Sustainability assessment of water resources systems

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PREFACE

The present report 'Sustainability assessment of water resources system' has been submitted as a part of the requirements for obtaining the Ph.D. degree at the Technical University of Denmark (DTU).

The study was carried out from February 1998 to September 2001 at the Department of Hydrodynamics and Water Resources (ISVA) now Environment & Resources DTU (E&R), Technical University of Denmark. Principal supervisor was Professor, dr. techn. Dan Rosbjerg with Dr. Jesper Knudsen, DHI Water & Environment as co-supervisor. I gratefully acknowledge their inspiration and supervision.

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The views expressed in this work are solely the responsibility of the author.

Kongens Lyngby, September 2001 Thomas Rodding Kjeldsen

Abstract

Despite incessant efforts by numerous research groups, still no universally agreed upon definitions of sustainability and sustainable development exist. Different policy options may lend themselves more or less to the underlying principles of sustainability, but no analytical tools are available for a more in-depth assessment of the degree of sustainability. The objective of this study is to develop such a practical tool for assessing the degree of sustainability of a water resources system. The methodology is building upon the water resources planning model MIKE BASIN developed by DHI Water & Environment. Two existing water resources systems situated in southern Africa were selected as case studies for testing of the methodology and the associated modelling system.

This thesis is divided into three main sections, a survey of available literature (chapter 2 and 3), development of methodology and associated modelling system (chapter 4 and 5) and, finally, two case studies from South Africa and Zimbabwe, respectively. **Chapter 2** is a general introduction to the concepts of sustainability and sustainable development. The main principles behind any sustainability assessment were identified. **Chapter 3** begins by discussing the concept of sustainability more specifically in relation to water resources systems. Different methodologies for assessing water resources systems sustainability were reviewed, with special emphasis on generic sustainability criteria. Especially, the extent to which these criteria encompass the essential elements was discussed. The conclusion of this literature review is that a new sustainability criterion containing two existing criteria will constitute a more balanced assessment tool. The first criterion is based on the three probabilistic system performance criteria reliability, resilience and vulnerability (R-R-V). The second criterion is an empirical measure of inter- and intra-generational fairness in allocation of impacts. In **Chapter 4** an operational version of the new sustainability criterion is developed. The practical applicability of R-R-V in a multi-criteria analysis as well as choice of appropriate estimators were investigated with respect to uniformity and overlap. These investigations were conducted through behaviour analysis of reservoirs using time series of historical and stochastically generated monthly runoff from eight different rivers. It was found that certain estimators of resilience and vulnerability are not defined uniformly with respect to increasing (or decreasing) water resources stress, measured in terms of water demand or reservoir storage, when estimation is based on time series of historical extension due to the low number of failure events. The problem was partly solved by using stochastic models for generation of time series of monthly runoff with an extension of 1000 years. The overlap between estimators of R-R-V was investigated by calculating the correlation between pairs of sample estimates from 100 reservoir simulations generated by stochastic models. It was found that pairs of estimators based on similar summarising statistics are almost completely overlapping, a fact that should be taken into account when R-R-V criteria are used in practise. Certain aspects of a complete sustainability assessment could not be related to water demand and, therefore, not evaluated in terms of R-R-V. Consequently, the final sustainability criterion (the relative sustainability) was defined as a combination of the criterion described above and a second criterion derived from the outcome of an environmental impact assessment conducted using the rapid impact assessment matrix (RIAM) tool. **Chapter 5** describes a modelling system consisting of the MIKE BASIN model coupled with a multivariate stochastic streamflow model. The stochastic model is based on a method of non-parametric disaggregation of annual runoff into monthly runoff. The modelling system is used for estimation of R-R-V in different time periods for the considered complex water resources systems. **Chapter 6** and **Chapter 7** contain descriptions and analyses of two existing surface water resources systems, a system encompassing the Mgeni and Mkomazi catchments in KwaZulu-Natal, South Africa and a system in the Mupfure catchment situated in central Zimbabwe. The modelling system developed in chapter 5 was setup for each of the two considered water resources systems and a RIAM investigation conducted. The existence of entire years with no observed runoff from several sub-catchments within the Mupfure catchment necessitated the development of an appropriate adjustment procedure of the multisite stochastic model. Through the analysis of multiple scenarios exposing different combinations of demand management and capacity expansion, reservoir construction was found to be the option with the highest degree of relative sustainability. This is conditional upon construction in an area where the negative impacts are minimised. Demand management proved to be less significant in terms of relative sustainability when considering the entire water resources systems. **Chapter 8** contains a final discussion and conclusion to the results obtained in the study. The usefulness of the developed methodology is discussed based on both the theoretical properties of the generic criterion and the results obtained through the two case studies. The final criterion appears as a strongly aggregated measure of sustainability, and caution is required when applied to an existing water resources system. Finally, unresolved issues and ideas for further research are listed.

RESUMÉ

BÆREDYGTIGHEDSVURDERING AF VANDRESSOURCE SYSTEMER

På trods af de store anstrengelser mange forskellige forskningsgrupper har gjort sig eksisterer der til dags dato stadig ingen universelle definitioner af begreberne bæredygtighed og bæredygtig udvikling. Forskellige forvaltningsprincipper må antages i større eller mindre udstrækning at leve op til det ideelle krav om bæredygtighed, men det der mangler er analyseredskaber til en nærmere bestemmelse af bæredygtighedsgraden. Formålet med denne afhandling er at udvikle en praktisk anvendelig metode til bestemmelse af graden af bæredygtighed af et vandressourcesystem. Metoden er opstillet til at kunne anvendes sammen med en eksisterende vandressourceforvaltningsmodel MIKE BASIN udviklet af DHI Water & Environment. To eksisterende vandressourcesystemer i det sydlige Afrika blev udvalgt som case studier for afprøvning af den opstillede metode med tilhørende modelsystem.

Afhandlingen er opdelt i tre hovedafsnit: en litteraturgennemgang (kapitel 2 og 3), udvikling af metode og tilhørende modelsystem (kapitel 4 og 5) og til sidst to case studier i hhv. Sydafrika og Zimbabwe (kapitel 6 og 7). **Kapitel 2** er en generel introduktion til begreberne bæredygtighed og bæredygtig udvikling . Hovedelementerne i en vurdering af graden af bæredygtighed er identificeret. **Kapitel 3** indleder med at diskutere begrebet bæredygtighed set mere specifikt i relation til vandressourcesystemer. Herefter følger en vurdering af forskellige metodiske tilgange til bæredygtighedsvurderinger, specielt med fokus på universelle bæredygtighedskriterier og i hvilken grad de inkluderer de essentielle elementer for en vurdering af et vandressourcesystem. Konklusionen på denne litteraturgennemgang er, at et nyt bæredygtighedskriterium bestående af elementer fra to eksisterende kriterier vil repræsentere et mere fuldstændigt kriterium. De to metoder er hhv. et kriterium, der bygger på de tre sandsynlighedsteoretiske systemevalueringskriterier pålidelighed (reliability), evnen til at returnere fra en fejlperiode til en acceptabel systemtilstand (resilience) og sårbarhed (vulnerability) (forkortet R-R-V), samt et empirisk mål for en retfærdig ressourceallokering (fairness) indenfor og imellem generationer. I **kapitel 4** er en operationel version af det førnævnte nye bæredygtighedskriterium udviklet. Den praktiske anvendelsen af R-R-V i en multiobjektiv analyse samt valg af estimatorer er undersøgt mht. egenskaber relateret til entydighed og overlapning. Disse undersøgelser blev foretaget vha. reservoirsimulering og tidsrækkeanalyse af både historiske og stokastisk genererede månedlige afstrømninger fra otte forskellige floder. Det blev vist, at visse estimatorer af resilience og vulnerability ikke er entydigt bestemt, når der anvendes tidsrækker af historisk længde pga. et lavt antal fejlperioder. Problemet er delvist løst ved at anvende stokastiske modeller estimeret ud fra de historiske data til at generere tidsrækker af en længde på 1000 år. Overlappet imellem de forskellige estimatorer af R-R-V blev undersøgt ved at beregne korrelationen mellem par af estimater estimeret ud fra 100 reservoirsimuleringer genereret vha. de stokastiske modeller. Ingen optimal kombination kunne udpeges, men estimatorer af hhv. resilience og vulnerability baseret på samme estimeringsmetode har næsten fuldstændigt overlap og bør derfor ses i lyset heraf. Visse aspekter af en komplet bæredygtighedsvurdering kan ikke umiddelbart tillægges et vandforbrug og derfor ikke evalueres baseret på R-R-V. Derfor blev det endelige bæredygtighedskriterium defineret som en kombination af førnævnte R-R-V kriterium og et kriterium beregnet ud fra resultaterne fra en environmental impact analysis (EIA analyse) udført vha. rapid impact assessment matrix (RIAM) metoden. **Kapitel 5** beskriver et modelsystem bestående af MIKE BASIN modellen koblet med en multidimensional stokastisk tidsrække model. Den stokastiske tidsrække model er baseret på nedskalering af årlig afstrømning til månedlig afstrømning vha. en parameterfri metode. Modelsystemet skal bruges ved analyse af komplekse vandressourcesystemer og til estimering af R-R-V kriteriet over en række specificerede tidsperioder. **Kapitel 6** og **kapitel 7** indeholder beskrivelser og analyser af to eksisterende vandressourcesystemer i hhv. et system bestående af Mgeni og Mkomazi flodernes oplandene i KwaZulu-Natal provinsen i Sydafrika og Mupfure flodens opland beliggende i det centrale Zimbabwe. Det i kapitel 5 udviklede modelsystem blev sat op for hvert af de to systemer og en tilhørende RIAM undersøgelse gennemført. Eksistensen af hele år uden observeret afstrømning i visse deloplande i Mupfure oplandet nødvendiggjorde en modifikation til den eksisterende stokastiske model. Ved at analysere en række mulige fremtidsscenarioer bestående af kombinationer af forbrugskontrol og udvidelse af en eksisterende vandforsyningsinfrastruktur blev det i begge tilfælde konkluderet, at opførelsen af et nyt overfladevandsreservoir er at betragte som den mest bæredygtige mulighed, såfremt det sker i et område, hvor de negative konsekvenser kan minimeres. Forbrugskontrol viste sig at være af mindre betydning når hele vandressourcesystemet blev inddraget i analysen. **Kapitel 8** indeholder en diskussion og konklusion på de opnåede resultater. Den udviklede metodes brugbarhed diskuteres baseret på både de teoretiske egenskaber og de resultater opnåede igennem de to case studier. Det endelige kriterium fremkommer igennem en høj grad af summation af data og bør anvendes med forsigtighed i forbindelse med analyse af eksisterende vandressourcesystemer. Afhandlingen afsluttes med en liste af uafklarede emner og ideer til fortsat forskning inden for emnet.

CONTENTS

Chapter 1
INTRODUCTION

Sustainable development has been high on the international agenda since the publication of the report *Our Common Future* in 1987 (WCED, 1987). The report, which stresses the need to balance current development of human civilisation with the needs of the environment and future generations, has initiated intense discussions on how to achieve and measure progress in the direction of sustainable development.

Water is a vital resource for health, development, food security and for the entire ecological system and, hence, of direct importance to sustainable development (Falkenmark, 1997). Africa, and especially Southern Africa, has often been put forward as a region facing severe problems in terms of water resources, especially concerning extreme hydrological events (droughts and floods) and population growth, see, for example, Falkenmark (1989), World Bank (1996), Sehmi & Kundzewicz (1997), Kjeldsen et al. (1999a), Kjeldsen et al. (2000) and Kjeldsen et al. (2001; 2002). These factors have triggered conflicts over scarce resources (Kjeldsen et al., 1999b), and the need for a more holistic and sustainable utilisation of available resources is clearly evident as is the need for appropriate tools to assist decision-makers in this process. In its policy paper on water resources management the World Bank (1993) stresses the need for practical policy analysis tools for a holistic consideration of the ecosystem and socio-economic activities within a river basin. A number of research projects has been initiated by various research groups (Simonovic, 1997) attempting to define and develop practical methods for assessing the degree of sustainability of water resources development options. However, progress has been moderate and very little has been published in terms of case studies and further development of existing first generation methods.

The objective of this research project is to gain insight into sustainable development in relation to the management of water resources and to develop a generic methodology for assessing the contribution towards sustainable development (sustainability) by water resources projects. The methodology should be used to distinguish between a number of development scenarios in terms of sustainability, and will be tested on two case studies of existing water resource systems in South Africa and Zimbabwe. Both water resources systems support several user groups and have previously been impacted by severe droughts. Moreover, both systems are planned extended by the construction of large reservoirs to prepare for anticipated future water scarcity.

This thesis consists of three major parts: a literature review (chapter 2 and 3), development of a methodology and a corresponding modelling system (chapter 4 and 5) and, finally, two case studies from South Africa and Zimbabwe, respectively (chapter 6 and 7).

Chapter 2 is an introduction to the general concept of sustainable development and sustainability. The key components defining the terms will be discussed and a checklist of issues that need to be addressed by a sustainability assessment method will be outlined.

Chapter 3 discusses sustainability in a water resources context. The chapter begins with a discussion of the issues of importance to sustainability and water resources and is followed by a short review of the application of the concept of system analysis to water resources development projects. The different methods of assessing sustainability, and especially the proposed sustainability criteria, are reviewed based on the findings of chapter 2 and the beginning of this chapter.

Chapter 4 presents a unified framework for assessing and comparing the sustainability of different water resources system development projects. The framework is based on the traditional system analysis decision-making procedure and combines two existing sustainability criteria related to fairness and to the risk criteria reliability, resilience and vulnerability (R-R-V). This new criterion was coupled with an impact assessment method known as the Rapid Impact Assessment Matrix (RIAM) to obtain a more holistic evaluation. The statistical properties of R-R-V are investigated using a combination of historical and synthetic runoff series generated using stochastic models to identify the best combination of estimators.

Chapter 5 describes the model system developed to assist in the sustainability assessment. An existing water resources planning model, MIKE BASIN, has been coupled with a multivariate time series model for generation of stochastic monthly runoff sequences using a CARMA(p,q) model for annual runoff and the method of fragments for disaggregating annual runoff into monthly runoff. The time series generation module enhances the ability of the MIKE BASIN model to perform risk assessments of water resources systems.

Chapter 6 presents a case study from the KwaZulu-Natal province, South Africa. The Mgeni River is the main source of water for the metropolitan areas of Durban and Pietermaritzburg and for commercial agriculture. Due to expected future water scarcity, interbasin transfer from the neighbouring Mkomazi River through the construction of major infrastructure projects is currently being considered along with other options such as extension of existing storage facilities and demand management. The newly adopted water law of South Africa requires explicit considerations of the environmental demand for water, which is relevant for any new development, especially major reservoirs.

Chapter 7 presents a case study from the Mupfure catchment, Zimbabwe. The Mupfure River supplies water mainly to commercial agriculture and, to a lesser degree, urban areas. Due to the prolonged droughts experienced in Zimbabwe at the end of the previous millennium a number of conflicts over the right to the existing water resources has occurred. To alleviate the problems of drought and over exploitation of water resources in the catchment, the construction of large-scale reservoirs on the Mupfure catchment is being considered.

In both case studies the construction of large surface reservoirs is considered as possible solution to the problems of anticipated water scarcity and shortage, as it is considered a solution in many other cases in the area as well. The construction of large reservoirs is always highly controversial due to the magnitude and multitude of impacts on the connected environmental, social and economic subsystem as highlighted by, for example, Takeuchi et al. (1998) and WCD (2000). Hence, these two case studies might provide an insight useful for the analysis of other cases.

Chapter 8 contains a final discussion and conclusion to the results obtained in the study. The usefulness of the developed methodology is discussed based on both the theoretical properties of the generic criterion and the results obtained through the two case studies. The final criterion appears as a strongly aggregated measure of sustainability, and caution is required when applied to an existing water resources system. Finally, unresolved issues and ideas for further research are listed.

Chapter 2

THE CONCEPT OF SUSTAINABLE DEVELOPMENT

This section constitutes an introduction and background to the term sustainable development followed by a discussion of the conceptual elements important for a general sustainability assessment method.

2.1 BACKGROUND

A growing awareness of global and local environmental problems and the need to balance the development of the human civilisation with the need of the life supporting environmental system led to the publication of the Brundtland report (WCED, 1987). Since then the concept of sustainable development has been accepted within a wide range of public and private sectors as being the ultimate goal to plan and manage for. The UN conference on Environment and Development in Rio Janeiro (UNCED, 1992) further developed the idea of sustainable development and presented recommendations for sustainable development in the future (Agenda 21). UNESCO (1999) states that the real value of the Rio Conference and other related conferences and publications "*was not that they led to actions that saved our planet. Their value may well be that they changed the way many view the environment and ecosystems as they worked to advance economic development and equity*" and some researchers argue that the concept is becoming a new paradigm for science (Haimes, 1992; Schultz, 1998). As a result of the popularity and widespread, non-critical use of the term, it is in real danger of loosing its significance. Kundzewicz (1999) states that the term often is "*being used and abused as a convenient argument to justify or criticise actions (or lack of actions), decisions etc*". The general acceptance of sustainable development has initiated research towards the development of methods for assessing whether development can be considered sustainable or not, see for example, Raskin et al. (1996), Simonovic et al. (1997) and UNESCO (1999), or, in other words, the search for a "*metrics of sustainable development*" (Baetz & Korol, 1994). However, as noted by Hersh (1999) "*progress has been moderate to date*".

2.2 DEFINITIONS

Despite, or because of, an abundant amount of conferences and literature there is still no consensus on the precise definition on what exactly sustainability means, what it implies, how to measure it and, especially, how to reach it. This, perhaps, is best illustrated by the fact that Pezzy (1992) (cited by Kundzewicz, 1999) was able to collect more than 190 definitions of the term sustainable development. A few commonly applied definitions are shown in Table 2.1. The most generally used definition of sustainable development originates from the Brundtland Report (WCED, 1987):

”*Humanity has the ability to make development sustainable – to ensure that it meets the needs of the present without compromising the ability of future generations to meet their own needs.*”

This definition emphasises the importance of explicitly considering the needs of the future generations, and thereby advocating a holistic approach stressing fairness when considering multiple economic, environmental and social objectives (UNESCO, 1999).

Table 2.1: Definitions of sustainability.

Source	Definition
WCED (1987)	Humanity has the ability to make development sustainable – to ensure that it meets the needs of the present without compromising the ability of future generations to meet their own needs.
Shamir (1996)	Making decisions that minimize the probability of future regret.
Raskin et al. (1996)	Reconciling the objectives of socio-economic development, environmental quality and ecosystem preservation into a resilient foundation for the future.
Kundzewicz (1999)	Improving the quality of human life (attaining non-decreasing human welfare over time) within the carrying capacity of the ecosystem.

Most definitions highlight the need for a multiobjective approach to development, recognising the interrelationship between social, economic and environmental subsystems. The total welfare derived from all the systems should ideally increase over time. The definition proposed by Shamir (1996) in Table 2.1 is a reminder that the recognition of risk and uncertainties is important when discussing sustainability.

2.3 CONCEPTUAL COMPONENTS OF SUSTAINABLE DEVELOPMENT

Takeuchi et al. (1998) defined need, generations and fairness as the three key conceptual notions that should be addressed when considering sustainable development.

Needs

Estimating the needs of the present can be very difficult due to e.g. lack of data, difficulties in perception (differences in what A and B think A needs), unknown factors (e.g. the need of an undiscovered population of frogs). The needs of the future generations can, at best, only be estimated by extrapolation from current available knowledge introducing additional uncertainty into an analysis. Falkenmark (1997) argues that backcasting from a sustainable future is more interesting than forecasting from a non-sustainable future. However, due to difficulties in defining just what a sustainable future implies this might be difficult in practise.

Fairness

Klauer (1999) argues that intra-generational fairness is the most important characteristic of sustainable development. When abundant resources are available for all purposes at all times management of natural resources is trivial. As soon as a resource becomes scarce, however, trade-offs between different needs have to be made with respect to both inter-and intra-generational needs, calling for both inter and intra-generational fairness.

Future generations

Sustainable development requires explicit consideration of the well-being of the future generations. However, no unambiguous definition of the extent of the required temporal planning horizon is available. Does the present generation need to consider all generations into the infinite future or only the next generation, which then consider the following generation etc.?

No unequivocal definitions or methods for considering these three conceptual components have been agreed upon, but a method claiming to assess sustainability should, at least, attempt to include them into the analysis.

2.4 MULTIOBJECTIVE DECISION-MAKING

The use of multicriteria decision methods for meaningful consideration of multiple stakeholders with multiple and often conflicting goals and preferences is important when discussing sustainable decision-making as stressed by, for example, Takeuchi et al. (1998) and Hersh (1999). Rohde & Rouvé (1977) distinguished between methods that aim at presenting a comprehensive array of relevant data to the decision-makers and methods that attempt to aggregate all available information into one common metric. Rohde & Rouvé (1977) argued in favour of the last method. Thompson (1990), on the other hand, argued that too much aggregation of information makes the decision-making non-transparent. The two approaches appear suitable for different circumstances. In a project evaluation attended by various stakeholders not familiar with the evaluation methodology, a transparent method is needed. For an in-house evaluation carried out by and for involved professionals familiar with the methodology, a single number may be sufficient. Often weighting schemes are applied to express preferences for each subcriterion in a multiobjective decision-making model.

Given the broad nature of the term sustainable development, it might be difficult to separate a sustainable development from one that is not, perhaps with the extreme and obvious cases of non-sustainable development, such as the destruction of the Aral Sea, as an exception (Kundzewicz, 1999). Therefore, the notion of relative sustainability (Loucks, 1997) may be more useful for comparing different development options.

2.5 ASSESSMENT OF SUSTAINABLE DEVELOPMENT

When considering a general framework for assessing sustainable development it is important to consider the following issues and include them in the analysis:

- Need
- Fairness
- Future generations
- Ability to include multiple criteria such as economy, environment and social issues

These issues will be used in the next chapter, together with other issues related specifically to water resources systems, to select the most appropriate methods for assessing water resources system sustainability. It has been argued that the type of methodology chosen for evaluation of sustainability depends on the objective of the exercise. It is the ambition of this project to develop a single criterion for expressing the degree of sustainability, aiming at a methodology with a high degree of information-aggregation.

Chapter 3
WATER RESOURCES SYSTEM SUSTAINABILITY

According to Rogers et al. (1997) the concept of sustainability has little relevance when "*applied to the level of a single resource or even a single nation over a short time span*." Despite this statement, several research groups such as Jordaan et al. (1993), Baan (1994), Duckstein & Parent (1994), Loucks (1994), Shamir (1996), Loucks (1997), Simonovic (1997), Simonovic et al. (1997), Hornbogen (1998), Takeuchi et al. (1998), UNESCO (1999), Makoni et al. (2001) and Nachtnebel (2002) have attempted to define and develop methodologies for assessing the sustainability of water resources systems and projects. Common for all the above-mentioned studies is the use of system analysis to quantify sustainability criteria and to assist in the decision-making procedure.

This chapter reviews the proposed methodologies and, in particular, the proposed sustainability criteria for water resources systems, which will be evaluated against the issues a potential sustainability criterion should possess. The review will lead to a decision-making procedure for sustainable water resources management, merging different methodologies into one united framework based on a system analysis approach.

3.1 DEFINITION OF WATER RESOURCES SYSTEM SUSTAINABILITY

There are a number of important issues that needs to be considered when trying to define water resources system sustainability in addition to those belonging to the general framework as identified in the previous chapter. According to Loucks (2000) the most important issues are change, scale, technology, risk and training. The first four issues are of direct importance to methodologies for assessing sustainability and, hence, will be further discussed.

Change

Most environmental systems are non-stationary, i.e. conditions change over the course of time. As the natural, economic, environmental and social subsystems are interrelated, change in one system will have an effect on the other systems and, hence, have an effect on the entire system. Also, the management objectives might change over time. Guesses about the future will almost certainly be wrong and therefore need to be revisited frequently. Adaptive management was introduced in natural resources management as a means to cope with dynamic systems (Holling, 1978 referred in Loucks, 2000). According to Loucks (2000) "*Adaptive management is a process of adjusting management actions and directions, as appropriate, in the light of new information on the* *current and likely future conditions of our total environment and our progress towards meeting our goals and objectives*". A method for assessing sustainability should be able to work within an adaptive management framework.

Scale

The appropriate scale, both temporal and spatial, for assessing sustainability needs to be considered. The temporal scale requires considerations of both the planning horizon and the duration of the time steps applied in the analysis. The need of the future generations is the pivot of the sustainability term, although no operational guidelines as to how many generations into the future should be considered can be found in the literature. The choice of appropriate duration of time steps has to take the variability of the natural water supply into account. Extreme events, such as droughts and floods, are naturally occurring in the hydrological cycle, but they can temporarily jeopardise efforts to achieve sustainable development (Kundzewicz, 1999). Therefore, the assessment of water resources system sustainability should adopt a time scale making the inclusion of extreme events possible. According to Loucks (1997) the duration of the time steps used for the sustainability analysis should "*be such that natural variation in a resource, like water, are averaged out over the period*". Mathson et al. (1997) recommended using time steps of 1-5 years and applied a planning horizon of 40 years in a sustainability analysis of different power supply systems. Loucks (1997) adopted a planning horizon of 50 years.

Conflicting views on the appropriate spatial scale for assessing sustainability are evident in the literature. Roger et al. (1997) state that sustainability cannot be properly analysed when applied to a spatial scale limited to a single nation. On the other hand, much effort has been devoted to the development of sustainability assessment methods at project level, see, for example, Baan (1994), Loucks (1997) and Simonovic et al. (1997). Hoekstra (1997) analysed the consequences of different management scenarios on a global scale and for the entire 1 360 000 km^2 Zambezi River Basin respectively. Lane et al. (1999) analysed the economic, social and environmental impacts of different climatic scenarios in the USA. Examples of sustainability assessments conducted on a more local scale can be found in McLaren & Simonovic (1999a,b) who analysed the 3 885 km^2 Assiniboine delta aquifer in Winpenny, Canada, and Makoni et al. (2001) who analysed the upper 5 450 km^2 of the 12 000 km^2 Mupfure catchment situated in Zimbabwe.

Adopting too broad a perspective, e.g. global, make it difficult to determine progress towards sustainable development, in particular more local attributes can easily be overlooked. On the other hand, too local a focus might fail to give a realistic picture of the overall system state (Loucks, 2000). The river basin is often defined as the appropriate spatial scale for water resources management, see for example World Bank (1993). Practical issues such as river basins divided into two or more political and administrative units can be a serious barrier towards sustainable development, see for example Kjeldsen et al. (1999b). In this study, the river basin will be adopted as the appropriate spatial scale.

Risk

Design and planning of water resource systems need careful consideration of system failure resulting from floods, droughts or unacceptable low level of water quality. Due to political and economic constraints most water resources systems are not fail-safe, but a sustainable water resources system should experience a decrease in frequency and severity of failures over time (Loucks, 2000). Traditionally, water resources systems, such as reservoirs, have been designed to have a high degree of *reliability* (low probability of failure). Due to research published by Hashimoto et al. (1982a) and Fiering (1982) substantial efforts have been devoted to the additional risk criteria of *resilience* (likelihood of return to normal operation after a failure) and *vulnerability* (likely magnitude of failure). A sustainable system should have a high degree of resilience and low vulnerability (Duckstein & Parent, 1994). The estimation of reliability, resilience and vulnerability (R-R-V) will be further discussed in section 4.2. Issues concerning sustainable management of extreme hydrological events have been highlighted by, for example, Kundzewicz (1999) and Kundzewicz & Kaczmarek (2000).

Technology

The use and development of computer-based modelling technologies are important components in a sustainability assessment tool (Hersh, 1999 and UNESCO, 1999). These modelling systems are supposed to assist the decision-makers in making informed decisions, hence the name Decision Support Systems (DSS). As stakeholder participation is a vital component in the management of water resources (World Bank, 1993) it is recognised that relatively simple models with an easy to understand graphical user interface enhancing the possibilities for achieving a useful shared-vision model set-up (Loucks, 2000). This study aims at developing a framework for assessing sustainability, which is applicable with existing water resources planning and management tools.

The four issues discussed above, together with the three conceptual components: need, future generations and fairness plus the ability to handle multiple objectives should be addressed by a method attempting to assess water resources system sustainability. The usefulness of previously proposed sustainability criteria will be reviewed considering whether the criteria take these issues into account.

3.2 OPERATIONALISATION

The following sections will present a brief review of the use of system analysis in water resources planning followed by a review of how to quantify sustainability in a system analysis framework. According to Takeuchi et al. (1998) and Bender & Simonovic (2000) system analysis provide a useful tool for the analysis of complex systems and their economic, social and environmental implications, hence system analysis appears to be a useful for sustainability assessments of complex water resources systems. Takeuchi et al. (1998) defined system analysis as "*a discipline for seeing entities. It is a framework for seeing interrelationships rather than things and* *seeing patterns rather than static snapshots. It is a set of general principles. It is also a set of specific tools and techniques.*" A more operational definition of system analysis provided by Duckstein & Parent (1994), is presented in section 3.3.

3.3 A SYSTEM VIEW OF WATER RESOURCES

Mathematical modelling has been widely applied to analyse water resources systems, see for example Loucks et al. (1981), Simonovic (1989) and Duckstein & Parent (1994). The framework for applying system analysis in a water resources context presented by Duckstein & Parent (1994) is visualised in Fig. 3.1. Considering a system which is discrete in time and state, then the system consists of the following components: a time horizon *T*, a set of system states *S*, an input set *X*, an output set *Y*, state transition functions *F* and output functions *G*.

Time scale

The time scale of the system simulation is defined by a time horizon *T*

$$t = 1,..,T \tag{3.1}$$

where *T* may be finite or infinite.

State variables

A state variable is an array *s(t)* whose elements belong to the set *S*. Examples of state variables are reservoir storage, cumulative shortage for each user etc.

$$s(0,T) = \left(s(0), s(1), ..., s(T)\right) \tag{3.2}$$

Using a state transition function *F*, the state at the time *t+1* is estimated as a function of the state and input at time *t* as

$$s(t+1) = F\left(s(t), x(t)\right) \tag{3.3}$$

Input

Three types of input to the system exist: controlled variables *U*, non-controllable variables *W*, and system parameters Θ, hence $X = (U, W, \Theta) = x(t)$. As an example of controlled input $u(t) \in U$, Duckstein & Parent (1994) mentioned rules for allocation of scarce resources. Uncontrollable input $w(t) \in W$ are, e.g., hydroclimatic variables such as streamflow and evaporation. The system parameters Θ are assumed constant during the entire system simulation. In system analysis the non-controllable variables *w(t)* are often modelled using stochastic methods.

Output

The output comprises of a set of variables $y(t) \in Y$. The output *y(t)* is estimated as a function of input and state variables as

$$y(t) = G\big(s(t), x(t)\big) \tag{3.4}$$

where *G* is an output function.

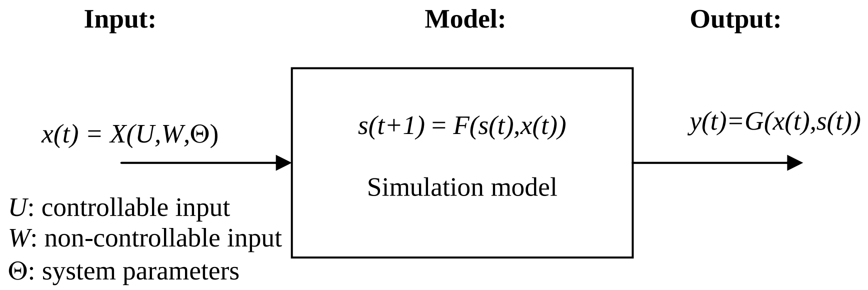


Fig. 3.1: Water resources system analysis using system analysis.

Criteria for system performance can be calculated from the output, and specific system performance criteria related to sustainability will be discussed in section 3.5. By changing the controllable input *U* or the system parameters Θ and observe the resulting system performance criteria the optimal system structure can be identified. A decision-making procedure in a system analysis framework usually consists of the following eight steps (Simonovic, 1989; Acreman, 2000):

1. Defining the problem.
2. Setting objectives, evaluation criteria and constraints.
3. Developing alternatives.
4. Modelling of alternatives.
5. Evaluating alternatives.
6. Selecting alternative.
7. Implement chosen alternative.
8. Monitoring [IF problem THEN GOTO 1].

This procedure assumes that modelling tools capable of considering all factors of importance are available, which is seldom the case when considering the complexities involved in quantifying sustainable development. However, the process will be used as the foundation of a procedure for ensuring sustainable decision-making.

The literature generally distinguishes between two different methods for deriving optimal system performance, viz. simulation and optimisation (Simonovic, 1992). A simulation model is a mathematical model of the physical system under consideration and can be used to predict system behaviour under a specific set of conditions. A simulation model is not an optimisation model in itself, but by performing numerous simulations with different values attached to the decision variables a near optimal solution can be found. Simulation models are widely used for comparing a set of future scenarios. Optimisation techniques have been used in the analysis of water resources systems as reported by, for example, Loucks et al. (1981) and Dyrbak (2001). These techniques often involve some type of mathematical programming technique. From an objective function (specified through one or more system performance criteria), a set of system constraints and a set of decision variables, the value of decision variables that maximises (or minimises) the objective function can be identified.

Simonovic & Fahmy (1999) applied a system analysis approach to analyse the consequences of national water resources policies in Egypt. They introduced a series of modelling guidelines for simplifying the complex task of integrating the hydrological system with other subsystems, such as irrigation or economy, into a manageable task. These modelling guidelines will be further discussed in chapter 4. However, the guideline named *water driven approach* is essential for the review of potential sustainability criteria and, therefore, defined herein. In a water driven approach water supply and water demand are the main policy variables, and scenarios will be evaluated based on the balance between water supply and demand. Other studies, such as Schultz & Hornbogen (1997), analysed long term effects of water resources development based on predictions of water supply and demand, hence through the water driven approach.

3.4 REVIEW OF SUSTAINABILITY ASSESSMENT METHODS

According to Takeuchi et al. (1998) and Kundzewicz (1999), three different methods can be used for assessment of sustainability of water resource systems:

1. A list of indicators of sustainability.
2. A set of guidelines that should be followed to ensure sustainability.
3. One or more sustainability criteria for comparison of different options or policies.

Indicators

The role of an indicator is to show the state or health of the subject under consideration and is a widely applied method for assessing sustainability, especially, through the pressure-state-response framework, which is linking different policy areas and thereby minimising the risk of solutions overlooking important interconnections between subsystems (Rogers et al., 1997). Furthermore, indicators monitor progress, show trends towards sustainability, assist decision-makers and act as a public relation tool (Takeuchi et al., 1998). However, as pointed out by Simonovic et al. (1997) and McLaren & Simonovic (1999a,b), they cannot be used directly in the decision-making process as they only present a static view of the state of the system. An example of useful indicators for the analysis of water resources systems was given by Takeuchi et al. (1998).

Guidelines

The vague notion of sustainable development and the subsequent difficulties associated with the identification of potential methods for quantification led Loucks (1997) to conclude that " *mathematical and technical language alone is not sufficient to measure sustainability fully*" and WCD (2000) to conclude that debate over how to develop water resources, especially large dams, "*is complex because issues are not confined to the design, construction and operation of the dams themselves but embrace the range of social, environmental and political choices on which the human aspiration to development and well-being depends.*". The response to these difficulties has been to develop guidelines (or checklists) for sustainable water resources development and management. Guidelines have been developed at different levels. Considering general policy principles ICWE (1992) listed four principles for water and sustainable development (the Dublin principles):

1. Fresh water is a finite and vulnerable resource, essential to sustain life, development and the environment.
2. Water development and management should be based on a participatory approach, involving users, planners and policy makers at all levels.
3. Women play a central part in the provision, management and safeguarding of water.
4. Water has an economic value in all its competing uses and should be recognised as an economic good.

Other guidelines are more specific and project oriented such as Takeuchi et al. (1998) and UNESCO (1999). Attempts to actually quantify sustainability using guidelines have been reported by Baan (1994), Neupane & Young (1997) and Makoni et al. (2001), all based on procedures used for Environmental Impact Assessments (EIA). The different aspects of the guidelines are quantified on a predefined numeric scale according to how well this particular aspect is met. Afterwards all scores are lumped together using a weighting scheme. Some methods consider a predefined set of issues that needs consideration. Other methods are more open-ended and require the analyst to identify issues of importance. These methods are generally accepted as a valid tool but have a number of build-in flaws. They suffer from a large degree of subjective judgement involved in determining which issues to include in the analysis, the magnitude of impacts and the weight attached to each impact as a measure of relative importance. Rogers et al. (1997) criticised the use of weighting schemes in decision problems as the final decision is too dependent on the chosen weights. Other research groups have tried to develop methods for assessing the weights through stakeholder consultation see, for example, Steward et al. (1997), McLaren & Simonovic (1999a,b). The actual assessment of preferences, hence weights, through stakeholder consultation is beyond the scope of this research.

Criteria

A criterion is defined by Takeuchi et al. (1998) as "*a standard on which a decision may be based*". Hence, a criterion can by used to compare and decide among different scenarios or development options. Several criteria of sustainability have been proposed in literature, for example, an *entropy* analogy (Nachtnebel, 2002), *consensus* (Bender & Simonovic (1997), *reversibility* (Fanai & Burn, 1997), risk (Kroeger & Simonovic, 1997), *reliability*, *resilience* and *vulnerability* (Duckstein & Parent, 1994; Loucks, 1997; UNESCO, 1999), and inter and intra-generational *equity* (Matheson et al., 1997). Another criterion suggested by Simonovic et al. (1997) is *robustness*, defined in a hydro-economic framework by Hashimoto et al. (1982b), but it has not yet been attempted defined in a sustainability context.

Most of the proposed sustainability criteria are based on the system analysis approach, and in the following a general framework for using most criteria will be devised. The framework is applicable within the system analysis approach presented in section 3.3. A broad classification of the proposed sustainability criteria divides them into two groups. The first group consists of criteria that can be estimated based on a direct comparison of demand and supply, i.e. a measure of the degree to which specific demand is fulfilled by a certain scenario. This class of criteria will be labelled water driven criteria (WDC) as they are directly applicable ith the water driven approach defined in section 3.3. It should be noted that the term WDC does not imply that these criteria are confined to compare water demand and supply. Most of these criteria have been developed for application in a broader multicriteria framework comparing data with different units from different sectors, and uses a variety of aggregation methodologies. The definition of the second group of criteria is fuzzier and basically consists of all the criteria that does not fit into the first group.

3.5 WATER DRIVEN CRITERIA

The water driven criteria encompass four different criteria suggested in the literature:

1. The criterion based on reliability, resilience and vulnerability.
2. The risk criterion.
3. The reversibility criterion.
4. The inter- and intra-generational fairness.

All four criteria are further explained in this section.

The procedure when using a WDC consists of the following four steps:

1. Identify the different scenarios *i; i* = 1,..., *NI*, where *NI* is the number of scenarios.

2. Identify the users c (c = 1,...,NC, where NC is the number of users) that potentially will be affected by the development under consideration. The need of each user is defined directly in terms of a specified threshold value of one or more water resources systems variable $s(t)$ such as water demand, reservoir storage, flow etc. The term users should be understood as not only related to human activities but all issues considered important, e.g. environmental water demand. Some methods use more sophisticated methods for evaluation, for example sub-dividing the impacts acting on a user into different categories, such as environmental, economic and social impacts, where the number of categories is denoted NJ. Other methods consider a number of impacts NG acting on a predefined number of users, hence in this case a user can be affected by several impacts.

3. Each of the NI scenarios are modelled using an appropriate hydrological model and the time series of the important state variables $s(t)$, t = 1, …, T extracted.

4. The value of the chosen sustainability criterion is calculated by comparing demand and supply.

5. Evaluate and rank scenarios based on the selected criteria.

Kjeldsen & Rosbjerg (2001a, 2001b) argued that a WDC cannot consider all factors of importance to sustainability. This aspect will be further discussed in section 4.9. Kay (2000) reached the same conclusion but stressed that a WDC is a vital part of a sustainability assessment.

Reliability, resilience and vulnerability (R-R-V)

This criterion was proposed by Loucks (1997) based on the idea that system sustainability is related to a high degree of reliability and resilience and low vulnerability as suggested by Duckstein & Parent (1994). The starting point is the identification of the NC users to be included in the analysis, note that this method does not consider impact categories. For each user c a threshold level separating satisfactory values of a state variable from non-satisfactory states has to be specified. Next, the planning horizon T is subdivided into a number of periods N each of length T/N . Loucks (1997) applied a planning horizon of T = 50 years and N = 5 periods each with a duration of 10 years.

The NI scenarios are modelled and for each time period the relevant state variables $S(c,t)$, where t = 1,...,T; c = 1,…,NC, are extracted and compared to the specified threshold levels. The duration of the j-th excursion into a failure period is denoted $d(j)$ and $v(j)$ is the corresponding deficit volume, j = 1,…,M, where M is the total number of failure events. The summary statistics of $d(j)$ and $v(j)$ can be used to estimate R-R-V, which will be further discussed in Chapter 4.2. A sustainability index $Sust(c,n,i)$ for user c in time period number n, where n = 1,…,N, for each scenario i is given as

$$Sust(c,n,i) = \mathrm{Rel}(c,n,i)\,\mathrm{Res}(c,n,i)\,[1 - \mathrm{rVul}(c,n,i)] \tag{3.5}$$

where relative vulnerability, rVul, is defined as

$$\mathrm{rVul}(c,n,i) = \frac{\mathrm{Vul}(c,n,i)}{\max_i \{\mathrm{Vul}(c,n,i)\}} \tag{3.6}$$

Finally, the relative sustainability *rSust* of the period under consideration for scenario *i* is estimated as a weighted average of the *Sust(c,n,i)* values of all users

$$rSust\,(n,i) = \sum_{c=1}^{NC} w(c) Sust(c,n,i), \quad n = 1,..., N \tag{3.7}$$

where *w(c)* is the weight assigned to each user ranging from 0 to 1 and summing up to 1. Each scenario is characterised by a *rSust* value in each of the *N* time periods. These *rSust* values should increase if the sustainability of the system is increasing. Actual guidelines on how to apply the temporal variability of the sustainability criterion into the decision-making process are not available, and Loucks (1997) refers to this as an unresolved issue that needs further attention. Kay (2000) used the R-R-V criterion to measure the sustainability of national water resources management policies in Israel, and Kjeldsen & Rosbjerg (2001a, 2001b) further developed this criterion, which will be discussed in Chapter 4.

Risk measure

Kroeger & Simonovic (1997) developed a risk measure for sustainable project selection. The criterion was developed as a part of a general framework reported by Simonovic et al. (1997) and should be regarded as one among several possible criteria, such as reversibility, fairness and consensus, all three described later. The risk criterion measures the product of the magnitude of a negative effect and the probability of occurrence. To apply the risk criterion for comparing the sustainability of the *NI* scenarios the following procedure was presented. First identify *NC* users and *NJ* impact categories corresponding to a social, environmental and economic category. Furthermore, for each scenario *i* identify the risks *r(j)* , where *r* = 1, .., *R(j)*, associated with the *j*-th impact category. Next, the probability of occurrence *p(r,j,c)* of each risk *r(j)* under each scenario *i* is calculated. According to Kroeger & Simonovic (1997) a specialist in the area of concern should undertake this. If the risk is not present under a specific scenario the probability of occurrence equals zero. Each user *c* is now asked to assign weights *d(j,c)* according to how important they consider category *j* to be. The weights should sum up to 100. Furthermore, each user *c* is asked to assign their preference to avoid the full magnitude of each risk, named the risk weight $k(r,j,c)$. The value of $k(r,j,c)$ should be between 0 (indicating no risk aversion) and 10 (indicating highest aversion to risk). Now the values $\upsilon(r,j,c)$ for risk *r(j)* for user *c* can be estimated as

$$\upsilon(r,\, j,c) = \frac{d(j,c)k(r,\, j,c)}{\sum d(j,c)} \tag{3.8}$$

The risk estimate *e(i,c)* obtained by user *c* under scenario *i* is calculated as

$$e(i,c)=\sum_{j=1}^{NJ} p(r,\, j, c)\upsilon(r,\, j, c) \tag{3.9}$$

Finally, an aggregated score x_i for each scenario i is obtained as a weighted average of the $e(i,c)$ values as

$$x(i)=\sum_{c=1}^{NC} w(c)e(i,c), \quad i=1,...,NI \tag{3.10}$$

where $w(c)$ are the weights attached to each user and sum up to one. Application of the risk criterion has been reported by Kroeger & Simonovic (1997) and McLaren & Simonovic (1999a). The later reported on difficulties in obtaining risk preferences from participating stakeholders.

Reversibility

Reversibility was presented as a part of the general framework for assessing sustainability in Simonovic et al. (1997), and is supposed to measure how well the impacts of a given development can be mitigated. The following procedure for estimating reversibility was presented by Fanai & Burn (1997).

First, identify all the affected users NC in each of the NJ = 3 categories environmental, economic and social. Next, for each user c in each category j determine the best impact magnitude $M(c,j)$ and the worst impact magnitude $m(c,j)$. Determine the impact magnitude $f(c,j,i)$ acting on user c in category j under scenario i. Estimate the reversibility $R(j,i)$ for category j under scenario i using a distance metric

$$R(j,i)=\left(\sum_{c=1}^{NC} w^2(c,\, j)\left|\frac{M(c,\, j)-f(c,\, j,i)}{M(c,\, j)-m(c,\, j)}\right|^2\right)^{\frac{1}{2}} \tag{3.11}$$

where $w(c,j)$ is the weight assigned by user c to category j, the weights sum up to 1. An R-metrics will be attached to each of the NJ categories in each of the NI scenarios. The scenario with the lowest R-value has the highest degree of reversibility. The metric presented in Eq. (3.11) is equal to the objective function used in compromise programming as presented by Loucks et al. (1981).

Inter- and intra-generational fairness

Inter- and intra-generational fairness was presented as a sustainability criterion by Matheson et al. (1997) as a part of the general framework presented by Simonovic et al. (1997). The criterion is based on three objectives for fair allocation known as perfect proportionality, perfect equality and satisfaction off needs. Each of the objectives can be expressed mathematically as

$$\begin{aligned} B_1 &= \sum_{c=1}^{NC} \left| E(c) - A(c) \right| \\ B_2 &= \sum_{c=1}^{NC} \sum_{d=1}^{NC} \left[\frac{\left| E(c) - E(d) \right|}{2NC^2 \overline{E}} \right] \\ B_3 &= \sum_{c=1}^{NC} \left| E(c) - Z(c) \right| \end{aligned} \tag{3.12}$$

where B_1, B_2 and B_3 measures deviation from a proportional impact allocation, an equal impact allocation and an impact allocation that exactly satisfies the needs of all users, respectively. The impact magnitude acting on the *c*-th group is given as *E(c)*. Correspondingly, the impact magnitude that group *c* deserves and the impact magnitude that user *c* requires to satisfy its needs are given as *A(c)* and *Z(c)*, respectively. Matheson et al. (1997) extended Eq. (3.12) to encompass both inter- and intra-generational fairness. First, identify the *NC* groups involved in the analysis and the *NG* different impacts. The magnitude of the *g*-th impact acting on the *c*-th group at time *t* under scenario *i* is denoted *E(c,g,t,i)*. Correspondingly, *A(c,g,t)* and *Z(c,g,t)* are the impact magnitude that group *c* deserves and the impact magnitude that group *c* requires to satisfy its needs with respect to the *g*-th impact at time *t*. The inter- and intra-generational fairness are estimated as

$$\begin{aligned} B_1(i) &= \frac{1}{NG \cdot T} \sum_{t=1}^{T} \sum_{g=1}^{NG} \left[w(g) \sum_{c=1}^{NC} \left| E(c,g,t,i) - A(c,g,t) \right| \right] \\ B_2(i) &= \frac{1}{NG \cdot T} \sum_{t=1}^{T} \sum_{g=1}^{NG} \left[w(g) \frac{\sum_{c=1}^{NC} \sum_{d=1}^{NC} \left| E(c,g,t,i) - E(d,g,t) \right|}{2NC^2 \overline{E}_{gci}} \right] \\ B_3(i) &= \frac{1}{NG \cdot T} \sum_{t=1}^{T} \sum_{G=1}^{NG} \left[w(g) \sum_{c=1}^{NC} \left| E(c,g,t,i) - Z(c,g,t) \right| \right] \end{aligned} \tag{3.13}$$

$$\begin{aligned} B_1^*(i) &= \frac{1}{NG \cdot NC} \sum_{g=1}^{NG} \left[w_j \sum_{c=1}^{NC} \sum_{t=1}^{T} \left| E(c,g,t,i) - A(c,g,t) \right| \right] \\ B_2^*(i) &= \frac{1}{NG \cdot NC} \sum_{g=1}^{NG} \left\{ w_j \sum_{c=1}^{NC} \left[\frac{\sum_{s=1}^{T} \sum_{t=1}^{T} \left| E(c,g,t,i) - E(d,g,t,i) \right|}{2T^2 \overline{E}_{cgi}} \right] \right\} \\ B_3^*(i) &= \frac{1}{NG \cdot NC} \sum_{g=1}^{NG} \left[w_j \sum_{c=1}^{NC} \sum_{t=1}^{T} \left| E(c,g,t,i) - Z(c,g,t) \right| \right] \end{aligned} \tag{3.14}$$

where B_1, B_2 and B_3 are average intra-temporal fairness measures and B_1^*, B_2^* and B_3^* are average inter-temporal fairness measures. Matheson et al. (1997) merged Eq. (3.13) and Eq. (3.14) into one overall measure of inter-and intra-generational fairness and expressed α and ψ, respectively, using a normalized geometric-based distance metric as

$$\alpha(i) = \left[\sum_{v=1}^{3} q_v^2 \left| \frac{B_v(i)}{B_{\max} - B_{\min}} \right|^2 \right]^{\frac{1}{2}}$$
$$\psi(i) = \left[\sum_{v=1}^{3} q_v^2 \left| \frac{B_v^*(i)}{B_{\max}^* - B_{\min}^*} \right|^2 \right]^{\frac{1}{2}} \tag{3.15}$$

where v is the index for different fair allocation objectives, B_{min} and $B_{\min}^*$ are minimum values of B_v and B_v^* for v = 1,2,3. B_{max} and $B_{\max}^*$ are maximum values of B_v and B_v^* for v = 1,2,3. The relative weights q_v are between zero and one and should sum up to one. A closer look at Eq. (3.13) and Eq. (3.14) reveals that the summation parts of the equations for inter- and intra-generational fairness are in fact identical, i.e. inter- and intra-generational fairness is only different in terms of their denominators.

3.6 OTHER CRITERIA

The term “other criteria” is covering the criteria that cannot be evaluated based on a comparison of water supply and demand. These criteria are not considered further in this study but are included in the review to obtain a complete picture of suggested sustainability criteria. These criteria are consensus, an entropy analogy and robustness.

Consensus

Consensus was presented by Bender & Simonovic (1997) and attempts to quantify the degree of agreement or disagreement among users involved in a decision problem. Assume that *ND* decision-makers have been involved in a decision problem trying to choose among *NI* different scenarios. For each decision-maker *u* in each scenario *i* an evaluation criterion *x(u,i)* has been estimated. On this foundation Bender & Simonovic (1997) defined five different operational measures of consensus as

$$
\begin{aligned}
&\gamma^1(i) = 1 - \min_{u \neq d} \left| w(u)x(u,i) - w(d)x(d,i) \right|, \quad u,d = 1,\ldots,ND \\
&\gamma^2(i) = 1 - \max_{u \neq d} \left| w(u)x(u,i) - w(d)x(d,i) \right| \quad u,d = 1,\ldots,ND \\
&\gamma^3(i) = 1 - \frac{1}{ND}\sum_{u=1}^{ND} \left| w(u)x(u,i) - \bar{x}(i) \right| \quad u = 1,\ldots,ND \\
&\gamma^4(i) = 1 - \frac{2}{ND(ND-1)}\sum_{u=1}^{ND-1}\sum_{d=u+1}^{ND} \left| w(u)x(u,i) - w(d)x(d,i) \right| \quad u,d = 1,\ldots,ND \\
&\gamma^5(i) = 1 - \max\left| w(u)x(u,i) - \bar{x}(i) \right| \quad u = 1,\ldots,ND
\end{aligned}
\tag{3.16}
$$

where *w(u)* is the relative weight of decision-maker *u* and

$$
\bar{x}(i) = \frac{1}{ND}\sum_{c=1}^{NC} w(u)x(u,i) \tag{3.17}
$$

The criteria $\gamma^1(i)$, $\gamma^3(i)$ and $\gamma^5(i)$ measure the degree of agreement and $\gamma^2(i)$ and $\gamma^4(i)$ the degree of disagreement in the ranking of the *NI* scenarios. Bender & Simonovic (1997) argue that the metric of consensus sustainability as presented in Eq. (3.16) does not attempt to identify the best solution but rather provides feedback into the decision-making procedure in order to reduce the number of appropriate scenarios and to identify sources of (dis)agreement.

Robustness

An operational definition of robustness for water resources systems planning based on an economic analysis was presented by Hashimoto et al. (1982b). Consider a proposed design *D* of a water resources system planned for satisfying the future demand *q*. The cost of accommodating *q* with *D* is defined as *C(q|D)*. The optimal economic design *L(q)* minimises the cost of satisfying *q* with *D*, viz.

$$
L(q) = \min_{D}\left\{C(q \mid D)\right\} \tag{3.18}
$$

The opportunity costs of selecting a design *D* for a given demand *q* is the difference between *C(q|D)* and *L(q)*. A merit of a robust system is that the opportunity costs for a given design *D* are small regardless of the value of the future demand q, i.e. the system is able to adopt a wide range of future conditions at small costs. Hashimoto et al. (1982b) argued that the opportunity costs should be smaller or equal to a fraction β of the minimum costs, hence

$$
\begin{aligned}
&C(q \mid D) - L(q) \le \beta L(q) \\
&\Updownarrow \\
&C(q \mid D) \le (1+\beta)L(q)
\end{aligned}
\tag{3.19}
$$

Consequently, robustness R_β is defined as

$$R_\beta = P\{C(q \mid D) \le (1+\beta)L(q)\} \tag{3.20}$$

where a robust system will be characterised by having a high R_β value.

Simonovic et al. (1997), on the other hand, define robustness as "*a reasonable intuitive result of [a] sensitivity analysis*".

Entropy analogy

Due to large uncertainties involved in extended planning horizon and broadened range of considered factors prerequisite by the sustainability concept, reversibility becomes an important system characteristics. Where robustness attempts to identify systems that can adapt to future conditions, the entropy criteria attempt to identify solutions that can be bring the system back to the pristine non-engineered state using a minimum of time and energy. Nachtnebel (2002) presented two examples of defining reversibility based on entropy concepts from economy and physics, respectively. Nachtnebel (2002) emphasises the use of energy and time rather than costs, as in robustness, for measuring reversibility, as costs can change dramatically over the course of time due to e.g. development of new technologies

3.7 REVIEW OF CRITERA

The review of the sustainable development concept and of issues concerning the sustainability of water resource systems highlighted a number of issues of importance. A criterion, or series of criteria, claiming to be a practical metric of water resources system sustainability should seriously consider the following issues

Sustainable development:

1. Need of present and future generation.
2. Inter- and intra-generational fairness.
3. Allow for a multiobjective approach.

Water resources system sustainability:

1. Consider risk.
2. Can be incorporated in a water resources DSS.
3. Useful in a changing system.
4. Be applicable at different scales (temporal and spatial).

Table 3.1 shows a score card according to how well each criterion consider each of the issues highlighted above. Due to the limited practical experience with most criteria the score card is based on a subjective evaluation, but is believed to give a general impression of the usefulness of the criteria. Two stars indicate that the issue is considered by the criterion. One star implies that the issue is being considered, but not in a way that is perceived as satisfactory. No star means that the issue was not considered by the criterion. The criteria are not evaluated based on scale. As no general method for applying entropy as a criterion has been developed, it is not considered.

Table 3.1 Scorecard for the reviewed sustainability criteria.

	RRV	Fairness	Risk	Reversibility	Consensus	Robustness
Sustainable development						
Need	★★	★★	★	★		★
Fairness		★★		★	★	
Multiobjective	★★	★★	★★	★★		
Water Resources System Sustainability						
Risk	★★		★★			★
DSS	★★	★★	★★	★★	★	★
Change	★★	★★	★	★	★	★★

From Table 3.1 it is evident that a combination of a criterion based on R-R-V and the fairness criterion should give the most comprehensive assessment of water resources system sustainability. In Chapter 4 the R-R-V criteria suggested by Loucks (1997) and UNESCO (1999) will be extended to also consider inter- and intra-generational fairness, following the approach outlined by Matheson et al. (1997). The criteria are all very much alike. Additional issues for comparison could be ability to function over a wide spectrum of case studies and practical applicability (McLaren & Simonovic, 1999a). However, due to the limited amount of published case studies this was not considered herein.

3.8 A UNIFIED FRAMEWORK FOR MAKING DECISIONS LEADING TO SUSTAINABLE DEVELOPMENT

Each of the three methods for assessment of sustainability (indicators, guidelines and criteria) has a function in the decision-making procedure outlined in section 3.3. Indicators are useful in step (1) and (2) as they point to factors of importance to sustainability. Guidelines should be followed in step (3) and (7), i.e. alternatives should not be developed and the selected project should not be implemented without consulting these guidelines. The sustainability criteria are selected in step (2) and used in (6) for the final project selection. Using the three methods (indicators, guidelines and criteria) at different stages could potentially force the decision-making procedure to consider sustainability explicitly. Still to be developed, the decision-making procedure should be used in a more comprehensive quantification of sustainability by giving each alternative developed in step (3) credits according to how well the guidelines are followed. Subsequently, by combining with the modified sustainability criterion, this can lead to a real sustainable decision-making procedure (SDMP), which would define sustainability of water resources systems as a process rather than a static situation as discussed by Kay (2000).

3.9 BARRIERS TO SUSTAINABLE WATER RESOURCES SYSTEMS

Having identified a potential criterion for assessing relative sustainability of development options, and developed a framework for its practical application is no guarantee for sustainable development of water resources systems. The World Bank (1993) states that most problems related to misallocation and wastage of water, as well as environmental damage, are a result of failures in the institutional setup responsible for water resources management. More specific they refer to issues of over-reliance on central government control, fragmented and sectorial management and general neglect of water quality, health and environmental aspects in public investments. Recently, research has been initiated to identify barriers to sustainable water resources management, both with respect to water quantity and quality. Chen & Xia (1999) addressed the problems concerning population growth, extreme events (droughts and floods) and environmental degradation jeopardising the sustainability of the water resources in China and listed a number of suggestions for enhancing water resources management. In another case study Kjeldsen et al. (1999b) studied barriers towards sustainable water resources management in Zimbabwe, where they highlighted barriers related to the existing water law and the traditional perception of water resources, respectively. In a review of barriers to sustainable water-quality management, Huang & Xia (2001) highlighted a number of issues in need of more research.

Another important barrier towards sustainable water resources systems is the lack of adequate information and data material for decision-making. It has been suggested by Kundzewicz (1997) that the density of the hydrological gauging network could be used as a potential criteria of water resources system sustainability. Furthermore, he stresses that the minimum requirement should be adequate information to avoid gross mistakes in decision-making. Burn (1997) foresees that the extended spatial and temporal scale required by considerations of sustainability will require properly designed and managed hydrological gauging networks; something which is not the case in many regions of the world.

3.10 SUMMARY

This chapter has reviewed the existing methods and criteria for assessing sustainability of water resources systems. Based on the issues of importance for sustainable development in general and, more specifically, the sustainability of water resources systems, the existing sustainability criteria were reviewed. The conclusion to this analysis was, that a combination of the R-R-V criterion by Loucks (1997) and the intra-and inter-generational fairness criterion by Matheson et al. (1997) would give the conceptually most sound measure of water resources system sustainability. The following chapter presents a sustainability criterion, which is a combination of the two existing criteria mentioned above. Furthermore, the statistical properties of the criterion will be examined in order to develop guidelines for its use.

Chapter 4

A REVISED WATER RESOURCES SUSTAINABILITY ASSESSMENT PROCEDURE

This chapter presents a procedure for assessing the sustainability of water resources development scenarios, including an investigation of both the technical aspects and guidelines for practical application. The procedure is a part of the united framework presented in section 3.8 for making decisions ensuring selection of the water resources development scenario with the highest degree of sustainability. The actual assessment consists of two components:

1. A revised water driven sustainability criterion based on the criteria presented by Loucks (1997) and Matheson et al. (1997).

2. A checklist to correct for factors not encompassed by the water driven criterion. The checklist is based on a tool for EIA named RIAM as presented by Jensen (1998) and applied by, for example, Makoni et al. (2001).

The final sustainability score is defined as the product of the two components.

First, a revised water driven sustainability criterion is presented followed by an investigation of the most suitable combinations of estimators of R-R-V to be used. This investigation will be based on historical runoff data from the Southern African region as well simulation studies based on linear stochastic models estimated from the historical data. Next, the RIAM procedure will be presented and, finally, practical guidelines for coupling of the assessment procedure with existing hydrological modelling tools will be given.

4.1 LOUCKS (1997) REVISITED

Two aspects of Loucks' criterion have been further developed. First, the criterion was adjusted to give more credible results and, secondly, extended to encompass inter- and intra-generational fairness. The sustainability criterion proposed by Loucks (1997), and presented in section 3.5, is based on estimates of reliability, resilience and vulnerability (R-R-V), which are combined into a sustainability index for each user c for each scenario i and for each time period n as

$$Sust(c,i,n) = \mathrm{Rel}(c,i,n)\,\mathrm{Res}(c,i,n)\left[1-\mathrm{rVul}(c,i,n)\right] \tag{4.1}$$

where relative vulnerability, rVul, is given as

$$\mathrm{rVul}(c,n,i) = \frac{\mathrm{Vul}(c,n,i)}{\max\limits_{i}\left\{\mathrm{Vul}(c,n,i)\right\}} \tag{4.2}$$

The values of *Sust* fall in the interval between zero and one. Kjeldsen & Rosbjerg (2001a) argued that the form of the sustainability index as presented in Eqs. (4.1)-(4.2) introduces bias in the ranking of the considered scenarios as the scenario with the maximum vulnerability always will be assigned a sustainability score of zero regardless of the values of Rel and Res. Instead, Kjeldsen & Rosbjerg (2001a) suggested a non-sustainability criterion, *nSust*, given as

$$nSust(c,n,i) = \left[1-\mathrm{Rel}(c,n,i)\right]\left[1-\mathrm{Res}(c,n,i)\right]\left[\mathrm{rVul}(c,n,i)\right] \tag{4.3}$$

where

$$\mathrm{rVul}(c,n,i) = \frac{\mathrm{Vul}(c,n,i)}{\frac{1}{NI}\sum\limits_{i}\mathrm{Vul}(c,n,i)} \tag{4.4}$$

Alternatives with small values of *nSust* should be selected. The non-sustainability number, as defined above, has the obvious disadvantage that is has no upper bound. A more appropriate approach than the non-sustainability criterion was presented by Kjeldsen & Rosbjerg (2001b) and can be obtained by using the original sustainability criterion, Eq. (4.1) together with a modified definition of rVul given as

$$\mathrm{rVul}(c,n,i) = \frac{\mathrm{Vul}(c,n,i)}{\sum\limits_{i}\mathrm{Vul}(c,n,i)} \tag{4.5}$$

which makes the sustainability number fall into the interval between zero and one, as the original criterion, but without the bias described above. This last form of the criterion is believed to be the most appropriate and, hence, adopted for further analysis in this study. Kjeldsen & Rosbjerg (2001b) presented a further extension of the criterion in which it has been combined with the fairness criterion presented by Matheson et al. (1997) as described in section 3.5. This extension of the criterion resolved the problem outlined by Loucks (1997) of how to include the temporal trend of the sustainability criterion into the decision-making procedure.

Matheson et al. (1997) considered several methods for quantifying inter- and intra-generational fairness. One of the methods measures the deviation from an impact allocation that exactly satisfies the needs of all groups involved in the analysis in terms of

$$
\begin{aligned}
B_3(i) &= \frac{1}{NG \cdot T} \sum_{t=1}^{T} \sum_{g=1}^{NG} \left[w(g) \sum_{c=1}^{NC} \left| E(c,g,t,i) - Z(c,g,t) \right| \right] \\
B_3^*(i) &= \frac{1}{NG \cdot NC} \sum_{g=1}^{NG} \left[w(g) \sum_{c=1}^{NC} \sum_{t=1}^{T} \left| E(c,g,t,i) - Z(c,g,t) \right| \right]
\end{aligned}
\tag{4.6}
$$

where NG is number of impact, NC is number of groups, T is number of time steps, i is the scenario under consideration, $E(c,g,t,i)$ is the magnitude of the g-th impact felt by the c-th group in time t under the i-th scenario, and $Z(c,g,t)$ is the magnitude of the g-th impact needed by the c-th group at time t. The weights $w(g)$ indicate the importance of category j and must sum up to one. The term B_3 indicates intra- and B_3' inter-generational equity, respectively. Small values indicate alternatives to be preferred.

The system performance indices R-R-V used by Loucks (1997) are equivalent to the difference between E and Z used by Matheson et al. (1997) as system performance criterion. Identical averaging procedures could be used to accommodate the problem of how to incorporate the temporal variation of the sustainability criterion in decision-making. Matheson et al. (1997) considered both impacts NG and groups NC, whereas Loucks (1997) considered the users directly, i.e. the corresponding simplified averaging schemes are given as

$$
\begin{aligned}
rSust(i) &= \frac{1}{N} \sum_{n=1}^{N} \sum_{c=1}^{NC} w(c) Sust(c,n,i) \\
rSust^*(i) &= \frac{1}{NC} \sum_{c=1}^{NC} w(c) \sum_{n=1}^{N} Sust(c,n,i)
\end{aligned}
\tag{4.7}
$$

where $rSust$ is the relative sustainability of the i-th scenario. The weights $w(c)$ indicate the importance of the c-th user and sum up to one. Eq. (4.7) can be interpreted as sustainability criteria with consideration of inter- and intra-generational fairness, respectively. High values of $rSust$ and $rSust^*$ are preferable and indicates a high degree of fairness. However, as for the fairness criteria proposed by Matheson et al. (1997), the two criteria are identical except for the value of the denominators as explained in section 3.5. As only the inter-generational criterion in Eq. (4.7) will give values between zero and one, it is adopted as the final water driven sustainability criterion

$$
rS_{WDC} = \frac{1}{N} \sum_{n=1}^{N} \sum_{c=1}^{NC} w(c) Sust(c,n,i) \tag{4.8}
$$

The index WDC indicates that it is water driven criterion. The rS_{WDC} criterion ranges from zero to one, with high scores being preferable to low scores. The criterion equals the average of the $rSust$ criterion in Eq. (4.1) over the N time periods and do not consider the direction of change.

4.2 DEFINITION OF RELIABILITY, RESILIENCE AND VULNERABILITY

The objective of this section is to review the estimators of R-R-V proposed in the literature and to examine which of these estimators are the most appropriate for use in connection with the criterion by Loucks (1997). Since the publications of Hashimoto et al. (1982a) and Fiering (1982), the use of R-R-V in the risk analysis of water resources systems analysis has been widely discussed, see for example, Moy et al. (1986), Kindler & Tyszewski (1989), Kundzewicz (1989), Burn et al. (1991), Tickle & Goulter (1994), Jinno et al. (1995), Kundzewicz & Chalupka (1995), Kundzewicz & Kindler (1997), Kundzewicz & Laski (1995), Vogel & Bolognese (1995), Vogel & McMahon (1996), Schumann (1997), Cancelliere et al. (1998), Zongxue et al. (1998), Montaseri & Adeloye (1999), Srinivasan et al. (1999), Vogel et al. (1999) and Adeloye & Montaseri (2000). Several estimators of R-R-V have been proposed, but few of the studies mentioned above discuss which is the most appropriate estimator to use under different conditions. Common for all estimators is that they rely on the statistical characteristics of failure periods for their estimation. The following section introduces the concept of a failure period and reviews the estimators of R-R-V proposed in the literature.

Characteristics of failure spells

The following description of R-R-V is based on the assumption, that the system under consideration at a given time *t* can be in either a satisfactory state (non-failure) *NF* or in a an unsatisfactory state (failure) *F*. The *NF* state occurs when water supply is able to meet water demand and, hence, the *F* state is when supply cannot meet demand. Moving from time step *t* to *t*+1, the system can either remain in the same state or migrate to the other state. Vogel (1987) illustrated the separation of and transaction between satisfactory and unsatisfactory system state as shown in Fig. 4.1.

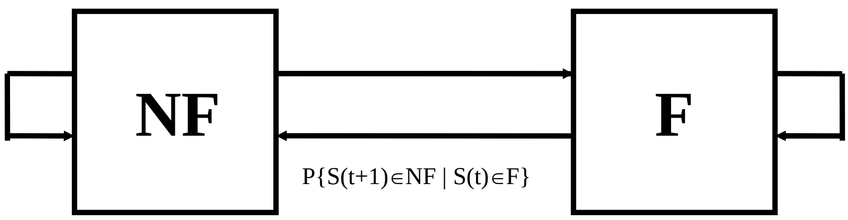


Fig 4.1 Transaction between failure and non-failure of a water resources system.

