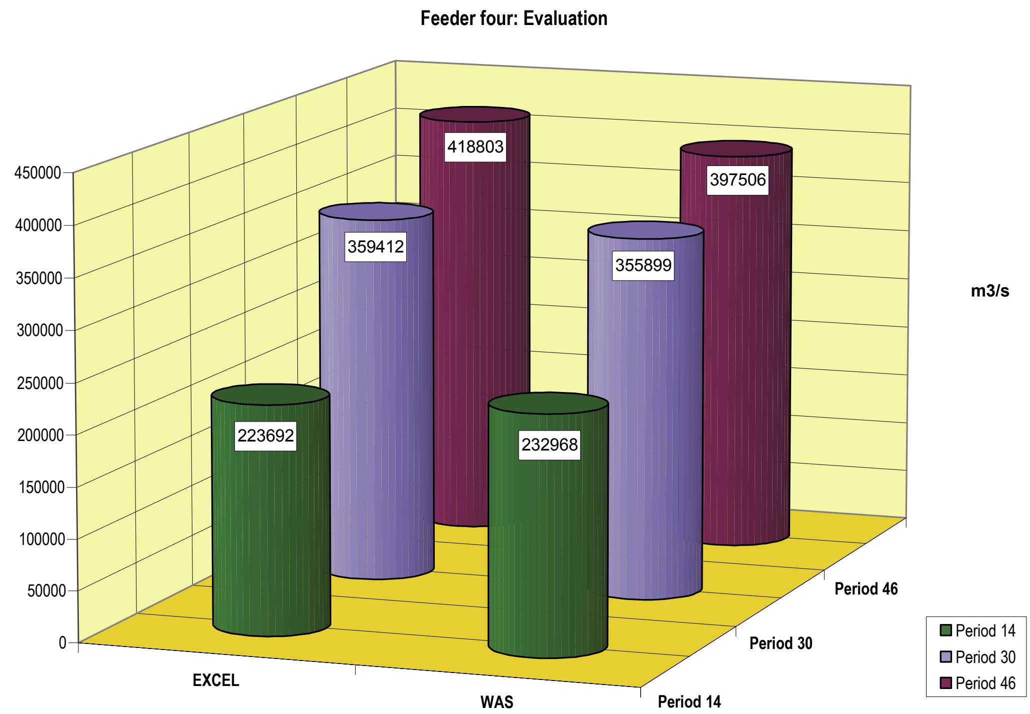

Figure 4
WAS vs. Excel: Final calibration results

therefore be concluded that all necessary data were updated and verified correctly as final calculations proved to correspond. The verified dataset can now be applied to the VHWUA. Since F4 was successfully validated, it may be assumed that the same method of validations and calibrations can be applied to any of the other feeder canals in the VHWUA. The rest of the canal network could therefore be built into this basis of data verification. By completing this exercise, trustworthy results can be obtained and the WAS program can be implemented fully.

Conclusion

At the beginning of the project the first three modules were implemented on the scheme while the 4th, the water release module, was still outstanding. Presently, however, Jan Kempdorp operates the Vaalharts Irrigation Scheme with all four modules but the release module is not in use at the moment Author: Please rephrase. Although the 4th module has not been completed due to ongoing calibrations and validations, it is anticipated that all data on the program can be verified and proved by the same method. The correct water release volume can therefore be calculated. As with all other similar computer-based managerial programs, a time of calibration is needed to evaluate and verify the performance of the system in operation.

The WAS has already been implemented on a number of other irrigation schemes in South Africa with satisfactory results. Some schemes use the WAS only for accounts, while others use it for administrator benefits. The intention is to implement the WAS fully at the VHWUA, making it the water management tool of the scheme. The VHWUA also lends itself to future developments of the WAS and water management. With the current water agreements between South Africa and Lesotho, and with the next phase in the Lesotho Highlands Water Project about to start, South Africa will definitely have more water available for irrigation. This will also be in line with new proposed community projects where agriculture needs to be improved (WRC, 2000). An adequate water supply is a much-needed commodity for any upcoming farmer. Certain assumptions can therefore be made that management tools such as the WAS program will always be in demand to deal with and manage new irrigation schemes. Other possibilities include the scheduling of water usage (Benadé et al., 2002). There are certainly application possibilities for new or already established irrigation schemes that need a computer-based management tool.

The WAS is most certainly a real-time possibility at the VHWUA. It offers all the benefits and provides in every need of the management office. Once the program is in full use, it should be implemented on a continuous basis, i.e. the scheme should not revert back to the old method of calculation. The more the program can be run on a continuous basis, the more satisfactory and trustworthy results can be obtained. The WUA must use correct and updated data in their weekly calculations. The validity of information and results can only be seen with time, thus verification and error checking will provide results that are correct and that can be used for years to come.

Sustainable water resource utilisation can only be achieved through proper management. Applying the most effective management procedure will ensure a cost effective and optimised process at the VHWUA. As the WAS can definitely calculate the correct release volume of water into a canal or river system, it is the ideal package to implement in any irrigation board or water users' association.

Reference

AYOADA JO (1988) *Tropical Hydraulics and Water Resources* (1st edn.). London. Macmillan Publishers. 189-239.

BENADÉ N, ANNANDALE GW and JOVANOVIC NZ (2002) The Development of an Integrated Information System for Irrigation Water Management Systems for Irrigation Water Management using WAS, SWB and RISKMAN Computer Models. WRC Report No. 946/1/02. Water Research Commission, Pretoria, South Africa.

COUSENS DWH (1998) Surface Water Resources of South Africa: Research Needs. WRC Report No. AV 1/88. Water Research Commission, Pretoria, South Africa.

GÖRGENS A, PEGRAM G, UYS M, GROBICKI A, LOOTS L, TANNER A and BENGU R (1998) Guidelines for Catchment

Management to Achieve Integrated Water Resource Management in South Africa. WRC Report No. KV 108/98. Water Research Commission, Pretoria, South Africa.

SOUTH AFRICAN GOVERNMENT ONLINE Government Communications (1998a) White Paper on a National Water Policy for South Africa. Pretoria, South Africa. Available from: http://www.gov.za/whitepaper/1997/water.pdf (Accessed October 2003).

SOUTH AFRICAN GOVERNMENT (1998) National Water Act, Act 36 of 1998b. Pretoria: Government Printers.

WATER RESEARCH COMMISSION (WRC) (2000) Improving Agricultural Water Management. WRC Report No TT 127/00. Pretoria. Water Research Commission, Pretoria.

Available on website http://www.wrc.org.za
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