The duration of the *j*-th excursion into a failure period is denoted *d(j)* and corresponding deficit volume is denoted *v(j)*, *j* = 1,…,*M*, where *M* is the total number of failure events. The definitions of *d(j)* and *v(j)* are illustrated for a single failure event in on Fig. 4.2. A failure event begins at the time step where water supply cannot fulfil water demand and continues until supply once again exceeds demand. The deficit volume of the failure even is calculated as the cumulative difference between demand and availability as

$$v(j) = \sum_{t=1}^{d(j)} \left[D(t) - Y(t) \right] \tag{4.9}$$

where *d(j)* is the duration of the failure, *D(t)* and *Y(t)* are the water demand and the water actually supplied, respectively, at time *t*.

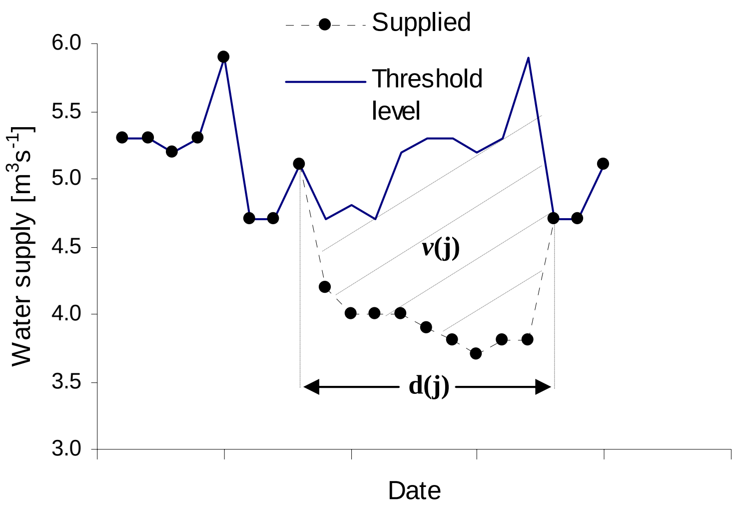


Fig. 4.2 Characteristics of duration and deficit volume of a failure event.

The following section describes how to estimate R-R-V from the extracted series of failure duration and deficit volume.

Reliability

The oldest and most widely used performance criteria for water resources systems is reliability, which is defined by Hashimoto et al. (1982a) as

$$\mathrm{Rel} = P\{S(t) \in NF\} \tag{4.10}$$

S(t) = the system state variable under consideration at time t.

Kundzewicz & Kindler (1995) have listed four definitions of reliability:

1. Occurrence reliability.
2. Volumetric reliability.
3. Temporal reliability.
4. Annual reliability.

The most widely accepted and applied definition is occurrence reliability, which can be estimated as

$$\text{Rel} = 1 - \frac{\sum_{j=1}^{M} d(j)}{T} \tag{4.11}$$

where

d(j) = duration of *j*-th failure event,
M = number of failure events, and
T = total number of time intervals

Also volumetric reliability has found some application and is defined as

$$\text{Rel} = 1 - \frac{\sum_{t=1}^{T} \left[D(t) - Y(t) \right]}{\sum_{t=1}^{T} D(t)} \tag{4.12}$$

where

Y(t) = water supplied
D(t) = water demand
T = Length of time series

Klemes et al. (1981) argued that the advantage of volumetric compared to occurrence reliability is the inclusion of water deficit, which has more socio-economic relevance. However, with the inclusion of resilience and vulnerability, this aspect has no relevance for the choice of reliability estimator. The last two definitions have found little use in practise and is not considered further in this study. Based on these findings, this study has adopted the occurrence reliability, which will be denoted Rel.

Resilience

Resilience is a measure of how fast a system is likely to return to a satisfactory state once the system has entered an unsatisfactory state. Fiering (1982) suggested eleven different estimators of resilience, but the most operational suggestion originates from Hashimoto et al. (1982a), who define resilience as a conditional probability

$$\text{Res} = P\{S(t+1) \in NF \mid S(t) \in F\} \tag{4.13}$$

where *S(t)* is the system state variable under consideration at time *t* and the terms F and NF refer to the failure and non-failure states defined in Fig. 4.1. They also showed that this definition of resilience is equal to the inverse of the mean value of the time the system spends in an unsatisfactory state, i.e.

$$\text{Res}_1 = \left\{ \frac{1}{M} \sum_{j=1}^{M} d(j) \right\}^{-1} \tag{4.14}$$

where

d(j) = duration of the *j*-th failure event
M = total number of failure events

Moy et al. (1986) defined resilience as the maximum consecutive duration the system spends in an unsatisfactory state To make this definition comparable with the definition in Eq. (4.14), resilience is expressed as the inverse of the maximum duration as

$$\text{Res}_2 = \left\{ \max_j \{d(j)\} \right\}^{-1} \tag{4.15}$$

where

d(j) = duration of *j*-th failure event

In a series of papers Kundzewicz & Chalupka (1995), Kundzewicz & Kindler (1995) and Kundzewicz & Laski (1995) argued that the definition based on maximum value is better than the event based mean value, as the presence of small insignificant events may lower the mean value compared to the same situation but without the small events as illustrated by Fig. 3 in Kundzewicz & Kindler (1995), which might lead to estimates that are not defined uniformly in terms of increasing (decreasing) water demand or storage volume. For example, greater water demand should always lead to a greater (smaller) value of the estimate. The extent of non-uniform behaviour observed in practical estimation of resilience and vulnerability will be further discussed in section 4.5. Srinivasan *et al.* (1999) highlighted the same problem but further argued that using the maximum duration might mask resilient behaviour in the rest of the series compared to other series with shorter maximum duration but more non-resilient behaviour in the remaining series. The estimators of Res_1 and Res_2, as defined by Eq. (4.14) and Eq. (4.15), are adopted for further investigation.

Vulnerability

Vulnerability is a measure of the likely damage of a failure event and was defined by Hashimoto *et al.* (1982a) as

$$\mathrm{Vul} = \sum_{j \in F} e(j)v(j) \tag{4.16}$$

where

v(j) = the most severe outcome of the *j*-th sojourn in unsatisfactory state
e(j) = probability of *v(j)* being the most severe outcome of a sojourn into the unsatisfactory state

Hashimoto *et al.* (1982a) considered the system state at each time step and denoted by *v(j)* the worst value of the system variable in the *j*-th sojourn into the failure state. Jinno *et al.* (1995) based their vulnerability measure on the total water deficit experienced during the entire *j*-th sojourn into a failure state, i.e. deficit volume as defined in section 4.2. The later definition is better suited for analysis of water supply systems as the most severe state often is no water availability *v(j)* = 0. As a further simplification of of Eq (4.16), Jinno et al. (1995) considered the probability of each event to be equal, i.e. *e(1) = ... = e(M) = 1/M*, where *M* is the number of failure events. Therefore, they estimated vulnerability as the mean value of the deficit events *v(j)* as

$$\mathrm{Vul}_1 = \frac{1}{M} \sum_{j=1}^{M} v(j) \tag{4.17}$$

Afterward, they normalised Eq (4.20) with the total water demand from the deficit periods. This approach, however, is not followed here because the normalisation eventually can assign the same vulnerability to events of different magnitude. It is not the relative but the absolute deficit that is responsible for the damage. Again, Kundzewicz & Kindler (1995) argues that the maximum event as proposed by Moy et al. (1986) might be a better estimator than the event based mean value, i.e.

$$\mathrm{Vul}_2 = \max_j \{v(j)\} \tag{4.18}$$

where

v(j) = deficit volume of the *j*-th failure event

The estimators Vul_1 and Vul_2, as defined by Eq. (4.17) and Eq. (4.18) are adopted for further investigation.

As an alternative to the maximum observed value, estimates of resilience and vulnerability can be based on the *p*-th fractile in the empirical cumulative distribution function (cdf) or a standard cdf fitted to either the duration or the deficit volume of the observed failure events as

$$\begin{aligned} \mathrm{Res}_3 &= \left\{F_d^{-1}(p)\right\}^{-1} \\ \mathrm{Vul}_3 &= F_v^{-1}(p) \end{aligned} \tag{4.19}$$

where F_d and F_v are the cdf of duration and deficit volume, respectively. Difficulties associated with the use of standard cdf for modelling of duration and deficit volume of failure events from Southern African rivers have been reported by Kjeldsen et al. (2000), and in this study the use of empirical cdf was attempted only with p = 0.9.

4.3 SYSTEM EXPERIMENTS

The use of both reliability, resilience and vulnerability in a multiobjective analysis of scenarios requires consideration of the following aspects

1. The considered criteria should be defined uniformly in terms of increasing and decreasing water stress (Lane et al. 1997; Rogers et al., 1997).
2. Minimum overlap between criteria as this will introduce bias into the decision-making procedure (Rogers et al., 1997; Simonovic et al., 1997).

The uniformity of and overlap between estimators of R-R-V were investigated using the series of d and v extracted using behaviour analysis. Furthermore, it was found necessary to apply time series of extended length to obtain uniform estimates of resilience and vulnerability. These time series were generated using univariate stochastic time series models.

Behaviour analysis

The investigations of the statistical properties of R-R-V regarding uniformity and overlap are based on samples of failure events, *d(j)* and *v(j)*, extracted from time series using behaviour analysis as described by, for example, McMahon & Mein (1986). Time series of monthly runoff are routed through a reservoir with a specified storage volume *S(t)* and demand *D(t)* as

$$\begin{aligned} &S(t+1) = S(t) + Q(t) - D(t) \\ &S(t+1) < 0 \Rightarrow S(t+1) = 0 \\ &S(t+1) > S_{\max} \Rightarrow S(t+1) = S_{\max} \end{aligned} \tag{4.20}$$

where

S(t) = reservoir storage at beginning of time step *t*
Q(t) = Inflow to reservoir in time step *t*
D(t) = demand from reservoir in time step *t*
S_{max} = reservoir storage capacity

Surplus water is spilled downstream, and the reservoir is assumed to be full at the beginning of each simulation.

The effect of reservoir operation on R-R-V has been investigated by, for example, Hashimoto et al. (1982a), Kindler & Tyszewski (1989), Schumann (1997) and Chancelliere et al. (1998). In this investigation the reservoir is operated according to a standard operating policy as defined in Fig. 4.3.

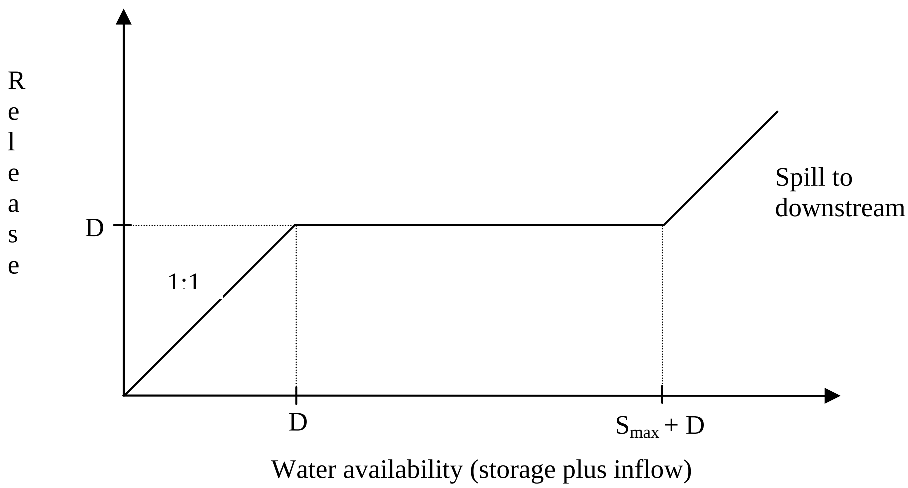


Fig 4.3 Standard reservoir operating policy.

Demand is always fulfilled unless water availability (storage ultimo last time step plus inflow during present time step) drops below water demand, in which case all available water will be delivered, hence the 45 degree slope of the rule curve. If available water exceeds water demand and storage capacity, the surplus is spilled downstream.

4.4 DATA MATERIAL

To investigate the behaviour of different estimators of R-R-V time series of monthly discharge from eight rivers in South Africa (4) and Zimbabwe (4), were collected. All gauging stations were reported recording natural flow, i.e. with a minimum of anthropogenic influence. The data series are chosen based on record length and data quality, especially concerning missing data. The Zimbabwean time series were patched using a rainfall-runoff model as reported by Kjeldsen et al. (2000). The South African series were patched using log-linear regression on a monthly basis with highly correlated neighbouring discharge series as recommended by Salas et al. (1980). The locations of the discharge gauging stations are displayed in Fig. 4.4 and annual and monthly flow statistics of the historical series shown in Table 4.1 and Table 4.2, respectively.

Table 4.1: Annual runoff characteristics of selected Southern African gauging stations.

River	Gauging Station	Country	Length of Record	Catchment area [km^2]	Mean annual runoff [10^6 m^3]	C_v of annual runoff	Percentage of zero years [%]
Palala	A5H004	South Africa	1956 – 95	638	71.5	1.19	0
Quencwe	R2H008	South Africa	1947 – 97	62	0.72	0.91	0
Mkomazi	U1H005	South Africa	1960 – 97	1743	67.0	0.53	0
Mpofana	U2H007	South Africa	1957 – 96	353	79.5	0.62	0
Munyati	C18	Zimbabwe	1960 – 95	2631	189.9	1.18	2
Umvumi	C41	Zimbabwe	1956 – 95	855	56.0	1.18	3
Mazowe	D28	Zimbabwe	1927 – 95	223	19.8	0.87	0
Mushagashi	E2	Zimbabwe	1929 – 94	541	43.3	1.10	3

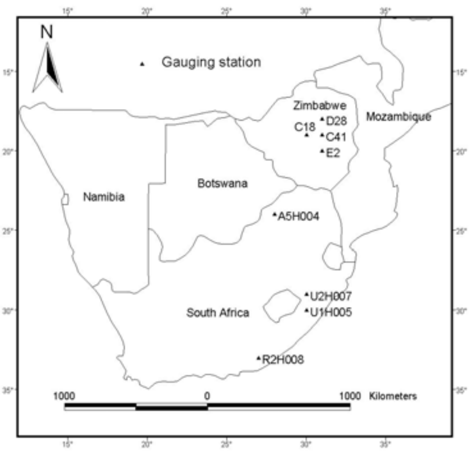


Fig. 4.4 Location of gauging stations.

For investigating the uniformity of the estimators, time series of historical monthly runoff is applied. For investigating the overlap between estimators and the required record length it was necessary to apply time series of monthly runoff generated from stochastic models estimated from the historical data. First, time series of annual runoff (transformed to normality) were modelled using ARMA(p,q) models. Afterwards, time series of monthly runoff were obtained using the disaggregation method described by Svanidze (1980) and known as the method of fragments. An in-depth discussion of the identification, estimation and validation of the univariate time series models estimated from the historical data is given in Appendix A.

Table 4.2: Monthly runoff statistics for the selected gauging stations

River	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
						Mean [10^6 m^3]						
A5H004	12.05	13.94	11.10	5.51	4.78	2.69	1.77	1.31	0.90	0.89	2.35	9.22
R2H008	1.04	1.01	1.13	0.64	0.43	0.08	0.09	0.36	0.16	0.46	0.89	0.58
U1H005	121.30	143.84	123.43	55.14	17.48	11.54	11.21	9.54	14.47	37.81	46.07	79.06
U2H007	14.88	14.36	13.62	7.85	4.14	2.72	2.52	2.07	2.53	3.96	4.72	8.56
C18	40.94	63.11	33.40	7.18	1.61	0.68	0.46	0.25	0.12	0.22	3.13	38.82
C41	14.27	16.29	9.88	3.08	0.76	0.35	0.25	0.15	0.10	0.30	1.44	9.10
D28	3.75	5.13	4.14	1.81	0.93	0.65	0.57	0.46	0.31	0.26	0.48	1.62
E2	9.65	12.08	7.78	2.89	1.40	0.68	0.41	0.21	0.10	0.07	0.48	5.48
						Standard deviation [10^6 m^3]						
A5H004	16.22	18.39	15.43	6.25	8.40	4.34	2.17	1.59	1.13	1.03	2.44	17.61
R2H008	1.79	1.59	1.79	1.29	1.56	0.09	0.15	1.36	0.31	1.02	1.81	0.77
U1H005	88.54	109.52	102.71	48.23	7.46	5.98	10.97	8.28	28.14	102.38	47.47	58.40
U2H007	13.28	11.90	12.24	6.46	3.34	1.35	1.69	1.01	2.70	6.11	3.68	8.56
C18	57.03	107.97	61.76	12.01	2.39	1.13	0.74	0.42	0.22	0.67	7.45	65.29
C41	20.65	23.05	15.63	6.55	1.55	0.66	0.42	0.25	0.16	1.02	2.98	15.62
D28	3.53	4.95	4.27	2.11	1.06	0.70	0.57	0.46	0.33	0.28	0.83	2.02
E2	14.93	16.79	10.80	3.90	2.04	1.01	0.61	0.36	0.21	0.19	1.01	9.72
						Correlation coefficient with previous month's flow						
A5H004	0.54	0.64	0.77	0.85	0.74	0.95	0.87	0.91	0.96	0.79	0.41	0.63
R2H008	0.33	0.39	0.57	0.52	0.09	0.10	0.15	0.36	0.12	-0.04	0.51	0.32
U1H005	0.49	0.64	0.65	0.48	0.67	0.41	0.39	0.46	0.21	0.97	0.87	0.06
U2H007	0.57	0.73	0.67	0.73	0.57	0.81	0.62	0.68	0.56	0.88	0.76	0.32
C18	0.41	0.56	0.42	0.58	0.85	0.92	0.99	0.98	0.98	0.73	0.19	0.72
C41	0.55	0.56	0.36	0.80	0.87	0.98	0.99	0.98	0.95	0.21	0.05	0.78
D28	0.49	0.65	0.75	0.90	0.93	0.98	0.99	0.99	0.98	0.85	0.36	0.19
E2	0.43	0.69	0.69	0.81	0.92	0.95	0.97	0.96	0.90	0.35	0.33	0.06
						Percentage of zero flows						
A5H004	0	0	2	0	2	2	5	5	10	15	0	0
R2H008	16	10	10	10	22	22	18	30	22	12	4	6
U1H005	0	0	0	0	0	0	0	0	0	0	0	0
U2H007	0	0	0	0	0	0	0	0	0	0	0	0
C18	0	0	3	6	6	6	6	9	11	9	3	0
C41	8	5	10	23	33	41	44	49	59	59	36	13
D28	0	0	0	1	3	3	3	3	3	4	6	1
E2	11	12	14	18	26	38	42	46	54	58	38	20

4.5 UNIFORMITY

To introduce the concept of a uniform estimator (Lane et al, 1999) consider a storage-yield-reliability relationship as shown in Fig. 4.5. If storage increases (decreases) or yield decreases (increases) the reliability will increase (decrease) over the entire range of the storage-reliability-yield curve, hence reliability is considered a uniform estimator with respect to storage volume and draft.

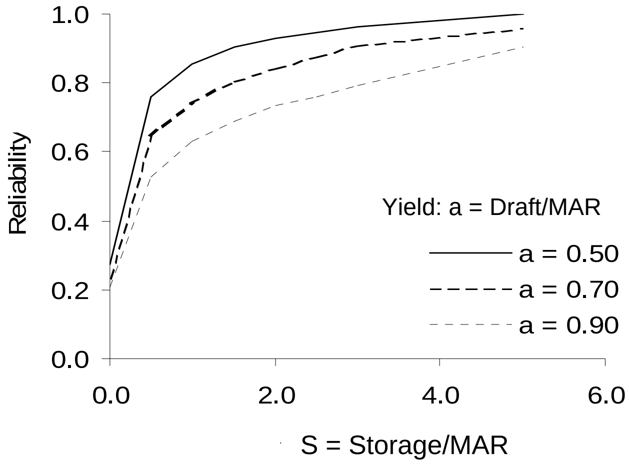


Fig. 4.5 Storage-Reliability-Yield relationship for gauging station E2.

For investigating the uniformity of the estimators of resilience and vulnerability, the following combinations of active reservoir storage capacity and draft, both normalised with mean annual runoff (MAR), have been used.

Storage: $S = 0.5, 1.0, 2.0$

Draft: $a = 0.55, 0.60, 0.65, 0.70, 0.75, 0.80, 0.85, 0.90, 0.95$

Each historical time series of monthly runoff is routed through the reservoir as described in section 4.3 and the number of failure month, the duration of each failure and the water deficit accumulated during each failure are recorded. Based on the statistics of the failure events sample estimates of resilience and vulnerability are obtained through Eq. (4.14) - Eq. (4.15), Eq. (4.17) - Eq. (4.18) and Eq. (4.19). The water demand is specified on an annual basis and afterwards disaggregated equally on each month. Fig. 4.6a to Fig. 4.6f show the sample estimates of resilience and vulnerability as a function of draft and active storage volume for each of the eight rivers.

When estimation is based on mean values of duration and deficit volume the sample estimates generally exhibit a non-uniform behaviour. Considering a specified storage volume, the sample estimates do not increase (decrease) monotonously as the draft increases (decreases). This non-uniform behaviour is observed for both resilience and vulnerability estimated through Eq. (4.14) and Eq. (4.17). Also, when considering resilience and vulnerability as a function of storage volume with a fixed draft, a similar non-uniform behaviour can be observed, see Fig. 4.6a and Fig. 4.6b. The same tendency can be observed when estimating resilience and vulnerability through Eq. (4.19), i.e. as the 0.9 fractile in the empirical cdf of failure duration and deficit volume, respectively, as evident from Fig. 4.6c and Fig. 4.6e. The use of maximum observed values of duration and deficit volume for estimation of resilience and vulnerability, Eq. (4.15) and Eq. (4.18), gives sample estimates that are uniform both with respect to draft and storage volume, see Fig. 4.6c and Fig. 4.7d.

The results from this investigation support the observations reported by Kundzewicz & Kindler (1995) that obtaining sample estimates of resilience and vulnerability based on mean values of failure duration and deficit volume using time series of historical length is not appropriate. Likewise, system experiments show that sample estimates obtained using the 0.9-th fractile of the empirical cdf of failure duration and deficit volume exhibit the same non-uniform behaviour. Maximum values appear to be more applicable in terms of uniformity, however, as noted by Vogel & Stedinger (1988) "*the distribution of that single value is quite unstable, just as the largest flood to occur in a n-year period has a very large sampling variance.*" Therefore, no recommendation can be made based on an investigation of uniformity alone. The non-uniform behaviour observed when using mean values or the 0.9-th fractile in the empirical cdf is related to the limited number of failure events experienced when using time series of historical extent. The estimation of both resilience and vulnerability could, therefore, be enhanced by the introduction of stochastic streamflow models. By generating long series of synthetic streamflow, the number of failure events can be increased leading to more robust estimates of resilience and vulnerability, both concerning mean values and maximum values. Hashimoto et al. (1982a) used 10 000 years of synthetic data for evaluation of R-R-V, Vogel & Bolognese (1995) used 100 million years of data and Vogel & McMahon (1996) used 30 million years of data. The record length required for robust estimation of resilience and vulnerability will be discussed in section 4.7.

4.6 OVERLAP

Overlap between different criteria in multiobjective decision-making may lead to inaccurate ranking of the considered scenarios (Simonovic et al., 1997). To illustrate the effect of overlap consider the comparison between two policy options with respect to two criteria *a* and *b*. The final ranking of the policy options is based on the product of the two criteria. If *a* and *b* are significantly correlated then *b* can be expressed as a linear function of *a* as $b = \alpha a$. If α is considered constant then the product of *a* and *b* is given as $ab = a(\alpha a) = \alpha a^2$, and the ranking of the two policy options is based on the outcome of criterion *a* only. Instead of being a multiobjective investigation with respect to a and b it is a single objective analysis. Hence, the existence of overlap effectively reduces the amount of information concerning system performance compared to using independent criteria. Rogers et al. (1997) investigated overlap between selected environmental indicators using principal component analysis. In this study, the overlap between different estimates of reliability, resilience and vulnerability is investigated by generating many synthetic time series of monthly runoff, estimating R-R-V for each time series and, finally, quantifying the degree of overlap in terms of correlation coefficients between series of sample estimates. The procedure for each of the eight rivers is outlined below:

1. Identify, estimate and validate stochastic streamflow model.
2. Generate synthetic time series of monthly runoff of historical length.
3. Route time series through reservoir with specified storage volume and water demand.
4. Estimate R-R-V through Eq. (4.12), Eq. (4.14) - Eq. (4.15), Eq. (4.17) - Eq. (4.18) and Eq. (4.19).
5. Repeat 2. – 4. 100 times.

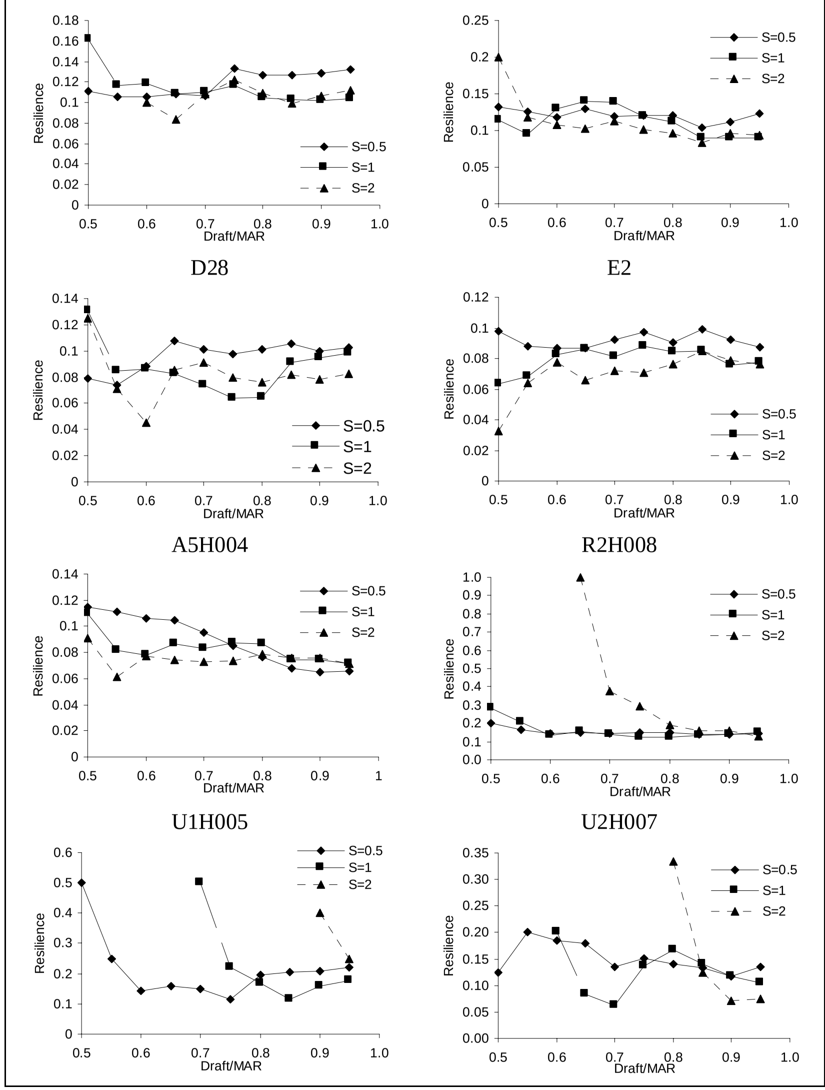


Fig. 4.6a Resilience Res_1 estimated through Eq. (4.14) using mean value of failure duration.

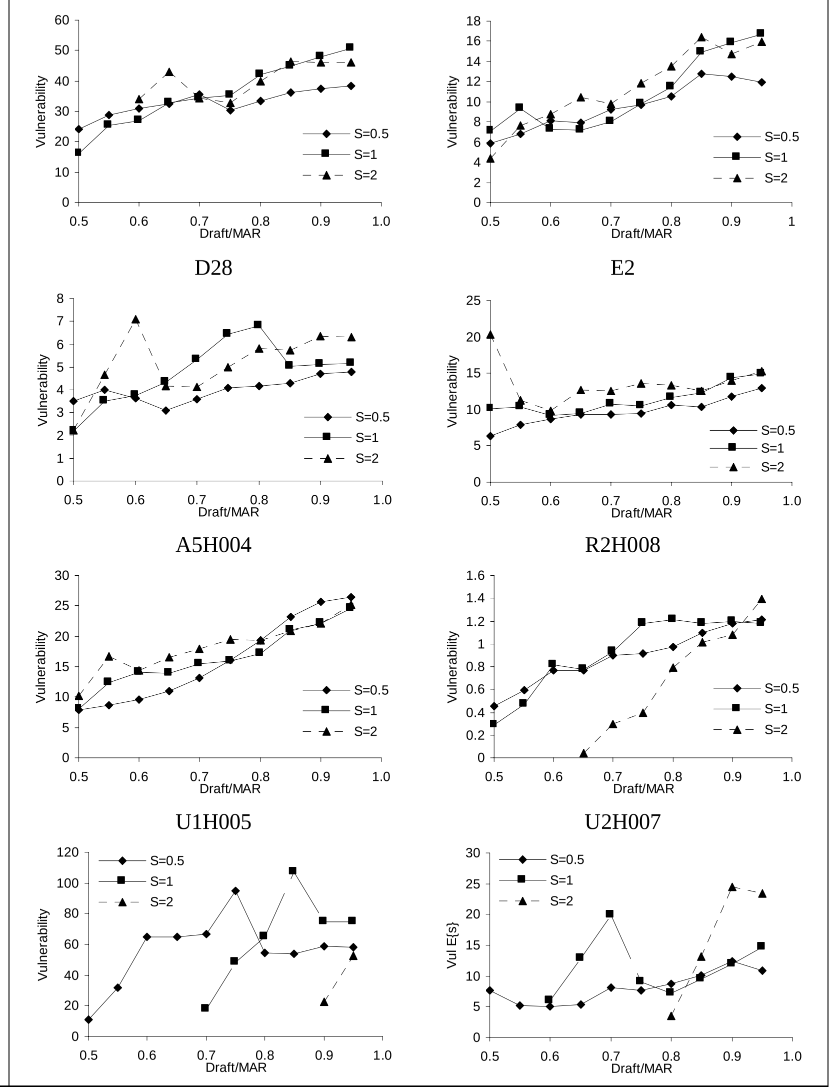


Fig 4.6b Vulnerability Vul_1 estimated through Eq.(4.17) using mean value of failure deficit volume. The unit of vulnerability is [m^3/s], corresponding to the runoff in one month.

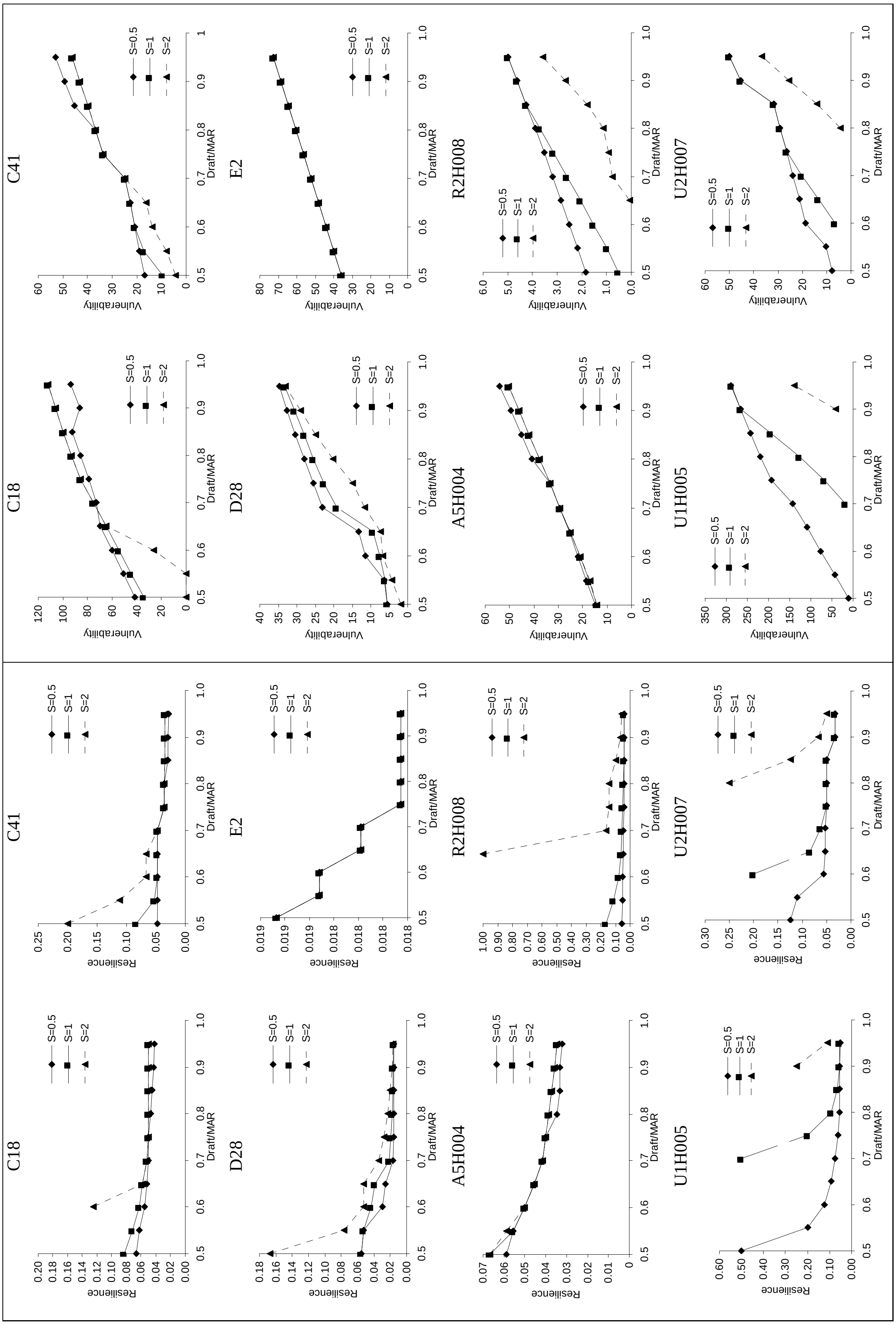


Fig. 4.6c Resilience Res_1 estimated through Eq. (4.15) using maximum failure duration.

Fig 4.6d Vulnerability Vul_2 estimated through Eq. (4.18) using maximum deficit volume. The unit of vulnerability is $[m^3/s]$, corresponding to the runoff in one month.

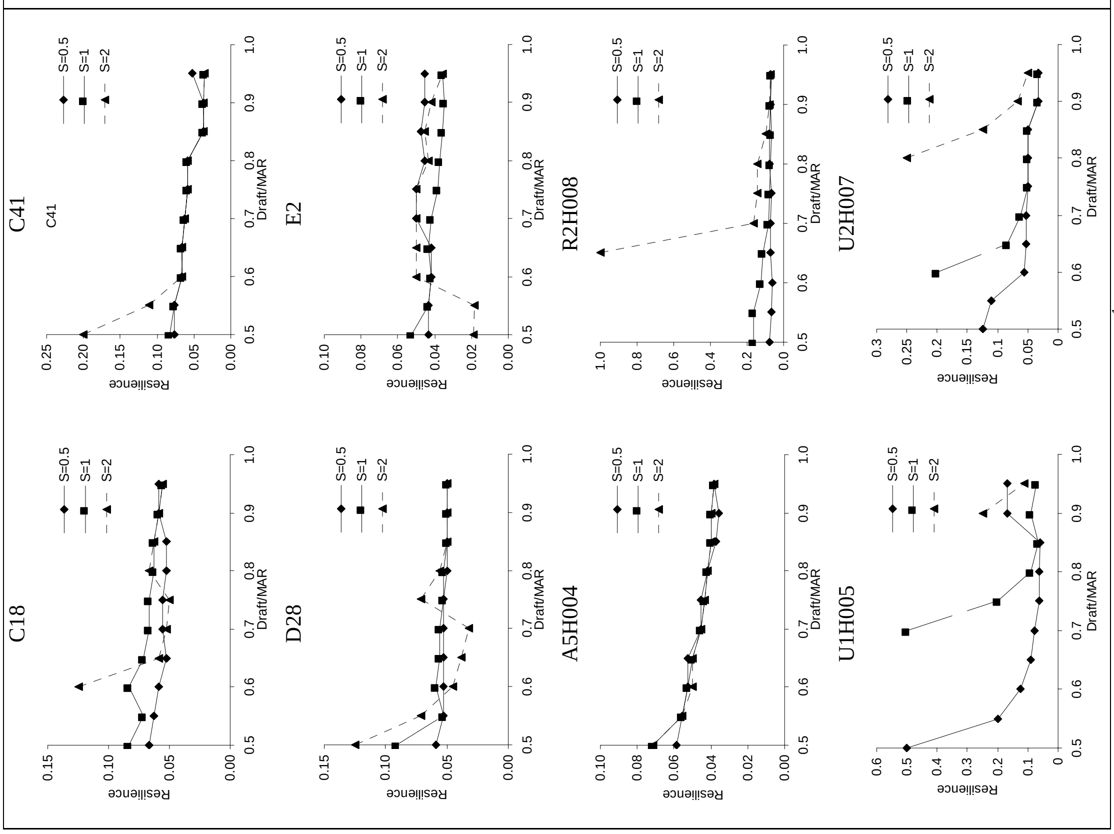


Fig. 4.6e Resilience estimated through Eq. (4.19) using F^{-1} (0.9) in the cdf of duration.

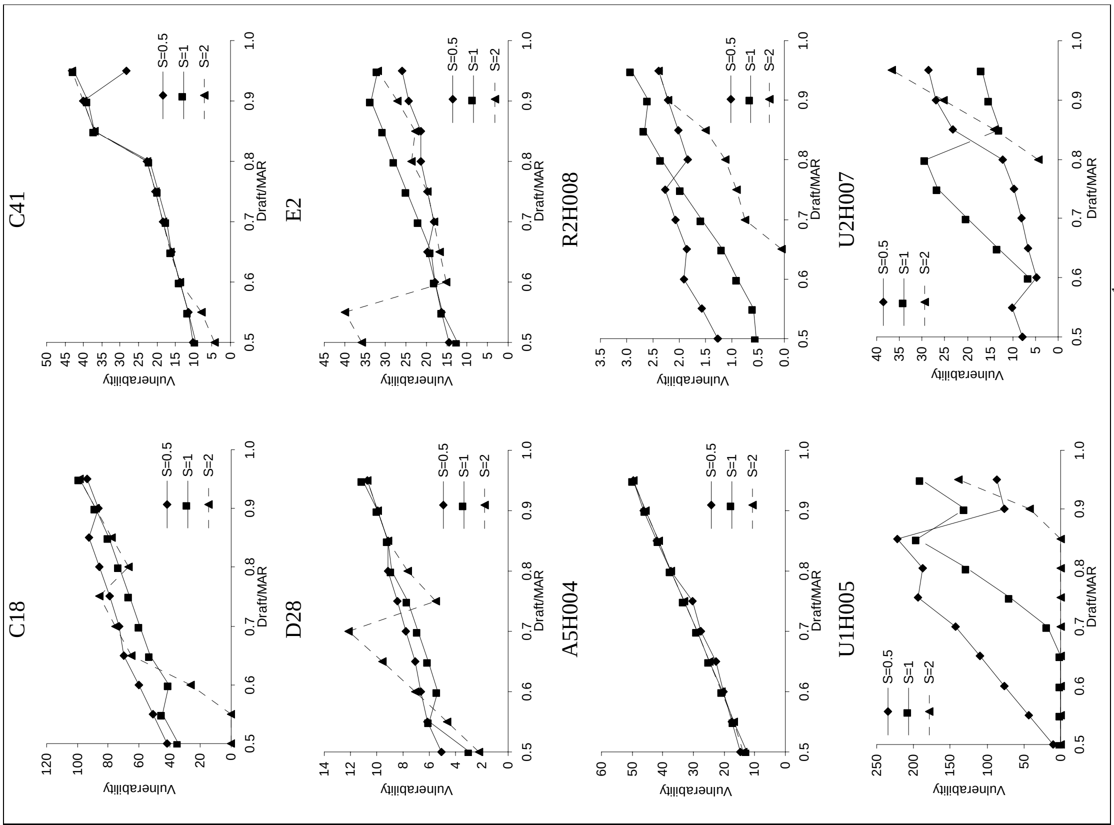


Fig 4.6f Vulnerability estimated through Eq. (4.19) using F^{-1} (0.9) in the cdf of deficit volume. The unit of vulnerability is [m^3/s], corresponding to the runoff in one month.

The identification, estimation and validation of the stochastic streamflow models are explained in-depth in Appendix A. The following combinations of storage volume and water demand have been used (both normalised with MAR).

Storage: S = 0.5, 1.0, 2.0
Draft: a = 0.5, 0.70, 0.90

The streamflow generation algorithm is also explained in Appendix A. The generated time series of monthly runoff are routed through the reservoir using behaviour analysis as described in section 4.2. For each combination of storage volume and water demand, the correlation coefficient between the 100 coherent estimates of R-R-V is calculated as

$$\rho = \frac{Cov\{\bullet,\bullet\}}{\sqrt{(Var\{\bullet\}Var\{\bullet\})}} \tag{4.21}$$

expressing the linear relationship between the estimates of R-R-V. The higher the value of the correlation coefficient is, the more the two considered estimators overlap each other thereby reducing the amount of information gained by using both criteria compared to using two independent criteria. The following combinations of estimators are investigated and the results plotted of each combination of storage volume and water demand.

Table 4.1 Combinations of R-R-V for investigation of overlap.

	Vul_1	Vul_2	Vul_3	Rel
Res_1	✓	✓	✓	✓
Res_2	✓	✓	✓	✓
Res_3	✓	✓	✓	✓
Rel	✓	✓	✓	

The inverse of the resilience estimators, rather than the estimators themselves, as presented in Eq. (4.14), Eq. (4.17) and Eq. (4.19), have been used in order to obtain linear relationship, hence making the correlation coefficient a useful measure of overlap. The adoption of the correlation coefficient measuring the linear relationship between the series of estimates should be interpreted as a general indicator of the degree of overlap rather than a strict measure of linear relationship.

Reliability-Resilience

As seen from Fig. 4.7d the correlation between reliability and resilience expressed through either mean values, fractiles or maximum values is of the same order of magnitude between -0.3 and -0.8. Considering the mean value of failure duration there appears to be a tendency of increasing numerical correlation with decreasing storage capacity and increasing water demand. The opposite mechanism is observed when considering the maximum value or the 0.9-th fractile of observed failure duration, i.e. increasing storage volume and decreasing water demand. The high negative correlation indicates that reliable systems tend to have a high degree of resilience (keeping in mind that the experiment considers the correlation between reliability and the inverse of resilience), which corresponds with the findings of others, for example, Kundzewicz & Kindler (1995) and Srinivasan et al.(1999). All three methods of estimating resilience gave correlation coefficients of the same order of magnitude and, based on these results, no recommendations as to which estimator of resilience to use together with the occurrence reliability can be given.

Reliability-Vulnerability

Fig. 4.7e shows the correlation coefficient between reliability and vulnerability expressed through mean values, the 0.9-th fractile and maximum values respectively. The order of magnitude of the correlation coefficient is generally between –0.3 to –0.8, i.e. the same order of magnitude as for reliability and resilience. The negative correlation indicates that systems with high reliability will have a low vulnerability, which corresponds with Kundzewicz & Laski (1995). The correlation coefficients behave similar to the pattern observed in the correlation between reliability and resilience, and no obvious choice of estimators of reliability and vulnerability can be identified.

Resilience-Vulnerability

Nine different combinations of resilience and vulnerability were investigated. Fig. 4.7a, Fig. 4.7b and Fig. 4.7c show the correlation coefficient between estimators of the inverse of resilience and vulnerability, i.e. each figure contains results from one of the rows in Table 4.1. In general the correlation coefficients between vulnerability and the inverse of resilience are positive and between 0.5 and 1.0. This shows that a system with high sample resilience will also have a low sample vulnerability, which corresponds with the findings of Kundzewicz & Laski (1995). It should be noted that the correlation coefficients between pairs of estimators based on the same summary statistics, such as mean value, maximum value or the 0.9-th fractile, are close to one, indicating a total overlap between resilience and vulnerability in these cases.

Based on this investigation of overlap it must be concluded that a significant correlation between reliability, resilience and vulnerability is evident and great care should be taken when selecting combinations of estimators to be used in combination with the sustainability criterion by Loucks (1997). Reliability is correlated with both resilience and vulnerability and the order of magnitude of the correlation appears independent of whether resilience and vulnerability are estimated using mean values, maximum values or the 0.9-th fractile, respectively. A higher correlation coefficient is found between resilience and vulnerability. In the case where estimators of both resilience and vulnerability are based on the same summary statistics, a correlation coefficient close to one is obtained, i.e. complete overlap. If the estimators are paired so resilience and vulnerability are estimated using different summary statistics, then a significantly lower correlation coefficient is obtained, but still higher than the correlation between reliability and resilience or vulnerability. However, no obvious answer to the most appropriate combination of estimators can be derived from this investigation alone, but will be discussed in section 4.8 in connection with the results from the other investigations.

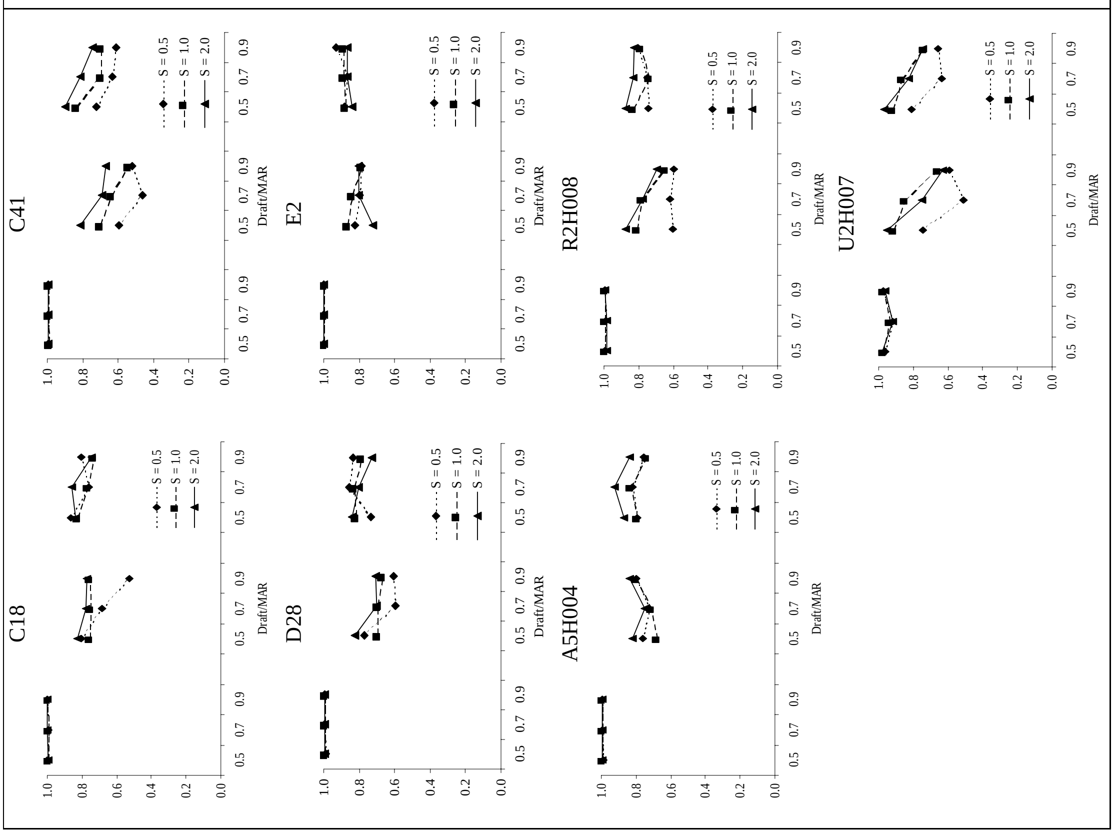


Fig. 4.7a Correlation between inv{Res$_1$} and Vul$_1$, Vul$_2$ and Vul$_3$ (mean, max, fractile).

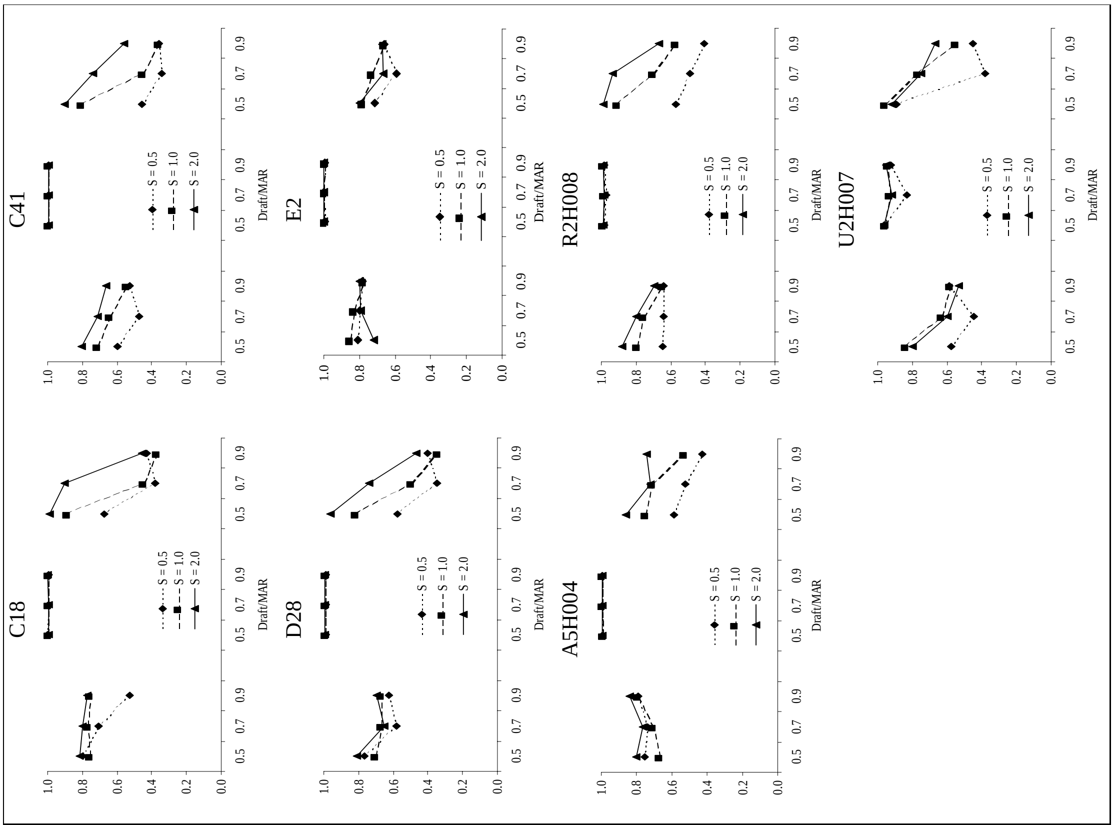


Fig. 4.7b Correlation between inv{Res$_2$} and Vul$_1$, Vul$_2$ and Vul$_3$ (mean, max, fractile).

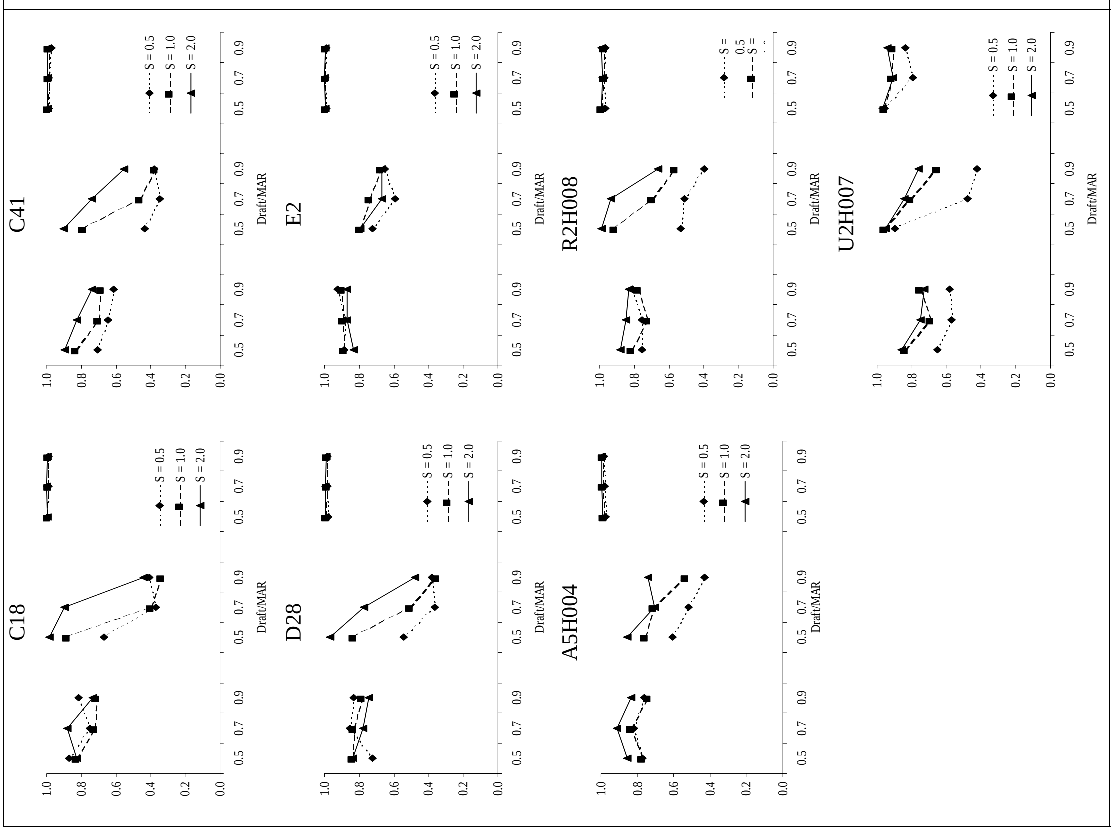


Fig. 4.7c Correlation between inv{Res$_3$} and Vul$_1$, Vul$_2$ and Vul$_3$ (mean, max, fractile).

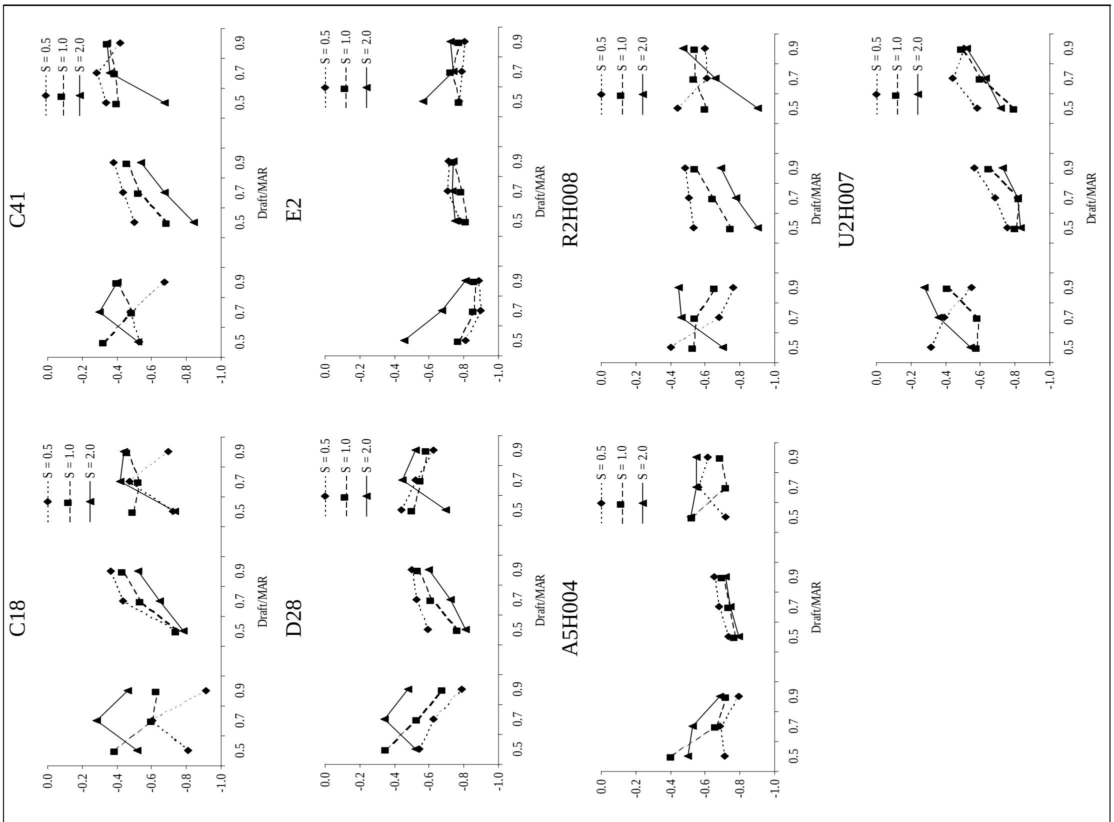


Fig. 4.7d Correlation between Rel and Res$_1$, Res$_2$ and Res$_3$ (mean, max, fractile).

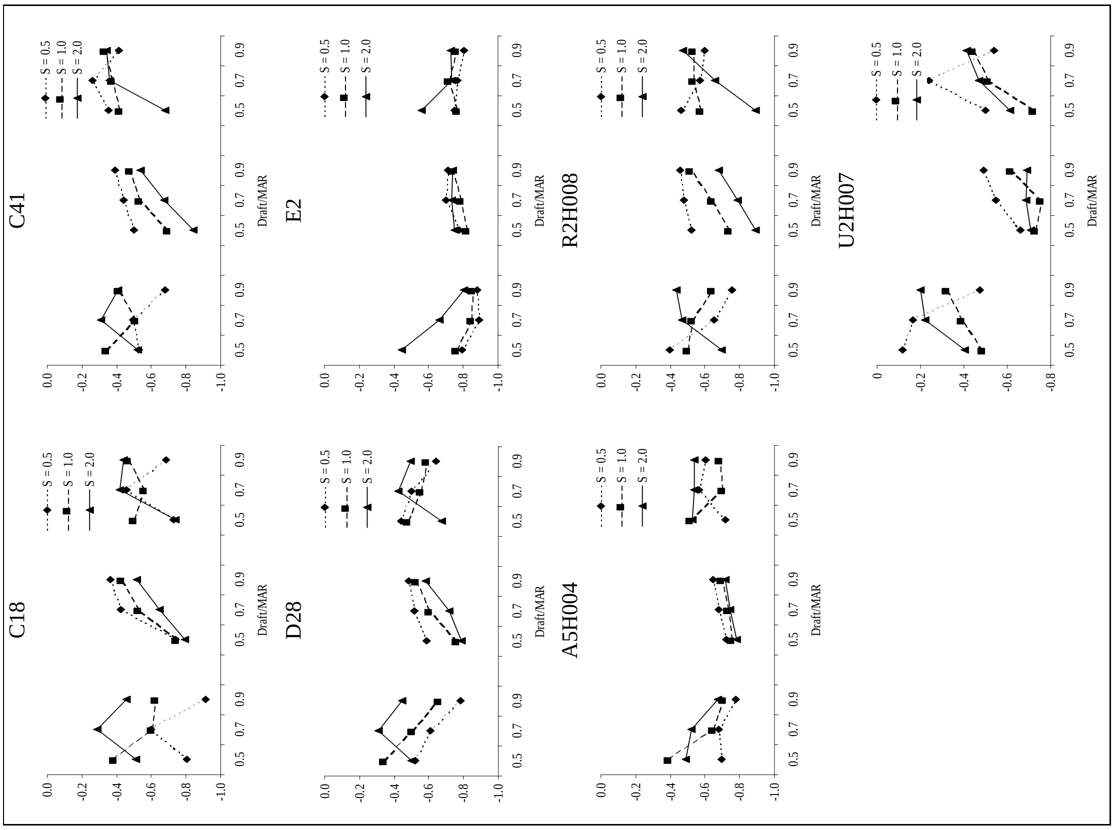


Fig. 4.7e Correlation between Rel and Vul_1, Vul_2 and Vul_3 (mean, max, fractile).

4.7 REQUIRED RECORD LENGTH

As evident from the investigations concerning uniformity the use of time series of historical length to estimate resilience and vulnerability leads to non-uniform behaviour when using mean values or the *p*-th fractile due to the limited number of failure events. To enhance the estimation of resilience and vulnerability, the stochastic streamflow models developed in Appendix A will be used to generate longer series and thereby more failure events. The objective of this investigation is to determine the record length required to give robust estimates of resilience and vulnerability. The procedure for estimating the required record length is outlined below:

1. Identify, estimate and validate stochastic streamflow model.
2. Select record length (T years).
3. Generate synthetic time series of monthly runoff with a record length of T.
4. Route time series through reservoir with specified storage capacity water demand.
5. Estimate R-R-V through Eq. (4.14), Eq. (4.17) and Eq. (4.19) (Res_1, Res_3, Vul_1 and Vul_3).
6. Go to 2.

As before the identification, estimation and validation of the stochastic streamflow models and the generation of synthetic time series are explained in Appendix A. The investigation is made considering a reservoir with a storage volume equal to MAR and the following water demand:

Water demand relative to MAR: a = 0.5, 0.7, 0.9

The following record lengths were considered:

T [years] = historic, 100, 200, 500, 1000, 5000, 10 000, 100 000

The required record length is assumed reached when a further extension of the time series does not significantly change the estimate of the resilience or vulnerability. Resilience and vulnerability defined in Eq. (4.15) and Eq. (4.18) as the maximum duration and deficit volume of observed failure periods, will increase as the record length increases, hence, no required record length can be determined and these estimators were not considered. The procedure outlined above is repeated for each of the eight rivers and the results shown for Res_1, Res_3, Vul_1 and Vul_3 in Fig. 4.8a to Fig. 4.8d. The general tendency is that the same pattern is observed in most rivers for all estimators and, therefore, tentative guidelines for the required record length can be given. For all rivers the estimates of resilience and vulnerability obtained using mean values and the 0.9-th fractile reach a constant level when using record lengths of 1000 years. More robust estimates might be obtained using longer records, but for practical reasons 1000 is the minimum required record length. This is of importance when considering the calculation time of the modelling system presented in chapter 5.

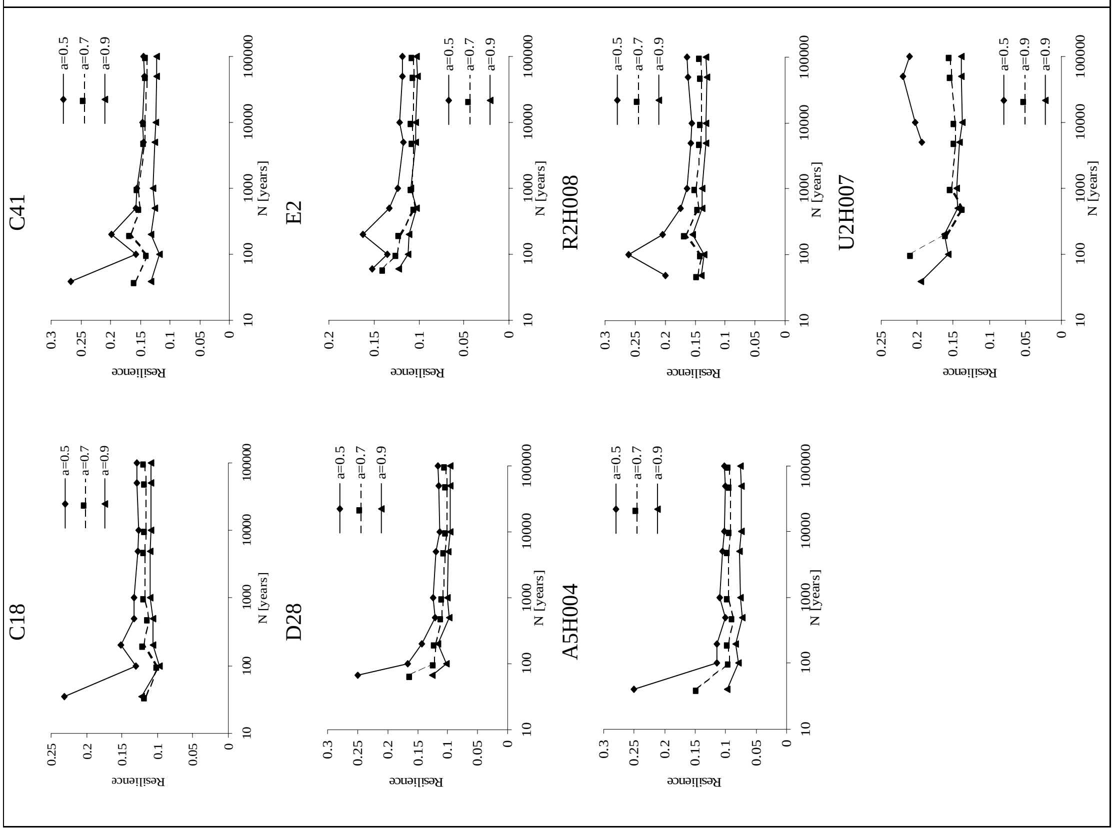


Fig. 4.8a Required record length for Res_1.

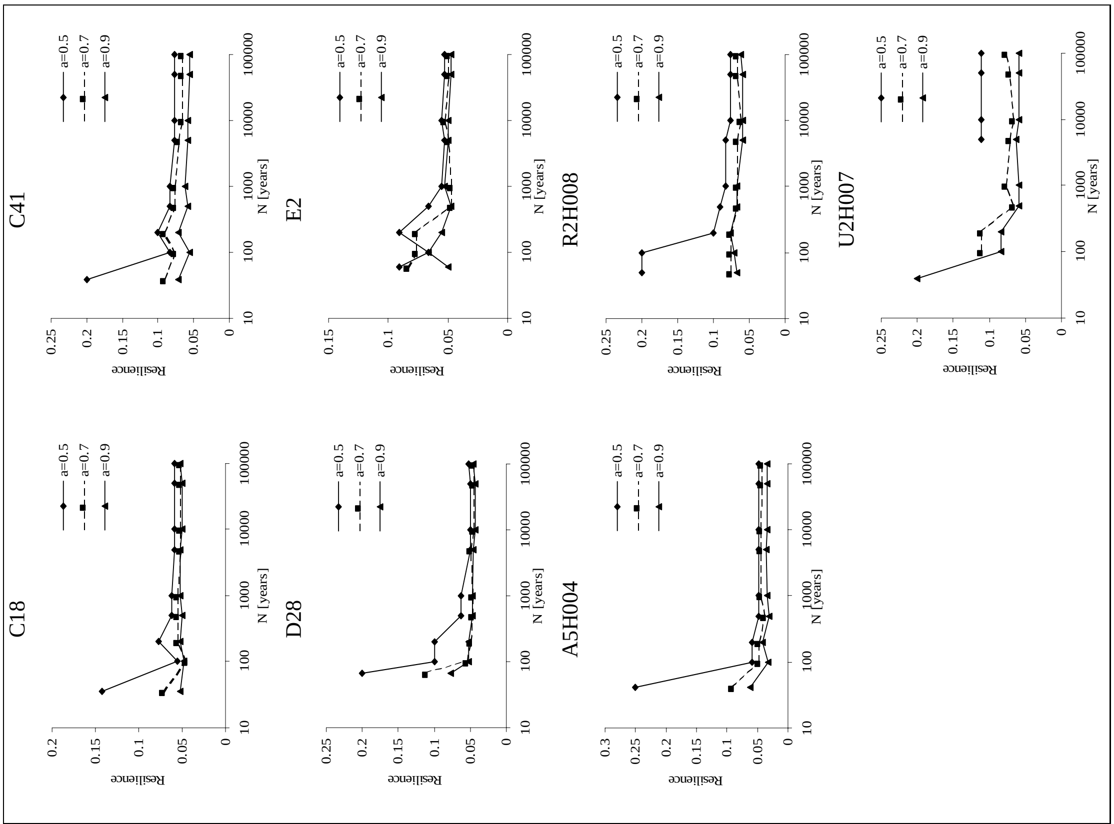


Fig. 4.8b Required record length for Res_3.

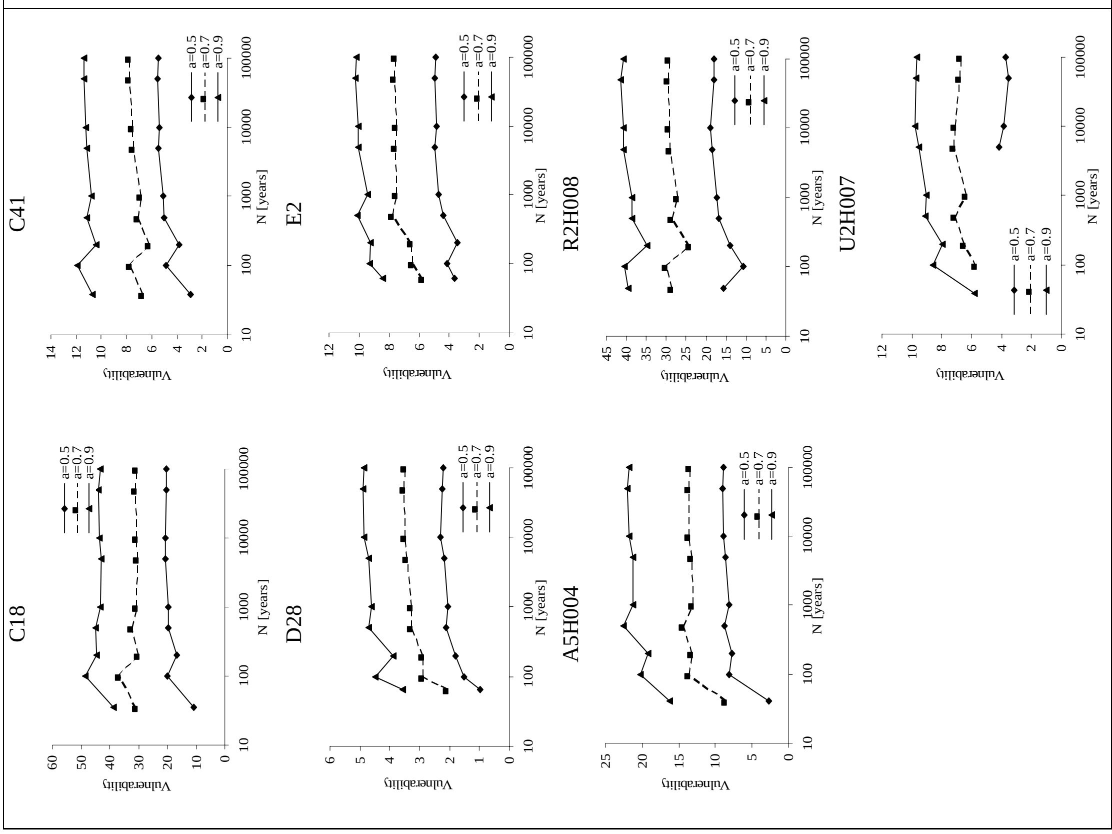


Fig. 4.8c Required record length for Vul_1. The unit of vulnerability is [m^3/s], corresponding to the runoff in one month.

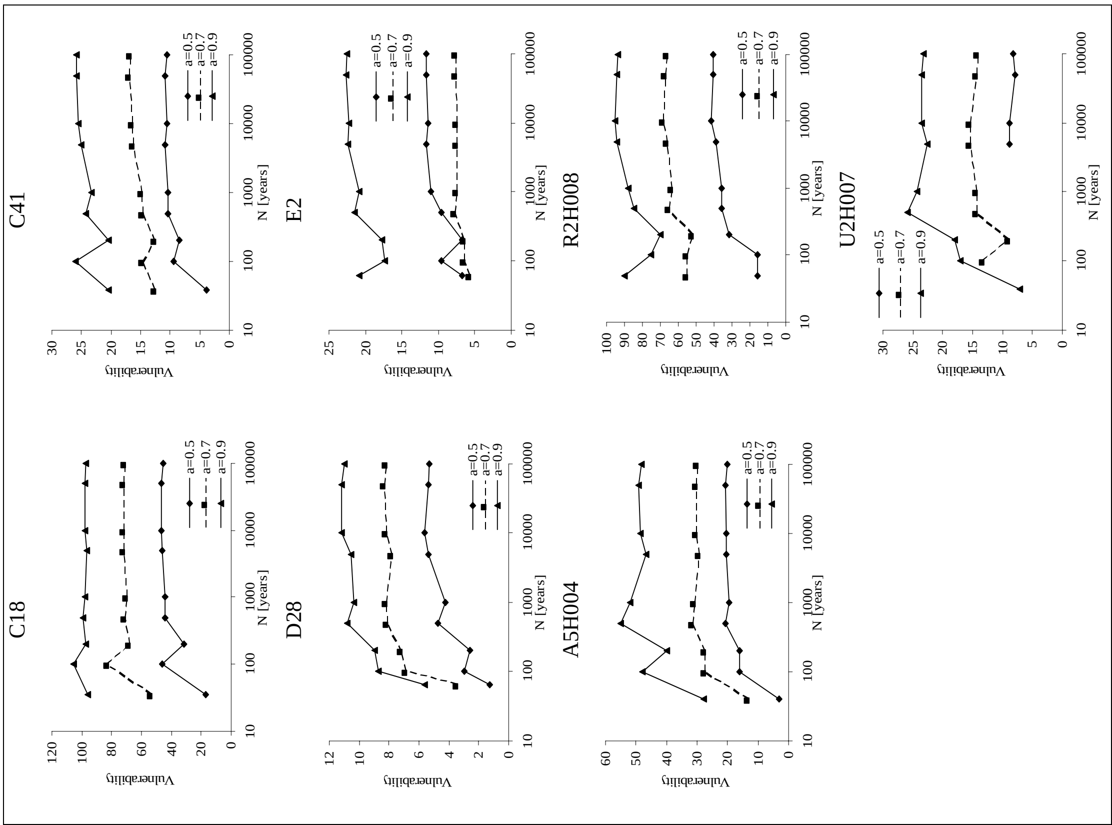


Fig. 4.8d Required record length for Vul_3. The unit of vulnerability is [m^3/s], corresponding to the runoff in one month.

4.8 SUMMARY OF R-R-V INVESTIGATION

Non of the conducted investigations on its own could give a clear answer as to the most appropriate combination of estimators of R-R-V to be used. Therefore, the final decision is based on a weighting of the results from each of the investigations. Using time series of historical length is problematic, especially when estimating resilience and vulnerability through the use of mean values or the *p*-th fractiles of failure event characteristics, as it leads to a non-uniform behaviour of the estimates. Using maximum values of duration and deficit volume proved to give more uniform estimates, however, criticism regarding masking of resilient behaviour in the remaining of the series and large uncertainties makes maximum values estimated from historical time series less appealing. The non-uniform behaviour of the estimators led to the investigation of the required record length, from which it is clear that the length of the time series used for estimating resilience and vulnerability should be at least 1000 years. Estimators based on maximum values were not considered together with the use of stochastic streamflow, as they are increasing as the record length increases, thus never converging towards a robust estimate.

Considering the overlap between R-R-V it became clear that combinations of resilience and vulnerability based on same summary statistics should not be used to avoid total overlap. The need for using estimators based on different summary statistics finally ousted the use of estimators based on maximum values as this estimator cannot be used together with stochastically generated long time series required by the remaining estimators. Returning to the comments by Simonovic et al. (1997) that overlap causes bias in the ranking of the scenarios it might be beneficial to abandon either resilience or vulnerability from the sustainability criterion in Eq.(4.1), alternatively together with abandoning reliability as well. Schumann (1997) recommended using reliability and vulnerability for analysis of reservoir systems. Similarly Simonovic (1992) used reliability and vulnerability in optimisation of reservoirs. On the other hand, keeping resilience in the assessment will further enforce the difference between scenarios with different risk characteristics, and as no total overlap was found additional information concerning system performance is gained by including resilience. Based on the lack of clear incentives for removing any of the risk criteria it was decided to keep all three in the assessment procedure. Based on these conclusions it is recommended that either resilience should be estimated based on mean values of failure duration and vulnerability on the 0.9-th fractile of the empirical distribution of deficit volumes, or vice versa, as these estimators have minimum overlap. The first of the two options was adopted in this study as it corresponds with the most widespread estimator of resilience. Thus, the following estimators were used

$$Sust(c,i,n) = \mathrm{Rel}(c,i,n)\,\mathrm{Res}_1(c,i,n)\left[1-\mathrm{rVul}(c,i,n)\right]$$

$$\mathrm{Rel} = 1 - \frac{\sum_{j=1}^{M} d(j)}{T}$$

$$\mathrm{Res}_1 = \left\{\frac{1}{M}\sum_{j=1}^{M} d(j)\right\}^{-1}$$

$$\mathrm{Vul}_3 = F_v^{-1}(p) \tag{4.22}$$

$$\mathrm{rVul}(c,n,i) = \frac{\mathrm{Vul}_3(c,n,i)}{\sum_i \mathrm{Vul}_3(c,n,i)}$$

$$rS_{WDC}(i) = \frac{1}{N}\sum_{n=1}^{N}\sum_{c=1}^{NC} w(c)Sust(c,n,i)$$

where *M* is the number of failure events, *d(j)* is the duration of the *j*-th failure and *v(j)* is the deficit volume of the j-th failure. These recommendations require the use of stochastic streamflow models to generate time series with a length of at least 1000 years.

4.9 MODELLING PROCEDURE

A number of practical issues concerning choice of modelling system and simulation procedure have to be considered when conducting a water resources policy analysis. The modelling procedure presented here is of general nature and, therefore, applicable independently of the chosen modelling system. More model-specific recommendations are presented in chapter 5.

Meaningful evaluation of R-R-V requires stationary conditions, i.e. static simulation. However, the temporal considerations required by Loucks' sustainability criterion involve systems changing over time, e.g. increasing water demand, i.e. dynamic simulation. To facilitate the estimation of R-R-V, a stepwise dynamic modelling approach has been adopted. The planning period *T* is divided into *N* periods each of length *T/N*. For a period ranging from $(i/N)T$ to $([i+1]/N)T$, the water demand is specified as the average demand from that period as illustrated on Fig. 4.9. For each period the R-R-V are evaluated by generating 1000 years of runoff and using the average demand of that period. In this study the monthly stochastic runoff sequences are generated for different runoff schemes within the catchment by first using a multivariate time series model, a CARMA(*p*,*q*) model as described by in section 5.2, to generate annual runoff and afterwards disaggregating the annual runoff into monthly runoff by using the method of fragments.

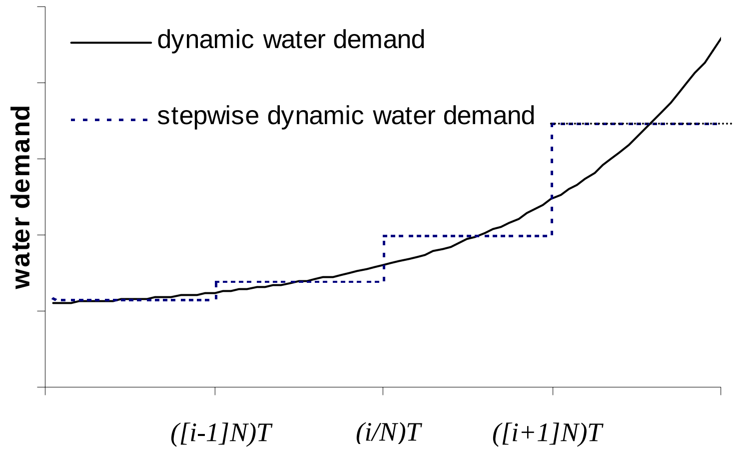


Fig 4.9 Modelling procedure for evaluation of modelling procedure.

The rapid impact assessment matrix (RIAM)

In a series of feasibility studies concerned with the application of R-R-V as a sustainability criterion Kjeldsen & Rosbjerg (2001a, 2001b) found the modelling procedure outlined above to be insufficient. A number of impacts of concern to a sustainability assessment of a series of water resources system development options could not be evaluated based on a water driven approach using the adopted modelling system (chapter 5), such as, e.g., loss of habitat, inundation of archaeological, historical and cultural sites, inundation of human settlements etc. In this study these impacts have been included in the sustainability by using an environmental impact assessment tool, know as the rapid impact assessment matrix (RIAM) presented by Jensen (1998), and applied by Makoni et al. (2001) as a component in a sustainability assessment of different reservoir development options. For the purpose of this study, the output of the EIA has to be a single aggregate number between zero and one, and the RIAM methodology has been modified accordingly as further elaborated later.

An abundance of EIA tools are available advocating different formal methodologies for evaluation and comparison development options. This is illustrated by the fact that Thompson (1990) was able to identify 24 different methodologies. He divided these methods into six different groups based on how each method fulfilled the following criteria:

1. Is the issue of impact significance handled separately from impact magnitude?
2. Are guidelines for determining the significant available?
3. Does the method apply information aggregation?
4. Are methods proposed to ensure public participation?

The RIAM methodology does differentiate between impact significance and magnitude and incorporate both in the assessment. Furthermore, the method does provide some guidance as how to determine impact significance. However, the method does not use information aggregation, but rather presents an overview of the impacts. In fact, Jensen (1998) defines the use of information aggregation and the thereof following lack of transparency as the major misgiving of most EIA methods. Finally, the method does not ensure public participation in the evaluation procedure.

Scoring system When conducting an EIA using RIAM the considered impacts are grouped into categories according to whether they can be considered physical/chemical, biological/ecological, social/cultural or economic/operational. The method is based on the use of a standardised scoring system for each of the considered impact providing a measure of the benefit/dis-benefit of the particular impact. The development options are evaluated against each other based on the scores obtained from all impacts. The standardised scoring system consists of two types of assessment criteria (Jensen, 1998)

1. Group A: criteria that are of importance to the considered impact, and which can individually change the score and which assess the importance of the impact.

   Importance of condition (A_1) A measure of the importance of the impact, which is assessed against the spatial boundaries or human interests it will affect.

   Magnitude of impact (A_2) A measures the magnitude of the impact, i.e. a scale of the benefits/dis-benefits of the impact.

2. Group B: criteria that are of value to the situation, but individually should not be able to significantly change the score obtained.

   Permanence (B_1) This defines whether an impact is temporary or permanent, and should be seen as a measure of the temporal status of the impact itself and not the effects following the impact.

   Reversibility (B_2) This defines the degree of reversibility of the effect of the impact, i.e. if the negative effects can be mitigated.

   Cumulative (B_3) a measure of the cumulative effect of the impact, i.e. if the effect is passed on in the system and has effect after the impact itself has ceased.

The entire scoring system for each criteria is shown in Table 4.3. A final assessment score (FAS) for each impact is estimated as

$$FAS = (A_1+A_2)(B_1+B_2+B_3) \quad (4.23)$$

Again negative FAS values indicate adverse impacts while positive values indicate beneficial impacts. The use of multiplication for group A criteria is important as it ensures a high score only if both criteria have high scores, i.e. adding a high weight to both criteria in line with the philosophy behind the water driven criterion based on R-R-V. Scores for the B criteria are added together to provide a single sum. This ensures that the individual values cannot influence the overall score, but the collective importance of all values is important (Jensen, 1998). Explicit consideration of issues such as magnitude, permanence and reversibility, though in a subjective format, bears resemblance to the use of the risk criteria R-R-V, hence the RIAM methodology is considered useful in combination with the water driven criterion defined in Eq. (4.22). The use of a pre-defined scale for assessing the magnitude and importance of each impact is believed to decrease the high degree of subjectivity and to increase the transparency. However, being an open-ended methodology, the impacts included in the analysis will still largely be determined by the analysts. Jensen (1998) presented six case studies, and in all cases the scoring was determined by the analyst, mostly based on preliminary interviews with involved stakeholders.

Table 4.3 Scoring system for RIAM methodology (Jensen, 1998).

Importance of condition (A_1)	Magnitude of change/effect (A_2)	Permanence (B_1)	Reversibility (B_2)	Cumulative (B_3)
4 = important to national/international interests	+3 = major positive benefit	1 = no change/not applicable	1 = no change/not applicable	1 = no change/ not applicable
3 = important to regional/national interests	+2 = significant improvement in status quo	2 = temporary	2 = reversible	2 = non-cumulative/ single
2 = important to areas immediately outside the local conditions	+1 = improvement in status quo	3 = permanent	3 = irreversible	3 = cumulative/ synergistic
1 = important only to local conditions	0 = no change/status quo			
0 = no importance	-1 = negative change in status quo			
	-2 = significant negative dis-benefit or change			
	-3 = major dis-benefit or change			

Information aggregation Jensen (1998) distinguishes between an EIA and an initial environmental evaluation (IEE) with the former being more superficial. The evaluation performed in this study is classified in an IEE. In order to compare the outcome of the RIAM analysis with the sustainability criterion described in section 4.8 and 4.9, the outcome of an IEE analysis have been transformed into a score between zero and one, denoted rS_{IEE}. This transformation was obtained based on the minimum geometrical distance as suggested by Loucks et al. (1981)

$$rS_{IEE}(i) = 1 - \frac{1}{4}\sum_{j=1}^{4}\left\{\sum_{g=1}^{NG(j)} w^2(g,j)\frac{\left(108 - FAS(g,j,i)\right)^2}{\left(108-(-108)\right)^2}\right\}^{\frac{1}{2}} \tag{4.24}$$

where i indicate scenario number, j is the impact category number (1 = physical/chemical, 2 = biological/environmental, 3 = social/cultural and 4 = economic/operational), NG is the number of impacts within each group and $FAS(g,j,i)$ is the final assessment score for the g-th impact in the j-th impact category. The weights $w(g, j)$ attached to the g-th impact in the j-th impact category are distributed equally among the $NG(j)$ impacts and sum up to one within each group. The number 108 is the maximum possible score in the RIAM methodology and the denominator is the distance between the best (108) and the worst score (-108) possible. The criterion rS_{IEE} ranges between zero and one with high scores being preferable over lower scores. The index IEE indicate that the criterion is calculated based on the IEE analysis. The scheme for aggregating multiobjective information into a number between zero and one applied above is equal to the scheme used by Fanai & Burn (1997) for their reversibility criterion presented in section 3.5.

Final sustainability score

Both the water driven criteria rS_{WDC} as defined in Eq. (4.22) and the initial environmental evaluation criterion rS_{IEE} in Eq. (4.24) range from zero to one with high scores being preferable over low scores. Next step in the procedure is a meaningful combination of the two criteria into one operational criterion. In the definition of his sustainability criterion Loucks (1997) argues that multiplicity of sub-criteria is preferable as it gives added weight to criteria having low values. Furthermore, if any of the sub-criteria equals zero the relative sustainability criterion is set to zero, i.e. no aspects of sustainability can be ignored, if a high score should be obtained. Therefore, final sustainability score for each scenario, $rS(i)$, is given as a product of the two criteria

$$rS(i) = rS_{WDC}(i) \cdot rS_{IEE}(i) \tag{4.25}$$

Estimates of rS will be in the interval between zero and one, with high scores indicating a high degree of sustainability.

4.10 DISCUSSION

This chapter has been concerned with the development of an actual water resources system assessment methodology based on the findings of the two previous chapters. Initially, the methodology suggested by Loucks (1997) and UNESCO (1999) was adopted as a potential assessment methodology, but a number of adjustments were introduced to enhance the practical applicability of the criterion. First, the use of maximum observed vulnerability to obtain the relative vulnerability for each user in each scenario leads to bias in the evaluation procedure. This has been corrected by using the sum of observed vulnerability to obtain the relative vulnerability of each user in each scenario. The temporal trend in water demand and/or water supply constitutes a problem, as the evaluation of R-R-V requires stationary conditions (Hashimoto et al., 1982a). Furthermore, as noted by Loucks (1997) " *Any worsening (or improving) situation in the future should not be hidden by including poorer future (or present) values with the better present (or future) values when calculating these statistical measures*". Hence, the existence of trend will lead to a sustainability criterion that is time dependent. To overcome the problem of non-stationarity, this study has adopted a stepwise dynamic simulation approach as shown in Fig. 4.9. For each time step the water demand was estimated as the average water demand for that period and R-R-V evaluated based on this average water demand. The result is an estimate of the sustainability criterion for each time step. To accommodate the temporal development of the sustainability criterion from different time periods, the averaging schemes designed for evaluation of intra- and inter-generational fairness by Matheson et al. (1997) were combined with the methodology proposed by Loucks (1997). This extended the method by Loucks (1997) to also consider the important aspects of inter-generational fairness. However, it should be noted that the criterion in its present form is an average over the scores of *rSust* over the adopted number of planning periods but does not consider the direction of change, i.e. if the criteria increase or decrease over time.

The relatively short time periods (10 years) used for evaluation of R-R-V in each time step, as adopted by Loucks (1997) and UNESCO (1999), has proven not to be viable due to the non-uniform behaviour of the estimates of resilience and vulnerability when based on mean values or the 0.9-th fractile of failure duration and deficit volume. The use of stochastic time series models for generation of extended synthetic time series of monthly runoff was found to give robust estimates of resilience and vulnerability. It was found that using time series with a length of 1000 years was adequate to obtain robust estimates of resilience and vulnerability for all rivers considered. A more in-depth investigation should consider the required record length as a function of reservoir size, water demand and C_v of inflow time series.

Several estimators of R-R-V were identified in the literature and the most suitable combination of these estimators was investigated. In order to avoid bias and to maximise the information concerning system performance, minimum overlap between R-R-V is desirable. A series of Monte Carlo experiments was conducted to quantify the degree of correlation between the different estimators. In the case where both estimators of vulnerability and the inverse of resilience were based on the same summation statistics, a correlation coefficient between vulnerability and the inverse of resilience close to one was observed. A lower correlation was obtained when comparing estimators obtained through different summation statistics. Based on these findings it was decided to estimate resilience through the mean value of failure duration and vulnerability as the 0.90-th fractile in the empirical cdf of observed deficit volumes, both defined in Eq. (4.22). The existence of correlation does not disqualify the R-R-V criterion, but the use of different risk criteria adds no or only little new information concerning system performance. The correction adopted in Eq. (4.5) concerning the estimation of the relative vulnerability was introduced to enhance the multiobjective aspect of the criterion. However, with the existence of significant correlation between the R-R-V the practical importance of this adjustment might be limited.

As reported by Kjeldsen & Rosbjerg (2001a, 2001b) the use of a water driven criterion for evaluation of water resources system sustainability does not allow for the inclusion of factors of importance to sustainability but not directly related to water demand and supply. The concepts of an existing EIA tool have been applied to introduce these factors into the analysis. This particular tool was selected based on the fact that its evaluation procedure is based on the same set of ideas as the water driven sustainability criterion. The existing tool was equipped with an averaging scheme for producing a single output value.

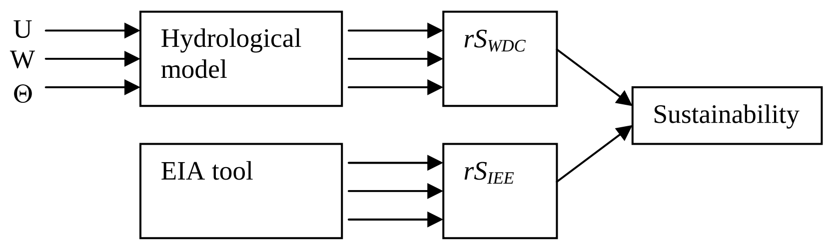


Fig. 4.10 Water resources system sustainability assessment procedure.

The present state of development of the methodology is illustrated in Fig. 4.10. For each scenario there is a number of factors influencing the sustainability of the water resources system. Some of the factors can be evaluated based on a comparison of water demand and water supply, but other have to be evaluated through an EIA procedure. The final sustainability score for the scenario is a combination of both the water driven criterion and the EIA. Future research into the topic could focus on how to decrease the number of factors that needs to be evaluated through the EIA and include them into the water driven criterion. This will often require more sophisticated modelling tools and a multidisciplinary team effort than was considered realistic for this study.

Chapter 5
MODELLING SYSTEM

This chapter introduces the practical water resources engineering tools applied in this study. An existing water resources planning and management tool, MIKE BASIN (DHI, 2000), has been coupled with a multivariate time series model for the generation of time series of monthly runoff. The application of stochastic runoff enables MIKE BASIN to estimate the sustainability criterion presented in section 4.1 using the modelling procedure outlined in section 4.9. As scenario analysis is an integrated part of the developed methodology, the use of scenarios as a planning tool in water resources management is outlined and methods for water demand predictions are reviewed.

5.1 RIVER BASIN MODELLING USING MIKE BASIN

MIKE BASIN is a mathematical representation of the river basin encompassing the configuration of the main river and their tributaries, the hydrology of the basin in time and space, as well as existing and potential major schemes and their various demand for water. The model is structured as a network model in which the rivers and their tributaries are represented by branches and nodes. The branches represent individual stream sections while the nodes represents confluences, locations where water activities occur or model results are required. Once the model is set-up for a river basin including hydrology, water user schemes, reservoirs and management, the water quantity mass balance is calculated in every branch and node and for every scheme. The model output consists of time series of information concerning performance of every scheme and simulated flow in every node and branch (DHI, 2000). Hughes (1992), Basson et al. (1994) and Basson & Van Rooyen (2001) have previously successfully tested a similar modelling approach under South African conditions. The MIKE BASIN model fits well into the system analysis framework for water resources as described in section 3.3 and the modelling procedure for estimation of the water driven sustainability criterion presented in section 4.8 and 4.9.

The model is set-up and operated via a user-friendly graphical user interface, making the model a potential powerful tool for enhancing communication between stakeholders involved in water resources management. This, however, is beyond the scope of this study. In addition, the model can be coupled to other external applications such as EXCEL for more advanced features such as optimisation of reservoir operation rules, see, for example, Dyrbak (2001). In this study, the MIKE BASIN model has been coupled with a stochastic multivariate time series contemporaneous ARMA(p,q) model, denoted a CARMA(p,q) model, for the generation of synthetic time series of monthly runoff. The properties of the CARMA(p,q) model are further elaborated in section 5.2.

Model components

In the following the different components in MIKE BASIN and the corresponding data requirements are described. A MIKE BASIN set-up consists of:

1. A physical layout of the river basin and the water resources system infrastructure.
2. Specification of runoff from incremental catchments.
3. Different water user schemes including their water demand.
4. Water resources infrastructure and management.

System Layout First it is important to know the physical layout of the catchment. Additionally, the different water users should be identified, see Fig. 5.1(a). Next, the river is digitised and nodes are inserted in points of interest, see Fig. 5.1(b).

Runoff schemes The model automatically generates runoff schemes between nodes in the model set-up, see Fig. 5.1(c), and the naturalised runoff from each of the schemes has to be specified. The term naturalised runoff refer to time series of runoff where the anthropogenic influence, such as water abstraction, reservoir development, land-use changes etc., has been removed. The method for retrieving time series of naturalised runoff depends on the purpose of the study and the features influencing runoff. In some simple cases the observed runoff can be used, but often this information is needed from ungauged catchments or concerning the influence of the above mentioned anthropogenic activities. In the latter cases the use of appropriate hydrological models is needed. Again, the choice of model rest with such factors as the objective of the study, data availability, preference of the analyst etc. and can range from simple statistical regression models to full scale numerical models of the entire hydrological cycle.

Water user schemes Two types of water user schemes can be specified in MIKE BASIN Water Supply and Irrigation, see Fig. 5.1(d). Water supply schemes require time series of water demand, fraction of demand covered by groundwater and return flow fraction. Irrigation schemes require the same information plus a linear routing coefficient for returning water to the river system. Depending on the level of aggregation involved in the definition of the irrigation schemes, these parameters will often be based on engineering experience or used as calibration parameters.

Infrastructure Data requirements for reservoirs can be divided into design and management data. Design data includes height-volume-area (HVA) relationship, time series of evaporation and precipitation, water user connections, seepage and downstream user loss factors. The HVA-curves can be specified either as table data or using pre-specified relationships between height and area and between height and volume, as

$$\begin{aligned} V(h) &= a(h-b)^c \\ A(h) &= d(h-e)^f \end{aligned} \tag{5.1}$$

where a,b,c,d,e,f are parameters, V is volume [10^6 m^3] and A is area [10^6 m^2]. In practise, however, often only a general relationship between area and volume is given as for example the area-volume relationship derived by Michell (1982) for large reservoirs in Zimbabwe as

$$A = 0.53V^{0.66} \tag{5.2}$$

where the unit of the area is [ha] and the unit of volume is [10^6 m^3]. A method for transforming these general relationships into relationships used in MIKE BASIN as specified in Eq. (5.1) is given in Appendix B.

The operation data includes operational control and user priorities. The operational control is related to the amount of water presently stored in the reservoir. The storage can be divided into five different zones, see Fig. 5.2.

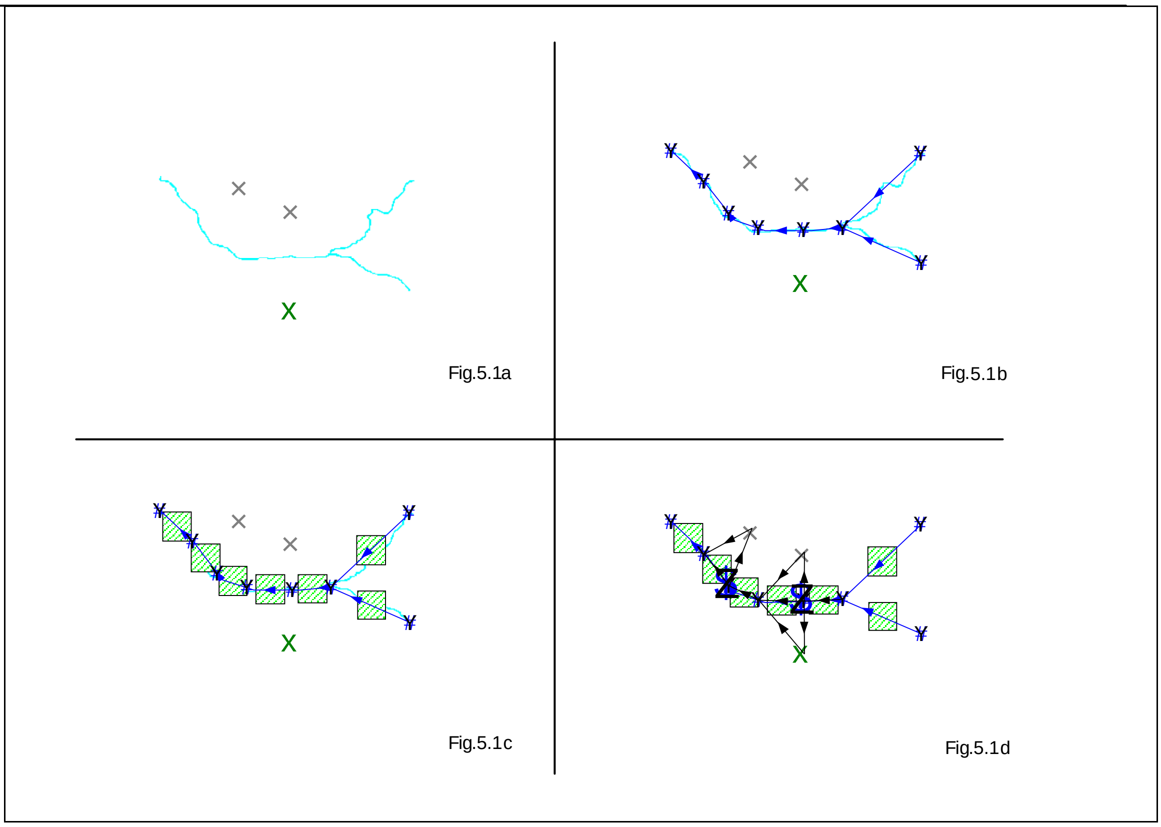


Fig. 5.1 Schematic setup of the MIKE BASIN model.

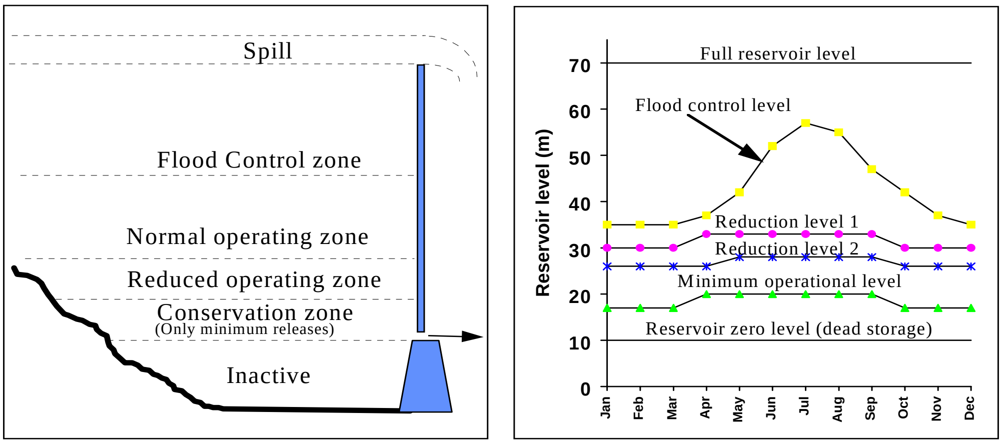


Fig. 5.2 Types of zones and operational rules for individual reservoirs (DHI, 2000).

Flood control zone This zone serves as a storage buffer to diminish the impacts of high floods. Under normal circumstances the water level in the reservoir is kept at this level to maintain optimal protection and reserve water for water supply. If the water level enters the flood control zone, a maximum downstream release, defined by the user, is released.

Normal operating zone In this zone all demands are fulfilled.

Reduced operating zone In this zone the demands are only partially fulfilled. Two reduction level curves and two corresponding reduction factors must be specified for each reservoir user connection. If the reservoir storage drops below reduction level 1 for a specific user, the actual abstraction is reduced according to the reduction factor 1. If the storage falls below reduction level 2, the abstraction is reduced according to the reduction factor 2.

Conservation zone In this zone only the specified minimum downstream release is maintained. No other abstractions take place.

Dead storage zone Water in this zone cannot be abstracted for water use or released downstream.

Priority system

Each water user in MIKE BASIN is given a local priority. If more users are drawing water from the same node or reservoir, the user with the highest priority will have a 100% fulfilment before the user with the second priority is allowed to draw water. There is no upper limit in the model to the number of users that can be attached to a specific water source.

Modelling principles

To apply a modelling system as MIKE BASIN for a policy analysis or evaluation of scenarios, a number of principles to reduce the complexity of the problem has to be introduced. Simonovic & Fahmy (1999) applied a series of such principles to model and evaluate different water policies in Egypt on a national scale. This study focuses on the evaluation of water policies at a catchment scale and, therefore, the principles have to be adjusted accordingly. The following principles have been applied in this study:

Simulation Principle According to Simonovic & Fahmy (1999) they applied simulation technique instead of optimisation technique as not all decision variables were under the full control of one stakeholder. The same principle is applied in this study.

Aggregation Principle The aggregation principle considers both the spatial and the temporal scale of modelling. Before starting the model set-up, it is important to define what is the appropriate representation of the river basin. An attempt to model every tributary and water user in a large catchment will often turn out to require enormous resources. It is therefore advisable to consider a schematisation that reflects the overall natural condition and are based on the objective of the modelling exercise. A principle of modelling at highest appropriate level of aggregation is therefore introduced as a guideline. Such a principle will assist the modeller in the set-up of the model. Generally, the aggregation of data will include:

1. Lumping of small rivers and tributaries into a single branch.
2. Lumping of small irrigation schemes into one single scheme.
3. Lumping of domestic and industrial water demand into one demand.
4. Lumping of small farm dams into one dam representing all dams in a certain area.

In difference to Simonovic & Fahmy (1999) the water users in MIKE BASIN are not grouped together into sectors but modelled as individual water users.

Water Use Principle The water use principle and the water driven approach have been discussed in section 3.5.

5.2 MULTIVARIATE TIME SERIES MODELLING

The use of stochastically generated runoff for the analysis of complex water resources systems requires a coupling between the MIKE BASIN model and a multivariate stochastic time series model. This chapter is a review of the adopted models whereas the actual application is described in each of the case studies. As discussed in section 4.4, the expected occurrence of months with zero flow during the dry season has lead to the adoption of a non-parametric disaggregation method, the method of fragments, for generation of monthly flow based on the generated annual flow. In this study, a multivariate extension of the method of fragments has been adopted.

As for univariate time series modelling, described in Appendix A, the development and use of multivariate time series models follow the six-step procedure outlined by Stedinger & Taylor (1982):

1. Obtain streamflow data.
2. Select models to describe marginal probability distributions of flow in different seasons and estimate the models' parameters.
3. Select appropriate model for the spatial and temporal dependence of the streamflow.
4. Verify the computer implementation of the model performs as specified.
5. Validate the model for water resources system simulation.
6. Use the model.

To obtain time series of naturalised monthly streamflow, which are added up to annual streamflow, at all sites under consideration, a physical-conceptual hydrological rainfall-runoff model, the ACRU model (Schulze, 1995), has been applied. Naturalised streamflow implies that all anthropogenic influences on streamflow, such as reservoirs and water abstraction, have been removed. The following discussion of the identification and estimation of multivariate time series models is divided into annual and monthly runoff, respectively.

Annual runoff

Consider a MIKE BASIN set-up including time series of annual runoff from M sites $q(i,t)$, i = 1, …, M and t = 1,…, $T(i)$ where $T(i)$ is the number of year in time series number i. The first step in the model selection procedure is to select an appropriate marginal probability distribution to model the annual runoff from each site. Based on the findings of the univariate modelling, the time series of annual runoff from each of the M sites are either Box-Cox transformed according to Eq. (A7) or transformed according to the LN3 distribution as in Eq.(A5)-Eq.(A6). The actual method of transformation is based on the outcome of the PPCC test, as described in Appendix A. Thus, at each site the 2-parameter normal distribution is adopted as marginal probability distribution together with either a site specific Box-Cox parameter λ or the ξ parameter in the LN3 distribution. In the following transformed annual runoff at site i at time t is denoted $x(j,t)$.

Next, consider the spatial and temporal correlation structure in the annual time series. Salas et al. (1985) defined the three types of relationship between two time series $x(i,t)$ and $x(j,t)$ contemporaneous, unidirectional and feedback. A contemporaneous relationship implies that interaction between $x(i,t)$ and $x(j,t)$ is instantaneous, see Fig. 5.3 and prevails when, for example, streamflow at several stations in a region have no connection by either natural courses or man made intervention but are impacted by the same regional precipitation. According to Salas et al. (1985) time series of annual runoff from conditions as described above will have a contemporaneous correlation structure.

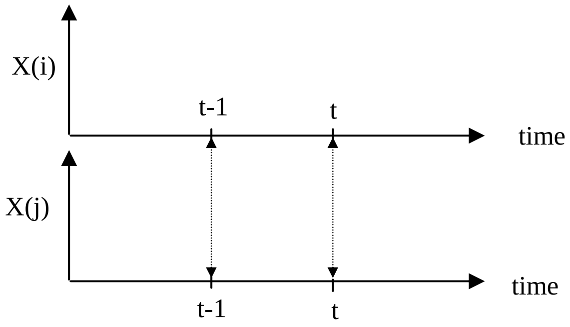


Fig. 5.3 A contemporaneous relationship between two time series *x(i)* and *x(j).*

A unidirectional relationship is when a time series *x(i,t)* causes another time series *x(j,t),* hence past and present values of *x(i,t)* are useful for prediction of present and future values of *x(i,t)* and exists, for example, when modelling real time streamflow at two gauging stations on the same river. Previous measurements from the upstream station may be useful to predict streamflow further downstream (Salas et al., 1985). A unidirectional lag-one correlation structure is shown in Fig. 5.4.

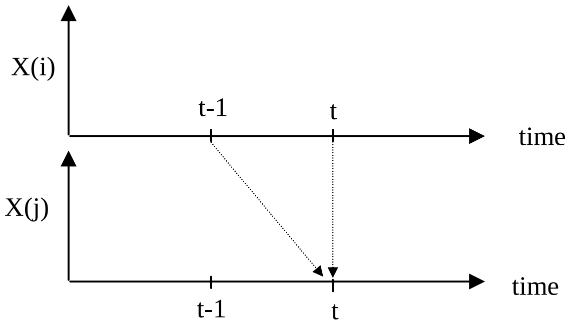


Fig 5.4 Unidirectional relationship between two time series *x(i)* and *x(j)* illustrated for lag zero and lag one correlation.

A feedback relationship between two time series *x(i,j)* and *x(j,t)* implies that a unidirectional relationship exists in both directions, i.e. that *x(i,t)* causes *x(j,t)* and vice versa. According to Salas et al. (1985) a feedback relationship may be useful when modelling, for example, evaporation and precipitation over large tropical catchments. The correlation structure for a lag-one feedback correlation structure is shown in Fig. 5.5.

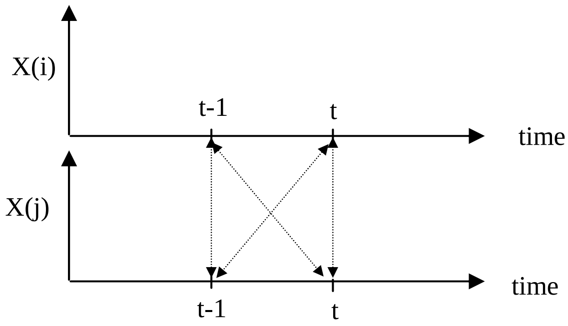


Fig 5.5 A Feedback relationship between two time series *x(i)* and *x(j)* illustrated for lag one.

As the runoff from the *M* sites in the MIKE BASIN model is naturalised runoff and is modelled independently of each other, a model with a contemporaneous correlation structure is adopted, a CARMA(*p*,*q*) model, for modelling of transformed annual streamflow. The CARMA(*p*,*q*) model is a simple case of the general multivariate ARMA(*p*,*q*) model given as

$$(\mathbf{x(t)}-\boldsymbol{\mu}) = \sum_{i=1}^{p} \boldsymbol{\varphi}\mathbf{(i)}(\mathbf{x(t-i)}-\boldsymbol{\mu}) + \sum_{i=1}^{q} \boldsymbol{\theta}\mathbf{(i)}\boldsymbol{\varepsilon}\mathbf{(t-i)} + \boldsymbol{\varepsilon}\mathbf{(t)} \tag{5.3}$$

where $\mathbf{x(t)} = \{x(1,t),\ldots,x(M,t)\}^T$ with mean value $\boldsymbol{\mu} = \{\mu(1),\ldots,\mu(M)\}$ and $\varepsilon\mathbf{(t)} = \{\varepsilon(1,t),\ldots,\varepsilon(M,t)\}^T$ is normally distributed with mean value of zero and Δ as the variance-covariance matrix. The parameter matrices of the CARMA(*p*,*q*) model are diagonal and given as

$$\phi(i) = \begin{bmatrix} \phi(1,1,i) & & & \\ & \phi(2,2,i) & & \\ & & \ddots & \\ & & & \phi(M,M,i) \end{bmatrix}, \; i = 1, \ldots, p \tag{5.4}$$

$$\theta(i) = \begin{bmatrix} \theta(1,1,i) & & & \\ & \theta(2,2,i) & & \\ & & \ddots & \\ & & & \theta(M,M,i) \end{bmatrix}, \; i = 1, \ldots, q \tag{5.5}$$

Hence, unidirectional and feedback mechanisms between runoff from different sites are not considered. The independent elements have zero mean and are uncorrelated in time but are contemporaneously correlated in space

$$\boldsymbol{\Delta} = Cov\{\varepsilon(i,t), \varepsilon(j,t+l)\} = \begin{cases} c(i,j), & l = 0 \\ 0, & l \neq 0 \end{cases}, \quad i,j = 1, \ldots, M \tag{5.6}$$

where l is the lag time in years and $c(i,j)$ is the covariance between $\varepsilon(i,t)$ and $\varepsilon(j,t)$. According to Hipel & McLeod (1994) a CARMA(p,q) model can be considered as M univariate ARMA(p,q) models linked together only by the variance-covariance matrix of the normally distributed element $\varepsilon(i,t)$. The order of the CARMA(p,q) model is defined as the maximum order observed among the M univariate ARMA(p,q) models, hence $p = \max\{p(1), \ldots, p(M)\}$ and $q = \max\{q(1), q \ldots, q(M)\}$.

Parameter estimation

According to Stedinger et al. (1985) and Hipel & McLeod (1994) the method of moments (MOM) should not be used for estimation of parameters in a multivariate time series model. Instead, both studies advocate for the maximum likelihood (ML) method. Stedinger et al. (1985) recommended using the unconditional maximum likelihood (UML) estimator, as described by Box & Jenkins (1976), to estimate the parameters of each univariate model as the ML estimator performed better than the MOM estimator, especially when correlation between time series was high as in the case of runoff from within the same region. To estimate the parameters of an ARMA(p,q) model using the UML estimator, consider the general univariate ARMA(p,q) model

$$x(t) + \phi(1)x(t-1) + \ldots + \phi(p)x(t-p) = \varepsilon(t) + \theta(1)\varepsilon(t-1) + \ldots + \theta(q)\varepsilon(t-q) \tag{5.7}$$

where $\theta^T = \{\theta(1), \ldots, \theta(p), \phi(1), \ldots, \phi(q)\}$ are the unknown parameters and $\mathbf{x}^T = \{x(1), \ldots, x(T)\}$ contains all observations. The independent element $\varepsilon(t)$ is distributed according to a normal distribution with variance $\hat{\sigma}_\varepsilon^2$. The likelihood function L is equal to the T-dimensional normal distribution of $\mathbf{x}$ conditional on $\boldsymbol{\theta}$ and σ_ε^2 and given as

$$L(\mathbf{x} \mid \boldsymbol{\theta}, \sigma_\varepsilon^2) = f(x \mid \boldsymbol{\theta}, \sigma_\varepsilon^2) \tag{5.8}$$

The UML estimator of $\boldsymbol{\theta}$ is obtained by minimising the sum of squares SS with respect to the parameter array $\boldsymbol{\theta}$ (Box & Jenkins, 1976), i.e.

$$\min_{\boldsymbol{\theta}} \left\{ SS(\boldsymbol{\theta}) = \sum_{t=-\infty}^{T} \varepsilon_t^2(\boldsymbol{\theta}) \right\} \tag{5.9}$$

The sum of squares $SS(\theta)$ in Eq. (5.9) is calculated using backcasting as described by Box & Jenkins (1976). An Excel function containing the backcasting algorithm has been developed and the minimisation problem solved using a build-in optimisation routine in Excel based on a gradient method.

The cross correlation structure of the residuals of the transformed annual runoff is specified through the variance-covariance matrix Δ as defined in Eq. (5.6). An estimate of Δ was obtained by first calculating the *M* time series of the innovations $\varepsilon(j,t)$ as

$$\varepsilon(i,t) = \left(x(i,t) - \mu(i)\right) - \sum_{j=1}^{p} \phi(i,i,j)\left(x(i,t-j) - \mu(i)\right) - \sum_{j=1}^{q} \theta(i,i,j)\varepsilon(i,t-j) \tag{5.10}$$

From the residual series the residual cross correlation coefficient (RCCC) is estimated for different lags as

$$\rho(i,j,l) = \frac{c(i,j,l)}{\sqrt{c(i,i,0)c(j,j,0)}} \tag{5.11}$$

where *c(i,j,l)* is the estimated cross covariance between series *i* and *j* and at lag *l* and is estimated from the residual series as

$$c(i,j,l) = \begin{cases} \dfrac{1}{T}\displaystyle\sum_{t=1}^{T-l} \varepsilon(i,t)\varepsilon(j,t+l), & l \ge 0 \\ \dfrac{1}{T}\displaystyle\sum_{t=1-l}^{T} \varepsilon(i,t)\varepsilon(j,t+l), & l \ge 0 \end{cases} \tag{5.12}$$

As defined in Eq. (5.6), when considering a CARMA(*p*,*q*) model, $c(i,j,l) \neq 0$ for $l = 0$ and $c(i,j,l) = 0$ for $l \neq 0$.

To check if the estimated CARMA(*p*,*q*) adequately models the observed correlation structure between residuals of the time series of the transformed annual runoff Hipel & McLeod (1994) recommend plotting the RCCC, defined in Eq. (5.11) as a function of lag time *l* for each pair of time series. If estimated values of $\rho(i,j,l)$ do not exceed the approximate 95% confidence interval around zero given as $\pm 2T^{1/2}$ then the CARMA(*p*,*q*) model is sufficient otherwise the more general ARMA(*p*,*q*) model should be used.

Monthly runoff

A multivariate extension of the method of fragments was presented by Basson et al. (1994). At each of the *M* sites under consideration a set of fragments *f(t,i,j)* is estimated for each hydrological year as

$$f(i,t,j) = \frac{q_m^h(i,t,j)}{\sum_{j=1}^{12} q_m^h(i,t,j)} \tag{5.13}$$

where $q_m^h(i,t,j)$ is the historical runoff at site *i* (*i* = 1, …, *M*), in year *t* (*t* = 1,….*T(i)*) and in month *j* (*j* = 1,…,12). Annual runoff q*(i,t)* is generated at the *M* sites. For a number of key stations *nk*, the sum of squares between simulated $q^s(i,t)$ and observed $q^o(i,t)$ is calculated as

$$\min_t \left[SS(t) = \sum_{i=1}^{nk} \left(q^s(i,t) - q^o(i,t) \right)^2 \right] \tag{5.14}$$

For each simulate annual runoff, $q^s(i,t)$ the hydrological year *t* that minimises *SS(t)* is chosen for disaggregation at all *M* sites. A similar approach has been adopted in this study, however, the number of key-stations has been set equal to the total number of sites *M*, thus

$$\min_t \left[SS(t) = \sum_{i=1}^{M} \left(q^s(i,t) - q^o(i,t) \right)^2 \right] \tag{5.15}$$

Finally, the monthly runoff $q_m(i,t,j)$ at the *i*-th site, in the *j*-th month in year *t* is estimated using the generated annual runoff $q^s(i,t)$ and the derived fragment *f(i,t,j)* as

$$q_m(i,t,j) = q^s(i,t)\, f(i,t,j) \tag{5.16}$$

Model verification and validation

The model verification and validation of a multivariate time series model is essentially the same as for the univariate models. The procedure is similar to the verification-validation procedure explained in Appendix A and is based on a sample of *N* = 5000 generated time series of historical length. From each of the *N* = 5000 generated time series the verification and validation statistics of interest were estimated. From the *N* = 5000 estimated verification/validation statistics the *5-number summary* (Hipel & McLeod, 1994) is estimated, i.e. the maximum and minimum values and the 0.25, 0.50 and 0.75 quantiles. If the historical estimates of the verification/validation statistics fall within the 0.25 to 0.75 quantile of the generated values, then the estimated model is assumed to perform satisfactorily. Basson et al. (1994) warn that a limited number of validation statistics should be selected, otherwise the validation tasks becomes too cumbersome even for a modest number of time series in a multivariate analysis.

Generation of runoff

The generation of synthetic runoff requires a random number generator for the generation of N(0,Δ) distributed residuals ε(t) in Eq. (5.3), where Δ is the variance-covariance of the residuals and specified in Eq. (5.6). The actual generation of correlated runoff at each of the *k* sites is mainly adopted from Hipel & McLeod (1994) and carried out as outlined below.

1. Determine the lower triangular matrix **B** by Cholesky decomposition of the variance-covariance matrix **Δ**

$$\mathbf{\Delta} = \mathbf{B}\mathbf{B}^T \tag{5.17}$$

2. Use the random number generator to obtain *M* realisations *e(i,t)* of a U(0,1) distributed random variable.

3. Transform *e(i,t)* into realisations η from a NID(0,1) random variable as

$$F\big(\eta(i,t)\big) = e(i,t) \Leftrightarrow \eta(i,t) = F^{-1}\big(e(i,t)\big) \tag{5.18}$$

   where *F(•)* is the cdf of the 2-parameter normal distribution.

4. The variance-covariance structure is added to the NID(0,1) elements $\eta(i,t)$ to obtain a realisation from the multivariate normal distribution describing the residuals $\varepsilon(i,t)$ defined in Eq. (5.10)

$$\varepsilon(i,t) = \sum_{j=1}^{i} b(i,j)\eta(i,t) \tag{5.19}$$

   where the *b(i, j)*s are elements in the matrix **B** obtained through Cholesky decomposition of the variance-covariance matrix **Δ**.

5. Obtain the transformed annual runoff *x(i,t)* at each site *i* = 1,…,*M* by inserting the residuals from Eq. (5.19) into Eq. (5.3).

6. The actual annual runoff *q(i,t)* at each site is obtained through inverse transformation of *x(i,t)* according to the transformation procedure initially applied (Box-Cox or LN3) at the site under consideration.

7. Find the set of fragments corresponding to generated annual runoff *q(i,t)* and calculate the monthly runoff in each months through Eq. (5.15) to Eq. (5.16).

8. Go to 2. until a time series of sufficient length has been generated.

9. Repeat the sequence defined by 2. – 7. for as many time series as required.

The computer routines required for Cholesky decomposition, generation of U(0,1) realisations (*e*) and transformation of these into NID(0,1) realisations (η) have been adopted from Press et al. (1992). To alleviate the influence of initial values, the first 100 generated values were discharged before the simulation of each time series. This also helps to minimise the influence from a generated time series on the following time series.

The transformation of time series of annual runoff *q(i,j)* into time series of normally distributed realisations using any transformation T is defined as

$$T(Q \mid \alpha) \rightarrow Y \in N(\mu, \sigma^2) \tag{5.20}$$

where α is a set of transformation parameters. The distribution of annual runoff will have a lower bound at zero, as negative runoff cannot occur, but the normal distribution is unbounded as illustrated in Fig. 5.6.

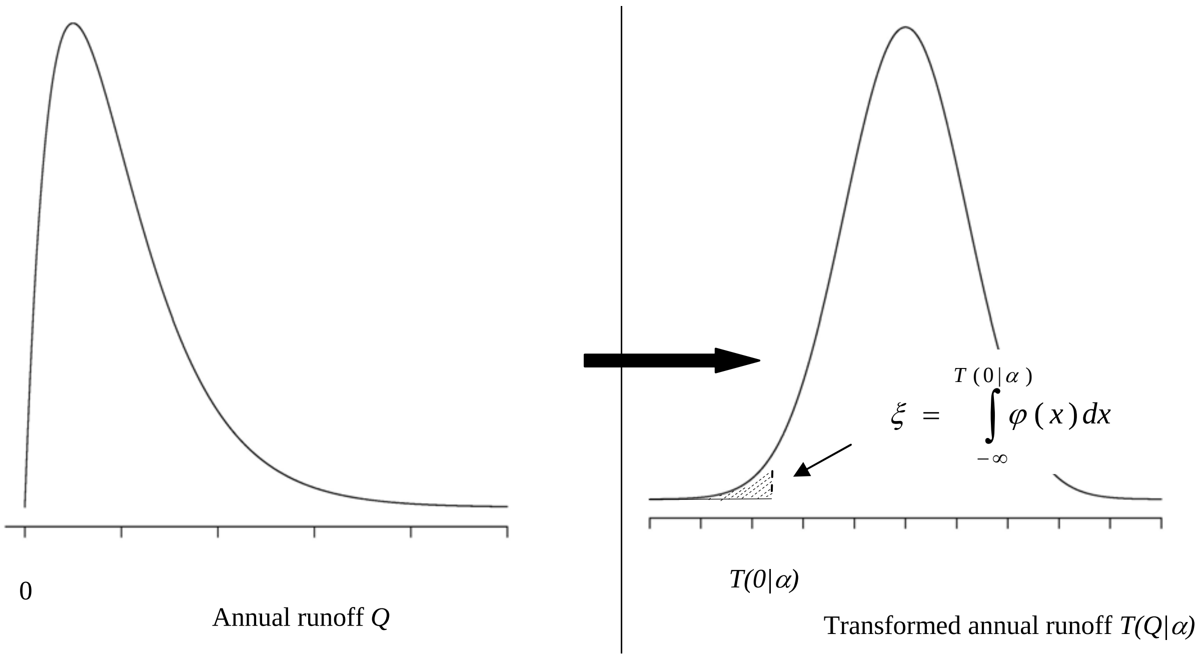


Fig. 5.6 Error committed by transformation of annual runoff.

As the normal distribution is unbounded, apart of the probability mass ξ will fall below the lower limit defined by the transformation as *T(0,α)*. When generating realisations of transformed runoff a fraction corresponding to the probability mass ξ will lead to undefined values of annual runoff, thus a censoring of the generated data is needed. The type of censoring depends on the adopted marginal probability distribution of annual runoff. In the case of the LN3 distribution, Basson et al. (1994) recommended changing the sign of generated negative values followed by division by ten. No corresponding recommendations were identified in the case of the Box-Cox transformation. In this study a generated values below *T(0|α)* is set equal to *T(0|α)*.

Model verification

The streamflow models will be verified by examining if the implemented version is able to reproduce MAR, standard deviation of annual runoff at each site. Furthermore, the model should be able to reproduce the specified cross-correlation structure between series of annual runoff. The correlation structure specified in the model is the cross correlation between the residuals of the transformed time series of annual runoff. As noted by Stedinger (1981) it is correlation between actual streamflow rather than the correlation structure of the multivariate normal distribution that is of interest in the planning situation. The actual lag-zero cross correlation between annual runoff is not specified in the model and, therefore, investigated as a part of the model validation.

Model validation

As for the univariate models (appendix A) the estimated model's ability to reproduce the mean and standard deviation of monthly runoff, the correlation between successive month at the same site and the maximum observed deficit volume at each site are all considered model validation. Two additional criteria for validation of multivariate time series models are the ability to reproduce the lag-zero correlation between annual runoff at different sites and the correlation between drought events at different sites, respectively.

The lag-zero cross correlation between annual runoff a different sites $r_a(i,j)$ is estimated as

$$r_a(i,j) = \frac{\sum_{t=1}^{n}\left(q(i,t)-\bar{q}(i)\right)\left(q(j,t)-\bar{q}(j)\right)}{\left[\sum_{i=1}^{n}\left(q(i,t)-\bar{q}(i)\right)^2 \sum_{j=1}^{n}\left(q(j,t)-\bar{q}(j)\right)^2\right]^{1/2}} \tag{5.21}$$

where $q(i,t)$ is the annual runoff at site i (i = 1,…,M) in year t (t = 1,…,T).

For estimation of correlation between drought events at different sites, Stedinger et al. (1985) defined a hypothetical bottomless reservoir and a demand D for each of the M sites. The required storage capacity in each time step was calculated as

$$w(i,t+1) = \max\{0, w(i,t) + D - q(i,t)\} \tag{5.22}$$

where $q(i,t)$ is the inflow to the reservoir during time step t. Hence, the cumulative drought impact at each site in each time step is denoted $w(i,t)$. According to Stedinger et al. (1985), a reasonable measure of multivariate drought correlation is the correlation coefficient between drought impact $w(i,t)$ and $w(j,t)$ denoted $r_d(i,t)$ and estimated as

$$r_d(i,t) = \frac{\sum_{t=1}^{n}\left(w(i,t)-\overline{w}(i,t)\right)\left(w(j,t)-\overline{w}(j,t)\right)}{\left[\sum_{t=1}^{n}\left(w(i,t)-\overline{w}(i,t)\right)^2 \sum_{t=1}^{n}\left(w(j,t)-\overline{w}(j,t)\right)^2\right]^{1/2}} \tag{5.23}$$

Stedinger et al. (1985) used the criterion in connection with annual runoff. However, in this study it will be used on a monthly time scale.

5.3 LINKING MIKE BASIN TO THE MODELLING PROCEDURE

For the MIKE BASIN model to be used with the modelling procedure outlined in section 4.9 for estimation of the water driven part of the sustainability assessment rS_{WDC}, the model was linked with three external procedures. The following three programs were written in the programming language C:

1. Generation of multivariate time series of monthly runoff.
2. Prediction of average water demand for each modelling period.
3. Estimation of R-R-V for each user.

In each of the three procedures the necessary parameters can be specified. The entire evaluation procedure was linked together in EXCEL using Visual Basic for Applications as schematically illustrated in Fig. 5.7, where

i = i-th planning period
N = Total number of planning periods each of duration 10 years

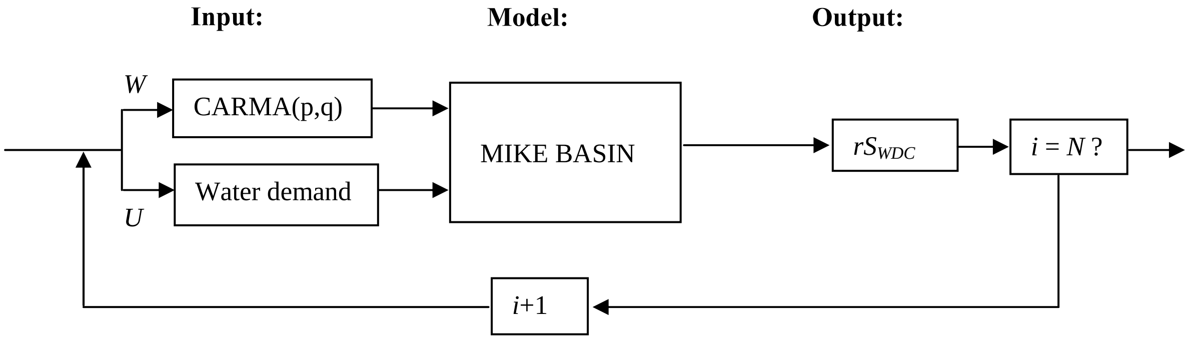


Fig. 5.7 Schematic setup of the entire modelling system.

As can be seen by comparing Fig. 3.1 with Fig. 5.7, the developed modelling system lend itself well to the system analysis framework by Duckstein & Parent (1994). All user demands are specified as a predetermined monthly time series.

5.4 SCENARIOS

The use of scenarios in water resources planning for assessing the impacts of different management options has been recommended and found widespread use in the water resources literature such as Simonovic & Fahmy (1999) and Acreman (2000). Often the scenarios are constructed by altering certain model parameters in a calibrated hydrological model and comparing the resulting model output, making the difference between scenario planning and a sensitivity analysis rather fuzzy. Frequently a scenario is defined as being “not a forecast but rather a possible future state” (Porter, 1985). Despite the apparent popularity of scenario planning in water resources literature it is necessary to turn to the economic literature for a formal introduction to and definition of scenario planning. A number of practical issues related to the use of the scenario planning technique needs to be considered. Planning is often divided into long-term and short-term planning. The difference, according to Salvatore (2001) short-term planning is defined by at least one fixed input variable and long-term planning by having no fixed input variables. Hence, scenario planning falls into the long-term planning category as all input variables can be subject to scrutiny. The input variables can be sub-divided into three categories considering controllable input, non-controllable input and system parameters as defined in section 3.1.

Porter (1985) argued that when constructing a scenario, viz. a possible future situation, it is important chose combinations of variables that can be considered consistent. A consistent scenario consists of combinations of scenario variables which are consistent, i.e. that the assumptions made about the value of a certain variable are possible or likely considering the values of the other scenario variables. In practise the number of considered scenarios should not exceed seven (Mintzberg, 1994). Once the scenarios have been analysed, the planners are still left with the problem of how to use the knowledge obtained to get the best result. According to Mintzberg (1994) the managers are left with the following options:

1. Bet on the most likely scenario.
2. Bet on the “best” scenario.
3. Hedge.
4. Preserve flexibility.
5. Influence.

This part of the planning procedure is beyond the scope of the present research.

A complete evaluation of the sustainability of a water resources system requires knowledge of all involved factors and their needs, now as well as in all infinite future. In practise this is not possible and, therefore, it is important to consider which factors are included and which are left out, and why. Furthermore, it is important to consider an applicable planning horizon.

The first step in the evaluation procedure is to select the water users to be included in the sustainability assessment. For each user two aspects need to be considered before inclusion in the analysis:

1. User is impacted by one or more scenarios.
2. User is likely to experience failure periods in one or more scenarios.

5.5 WATER DEMAND PREDICTION

The estimation of the sustainability criterion is carried out through the water driven approach as explained in section 3.3. Therefore, the prediction of the future water demand (need) in the considered sectors is of vital importance to reliable estimates.

The term “demand” is not unambiguous. Kindler & Russel (1984) distinguish between “demand” and “requirements” and use economic terms to define them. Demand is the quantity consumers are willing to buy at a given price, demand is, therefore, subject to price control. Requirements, on the other hand, cover the basic need for water and, therefore, is not subject to price control. Kindler & Russell (1984) present two different methods for forecasting future water demand: the statistical and the engineering approach. The statistical approach is the classical black box method where a dependent variable (water demand) can be predicted by one or more explanatory variable, i.e. water demand depends on number of people. The Engineering approach is a deterministic approach which aims at identifying the water demand by investigating the actual processes that leads to the water demand.

Urban water demand

When trying to determine the water demand for urban areas Kindler & Russell (1984) recommend the statistical method. McMahon (1993) presented a number of regression equations for estimating water demand from exploratory variables such as price of water, market value of housing unit, weather conditions, conservation programs and number of users. Most of these equations, however, are made for urban areas in USA and Australia. No readily available models for prediction of water demand in urban areas in Southern Africa were identified in literature. In this study, the future annual abstractions from the urban areas are predicted using a simple growth model based on available records of historical water abstraction. The general growth model is given as

$$\frac{dC}{dt} = K_n C^n \tag{5.24}$$

where

C = annual water demand
K_n = growth coefficient
n = order of growth

The solution to Eq. (5.24) depends on the order n. For n = 0 and n = 1 the annual water demand $C(t)$ is given below

$$C(t) = \begin{cases} C(t) = C_0 + K_0(t - t_0) & , n = 0 \\ C_0 e^{K_1(t-t_0)} & , n = 1 \end{cases} \tag{5.25}$$

The parameters in the growth models are estimated using the least square method and using historical annual abstraction. The monthly water demand $D(t,\tau)$in the τ-th month in year t is predicted as

$$D(t,\tau) = C(t)\delta(\tau) \tag{5.26}$$

where

$C(t)$ = annual water demand at time t as defined in Eq. (5.25)
$\delta(\tau)$ = correction factor for the τ-th season

In this study the seasonal correction factor $\delta(\tau)$ will be calculated from historic data and on a monthly basis as the fraction of monthly demand compared to the total annual demand, i.e.

$$\delta(\tau) = \frac{\sum_{t=1}^{T} D(t,\tau)}{\sum_{t=1}^{T}\sum_{j=1}^{12} D(t,j)} \tag{5.27}$$

This simplified perception on growth of water demand may not necessarily represent the actual development, especially when regarding long-term predictions. External factors such as, for example, demand management programs will affect the future development of water demand. Such factors are not included in this model and, hence, not explicitly accounted for. However, by adjusting the model parameters in the growth model the anticipated effect of a demand management program can be included in the prediction of future water demand.

Agricultural Water Demand

As for the urban sector, the agricultural water demand can be estimated based on statistical methods or on a more physically based model approach. However, reliable historical data of agricultural water demand are rarely available and, hence, the model approach is used to estimate irrigation requirements. This study uses the irrigation module available within the ACRU model and documented by Schulze (1995).

Environmental Water Demand

Determination of the water requirements of the ecological system is very complex and requires extensive monitoring and research. McMahon (1993) reviewed a number of existing methodologies varying from empirical to complex eco-hydrological models such as the physical habitat simulation (PHABSIM) model. In South Africa a method known as the building block method has been adopted for determining the instream flow requirement (IFR) of a certain river needed to sustain the existing environment and ecosystem. The IFR are determined at a multidisciplinary workshop and given as a set of minimum monthly flow values and desired peak flow values during the flooding season. For a further discussion of the BBM see King & Louw (1998).

Chapter 6

THE MGENI-MKOMAZI WATER RESOURCES SYSTEM

This chapter is the first of two case studies where the developed sustainability assessment methodology has been tested. The 4353 km^2 Mgeni catchment is situated in the KwaZulu-Natal province of South Africa and the Mgeni river is the major source of water supply to the Metropolitan areas of Durban and Pietermaritzburg, in which approximately 45% of the total population in the province live. The region contributes close to 20% of the South African GNP (Kienzle et al., 1997). Five major reservoirs have been constructed in the Mgeni catchment to secure reliable water supply to the urban areas and the industry. The total storage capacity of these reservoirs is 734 10^6 m^3 which constitutes approximately 110% of the mean annual runoff (MAR). The Mkomazi River is situated close to the Mgeni catchment and it is one of the last natural rivers left in South Africa. The Mkomazi catchment covers an area of approx. 4400 km^2. Both rivers rise in the Drakensberg Mountains 2000 m above sea level and flows into the Indian Ocean near Durban, see Fig. 6.1. The mean annual precipitation of the study area varies between 700 mm to 1200 mm with highest rainfall in the western part in the Drakensberg Mountains. The rainfall pattern reflects a seasonal variation with the bulk of the rain falling between October and March. The annual potential evaporation is estimated to be in the range of 1400 mm to 1800 mm.

South Africa recently adopted a new Water Act founded on the principles of sustainability and equity in water resources management. One of the major objectives of the post-apartheid government is the provision of clean water to all South Africans. Populations growth in the metropolitan areas of the Mgeni catchment, which is expected to reach 9-12 million in 2025 (Kienzle et al., 1997), have led to concerns of future water scarcity. At the same time, commercial agriculture and forestry are highly developed, especially in the upper parts of the catchment putting more pressure on the existing water resources. It is generally agreed that failure to supply water to the urban and industrial areas would have very severe undesirable consequences for the whole region. In the light of these grim forecasts, plans were laid out for the possible future development of the existing water resources system. The options include demand management in the Durban metropolitan region, raising of an existing dam wall and construction of a new reservoir, the Smithfield Dam, in the neighbouring Mkomazi catchment (see Fig. 6.1) followed by inter-basin transfer of water to the Mgeni catchment. Meanwhile, as a fast growing population demands more water, the new Water Act is attempting to encompass the needs of the environment. Before any new water resources development project can be authorised, an explicit assessment of the environmental and ecological needs of the affected river system has to be carried out. McKay (2001) highlighted the research and policy development undertaken in South Africa in order to transform the requirement of the new water act into a practical tool for improved water resources management, particular with reference to the needs of the hydro-ecological system. Currently, the need is specified as in-stream flow requirements (IFR) given as monthly minimum flow values at specific sites on the river.

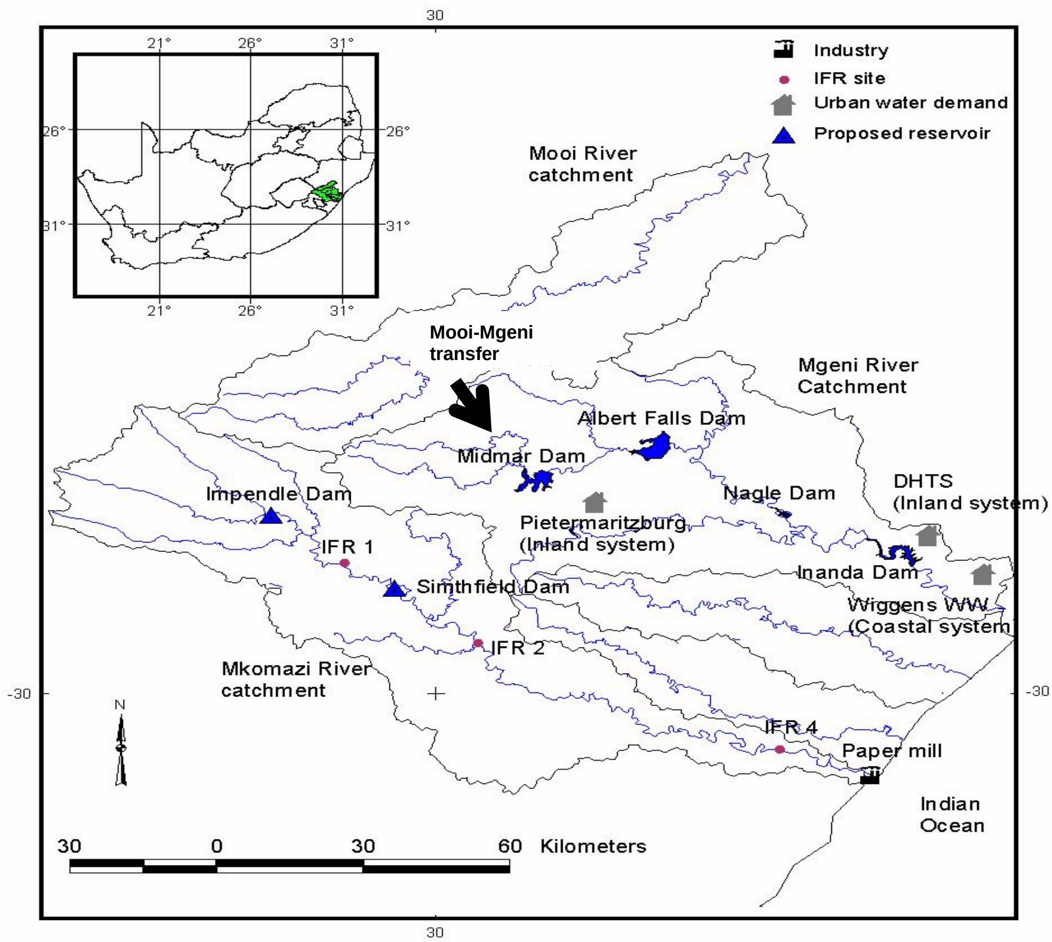


Figure 6.1 Mgeni-Mkomazi river basins with major infrastructure.

This case study consists of two major parts 1) setup and calibration of the MIKE BASIN model for the Mgeni-Mkomazi water resources system based on historical data, and 2) evaluation of different development scenarios based on the outlined methodology. The following sections 6.1 and 6.2 present the available data material from the Mgeni and the Mkomazi and the setup of the MIKE BASIN model for the entire water resources system. Section 6.3 describes the procedures applied for calibration of the modelling system. Section 6.4 outlines the estimation, verification and validation of the CARMA(p,q) model used to generate synthetic time series of monthly runoff from different runoff schemes within the water resources system. Finally, section 6.5 – 6.9 describe the development and evaluation of future scenarios of water resources development of the system.

The entire water resources system has been divided into twelve sub-catchments, six in each of the two catchments. The sub-catchments in the Mgeni catchment are denoted Mge1-Mge6 and, correspondingly, Mko1-Mko6 in the Mkomazi catchment. The geographical extension of each sub-catchment is defined in Table 6.1. The Mkomazi cachment was subdivided based on points of potential interest. Plans for constructing the Impendle dam were ousted by Umgeni Water in favour of the Smithfield dam, and only IFR site number four was included in the final analysis.

Table 6.1 Sub-catchments and their geographical extension.

Mgeni catchment	Geographical extension	Mkomazi catchment	Geographical extension
Mge 1	Inflow to Midmar Dam	Mko 1	Inflow to Impendle Dam
Mge 2	Between Midmar and Albert Fall Dam	Mko 2	Between Impendle Dam and IFR 1
Mge 3	Between Albert Fall and Nagle Dam	Mko 3	Between IFR 1 and Smithfield Dam
Mge 4	Between Nagle and Inanda Dam	Mko 4	Between Smithfield Dam and IFR 2
Mge 5	Inflow to Henley Dam	Mko 5	Between IFR 2 and IFR 4
Mge 6	Between Henley and Inanda Dam	Mko 6	Between IFR 4 and Paper mill

For each sub-catchment the water demand, infrastructure and time series of monthly runoff were estimated and used as input to the MIKE BASIN model. A schematic setup of the model for the entire water resources system is illustrated in Fig. 6.2.

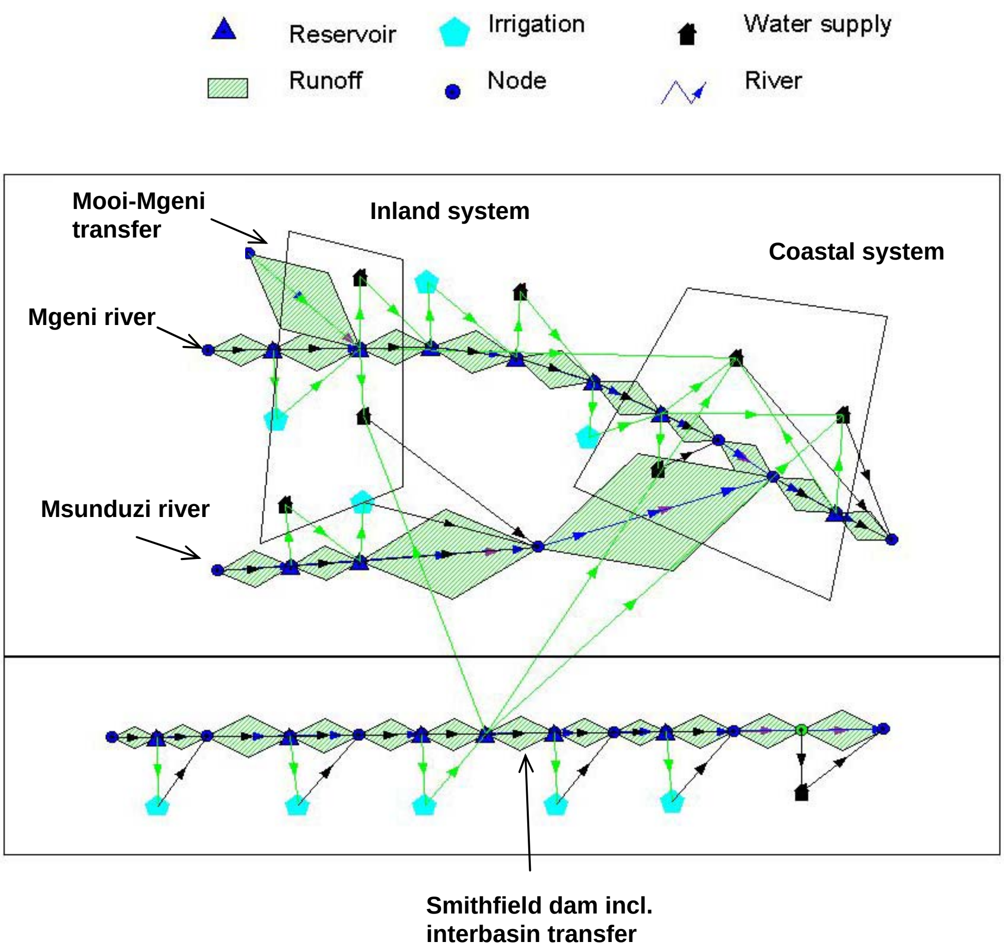


Fig. 6.2 Schematic MIKE BASIN setup of the Mgeni-Mkomazi water resources system.

6.1 MGENI SYSTEM

Due to the presence of important urban areas, the Mgeni catchment has been the subject of intense research into water related topics such as reported by Tarboton & Schulze (1992), Tollow (1995), DWAF (1997), Kienzle et al. (1997), Umgeni Water (1998), Preston (1999) and Naidoo & Constantinides (2000). Data material for the Mgeni catchment was collected in South Africa through interviews, literature and site visits.

Water demand

Data on present and future water demand was collected and compiled from Umgeni Water and from the University of Natal. Below is a description of the different water users identified within the catchment including a description of the anticipated future demand.

Urban Water The urban water demand is concentrated in the metropolitan areas of Durban (Coastal system) and Pietermaritzburg (inland system). Historically, the reservoir system has been used for supplying the urban areas, while rural areas relied mainly on groundwater sources. According to Naidoo & Constatinides (2000) the coastal system represents 80% of the total water demand in the Mgeni catchment. The historical growth rate of water demand in the coastal system has been approximately 6% per annum over the last ten years. Historical data for urban water abstraction from the reservoir system include both domestic and industrial consumption and, according to Tollow (1995), industry uses the bulk of the water. No attempts were made to separate the two demands. Time series of total annual abstraction from the inland and coastal systems are shown in Fig. 6.3.

Umgeni Water anticipates that water demands from both the inland and the coastal system will continue to increase in the future. Prediction of the future water demand is further discussed in section 6.6.

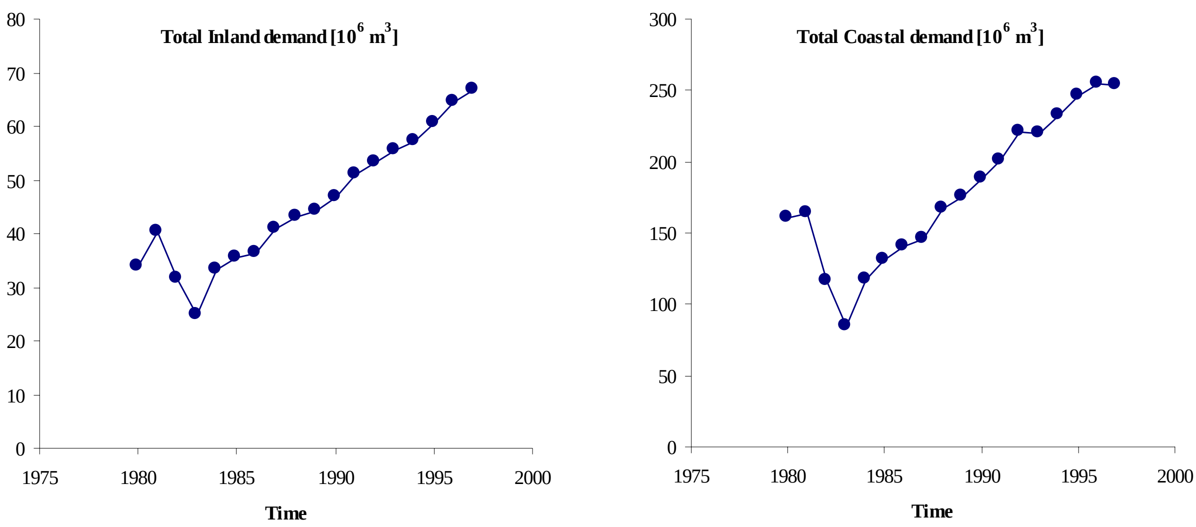


Figure 6.3 Total annual abstraction for urban use from Inland and Coastal systems.

Agriculture Detailed surveys of irrigation in the Mgeni catchment are reported by Tarboton & Schulze (1992) and Kienzle et al. (1997). In the two studies parameters of irrigated area, soil properties and crop characteristics were estimated using satellite images. Other parameters such as mode of irrigation scheduling, irrigation cycle, conveyance losses and source of irrigation water were collected at site, and historical time series of water demand were calculated using the irrigation routine in the ACRU model (Schulze, 1995). The calculated water demand is the net demand, i.e. the amount of water that would have been applied to a perfectly irrigated system. The average annual water demand for irrigation is shown in Table 6.2. Water demand for irrigation is not expected to increase in the future.

Table 6.2 Agricultural water demand.

	Irrigation demand [l/s]			
	Mge1	Mge2	Mge3	Mge6
January	0.851	0.316	0.115	0.003
February	1.158	0.397	0.125	0.004
March	0.861	0.335	0.113	0.003
April	1.425	0.472	0.125	0.003
May	1.745	0.605	0.153	0.004
June	1.679	0.511	0.150	0.004
July	1.637	0.461	0.147	0.004
August	1.419	0.391	0.132	0.004
September	1.213	0.353	0.122	0.004
October	0.914	0.255	0.119	0.003
November	0.960	0.301	0.128	0.003
December	0.934	0.308	0.138	0.004
Annual [10^6 m^3]	38.9	12.4	4.1	0.1

Environment No IFR data has been specified within the Mgeni catchment. Each of the major dams has to release a minimum flow in a non-shortage situation, defined as normal flow. Values of normal flow for each of the major dams are given in Table 6.3.

Table 6.3 Minimum release [m^3/s] from major dams in the Mgeni catchment.

Dam	Minimum release [m^3/s]
Midmar	0.90
Albert Falls	*
Nagle	0.71
Inanda	1.50

* No minimum release specified for Albert Falls Dam

Recreation Recreational use of the Mgeni system includes swimming, yachting, fishing, canoeing, and bird watching, and both Midmar Dam and Albert Falls Dam are popular recreational resorts. However, the vital use of the reservoirs for water supply has highest priority. Therefore, no constraints to system management due to recreational use were identified.

Water supply infrastructure

Information concerning the water supply infrastructure and its operation including major reservoirs and farm dams has been compiled.

Major reservoirs As the metropolitan areas gradually became larger, additional storage facilities were constructed within the Mgeni catchment to secure a reliable supply of water. The present state of the reservoir system is shown in Fig. 6.2. The dimensions of the existing dams are given in Table 6.4. Umgeni Water provided dimensions and HVA curves for each of the five major dams.

Table 6.4 Dimensions of major dams in the Mgeni catchment.

Dam	Year of Construction	Full Supply Capacity $[10^6\ m^3]$	Area at Full Supply Level $[km^2]$
Midmar	1963	177.8	15.6
Albert Falls	1976	289.7	23.5
Nagle	1948	23.2	1.6
Inanda	1989	241.7	14.3
Henley*	1942	1.5	0.3

* Henley Dam is situated on the Msunduzi river, but was taken out of operation in 1996 due to operation costs.

Midmar Dam supplies water to the Pietermaritzburg area (inland system). Nagle and Inanda Dam supply water to Durban area (coastal system). Albert Falls Dam is a back up reservoir for the coastal system.

System operation The urban areas are supplied by water works, which draw water from the reservoir system. The inland system is supplied by the water abstracted from Midmar Dam and in MIKE BASIN represented by one water supply scheme. A second scheme is added to model the historical abstraction from Henley Dam. The coastal system consists of two major water works, Durban Heights water work (DBHS) and Wiggens water work. DHTS draw water primarily from Nagle Dam, but can be supplied by Inanda Dam or even Midmar Dam when needed. Wiggens pumps water from Inanda Dam, but can be supplied by Nagle Dam if necessary. Historical data on abstraction from the reservoirs are available but not information concerning which water work is receiving the water. In the model setup the water abstracted from Nagle Dam has been defined as the historical demand of DHTS water work, and the water abstracted from Inanda Dam has been defined as the historical demand of Wiggens water work.

According to Umgeni Water 6% of the water released from the Midmar Dam is lost between the dam and the water work due to leakage. In the coastal system the loss is estimated to be 5%. No fixed operating rules exist for the reservoir system as the reservoirs are operated on a month to month basis based on reservoir levels and engineering experience. Information on how much water to draw from each reservoir under certain operating options was provided by Umgeni Water (rule I – O) and is shown in Table 6.5. However, no guidelines as to which operating rule should be applied under which conditions exist.

Table 6.5 Reservoir operation guidelines in % of total demand of the coastal and inland system in the Mgeni catchment.

Water work	Reservoir	I	J	K	L	M	N	O
Midmar	Midmar	100	100	100	100	100	100	100
DHTS	Nagle	100	89.9	79.8	69.6	55.0	42.3	37.2
	Inanda	0	11.1	20.2	30.4	45.0	57.7	62.8
Inanda	Nagle	100	100	100	100	100	100	100
	Inanda	0	0	0	0	0	0	0

Urban return flow From the inland system the bulk of the waste water is discharge into the Msunduzi River, which is a tributary to the Mgeni River (See Fig. 6.2). Umgeni Water (1998) has estimated the return flows from the inland system to be 46% of the water abstracted from Midmar Dam. Waste water from the coastal system comes from DHTS and is discharged into the Indian Ocean. According to Umgeni Waters, 49% of the water abstracted for DHTS is returned to the system downstream of Inanda Dam. Wiggens does not contribute with return flow to the system.

Farm dams Tarboton & Schulze (1992) identified 1133 farm dams within the catchment. The development of farm dams is a dynamic process as dam construction takes place continuously over time. Umgeni Water (1998) has estimated the historical development of volume and full supply area of farm dams within four areas of the catchment: upstream of Midmar, between Midmar Dam and Albert Falls Dam, between Albert Falls Dam and Nagle Dam and on the Msunduzi river downstream of Henley Dam. On this basis time series of volume and area can be calculated from estimated volume and area at present. In Mike Basin the farm dams within each runoff area are aggregated into one dam. In the Mgeni catchment the farm dams are represented by four reservoirs, see Table 6.6, each situated at the outlet of the sub-catchment under consideration. The procedure used for estimating HVA curves for the farm dams is outlined in Appendix B.

Table 6.6 Dimension of farm dams in the Mgeni catchment.

Farming area	Full Volume $[10^6\ m^3]$	Area $[km^2]$
Mge1	13.4	5.1
Mge2	8.3	3.7
Mge3	7.9	4.2
Mge6	7.8	1.7

A standard operating rule was specified for the farm dams and the minimum required downstream release was set equal to zero.

Mooi transfer Since the early 1980s water has been transferred from the Mooi River and into the Mgeni system upstream of Midmar Dam, see Fig. 6.1, when needed. However, no records of the pumped volumes exist and the transfer is, therefore, not included in the model setup.

Runoff

An existing ACRU model setup of the hydrology of the Mgeni catchment was made available through the University of Natal. The Mgeni catchment was delineated into 137 subcatchments covering the catchment to the outlet of the Inanda Dam, near the Indian Ocean. Daily runoff has been simulated for the period of 34 years 1.1.1960 – 31.12.1993. Kienzle et al. (1997) compared monthly simulated streamflow values to observed runoff at six different sites within the catchment and found values of the coefficient of determination R^2 between 0.79 and 0.88. The naturalised runoff was obtained by running the ACRU model disabling the influence by dams and abstraction for irrigation and water supply. Finally, time series of monthly runoff were calculated. The runoff from each of the six sub-catchments is shown in Table 6.7. The seasonal variation of monthly runoff is illustrated in Fig. 6.4. Most runoff occurs during the wet season from October to March.

Table 6.7 Runoff schemes in the Mgeni catchment.

Runoff scheme	MAR $[10^6\ m^3]$	Area $[km^2]$	MAR [mm]
Mge1	196.4	927	212
Mge2	139.8	725	193
Mge3	112.2	907	124
Mge4	102.4	668	153
Mge5	41.6	220	189
Mge6	81.1	680	119

6.2 MKOMAZI SYSTEM

The Mkomazi catchment is situated south of the Mgeni catchment. Having a lower degree of utilisation the water resources of the Mkomazi catchment have been subject to less research activities than the Mgeni catchment. However, current research at the University of Natal is focusing on the hydrology of the catchment and a setup of the ACRU model for the catchment has been completed (Taylor, 2001)

Water demand

Data on present and future water demand has been collected and compiled from Umgeni Water and University of Natal. Below is a description of the different water users identified within the catchment including a description of the anticipated future demand.

Domestic The population of the Mkomazi catchment is approximately 216 000, with 98% living in rural areas (1992). Water consumption in rural areas of KwaZulu-Natal is estimated to be between 10 l/(cd) and 60 l/(cd), most often closest to 10 l/(cd). This low number combined with the limited population and the fact that only 30% of the domestic water comes from rivers (IWRMS, 1998) makes the domestic water consumption from rural areas negligible compared to volumes consumed by other users. Contributing with a consumption between 0.2% and 0.7% of the MAR and scattered all over the catchment, domestic water consumption is not included in the analysis.

Industry The only significant industry in the catchment is a SAPPI-SAICOR paper mill plant situated at the river mouth. The current consumption is 50 10^6 m^3/y and is not anticipated to increase in the future.

Agriculture Information concerning the water demand for irrigation has been compiled and modelled using the ACRU model at University of Natal (Taylor, 2001), see Table 6.8.

Table 6.8: Agricultural water demand in the Mkomazi catchment [l/s].

	Mko1	Mko2	Mko3	Mko4	Mko5
January	0.119	0.005	0.000	0.958	0.639
February	0.111	0.004	0.000	0.980	0.597
March	0.140	0.005	0.000	1.634	0.728
April	0.185	0.007	0.029	1.408	0.699
May	0.210	0.009	0.033	1.444	0.681
June	0.195	0.008	0.033	1.279	0.601
July	0.195	0.009	0.033	1.146	0.522
August	0.166	0.009	0.033	0.943	0.425
September	0.132	0.008	0.030	0.806	0.356
October	0.110	0.008	0.000	0.787	0.410
November	0.122	0.006	0.000	0.916	0.602
December	0.128	0.005	0.000	0.960	0.685
Annual [10^6 m^3]	4.8	0.2	0.5	34.8	18.3

Environmental Three Instream Flow Requirement (IFR) sites have been identified on the Mkomazi River, see Fig. 6.2. IFR are specified as a required monthly flow in order to maintain the environmental state of the river. The IFRs have been determined at an interdisciplinary workshop (DWAF, 1998) and Fig. 6.5 shows the required minimum monthly flow at the three IFR sites specified on the Mkomazi River.

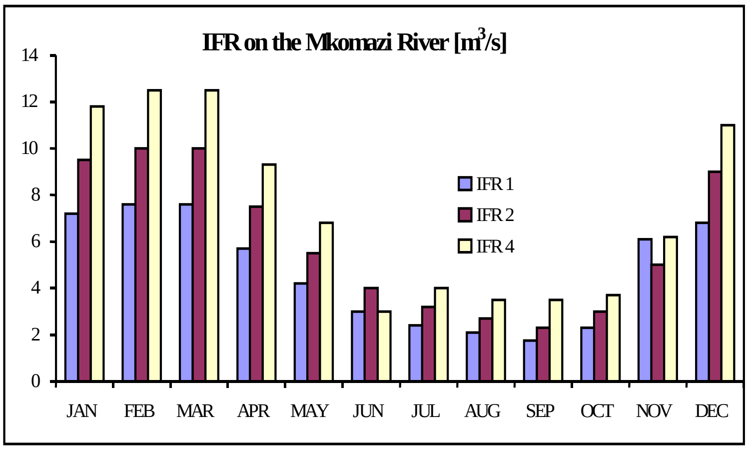


Figure 6.3 IFR at the three sites on the Mkomazi River.

Water supply infrastructure

The Mkomazi catchment is much less developed with regards to water supply infrastructure than the Mgeni catchment. No major metropolitan area exits within the catchment and the majority of the population is supplied by groundwater. The only significant infrastructure is farm dams.

Farm dams Umgeni Water (1998) has estimated the historical development of volume and full supply area of farm dams within five sections of the catchment. The same procedure as described in connection with the Mgeni catchment, is used to calculate historical time series of capacity and area for farm dams and the HVA curves. The dimensions of the farm dams are shown in Table 6.9.

Table 6.9 Dimension of farm dams in the Mkomazi catchment.

Farming area	Full Volume $[10^6\ m^3]$	Area $[km^2]$
Mko1	2.7	1.0
Mko2	1.0	0.4
Mko3	1.7	0.7
Mko4	3.0	1.4
Mko5	0.5	0.3

Runoff

The initial ACRU setup used in this study has subdivided the Mkomazi catchment into 21 sub-catchments. Each sub-catchment is divided into eight catchments according to land use, hence a total of 168 sub-catchments. Daily runoff is simulated from 1.1.1945 to 31.9.1996, however only data from 1.1.1960 – 31.12.1993 is included in this study. Naturalized monthly runoff was obtained by running the ACRU model without the influence of reservoirs and water abstraction. The catchment runoff was disaggregated into six runoff schemes, see Table 6.10.

Table 6.10 Runoff schemes in the Mkomazi catchment.

Runoff schemes	MAR [10^6 m^3]	Area [km^2]	MAR [mm]
Mko1	550.1	1407	391
Mko2	143.5	325	441
Mko3	69.8	310	225
Mko4	156.9	976	161
Mko5	213.4	1413	151
Mko6	9.2	35	263

6.3 CALIBRATION OF THE MIKE BASIN MODEL

The MIKE BASIN model is calibrated by adjusting the reservoir operation policies. The reservoirs in the Mgeni system are operated based on the prevailing supply/demand situation, experience of the managers and according to certain legislative requirements such as minimum downstream release, i.e. no fixed procedure for system operation exists. The available data amounts to historical records of reservoir storage in each of the four major reservoirs and abstraction for the urban areas. For each reservoir the reduction factors controlling the curtailment the downstream release was adjusted using trial and error until the best agreement between observed and simulated reservoir storage was obtained. The historical time series of abstraction was specified in the model as the desired draft from the reservoirs. In periods with water scarcity the actual draft can be less than the specified desired draft, which will indicate a discrepancy between the model and the historical data. The model's ability to fulfil the desired drafts is an indication that enough water was available, however, it could also be the case that more water than needed was available as well. Hence, the ability to reproduce historical time series of abstraction was given less weight than the ability to reproduce observed reservoir storage. As no major reservoirs are situated on the Mkomazi River, no system calibration is performed for the catchment.

The system is modelled for the period January 1960 to December 1993 using monthly time steps. Historical urban water abstraction is drawn from each reservoir, i.e. no demand reduction is specified in the model. The downstream release is adjusted to minimize the difference between observed and simulated reservoir storage.

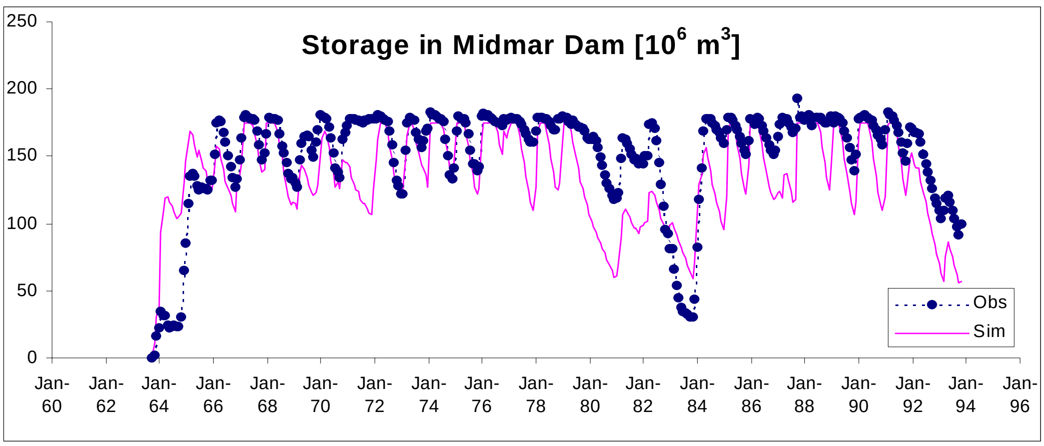
The minimum required releases from the reservoirs are given in Table 6.3, and the operation is controlled using the following parameters:

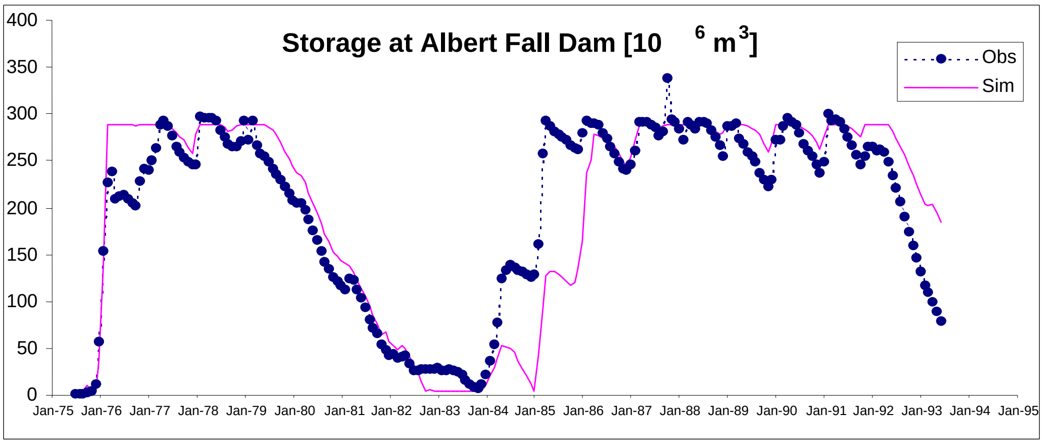
1. Reduction level 1 and 2.
2. Reduction factor 1 and 2.
3. Normal release.

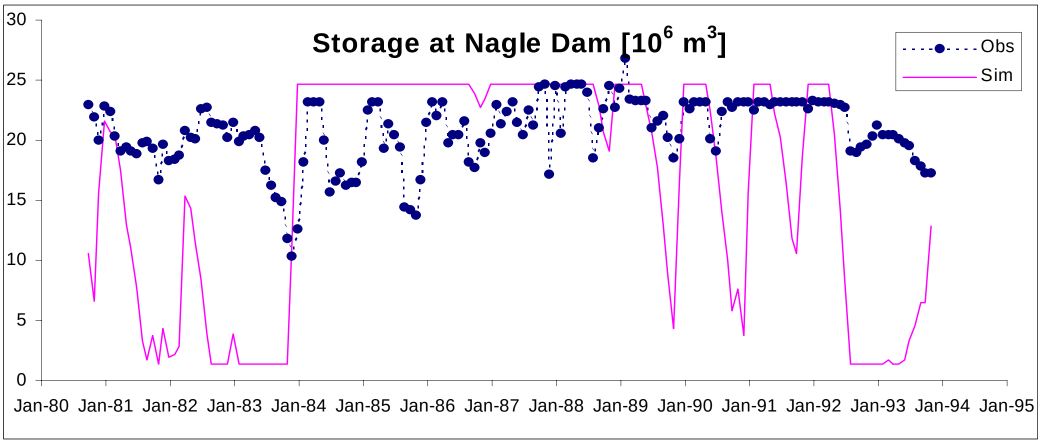
Additionally, the following constraint was introduced: (normal release)*(reduction factor) = minimum required release. Furthermore, information for determining reduction level 1 and 2 was obtained from the technical description of the dams. Information concerning normal release was taken from recorded flow just downstream of the dams. Flow values from periods where the dams were not spilling or had reduced downstream release were used as estimates of normal release. The calibration of the downstream release operation is based on visual comparison of observed and simulated reservoir storage. For Inanda Dam no downstream release operation has been specified, only a required minimum downstream release of 1.5 m^3/s. No operating rules were specified for Henley Dam and the farm dams. The calibration parameters are shown in Table 6.11, and the observed vs. simulated reservoir storage graphs are shown in Fig. 6.5.

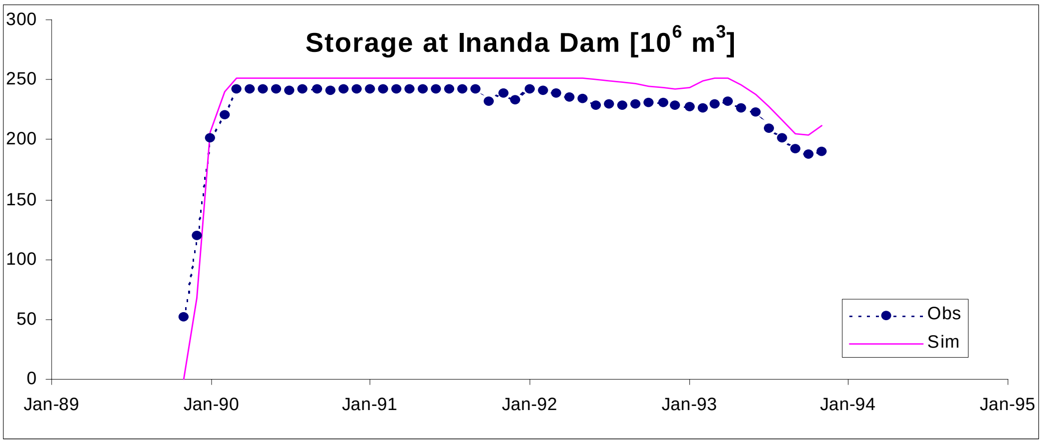
Table 6.11 Parameters controlling downstream release from reservoirs.

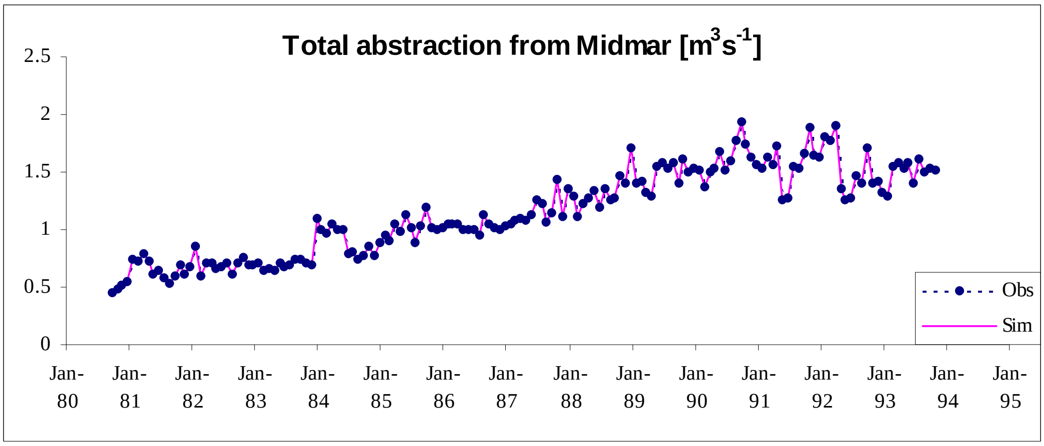
Reservoir	Full reservoir level [m]	Min. op level [m]	Red. level 1 [m]	Red. level 2 [m]	Red. fact 1 [-]	Red. fact 2 [-]	Qnorm [m^3/s]
Midmar	1043.9	1032	1041	1041	0.5	0.273	3.295
Albert Fall	655.9	637.1	638	638	0.5	0.1	4.48
Nagle	403.8	379.8	399	387.5	0.71	0.1	1.0




Storage at Albert Fall Dam [10^6 m^3]
Obs
Sim
400
350
300
250
200
150
100
50
0
Jan-75 Jan-76 Jan-77 Jan-78 Jan-79 Jan-80 Jan-81 Jan-82 Jan-83 Jan-84 Jan-85 Jan-86 Jan-87 Jan-88 Jan-89 Jan-90 Jan-91 Jan-92 Jan-93 Jan-94 Jan-95


Storage at Nagle Dam [10^6 m^3]
Obs
Sim
30
25
20
15
10
5
0
Jan-80 Jan-81 Jan-82 Jan-83 Jan-84 Jan-85 Jan-86 Jan-87 Jan-88 Jan-89 Jan-90 Jan-91 Jan-92 Jan-93 Jan-94 Jan-95


Storage at Inanda Dam [10^6 m^3]
Obs
Sim
300
250
200
150
100
50
0
Jan-89 Jan-90 Jan-91 Jan-92 Jan-93 Jan-94 Jan-95



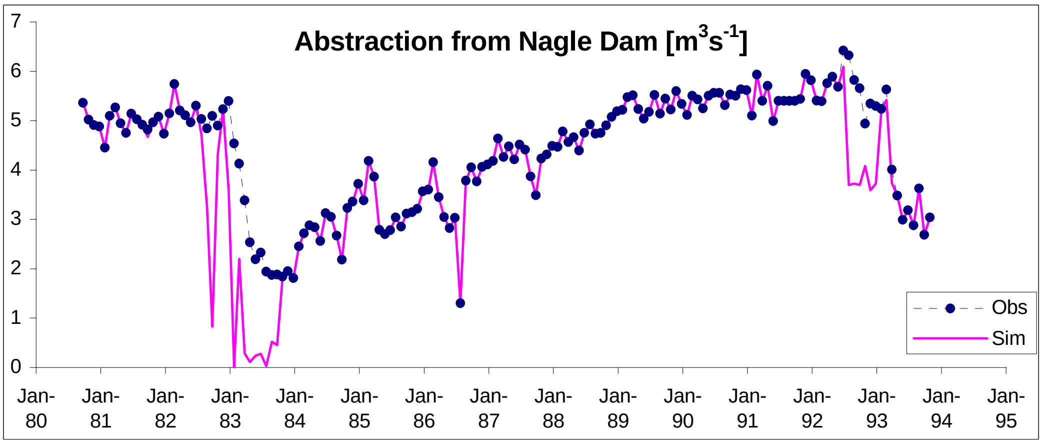


Fig. 6.5(a)-(f) observed vs. simulated storage and abstraction at the major dams.

As can be seen from Fig. 6.5(a)-(d) the simulated reservoir storage generally follows the observed storage. For Midmar Dam the period before 1980 is well described, but after 1980 the simulated storage is generally lower than the observed storage. This might be explained by the fact that interbasin transfer from Mooi River to Midmar Dam was initiated around 1980 but not included in the model due to the lack of information. The simulated and observed storage for Albert Falls Dam is well described, except for the period Jan 1984-Jan 1987 where the simulated storage is estimated with a one year lag time compared to the observed. The simulated reservoir has a slower recovery from the 1981-1983 drought than the actually observed. The simulated vs. observed storage for Nagle Dam, see Fig. 6.5(c), gives the worst results. In the dry periods the observed storage is generally observed above 15.0 10^6 m^3, whereas the simulated storage often falls below 10.0 or 5.0 10^6 m^3.

The discrepancy between simulated and observed storage, especially in the dry periods, indicate that either 1) the system receives water from a source not included in the model setup (e.g. Mooi River transfer) or 2) Nagle Dam is operated in conjunction with Albert Falls Dam, a linkage MIKE BASIN is not able to model. Looking at Fig. 6.5(e), the observed and simulated abstraction from Nagle Dam, it is noticed that in the dry periods from 1981-1983 and again in 1992, the model is not able to reproduce the specified abstraction, once again indicating the existence of sources not included in the model or a conjunctive operation with Albert Falls Dam. The demand reduction constraints specified at reduction level 1 and reduction level 2 reduces the downstream release below the required minimum. This has been necessary in order to obtain a better simulation of the storage. The storage of Inanda Dam is well described, however only based on the period 1989-1993. The HVA curve obtained from Umgeni Water overestimates the actual storage capacity of the reservoir and, therefore, the simulated storage is generally larger than the observed.

The fact that too little water is included in the model affects the estimation of the downstream release operation, as it will tend to hold water back in the reservoirs to compensate for the missing volume. The model was found to be robust concerning the downstream release, allowing for a wide spectrum of values to be accepted. These factors introduce uncertainty in the estimation of downstream release. A more thorough investigation could be obtained, if knowledge of the Mooi River transfer was available. Despite these weaknesses the model setup was regarded as satisfactory and adopted for the further studies.

6.4 ESTIMATION OF MULTIVARIATE STOCHASTIC MODEL

The identification and estimation of the multivariate time series model of monthly runoff are carried out as outlined in section 5.2. For any given year the annual runoff is generated at all sites under consideration and afterwards the monthly runoff is obtained using the method of fragments disaggregation approach. This section presents the identification, estimation, verification and validation of the CARMA(p,q) model to be coupled with the MIKE BASIN setup presented in the previous section. The Mgeni-Mkomazi water resources system includes twelve runoff schemes, six in the Mgeni catchment and six in the Mkomazi catchment. As described in section 5.2, a CARMA(p,q) model can be considered as a number of univariate ARMA(p,q) models linked together by the variance-covariance structure of the stochastic elements.

Univariate ARMA(p,q) models

At each of the twelve runoff schemes a time series of naturalised annual runoff from the period 1960 to 1993, i.e. 34 years, is available. Before estimating the univariate ARMA(p,q) models, the hydrological year was defined based on minimum month to month correlation. For all sites the hydrological year was defined equal to the calendar year ranging from January to December. The annual runoff from each of the twelve schemes was investigated for the existence of trend using the Mann-Kendal test with a level of significance of 5%, but no trend was detected in any of the time series. The estimation of the twelve univariate ARMA(p,q) to the either Box-Cox or LN3 transformed annual runoff from each scheme was performed using the unconditional maximum likelihood method as described in section 5.2 and a summary of the results is shown in Table 6.12. The model selection was based on Akaike's information criterion (AIC) as described in Appendix A.

From the results in Table 6.9 the resulting model is a CARMA(3,0) model.

Table 6.12 Univariate time series models used in the CARMA(3,0) model of annual runoff in the Mgeni-Mkomazi catchment.

Runoff Scheme	ARMA(p,q)	Box-Cox λ	LN3 ξ	Runoff Scheme	ARMA(p,q)	Box-Cox λ
Mge1	AR(2)	0.26		Mko1	AR(0)	0.05
Mge2	AR(2)		8418.4	Mko2	AR(0)	0.41
Mge3	AR(1)		3214.6	Mko3	AR(0)	0.34
Mge4	AR(1)		-3406.5	Mko4	AR(2)	0.12
Mge5	AR(2)	0.05		Mko5	AR(3)	0.03
Mge6	AR(2)		5894.1	Mko6	AR(3)	0.35

Correlation structure

The spatial correlation structure of the CARMA(3,0) model is specified through the cross-correlation of the time series of $\varepsilon(i,t)$, the residual cross correlation coefficient (RCCC), as defined by Eq. (5.11) – Eq.(5.12). Estimation of the variance-covariance matrix Δ is based on a two step procedure. First the time series of the transformed time series of annual runoff from each is scheme is pre-whitened through Eq. (5.10) to obtain the time series of normally distributed residuals $\varepsilon(i,t)$ with mean value of zero. Next, the RCCC between time series $\varepsilon(i,t)$ and $\varepsilon(j,t)$ at lag l, $\rho(i,j,l)$ is estimated through Eq. (5.11) and Eq. (5.12). Fig 6.5 shows the RCCC for all possible 66 combinations of i and j for the 12 runoff sites and for lag-time l = -2, 1, 0, 1, 2. The 95% confidence interval around $\rho\ (i,j,l) = 0$ is given as $\pm 2n^{-1/2}$ (Hipel & McLeod, 1994) with n = 30 years in this case. As evident from Fig 6.6 the cross correlation is significantly different from zero at lag-time l = 0. For the remaining lag-times, at least 95% of the estimated values of cross correlation fall inside the 95% confidence interval. For l = 2 only 92.4% of the estimated RCCC is below the +95% threshold level.

However, it was assumed that the possible gains from including the l = 2 correlation, and thereby moving from a CARMA(p,q) model to a more general ARMA(p,q) model, would less than balance the increased model complexity. Hence, the CARMA(3,0) model was assumed a reasonable choice of model.

Model verification

Model verification compares the mean annual runoff (MAR), and the standard deviation of annual runoff of the generated time series with the similar statistics estimated from the historical time series of annual runoff from each site. To ease visual comparison, all estimates are normalised with the historical estimate of the factor under scrutiny. From Fig. 6.6 it can be seen that each of the historical estimates of MAR are close to the corresponding median of the N = 5000 generated values of MAR. For all time series the historical estimates of MAR is between the 0.25 and 0.75 quantiles of the generated MAR sample. The historical standard deviation of annual runoff is compared to the corresponding generated values in Fig. 6.7.

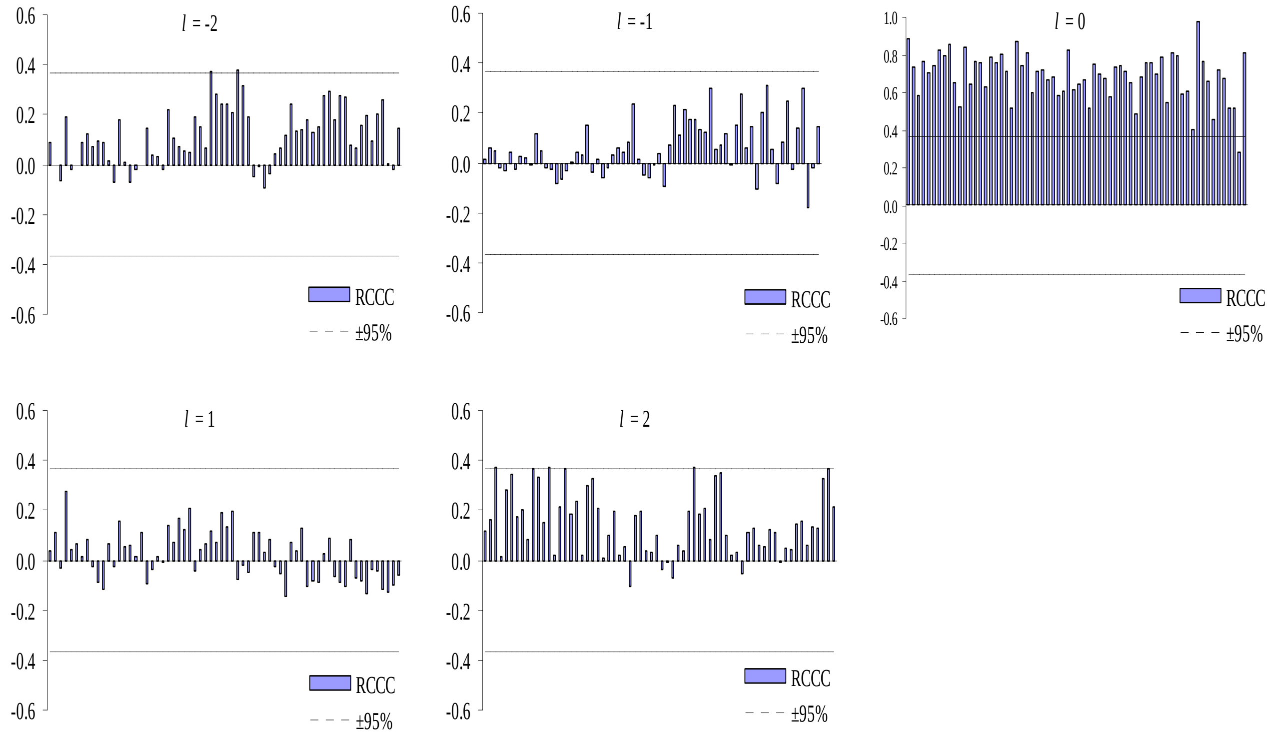


Fig. 6.5 RCCC for different lag times *l*. Only RCCC for *l* = 0 is significantly different from zero.

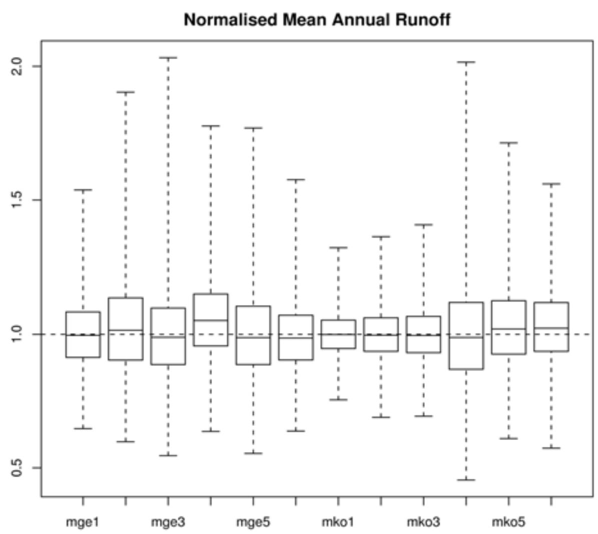


Fig 6.6 Simulated MAR at each site.

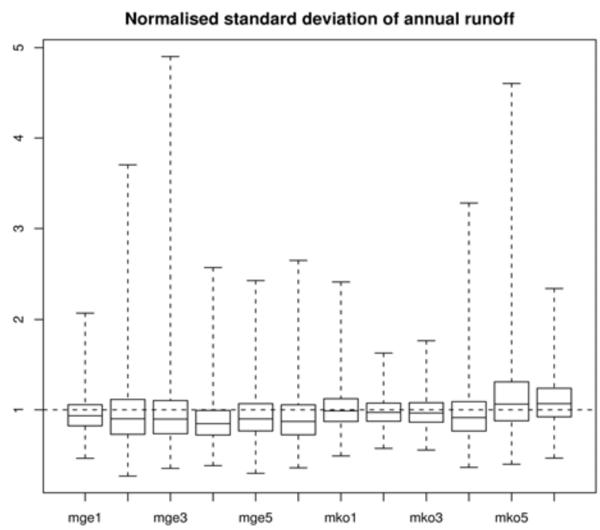


Fig. 6.7 Simulated standard deviation of annual runoff at each site.

As for MAR the generated estimates of the standard deviation of annual runoff generally correspond well with the historical estimates, which fall within the 0.25 to 0.75 quantile interval of the generated values. The model verification shows that the implemented CARMA(3,0) model reproduces the specified historical statistics and therefore can be considered correctly implemented.

Model validation

Validation of the multisite extension of the method of fragments is concerned with the following aspects of model performance: ability to generate mean monthly runoff (MMR) and standard deviation of monthly runoff (SMR), the correlation between successive months at the same site, the ability to reproduce maximum observed deficit volume at each site, the lag-zero cross correlation between annual runoff at different sites and, finally, the lag-zero correlation between droughts at different sites through Eq. (5.23).

Mean monthly runoff The historical MMR and the corresponding 5-number summary, as explained in section 5.2, of the generated values for each of the 12 runoff schemes within the Mgeni and Mkomazi catcments are shown in Fig. 6.8. During the rainy season characterised by high runoff (October-April) the historical values are generally within the interval defined by the 0.25 - 0.75 quantiles of the generated values, i.e. the model is performing satisfactorily. Slightly less satisfactory results are obtained during the dry season (May-September) where the method of fragments tends to overestimate the MMR compared to the historical values.

Standard deviation of monthly runoff Considering the standard deviation of monthly runoff, a pattern similar to the results for MMR was observed as shown in Fig. 6.8. Generally, the results are satisfactory during the rainy season even though historical values outside the 0.25 - 0.75 quantile interval are observed for a number of gauging stations. The estimates from the generated time series during the dry season generally overestimate the observed values. The months of September and, to a lesser degree, October constitute a problem. The coefficient of variation of the historical runoff is significantly higher in these months than any other months. However, for most runoff-schemes and for most months the method of fragments generate MMR with a standard deviation within the interval defined by the 0.25 - 0.75 quantiles of the generated values. An attempt to relate fraction, as defined in Eq.(5.13), to annual runoff through a functional relationship for a more smooth disaggregation proved to be fruitless.

Correlation between runoff in successive months The correlation between runoff in a specific month and the runoff in the preceding month obtained through time series generation is compared to the corresponding historical estimates in Fig. 6.10. For most sites and for most months the historical estimates fall within the 0.25 - 0.75 quantile interval of the estimates from the generated time series. For some sites the month-to-month correlation is underestimated during the dry season, especially the correlation between May-June and June-July. The method of fragments does not preserve the correlation between the last month of a hydrological year and the first month of the following hydrological year. Here, the hydrological year has been defined from January to December and, hence, the historical correlation between these two months is not preserved by the method of fragments, which is clearly seen on Fig. 6.10.

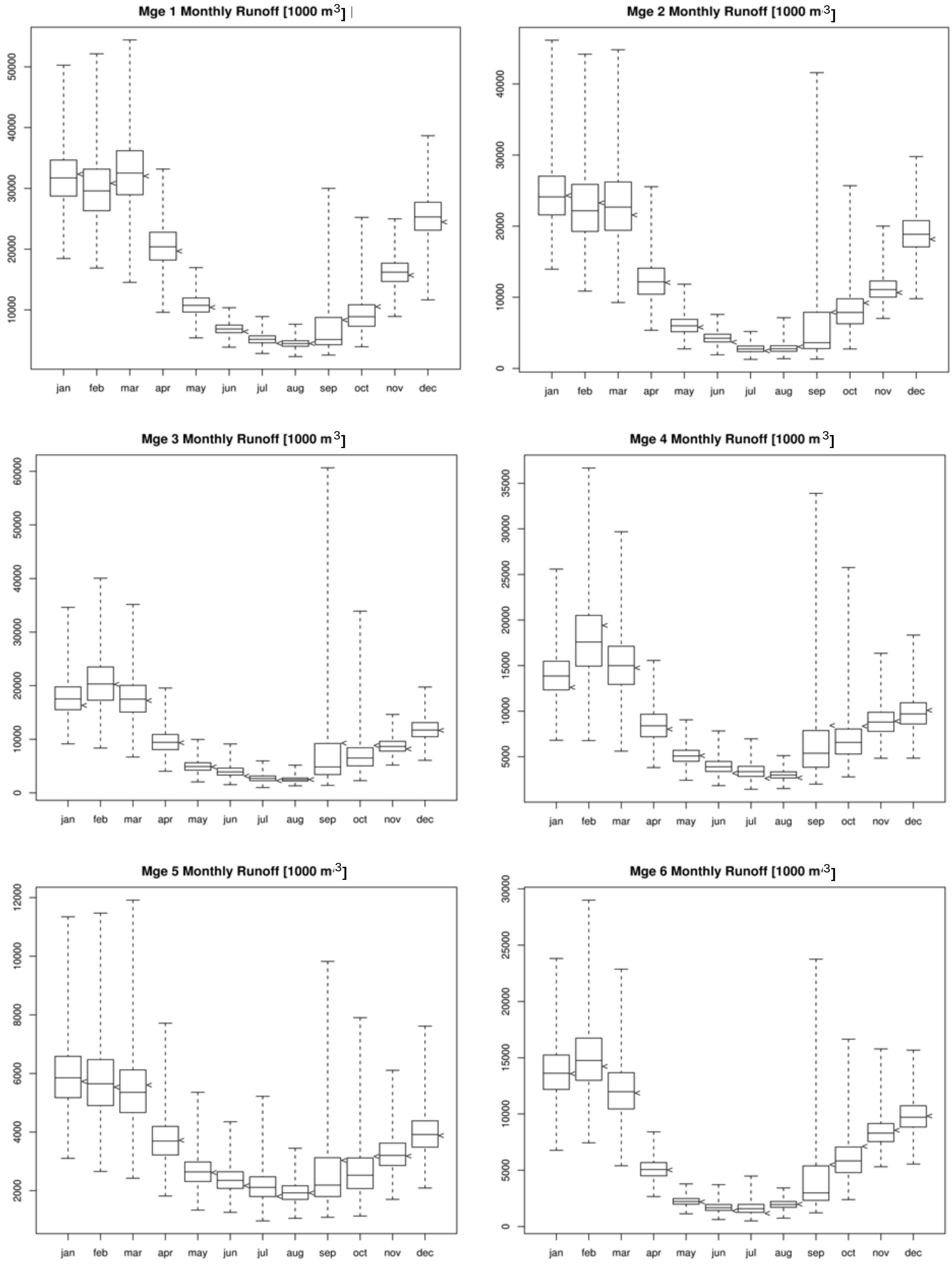


Fig. 6.8 Observed and generated monthly runoff at each site. Estimates obtained from historical time series are indicated by a ”<”.

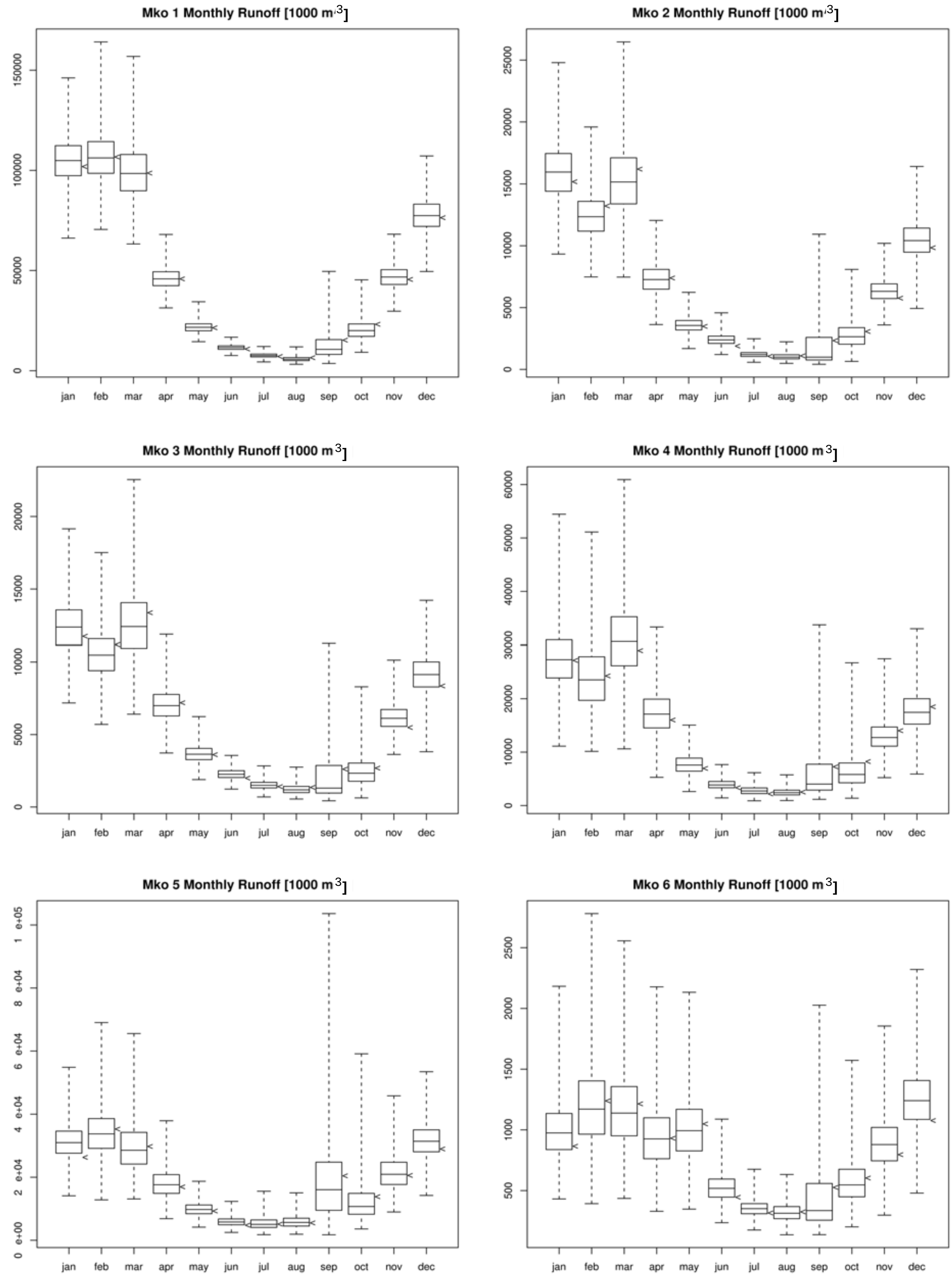


Fig. 6.8 Observed and generated monthly runoff at each site. Estimates obtained from historical time series are indicated by a ”<”.

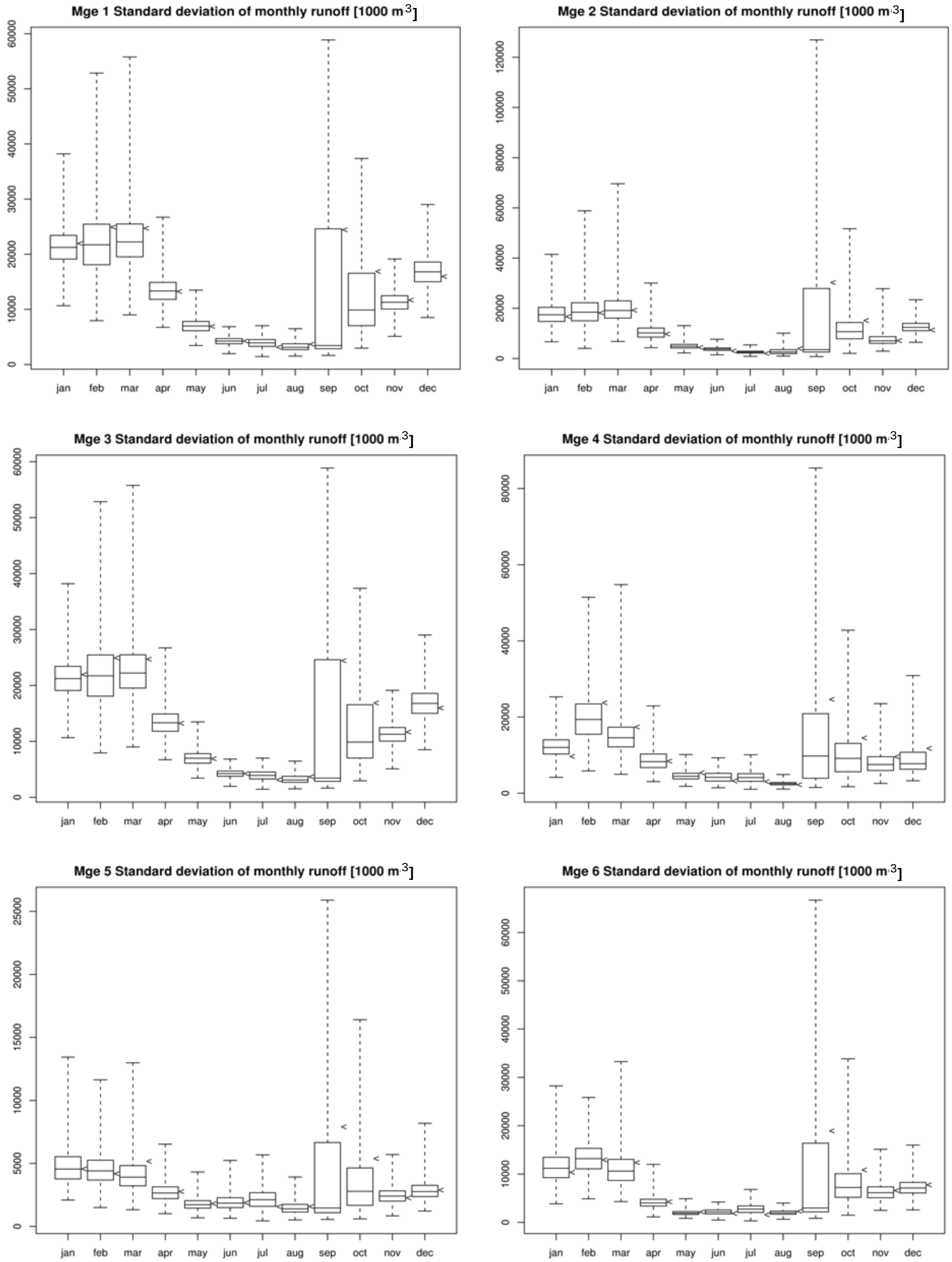


Fig. 6.9 Observed and generated standard deviation of monthly runoff at each site. Estimates obtained from historical time series are indicated by a "<".

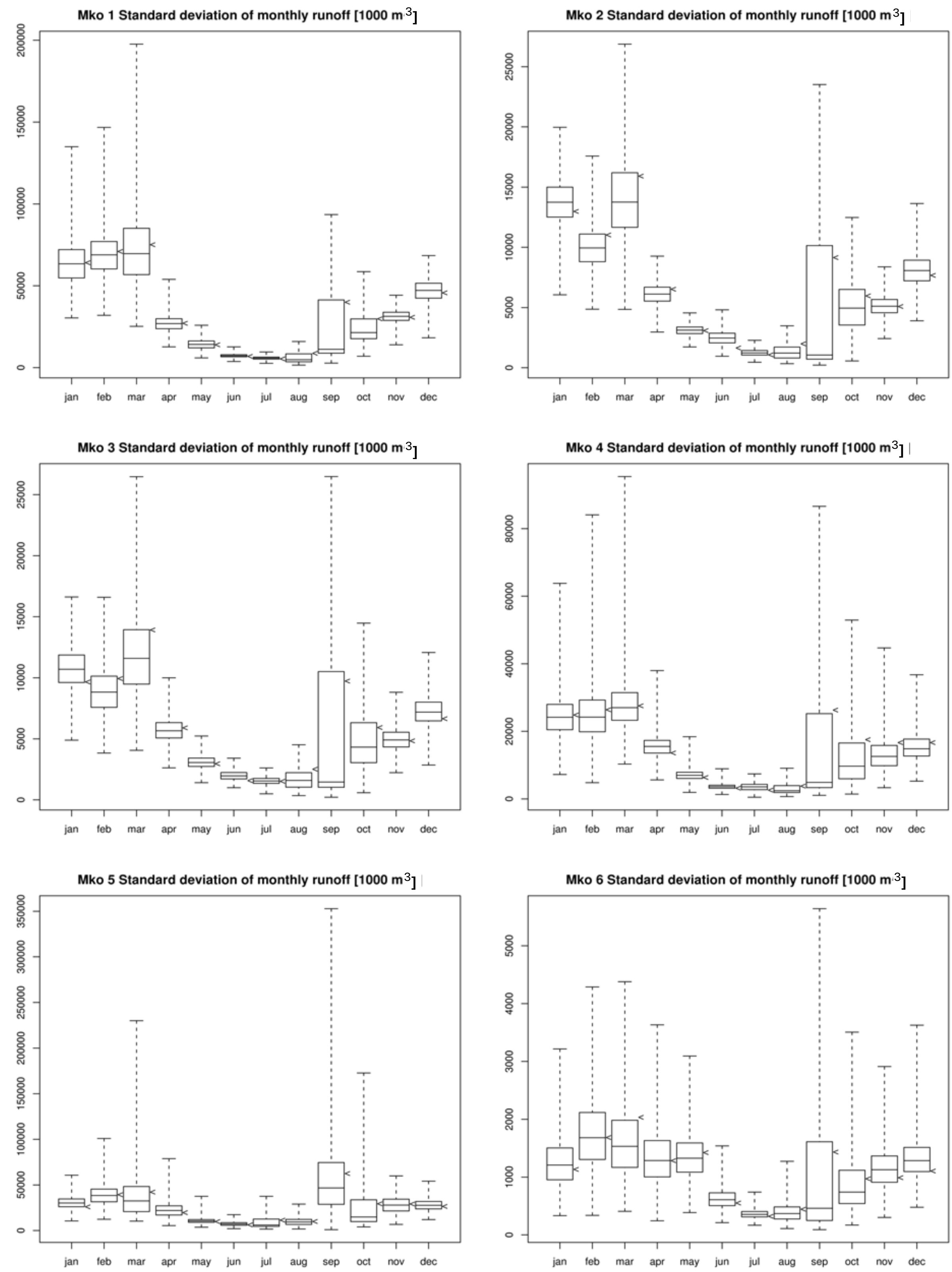


Fig. 6.9 Observed and generated standard deviation of monthly runoff at each site. Estimates obtained from historical time series are indicated by a "<".

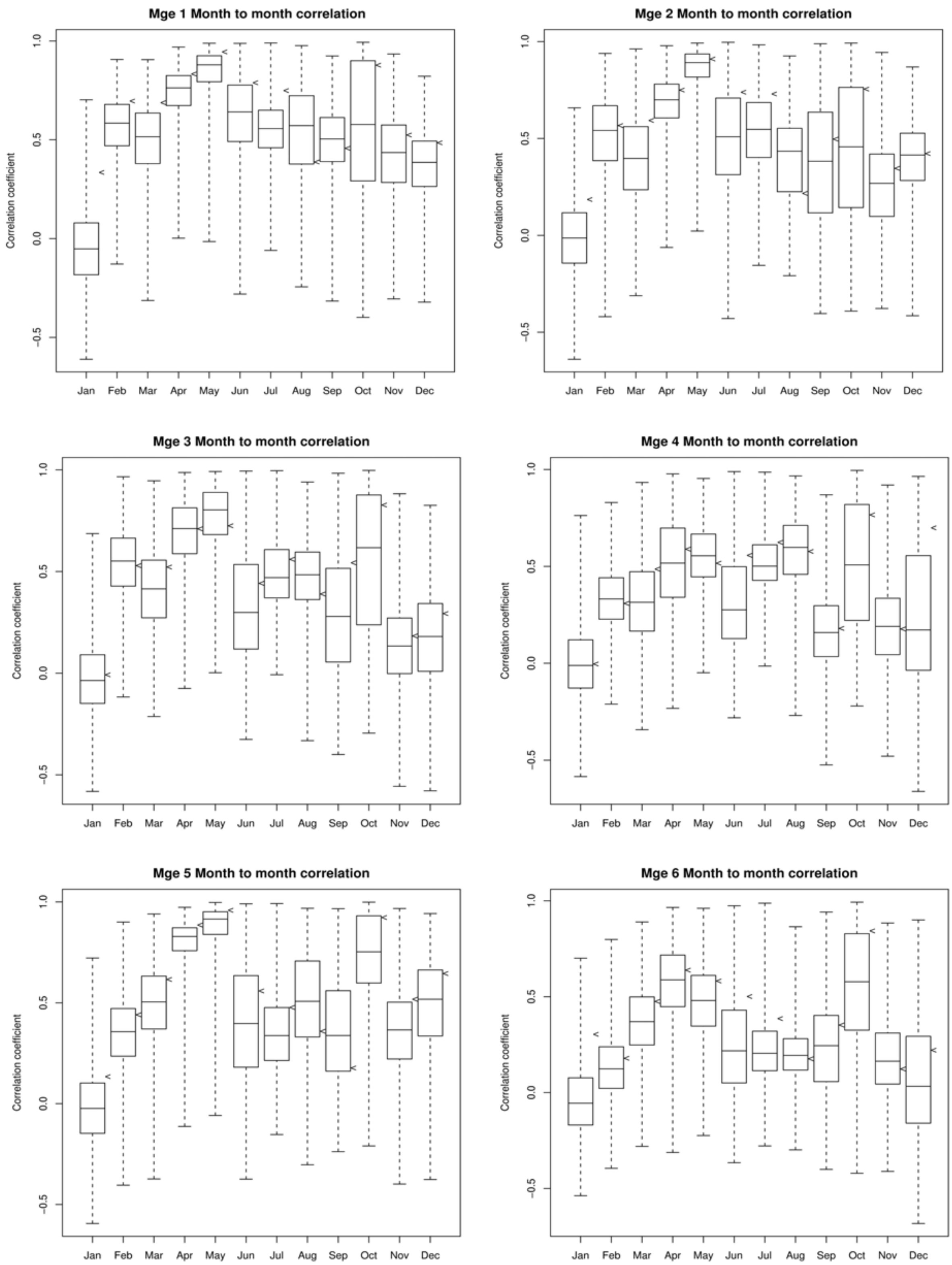


Fig. 6.10 Observed and generated month to month correlation at each site. Estimates obtained from historical time series are indicated by a ”<”.

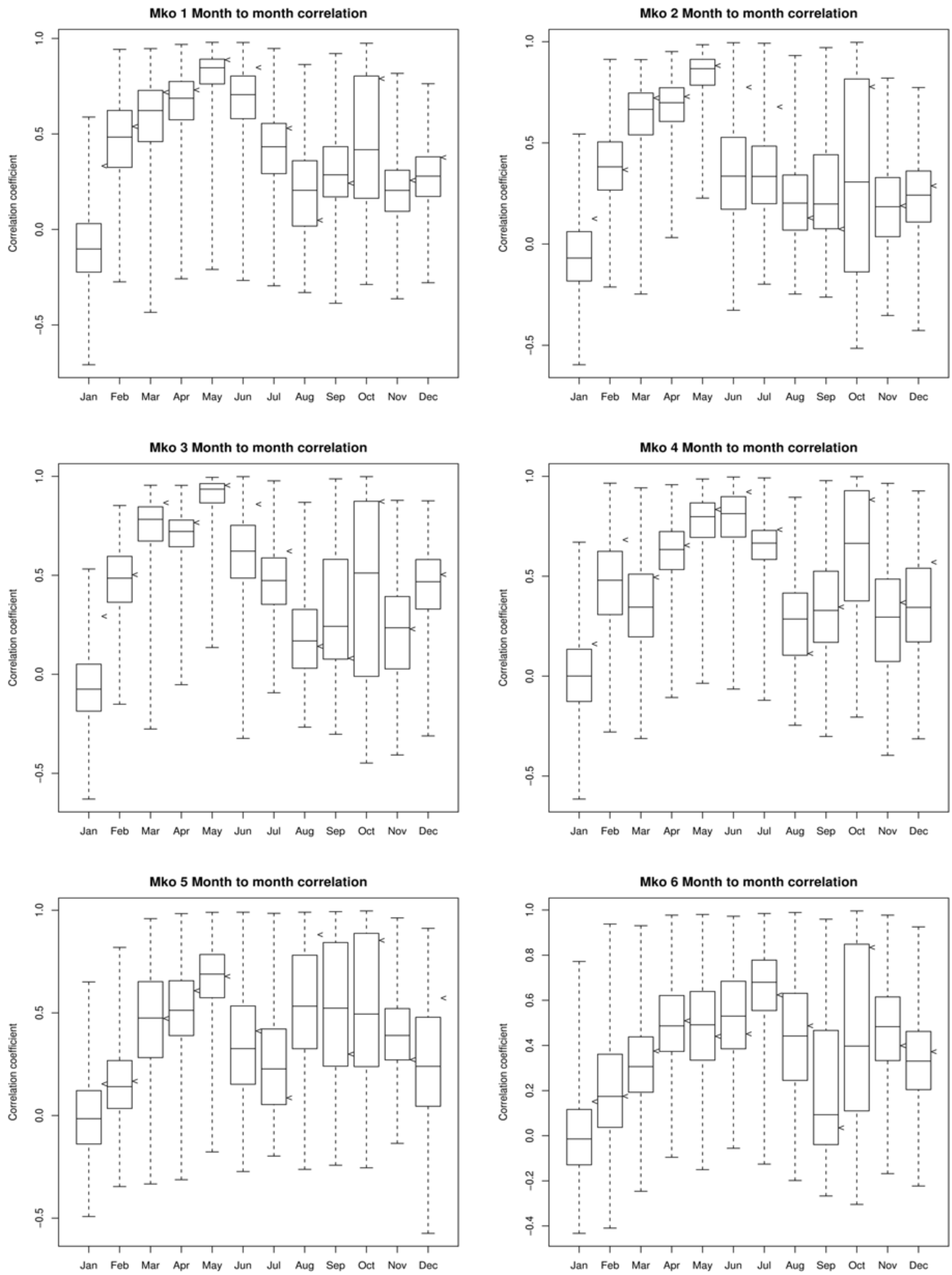


Fig. 6.10 Observed and generated month to month correlation at each site. Estimates obtained from historical time series are indicated by a ”<”.

Maximum deficit volume The maximum observed deficit volume observed in historical time series was compared to the corresponding estimates obtained from the generated time series in Fig. 6.11 with an annual demand corresponding to 80% of MAR. For all twelve sites the historical estimates falls within the 0.25 – 0.75 quantile interval of the estimates from the generated time series.

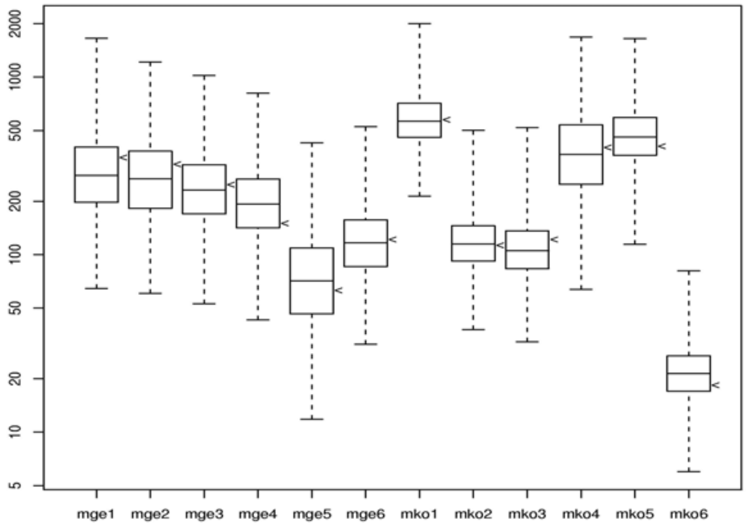


Fig. 6.11 Observed vs. simulated maximum deficit volume. Estimates obtained from historical time series are indicated with “<”.

Hence, the estimated CARMA(3,0) model is considered able to reproduce the maximum deficit volumes observed in the historical time series. This is considered an important result, as these events are responsible for the failure events measured by the water driven sustainability criterion.

Lag-zero cross correlation The lag-zero cross correlation between historical time series of annual runoff at the twelve different sites was compared to the corresponding estimates obtained from the generated time series. Due to the high number of possible combinations of sites (12*(12-1)/2 = 66), the historical and the mean value of the N = 5000 generated estimates are compared by plotting them against each other as shown in Fig. 6.12.

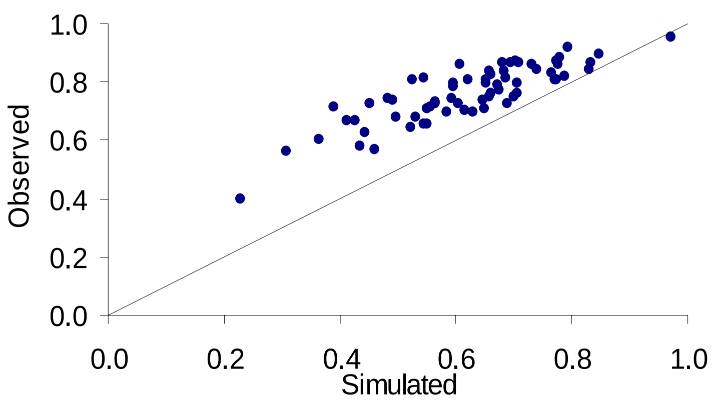


Fig. 6.12 Observed vs. simulated lag-zero cross correlation between time series of annual runoff.

The mean values of the estimates from the generated time series are lower than the corresponding estimates obtained from the historical time series. The absolute difference between the two estimates tends to decrease for increasing correlation. The average standard deviation of the estimates obtained through simulation equals 0.12, but this cannot explain the observed bias. The lag-zero variance-covariance matrix structure of the non-transformed multivariate normal distribution is reproduced in a satisfactory manner by the model as shown in Fig. 6.13.

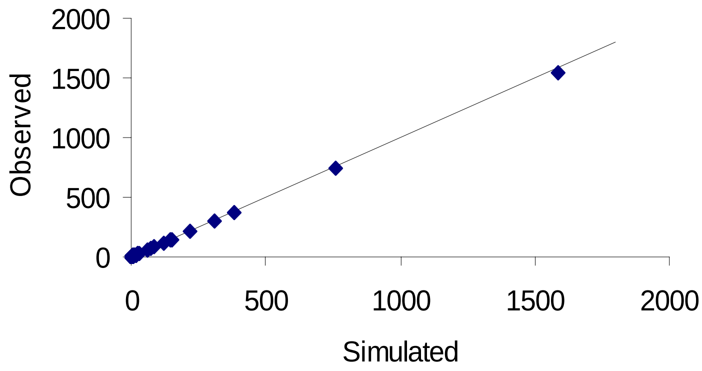


Fig. 6.13 Lag-zero variance-covariance of non-transformed annual runoff.

Hence, the bias must originate from the inverse Box-Cox transformation.

Lag-zero drought correlation The lag-zero correlation between deficit volume at different sites $r_d(i,j)$ is specified by Eq. (5.23). Estimates of $r_d(i,j)$ obtained from historical time series were compared to the mean value of the estimates from the N = 5000 generated time series, see Fig. 6.14.

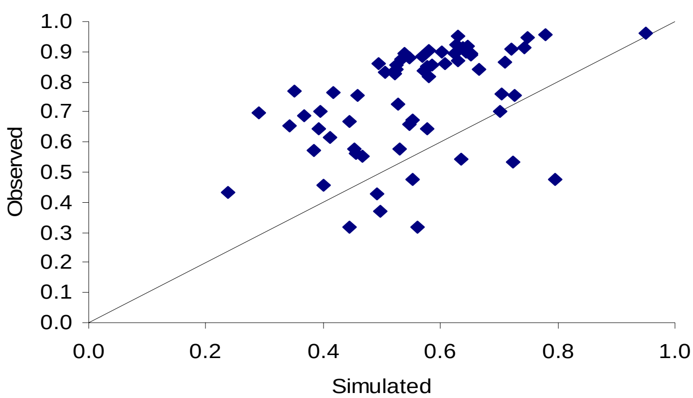


Fig. 6.14 Lag-zero cross correlation $r_d(i,j)$ between deficit volume at different sites.

From Fig. 6.14 it can be observed that the drought correlation obtained from the historical time series are slightly larger than the corresponding estimates obtained from the simulated time series.

Summary of CARMA modelling

A multivariate CARMA(3,0) time series model was developed based on 34 years of annual runoff. The model was verified and validated to ensure acceptable reproduction of the statistical properties of the historical data. In both the verification and validation the estimated model demonstrated the ability to reproduce these statistics with a satisfactory precision. Hence, the model is accepted and applied in the further analysis of the Mgeni-Mkomazi water resources system.

6.5 DEVELOPING SCENARIOS

The data material concerning the options for future water resources development has been compiled through interviews, literature surveys and visits to field sites. Following the terminology outlined in section 3.3 and section 5.4, the scenarios were constructed considering the following variables

Non-controllable input:	The future water availability is simulated using the specified CARMA(3,0) model.
Controllable input:	Parts of the MIKE BASIN model setup are changed to adequately model the defined scenarios, in particular water demand and additional reservoir development.
System parameters:	Remaining parts of the MIKE BASIN model setup, which are not considered as either a non-controllable or controllable inputs, are kept unchanged.

Planning horizon

An important issue in a sustainability analysis is the consideration of inter-generational equity and thereby the planning horizon. As discussed in section 3.1, considerations of future generations require an extended planning horizon. In this study the planning horizon is set to 30 years, starting from 2000 and ending 2030, as the historical records extend 34 years back in time. This may not be regarded as an extended planning horizon but limitations in the available data material was considered a barrier towards a further extension of the planning horizon. The planning period is sub-divided into three ten-year periods as required by the assessment methodology.

Water demand

In this study only increase in the water demand of the urban sector is considered, i.e. the water demand from the agricultural sector and the environment is assumed constants. The water demand data obtained from Umgeni Water show the monthly quantity of water abstracted from each reservoir. Therefore, these data do not coincide with any of the definitions of water demand offered by Kindler & Russell (1984) and discussed in section 5.3. It is not the specific demand belonging to a certain price level as the actual abstraction is subject to reduction in times of shortage, e.g. the period 1982-1983, as clearly seen on Fig. 6.3. Neither is it the requirement, as in times of sufficient supply the delivery will exceed the basic requirements. To estimate the future monthly water demand of the Mgeni-Mkomazi system a number of steps has to be followed. First, the future annual abstraction for the planning horizon is estimated for the inland and the coastal system. The inland system is represented only by the Midmar Dam. The coastal system consists of two dams, Nagle and Inanda and, therefore, the total coastal demand has to be divided into abstraction from the two dams, respectively. Finally, the annual abstraction is disaggregated into monthly demand based on historical patterns of water abstraction.

The future annual abstraction from the reservoirs is predicted by estimating a growth model based on the historical data and assuming this model to be valid within the planning period. The general growth model is discussed in section 5.3, Eq. (5.25) and its parameters are estimated using the least squares method together with historical annual abstraction from the inland and the coastal system from the period 1984-1997. This period was chosen, as the data from this period appear to be unaffected by climatic fluctuations like droughts. The 1. order growth model is considered to overestimate the future demand, hence the zero-order (linear) growth model is adopted. The annual total water demand in the coastal and inland systems are estimated using the following regression models

$$\begin{aligned} &\text{Inland system:} && D(t) = 32274872 + 2597623\,(t - 1984), \quad t \geq 1984 \\ &\text{Coastal system:} && D(t) = 119327424 + 11225068\,(t - 1984), \quad t \geq 1984 \end{aligned} \tag{6.1}$$

The outcome of these models corresponds with the development of water demand anticipated by Umgeni Water. From the historical data it can be seen that the total abstraction from the coastal system is distributed with 75% at Nagle Dam and 25% at Inanda Dam, i.e. the future total abstraction for the coastal system is disaggregated accordingly. The final step is to disaggregate the annual abstraction at each dam into monthly abstraction. To do this the fraction of monthly abstraction divided by total annual abstraction is calculated for each year. Next, the mean values of the monthly fractions are calculated for each month, and, finally, the predicted annual abstraction is disaggregated according to the mean fractions.

Water management options

The present system is considered unable to supply sufficient quantities of water in the future unless appropriate measures are taken to increase the yield of the system. A number of measures has been proposed, including both supply and demand oriented solutions:

1. Construction of a new reservoir on the Mkomzi River (interbasin transfer).
2. Raising the Midmar Dam.
3. Demand management in the coastal system.

Option 1 and 2 represent traditional supply oriented solution where a balance between demand and supply is obtained by increasing system capacity. Option 3 represents a demand-oriented approach, where demand management is introduced in an attempt to level supply and demand.

The Smithfield Dam option One solution to the problem of anticipated water scarcity is to build a reservoir on the Mkomazi River, the Smithfield Dam. Water from the dam is ultimately transferred to Umlaas road water works, which currently is fed with water from Midmar dam and supplies water to the inland system. The technical specifications of the reservoir are shown in Table 6.13.

Table 6.13: Dimensions of the Smithfield Dam.

Smithfield Dam	
Full Supply Capacity [10^6 m^3]	560
Height [m]	97
Transfer Capacity [m^3/s]	13.0
Inudated Area at Full Supply Level [km^2]	20

Rehabilitation of existing reservoir The Midmar Dam was constructed in 1965 to ensure reliable water supply to the urban, industry and agricultural sectors. In 1972 the water body and the surrounding area were handed over to the KwaZulu-Natal Nature Conservation Service and managed as a recreational resort, including a game park, camping area, picnic sites, sailing, fishing etc. It has been proposed to raise the existing Midmar Dam with an additional 3.61 m, from the existing 1043.9 m to 1047.5 m, in order to secure the water supply for the inland system. The expansion of Midmar Dam will increase the full supply level from currently 175 10^6 m^3 to 235 10^6 m^3 and the corresponding area of the water surface from 1564 ha to 1788 ha.

Demand management The main user of the reservoir system in the Mgeni catchment is Durban metropolitan. With an annual increase in connections of 25 000 and an unaccounted for water estimated to be around 40% (Durban Metro, 1999), the Durban Metro Water Service decided to initiate a demand management program. The program is aimed at achieving a zero percentage growth in water demand over a 4-year period (1999-2003). No demand management programs have been initiated for the inland system yet. Assuming zero order growth, the prediction of the future water abstraction in both the inland and the coastal systems are displayed in Fig. 6.15. To obtain the monthly abstraction the previously described disaggregation procedure is applied.

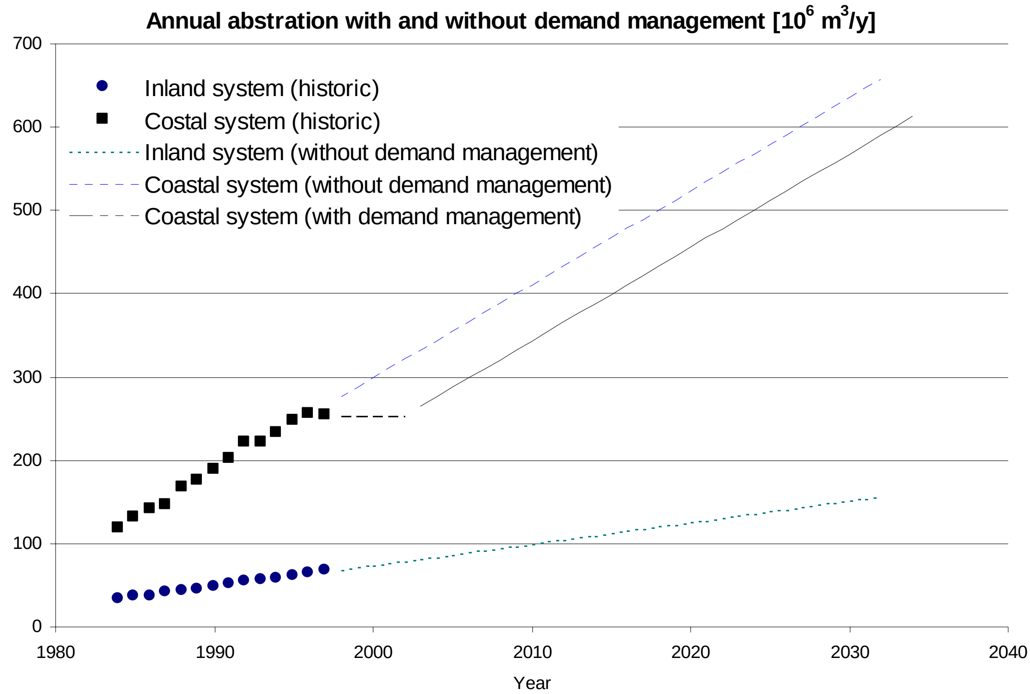


Fig. 6.15 Future annual abstraction with and without demand management [10^6 m^3/y].

According to Preston (1999) this water demand management program is widely regarded as the most successful program within South Africa. Naidoo & Constatinides (2000) classified the specific actions taken by the Durban Metro into five categories:

1. Passive operational and maintenance measures on the distribution system.
2. Proactive maintenance measures on the distribution system.
3. Customer demand-management measures.
4. New consumer demand-management measures.
5. Return flow management.

The specific measures initiated in each category are listed in Table 6.14 with reference to Naidoo & Constatinides (2000).

Table 6.14 Demand management actions initiated by the Durban Metro (Naidoo & Constatinides, 2000).

Demand management category				
Passive maintenance on distribution system	Proactive maintenance on distribution system	Customer demand management	New consumer demand management	Return flow management
Computerised 24h leak control	Pressure management	Introducing raising block-rate tariffs	Taylor made solutions	Waste water is sold to industry
Centralised control of reservoirs and pumps	Pipeline replacement	Zero rate of first 6000 l per months	Participatory approach	
Water balance between zone meters to detect leaks	Consumer meter management and replacement	Repair pluming within households in previously neglected areas.		
Monitoring of minimum night flow	New on-site leak detection tools	Increase frequency of meter readings		
	Upgrading mid block distribution system in previously neglected areas.	Introduction of informative billing		
		Credit control measures		

Scenario matrix

Six different scenarios have been constructed to account for different development paths, see Table 6.15

Table 6.15 Scenario matrix for the Mgeni-Mkomazi system.

Scenario	Demand	Smithfield Dam	Raise Midmar
1	linear growth		✓
2	linear growth	✓	
3	linear growth	✓	✓
4	linear growth + DM§		✓
5	linear growth + DM§	✓	
6	linear growth + DM§	✓	✓

§ DM = demand management

The scenarios have been selected to cover different combinations of future infrastructure and demand development. The Mooi-Mgeni transfer has been included in all scenarios, which is further elaborated in section 6.6. All scenarios are considered to be consistent, i.e. the specified demand and supply combinations are considered possible.

6.6 MODELLING SCENARIOS

A number of decisions has to be made concerning the modelling of the system. Most choices concerning spatial and temporal modelling scale have already been made through the set-up and calibration of the MIKE BASIN model. However, certain parameter values and modelling method still need to be addressed.

Modelling methodology

Each of the scenarios are modelled according to the methodology outlined in section 4.9 and Fig. 4.9. The planning horizon is set to 30 years and subdivided into 3 periods each of duration 10 years. For each 10 year period the average annual water demand is estimated and afterwards disaggregated into average monthly water demand. Next, 1000 years of synthetic monthly runoff is generated using the CARMA(3,0) model. Finally, water demand and availability are compared and sample estimates of R-R-V obtained. A number of choices concerning the operation of the water resources system has to be made beforehand.

Operating rules Each of the six scenarios requires a MIKE BASIN set up. The main difference between the model setup for calibration and the modelling of the scenarios is the operating rules of the reservoir system. During model calibration abstraction from reservoirs was specified from historical abstraction data. When modelling scenarios system operation guidelines must be be specified. In the following the operating rules adopted for different system components are described.

Inland system The Henley Dam was decommissioned in 1996 and is therefore not included in the future scenarios. The water for Pietermaritzburg is supplied by the Midmar Dam and also by the Smithfield Dam in scenario 2,3,5 and 6. The minimum downstream release (MDR) from Midmar Dam is specified in Table 6.1 to 0.9 m^3/s. From the model calibration it was found that release from the dam in a normal situation is 3.3 m^3/s. In scenarios where Midmar was raised to 235 10^6 m^3 the downstream release was reduced to MDR when active storage in the dam reached 50% of full capacity. In scenario 2 and 5, where the full capacity is 175 10^6 m^3, the downstream release was reduced to MDR when active storage reached 75% of full capacity.

Coastal system A number of pre-defined operating rules for the coastal system can be found in Table 6.5. In MIKE BASIN two reduction levels can be specified for reservoir operation which allows for a normal operating rule and two reduction rules. The following fixed rules have been chosen to imitate the anticipated operation of the coastal system: The DHTS water work draws water from Nagle and Inanda Dam. respectively. Initially, the system follows rule I but if the water level in Nagle Dam falls below the two specified reductions levels the system switches to rule L and finally rule O. Table 6.16 shows the distribution of DHTS's water demand as a function of reservoir level in Nagle Dam.

Table 6.16 Operating rules for Coastal system.

	I	L	O
Nagle Dam	100 %	69.6 %	37.2 %
Inanda Dam	0 %	30.4 %	62.8 %
Nagle water level [m]	> 396	396	387.5

The threshold values for shifts in operating rules are determined by the design of Nagle Dam. Wiggens water work draws water from Inanda Dam and only in the rare case that Inanda is unable to supply enough water, the remaining water will be drawn from Nagle Dam. The MDR from Inanda Dam is kept at a constant rate of 1.5 m^3/s. MDR from Nagle Dam is specified as 0.71 m^3/s. When the storage content in Nagle Dam is more than 71% of full storage capacity, 1.0 m^3/s is released. When storage volume falls between 71% and 20% only 0.4 m^3/s is released. Below 20% of full storage capacity, only 0.1 m^3/s is released. The chosen values were found to mimic the behaviour of the historic system better than the specified MDR. Finally, Albert Falls Dam was specified to release 4.48 m^3/s in order to obtain a reasonable fit between observed and simulated historic reservoir storage during model calibration. When the active storage falls below 2% of the full capacity, the downstream release is reduced to 0.45 m^3/s.

Inter-basin transfer The transfer from Smithfield Dam to the Mgeni system is modelled as a direct transfer from the reservoir to each of urban water demand centres. Reduction coefficients of (1,1) has been applied, i.e. no reduction in water transfer due to low water level in Smithfield Dam when water is needed in the Mgeni system. The downstream release from Smithfield Dam is specified to fulfil the IFR at site 4 downstream of the dam.

The reservoirs on the Mgeni River are all assumed to be full at the beginning of the simulation. Concerning the raising of the Midmar Dam, the term full is defined as the reservoir volume before raising of the dam wall. Smithfield Dam is assumed to be empty at the beginning of a simulation.

Mooi transfer Another supply-oriented solution is to transfer water from the nearby Mooi River. A constant transfer of water from the nearby Mooi River, see Fig. 6.2, of 4.3 m^3/s has been proposed. The transfer will enable the IFR values determined for the Mooi River to be fulfilled (DWAF, 1997).

6.7 EVALUATING SCENARIOS

The evaluation of the scenarios will be based on the methodology outlined in section 4.9, hence a combination of the revisited sustainability criterion and a RIAM investigation.

The water driven sustainability criterion

The first step in the evaluation procedure is to select the water users to be included in the sustainability assessment. For each user two aspects need to be considered before inclusion in the analysis:

1. User is impacted by one or more scenarios.
2. User is likely to experience failure periods in one or more scenarios.

Table 6.7 shows the water users, divided into sectors, that have been included in the analysis. A number of irrigation schemes from the model setup has not been included in the assessment as they did not fulfil either 1) or 2), or both.

Table 6.17 Water user included in the sustainability analysis.

Sector	User
Urban	Inland system, and coastal system (DHTS and Wiggens water works)
Industry	Paper mill
Agriculture	Two schemes downstream of Smithfield Dam
Environment	IFR4 on the Mkomazi River

After having selected the users to be included in the analysis, the system is simulated and criteria of R-R-V are estimated as defined in Eq.(4.22), i.e. using occurrence reliability, resilience estimated using the mean value of failure duration and vulnerability as the 0.9-th fractile of observed deficit volumes. Finally, the relative sustainability, rS_{WDC}, for each of the six scenarios is calculated through Eq. (4.8). The weights *w(c),* indicating the relative importance of the *c*-th user, are assumed equally distributed to all users. The score-card is shown in full in Appendix C and a summary is found in Table 6.18.

Table 6.18 Ranking of scenarios based on rS_{WDC}
(weights *w(c)* are distributed equally among users).

Scenario no	Res_1 Vul_3	Rank
1	0.191	6
2	0.825	4
3	0.825	3
4	0.309	5
5	0.835	2
6	0.836	1

From the results in Table 6.18 it can be seen that scenarios where the supply of water is increased through the construction of the Smithfield Dam (scenario 2, 3, 5, 6) have a significantly higher rS_{WDC} score than the scenarios without the Smithfield Dam (scenario 1, 4). The effect of raising the existing Midmar Dam is only recognisable at the third decimal when carried out in parallel to constructing the Smithfield Dam. The introduction of a demand management program in the coastal system has more effect than raising Midmar Dam, but not as significant as the Smithfield Dam. Comparing the two scenarios without the Smithfield Dam, the demand management program has a significant effect on rS_{WDC}, which increases from 0.191 to 0.309. When introducing the demand management program as well as constructing the Smithfield Dam the effect of demand management is much less significant, leading to an increase in rS_{WDC} from 0.825 to 0.835. Construction of the Smithfield Dam will ensure a fail-safe operation of the urban and the agricultural users in the first two time periods leading to a high score. The conclusion to this analysis is that a more ambitious demand management program is needed before the construction of the Smithfield Dam becomes obsolete in terms of matching water supply with water demand and thereby reducing the risk of water shortage.

The initial environmental evaluation

The scenarios identified in this study all include impacts that cannot be evaluated based on a comparison of water demand and supply. The RIAM methodology was adopted to include these impacts into the final sustainability assessment. The initial environmental evaluation (IEE) was conducted by reviewing existing literature, visits to the sites of concern and informal interviews with local people and authorities. The following impacts were identified for the construction of the Smithfield Dam and the raising of the Midmar Dam, respectively. No IEE was conducted in terms of the demand management program.

According to Takeuchi et al. (1998) the impacts of a dam construction project may be distinguished according to the development phase they fall under. The major impacts may be divided into impacts

- caused by construction of the dam
- caused by the formation of a reservoir
- related to the operation of the reservoir

Taking all these phases into account, a list of anticipated long-term major impacts not accounted for by the water driven sustainability criterion was identified for development option.

Smithfield Dam

Physical/Chemical components

PC1 – Impact on the visual/aesthetic character of the area

Situated in a picturesque part of KwaZulu-Natal a 97m high dam wall is going to have a negative effect on the aesthetic appearance of the area. Additional, there will be many unsightly waste and excavation sites due to the need for construction materials, which will not only be isolated around the dam site.

PC2 - Noise from construction

The construction of the reservoir will create noise pollution affecting neighbouring communities.

Biological/Ecological components

BE1 – Loss of flora

Many medical plants plus a few species in need of conservation are present at the dam site. A fairly unique plant, *Hydrostachys polymorpha,* is growing around a waterfall at the site. This species requires a highly specialised habitat and resettlement is not likely to be successful.

BE2 - Loss of habitat for terrestrial and avifauna

A nesting site for the Bald Ibis was identified at the dam site. The bald ibis is considered in need of conservation.

Social/Cultural components

SC1 - Inundation of human settlement

Approximately 27-35 households will be directly affected by the reservoir, but the entire community may demand resettlement. As described by Takeuchi et al. (1998) and WCD (2000) appropriate resettlement of people is an extremely important part of the overall sustainability of the reservoir.

SC2 - Inundation of graves

Disturbance of ancestral graves is considered a serious matter. Approximately 40 graves were identified within the area with the possible existence of more.

SC3 - Inundation of river crossing points

During low flow conditions people cross the river at various points. These crossings will be inundated for the length of the dam basin, affecting social ties and relations.

SC4 - Inundation of roads and bridges

The following major transport routes will be flooded.

- Himeville to Impendle transport route
- Mkomazana bridge and access road to Bulwer

SC5 - Loss of river stretch for canoeing

Strong preference from canoeing organisations for the river to be maintained in as pristine condition as possible.

SC6 - Impact on public health

Impoundment of a large water body could potentially favour pest and problem species such as blackflies[1], mosquitoes and snail vectors of bilharzia. On the other hand, the easy access to water supply could potentially reduce the risk of water borne diseases such as cholera.

SC7 - Impact on migration away from the area

The construction phase of the reservoir will provide short-term employment whilst the long-term presence of a reservoir, provided water is accessible to local people, would enhance small-scale farming opportunities. These factors could potentially help reducing migration away from the area.

Economic/Operational components

EC1 - Inundation of arable land

The reservoir will inundate 115 ha of arable land and 870 ha of grazing land.

EC2 - Improvement of infrastructure

Due to construction existing roads, power and communication infrastructure will be upgraded.

EC3 - Impact on local economy

Short- and long-term economic benefits are likely to emerge from construction of the dam. The increased investment in the area is likely to encourage growth and job creation.

EC4 - Impact on food security

The establishment of a large reservoir is likely to have a positive influence on local agricultural production through increased water availability. However, WCD (2000) reported on food shortage as a direct result of resettlement of people.

[1] Black flies are vectors of filarial parasites of mammals.

Raising of Midmar Dam

Physical/Chemical components

PC1 – Noise from construction

The construction of the reservoir will create noise pollution affecting neighbouring communities.

Biological/Ecological components

BE1 - Inundation of conservation land

The raising water level will inundate parts of the Midmar Game Reserve and 81 ha of Southern Tall Grassveld[2], which is approximately 9% of the total area of the game reserve. Only 1% of this veld type is conserved in KwaZulu-Natal and of this 12% is within the Midmar Game Reserve. The impact can be mitigated to some degree by purchasing of a neighbouring farm for expansion of the game reserve.

BE2 - Inundation of flora

A number of plant species of conservation value will be flooded including Yellow wood.

BE3 - Loss of wetland, new wetlands created ad shifts in wetlands

The largest wetland exists on a tributary to the reservoir. This wetland will be lost in its present state, but will migrate upstream and a new wetland will be created. Two smaller wetlands will increase in size as a result of raising the dam wall.

BE4 - Loss of habitat for terrestrial and avifauna

The inundation of the Midmar Game Reserve will have an impact on the habitat for 18 red data species including reptiles, birds and mammals. An operation Noah is planned to mitigate the impacts.

Social/Cultural components

SC1 - Impact on interest groups

The facilities, including club houses and slipways, for the Hilton Yacht Club, Natal Aquatic Power Boat Club and Midmar Boat Rental will be affected by the raising water level and requires relocation.

SC2 - Inundation of recreational land

Approximately 41 ha of picnic and braai[3] sites and 5.5 ha of campsites will be inundated.

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[2] Veld is the South African term for grassland.
[3] The South African equivalent of a barbecue.

Economic/Operational components

EO1 - Inundation of general infrastructure

The inundation of existing infrastructure within the Mgeni Game Reserve is estimated to be in excess of one million rand including the loss of access roads (gravel, grass, concrete and tar), bridges and pump houses.

EO2 - Creation of temporarily employment

Emphasis will be put on the use of local labour, which will have a positive impact on local economy and job creation.

It should be noted that Midmar Dam has existed for more than 35 years and issues such as environmental and social impacts most likely were not considered as important then as they are today. Hence, the construction of the reservoir might have been considered unsustainable in terms of present-days knowledge and value system. Today, however, the reservoir is an integrated part of everyday life in the area and nobody would dream of removing the reservoir to restore pristine conditions. Whoever or whatever was impacted when the reservoir was first constructed seems to be forgotten in the years that have passed since then.

For each of the considered impacts the five criteria (A_1, A_2, B_1, B_2 and B_3) defined by the RIAM methodology were assigned scores according to the guidelines outlined in Table 4.3. The individual scores were based on the perception of the author and formed through literature reviews and informal interviews with people involved in water resources management and various stakeholders. The final scorecards for construction of the Smithfield dam and the raising of the Midmar Dam are shown in Table 6.19 and Table 6.20, respectively. From these scorecard values of rS_{IEE} was estimated through Eq. (4.24) for each of the six scenarios outlined. In order to compare the two options in a meaningful manner each option is valued against a common set of issues consisting of all impacts from both reservoir options. If an impact is not relevant for the option under consideration, i.e. noise from construction of Smithfield Dam when raising Midmar Dam, then this particular option is assigned a no-change score, i.e. FAS = 0. The results are shown in Table 6.21 where only the relevant issues are shown for each option.

The outcome of the RIAM investigation indicates that the impacts from the Smithfield Dam are more negative than the impacts from the raising of the existing Midmar Dam. The lowest (worst) score was obtained when considering both projects. However, differences between the estimated values of rS_{IEE} are small when comparing the different options. An alternative method of evaluation considered each option independently by including only impacts of direct relevance for the option. This method, however, was discharged as the number of impacts in each category was found to have a large influence on the final score. This investigation involved a great deal of choices concerning the score of each component for each impact. These scores are based entirely on the perception of the analysts. However, it was felt that the scoring system developed in RIAM is well designed to achieve as objective an assessment as possible. Therefore, the values of rS_{IEE} obtained in Table 6.21 are believed to reflect a valid ranking of the scenarios with respect to their positive and negative impacts.

Table 6.19 RIAM results for the construction of the Smithfield dam.

Construction of the Smithfield Dam	FAS	A_1	A_2	B_1	B_2	B_3
Physical/cemical components (P/C)						
Impact on visual/aesthetic character of the area	-7	1	-1	3	3	1
Noise from construction	-4	1	-1	2	1	1
Biological/Ecological components (B/E)						
Loss of flora	-18	2	-1	3	3	3
Loss of habitat for terrestrial and avifauna	-18	2	-1	3	3	3
Social/cultural components (S/C)						
Inundation of human settlement	-24	1	-3	3	2	3
Inundation of graves	-21	1	-3	3	3	1
Inundation of transport routes	-14	1	-2	3	3	1
Inundation of roads	-28	2	-2	3	3	1
Loss of river stretch for canoeing	-14	2	-1	3	3	1
Impact on public health	-24	4	-1	3	2	1
Impact on migration away from the area	16	2	1	3	2	3
Economic/operational components (E/C)						
Inundation of arable land	-14	2	-1	3	3	1
Improvement of infrastructure	21	3	1	3	3	1
Impact on local economy	12	2	1	2	1	3
Impact on food security	18	3	1	3	2	1

Table 6.20 RIAM results for the raising of the Midmar dam.

Raising of the Midmar Dam	FAS	A_1	A_2	B_1	B_2	B_3
Physical/cemical components (P/C)						
Noise from construction	-5	1	-1	2	2	1
Biological/Ecological components (B/E)						
Loss of conservation land	-21	3	-1	3	3	1
Loss of flora	-7	1	-1	3	3	1
Loss/gain /shits of wetlands	16	2	1	3	2	3
Loss of habitat for terrestrial and avifauna	-8	1	-1	3	2	3
Social/cultural components (S/C)						
Impact on interest groups	-6	1	-1	3	2	1
Inundation of recreational land	-6	1	-1	3	2	1
Economic/operational components (E/C)						
Inundation of general infrastructure	-6	1	-1	3	2	1
Creation of temporal employment	4	1	1	3	1	1

Table 6.21 Results of RIAM investigation.

Scenario no.	rS_{IEE}	*Rank*
1	0.780	1
2	0.775	3
3	0.712	5
4	0.780	1
5	0.775	3
6	0.712	5

Results

The final sustainability assessment of the six scenarios are calculated through Eq. (4.25) and shown in Table 6.22

Table 6.22 Results of sustainability assessment.

Scenario no	rS_{WDC}	rS_{IEE}	$rS = rS_{WDC} \cdot \times rS_{IEE}$	Rank
1	0.191	0.780	0.149	6
2	0.825	0.775	0.639	2
3	0.825	0.712	0.587	4
4	0.309	0.780	0.241	5
5	0.835	0.775	0.647	1
6	0.836	0.712	0.595	3

By comparing the final sustainability score rS in Table 6.22 with rS_{WDC} from table 6.18 is can be observed that a new ranking of the scenarios has been obtained. The most sustainable scenario, i.e. the scenario with the best balancing of the future water supply and demand with a minimum of negative consequences is no. 5, construction of the Smithfield Dam together with a demand management program in coastal system. The benefits in terms of R-R-V gained by raising the Midmar Dam is counter-weighted by the potential negative impacts imposed by this development option as quantified by rS_{IEE}. These negative impacts also do not counter-balance the gain from demand management, i.e. the second most sustainable scenario is no. 2 which includes the Smithfield Dam, but neither demand management nor raising of Midmar Dam.

6.8 SENSITIVITY ANALYSIS

In the course of developing an optimal methodology for assessing the relative sustainability using risk criteria related to reliability, resilience and vulnerability a number of choices was made. These choices were based on either theoretical considerations or the outcome of a series of system experiments. In the following section a sensitivity analysis is carried out to investigate the practical importance of the these choices with focus on the following three aspects:

1. Choice of estimators of R-R-V.
2. Using the original Eq. (4.1) – Eq. (4.2) or modified Eq. (4.1) and Eq. (4.5) sustainability index.
3. The introduction of synthetic runoff compared to a repetition of the historical runoff sequence.

Furthermore, the significance of the weights attached to each water user signifying its relative importance is investigated. The effect of these changes will be discussed in relation to the ranking of the scenarios obtained using the assessment methodology outlined in chapter 4. The outcome of the RIAM investigation is assumed constant throughout the sensitivity analysis.

Choice of estimators

Based on he findings concerning overlap and required record length, the rS_{WDC} criterion will be estimated using the following combination of estimators

1. Rel Res_1 Vul_1
2. Rel Res_3 Vul_1
3. Rel no Res Vul_3
4. Rel no Res no Vul

A comparison of the estimates of rS_{WDC} obtained using these combinations are shown in Table 6.23.

Table 6.23: Results of sensitivity analysis regarding choice of estimators of R-R-V on the water driven criterion rS_{WDC}.

Scenario no	Org.	Rank	1.	Rank	2.	Rank	3.	Rank	4.	Rank
1	0.191	6	0.202	6	0.112	6	0.393	6	0.848	6
2	0.825	4	0.800	4	0.774	4	0.954	4	0.989	4
3	0.825	3	0.823	3	0.774	3	0.954	3	0.989	3
4	0.309	5	0.321	5	0.202	5	0.613	5	0.920	5
5	0.835	2	0.832	2	0.791	2	0.961	2	0.990	2
6	0.836	1	0.832	1	0.791	1	0.961	1	0.990	1

In practice, the use of different combinations of estimators appears to have no influence on the ranking of the considered scenarios. Furthermore, a combination of estimators of resilience and vulnerability based on mean values proved to be almost identical to scores obtained using a combination of a resilience estimator based on mean values and a vulnerability estimation based on the 0.9-th fractile of the empirical cdf of failure deficit volume. This may be due to the fact that the ratio between mean value and the 0.9-th fractile is almost constant. In Table 6.24 the effect of different combinations of estimators on the final sustainability assessment *rS* is shown

Table 6.24: Results of sensitivity analysis regarding choice of estimators of R-R-V on the final sustainability criterion *rS*.

Scenario no	Org.	Rank	1.	Rank	2.	Rank	3.	Rank	4.	Rank
1	0.149	6	0.164	6	0.090	6	0.306	6	0.662	6
2	0.639	2	0.600	2	0.581	2	0.739	2	0.767	2
3	0.587	4	0.461	4	0.434	4	0.679	4	0.704	5
4	0.241	5	0.260	5	0.163	5	0.478	5	0.717	3
5	0.647	1	0.624	1	0.593	1	0.744	1	0.767	1
6	0.595	3	0.466	3	0.443	3	0.684	3	0.705	4

Choice of method

In chapter 4 it was argued that the estimation of relative vulnerability (rVul) as defined by Loucks (1997) in Eq. (4.2) is inappropriate and could lead to bias in the ranking of scenarios. Instead a modified rVul was suggested as defined in Eq. (4.5). The practical implication of this modification is investigated. However, due to overlap (correlation) between estimators of R-R-V the effect on rS_{WDC} is assumed to be negligible. The results of the comparison are shown in Table 6.25.

Table 6.25: Results of sensitivity analysis regarding choice of method.

Scenario no	This study	Rank	Loucks (1997)	Rank
1	0.191	6	0.000	6
2	0.825	4	0.783	4
3	0.825	3	0.783	3
4	0.309	5	0.123	5
5	0.835	2	0.798	2
6	0.836	1	0.799	1

A difference in the obtained rS_{WDC} scores is evident but the final ranking of the scenarios is kept unchanged. The modification of the methodology by Loucks (1997) was introduced to enhance the multiobjective aspect of using different risk criteria. However, due to overlap between the estimators of R-R-V, the modification was expected only to have a minor practical implication, which is clearly evident from this investigation.

Synthetic vs. historical runoff

In section 4.5 it was shown that using time series of historical length could lead to non-uniform behaviour of the sample estimates of resilience and vulnerability. As a possible solution stochastic models for generation of long time series of synthetic runoff were introduced. It was found that using 1000 years of synthetic runoff generally leads to uniform behaviour of the sample estimators. The effect of using historical runoff instead of synthetic runoff in each of the three time periods is investigated. For each time period the average water demand is estimated and the historical time series (408 months) was repeatedly routed through the system and, finally, sample estimates of R-R-V obtained according to Eq. (4.22). A comparison between estimates of rS_{WCD} obtained through the two methods is shown in Table 6.26.

Table 6.26: Results of sensitivity analysis of rS_{WDC} comparing the use of synthetic vs. historic runoff.

Scenario	Synthetic runoff	Rank	Historic runoff	Rank
1	0.191	6	0.143	6
2	0.825	4	0.891	4
3	0.825	3	0.896	3
4	0.309	5	0.281	5
5	0.835	2	0.896	2
6	0.836	1	0.897	1

In this case study the process of estimating and applying the multisite stochastic model was found to have a limited effect on the outcome of the analysis compared to a repetitive use of historical time series. This is attributed to the large difference between scenarios with and without the Smithfield dam. Construction of the Smithfield dam ensures fail-safe operation for most users in two out of the three periods.

6.9 DISCUSSION AND CONCLUSION

The methodology outlined and discussed in chapter 2 to chapter 5 has been tested on an actual case study encompassing of the Mgeni-Mkomazi water resources system situated in a region of Southern Africa well endowed with water resources in terms of rainfall and runoff. However, due to ever increasing water demand, especially in the urban areas, different development options were considered to alleviate the anticipated future water scarcity.

Two existing setups of the ACRU model for the Mgeni and the Mkomazi catchment, both based on years of research and data collection, were made available. From local authorities data on water abstraction and operation of existing reservoirs were available. The data availability was generally satisfactory. From the ACRU model, time series of naturalised monthly runoff from areas between points of interest was obtained. The MIKE BASIN model was setup using these time series of naturalised runoff together with the water resources system data on water abstraction and reservoir characteristics. Using historical information, rules of reservoir operation were identified (in terms of downstream release) enabling MIKE BASIN to reproduce the observed reservoir storage and water abstraction on a monthly basis. Problems were observed in terms of reproducing the observed storage volume in the Nagle Dam. This particular reservoir is operated in conjunction with the much bigger upstream Albert Falls Dam, but, at present, it is not possible in MIKE BASIN to specify conjunctive operation of two or more reservoirs. Additional historical data concerning the inter-basin transfer of water from the Mooi-River to the Mgeni catchment might have improved the modelling results. A CARMA(3,0) was estimated based on the annual runoff from the twelve sites. Verification and validation of the model were based on a Monte Carlo study using N = 5000 simulations and in general satisfactory results were obtained. The model was able to reproduce mean and standard deviation of both annual and monthly runoff and the maximum observed drought at each site, but had problems reproducing the lag-zero cross correlation between annual runoff. Both the MIKE BASIN and the CARMA(3,0) model were accepted for use in the sustainability analysis of the case study.

A number of options for future development of the Mgeni-Mkomazi water resources system was identified and combined into six different scenarios. To evaluate these scenarios it was deemed necessary to introduce a number of simplifications discussed in the following. Water demand in both urban areas was assumed to grow according to a zero order growth model estimated based on historical data. A more in-depth approach combining demographic data with available forecasts of economic activity may result in different estimates, however, the data material for such an analysis was not available for this study. The agricultural water demand is assumed constant, which may also be a simplification of the future development. The choice of users to be included in the analysis was based on information of water using activities within the two catchments. The urban areas were characterised based on historical time series of monthly abstraction. Information concerning agricultural water demand was estimated using the ACRU menu based on surveys of area under irrigation, crops grown and irrigation practise. The agricultural water demand was estimated as the aggregated water demand for each of the twelve sub-catchments where irrigation was present, i.e. based on geographical proximity, a method well suited for the MIKE BASIN model structure. Other criteria for aggregation of agricultural water users into one user could be defined based on types of agriculture such as areas with similar crops or farm size, which may result in users representing a more homogeneous institutional or social group. These aspects would require further investigations and development of appropriate tools and necessitates a more multi-disciplinary approach.

Evaluation of the six scenarios based on rS_{WDC} resulted in a ranking where scenarios including the Smithfield Dam are ranked higher than scenarios without the reservoir. Both the raising of the Midmar Dam and the introduction of the demand management program in the coastal system have little influence on the results. An initial environmental evaluation (IEE) was conducted using the RIAM tool and summarised into the rS_{IEE} criterion. The IEE included all the factors not considered by the rS_{WDC} criterion and was based on a survey of existing literature, informal interviews with local water managers and visits to the sites of interest. The final scoring attributes of each factor are based on the perception of the analyst. The final sustainability score rS was obtained as a product of rS_{WDC} and rS_{IEE} and a new ranking of the scenarios was obtained. Constructing the

Smithfield Dam together with a demand management program in the coastal region was the scenario with the highest degree of sustainability, i.e. the best balancing of future water demand and supply with the negative impacts. The negative impacts from raising the Midmar Dam are larger than the gain in terms of rS_{WDC} and, therefore, rated as having a lower degree of sustainability. No negative impacts were associated with a demand management program, hence such a program will always improve the sustainability score. The urban areas in the Mgeni catchment account for the bulk of the water withdrawal. Even though the Smithfield Dam can eliminate the problem of water scarcity within the considered time period, further development may be necessary to satisfy the ever raising water demand in the system as well as in other neighbouring water resources system. Therefore, continued growth of water demand in the urban areas could be considered a potential barrier towards a sustainable water resources system. The recently initiated water demand management program in the coastal system is an important long-term measure to improve the sustainability of the water resources system.

As this case study does not include any scenarios that are obviously sustainable or non-sustainable it is difficult to evaluate the precision of the methodology. Instead, a sensitivity analysis was carried out to investigate the effect of the methodological extensions developed in this study to the original sustainability assessment method. It was found that the ranking of the scenarios based on the water driven criterion rS_{WDC} was very robust and did not change regardless of choice of estimators of resilience and vulnerability. If either resilience or vulnerability was omitted from the sustainability index, then the final ranking of the scenarios based on rS changed. Removing resilience or resilience and vulnerability makes the numerical deviation between the score for each scenario smaller, but did not alter the ranking of the scenarios. A modification of the original method by Loucks (1997) was introduced in section 4.1 to enhance the use of multiple risk criteria in the evaluation of system performance. The observed overlap between R-R-V unfortunately limited the practical significance of this modification. This was clearly observed in this case study, as the choice of estimators had no influence on the ranking of the scenarios. Finally, the effect of using stochastically generated time series of monthly runoff with an extension of 1000 years was compared to repeated use of the corresponding observed time series. Also this factor was found to have no influence on the ranking of the scenarios. This does not diminish the problem at hand concerning non-uniform estimators, but rather highlights the large difference between the performances of the scenarios. Construction of the Smithfield Dam ensures a fail-safe operation of most users during the first two time periods.

Chapter 7

THE MUPFURE WATER RESOURCES SYSTEM

The second case study for testing the methodology encompasses the Mupfure catchment situated in Zimbabwe. This catchment is less developed in terms of infrastructure than the South African. However, due to significant variability of runoff, both intra- and inter-annual, the region has experienced severe droughts and seen many water related conflicts during the last three decades. Furthermore, the available water resources are planned utilised through construction of large-scale reservoirs within the catchment and, hence, chosen as a suitable case study. Zimbabwe is a land-locked country in the semi-arid part of Southern Africa with a MAP from below 400 mm in the south western parts close to Botswana and South Africa and up to the excess of 1000 mm in the Eastern highlands bordering Mozambique. The country is inhabited by approximately 11 million people with a population growth of 1.8% p.a. (World Bank, 2001). As most other countries in the sub-Saharan region the Zimbabwean economy is dominated by the agricultural sector, which is also reflected in the national water use statistics. According to Mtetwa (1999) and Xie et al. (1993) 79% - 80% of the allocated water resources are earmarked for agriculture. In 1999 Zimbabwe adopted a new water act instead of the much criticised pre-independence water act dating back to 1976 (van der Zaag & Röling 1996; Kjeldsen et al., 1999b; Mtetwa, 1999). The process of revising the national water act led to the nominations of a number of catchments as test catchments where pilot projects concerning water resources management were conducted. The 12 000 km^2 Mupfure catchment was selected as a test catchment based on the variety of water users in the catchment and due to the previously experienced conflicts over scarce water resources during the prolonged droughts in 1982/83 and, in particular, 1991/92. Kjeldsen et al. (1999b) analysed barriers towards sustainable water resources management based on a case study from the Mupfure catchment. They identifed two types of problems related to the 1976 water act and the traditional perception of water resources management in Zimbabwe (and most likely other places as well), respectively. To alleviate future problems concerning water shortage it has been suggested that one or two major reservoirs should be constructed on the main river, ensuring reliable water supply to urban areas, as well as small scale and large scale commercial agriculture. The two reservoir schemes are named the Mhondoro Dam and the Muda Dam, respectively (Makoni et al., 2001). The aim of this case study is to:

1. Test the developed methodology for water resources system sustainability.
2. Assess the relative sustainability of a number of possible future water resources development scenarios.

It should be noted, that the field work constituting the bulk of the material for this case study was collected before the latest political unrest in the country, and, therefore, the results may no longer reflect the current situation in the catchment, but rather constitute a test of the developed methodology.

The Mupfure catchment is situated in the north eastern part of Zimbabwe, and the Mupfure River raises in the highlands approximately 100 km south of the capital Harare and flows westward until it joins the bigger Sanyati River, which is a tributary to the Zambezi River and drains into Lake Kariba (Fig. 7.1).

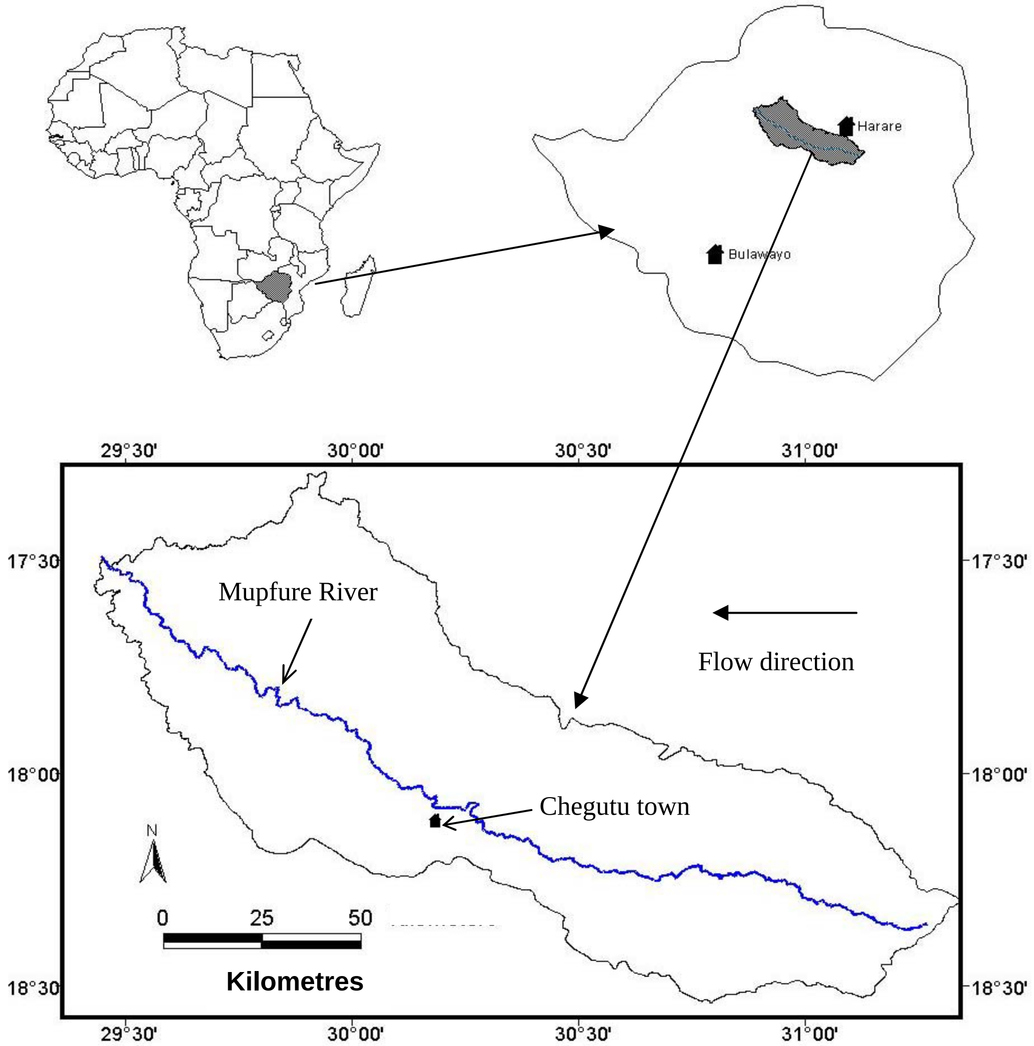

Fig. 7.1 Location of the Mupfure catchment.

This case study focuses on the 5180 km^2 upper part of the catchment, as this is where the water-related conflicts have occurred. Furthermore, virtually no records of hydrological measurements exist within the lower part of the catchment.

Data material concerning the Mupfure water resources system and the hydrology within the catchment was collected through interviews, literature, site visits and in collaboration with the Ministry of Lands and Water Resources, Zimbabwe and the University of Zimbabwe.

7.1 WATER RESOURCES ASSESSMENT FOR THE MUPFURE CATCHMENT

Before conducting a sustainability assessment as outlined in chapter 4 it was necessary to obtain information concerning spatial and temporal distribution of water resources within the upper part of the catchment. A water resources assessment was carried out through the use of the distributed hydrological modelling system ACRU developed at the University of Natal and documented by Schulze (1995).

The ACRU model

The ACRU modelling system was first developed in the early 1980s at the University of Natal, South Africa and is a physically conceptual model based on daily multi-layer soil water budgeting (Schulze, 1995), as illustrated in Fig. 7.2.

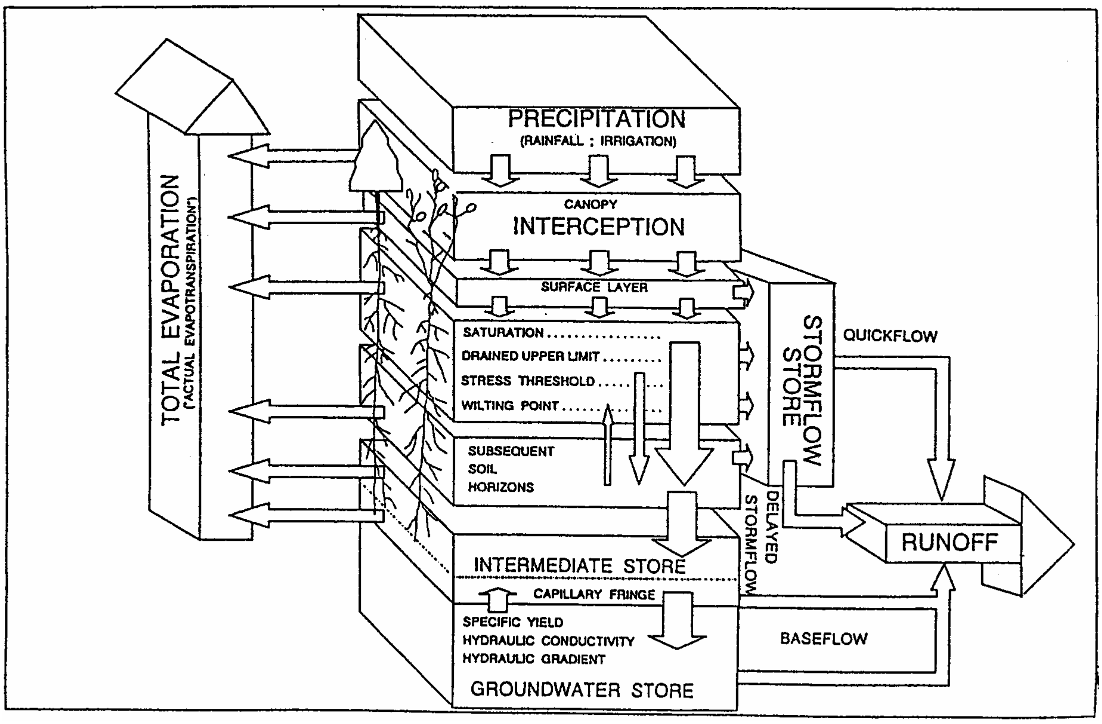

Fig. 7.2 Structure of the ACRU model (Schulze, 1995).

Since the first version the model has been further developed into the present-day system with the ability to simulate catchment hydrology and irrigation demand and supply. The model can be operated on a single catchment or as a spatially distributed cell-type model linking individual sub-catchments together in a network structure. The model is widely used in South Africa and its ability to model the rainfall-runoff processes in the Southern African region is documented through numerous applications as reported by Schulze (1995).

Data availability

The availability of hydrological data in Zimbabwe was a significant barrier throughout the study. Through close collaboration with the Ministry of Land and Water Resources and the University of Zimbabwe, sufficient data were collected for running the ACRU model for a 26 year period ranging from 1 January 1970 to 31 December 1996.

General catchment information

The upper part of the Mupfure catchment was delineated into 19 sub-catchments (Fig. 7.3) based on considerations of available streamflow gauging stations, existing and potential future reservoir sites, and socio-geographic conditions within the catchment.

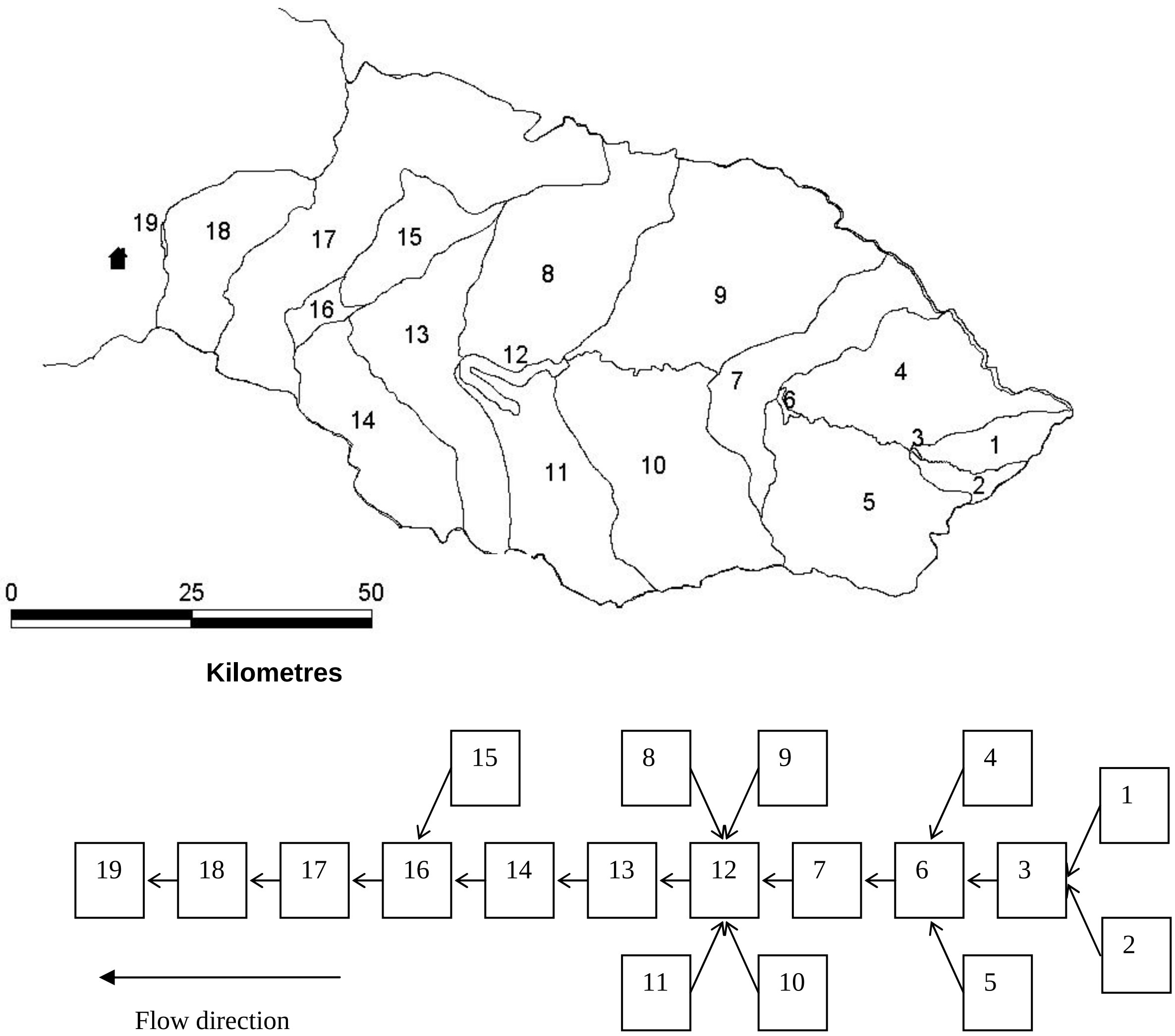

Fig. 7.3 Delineation and schematic setup of the ACRU model for the Mupfure catchment.

For each sub-catchment,, information concerning catchment area, mean altitude, latitude and longitude was obtained through the use of GIS and a digital elevation model. The catchment area for each sub-catchment is shown in Table 7.1.

Table 7.1 Area of sub-catchments specified in ACRU

Sub-catchment	Catchment area [km^2]	Sub-catchment	Catchment area [km^2]
1	93.7	11	349.8
2	43.0	12	39.5
3	0.9	13	392.8
4	382.3	14	326.9
5	453.4	15	167.9
6	4.8	16	40.6
7	287.2	17	776.6
8	515.1	18	273.7
9	695.7	19	1.9
10	578.0		

Schulze (1995) recommends that ACRU should not be applied to catchments smaller than 50 km^2. However, most sub-catchments are considerably bigger with an average size of 285 km^2. A further sub-division was not considered feasible due to, especially, the limited amount of available rainfall data.

Rainfall

Inadequately defined catchment rainfall input is, according to Hughes & Metzler (1998), the major problem when applying hydrological models to semi-arid areas. Therefore, a dense rainfall gauging network is of primary importance in Southern Africa. However, the socio-economic conditions prevailing in most regions of sub-Saharan Africa do not allow for the operation and maintenance of such networks, and the Mupfure catchment is no exception. Daily series of precipitation were collected from eleven sites within the considered part of the Mupfure catchment (Fig. 7.4).

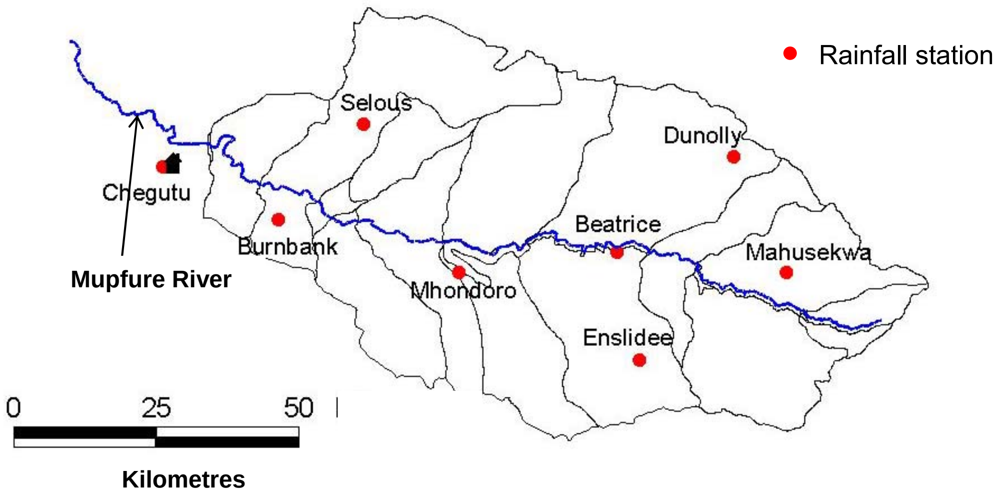

Fig. 7.4 Available rainfall stations.

The raw data were available only in the form of hand written records, which had to be digitised manually. Eight of the rain gauging stations closest to the river was used in the setup as this was found to give the best calibration/validation results. The location and description of the stations are shown in Table 7.2.

Table 7.2 Rain gauges used in ACRU set-up.

Name of rain gauge	MAP [mm]	Used in the following sub-catchments
Mahusekwa	776	1, 2, 3
Beatrice	625	4, 6, 7, 10
Enslidee	705	5
Mhondoro	687	8, 11 12
Dunolly	552	9
Burnbank	746	13, 14, 15, 16
Selous	684	17
Chegutu	787	18, 19

Inspection of the data revealed sequences of missing data from most stations and, therefore, extensive patching was necessary. The method used to patch the data is based on the assumption that the trend in the observed rainfall at two highly correlated neighbouring gauging stations is similar. The patching was carried out by selecting a highly correlated neighbouring station and, on days of missing data, in-fill the missing data with data from the selected neighbouring station, provided it had data on the days in question. Before filling in, the data were adjusted by the ratio of the mean monthly precipitation of the two stations under consideration. Double mass-plots were made to check the rainfall records for inconsistencies but no such was identified. For further description of the patching of the rainfall series from the Mupfure catchment see Makoni (2000).

Evaporation

No measurements of evaporation were available from within the catchment. However, based on available climatic data the ACRU modelling system provides a suite of build-in methodologies for estimating potential evaporation. For reasons listed by Schulze (1995) the ACRU model uses A-pan equivalents as reference potential evaporation. As only temperature data are available Schulze (1995) recommends using a slightly modified version of the method proposed by Hargreaves & Samani (1985) (referenced in Schulze, 1995) for estimation of monthly values of A-pan evaporation

$$E_{apan} = 1.25 \times 0.0023 R_a T_r^{0.5} \left(T_a + 17.8 \right) \tag{7.1}$$

where

T_r = range of monthly air temperature (°C)
T_a = monthly mean air temperature (°C)
R_a = extraterrestrial radiation (mm equivalents/day)

The extra terrestrial radiation R_a is given by Schulze (1995) and is a function of day-of-year and catchment position in terms of latitude. Time series of monthly maximum, minimum and average temperatures were collected from a climatic station situated in Mhondoro (sub-catchment no. 12) and used to estimated evaporation in the entire catchment.

Land cover

Land cover information was derived from the classifications of the Landsat TM image of 26 April 1998 obtained from the University of Zimbabwe. On the image land cover was grouped into eight categories. Each land use type was assigned to a standard build-in land use in the ACRU model as described by Schulze (1995) and shown in Table 7.3. The percentage distribution of each land use type within each of the 19 sub-catchments was estimated and, in each sub-catchment, the dominating type was used to specify the land-use, see Fig. 7.5.

Table 7.3 Land use classification adopted in ACRU from satellite images.

Landsat TM land-use classification	ACRU standard land use type (Schulze, 1995)	Description (Schulze, 1995)
Water	4020101	Dam
Wooded grassland/pasture	2010101	Woodland (indigenous savannah)
Cropland – late maturing crops	3020301	Double cropping-Maize and wheat
Forest	2030107	Bush/veld general
Wetland	4040102	Wetland-grasses
Grassland/pasture	2030102	Veld in poor condition
Cultivated land	3020103	Maize – general
Bare ground	1040201	residential informal, rural

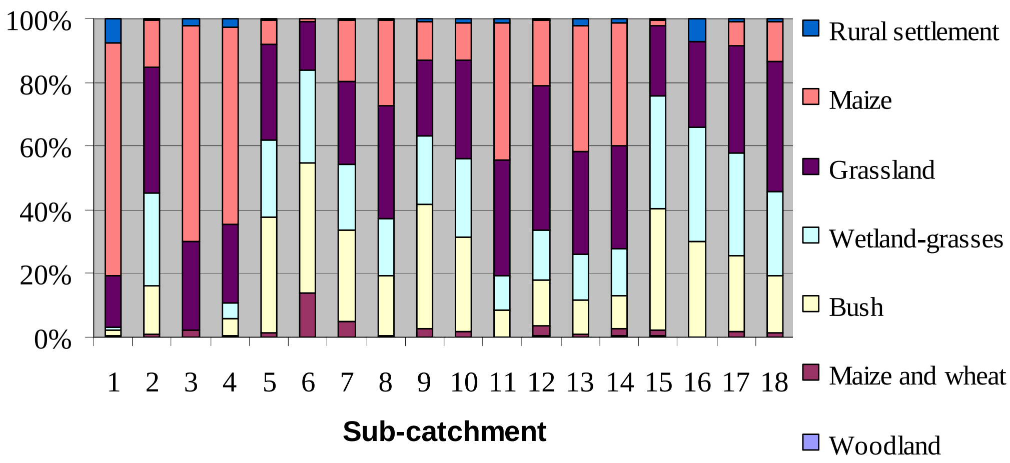

Fig. 7.5 Land use types within each sub-catchment.

For each standard ACRU land-use type a set of pre-programmed values of crop water coefficients, leaf area index, interception losses and fraction of active root mass in topsoil are available on a monthly basis.

Soils

Considering necessary catchment soil information, the ACRU model, as a minimum, requires soil texture class and soil horizon thickness (Schulze, 1995). Soil maps indicate that in the upper part of the catchment sandy clay loam dominates, whereas the lower part consists of loam (Makoni, 2000). From a study of the hydrology of a neighbouring (south) catchment Refsgaard & Knudsen (1996) reported soil depths between 0.5 m – 0.6 m. These values have been used in the ACRU setup for the Mupfure catchment.

Surface water reservoirs

At present no large reservoirs exist in the catchment but a number of smaller reservoirs have been constructed, especially in areas dominated by commercial agriculture. Reservoirs within the catchment can be categorised into private dams and government dams. Private dams are farm dams used to store water for irrigation of commercial agriculture, such as wheat, horticulture and tobacco. The Ministry of Lands and Water Resources, Zimbabwe, has compiled a database of granted storage rights, i.e. rights granted to users allowing them to store water for later use. As a direct consequence of the drought experienced in 1982/83 the number of storage rights in areas dominated by commercial agriculture increased to ensure future water supply for irrigation. To account for this anthropogenic influence, storage volume was specified for two periods, before 1983 and after 1983. For both periods the storage volume was estimated as the cumulative storage rights at the end of the period, see Table 7.4. It is assumed that a storage right is utilised to its full extend, i.e. the size of the farm dams corresponds to the granted storage right.

Table 7.4 Storage capacity within sub-catchments in 1997 and (1983).

Sub-catchment	Storage volume [1000 m^3]	Sub-catchment	Storage volume [1000 m^3]
1	0 (0)	11	7 (7)
2	0 (0)	12	0 (0)
3	0 (0)	13	2387 (2354)
4	21 (0)	14	2855 (2854)
5	1370 (1095)	15	64 (64)
6	0 (0)	16	196 (114)
7	5856 (1070)	17	5080 (3092)
8	109 (109)	18	1870 (1870)
9	11647 (4176)	19	0 (0)
10	2025 (1548)		

A number of minor government dams exist but are generally of limited storage capacity, see Table 7.5. The government dams have been added to the farm dams in the sub-catchments where both exist.

Table 7.5 Government owned dams.

Name of Dam	Storage capacity [1000 m^3]	Year of construction
Mahusekwa	3.0	1987
Upper Seignury	1.1	1970s
Lower Seignury	0.8	1950s
Maynard	1.8	1965
Poole	4.5	1940s
Clifton	11.0	1986
Twyford weir	0.9	1950s
Railway weir	0.2	1950s

The off river storage Clifton Dam has been added to the model as water from the river is pumped into the reservoir.

Abstractions

Abstraction of surface water for both the urban and agricultural sectors takes place within the catchment. The only significant urban user is the town of Chegutu situated at the outlet of the considered part of the catchment and, therefore, not included in the ACRU setup. Other water abstraction for domestic purposes is not included in the model setup as it was considered insignificant. Agriculture is the most water consuming sector and can be divided into three sub-sectors: commercial agriculture, communal agriculture and resettlement agriculture. A variety of crops is cultivated within the catchment such as wheat, tobacco, citrus, soybeans and mealies. The commercial farming sector alone occupies 65% of the area in Mupfure catchment, and is also by far the most water consuming sector. Most of these farms are developed using modern machinery and large irrigation systems as well as private-owned reservoirs. The commercial farming areas are placed along the Mupfure River and its tributaries, and all farms have permanent rights to abstract water from the river. The communal areas occupy approximately 30% of the area in Mupfure catchment. They were formed before the independence in 1980 by the former Rhodesian authorities as reservations for native Africans. These areas are characterised by the highest population density and the least fertile land. Only few irrigation systems, reservoirs, water rights and little machinery exist in the communal areas and production is minor compared to the commercial areas, and mainly used to supply the areas themselves. Water is mainly supplied as primary water[1] or water supply schemes owned and operated by the government. The resettlement areas were created after the independence by reallocating commercial farming areas to Africans coming from the communal areas. The resettlement areas occupy 4% of Mupfure catchment. These areas are not as densely populated as the communal areas, and the land is also more fertile.

As for storage rights, the Ministry of Lands and Water Resources has a data base of issued abstraction rights, i.e. rights granted allowing users to draw a certain volume of water, specified on a monthly level, from a river. Potentially, the water rights could be used as an estimate of the agricultural water demand within the catchment. However, no or only little control of is exerted to ensure that users do not abstract more water than they are entitled to, and illegal abstraction is a widespread problem throughout Zimbabwe (van der Zaar & Röling, 1996). On the other hand, the water right only specifies what users are allowed to abstract and in years of good rain they might abstract less water. In lieu the irrigation water demand routine in the ACRU model was used to estimate the net demand for irrigation within each sub-catchment. Based on pre 1980 records of water use and irrigation, Makoni (2000) found a figure of 12.0 10^3 m^3/ha as an estimate of average water demand in commercial agriculture. Using this figure together with the total volume of water issued as water rights within each sub-catchment, an estimate of the area under irrigation was obtained. In areas without water rights, which comprise some sub-catchments within the communal areas, no irrigation of significance was assumed to take place. Schulze (1995) gives a detailed description of the irrigation module. For modelling purpose all year irrigation was assumed of a fictive crop with crop coefficient of 0.8 and initial interception of 0.1 mm/rainday, which appears to be suitable values according to Schulze (1995) as an average crop. Despite the low potential for groundwater development, abstraction from the groundwater is evident within the catchment, especially for commercial agriculture during periods of low runoff. However, no records of number of wells or actual abstraction exist.

Runoff

A number of flow gauging stations exists on the main river and its tributaries, however only two of these have records of sufficient reliability to be used for model calibration and validation. The positions of the two gauging stations are shown in Fig. 7.6.

The most upstream of the two stations is C70, which is located on the main river approximately 7 km west of the Harare-Johannesburg highway at the outlet of sub-catchment no. 7. This gauging station started operating in 1969 and, according to MLWR (1997) the data quality is generally good. The second gauging station C12 is situated 80 km further downstream under the bridge where the Harare-Bulawayo highway crosses the main river at the outlet of sub-catchment no. 18, and has been operating since 1950. According to MLWR (1997) station C12 is not good at measuring low flow but no quantification of the extent of this error was available. The hydrological characteristics of the recorded flow are shown in Table 7.6.

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[1] Primary water is defined as water for covering basic human needs, which can be abstracted without a water right.

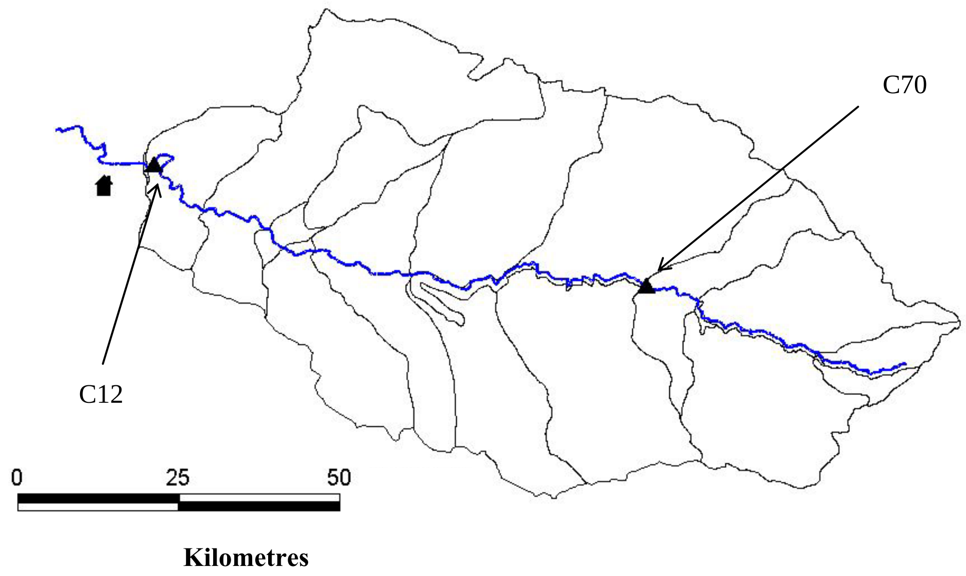


Fig. 7.6 Location of runoff gauging stations.

Table 7.6 runoff characteristics at available gauging stations.

Gauging station	Period of record	Contributing area [km^2]	MAR [m^3]	Cv of MAR	Comment
C70	Oct 1969 – Dec 1995	1215	104 10^6	1.42	Naturalised flow
C12	Oct 1950 – Dec 1995	5180	343 10^6	1.23	Problems with low flow

Time series plots of monthly runoff from both stations are shown in Fig 7.7a,b. These plots reveal significant inter- and intra-annual variation in runoff with the bulk of flow occurring during the wet season from October to March. Periods of no flow are often recorded during the dry season at both gauging stations. There appears to be a shift happening in the hydrological regime around 1981.

After this year a significant reduction of maximum runoff and inter-annual variation occurs. The reason for this shift is not obvious but could be related to factors such as errors in measurements, increasing water abstraction and impoundment or large-scale climatic fluctuations. Analysis of water right data showed a significant increase in both abstraction and impoundment after 1980. At the same time research by Makarau (1999) indicated a downward trend in Zimbabwean rainfall. Both factors could attribute to the observed shift in runoff but more research is needed for final conclusions to be drawn.

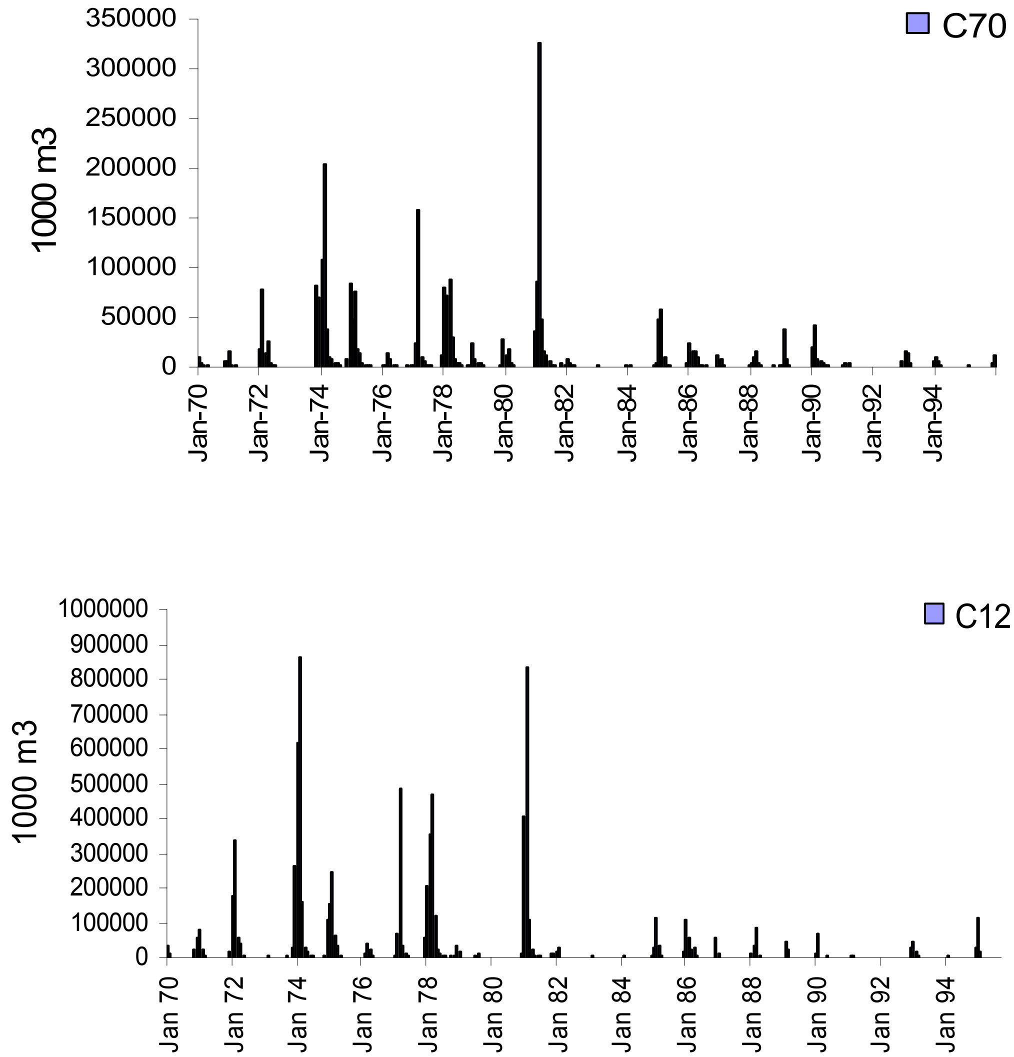


Fig. 7.7 Time series of observed monthly runoff at gauging weir a) C70 and b) C12.

Model calibration and validation

According to Schulze (1995) the ACRU model should not be calibrated in the case of unacceptable discrepancy between observed and simulated runoff. Rather additional data material should be collected to enhance knowledge of the hydrological system under consideration. Due to the shortage of hydrological and water resources system data this appealing procedure could not be applied in this study and, instead, the ACRU model was calibrated and validated against available runoff data by adjusting certain model parameters. To measure the performance of the model in terms of ability to reproduce observed streamflow a number of numerical criteria and illustrations were defined. These criteria have previously been used in studies of the performance of rainfall-runoff models in semi-arid Africa by Schulze (1995), Refsgaard & Knudsen (1996), Lørup et al. (1998) and Andersen et al. (2001).

1. Time series plots of observed versus simulated monthly flow

2. Plots of observed versus simulated annual runoff

3. Plots of observed and simulated mean monthly runoff

4. The overall mass balance D_v measuring the percentage difference between observed flow $q^{(o)}$ and the corresponding simulated flow $q^{(s)}$ over a time period of T months as

$$D_v = \frac{\sum_{t=1}^{T}\left(q_t^{(o)} - q_t^{(s)}\right)}{\sum_{t=1}^{T} q_t^{(o)}} \tag{7.2}$$

5. The coefficient of determination R^2 calculated based on monthly runoff as

$$R^2 = 1 - \frac{\sum_{t=1}^{T}\left(q_t^{(o)} - q_t^{(s)}\right)^2}{\sum_{t=1}^{T}\left(q_t^{(o)} - \bar{q}^{(o)}\right)} \tag{7.3}$$

where $\bar{q}^{(o)}$ is the mean value of monthly runoff.

According to Schulze (1995) the D_v for a semi-arid catchment should ideally be below 5% but more realistically below 10%, and the R^2 coefficient should be above 0.75 based on monthly runoff. Andersen et al. (2001) suggested four intervals for D_v and R^2 for assessing model performance, also based on monthly runoff (Table 7.7).

Table 7.7 Model performance criteria adopted from Andersen et al. (2001)

Performance	D_v [%]	R^2
Very good	< 5	> 0.95
Good	5 – 10	0.85 – 0.95
Fair	10 – 20	0.70 – 0.85
Poor	> 20	< 0.70

Model calibration

The calibration and validation exercise has been carried out through a split-sample test (Refsgaard & Knudsen, 1996) dividing the period of observed data into a calibration period (1970-1985) and a validation period (1986-1996). The model was calibrated using differential split-sample trial and error calibration (Refsgaard & Knudsen, 1996). For the calibration period the reservoir storage for each sub-catchment was set corresponding to the 1983 level as indicated in Table 7.4. For the validation period the 1997 level of reservoir storage volume was used. The number of parameters in the ACRU model combined with the relative sparse data material can indeed make a calibration a very cumbersome procedure. However, Makoni (2000) conducted a sensitivity analysis and found that the parameter CORPPT, which corrects daily precipitation and can be specified on a monthly basis for each sub-catchment, was the most sensitive parameter. The calibration was carried out mainly, but not solely, by adjusting this parameter. The results of the calibration, in form of the five specified model performance criteria, are shown in Table 7.8 and in Fig. 7.8 to Fig. 7.10.

Table 7.8 Numerical performance criteria for the calibration period (1970-1985).

	D_v [%]	R_2
C70	2.40	0.83
C12	0.86	0.78

In terms of both the numerical performance criteria the calibration results must be perceived as satisfactory with respect to both gauging station C70 and C12. The comparison of the time series of monthly runoff at both gauging stations in Fig. 7.8a and Fig. 7.8b shows that the timing of the occurrence of flow is well captured by the ACRU model. The magnitude of the flow is also reproduced in a satisfactory manner. Both over- and under-estimation do occur, especially in the beginning of the period, but no bias towards one or the other is evident from the figures.

The model’s ability to reproduce annual runoff was investigated by plotting observed versus simulated annual runoff as shown in Fig. 7.9a and Fig. 7.9b. The hydrological year has been defined as November to October, which is further elaborated in section 7.4. A line representing the 1:1 relationship between observed and simulated runoff as well as the linear relationship estimated using least squares estimation have been added to the plot.

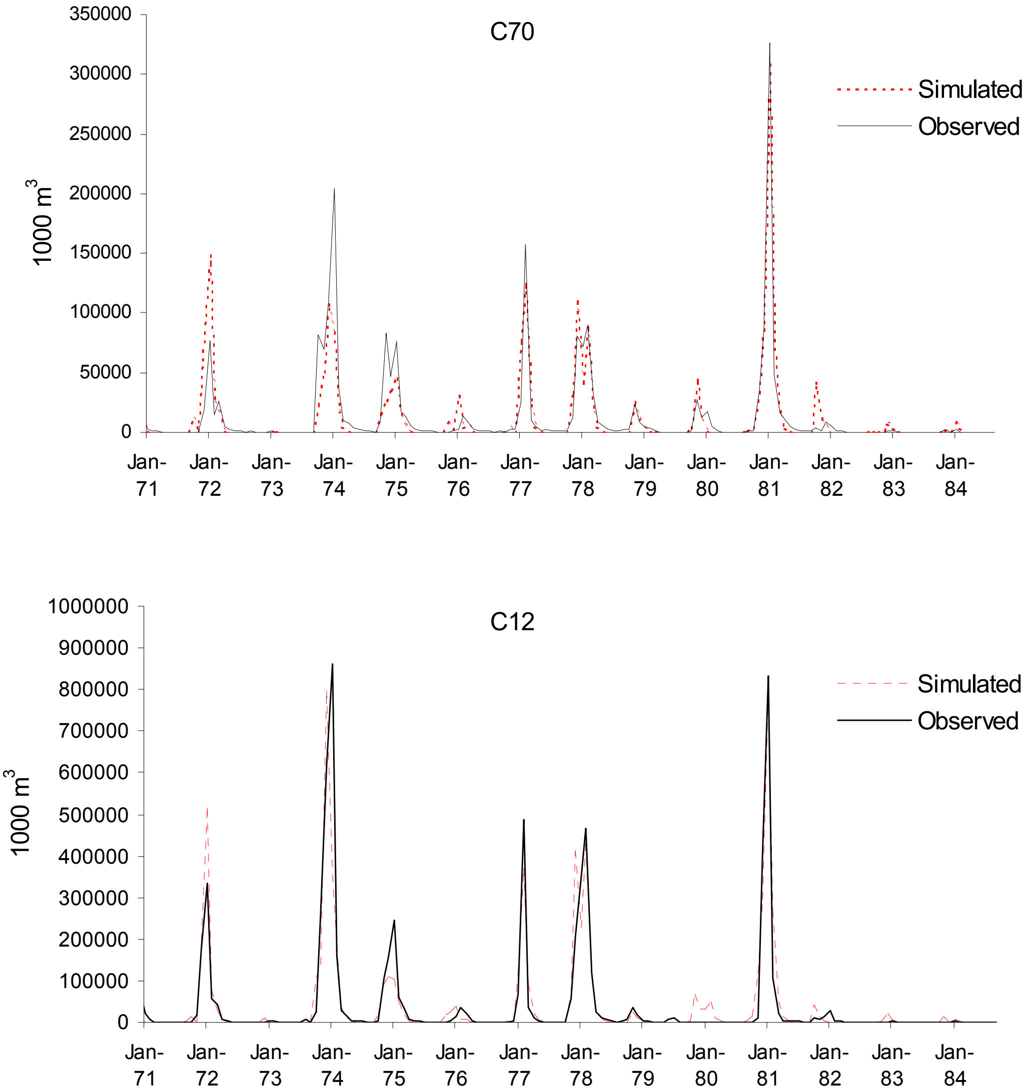


Fig. 7.8 Comparison of observed and simulated time series of monthly runoff (1970-1985) at C70 and C12.

Statistical tests as described by Montgomery (1991) were conducted to investigate if the linear regression model fitted to the data could be described by a line with unit slope and zero interception. The test statistics are shown in Table 7.9, and for both C70 and C12 the hypothesis of unit slope and zero interception is accepted at a 5% level of significance. These results indicate that no significant bias in terms of over- or under-estimation of annual runoff is present at the two gauging stations. This conclusion is supported by visual inspection of the plots on Fig. 7.9a and Fig. 7.9b.

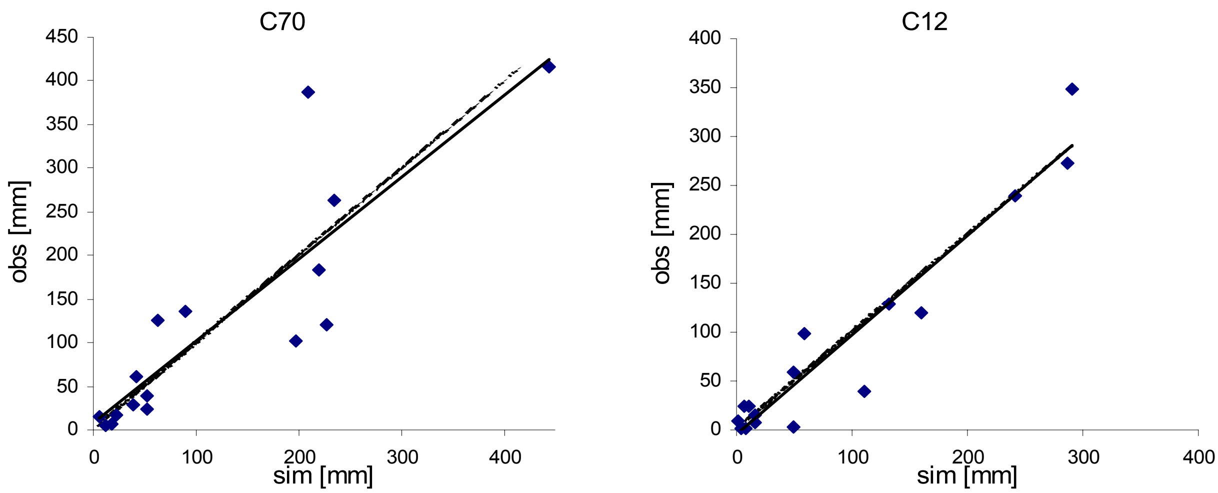


Fig. 7.9 Observed versus simulated annual runoff (1970-1985) at gauging weir a) C70 and b) C12.

Table 7.9 Statistics for comparison of simulated and observed annual runoff.

	Slope	Interception
C70	-0.41	-0.32
C12	0.18	-0.40

Finally, the observed and simulated mean monthly runoff is compared in Fig. 7.10a and 7.10b.

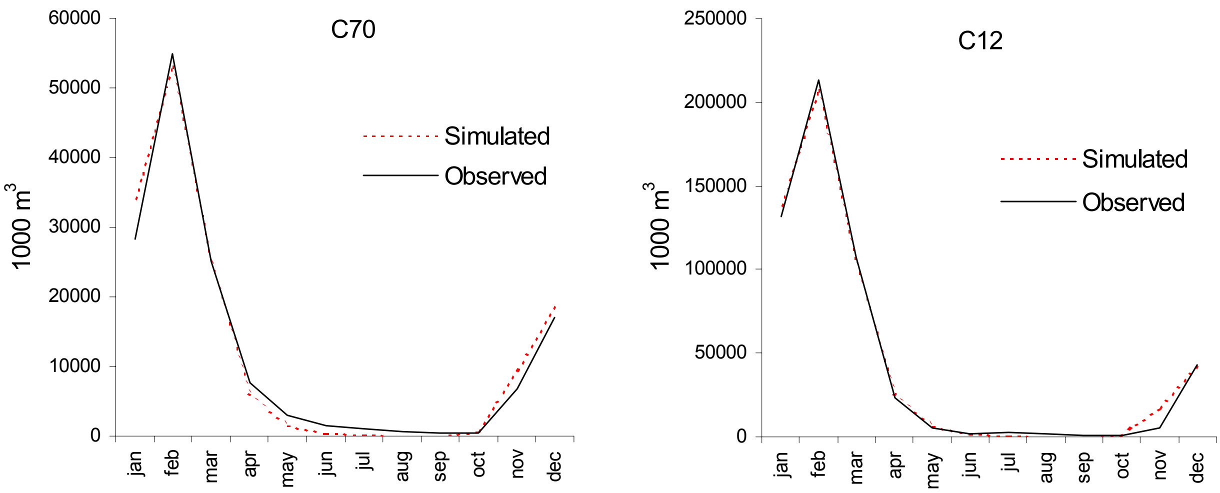
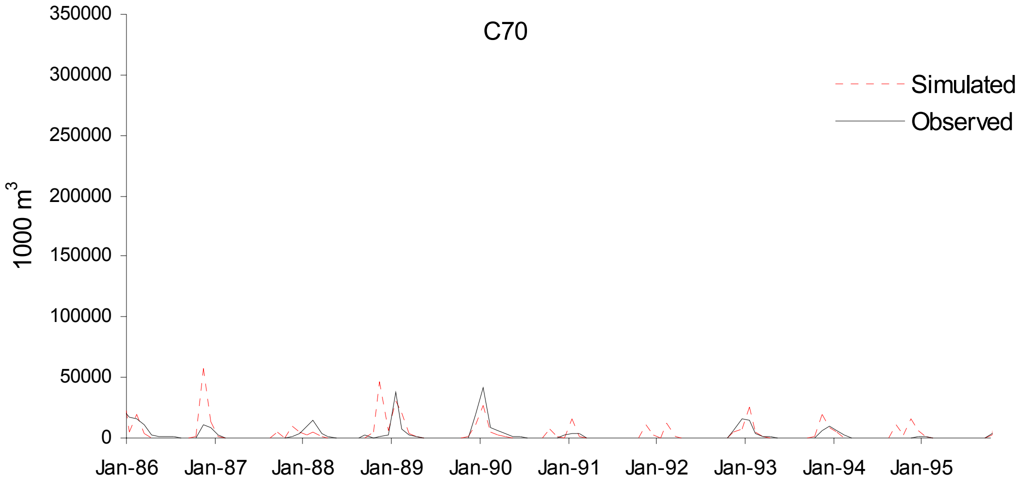


Fig. 7.10 Observed and simulated mean monthly runoff (1970-1985) at gauging weir a) C70 and b) C12.

Based on the outcome of the applied performance criteria the present setup of the ACRU model is accepted as being calibrated for the Mupfure catchment. No systematic errors were identified and the model generally performed satisfactorily. Further data collection is required, especially rainfall, for further physically based model improvements.

Model validation

Following a successful model calibration the model is validated on the subsequent ten year period from 1986 to 1996. The observed runoff during this period is significantly lower than during the calibration period, which is also reflected in the various performance parameters. Plots of observed versus simulated monthly runoff for the validation period are shown in Fig. 7.11.



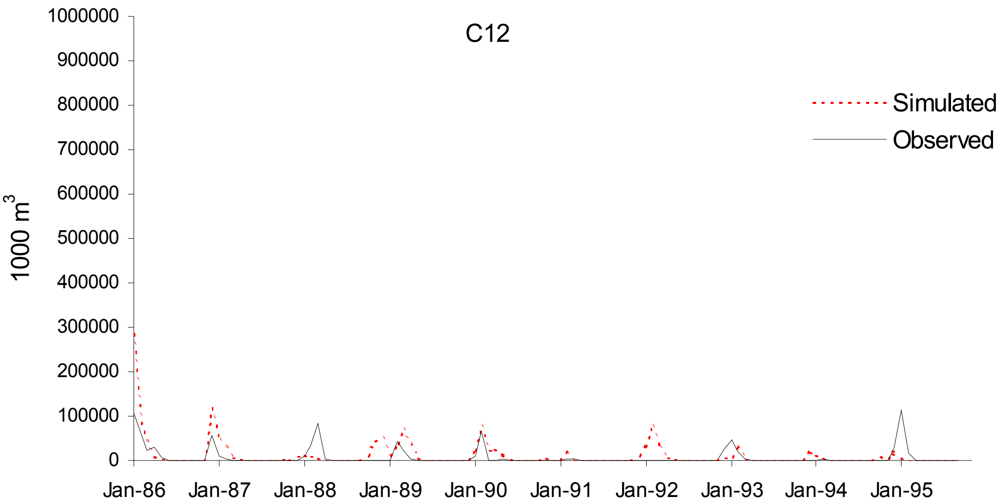


Fig. 7.11 Comparison of observed and simulated monthly runoff (1986-1996) at a) C70 and b) C12.

These plots reveal that despite the low runoff, the model is still able to fit the timing of the flow occurrence at both C70 and C12. Visual comparison of observed and simulated runoff magnitude does not signify critical deviations, but the relative difference due to low runoff might be significant compared to the calibration period. This observation is supported by the values of the numerical performance criteria, as shown in Table 7.10

Table 7.10 Numerical performance criteria for the validation period (1986-1996).

	D_v [%]	R^2
C70	20.51	0.56
C12	52.20	-0.74

The overall mass balance D_v and the coefficient of determination R^2 have lower values than obtained during model calibration. In fact, the results appear so poor that they do not fulfil the requirement by Schulze (1995) and Andersen et al. (2001) for good model performance. Especially, the results obtained at gauging weir C12 are poor. However, the runoff during the validation period is low, which makes the numerical performance criteria sensitive to differences between observed and simulated runoff. As noted before, gauging weir C12 is not well suited for measuring low flow. Furthermore, a number of external factors might influence runoff in a catchment during periods of droughts and low water availability such as illegal abstractions and increased groundwater abstraction. The operation of reservoirs also becomes an important factor for determining runoff in the river when availability is low. All of these factors may explain for some of the difference between results obtained for the calibration and validation period, respectively, as non of these factors have been included in the ACRU setup. As gauging weir C12 is situated at the outlet of the considered catchment and downstream of area with large-scale commercial agriculture and numerous impoundments, it is expected that the results obtained here are less favourable than for gauging weir C70, which measures less influenced streamflow.

Observed and simulated annual runoff for both C70 and C12 are compared in Fig. 7.12a and Fig. 7.12b. A line with unit slope and zero interception has been added to the plots together with the linear relationship estimated using least squares method.

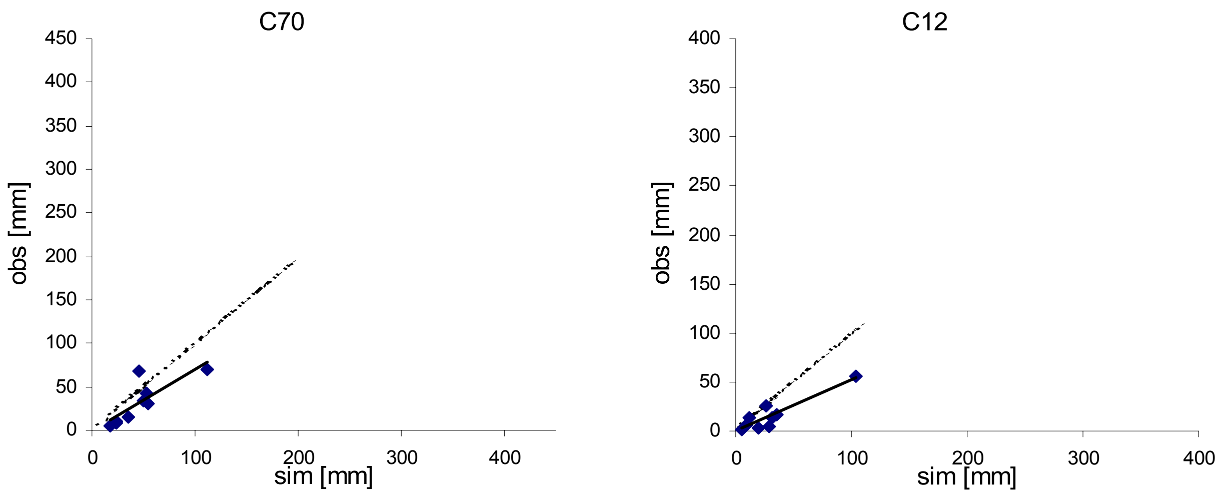


Fig. 7.12 Observed versus simulated annual runoff (1986-1996) a) at gauging weir a) C70 and b) C12.

Again it is found that runoff during the validation period is lower than during the calibration period. Also a tendency of simulating more runoff than observed can be seen at both C70 and C12. However, it should be noted that the estimated linear relationship tends to have zero interception. The test statistics for unit slope and zero interception are shown in Table 7.11.

Table 7.11 Statistics for comparison of simulated and observed annual runoff.

	Slope	Interception
C70	-1.54	-0.13
C12	-5.41	-0.07

Again, the poor results obtained at, especially gauging weir C12, might be due to factors not included in the model setup. Finally, the observed and simulated mean monthly runoff at both gauging weir C70 and C12 are compared in Fig. 7.12a and Fig 7.12b. According to Schulze (1995) a zero interception indicates that no systematic error or over or under-simulation is evident.

Finally, the simulated and observed mean monthly runoff are compared in Fig. 7.13a and Fig. 13b. At both gauging weirs the ACRU model simulated more runoff than observed during the rainy season from December to March, where as flow during the dry season April to November is well described at both weirs.

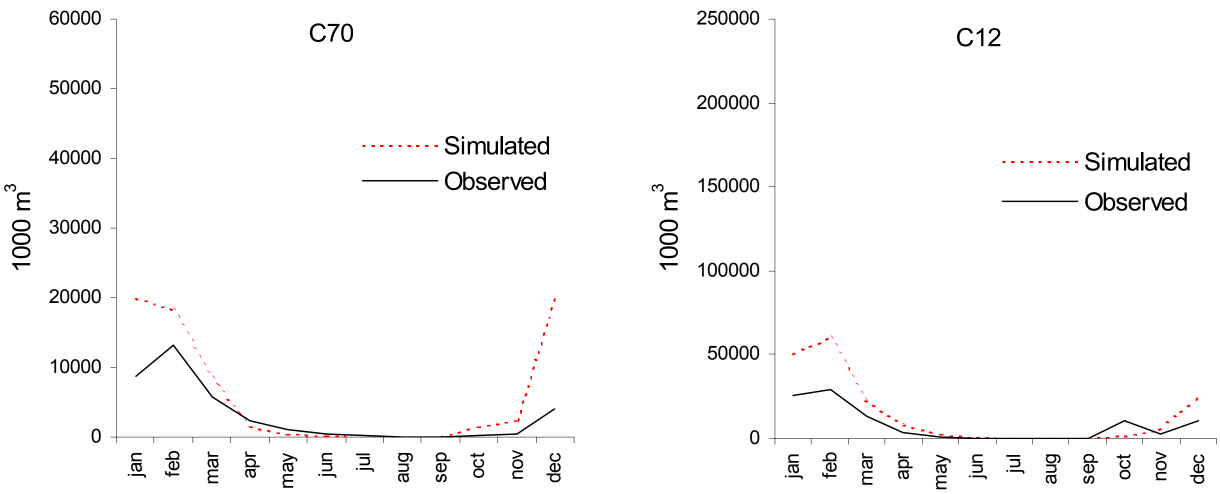


Fig. 7.13 Observed and simulated mean monthly runoff (1986-1995) at gauging weir a) C70 and b) C12.

A reduction in runoff during the rainy season in a period of low runoff might be influenced by increased storage for later irrigation, which is believed to increase during droughts. However, more research is needed before final conclusions can be drawn.

Discussion

Based on the results obtained during model calibration and validation the current setup of the ACRU model for the Mupfure catchment is accepted and applied in the succeeding sustainability assessment. The selected performance criteria showed that the model calibration was successful. The succeeding model validation was carried out in a period of low runoff, and the performance criteria indicated problems related to over-simulation of runoff during this period. However, a number of external factors might influence the observed runoff leading to further aggravation of the results. Further research into past management practise and quality control of data material are needed before more in-depth conclusions regarding model performance can be drawn.

7.2 MIKE BASIN SETUP OF THE MUPFURE WATER RESOURCES SYSTEM

Following the water resources assessment the MIKE BASIN model has been setup for the water resources system. The model setup was made taking into consideration the different water users as well as existing and planned water resources infrastructure. The considered part of the catchment was divided into eight sub-catchments based on aggregation of the 19 sub-catchments used in the setup of the ACRU model. The sub-catchments are denoted Mup1 – Mup8 and their geographical characteristics are defined in Table 7.12, with reference to Fig. 7.3.

Table 7.12 Runoff schemes used in the MIKE BASIN setup of the Mupfure catchment.

Sub-catchment		ACRU sub-catchments
Mup 1	Inflow to Makusekwa dam	1, 2, 3
Mup 2	Commercial farm land	5, 6
Mup 3	Communal area	4
Mup 4	Commercial farm land	7
Mup 5	Commercial farm land	9, 10
Mup 6	Mhondoro communal area	8, 11, 12
Mup 7	Commercial farm land	13, 14, 15, 16, 17
Mup 8	Commercial farm land	18

The MIKE BASIN setup consists of four irrigation schemes, one urban user, one industrial user, five reservoirs and eight runoff schemes as shown in Fig. 7.14. The level of information aggregation chosen for the model setup reflects the objective of the policy analysis. The aggregation of the agricultural sector into four irrigation schemes emphasises the difference between commercial and communal agricultural sectors. Furthermore, the conflicts between up- and down stream users regarding the demand for water can be included in the analysis. However, conflicts between farmers within the same sector and situated in the same part of the catchment cannot be analysed. With the level of detail of the available data material, and maintaining a focus primarily on the entire water resources system and the conflicts emphasised by users within the catchment (Kjeldsen et al., 1999b), it was decided that the current MIKE BASIN model setup is appropriate for the sustainability assessment.

The specifications for each water user, reservoir and runoff scheme are described in the following.

Agricultural water demand

The agricultural water users have been divided into four different water user-groups based on socio-geographic conditions. Hence two irrigation schemes representing small-scale communal agriculture and two schemes representing large scale commercial agriculture. The irrigation water demand for each scheme is calculated by summing up the monthly net irrigation requirements calculated by the irrigation module in the ACRU model as described in section 7.1. Table 7.13 shows from which ACRU catchments the irrigation demand for each of the four users is specified with reference to Fig. 7.3 and Fig. 7.14.

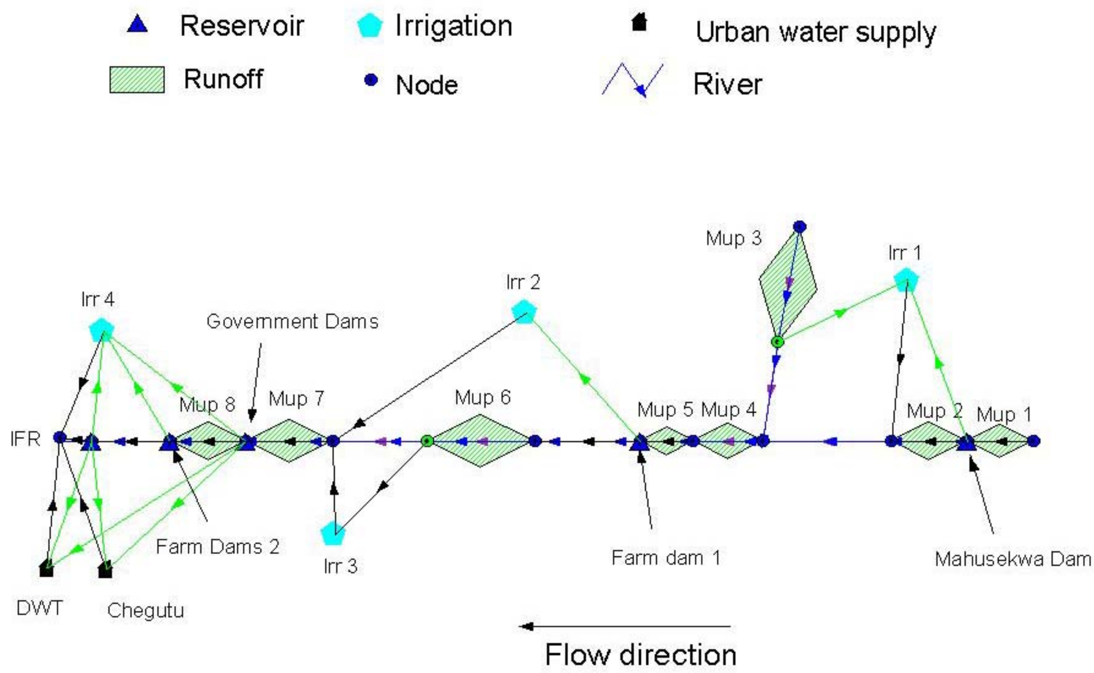


Fig. 7.14 Schematic setup of the MIKE BASIN model for the Mupfure water resources system.

Table 7.13 Irrigation schemes used in MIKE BASIN.

	Type of agriculture	ACRU sub-catchments
Irr 1	Small-scale communal	4
Irr 2	Large-scale commercial	5, 7, 9
Irr 3	Small-scale communal	8, 10
Irr 4	Large-scale commercial	13, 14, 15, 16, 17, 18

The average monthly demand for each of the four irrigation schemes is shown in Table 7.14. It is obvious that the bulk of the demand originates from the commercial agriculture.

Losses due to inefficient conveyance of water from source to field are not included in the demand estimates obtained from ACRU and have to be specified in the MIKE BASIN setup. Xie et al. (1993) collected estimates of irrigation efficiency from a variety of studies world-wide. They defined the term irrigation efficiency in terms of conveyance E_c, distribution E_d and field efficiency E_f. The field efficiency is included in the net requirements in Table 7.14. Conveyance efficiency is estimated as the ratio between water diverted from a source and the amount of water received by the user. Distribution efficiency is the ratio between water received by the user and the amount of water applied to the farm. Irrigation network efficiency E_n is defined as a combination of both conveyance and distribution efficiency, i.e. $E_n = E_c E_d$, and can be specified in the MIKE BASIN model. Based on the figures presented by Xie et al. (1993) the irrigation network efficiency was assumed to be 68% in the communal areas, which equals the category “Developing Countries” and 80% in the commercial agricultural sector. No return flow from the irrigation schemes back to the system was assumed.

Table 7.14 Agricultural water demand [m^3/s].

	Irr 1	Irr 2	Irr 3	Irr 4
Jan	0.05	0.56	0.12	0.71
Feb	0.05	0.81	0.19	1.34
Mar	0.11	1.23	0.27	1.71
Apr	0.12	1.45	0.31	2.74
May	0.14	1.83	0.69	3.32
Jun	0.15	1.77	0.49	3.4
Jul	0.15	1.80	0.64	3.44
Aug	0.13	1.78	0.54	3.04
Sep	0.16	1.81	0.52	3.36
Oct	0.14	1.69	0.58	2.90
Nov	0.10	1.26	0.29	2.15
Dec	0.06	0.67	0.15	1.29
*TAD [10^6 m^3]	3.6	43.8	12.6	83.0

*TAD = total annual demand.

Urban water demand

The only major urban centre within the catchment is the town of Chegutu with a population of approximately 35 000 (CSO, 1997). Abstracting water from the neighbouring Mupfure River covers the town’s water demand. Water is abstracted from the Railroad weir and via a treatment plant distributed to water users. Additional water is stored in the upstream government reservoirs and is released down to the Railroad weir when needed. Records of monthly water abstraction from the period 1979 to 1991 were collected from the office of the Town Engineer in Chegutu. After this period data were either missing or deemed too unreliable or erroneous for further analysis. The annual abstraction is shown in Fig. 7.15 and it is evident that the major droughts in 1982/83 and 1986 reduced the volume of abstracted water.

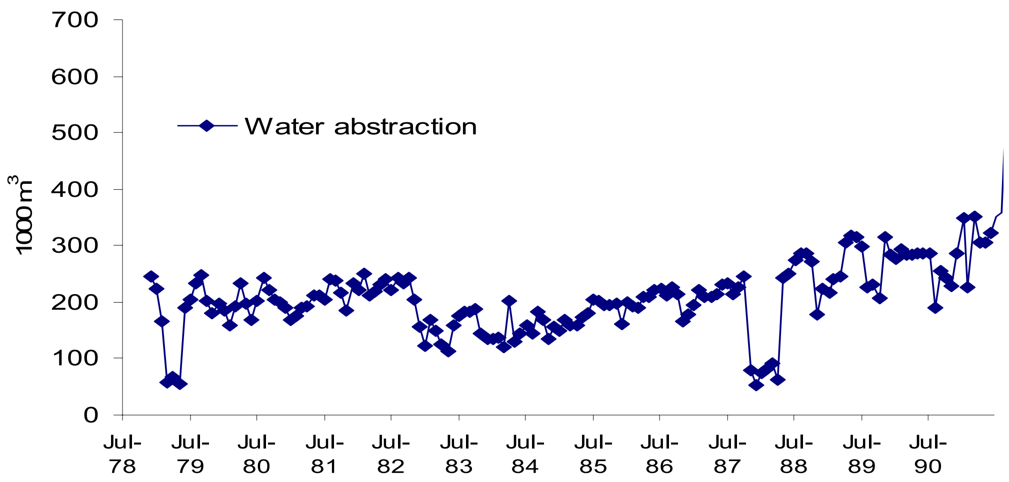


Fig. 7.15 Observed record of monthly abstraction from Chegutu.

Water demand for the period ranging from 1970 and up to the beginning of the observed data in January 1979 was assumed constant at 200 000 m^3/month. From the end of the observed data in September 1991 and up to the end of the simulation period in December 1996, the water demand was predicted using a linear model as further described in section 7.5.

Industrial Water Demand

The David Withfield textile (DWT) factory is situated within Chegutu town. According to separate water use data from the town engineer in Chegutu the factory has a constant monthly water demand of 70 000 m^3. This water demand is assumed constant during the calibration period.

Water Supply infrastructure

The existing reservoirs in the catchment can be divided into private and governmental reservoirs. Private dams are primarily used for storing water for irrigation of large-scale commercial agriculture during the dry season. Water in the government's reservoirs are used for supplying urban areas, mainly Chegutu, and to some extent for irrigation. In the MIKE BASIN setup the farm dam volumes listed in Table 7.4 have been aggregated into two representative reservoirs. The first reservoir, farm dams 1, represents the impoundment in the upper part of the catchment and is situated at the outlet of sub-catchment Mup 5. The second reservoir, farm dams 2, represents the reservoirs in the lower part of the catchment and is situated at the outlet of sub-catchment Mup 8. The HVA curves for each reservoir are constructed using the procedure outlined in Appendix B. The government's reservoirs have been modelled as three individual reservoirs. The most upstream reservoir is the Mahusekwa dam situated at the outlet of sub-catchment Mup 1. A HVA curve for this reservoir was available from MLWR (1999). The second reservoir consists of all the reservoirs supplying water to Chegutu, i.e. Upper and Lower

Seignury, Maynard, Poole, Clifton and Twyford weirs. This reservoir is situated at the outlet of sub-catchment Mup 7. Finally, a reservoir representing the Railway weir is located after farm dams 2, i.e. the last reservoir before the water leaves the model setup. The Railway weir is model separately because of its importance in the allocation of water between Chegutu and the surrounding agricultural users (irrigation scheme no. 4).

Because of the principle of prior rights embedded in the old Water Act (Kjeldsen et al., 1999b) a number of complications regarding the system operation had to be taken into consideration in the model setup. First, water is sold from the government's reservoirs to both urban, industry and agricultural users in listed order of priority in times of shortage. However, at Railway weir the water rights owned by the agricultural sector have priority to the government's water rights. This system of user priorities has been build into the model setup and the actual policy will be determined during calibration of the model. Secondly, the water rights owned by the downstream agricultural users (irrigation scheme no. 4) generally have prior right to the agricultural users further upstream, represented by irrigation scheme 2. Therefore, the upstream users have to release a certain amount of water to downstream users before they can start abstracting water. The minimum required downstream release is adjusted as a part of the model calibration.

Runoff

Time series of monthly runoff for each sub-catchment was obtained by aggregating time series of naturalised runoff from the ACRU sub-catchments listed in Table 7.15. Naturalised runoff implies that simulated runoff from the ACRU model with disabled irrigation and reservoirs has been used.

Table 7.15 Runoff in the Mupfure catchment.

Sub-catchment	Catchment area [km^2]	Mean annual runoff [mm]	Mean annual runoff [10^6 m^3]	C_v of annual runoff
Mup 1	137	115	15.8	1.07
Mup 2	458	96	44.0	1.11
Mup 3	382	70	26.7	1.31
Mup 4	287	94	27.0	1.26
Mup 5	1274	71	90.5	1.31
Mup 6	904	62	56.0	1.67
Mup 7	1705	63	107.4	1.69
Mup 8	274	79	21.6	1.67

A large variation in unit runoff from the different parts of the catchment is evident in Table 7.15. This could be influenced by the low data availability. However, a more in-depth analysis would require additional data, which at present do not exist.

Calibration of the MIKE BASIN model

The MIKE BASIN model was set-up for the Mupfure catchment, as shown in Fig. 7.14 for the 26 year period ranging from 1 January 1970 to 31 December 1996, and based on the water resources system data described above. Next, certain parameters in the model setup were adjusted to obtain the best agreement between observed and simulated data. In the following this adjustment will be referred to as a model calibration, even though it is not an actual model calibration/validation as often used in connection with hydrological models. The adjustment procedure is aimed at identifying a general model structure able to reproduce the observed time series of various water resources system variables with a satisfactory degree of agreement, and should represent a reasonable policy structure of the entire water resources system. No validation of the model setup was carried out as system operation is determined mainly by human influences and can vary according to external factors not represented in the model. Furthermore, the observed time series covers different time periods making identification of calibration and validations period difficult. The following time series of water resources system data were obtained:

1. Monthly runoff at gauging weir C70 and C12 (1970-1996).
2. Monthly water abstraction for Chegutu town (1978-1991).
3. Monthly storage of water in the government reservoirs (1991-1996).

The model setup was calibrated using trial and error and by adjusting minimum downstream release from farm dam 1 and system operation for supply of water to Chegutu and irrigation scheme no. 4. Introducing a minimum downstream release from the upstream farm dams corresponds well with the management of water rights within the catchment. The 1976 Water Act is based on the principle of prior rights, i.e. water rights issued with a prior date should be fulfilled before more recently issued water rights (Kjeldsen et al., 1999b). The oldest water rights in the Mupfure catchment are generally situated in the lower part of the catchment, requiring downstream release of water from users in the upper part. Next, the operation of the reservoir system controlling water allocation to Chegutu, the textile factory and irrigation scheme no. 4 were adjusted. The results of the model adjustment in terms of comparisons between simulated and observed time series are described below.

The observed and simulated time series of monthly runoff were compared according to the numerical model performance criteria used during calibration and validation of the ACRU model, D_v and R^2, presented in section 7.1.

Table 7.16 Goodness of fit statistics for observed and simulated monthly runoff in the MIKE BASIN model.

	C70		C12	
	D_v [%]	R^2	D_v [%]	R^2
1970-1985 (cal)	-7.1	0.79	-12.9	0.82
1986-1996 (val)	83.3	-0.56	-3.54	-0.06
1970-1996	4.8	0.73	-11.9	0.84

The numerical performance criteria were calculated for the entire water resources period (1970-1996) and for the calibration (1970-1985) and validation (1986-1996) periods defined in section 7.1 in connection with the ACRU model. The results showed in Table 7.16 compare well to the corresponding estimates obtained during calibration and validation of the ACRU model.

The observed abstraction of water from the Mupfure River for Chegutu town has been specified in the MIKE BASIN model as the water demand. Therefore, periods where simulated abstraction cannot fulfil the specified demand should be minimised. It should be noted that only periods of too little water can be identified. In periods of ample water availability, only a volume of water corresponding to the specified water demand will be abstracted. A comparison between specified and simulated water abstraction is shown in Fig. 7.16, and it can be seen that enough water is available in the time period covered by the observed data, except the last month which corresponds with the onset of the severe 1991/92 drought.

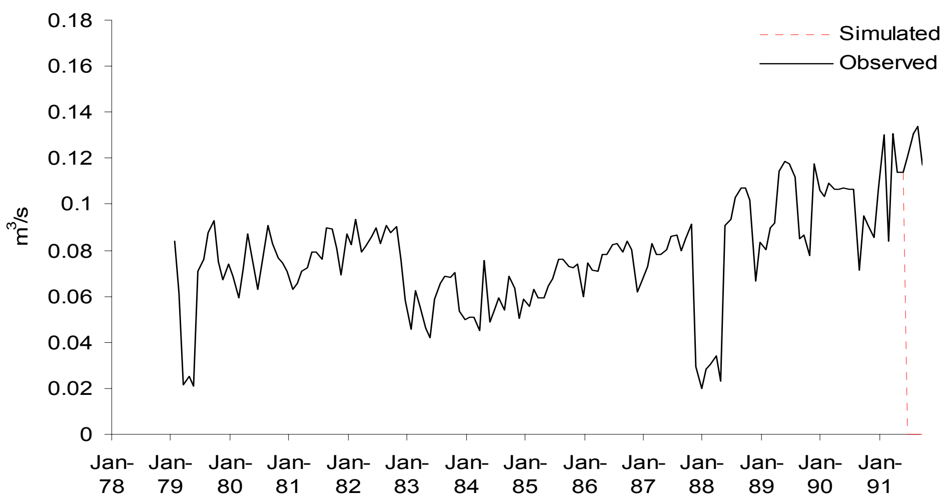


Fig. 7.16 Observed and simulated water abstraction by the Chegutu town.

Finally, the simulated storage volume in the government's reservoirs is compared to the corresponding observed data. The observed data consist of the storage volume for each of the last seven dams listed in Table 7.5 added together. The data were available on a weekly basis, however only measurements closest to the last day of each months were used. In the model setup the government's dams have been conceptualised by a single reservoir situated at the outlet of sub-catchment Mup 6. Because of this model simplification less emphasis is given to this aspect of the model calibration. The comparison between observed and simulated storage volume is shown in Fig. 7.17.

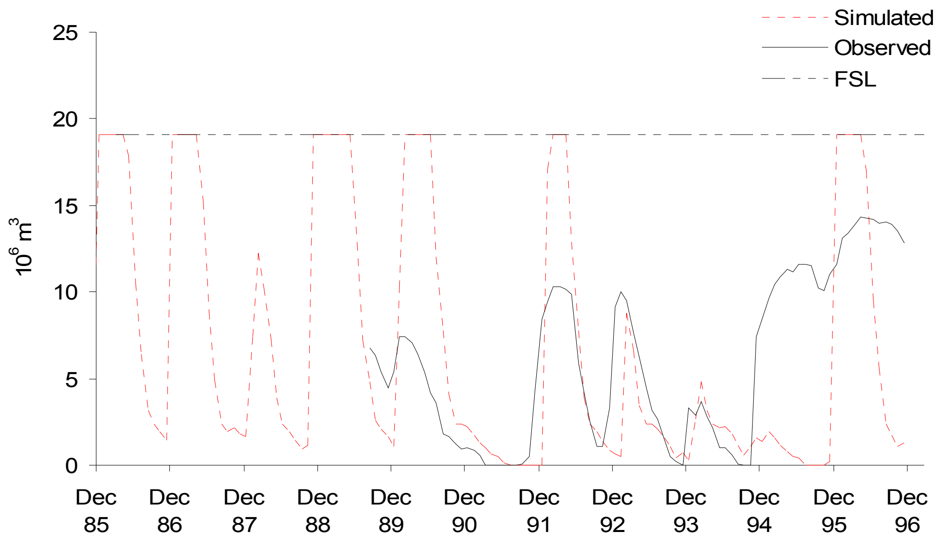


Fig. 7.17 Observed and simulated storage in the government's reservoirs.

Based on the reasonably successful comparison between observed and simulated water resources system variables it is concluded that the model setup is a reasonable description of the Mupfure water resources system and will be used in the following to analyse a number of different policy scenarios with respect to the degree of sustainability.

7.3 MULTIVARIATE STOCHASTOC STREAMFLOW MODEL

The identification, estimation and generation of the multivariate CARMA(p,q) time series model for the eight runoff schemes in the Mupfure catchment are carried out as outlined in section 5.2. However, the occurrence of years with no runoff (zero runoff) at four of the eight runoff schemes necessitated the development of an adjustment procedure to the CARMA(p,q) model to account for the frequency of occurrence of zero runoff at various sites. The modified procedure is described below

Adjustment procedure for generation of annual runoff

The stochastic generation of annual streamflow at multiple sites is carried out as the product of two separate stochastic processes as

$$q(i,t) = v(i,t)z(i,t) \tag{7.4}$$

where $z(i,t)$ is the non-zero annual runoff at the i-th site in time t and $v(i,t)$ is a binary process defined as

$$v(i,t) = \begin{cases} 0, \text{dry year at site } i \\ 1, \text{wet year at site } i \end{cases} \tag{7.5}$$

It is assumed that no serial correlation is present in the observed time series of annual runoff, which is commonly encountered phenomenon in arid and semi-arid environments (McMahon, 1979), and also supported by the data material used in this study. First, the observed non-zero annual runoff observed at each site are transformed into samples described by the normal distribution using an appropriate transformation T, which is either the Box-Cox transformation or the 3-parameter log-normal distribution as described in Appendix A. The transformed non-zero annual runoff at the i-th site $x(i)$ is given as

$$T(z(i) \mid \alpha_i) \rightarrow x(i) \in N\left(\mu, \sigma^2\right) \tag{7.6}$$

where α_i are the transformation parameters at the i-th site. Different methods for transformations can be used at different sites. The spatial and temporal dependencies of $x(i,t)$ are described by the CARMA(p,q) model

$$\left(\mathbf{x}(t) - \boldsymbol{\mu}\right) = \sum_{i=1}^{p} \varphi(i)\left(\mathbf{x}(t-1) - \boldsymbol{\mu}\right) + \boldsymbol{\varepsilon}(t) + \sum_{j=1}^{q} \boldsymbol{\theta}(j)\left(\boldsymbol{\varepsilon}(t-j)\right) \tag{7.7}$$

where $\mathbf{x}$(t) is the vector of transformed non-zero annual runoff at time t with mean value $\boldsymbol{\mu}$. The residuals $\boldsymbol{\varepsilon}$ have zero mean and a variance-covariance matrix $\boldsymbol{\Delta}$. As no significant temporal correlation is present the CARMA(p,q) model reduces to a multivariate normal distribution. The identification and estimation of a CARMA(p,q) model and the subsequent generation of series of stochastic runoff are similar to the procedure outlined in section 5.2. However, due to the consideration of only non-zero events the residual variance-covariance matrix $\boldsymbol{\Delta}$ was estimated using a modified procedure adopted from Madsen (1995) and described in Appendix D. Time series of non-zero runoff $x(i,t)$ are generated at each time step using the CARMA(p,q) model. The realisations of the binary $v(i,t)$ process are generated based on the probability of observing zero flow at each site. First, a discrete stochastic variable NZ is introduced, describing the number of sites with zero runoff observed in any given year. An example of an empirical cdf of NZ estimated from the eight time series of annual runoff used in the analysis of the Mupfure catchment is shown in Fig. 7.18.

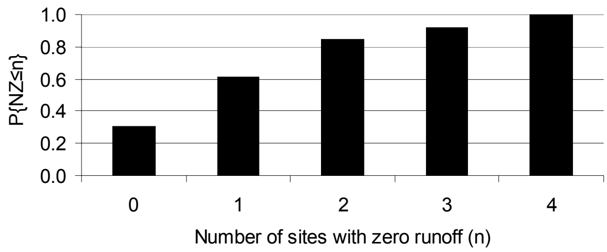


Fig. 7.18 Example of empirical cdf describing the number of zero runoff sites to be generated within a year

For each time step a uniform $U(0,1)$ random variable is generated and the number of sites with zero runoff (NZ) is found by inverting the cdf described in Fig. 7.18. Next, the $NZ = n$ sites with zero runoff need to be allocated to actual sites. This allocation process is based on the probability of occurrence of a year with zero flow at each specific site conditional on the total number of sites with zero runoff. The condition has been introduced as some sites only experience zero-runoff in very dry years, where other sites more frequently experience years of zero runoff. The probability of occurrence of zero flow at site i conditional on the total number of sites with zero runoff NZ within a year is given as

$$p(i,n) = P\{q(i,t) = 0 \mid NZ = n\} \tag{7.8}$$

The algorithm for distributing the NZ sites with zero runoff between all sites is outlined below and is executed for each year in the generation procedure:

1. Generate the number of sites with zero runoff within a year $NZ = n$.

2. If $NZ = 0$: $v(i,t) = 1$ for $i = 1, \ldots, M$. Go to 1.

3. If $NZ = n > 0$: for each site calculate the probability $p^*(i,1)$ of allocating this first zero-flow to the i-th site as

$$p^*(i,1) = \frac{p(i,n)}{\sum_{k=1}^{M} p(k,n)} \tag{7.9}$$

The estimated $p^*(i,1)$ will be between zero and one and summing up the probabilities over all M sites will equal one as illustrated in Fig. 7.19. The allocation of the first zero flow is determined by generating a uniform $U(0,1)$ realization and finding the first site where the cumulative values of $p^*(i,1)$ is larger or equal to the value of this realisation as illustrated in Fig. 7.19. In Fig. 7.19 the allocation probabilities p^* are estimated by summing up in ascending order, i.e. site 1, 2, …, M, but other orders of summation could have been used, which may led to a different allocation of zero-flows within each individual generated time series. However, the extent of the allocation probabilities defined in Eq. (7.9) is independent of the order of summation, and when the selection of the site is based on realisation of a uniform $U(0,1)$ distribution, on average, events of zero-runoff will be allocated according to the specified distribution.

If $NZ = 1$ then go to 1. If $NZ > 1$ then repeat the procedure outlined above, but with the site already allocated zero runoff removed from the calculations. Hence, the probability of allocating the second zero flow to the i-th site $p^*(i,2)$ is calculated as

$$p^*(i,2) = \frac{p(i,n)}{\sum_{k=1}^{M-1} p(k,n)} \tag{7.10}$$

The extent of the allocation probabilities $p^*(i,2)$ depends on, which site has been removed from the calculations. However, it is only the denominator in Eq. (7.10), which is altered and the ratio between the allocation probabilities is kept constant. Therefore, as before, the zero-flow events will be allocated according to the specified distribution.

4. Continue this procedure until all $NZ = n$ zero runoff sites have been allocated.

5. Go to 1.

Following the procedure outlined above, series of annual runoff with a frequency of occurrence of zero runoff equal to the observed frequency will be generated. By adopting a stochastic procedure for distribution of zero-flows it is possible to obtain combinations not included in the historical data, but sites where the historical records do not contain zero-flows events will not be allocated zero-flows by this method. However, if the $v(i,t)$ and $z(i,t)$ processes are generated independently of each other, the result will be an unrealistic occurrence of sites with zero runoff in years otherwise characterised by high non-zero runoff at sites where no zero runoff event is specified. Therefore, the probability of generating NZ sites with zero runoff was considered separately for wet and dry years.

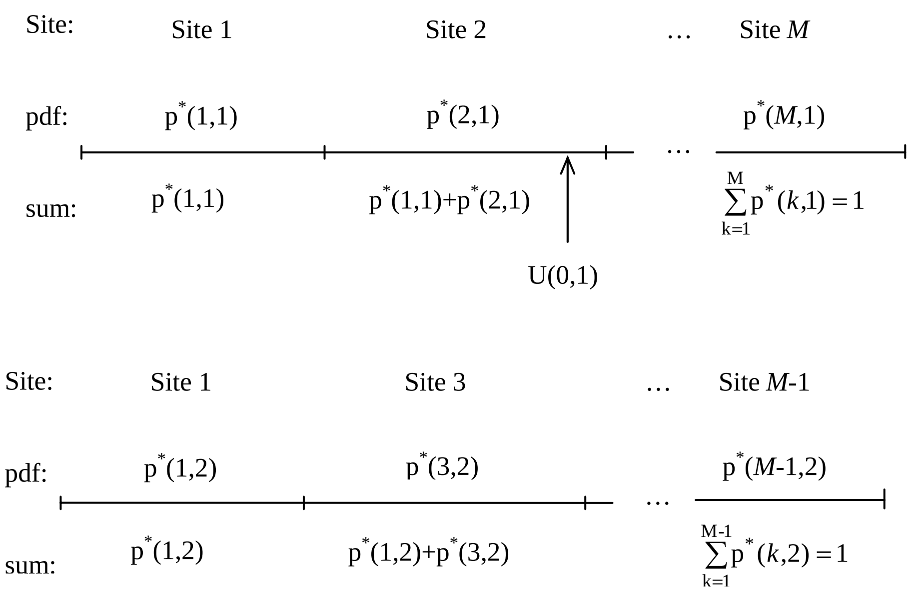


Fig 7.19 Procedure for allocating $NZ = n$ zero-flow events to n different sites within a year. A $U(0,1)$ distributed random variable is generated and the corresponding site is chosen based on the sum of allocation probabilities $p^*(i,1)$ (in this case Site 2 is chosen). For the chosen site (Site 2) the $v(i,t)$ process is set equal to zero. Next, the site is removed from the procedure and a new set of allocation probabilities $p^*(i,2)$ calculated through Eq. (7.10). This procedure continues until all $NZ = n$ sites are allocated.

To distinguish between the two instances it was considered whether the sum of non-zero runoff from all sites within a year is above or below a specified threshold level Q, and a probability distribution estimated for each of the two instances.

$$\begin{aligned} &P\left\{NZ = n \mid \sum_{i=1}^{M} z(i,t) > Q\right\} \\ &P\left\{NZ = n \mid \sum_{i=1}^{M} z(i,t) \le Q\right\} \end{aligned} \tag{7.11}$$

where the unconditional probability is given as

$$\begin{aligned} P\{NZ = n\} = {} & P\left\{NZ = n \mid \sum_{i=1}^{M} z(i,t) > Q\right\} P\left\{\sum_{i=1}^{M} z(i,t) > Q\right\} + \\ & P\left\{NZ = n \mid \sum_{i=1}^{M} z(i,t) \le Q\right\} P\left\{\sum_{i=1}^{M} z(i,t) \le Q\right\} \end{aligned} \tag{7.12}$$

In theory there is no limit to the number of probability distributions that can be specified, but in practice the available data material will be a limiting factor. The method, however, has a build-in flaw. When forcing all zero events to occur in years with low runoff the average simulated runoff will be biased upwards compared to the corresponding observed runoff, i.e. the method simulates too much water. To correct for this effect a recursive adjustment of the mean values of the transformed non-zero runoff was applied to minimise the difference between observed and simulated mean annual runoff, i.e.

$$\min_{\boldsymbol{\mu}} \left\{ \sum_{i=1}^{M} \left(q^{(o)}(i) - q^{(s)}(i) \right)^2 \right\} \tag{7.13}$$

where $q^{(o)}(i)$ is mean value of observed annual runoff at the *i*-th site and $q^{(s)}(i)$ is the mean value of the corresponding simulated mean annual runoff at the *i*-th site. The minimisation was performed using the downhill simplex method described by Nelder & Mead (1965) and Press et al. (1992). Subsequently to having estimated the mean values it was checked if approximately 50% of the years still fall under the specified threshold level. Attempts to improve the method by simultaneously adjusting the variance-covariance matrix of the non-transformed time series of annual runoff by assuming a fixed cross-correlation structure did not improve the results. Furthermore, an attempt to expand the objective function in Eq. (7.13) to also consider the deviation from observed standard deviation of annual runoff did not improve the method either.

Binary process

Two binary $v(i,t)$ processes were considered, one for wet years and one for dry years. The threshold level Q separating the two instances was specified as 1.1 times the median in the sample of sum of observed annual runoff from all sites. Hence, in a runoff generation context a wet and a dry year are defined as

$$\begin{aligned} Wet &: \sum_{i=1}^{M} z(i,t) > Q \\ Dry &: \sum_{i=1}^{M} z(i,t) \leq Q \end{aligned} \tag{7.14}$$

The specified cdfs for wet and dry years are shown in Table 7.17.

Table 7.17 Specified distribution for number of zero runoff events to generate each year.

	Wet		Dry	
NZ = n	pdf	cdf	pdf	cdf
0	1	1	0.31	0.31
1	0	1	0.31	0.62
2	0	1	0.23	0.85
3	0	1	0.08	0.92
4	0	1	0.08	1.00

Years with zero runoff only occurred in dry years and in only four out of the eight runoff schemes. The matrix containing the allocation probabilities $P\{q(i,t) = 0 \mid NZ = n\}$ as defined in Eq. (7.11) is shown in Table 7.18.

Table 7.18 Estimated values of $P\{q(i,t) = 0 \mid NZ = n\}$.

NZ = n	Mup 1	Mup 2	Mup 3	Mup 4	Mup 5	Mup 6	Mup 7	Mup 8
1	0	0	0	0	0	0.75	0	0.25
2	0	0	0	0.33	0.33	0.33	0.33	0.67
3	0	0	0	0	1	0	1	1
4	0	0	1	1	0	1	1	1

Due to the existence of only one year with NZ = 4 and NZ = 3 the allocation probabilities equal one at the sites where zero runoff occurred in these years.

Univariate ARMA(p,q) models

At each of the eight runoff schemes an ARMA(p,q) model was estimated to the time series of transformed non-zero observed runoff. Using the unconditional maximum likelihood method for parameter estimation and the Aikaike's information criteria for model selection as described in Appendix A it was found that the ARMA(0,0) model was preferable for all sites, thus the CARMA(p,q) model reduces to a multivariate normal distribution with no consideration of temporal persistence. The transformation parameters for each of the eight runoff schemes are shown in Table 7.19.

Table 7.19 Transformation parameters for each site.

	Box-Cox [λ]	LN3 [ξ]		Box-Cox [λ]	LN3 [ξ]
Mup 1	0.1914		Mup 5	0.2710	
Mup 2	0.2117		Mup 6	0.1338	
Mup 3	0.1908		Mup 7	0.1705	
Mup 4		-4.04	Mup8	0.1772	

Correlation structure

The spatial correlation structure of the CARMA(0,0) model for the non-zero events is specified through the cross-correlation of the time series of $\varepsilon(i,t)$, the residual cross correlation coefficient RCCC, as defined by Eq. (5.11) – Eq. (5.12). Estimation of the variance-covariance matrix Δ is based on a two step procedure. First the time series of the transformed time series of annual runoff from each site is pre-whitened through Eq. (5.10) to obtain the time series of normally distributed residuals $\varepsilon(i,t)$ with mean value of zero. Next, the RCCC between time series $\varepsilon(i,t)$ and $\varepsilon(j,t)$ at lag l, $\rho(i,j,l)$ is estimated using the modified estimation procedure for time series with missing observations outlined in Appendix D. Fig. 7.20 shows the RCCC for all possible 28 combinations of i and j for the eight runoff sites and for lag-time l = -2, 1, 0, 1, 2. The 95% confidence interval around $\rho\ (i,j,l)$ = 0 is given as $\pm 2n^{-1/2}$ (Hipel & McLeod, 1994) with n varying from site to site. In this case a general figure of n = 26 has been adopted. As evident from Fig 7.20 the cross correlation is significantly different from zero at lag-time l = 0. For the remaining lag-times, at least 95% of the estimated values of cross correlation fall inside the 95% confidence interval. This corresponds well with the assumption of no temporal persistence which reduced the CARMA(p,q) model to a multivariate normal distribution.

Fig. 7.20 RCCC for different lag times l. Only RCCC for l = 0 is significantly different from zero.

Model verification

Model verification compares the mean annual runoff (MAR), the standard deviation of annual runoff (SAR), and the percentage of zero runoff (pzf) of the generated time series with the similar statistics estimated from the historical time series of annual runoff from each site. From Fig. 7.21a it can be seen that each of the historical estimates of MAR are close to the corresponding median of the N = 5000 generated values of MAR. For all time series the historical estimates of MAR is between the 0.25 and 0.75 quantiles of the generated MAR sample. The historical estimate of SAR is compared to the corresponding generated values on Fig. 7.21b.

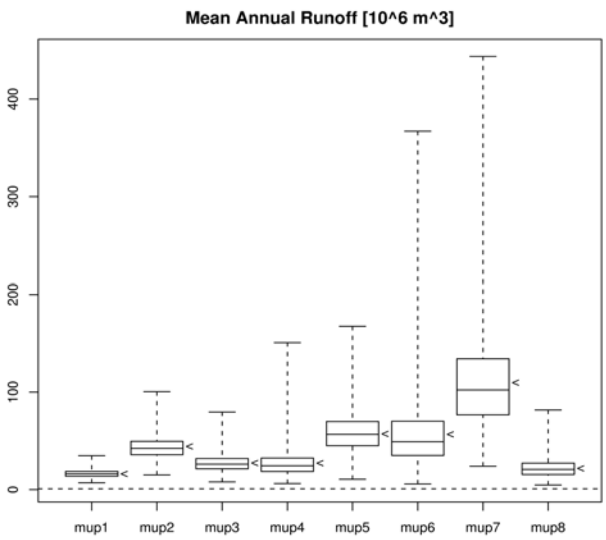


Fig 7.21a Comparison between simulated and observed MAR at each site [10^6 m^3].

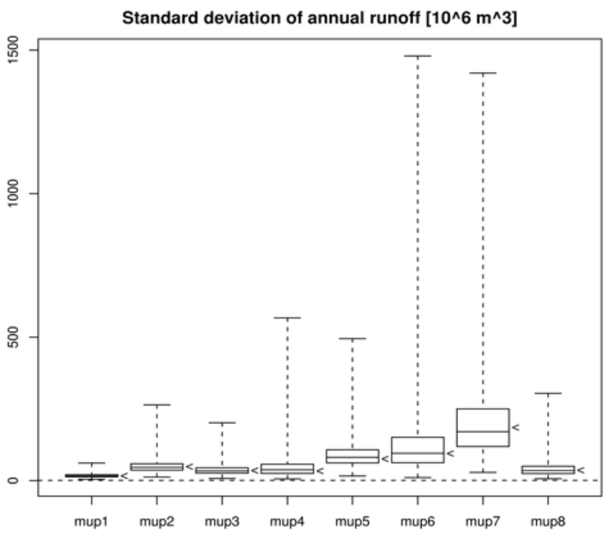


Fig 7.21b Comparison between observed and simulated SAR at each site [10^6 m^3].

As for MAR the generated estimates of SAR generally corresponds well with the historical estimates, which falls within the 0.25 to 0.75 quantile interval of the generated values. A comparison between observed and generated estimates of pzf is shown in Fig. 7.22.

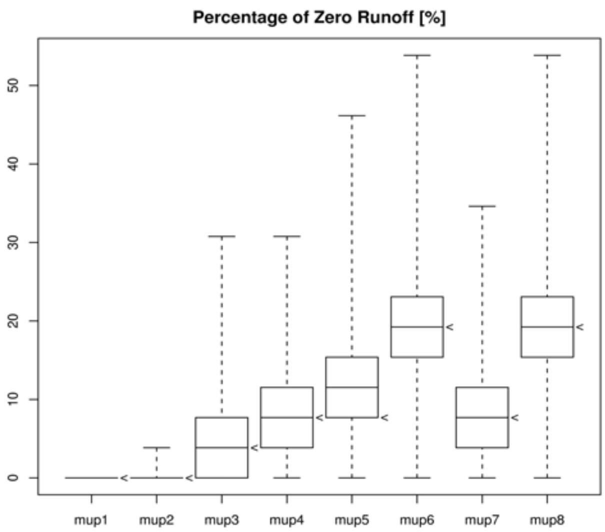


Fig. 7.22 Comparison between simulated and observed percentage of zero flow of annual runoff.

For most runoff schemes the specified and generated pzf correspond well. However, for Mup 5 the specified model generates time series with a higher frequency than specified. This is due to the effect described in section 5.2 and created by the transformation of a stochastic variable with a lower bound (Q = 0, i.e. zero flow) into an unbounded normally distributed stochastic variable. The transformation is described in Eq. (5.20). The CARMA(0,0) (a multivariate normal distribution) is used to model transformed non-zero part of the model as specified in Eq. (7.4). When a realisation from the CARMA(0,0) model falls below the threshold level defined by transformation of the lower bound (Q = 0), then a year with zero flow is assumed. Hence, the non-zero part of the model can also produce years with zero runoff and thereby increase the frequency of zero runoff in the generated time series. The same effect can be seen at Mup 2. Generally, the model verification shows that the stochastic model reproduces the specified historical statistics and therefore can be considered correctly implemented.

Model validation

Validation of the multisite extension of the method of fragments includes following aspects of model performance: ability to generate mean monthly runoff (MMR) and standard deviation of monthly runoff (SMR), the correlation between successive months at the same site, the ability to reproduce maximum observed deficit volume at each site, the lag-zero cross correlation between annual runoff at different sites and, finally, the lag-zero correlation between droughts at different sites through Eq. (5.23).

Mean monthly runoff The variation of generated MMR is very different for the wet (November to April) and the dry (May to October) seasons. A large variation is observed in the wet season and almost no variation in the dry season. However, the historical MMR and the corresponding *5-number summary*, as explained in section 5.2, of the generated values for each of the 8 runoff schemes are shown in Fig. 7.23. During both seasons the MMR is well captured by the method of fragments. In only few instances do the observed MMR fall outside the interval defined by the 0.25 - 0.75 quantile interval of the generated MMR.

Standard deviation of monthly runoff Considering the standard deviation of monthly runoff (SMR) a pattern similar to the results for MMR was observed as shown in Fig. 7.24. Again, a marked difference between the wet and the dry season is observed, with large variation during the wet season and no variation during the dry season. For both the wet and the dry season a generally good correspondence between observed and generated SMR is observed at all eight sites. The few sites and months where observed SMR is outside the 0.25 - 0.75 quantile interval of generated SMR are almost the same as for MMR.

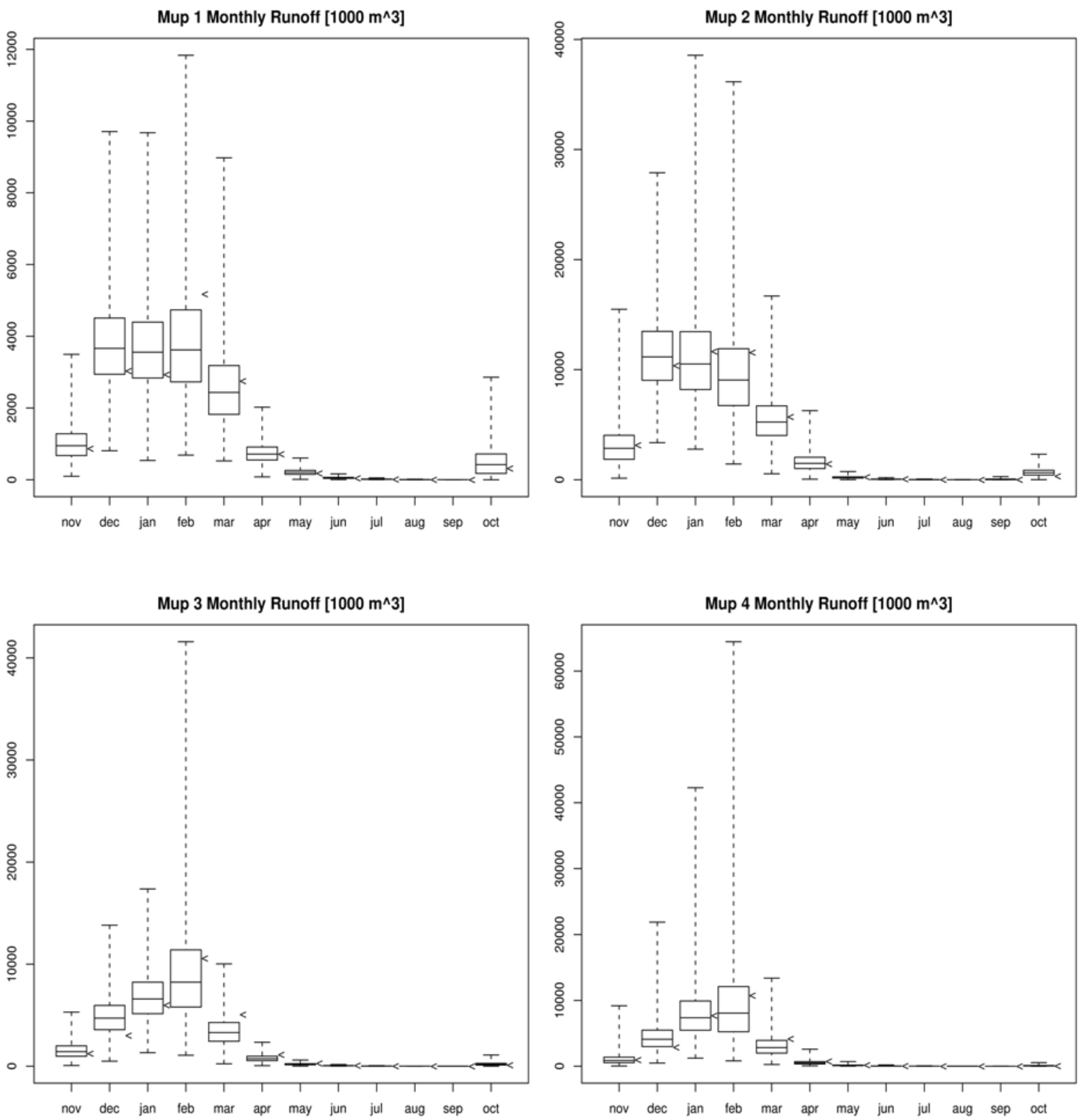


Fig. 7.23 Observed and generated mean monthly runoff at each site. Estimates obtained from historical time series indicated with “<”.

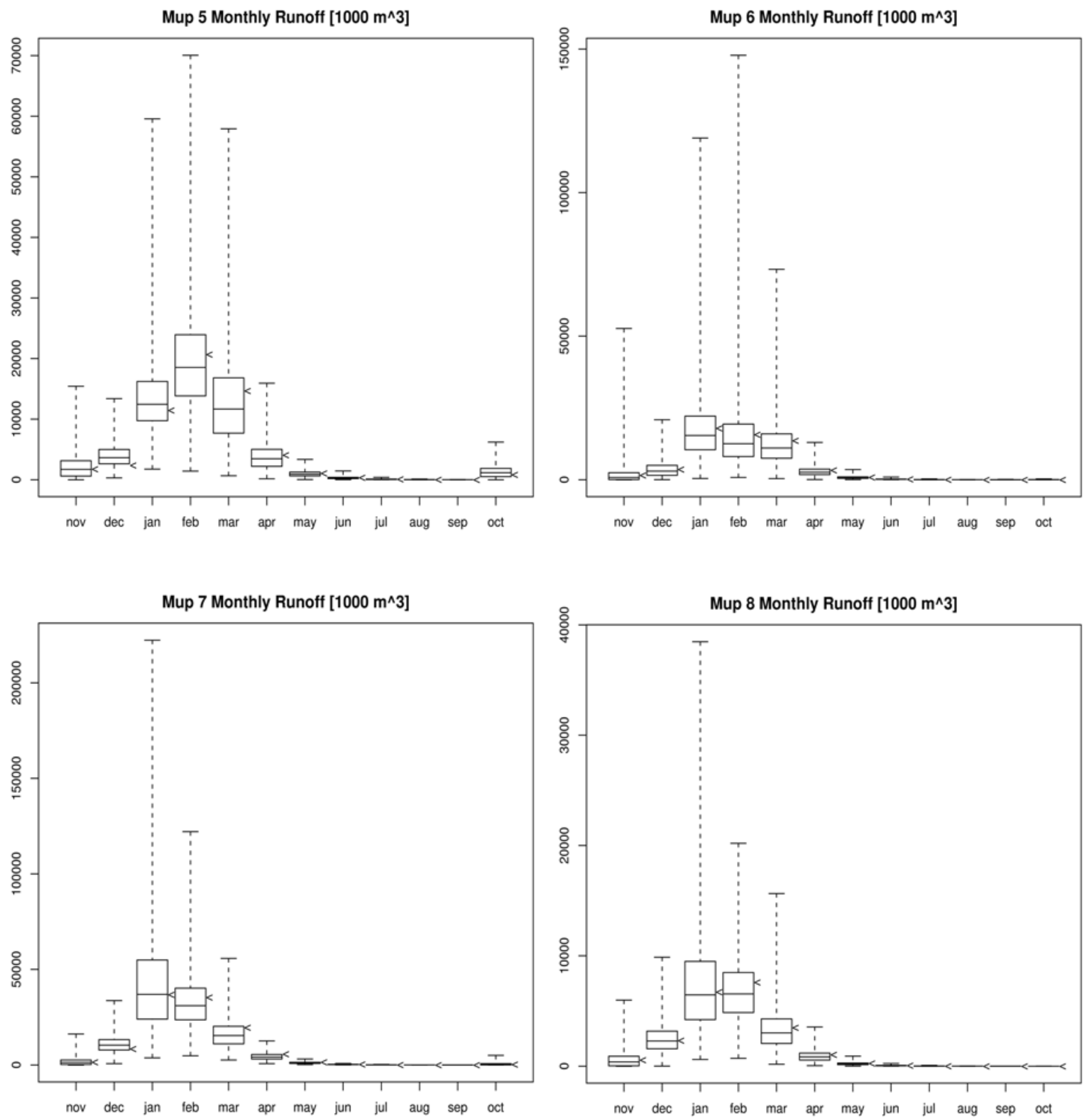


Fig. 7.23 Observed and generated mean monthly runoff at each site. Estimates obtained from historical time series indicated with “<”.

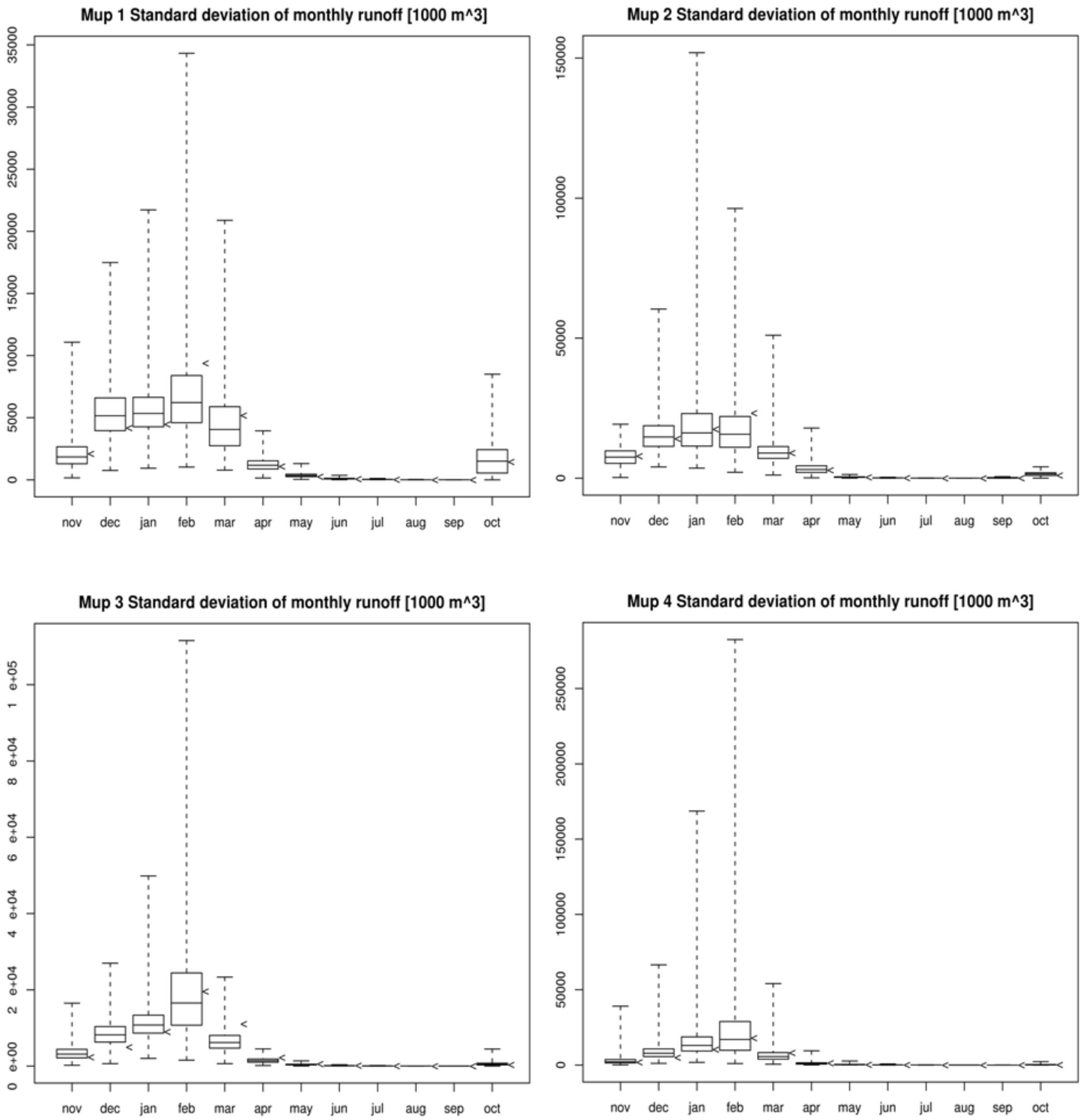


Fig. 7.24 Observed and generated standard deviation of monthly runoff at each site. Estimates obtained from historical time series indicated with “<”.

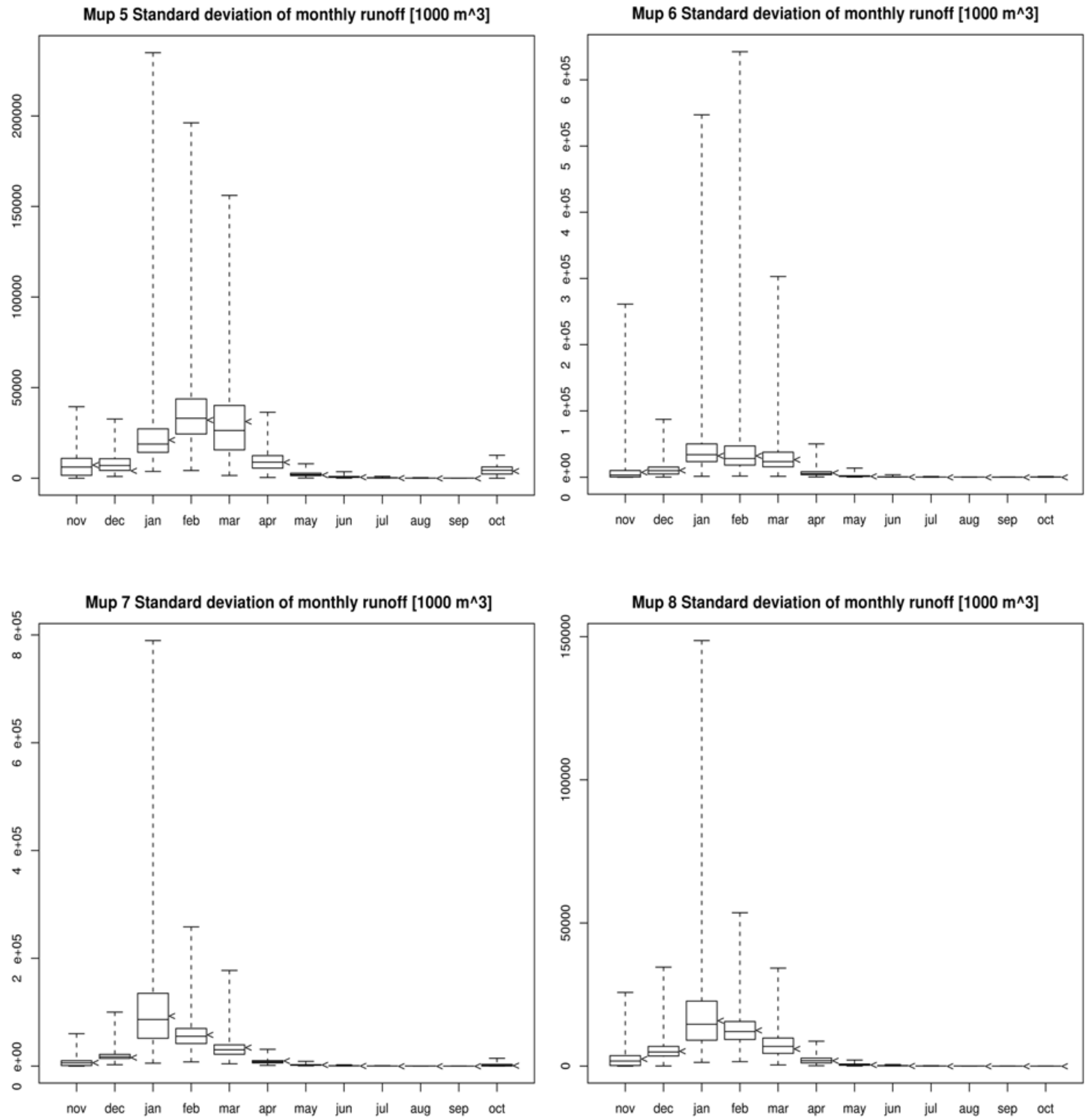


Fig. 7.24 Observed and generated standard deviation of monthly runoff at each site. Estimates obtained from historical time series indicated with “<”.

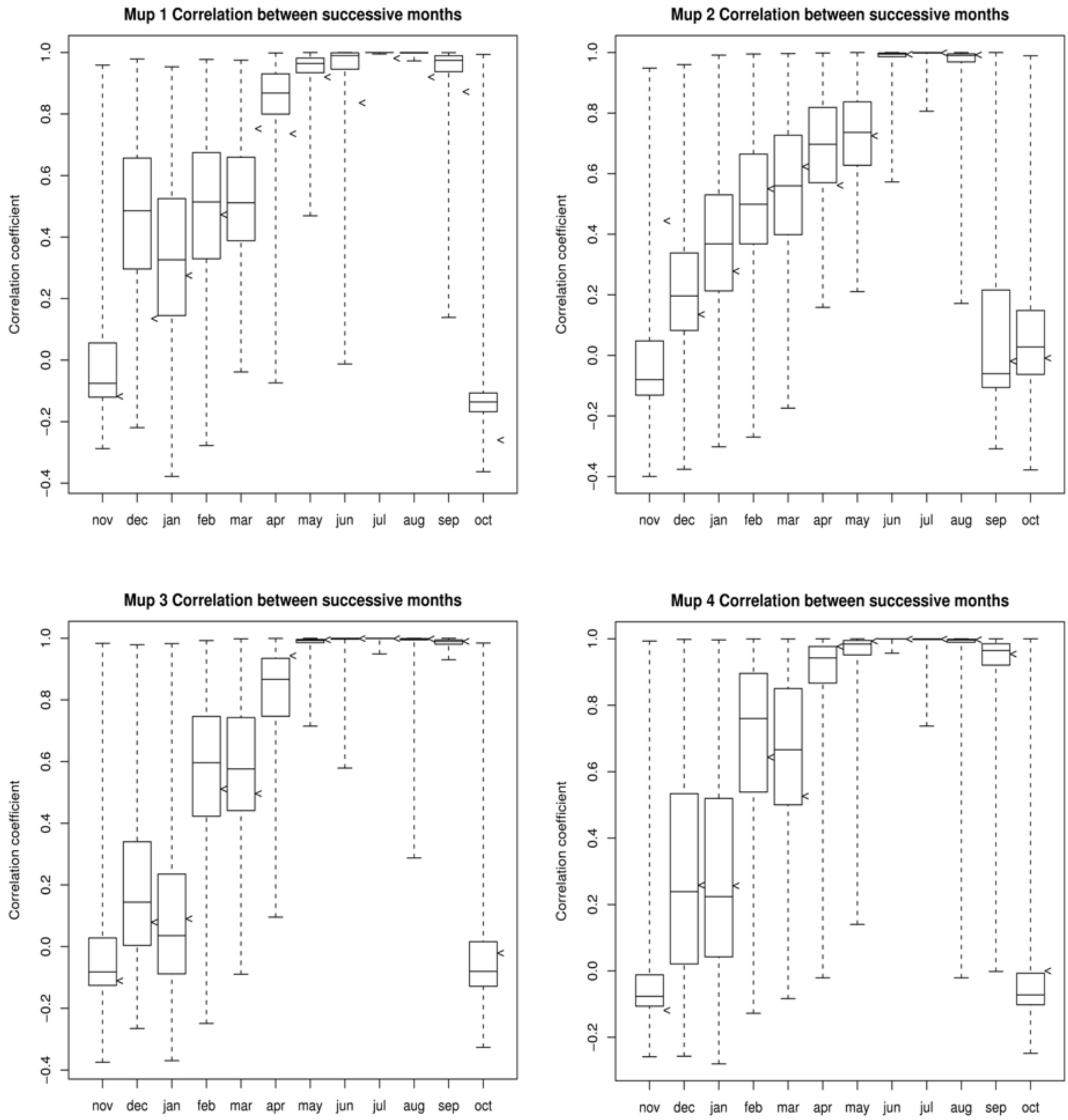


Fig. 7.25 Observed and generated correlation between successive months. Estimates obtained from historical time series indicated with “<”.

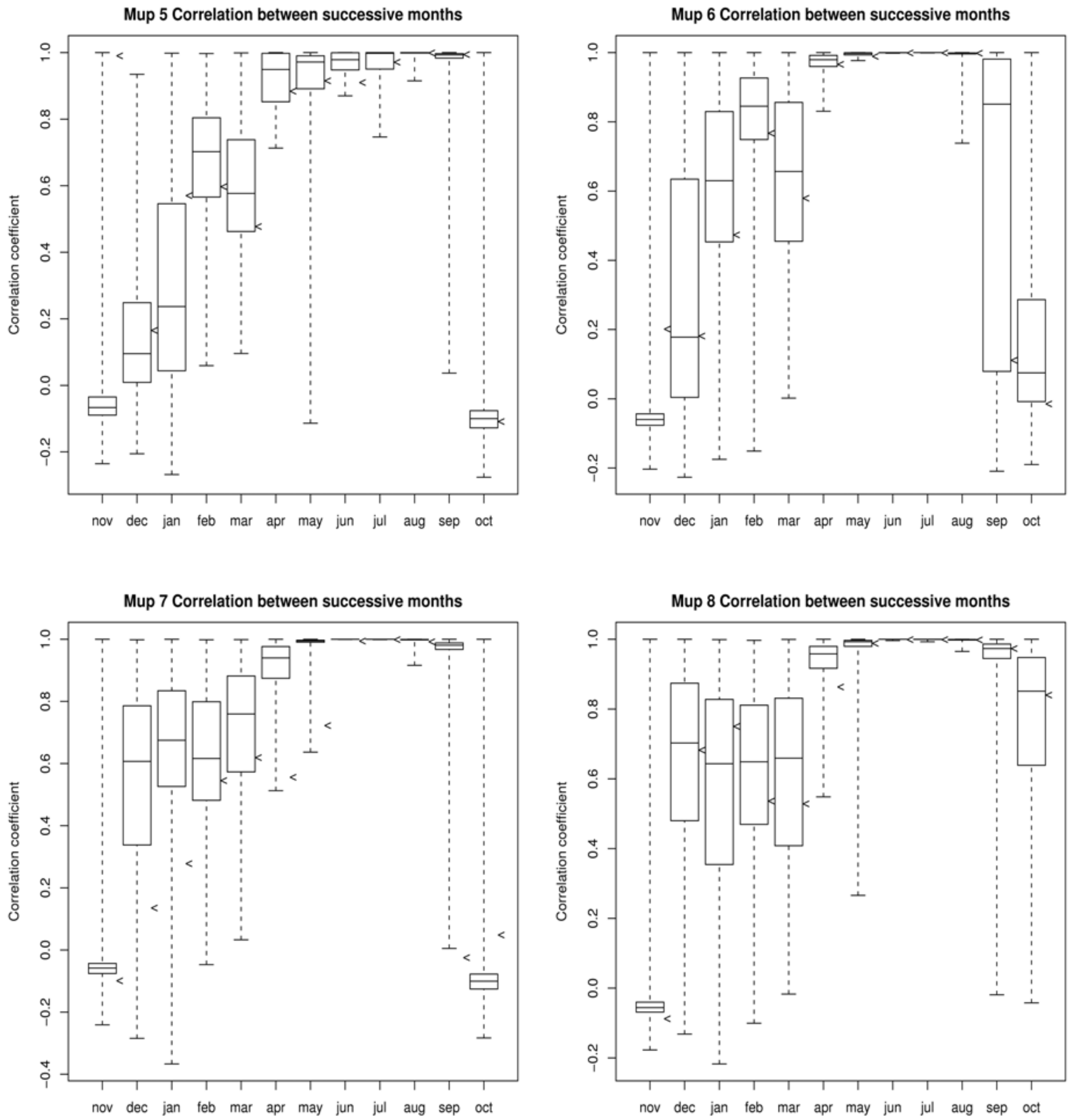


Fig. 7.25 Observed and generated correlation between successive months. Estimates obtained from historical time series indicated with “<”.

Correlation between runoff in successive months The correlation between runoff in a specific month and the runoff in the preceding month obtained through time series generation is compared to the corresponding historical estimates in Fig. 7.25. For most sites and for most months the historical estimates fall within the 0.25 - 0.75 quantile interval of the estimates from the generated time series. The method of fragments does not preserve the correlation between the last month of a hydrological year and the first month of the following hydrological year. In this case, the hydrological year has been defined from November to October, the historical correlation between these two months is not preserved by the method of fragments. As the hydrological year was defined based on the minimum correlation between runoff in successive months this model deficiency is considered not significant.

Maximum deficit volume The maximum observed deficit volume observed in historical time series was compared to the corresponding estimates obtained from the generated time series in Fig. 7.26 with an annual demand corresponding to 80% of the mean annual runoff distributed equally to all twelve months. For all eight sites the historical estimates falls within the 0.25 – 0.75 quantile interval of the estimates from the generated time series.

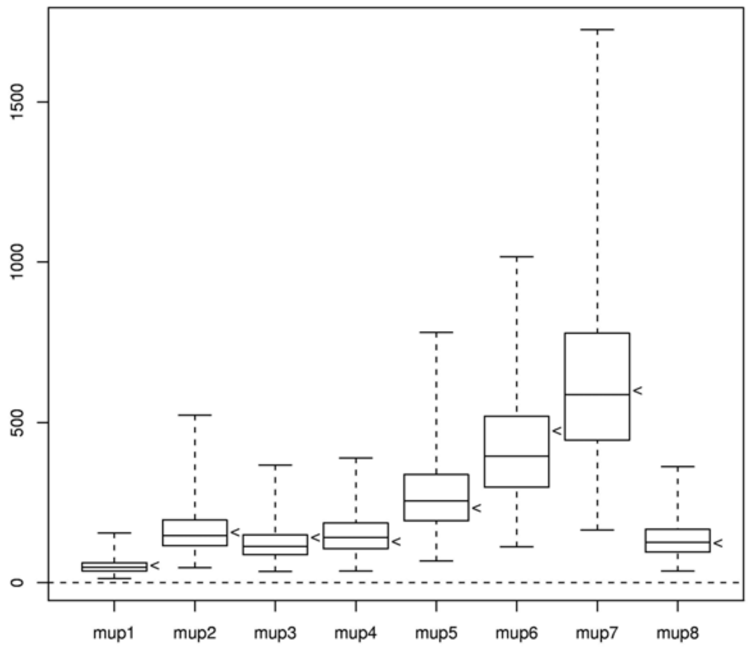


Fig. 7.26 Observed vs. simulated maximum deficit volume [10^6 m^3]. Estimates obtained from historical time series are indicated with “<”.

Hence, the estimated CARMA(0,0) model is considered able to reproduce the maximum deficit volumes observed in the historical time series. This is considered an important result, as these events are responsible for the failure events measured by the water driven sustainability criterion.

Lag-zero cross correlation The lag-zero cross correlation between historical time series of annual runoff at the eight different sites was compared to the corresponding estimates obtained from the generated time series. Due to the high number of possible combinations of sites (8· (8-1)/2 = 28), the historical and the mean value of the N = 5000 generated estimates are compared by plotting them against each other as shown in Fig. 7.27.

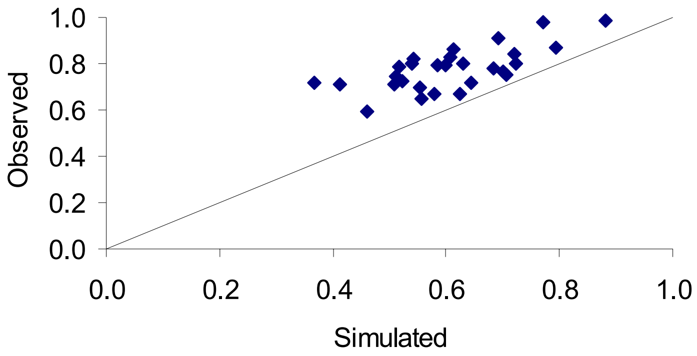


Fig. 7.27 Observed vs. simulated lag-zero cross correlation between time series of annual runoff.

The mean values of the estimates from the generated time series are lower than the corresponding estimates obtained from the historical time series. The absolute difference between the two estimates tends to decrease for increasing correlation. This is similar to the results obtained in the South African case study.

Lag-zero drought correlation The lag-zero correlation between deficit volume at different sites $r_d(i,j)$ is specified by Eq. (5.23). Estimates of $r_d(i,j)$ obtained from historical time series were compared to the mean value of the estimates from the N = 5000 generated time series, see Fig. 7.28.

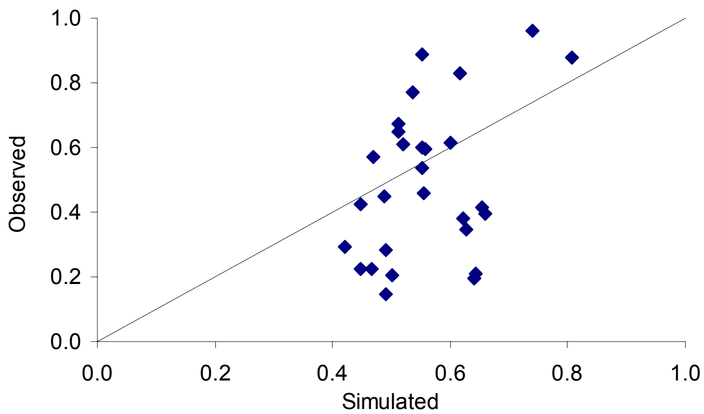


Fig. 7.28 Lag-zero cross correlation $r_d(i,j)$ between deficit volume at different sites.

From Fig. 7.28 it can be observed that there is a high degree of variation in the lag-zero correlation between deficit volume. For the time series of observed annual runoff r_d varies between 0.1 and 1.0, whereas the generated values of r_d are confined to the narrower interval between 0.4 and 0.8.

Summary of CARMA modelling

A multivariate CARMA(0,0) time series model was developed based on 26 years of annual runoff. The model was verified and validated to ensure acceptable reproduction of the statistical properties of the historical data. In both the verification and validation the estimated model demonstrated the ability to reproduce these statistics with a satisfactory precision. Hence, the model is accepted and applied in the further analysis of the Mupfure catchment water resources system.

7.4 DEVELOPING SCENARIOS

The data material and information for the development of water resources management scenarios within the Mupfure catchment have been collected through interviews, literature surveys and visits to field sites. Due to lack of data and available information concerning strategies for future development within the catchment, only very broad and general scenarios could be constructed. Following the terminology outlined in section 3.3 and section 5.4, the scenarios were constructed considering the following variables:

Non-controllable input: The future water availability is simulated using the specified CARMA(0,0) model together with the adjustments reported in section 7.3 for consideration of years with zero annual runoff.

Controllable input: Parts of the MIKE BASIN model setup are changed to adequately model the defined scenarios, in particular water demand and additional reservoir development.

System parameters: Remaining parts of the MIKE BASIN model setup, which are not considered as either non-controllable or controllable inputs, are kept unchanged.

Planning horizon

The outcome of the water resources assessment described in section 7.1 is time series of naturalised runoff for a period of 26 years. Based on these data a planning horizon of 30 years into the future was considered appropriate. This 30-year period was sub-divided into three planning periods, each with a duration of ten years.

Water demand

Predictions of future water demand were considered for three sectors urban, agriculture and the environment. The future water demand of the two first sectors was estimated based on extrapolation of existing data, whereas the water demand of the later sector was based on a reported policy decision.

Urban water demand

The only urban centre included in the analysis is the town of Chegutu. A model for predicting monthly water demand was developed according to the methodology outlined in section 5.5. Annual water abstraction data from October 1979 to September 1991, excluding the drought period from October 1987 to September 1988, were used to estimate the parameters in a zero order growth model for annual water demand [m^3/year] as

$$C(t) = 245877 \left(t - 1985\right) + 2077346 \ , \quad t \geq 1985 \tag{7.15}$$

where $C(t)$ is annual water demand in the t-th year. The monthly correction factors were estimated as the monthly fraction of annual demand. Through an interview with the municipal engineer in Chegutu conducted by Kjeldsen & Lundorf (1997) unaccounted for water was estimated to be approximately 30%. Xie et al. (1993) state that in developing countries in general unaccounted for water is often between 25% and 50%. A thoroughly planned and implemented demand management program may curtail the growth in future water demand. According to the town engineer the following actions need to be taken:

1. Fixing of leaking pipes,
2. Increase price of water, and
3. Installation of water meters.

The implementation of these steps might be complicated by the lack of funding and the fact that no plan of the existing pipe network is available. As an alternative to continued growth in demand, a demand management scenario with a constant demand equal to the average water demand from the first ten year planning period, i.e.

$$C(t) = 7.70\ 10^6\ \mathrm{m}^3/\mathrm{year} \tag{7.16}$$

Agricultural water demand

The historical estimates of agricultural water demand were based on the modelling results from the ACRU model as described in section 7.1. No information concerning predictions of future water demand was available for the Mupfure catchment. Historical data of irrigated area for the entire country were available from FAO (2001) and shown in Fig. 7.29.

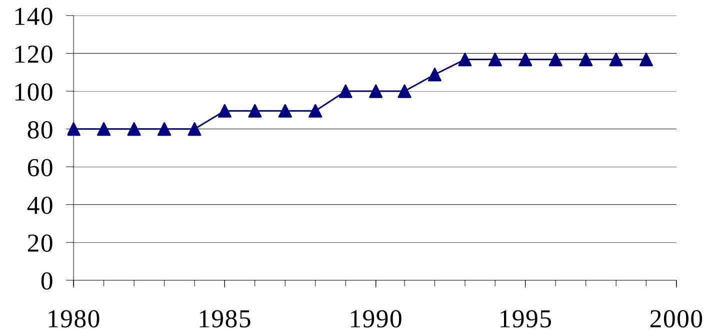


Fig. 7.29 Historical estimates of total irrigated area [1000 ha] within Zimbabwe (FAO, 2001).

These data show an increase of irrigated area in the range 7% - 12% approximately every ten years. Therefore, a 10% step increase in agricultural water demand every ten years, i.e. for every planning period, is assumed. No studies on the potential of increasing the irrigation efficiency in terms of water consumption in neither the communal nor the commercial areas were identified. To investigate the effect of agricultural water demand, a demand management program assuming zero growth in the commercial agricultural sector was introduced. No reduction in agricultural water demand from the communal areas was assumed.

Environmental water demand

The old Water Act had no considerations of the environmental water demand (Kjeldsen et al., 1999b). With the introduction of the new Water Act the environment has been accepted as a water user with a legitimate demand for water. However, no guidelines for the inclusion of environmental water demand into operational water resources management have been developed so far. Makoni (2000) refers to guidelines from MLWR stating that 5% of MAR should be saved for the environment, however no sites within the catchment have been identified for determination of the instream flow requirements. The determination of the environmental water demand is further complicated by the fact that it is not uncommon for rivers in Zimbabwe to have no flow during the dry season. In a review of the South African approach to determination of environmental need for water by King & Louw (1998), the case of water demand for an ephemeral river should be analysed considering the groundwater level. The groundwater level should be linked to information concerning water holes used by rural communities, livestock and wildlife and the root depth of riparian vegetation. As an attempt to include considerations of environmental water demand within this sustainability assessment only the occurrence of surface water was considered. A site of interest was defined at the outlet of the considered part of the catchment. At this point monthly target flow values were estimated as the 95% fractile in the flow duration curve for each month.

Infrastructure development

To overcome the threat of future water shortage as experienced during the recent droughts, a number of major reservoir schemes were considered constructed within the Mupfure catchment. The general flat topography of the catchment means that few sites are appropriate for construction of such major reservoirs. A survey carried out by MLWR in 1988 of possible dam sites resulted in the identification of two sites at Muda and Mhondoro, respectively, see Fig. 7.30 (Makoni, 2000). The corresponding names of the reservoir schemes are the Muda Dam and the Mhondoro Dam.

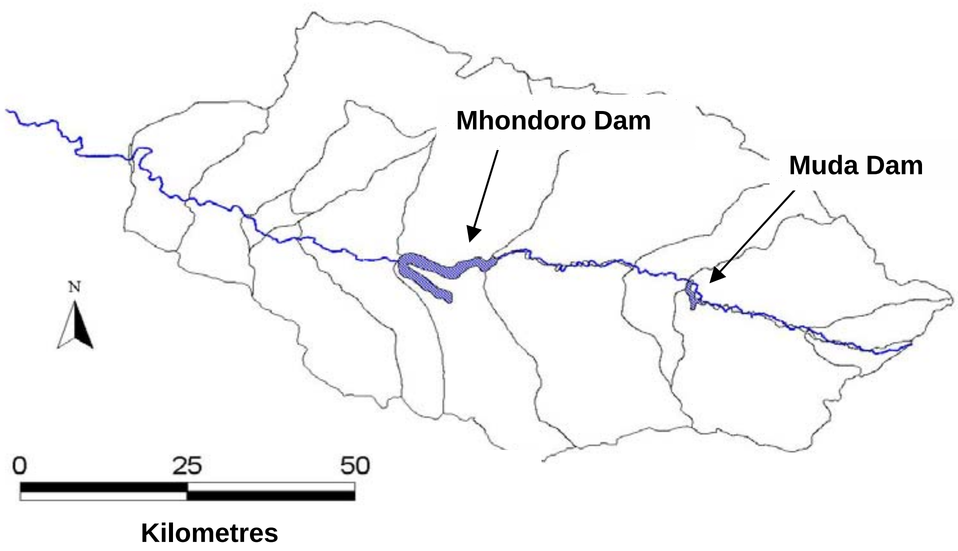


Fig. 7.30 Location of the Muda and Mhondoro dam sites.

The Muda Dam option

The Muda dam is the smallest of the two considered reservoir options, and is intended to supply water to both communal and commercial agriculture in the upstream part of the catchment. The technical specifications of the reservoir are given in Table 7.20.

Table 7.20 Technical specifications of the Muda Dam.

Muda Dam	
Full Supply Capacity [10^6 m^3]	98
Height [m]	31.5
Inundated area at full supply level [ha]	1400
Dead storage [10^6 m^3]	2.5

The zoning of the reservoir and the available HVA curves are shown in Fig. 7.31.

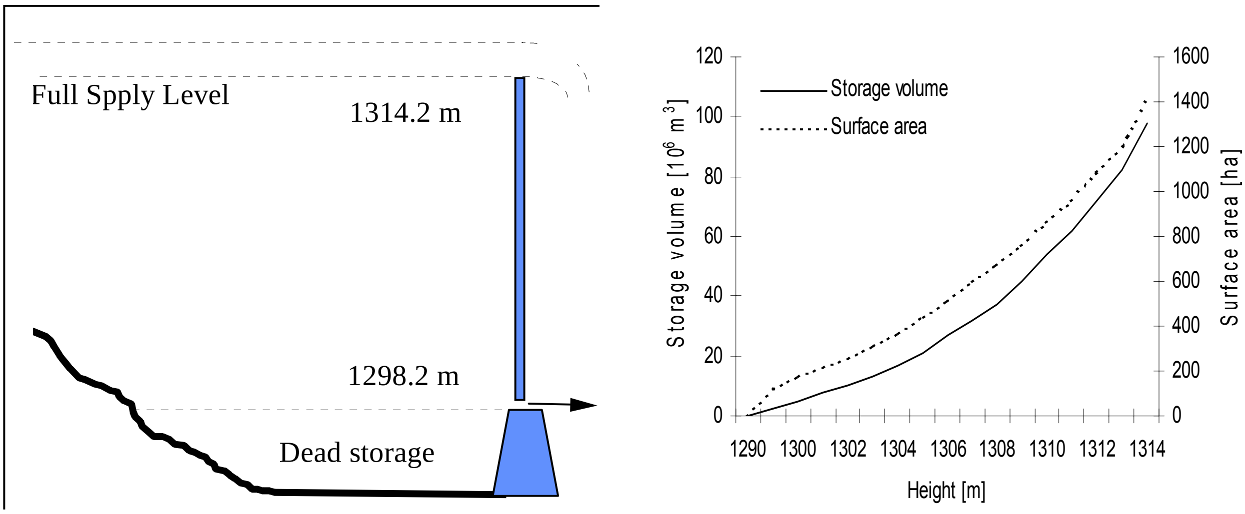


Fig. 7.31 Zoning and available HVA relationship for the Muda Dam.

When added to the existing MIKE BASIN setup, the Muda dam is assumed to deliver water directly to all users in the catchment. The loss factor for each user equals the previously specified values.

The Mhondoro Dam option

The Mhondoro Dam was first suggested in 1988 and since the plans have been revised numerous times. The scheme has been the source of disputes between farmers and the government, as the reservoir is so large that it will offer control of the bulk of the water resources within the catchment. In its current layout the reservoir will secure supply to the agricultural and urban sectors, both within and outside the Mupfure catchment. The technical specifications of the reservoir are shown in Table 7.21.

Table 7.21 Technical specifications of the Mhondoro Dam.

Full Supply Capacity [10^6 m^3]	444
Height [m]	42.5
Inundated Area at Full Supply Level [ha]	5612
Dead storage [10^6 m^3]	0.3

No HVA curves were readily available from MLWR for this reservoir, but based on full supply capacity, the surface area of the water body at full supply capacity and height of dam wall, the methodology outlined in Appendix B was used to estimate parameters for the MIKE BASIN description of HVA curve corresponding to the general Volume-Area relationship for large reservoirs in Zimbabwe developed by Michell (1982).

Water from the Mhondoro Dam will be used for irrigation of approximately 2000 ha situated outside the considered catchment. Assuming the specific water use in these areas equals the before mentioned figure of 12.0 10^3 m^3/ha (Makoni, 2000) with the same monthly pattern of water demand as observed in the Mupfure, an annual amount of 24.0 10^6 m^3 will be abstracted from the reservoir. The priority of this water use will be placed below the irrigation water demand in the Mupfure catchment, and no return flow back to the considered system occurs. Note, that this irrigation scheme is not included in the sustainability assessment. When added to the existing MIKE BASIN setup, the Muda dam is assumed to deliver water directly to all users in the catchment. The loss factor for each user equals the previously specified values. The irrigation scheme outside the catchment receiving water is assumed to have a loss factor of 0.40.

Scenario matrix

Based on the outlined development and management options a scenario matrix considering different combinations of demand and supply options was defined as shown in Table 7.22.

Table 7.22 Scenario matrix for future development in the Mupfure catchment. Numbers in ( ) refer to scenario number.

	No development	Mhondoro	Muda	Muda + Mhondoro
No DM	✓ (1)	✓ (5)	✓ (6)	✓ (10)
Commercial Agr. DM	✓ (2)		✓ (7)	
Urban DM	✓ (3)		✓ (8)	
Urban + Agr. DM	✓ (4)		✓ (9)	

Options considering both the construction of the large Mhondoro reservoir and the reduction of water demand were considered inconsistent and therefore not included as a valid scenario. Scenario 1 considering continued unconstrained growth in water demand in both the urban and agricultural sector and with no new reservoir development equals the “business as usual” scenario.

7.5 EVALUATING SCENARIOS

The evaluation of the ten scenarios in the scenario matrix in Table 7.22 will be conducted according to the methodology outlined in section 4.9, as a combination of the water driven criterion and a RIAM investigation.

The water driven sustainability criterion

All users included in the MIKE BASIN setup, as shown in Fig. 7.14, are impacted by at least one or more of the ten scenarios and, therefore, included in the sustainability assessment. Table 7.23 shows these users arranged according to sectors and with reference to Fig. 7.14.

Table 7.23 Users included in the sustainability assessment.

Sector	User
Urban:	Chegutu municipality
Commercial agriculture:	Irr 2 , Irr 4
Communal agriculture:	Irr 1, Irr 3
Industry:	DWT (textile factory)
Environment:	IFR site at outlet of MIKE BASIN setup

The sensitivity analysis carried out in section 6.8 showed little difference between estimates of rS_{WDC} when using different combinations of estimators of resilience and vulnerability. Therefore, the estimators defined in Eq. (4.22) were selected, i.e.

Reliability: Occurrence reliability,
Resilience: Estimated through mean value of failure duration, and
Vulnerability: Estimated based on the 0.9-th fractile in the empirical cdf of observed deficit volume.

Finally, rS_{WDC} is estimated though Eq. (4.8). The weights $w(c)$ indicating the relative importance of the c-th user are assumed equally distributed among the seven users. Implication of the choice of weights will be addressed in a proceeding sensitivity analysis. The score card in its full can be found in Appendix E and the corresponding estimates of rS_{WDC} are shown in Table 7.24.

Table 7.24 Ranking of scenarios based on rS_{WDC} (weights *w(c)* are assumed distributed equally among users).

Scenario no.	rS_{WDC}	Rank	Scenario no.	rS_{WDC}	Rank
1	0.136	10	6	0.163	6
2	0.147	8	7	0.187	2
3	0.139	9	8	0.165	5
4	0.154	7	9	0.190	1
5	0.174	4	10	0.184	3

From the results in Table 7.24 it can be observed that the sustainability score obtained for each scenario reflects the ratio between storage capacity and demand. The more storage and the lower demand the higher the estimated value of rS_{WDC}. The scenarios including the reservoir options have a higher ranking than the non-reservoir scenarios. The demand management options also have an influence on the ranking, but not as significant as the reservoirs. In fact, comparing the Muda dam option and the non-reservoir option, the demand management option is responsible for sub-ranking the scenarios within each group, but non of the non-reservoir scenarios have higher or equal score to any scenario including the Muda dam. Constructing both reservoirs is observed to give a smaller rS_{WDC} score compared to construction of the Muda dam and the introduction of demand management in the various sectors.

The obtained estimates of rS_{WDC} are closer to each other than observed in the South African case study, which is caused by a number of factors. First, in the Mupfure catchment, non of the proposed scenarios provide a total eradication of failure periods as in the previous case study. Persistent droughts will affect the entire catchment and non of the proposed reservoirs are able to provide total protection from this natural hazard. Next, The construction of either the Mhondoro Dam or the Muda Dam will have a relatively minor impact on the estimates of R-R-V obtained for Chegutu, DWT and the IFR site, as the water demand from the two users are taken care of by ample existing storage capacity. Finally, the large number of scenarios will reduce the estimates of relative vulnerability rVul as estimated through Eq. (4.5), resulting in smaller rS_{WDC} scores.

Initial environmental evaluation

As required by the developed sustainability assessment methodology an initial environmental evaluation (IEE) was conducted to include issues of importance not accounted for by the water driven rS_{WDC} criterion. The description of the impacts and the results presented here are based the findings of Makoni (2000) and presented in an aggregate format by Makoni et al. (2001). Slight adjustments have been necessary as some of the impacts identified by Makoni (2000), such as impact on aquatic ecosystem, are considered to be accounted for by the water driven criteria. Impacts associated with the following four options were considered: no reservoir development, construction of Muda dam, construction of Mhondoro Dam and construction of both Muda and Mhondoro dams. No impacts associated with the demand management options were included. The four options are valued against the following 13 issues subdivided into physical/chemical, biological/ecological, social/cultural and economic/operational components. The issues within each component were given equal weight.

Physical / Chemical component

PC1 – Impacts on soils

The establishment of embankment foundation and access road to and from the area will necessitate the removal of a considerable amount of soils. Furthermore, riverbed sand from upstream the reservoirs will have to be excavated and removed in order to make space for the dam foundations. Also pollution of soils from spills of fuel, oil or chemicals used during construction can occur. The impacts is likely to be more significant at the Mhondoro site rather than the Muda site due to the different size of the reservoirs.

PC2 – Impacts on the visual/aesthetic character of the area

Both reservoirs are planned constructed in picturesque areas with a preference for the Mhondoro site. Both construction and the final reservoirs are believed to have a negative effect on the aesthetic character of the areas.

Biological/Ecological components

BE1 – Loss of habitat for terrestrial mammals and avifauna

The area has no special conservation status or wildlife utilisation,. However, the inundation of land and removal of vegetation will have negative consequences for the current animal population. Also reduction in flood frequencies and magnitudes may affect floodplain habitat. The loss of habitat impacts associated with the Mhondoro dam are likely to be of greater significance than those associated with the Muda dam, due to the greater land area being inundated.

BE2 – Impacts on flora/vegetation

The areas inundated by the reservoirs contain indigenous flora and vegetation, which will be lost. The Muda site is situated on land already cleared for agricultural use and therefore host a less varied plant population than the larger Mhondoro site. Hence, these impacts will be of greater significance at the Mhondoro site.

Social/Cultural components

SC1 – Impacts on human settlements

Construction of the Mhondoro dam will inundate nine villages with a total of 250 households and one school. The construction of the Muda dam will inundate only a small number of farms.

SC2 – Loss of historical, archaeological and cultural sites

A total of five archaeological sites have been identified within the area to be inundated by the Mhondoro dam. These sites include war memorials, ruins and rock paintings. Furthermore, a number of graves exist within the area. No sites of interest were identified within the Muda dam site.

SC3 – Impacts on human health

The presence of large bodies of water may result in the increase of water borne diseases such as Bilharzia and Malaria. The negative effect is likely to be most significant at the Mhondoro site, as this area is more densely populated than the Muda site. Furthermore, the influx of migrant workers during dam construction may potentially lead to an increase in the spread of HIV/AIDS. Health education and vector control should be important aspects of the dam projects.

SC4 – Impacts on recreational activities

Both reservoirs offer the possibilities for recreational activities and occupational fishing for local people. Neither the reservoirs are likely to have significant potential for regional or national/international tourism.

SC5 – Impacts on migration away from the area

The construction phase of the reservoir will provide short-term employment whilst the long-term presence of the reservoirs, provided the water is accessible to local people, would enhance local farming opportunities. These factors should help to reduce migration away from the area. Due to its location in a more densely populated area, the Mhondoro dam is likely to have a more significant impact in this regard.

Economic/Operational components

EO1 – Impacts on local economy

There are likely to be substantial short- and long-term economic benefits from the dam construction. Short-term benefits will include hiring of local workforce, local service industry and producers and suppliers of building materials. On a longer term the increased access to water may facilitate increased agricultural production. The reservoirs may also initiate recreational activities and thereby generating income and creating employment. The Mhondoro dam is likely to have the more significant impact in this regard.

EO2 – Impacts on food security

The establishment of a large dam secures reliable water supply, which potentially can increase agricultural output enhancing food security at both local and regional level.

EO3 – Impacts on infrastructure

Due to construction existing roads, power and communication infrastructure will be upgraded.

EO4 – Impact on economy of external user

When constructing the Mhondoro dam, water will be transferred to an irrigation scheme outside the Mupfure catchment. The potential gain from increased agricultural production in this specific area will, potentially, have a positive effect on economy, food security, job creation etc. No inter-basin transfer is carried out by construction of the Muda dam alone.

The score for each component with respect to the RIAM parameters (A_1, A_2, B_1, B_2, B_3) are shown in Appendix F and a summary of the final FAS score for each component for each scenario is shown in Table 7.25

Table 7.25 Summary of FAS score for each reservoir development option.

Component no.	No reservoir	Muda dam	Mhondoro Dam	Muda + Mhondoro Dams
PC1	0	-14	-28	-42
PC2	0	-14	-28	-42
BE1	0	-27	-54	-54
BE2	0	-27	-54	-54
SC1	0	-18	-54	-54
SC2	0	0	-84	-84
SC3	0	-18	-36	-54
SC4	0	12	12	24
SC5	-24	24	48	72
EO1	0	48	72	72
EO2	0	18	18	36
EO3	0	36	54	54
EO4	0	0	72	72

The results in Table 7.25 show that while construction of reservoirs has a negative impact on the natural system, especially the anticipated economic gains are regarded as positive. However, the World Commission on Dams (WDC, 2000) warns that the expected scope in local economy following the construction of large dams rarely materialises quite as significantly as expected. The social and cultural impacts are divided into both positive and negative impacts where the cultural impacts (SC1 and SC2) are perceived as negative, while the social impacts (SC4 and SC5) are regarded as being more positive apart from influence on public health (SC3), which is also regarded as being mostly negative. In Fig. 7.32 the score for each sub-category (PC, BE, SC, EO) calculated through the part of Eq. (4.24) in brackets are shown for each option (note that low scores are preferable).

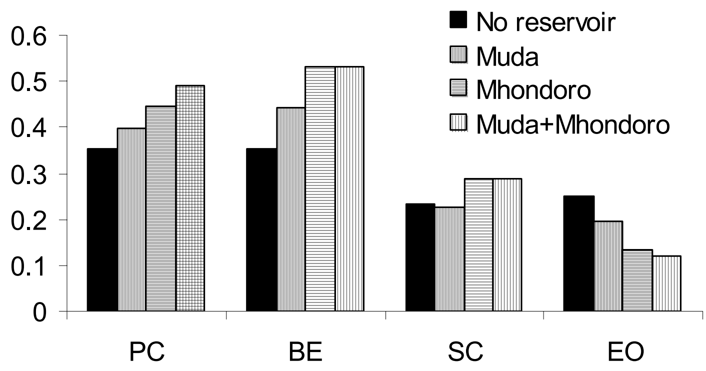


Fig. 7.32 RIAM scores for each sub-category.

From Fig. 7.32 it can be observed that options which perform poor in terms of PC and BE perform better in EO and vice versa. Thus, in average the obtained score (rS_{IEE}) for the different scenarios will be close to each other.

The final outcome of the RIAM investigation, see Table 7.26, shows that the no-reservoir option has the highest score followed by the Muda dam option, the Mhondoro Dam option and, at last, the Muda and Mhondoro Dams option with the lowest score. This indicates that the non-water demand related aspects of reservoir construction in the Mupfure catchment are perceived as being more negative than the positive benefits. However, the obtained scores are very close to each other.

Table 7.26 Results of RIAM investigation

Scenario no.	rS_{IEE}	Rank	Scenario no.	rS_{IEE}	Rank
1	0.702	1	6	0.684	5
2	0.702	1	7	0.684	5
3	0.702	1	8	0.684	5
4	0.702	1	9	0.684	5
5	0.650	9	10	0.642	10

Results

The final sustainability score for each of the ten scenarios are estimated through Eq. (4.25) using the results from Table 7.24 and Table 7.26 and shown in Table 7.27.

Table 7.27 Results of sustainability assessment for the Mupfure water resources system.

Scenario no.	rS_{WDC}	rS_{IEE}	$rS = rS_{WDC} \times rS_{IEE}$	Rank
1	0.136	0.702	0.094	10
2	0.147	0.702	0.102	8
3	0.139	0.702	0.096	9
4	0.154	0.702	0.106	7
5	0.174	0.650	0.111	5
6	0.163	0.684	0.111	6
7	0.187	0.684	0.127	2
8	0.165	0.684	0.112	4
9	0.190	0.684	0.129	1
10	0.184	0.642	0.116	3

When ranking the ten scenarios based on the rS scores, as shown in Table 7.27, it is observed that the outcome is very similar to the ranking of the scenarios based on the water driven criterion rS_{WDC} shown in Table 7.24. Thus, scenarios considering the Muda dam together with demand management in the agricultural and/or urban sector have a high degree of relative sustainability (scenario no. 6 to 9), indicating high scores with respect to both the water driven and the non-water driven criterion. Also, the scenario including the construction of the Mhondoro dam together with the Muda dam (scenario no. 10) attracts a high score. Considering the construction of the Mhondoro Dam by itself (scenario no. 5) only a mediocre ranking, i.e. the benefits from such a large reservoir in terms of supply and demand could not outweigh the negative impacts imposed by the reservoir, even though the possible benefits from the interbasin transfer were included. Therefore, construction of the Mhondoro dam must be regarded as being a less sustainable option than the Muda dam option. Scenarios with no consideration of reservoir construction (scenario no. 1 – 4) must be regarded as being the least sustainable, especially the business as usual scenario, which was ranked the lowest of all. Initiation of a demand management program in the commercial agricultural sector has a significant positive influence on the relative sustainability. This is not surprising considering that this sector is responsible for the bulk of the water demand within the catchment. Also a demand management program in the Chegutu municipality would have a marked effect on the relative sustainability of the entire water resources system.

7.6 SENSITIVITY ANALYSIS

A sensitivity analysis was carried out to investigate the effect of choices made concerning parameter values and methodologies. In the Mgeni-Mkomazi case study it was shown that the choice of estimators of R-R-V had no practical influence on the final ranking of the scenarios. Also, the ranking was found insensitive to the use of either historical or stochastically generated runoff. The latter result does not coincide with the findings concerning the non-uniform behaviour of estimators of resilience and vulnerability found in section 4.5, but was attributed to the large difference between the analysed scenarios. To further investigate this aspect, a similar sensitivity analysis is carried out for the Mupfure system. Also, the sensitivity of the relative sustainability *rS* with respect to the weights *w(c)* assigned to the *c*-th user in Eq. (4.8) representing its relative importance was investigated. The outcome of the RIAM investigation was not altered during this sensitivity analysis.

Synthetic vs. historical runoff

Instead of using the stochastic model for generation of 1000 year long time series of monthly runoff in each of the three planning periods, as required by the developed methodology, the corresponding historical time series of monthly runoff (322 months) were repeatedly applied. Estimates of rS_{WDC} were obtained through Eq. (4.22) with an equal weighting of the users. A comparison of the two methodologies is shown in Table 7.28.

Table 7.28 Results of sensitivity analysis of rS_{WDC} comparing the use of synthetic and historic runoff time series

Scenario no.	Synthetic runoff	rank	historical runoff	rank
1	0.136	10	0.128	10
2	0.147	8	0.133	6
3	0.139	9	0.131	7
4	0.154	7	0.140	4
5	0.174	4	0.145	2
6	0.163	6	0.129	9
7	0.187	2	0.144	3
8	0.165	5	0.131	8
9	0.190	1	0.147	1
10	0.184	3	0.140	5

As observed from the results in Table 7.28, a different ranking of the scenarios was achieved by using the short historical time series instead of the 1000 years long stochastically generated time series of runoff. Scenario number nine is still ranked as number one and scenario number one is still ranked as number ten, but the remaining scenarios in between have all obtained a new ranking. This new ranking no longer reflects the ratio between water demand and storage capacity. Together with the results from section 4.5 this investigation highlights the need for stochastic models to generate sufficiently long time series so that robust estimates can be obtained.

Choice of preferences

The sensitivity is investigated in terms of a group decision-making analysis. Raju et al. (2000) presented a technique for aggregation of preferences of multiple decision-makers into a single aggregate ranking of the relevant scenarios. Consider a problem involving multiple decision-makers *ND* each with different sets of preferences *w(c)* choosing between multiple scenarios *NI*. The additive ranking rule is defined by Raju et al. (2000) as

$$r^*(i) = \frac{1}{ND}\sum_{u=1}^{ND} r(u,i) \tag{7.16}$$

where

$r^*(i)$ = the aggregate ranking score for the *i*-th scenario considering the *ND* decision makers and the *NI* scenarios.

$r(i,u)$ = the ranking of the *i*-th scenario by the *u*-th decision maker.

The *NI* scenarios are ranked according to the obtained aggregate ranking scores, with preference given to low scores. In this case study the ten scenarios are presented to five hypothetical decision-makers each with the following preferences:

1. Equal preference with respect to all seven users.
2. Preference for the urban and industry sectors.
3. Preference for the commercial agricultural sector.
4. Preference for the commercial agricultural sector.
5. Preference for the environment.

Decision-makers with a special preference (2. – 5.) distribute half the relative weight (0.5) to the users within the preferred sector and the other half of the weight to the remaining users. The ranking of the ten scenarios obtained by each of the five decision-makers together with the aggregate rank are shown in Table 7.29.

As observed most decision-makers will rank the scenarios in a manner similar to the results obtained by assuming the weights distributed equally between the seven users included in this case study, suggesting a high degree of robustness. The results emphasise the previously obtained results, that scenarios including construction of the Muda dam are generally having the highest degree of relative sustainability, especially when combined with a strong focus on demand management in both the urban and the agricultural sectors. Alternatively, the scenario considering the construction of both reservoir options could be considered having a high degree of sustainability. The scenarios considering the no-reservoir option have a low degree of relative sustainability and should not be considered.

Table 7.29 Summary of ranking by each decision-maker and aggregate ranking.

Scenario no.	Equal weight	Urban, Industry	Commercial agr.	Communal agr.	Environment	$r^*(i)$	Aggregate rank
1	10	10	10	10	10	10.000	10
2	8	8	8	8	8	8.000	8
3	9	9	9	9	9	9.000	9
4	7	5	7	6	7	6.400	7
5	5	3	4	7	6	5.000	5
6	6	7	6	5	3	5.400	6
7	2	2	2	2	1	1.800	2
8	4	6	5	4	4	4.600	4
9	1	1	1	1	3	1.400	1
10	3	4	3	3	5	3.600	3

7.7 DISCUSSION AND CONCLUSION

The methodology outlined in section 4.9 for assessing the relative sustainability of various water resources management and development options has been tested on the Mupfure water resources system. The case study is characterised by a large inter- and intra-annual variability of water availability and a variety of sectors with competing water demands. In previous periods of droughts the water shortage has led to conflicts between the various users. To better cope with future droughts, plans for the construction of one or two large reservoirs within the catchment have been laid out. Before a sustainability assessment of different development options could be performed an extensive data collection effort was undertaken. Through field studies, literature surveys, and interviews with local stakeholders and water managers sufficient data to cover the minimum requirements for the evaluation were collected. However, more data especially concerning possible future development of water demand would have been valuable.

First, an assessment of water availability was conducted to obtain time series of naturalised monthly runoff. The assessment was carried out using the ACRU model. Enough data to cover the minimum requirements of the model was collected for the period 1970 to 1996. The performance of the model was assessed through a number of numerical and visual criteria. Taken into consideration the sparse data material, the setup of the ACRU model was accepted for a water resources assessment of the Mupfure catchment. From the ACRU model setup time series of naturalised monthly runoff, i.e. runoff without the influence of reservoirs and abstraction, at points of interest within the catchment as well as estimates of irrigation water demand were extracted. Using the time series of naturalised monthly runoff together with other relevant water resources system variables the MIKE BASIN model was setup and calibrated for the Mupfure water resources system. A multivariate stochastic model for generation of annual runoff at multiple sites taking into consideration the frequency of years with zero annual runoff was developed and successfully implemented. It was considered necessary to develop such a model to ensure a realistic representation of situations with low water availability on a catchment basis, as these periods are responsible for the failure events used to assess the relative sustainability. The model generates time series of annual runoff at each site with a mean value and frequency of occurrence of years with zero runoff corresponding to the observed time series. The standard deviation of annual runoff, however, is not a priori preserved, but validation results show that also this property was reproduced with a satisfactory degree of precision. Also, the observed maximum deficit volume and the correlation between deficit volumes at different sites were well preserved by the model. Based on the results obtained through verification and validation the estimated multivariate model was accepted and applied in the further assessment of the relative sustainability.

Through field studies, literature surveys and interviews with involved people and water managers a number of possible options concerning the future development of the water infrastructure within the catchment were identified and combined into ten scenarios. Currently, the construction of two major reservoirs is being considered. A very large 444 10^6 m^3 reservoir to be built in the Mhondoro communal area (the Mhondoro dam) and the smaller 98 10^6 m^3 Muda dam situated on sparsely inhabited agricultural land further upstream. No information concerning the potential future development of water demand in the various sectors was identified, thus a number of simplified assumptions were made to investigate the effect of demand management within different sectors. The water demand for sustaining the aquatic ecological system in the river system was included in the analysis as a monthly varying water demand at the outlet of the considered part of the catchment. Due to the lack of information concerning temporal development of demand the scenarios should be considered examples of future development rather with the potential of identifying points of influence regarding the relative sustainability. The ten scenarios were analysed according to the methodology outlined in section 4.9. The ranking of the scenarios obtained using the water driven criterion with equal weighting of the involved users reflects the ratio between storage capacity and water demand. The lower the demand and/or the higher the available storage capacity the higher the estimate of rS_{WDC} becomes. Therefore, options considering construction of reservoirs together with demand management in the agricultural and/or urban sectors were ranked the highest. Reservoir construction was found to have a more significant effect than demand management. The differences between the obtained estimates of rS_{WDC} are smaller than observed in the South African case study, which was attributed to several factors. These factors are 1) non of the specified options could provide fail-safe operation of the system, 2) a limited effect of additional reservoir construction on urban and industry users, and 3) the large number of scenarios giving small values of relative vulnerability, which leads to small values of rS_{WDC}. Considering the construction of the Mhondoro Dam, the transfer of water to an irrigation scheme outside the defined system will have a negative influence on the water driven criterion, calling for an extension of the system boundaries to include this external user in all scenarios. This, however, was not a viable option due to the lack of both hydrological and water resources system related data needed for an appropriate extension of the system to involve this user. Instead, the benefits from the water transfer were attempted explicitly included in the following RIAM investigation. The outcome of the RIAM investigation highlighted the problem associated with an objective assessment of the contribution of a reservoir in terms of sustainability. The larger the reservoir project is the more negative the physical/chemical and biological/ecological impacts are, but at the same time the anticipated economic/operational impacts are additionally more positive. The average score obtained for the various options are, therefore, close to each other. In this study, however, the negative impacts from the reservoir options outweigh the positive contributions. The lowest score was obtained for scenarios considering no reservoirs at all, even when demand management was included. Any non-water related impacts associated with the demand management programs were not included in the RIAM investigation.

The final relative sustainability score *rS* for each scenario showed that scenarios considering construction of the Muda dam together with the introduction of demand management in the commercial agricultural and/or urban sectors have the highest relative sustainability scores followed by scenarios including the Mhondoro reservoir option. Scenarios not considering construction of any reservoirs obtain a low degree of relative sustainability. The fact that the estimates of relative sustainability are very close to each other must be considered a weakness of the methodology and an illustration of the inherent difficulties in a sustainability analysis. It will be difficult for a decision-maker to justify a decision regarding whether or not to construct a reservoir based on the results from this study, also considering the large number of assumptions and uncertainties involved in the analysis. However, the obtained ranking of the scenarios was supported by a sensitivity analysis investigating the influence of different preferences regarding the importance of the water using sectors represented in the case study. The analysis also shows that an almost similar balance between the different objectives can be obtained through different policies, once again highlighting the difficulties inherent to the sustainability concept. The sensitivity analysis also illustrated the effect of non-uniform estimates of resilience and vulnerability can have a significant influence on the ranking of the scenarios.

There is much to do in Zimbabwe to enhance sustainable water resources management and development. Firstly, the already existing data material needs to be made more accessible in a format useful in relation to water resources management i.e. compiled on a catchment basis. Also, the initiations of further data collection efforts both with respect to hydrological and water resources system data are necessary. Sustainability assessment exercises such as conducted in this study might help to highlight and direct the limited financial resources towards the most urgent needs. In this case study, the most significant barrier was found to be information concerning water demand, both historical data and future predictions. Also more hydrological data would have been useful, but it was found that even with the available data material it was possible to make a reasonable assessment of the available water resources.

Chapter 8

DISCUSSION AND CONCLUSION

The objective of this study was to develop a generic methodology for assessing the degree of sustainability of water resources management and development options. The methodology was tested on two case studies from South Africa and Zimbabwe, respectively. In the case studies future scenarios encompassing both construction of large-scale reservoirs and introduction of demand management were considered.

Define sustainability in an operational form

Despite numerous efforts to develop methods for assessing sustainability, no common consensus on how best to do it has been reached yet. The outcome of most research efforts into the subject has been either 1) a list of indicators of sustainability, 2) a checklist that should be followed to ensure sustainability, or 3) one or more sustainability criteria for comparison of different options. Both indicators and checklists are case-specific, whereas criteria are generic, i.e. can be applied to a variety of case studies. As the objective of this study is to develop a generic method for assessing relative sustainability, the criteria option was selected. Based on a review of existing literature, a list of issues was drawn up defining the concepts of sustainable development and water resources system sustainability. Any criterion claiming to be a valid metric of sustainability should at least attempt to encompass needs, fairness (equity), future generations, and multi-objectivity. A tool for assessing the more specific water resources system sustainability should, in addition, also consider: ability to cope with changes, scale, risk and compliance to existing decision support systems. A range of existing and proposed sustainability criteria was reviewed and valued against how well they take into account the issues highlighted above.

Based on this review a combination of two existing criteria, the fairness criterion by Matheson et al. (1997) and a criterion based on measures of reliability, resilience and vulnerability as defined by Loucks (1997) was proposed as a balanced sustainability criterion. A modelling procedure was developed, enhancing estimation of R-R-V in terms of uniformity by introducing stochastic models for generation of long time series of monthly runoff. The problems of non-uniform estimation and the required record length are discussed later. The method by Loucks (1997) was modified to emphasis multiobjective risk analysis. Due to correlation between estimates of R-R-V this modification was found to be of limited practical significance. In fact, when analysing many scenarios it might lead to scores more close to each other. By coupling the R-R-V criterion with the inter-generational fairness criterion the problem of temporal evaluation of the R-R-V criterion was taken care of. However, the proposed criterion is a mean value over the adopted time steps, and cannot explicitly consider upward or downward trend of the sustainability criterion over time. Despite these shortcomings the new criterion and the associated modelling procedure are believed to be improvements of the existing methods. Future research efforts should further address the shortcomings listed above.

The criterion is based on a water driven approach for evaluation of water resources systems, , and, therefore, denoted rS_{WDC}, the relative sustainability assessed using a water driven criterion (WDC). As discussed later, the water driven approach was found to be inadequate for a sustainability assessment of an entire water resources system and an additional evaluation tool had to be introduced.

Estimation of reliability, resilience and vulnerability

When conducting an analysis of water resources systems with multiple objectives, it is important to consider if the adopted criteria are 1) defined uniformly in terms of increasing (or decreasing) water stress, and 2) independent of each other, i.e. they do not overlap. Despite being widely used in the analysis of water resources systems, though mostly in the scientific literature, few practical guidelines for the to appropriate selection and estimation of R-R-V exists. A series of system experiments was conducted to investigate the properties of uniformity and overlap of a range of proposed estimators of reliability, resilience and vulnerability. Using behaviour analysis, time series of historical and stochastically generated monthly runoff were routed through a storage reservoir with predetermined storage capacity and desired draft. From the observed failure periods, where the desired draft could not be met, estimates of R-R-V were obtained.

Uniformity: It was shown, in line with predictions made by other researchers, that estimates of resilience and vulnerability obtained from time series of historical length (considering annual or monthly runoff) lead to non-uniform behaviour of the estimates when the estimators are based on mean value or a specific fractile of the duration and deficit volume of the observed failure events. Using univariate ARMA(p,q) models to generate long time series of monthly runoff, and subsequently estimate resilience and vulnerability based on these generated time series, it was observed that uniform estimates were obtained, if 1000 years or more were used. It should be noted that this result should only be used as a rule of thumb, as it was based on a limited data material and a limited number of combinations of storage capacity and demand. A more thorough investigation should address the relation between the required record length and time series properties such as the coefficient of variation and the serial correlation coefficient of annual runoff and the ratio between demand and storage of reservoirs, as these properties are directly related to estimates of water resources system reliability, resilience and vulnerability.

Overlap: The existence of correlation between different objectives in a multi-objective decision-making problem introduces bias with respect to the favoured outcome and reduces the amount of information concerning system performance. The use of R-R-V to describe system performance is a multiobjective problem, which requires decisions concerning the relative importance of the three criteria. Using the product of R-R-V effectively reduces the multi-objective evaluation of system performance to a single-objective problem. However, the reason for using all three criteria is an underlying perception that they describe different aspects of system performance, thus assuming they are mutually independent. The maximum information concerning system behaviour is achieved where the correlation between the estimates of R-R-V is at its lowest. Using the specified univariate ARMA(p,q) models, 100 time series of monthly runoff were generated and routed through a reservoir with predefined storage capacity and desired draft, and sample estimates of R-R-V were obtained using all the proposed estimators. For each combination of estimators the degree of overlap was quantified as the correlation coefficient between the 100 pair of sample estimates. The inverse of resilience was used rather than resilience itself in this investigation in order to obtain a more linear relationship between estimates of R-R-V, making the correlation coefficient a more valid measure of overlap. It was found that when the estimates of resilience and vulnerability were obtained using the same type of summary statistic, such as mean value or maximum value, an almost complete overlap was observed, i.e. no extra information is obtained by moving from a single-objective to a multi-objective procedure. Using different combinations of summary statistics, a lower degree of overlap was achieved, however not solving the problem. In relation to the criterion by Loucks (1997), where sustainability was expressed as a product of R-R-V, this investigation did not yield any obvious answer as to which combination of estimators should be used to minimise overlap. Therefore, it was decided to use occurrence reliability, resilience estimated through mean value, as suggested by Hashimoto et al. (1982a), and vulnerability as the 0.9-th fractile of the observed events of deficit volume. A sensitivity analysis carried out in connection with the South African case study showed that in fact the choice of estimators had little practical significance concerning the final ranking of scenarios.

Modelling system

A modelling system was developed to support the estimation of the new water driven sustainability criterion, including the use of stochastic generated time series of monthly runoff with an extension of 1000 years for each planning period. An existing water resources planning tool for river basins, the MIKE BASIN model, was coupled with a number of modules for 1) prediction of water demand in each planning period 2) stochastic generation of time series of monthly runoff and, 3) estimation of R-R-V.

To enable simultaneous generation of monthly runoff at several sites within a MIKE BASIN model setup, the multivariate CARMA(p,q) model for modelling of annual runoff was combined with a non-parametric disaggregation method known as the method of fragments. The choice of a disaggregation approach rather than a more direct modelling of monthly runoff through a Markov model approach was based on two observations. First, a method disaggregating annual into monthly runoff preserves the characteristics of the annual runoff such as mean value, standard deviation and serial correlation, which are important when considering the probabilistic characteristics of water resources systems in regions with a high inter-annual variation of water availability such as Southern Africa. Secondly, rivers in arid and semi-arid areas are often ephemeral, implying that they frequently have no flow during the dry season. The frequent occurrence of zero flow is a problem for most conventional stochastic models as they specify a continuous distribution to non-zero flow events. The method of fragments is a non-parametric method that does not assume a distribution of monthly runoff but is based on the intra-annual pattern of occurrence of monthly runoff within the historical time series. The pattern observed within each year is defined by a set of twelve fragments, hence the name of the method. However, a number of flaws are evident when using a simple disaggregating method such as the method fragments. Firstly, the generated sequences of monthly runoff are constrained by the number of observed sets of fragments. All generated annual runoff events larger than the largest observed event will be disaggregated according to the same set of fragments belonging to the largest observed annual flow event. This will have a limited practical influence as no failure periods are recorded in years with ample water availability. Correspondingly, any generated annual runoff event lower than the lowest observed event will be disaggregated according to the set of fragments of the lowest observed event. This is a more serious misgiving, unless the lowest observed annual runoff event is zero. Secondly, the correlation between the last month of the previous year and the first months of the preceding year is not preserved by the method. However, by applying the method on the specified runoff schemes from each of the two case studies it was concluded that the statistical properties of both annual and monthly runoff could be satisfactorily reproduced with respect to all the specified validation criteria.

Non-water demand related factors

Through the project, and especially by analysing the case studies, it became increasingly clear that a water-driven analysis would represent only a limited dimension of a sustainability analysis, as certain issues, such as e.g. the loss of nature amenity or inundation of human settlement, could not be related to a specified value of one or more water resources system variables. Perhaps the use of a more sophisticated modelling system and more research on how to quantify the interaction between socio-economic and ecological systems and water resources system variables could alleviate the problem, but this was considered outside the scope of the study. To enable a more holistic evaluation of the different scenarios, the water driven criterion was coupled with a criterion derived from an initial environmental evaluation. An existing tool for conducting environmental impact assessments, the rapid impact assessment matrix (RIAM), was adopted and coupled with a compromise programming objective function for transforming the scores obtained into a single score between zero and one. This particular option was chosen based on its widespread popularity in the water resources literature and the fact that it has previously been used as the foundation of the reversibility sustainability criterion as reported by Fanai & Burn (1997) and review in section 3.5. This new criterion was denoted rS_{IEE}, the relative sustainability assessed through an initial environmental evaluation. In line with the philosophy of Loucks (1997) the final relative sustainability score rS was obtained through multiplication, ensuring a high total score if only both criteria have high scores. The scoring system developed for the RIAM method, striving towards making the analysis objective and transparent, was found applicable. Being an open-ended methodology, however, the choice of issues to be included is left for the analyst to decide, which might influence the final outcome of the analysis. As the method was introduced to supplement the water driven analysis only issues not taken into account by the water driven analysis should be included to avoid overlap between the two criteria. Furthermore, the value of the criterion depends on the choice of methodology for aggregating the information in the RIAM analysis into a single number. A method yielding a more distinct difference in the rS_{IEE} scores will have a higher influence on the final sustainability score for each scenario.

Case studies

The methodology outlined in the first part of the project was developed to be a generic tool, i.e. to be applicable for sustainability assessments over a wide range of water resources systems. For testing of the methodology two existing water resources systems were selected as case studies. Both case studies involve a number of future management options including supply as well as demand oriented solutions, and both systems are planned extended in the near future by construction of one or more large reservoirs.

Water driven criterion

For each case study the rS_{WDC} criterion was estimated using the outlined methodology, and in both case studies the water driven criterion rS_{WDC} was found to reflect the ratio between demand and storage capacity. The lower the demand and the higher the available storage capacity, the higher were the obtained scores. Thus, any measure aimed at matching water supply and demand will have a positive effect on the relative sustainability. In both case studies the reservoir options were found to have the most positive impact. The results of the sensitivity analysis of the Mgeni-Mkomazi case study indicated that the choice of estimators of R-R-V had little practical significance on the final ranking of the scenarios. This result further indicates that perhaps similar results could be obtained through a more simple and less time-consuming evaluation of water demand and availability at basin scale. The same sensitivity analysis also showed that the use of stochastic hydrology to enhance estimation of R-R-V had no practical influence on the obtained ranking, once again indicating that a more simple evaluation of demand and availability could be used as an alternative measure.

As water availability was kept at a constant level during all three time steps (same statistical properties of the CARMA(p,q) model) while the water demand increased, the water driven criterion showed a downward trend of relative sustainability in all scenarios. The aggregate water driven criterion rS as defined in Eq. (4.8) estimates the average over the three planning periods, thus the unidirectional behaviour of R-R-V as experienced in this study makes this criterion a valid measure of relative sustainability. However, if more advanced scenarios had been constructed, which implied a more varying behaviour of R-R-V from time period to time period, the criterion point to the scenario with the highest average score as the one with the highest degree of relative sustainability.

Initial environmental evaluation

The rS_{IEE} criterion was estimated for each scenario in each case study through the outcome of a RIAM investigation of impacts and their significance. The scoring system of the RIAM methodology was found to be easily applicable and providing a fairly objective evaluation. However, being an open-ended methodology the choice of which impacts to include was based on the perception of the analyst. Based on interviews, literature studies and visits to field sites a list of factors related to the various infrastructure projects, but no to the demand management programs, was compiled. Common for all reservoir projects was the indication of an overall negative effect. However, looking closer at the results it was found that the larger the reservoir is the more negative were the impacts imposed on the physical/chemical and biological/ecological systems, whereas the economic/operational benefits were increased with the size of the reservoir. The social/cultural impacts were generally considered more negative for larger reservoirs. This diversity of negative and positive impacts led to average scores for the different scenarios very close to each other. It should be noted that the WCD (2000) warns that the anticipated economic benefits for the local communities are often overestimated in the planning phase. The difference between the negative impacts on the natural system and the anticipated positive benefits in terms of socio-economic welfare highlighted by this criterion is in essence a reflection of the debate concerning large reservoirs, and the problems associated with an objective evaluation of the term sustainability.

Lessens learned (usefulness of method)

The final relative sustainability score was obtained as the product between the two criteria derived from the water driven criterion and the initial environmental evaluation, respectively. This method of aggregating the different criteria was found to be in line with the philosophy of Loucks (1997) that a high sustainability score can only be obtained, if both criteria show high scores. In both case studies the construction of reservoirs in areas where negative impacts could be minimized has the highest relative sustainability compared to the corresponding non-reservoir solutions. Due to the narrow range of scores obtained by the IEE analysis the final ranking of the scenarios is close to the ranking obtained by the water driven criterion, though some adjustment do occur. The fact that scenarios including reservoir construction have high rS_{WDC} and low rS_{IEE} scores, and vice versa for scenarios without reservoir construction, gave rise to the close final *rS* scores in the case studies. As before, this illustrates the complexity of the problem at hand and the need to consider trade-offs between positive and negative impacts. Considering the number of assumptions and uncertainties involved in a sustainability assessment as conducted in this study, it will be difficult for a decision-maker to base the final decision regarding whether or not to construct a reservoir on the obtained *rS* scores. A major benefit, however, derived from conducting a sustainability assessment is the fact that decision-makers would be forced to make a holistic consideration of the water resources system under scrutiny including identification of stakeholders, factors of importance and future development options. Furthermore, the comprehensive data collection efforts needed to apply the modelling tool will highlight gaps in the existing data collection network and procedures in towards optimal data collection. In this regard the exercise can be an important part of the necessary process of defining and facilitating a sustainable development route.

A number of issues identified through application of the methodology is believed to require further attention in future research on the subject.

1. The adopted methodology for assessing the non-water related impacts is a very simplified tool, and more sophisticated tools might be necessary for a more in-depth analysis of these factors.

2. The water driven and the non-water related impacts are weighted equally by the proposed methodology. This might not necessarily be the most appropriate choice and more research on how to involve the preferences of the involved stakeholders and decision-makers is clearly needed.

3. The formation of users, especially in the agricultural sector, was based mainly on geographical conditions. This might not be the most appropriate grouping of single users. Guidelines to assist in the formation of appropriate groups in water resources management are needed.

4. Predictions of water demand were based on simplified growth models based on historical development. Predictions of water demand 30 years into the future are associated with a high degree of uncertainty, stressing the need for the methodology to be a part of an adaptive management approach.

5. The availability of data was found to be a profound problem. The need for an appropriate hydrometric gauging network is often highlighted as being of importance to sustainable water resources systems. In the two case studies analysed in this study data related to the water resources system, such as water abstraction and anticipated future growth, were found to be very scarce.

6. The concept of R-R-V as system performance criteria might be too abstract for non-system engineers, hence not facilitating a participatory approach to water resources management.

Despite these shortcomings it is hoped that the research presented in this thesis will be a positive contribution towards a further understanding of the sustainability concept and to the continued development of useful methods for quantification of sustainability.

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APPENDIX A
UNIVARIATE TIME SERIES MODELLING

The surface water resources in Southern Africa are characterised by high variability of annual runoff (McMahon, 1979), strong intra-seasonal differences, with the bulk of the annual runoff volume occurring during the wet season and little or no runoff during the dry season, and the widespread existence of ephemeral and intermittent rivers. These conditions require special considerations when applying stochastic models of monthly runoff.

In the literature two approaches for stochastic modelling of monthly runoff exist 1) direct modelling of monthly runoff using the concept of the Markov model as described by, for example, Thomas & Fiering (1962) or 2) Modelling of annual runoff followed by disaggregation into monthly runoff (Valencia & Shaake, 1973). The use of the Thomas-Fiering model does not necessarily preserve the statistical properties of the historical annual time series when generated monthly runoff are added together. Based on this shortcoming disaggregation models were develop aimed at preserving the statistical properties of the historical time series both on monthly and annual level (Salas et al., 1980). According to Vogel & McMahon (1996) reservoirs can be characterised by either within-year or over-year behaviour. Reservoirs dominated by within-year behaviour tend to empty and refill within the same year, whereas over-year systems are controlled by the year to year variation of annual inflow and the water demand. Vogel & McMahon (1996) used the index m given as

$$m = \frac{1-\alpha}{C_V} \tag{A1}$$

where

α = water demand as a fraction of MAR
C_V = coefficient of variation of annual inflow

For $0 \leq m \leq 1$ the reservoir is dominated by over-year behaviour and for $m > 1$ dominated by within-year behaviour. As seen in Table 4.1, the coefficient of variation of annual runoff for rivers in the region is often larger than one, which indicates over-year behaviour even at low water demand. It is therefore considered important that the selected model is able to reproduce the statistical properties of the annual runoff as precise as possible. Hence, disaggregation models appear to be preferable.

Chebaane et al. (1995) presented a univariate model for generation of monthly runoff in ephemeral or intermittent rivers based on an extension of the Thomas-Fiering model. However, no extension to a multivariate model, as needed in this study, exists. Srikanthan & McMahon (1980) tested different approaches for generating monthly runoff in ephemeral rivers in Australia. They concluded that the non-parametric Method of Fragments

(MF), presented by Svanidze (1980) was the model performing best in terms of reproducing the statistics of the historical time series. They also concluded that the Markov model by Thomas-Fiering, using a three parameter log-normal distribution, was unable to reproduce the observed percentage of zero flows from the historical time series, hence, not applicable under the hydrological circumstances prevailing in the Southern African region. Klemes et al. (1981) applied the MF for modelling of emphemeral rivers in Australia. Phien & Vithana (1983) compared different methods for generating monthly runoff, including the Thomas-Fiering model, and found the MF to be preferable with respect to the ability to reproduce the statistical characteristics of the historical time series. Takeuchi et al. (1998) recommended the MF for analysis of reservoir systems. Basson et al. (1994) applied the MF for analysis of large-scale reservoir systems in South Africa and proposed a procedure for a multivariate extension of the univariate model. Based on these recommendations, the MF was adopted for testing and subsequent implementation in the MIKE BASIN model. The MF is a disaggregation model, i.e. first annual runoff volume is generated and afterwards the MF disaggregates this volume into monthly runoff according to a fixed pattern. A six-step procedure for stochastic streamflow generation was presented by Stedinger & Taylor (1982):

1. Obtain streamflow data.
2. Select models to describe marginal probability distributions of flow in different seasons and estimate the models' parameters.
3. Select appropriate model for the spatial and temporal dependence of the streamflow.
4. Verify the computer implementation of the model performs as specified.
5. Validate the model for water resources system simulation.
6. Use the model.

The procedure by Stedinger & Taylor (1982) will structure the following sections focusing on modelling of annual runoff and use of the MF to generate monthly runoff, respectively. The data material was presented in section 4.4.

A1 ANNUAL RUNOFF MODELLING

The historical time series of annual runoff is regarded as a realisation of a basic stochastic process so $q(t)$, $t = 1,\ldots, T$ is assumed to be only one realisation of the infinite number of possible realisation of the process $\{Q(t), t = 1,\ldots, T\}$. The following analysis of annual runoff will focus on the components: definition of the hydrological year, investigations of trends and shifts, marginal distribution of annual runoff, selection of appropriate model for the temporal persistence of the annual flow, verification and validation.

Definition of the hydrological year

The magnitude and the statistical properties of annual runoff vary according to the definition of the hydrological year. As further explained later, the MF does not preserve the correlation between from the last month in the year *t* and the first month in the following year *t+1*. Therefore, as recommended by McMahon & Mein (1986), the hydrological year should be defined according to where the connection between successive years are weakest, i.e. minimum correlation. Table 4.2 shows the correlation between successive months based on historical time series. As can be seen, the minimum correlation in most cases is close to zero at the time just before the onset of the rainy season, October, November or December. In fact, for all rivers the minimum correlation was observed between the month following the month with the lowest runoff and the next months. The hydrological year has been defined individually for each of the eight rivers, see Table A1.

Table A1: Hydrological year defined according to minimum month to month lag one correlation

	A5H004	R2H008	U1H005	U2H007	C18	C41	D28	E2
Hydrological year	Nov-Oct	Oct-Sep	Dec-Nov	Dec-Jan	Nov-Oct	Nov-Oct	Dec-Nov	Dec-Nov

Trends and shifts

In order to make statistical interference of the stochastic process assumed to have generated the historical time series, the process should be, at least, a second order weakly stationary process and ergodic (Hipel & McLeod, 1994). If the time series of the annual runoff is described by a stochastic process, then a second order weakly stochastic process is defined as

$$\begin{aligned} &E\{Q(1)\} = \ldots = E\{Q(t)\} = \ldots E\{Q(T)\} = \mu \\ &V\{Q(1)\} = \ldots = V\{Q(t)\} = \ldots V\{Q(T)\} = \sigma^2 \\ &Cov\{Q(1), Q(1+k)\} = \ldots Cov\{Q(t), Q(t+k)\} = \ldots = Cov\{Q(T-k), Q(T)\} \end{aligned} \tag{A2}$$

The existence of trends and shifts in the historical time series jeopardise the stationary assumption, thus on identification they must be removed. Visual inspection of the historical time series did not reveal any shifts. The presence of trend in the time series was investigated using the Mann-Kendall test (Hipel & McLeod, 1994) and a test based on linear regression. The test based on the linear regression assumes that the time series of annual runoff can be modelled using a linear trend model

$$q(t) = b_0 + b_1 t + \varepsilon \tag{A3}$$

where

b_i = model parameters

ε = stochastic component NID(0,1)

The test statistics for each of the two tests are shown in Table A2.

Table A2: Mann-Kendal and linear regression test statistics for the hypothesis H_0: no trend.

Station	Mann-Kendall	Linear regression
A5H004	-2.60	-3.25
R2H008	0.71	-0.93
U1H005	-0.26	-0.06
U2H007	0.10	0.36
C18	-1.38	-0.88
C41	-1.94	-1.01
D28	-2.19	-1.35
E2	-1.44	-0.45

With the exception of gauging station A5H004 the no-trend hypothesis is accepted on a 5% significance level for both tests at all station. The tests indicate a tendency towards a decrease in the annual runoff at all gauging stations except U2H007. But in the light of the extreme floods observed in the region during the flooding seasons in the year 2000 and 2001, which were not included in these time series, the no trend hypothesis was accepted for all time series.

Marginal probability distribution

According to Basson et al. (1994) the choice of appropriate marginal distribution of annual runoff from arid and semi-arid catchments has received far less attention in literature than required by its importance. Three distributions, which have found widespread use for modelling annual runoff have been tested: 2-parameter log-normal distribution LN2, 3-parameter log-normal distribution LN3 and normal distribution of Box-Cox transformed annual runoff.

LN2: A random variable Q is distributed according to the LN2 distribution if

$$X = \ln(Q) \tag{A4}$$

is normally distributed. The LN2 distribution is often used to model annual runoff in more humid area.

LN3: This distribution is an extension of the LN2 distribution, subtracting a lower bound parameter from the random variable *Q* before taking the logarithm

$$X = \ln(Q - \xi) \tag{A5}$$

where Y is normally distributed. According to Stedinger et al. (1993) the lower bound parameter can be estimated as

$$\hat{\xi} = \frac{q(1)q(n) - q_{median}^2}{q(1) + q(n) - 2q_{median}} \tag{A6}$$

where *q(1)* and *q(n)* are the biggest and smallest value of the time series, respectively, and q_{median} is the sample median.

Afterwards the sample mean and standard deviation of $x(t) = \ln\left(q(t) - \hat{\xi}\right)$ are calculated. This method is simple and better than the method of moments and competitive with the maximum likelihood estimation (Stedinger et al., 1993).

Box-Cox transformation: The Box-Cox transformation of data to obtain a normally distributed sample was recommended by Hipel & McLeod (1994) and defined as

$$x(t) = \begin{cases} \dfrac{q^{\lambda}(t) - 1}{\lambda}, & \lambda \neq 0 \\ \ln(q(t)), & \lambda = 0 \end{cases} \tag{A7}$$

where *x(t)* is normally distributed. The transformation parameter λ is estimated using an iterative procedure changing λ until the coefficient of skew of *x(t)* is equal to zero. The estimated λ parameters are shown in Table A3.

All three models attempts to transform observed data to a series that can be described by the normal distribution. Stedinger et al. (1993) recommended the probability plot correlation coefficient (PPCC) test to examine how well observations are described by the normal distribution. The test is based on the correlation coefficient between ordered observed data q(i) and the corresponding quantile of the fitted model $w(i) = F^{-1}(1-p_i)$, where p_i is the plotting position for the *i*-th observation. The test statistic *r* is given as

$$r = \frac{\sum_i \left(x(i) - \bar{x}\right)\left(w(i) - \bar{w}\right)}{\left[\sum_i \left(x(i) - \bar{x}\right)^2 \sum_i \left(w(i) - w\right)^2\right]^{\frac{1}{2}}} \tag{A8}$$

The closer r is to one the better the data are described by the normal distribution. The PPCC test statistics was estimated for each of the rivers and for each of the three methods, see Table A3. Both the LN3 distribution and the two-parameter normal distribution fitted to the Box-Cox transformed series of annual runoff are acceptable on a 5% significance level (Stedinger et al., 1993).

Table A3: PPCC test statistics for marginal probability distribution of annual runoff. In last column ( ) indicates Box-Cox transformation parameter.

Station	LN2	LN3	Box-Cox
A5H004	0.9888	0.9873	0.9894 (0.0667)
R2H008	0.9550	0.9895	0.9952 (0.3813)
U1H005	0.9864	0.9878	0.9881 (0.1174)
U2H007	0.9821	0.9863	0.9858 (0.1912)
C18	0.9129	0.9888	0.9927 (0.3164)
C41	0.9851	0.9929	0.9928 (0.2869)
D28	0.9550	0.9919	0.9966 (0.4002)
E2	0.8839	0.9918	0.9947 (0.3281)

Selection of model for temporal persistence

Temporal persistence is an important characteristic of hydrological time series quantifying the effect of one event on the following event(s). To quantify the persistence in each of the transformed time series of annual runoff, the autocorrelation function was estimated as

$$\rho(k) = \frac{\sum_{t=1}^{N-k}\left(x(t)-\bar{x}\right)\left(x(t+k)-\bar{y}\right)}{\sum_{t=1}^{N}\left(x(t)-\bar{x}\right)^2} \tag{A9}$$

where k is the lag time, $k = 0,\ldots,T/4$. To model the persistence in the time series of transformed annual runoff, the linear ARMA(p,q) models are adopted.

$$x(t)-\mu = \sum_{i=1}^{p}\left(x(t-i)-\mu\right)+\varepsilon(t)-\sum_{j=1}^{q}\theta(j)\varepsilon(t-j) \tag{A10}$$

where

$x(t)$ = transformed annual runoff at time t

μ = mean value of $x(t)$

ϕ = autoregressive parameter

θ = moving average parameter

$\varepsilon(t)$ = independent identically distributed normally random variable with mean zero.

The variance of the $\varepsilon(t)$ element in Eq. (A10) is denoted σ_ε^2.

Five ARMA(p,q) models were compared to find the most suitable model for each of the transformed time series: AR(0), AR(1), AR(2), AR(3) and ARMA(1,1). The choice of model was based on the Akaike information criterion (AIC) defined as

$$AIC(p,q) = N\ln\left(\hat{\sigma}_\varepsilon^2\right) + 2(p+q) \tag{A11}$$

The model with the minimum value of AIC should be preferred. The parameters of the AR(p) models are estimated using the Yule-Walker equations

$$\begin{aligned} &\rho(1) = \varphi_1 + \phi_2\rho(1) + \cdots + \varphi_p\phi(p-1) \\ &\rho(2) = \varphi_1\rho(1) + \phi_2 + \cdots + \varphi_p\rho(p-2) \\ &\vdots \qquad\quad \vdots \qquad \vdots \qquad\qquad \vdots \\ &\rho(\mathrm{p}) = \varphi_1\rho(p-1) + \varphi_2\rho(p-2) + \cdots\varphi_p \end{aligned} \tag{A12}$$

and the ARMA(1,1) model by combining the two equations

$$\begin{aligned} &\rho(1) = \frac{(1-\varphi_1\theta_1)(\varphi_1-\theta_1)}{1+\theta_1^2-2\varphi_1\theta_1} \\ &\varphi_1 = \frac{\rho(2)}{\rho(1)} \end{aligned} \tag{A13}$$

The chosen models are shown in Table A4.

Table A4: ARMA(p,q) models selected using the AIC criterion.

Station	Model
A5H004	AR(3)
R2H008	AR(0)
U1H005	AR(0)
U2H007	AR(2)
C18	AR(3)
C41	AR(0)
D28	AR(1)
E2	ARMA(1,1)

The identified models only take short-time persistence into account. Previous investigations of temporal persistence of hydrological time series from the South African region indicate that also long-term persistence may be present. Makarau & Jury (1997) studied Zimbabwean rainfall series and identified the possible existence of an 18 year cycle. Sene et al. (1998), however, did not find supportive evidence of this cycle in an investigation of runoff and rainfall in Lesotho. Much research has been devoted to investigations of long-term persistence in hydrological time series. However, for practical use of stochastic runoff simulation in water resources system analysis, Klemes et al. (1981) concluded that from a practical point of view the importance of long-term persistence models, as compared to short-term persistence models, are marginal compared to the socio-economic and hydrological uncertainties of the considered system. Thus, short-term persistence models were recommended.

Long-term persistence: The effect of long-term persistence in hydrological time series is often quantified by the Hurst coefficient H (Hipel & McLeod, 1994), which is defined as follows. Consider a time series of annual runoff $q(t)$, $t = 1,\dots, T$ with sample mean $\bar{q}$ and sample variance s_q^2, then the adjusted partial sums are

$$S(k) = \sum_{t=1}^{k} q(t) - k\bar{q}, \quad k = 1,\dots,T \tag{A14}$$

and the range is given as

$$R(T) = \max_{1\le k\le T}\left(S(T)\right) - \min_{1\le k\le T}\left(S(k)\right) \tag{A15}$$

The Hurst coefficient can now be estimated as

$$H = \frac{\log\left(R(T) / s_q \right)}{\log\left(T/2 \right)} \tag{A16}$$

Average values of the Hurst coefficient were given by McMahon (1979) as 0.72±0.08 for geophysical time series in general and 0.68±0.08 for annual hydrological time series from arid areas. Even though long-term persistence models were not considered in this study, the Hurst coefficient was calculated for the annual runoff for each of the eight river, see Table A5.

Table A5: The Hurst coefficient estimated using Eq. (A16).

Station	H
A5H004	0.83
R2H008	0.64
U1H005	0.61
U2H007	0.64
C18	0.78
C41	0.79
D28	0.69
E2	0.75

In general the estimated values of *H* are within the ranges given by McMahon (1979). However, there appears to be a difference between the runoff series from South Africa and Zimbabwe. With the exception of A5H004 (which is the most northern of the considered South African catchments) the South African time series have lower *H* values than the Zimbabwean time series. Due to the limited number of time series in this study, this should not be taken as an evidence but rather as an indication that regional differences may exist with regards to long-term persistence and cyclic behaviour.

A2 MONTHLY RUNOFF

The series of stochastic monthly runoff was obtained using the MF to disaggregate stochastic generated annual runoff into monthly runoff according to the within year pattern observed in the historical time series. To characterise the within year flow pattern, monthly fragments *f(t,j)* are calculated for each month *j* in each hydrological year *t* as

$$f(t,i) = \frac{q_m^h(t,j)}{\sum_{i=1}^{12} q_m^h(t,j)} \tag{A17}$$

where $q_m^h(t,j)$ is the historical runoff in the *j*-th month ($j = 1,\ldots,12$) in the hydrological year *t*. Svanidze (1980) suggested that the generated annual flow should be disaggregated according to a randomly selected set of fragments *f(•,j)* as

$$q_m^s(t,j) = q^s(t)\, f(\bullet, j), \qquad j = 1,\ldots,12 \tag{A18}$$

where $q_m^s(t,i)$ is the stochastic runoff in the *j*-th month in year *t*, and $q^s(t)$ is the stochastic annual runoff in year *t*. However, Srikanthan & McMahon (1980) found that the model performed better, if a set of appropriate fragments were selected for each generated annual runoff as the set of fragments originating from the historical annual runoff closest to the generated. Do to extension of the univariate disaggregation method into a multivariate model, the later approach was adopted in this study.

Three conceptual problems exist when applying this method 1) no preservation of correlation between last month in year t and first month in year $t+1$, 2) generated annual runoff bigger than historically recorded annual runoff will always be disaggregated according to the set of fragments originating from the year with the biggest historical runoff, and 3) runoff smaller than historically recorded annual runoff will always be disaggregated according to the set of fragments from the year with smallest annual runoff. The effect of the first misgiving is minimised by selecting the hydrological year so that the minimum observed month to month correlation is observed in the split between two years. Basson et al. (1994) argues that the two following misgivings will have little effect on the behaviour of a water resources system, and that generated annual runoff outside the historical observations does not occur frequently.

A3 MODEL VERIFICATION AND VALIDATION

According to Stedinger & Taylor (1982) the testing of stochastic streamflow models estimated from historical time series should include verification and validation. Verification is a demonstration that the estimated model has been implemented correctly and that the model is able to generate time series of streamflow possessing the specified statistical properties estimated from the historical time series. Validation of streamflow models is a demonstration that the model can reproduce time series characteristics not explicitly specified by the stochastic models through parameters estimated from the historical time series. Stedinger & Taylor (1982) considered, among other, ARMA(p,q) models for modelling time series of annual flow. For model verification they used mean, standard deviation and lag-one autocorrelation coefficient of annual streamflow, and for model validation reservoir storage capacity required for a failure free delivery of a specified target draft (69% of MAR) estimated using the SPA algorithm (Loucks et al., 1981). Srikanthan & McMahon (1980) investigated the performance of monthly streamflow models and used additional criteria of coefficient of skew, percentage of zero flows, extreme events, low flow sums and standard deviation of low flow sums.

In this study a verification-validation approach was adopted for testing the estimated streamflow models. The method of fragment requires estimation of an annual streamflow model through direct parameter estimation. The monthly flows, however, are not directly estimated and thus considered part of the model validation. Therefore, the following criteria are adopted for verification and validation:

1. Verification: mean, standard deviation and lag-one autocorrelation coefficient of annual flow.
2. Validation: mean, standard deviation and lag-one correlation of monthly flow and required storage capacity.

Both the verification and the validation are carried out as a simulation study. For each considered historical time series of length *n* years a stochastic streamflow model was estimated as described above. Each streamflow model was used to generate N = 5000 synthetic time series of historical length. For each of these generated time series the statistical properties included in the model verification and validation were estimated. Finally, the verification and validation were evaluated through the use of Box plots comparing the distribution of the statistical properties from the N = 5000 synthetic time series with the corresponding statistical properties of the historical time series. The simulation procedure is outlined in Fig. A1. The Box plot is based on the *5-number summary* (Hipel & McLeod, 1994) of the N = 5000 generated values, i.e. the smallest and the largest values, the median and the 0.25 and 0.75 quantile.

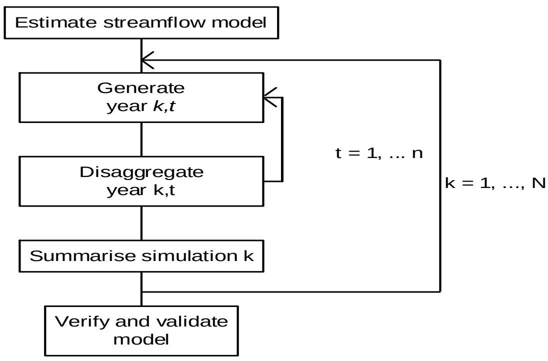

Fig A1: Flow diagram for simulation study.

Streamflow generation

Generation of time series of synthetic runoff requires a random number generator for generation of realisations of the independent $N(0,\sigma_\varepsilon^2)$ distributed element $\varepsilon(t)$ in Eq. (A10). For each time step a random number generator is used to generate a realisation *u* of a random variable distributed uniformly over the interval zero to one. The realisation *u* corresponds to the *u*-the fractile in the cdf of the ε element in Eq. (A10) as

$$F(\varepsilon_p) = u \Leftrightarrow \varepsilon_p = F^{-1}(u) \tag{A19}$$

where $F(\bullet)$ is the 2-parameter standard normal distribution. The transformed annual runoff at time *t*, *x(t)*, is calculated by inserting $\varepsilon_u = \varepsilon(t)$ into Eg. (A11). Finally, the inverse Box-Cox transformation was applied to get from the normally distributed *x(t)* to the actual annual runoff event *q(t)*. It is possible that the generated value of *x(t)* is below the lower limit of the Box-Cox transformation $-1/\lambda$, which is found by setting q = 0 in Eq. (A7). If the generated value of *x(t)* falls below this lower limit, then *x(t)* was set equal to the lower limit, hence giving a zero runoff event. A similar approach was used by Basson et al. (1994) when using the LN3 distribution for generation of annual runoff. The process is repeated for as many time steps as necessary. Computer routines for generation of realisations of a uniformly distributed random variable and transformation of these into normally distributed realisations are adopted from Press et al. (1992). To alleviate the influence of initial values the first 100 generated values were discharged before the simulation of each time series. This also helps to minimise the influence from a generated time series on the following generated time series.

Model verification

Model verification compares mean, standard deviation and lag-one autocorrelation of the generated time series of annual runoff to similar statistics estimated from the historical time series of annual runoff. The results of the verification, shown in Fig. A2 - A4 where the simulated values of mean, standard deviation and lag-one correlation, respectively, have been normalised by the corresponding historical values for comparative purposes. From Fig. A2 it can be seen that each of the historical estimates of MAR is close to the corresponding median of the N = 5000 generated values of MAR. For all time series the historical estimates of MAR are within the interval defined by the 0.25 – 0.75 quantiles of the generated samples of MAR. Similar results were obtained for the standard deviation of annual runoff as shown in Fig. A3.

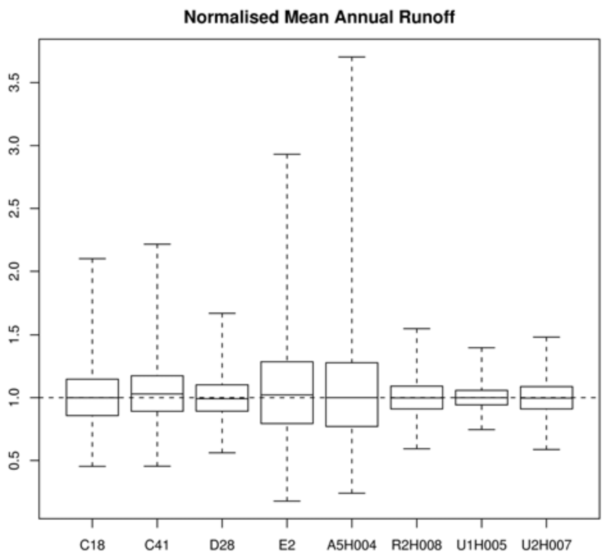

Fig. A2 Generated MAR normalised with the corresponding estimate obtained from the historical data.

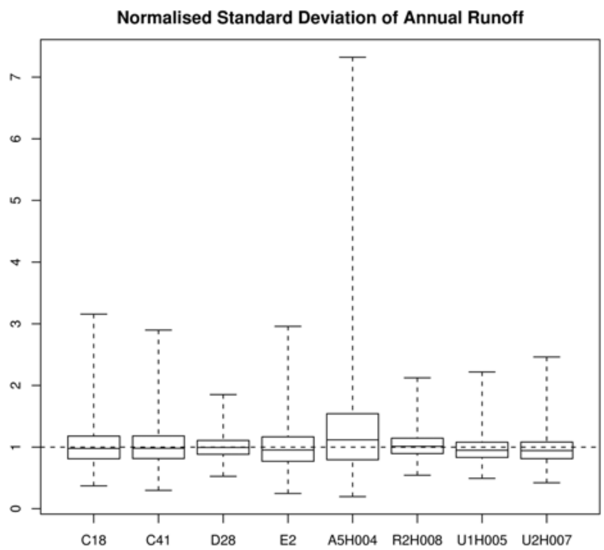

Fig. A3 Generated standard deviation of annual runoff normalised with the corresponding estimate obtained from the historical data.

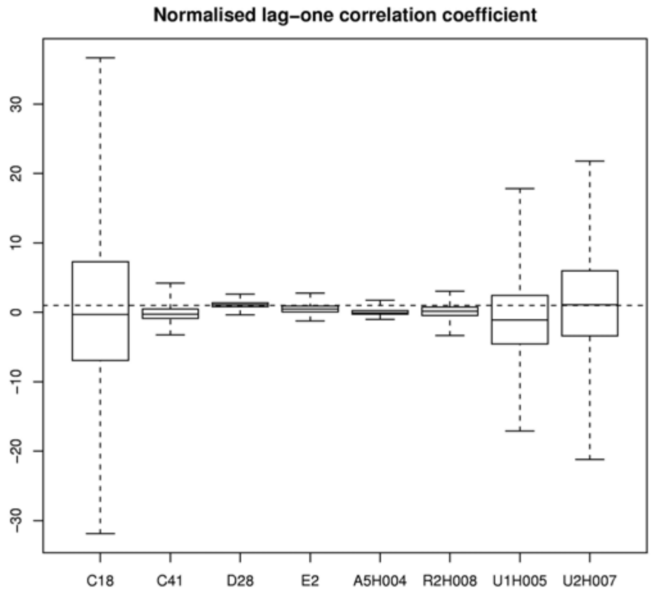

Fig. A4 Generated lag-one autocorrelation normalised with the corresponding estimate obtained from the historical data.

Less satisfactory results were obtained for the lag-one year autocorrelation. In the case of A5H004, R2H008, C41 and E2 the historical values of the lag-one autocorrelation coefficient falls outside the 25%-75% interval. The large fluctuations of the lag-one autocorrelation for C18, U1H005 and U2H007 are due to the low numerical value of the historical estimates used for the normalisation.

Model validation

The required reservoir storage capacity for providing a failure free delivery of a target draft *D* of 80% of MAR was estimated for each generated time series of monthly runoff and using the SPA algorithm as described by Loucks et al. (1981).

$$w(t) = \max\{0, w(t-1) + d - q(t)\} \tag{A20}$$

where

w(t) = required storage capacity at the beginning of time step *t*
q(t) = inflow to reservoir during period *t*
d = target draft
t = 0,1,…, *T**12

The required storage volumes of the generated time series are compared to the corresponding storage volume observed in the historical time series. All storage volumes are normalised with MAF, and the results displayed in Fig. A5.

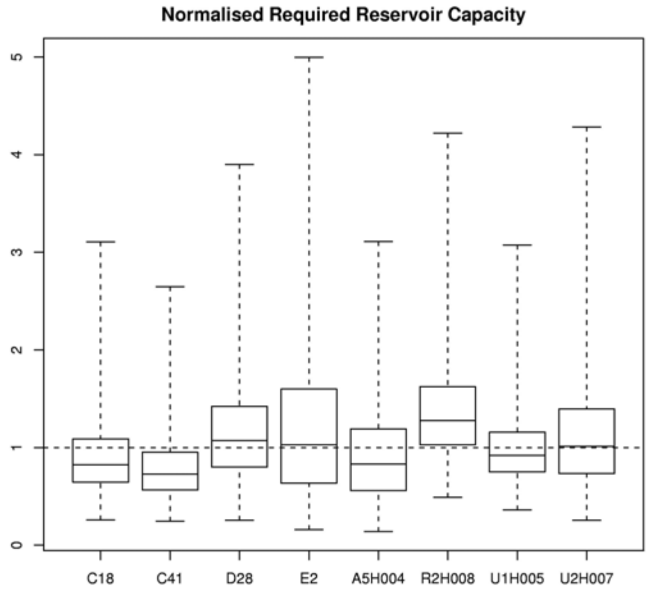

Fig. A5 Generated maximum deficit volume normalised with the corresponding estimate obtained from the historical data.

The mean and standard deviation were estimated for monthly runoff for each generated time series and compared to the corresponding statistics of the historical time series, see Fig. A6 and Fig. A7. The reproduced monthly mean runoff corresponds satisfactorily with the observed mean monthly runoff for all months and for all stations, except for gauging station U2H007. This particular gauging station is located in the KwaZulu-Natal province of South Africa and, in the past, has been impacted by tropical hurricanes as reported by, for example, Kjeldsen et al. (in press). Gauging station U1H005 is also situated in the KwaZulu-Natal province, but the bulk of the catchment is situated outside the zone impacted by the hurricanes (Kjeldsen et al., in press.). These extreme extreme events may create sets of fragments, as explained in section 4.4, with an unusually high percentage of annual runoff occurring during the flooding season (September-April). A similar pattern is observed for the standard deviation of the annual runoff. Again, all gauging stations are well described except for U2H007 during the flooding season.

The lag-one autocorrelation describing the month to month correlation coefficient was estimated, see Fig.A8. The eight gauging stations are located in different climatic zones, which is evident from the observed month-to-month correlation. The three South African gauging stations R2H008, U1H005 and U2H007 are located in the more humid South Eastern part of the country, whereas A5H004 and the four Zimbabwean gauging stations (C18, C41, D28 and E2) are all located in dry areas (MAP < 800 mm) with a strong intra-annual variation in runoff. In general, the month to month correlation is reproduced satisfactorily, especially for the gauging stations in the dry areas. Again, the MF has difficulties reproducing the observed statistics at gauging station U2H007 during the flooding season. As mentioned previously, the MF does not reproduce the correlation between the two months separating the hydrological year, which clearly is evident from Fig. A8.

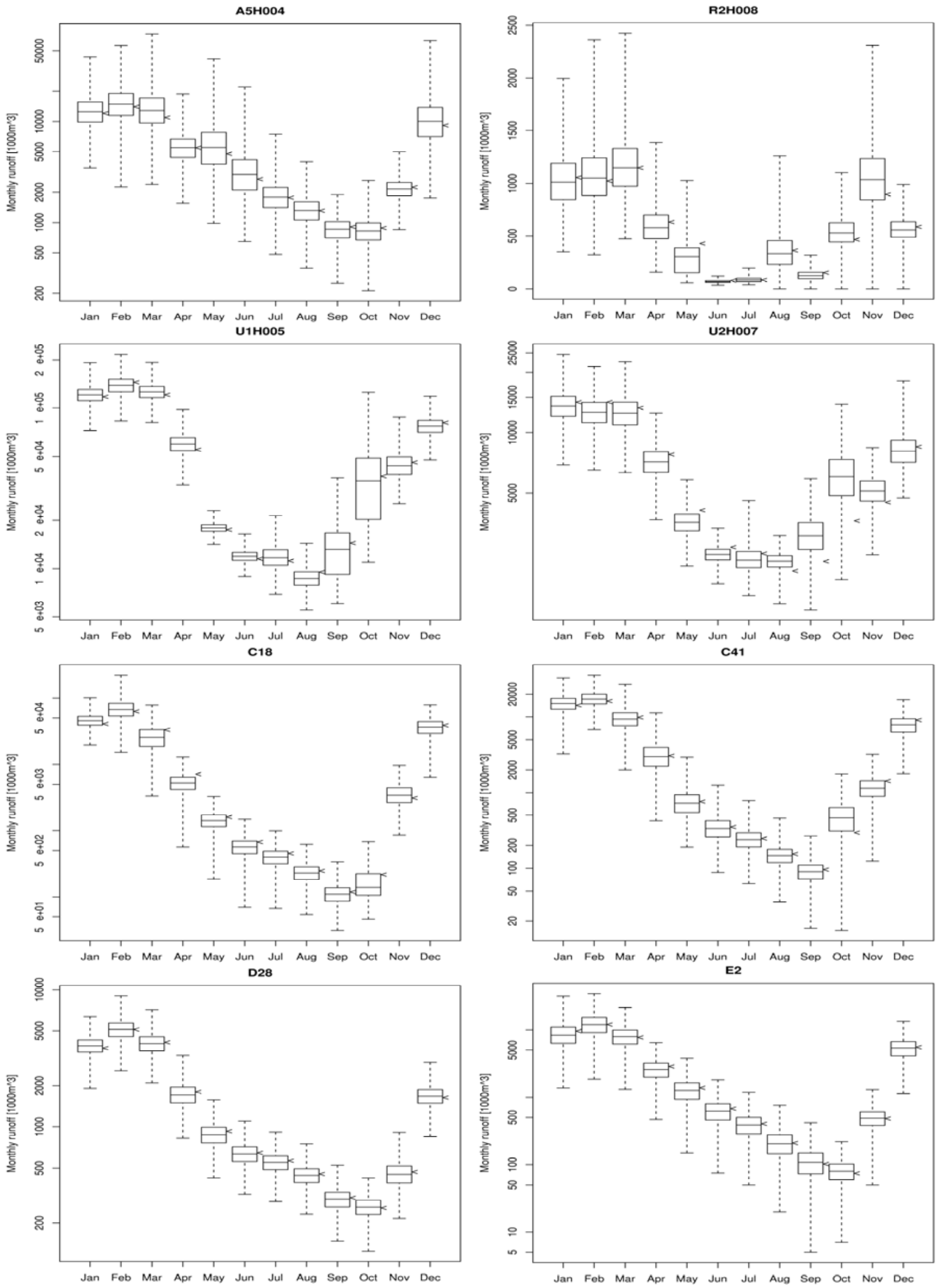

Fig. A6 Comparison of observed and generated mean monthly runoff for each of the eight gauging stations. Estimates obtained from historical time series are indicated with an "<".

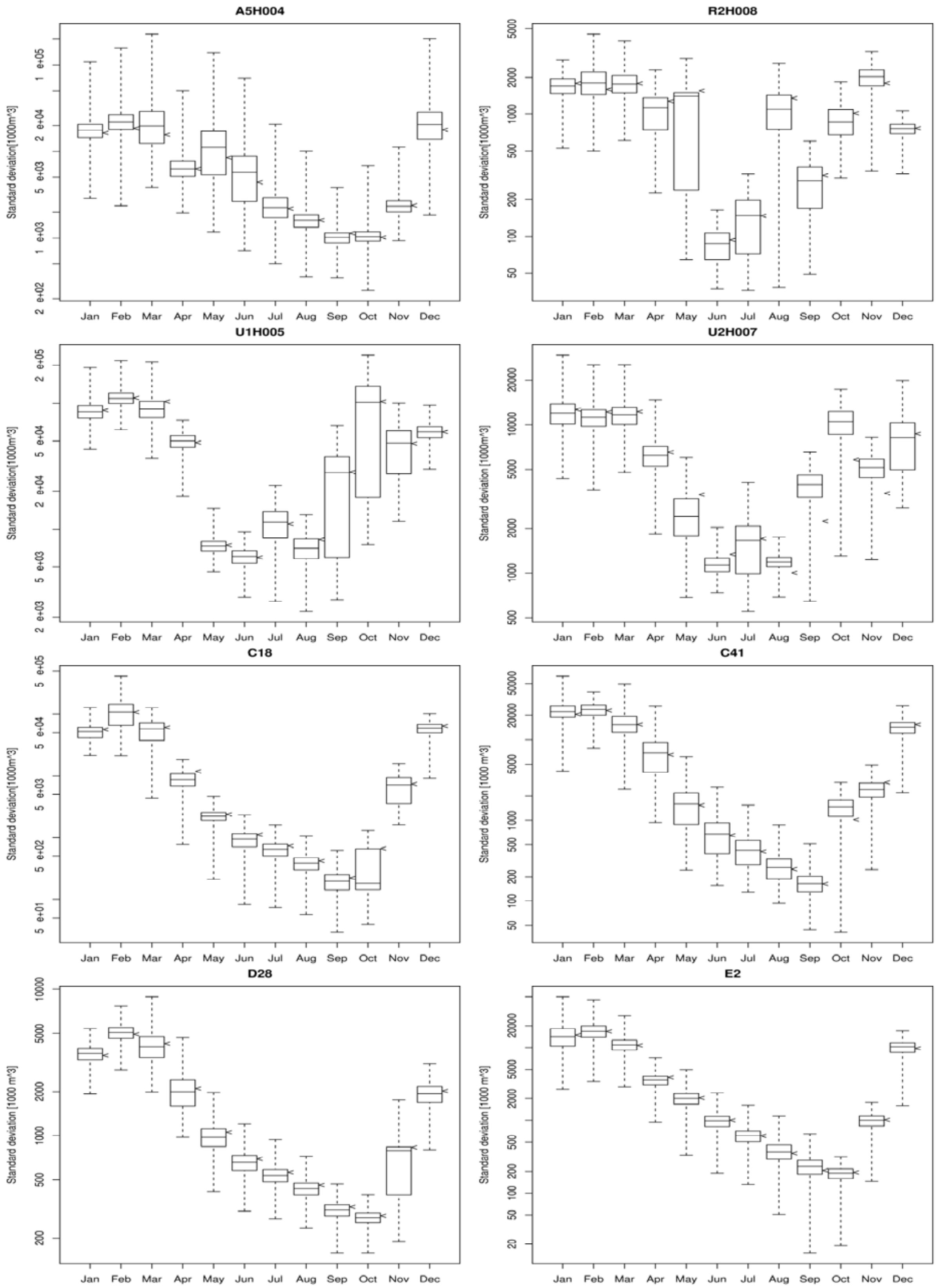

Fig. A7 Comparison of observed and generated standard deviation of monthly runoff for each of the eight gauging stations. Estimates obtained from historical time series are indicated with an “<”.

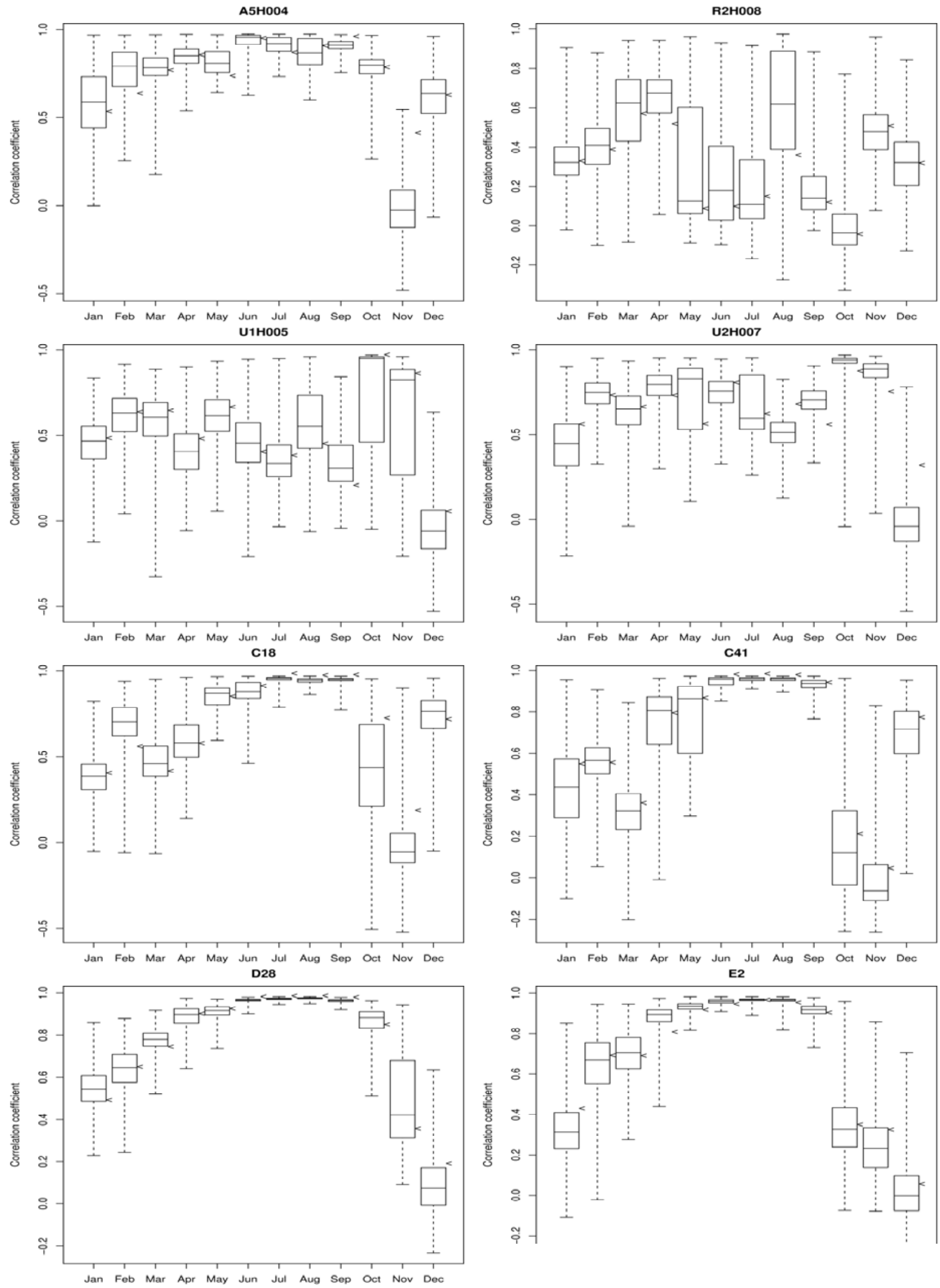

Fig. A8 Comparison of observed and generated correlation between runoff in successive months for each of the eight gauging stations. Estimates obtained from historical time series are indicated with an “<”.

In general, the outcome of the verification and validation exercise has proven the MF to be a useful tool for the generation of synthetic time series of monthly runoff, especially in area with a high intra-annual variability of monthly runoff. The MF has difficulties reproducing the observed statistics from a gauging station U2H007 during the flooding season. This is believed to due to the existence of extreme, extreme events creating sets of fragments with an unusually high percentage of annual runoff taking place during the flooding season.

APPENDIX B

MIKE BASIN RESERVOIR OPTION

MIKE BASIN has two options for specifying HVA relationships for reservoirs, either through table data or through formula data. Large reservoirs often have specified HVA relationships in tabular form, which can be entered directly into MIKE BASIN. Smaller reservoirs like farm dams often do not have available HVA curves. This appendix presents a procedure for estimating the parameters in the formula option from general HVA relationships.

The formula option in MIKE BASIN calculates the volume and free surface area of the reservoir as a function of the water level given as

$$\begin{aligned} V(h) &= a(h-b)^{c} \\ A(h) &= d(h-e)^{f} \end{aligned} \qquad \text{(B1)}$$

where

h = water level [m]
V = volume [m^3]
A = Surface area [m^2]
a, b, c, d, e, f = parameters to be estimated

First, the equations in (B1) are rearranged and it is assumed that b = 0 and e = 0.

$$\begin{aligned} h &= \left(\frac{V}{a}\right)^{\frac{1}{c}} \\ h &= \left(\frac{A}{d}\right)^{\frac{1}{f}} \end{aligned} \qquad \text{(B2)}$$

From (B2) the following general relationship can be found

$$A = \frac{d}{a^{\frac{f}{c}}} V^{\frac{f}{c}} \qquad \text{(B3)}$$

Maaren & Moolman (1985) identified the following relationship for farm dams in South Africa

$$A = 7.2V^{0.7} \tag{B4}$$

where

A= surface area [m^2]
V=storage [m^3]

If (B4) is compared to (B3) it can be seen that

$$\frac{f}{c} = \frac{3}{4} \tag{B5}$$

and

$$\frac{d}{a^{\frac{f}{c}}} = 7.2 \tag{B6}$$

Eq. (B1) is applied to the case where a dam is full, i.e.

$$a = \frac{V_{max}}{h_{max}^4} \tag{B7}$$

The height of the farm dams is assumed equal to 10 m. From (B5), (B6) and (B7) the parameters needed by MIKE BASIN to calculate the HVA relationship can be calculated from knowledge of the total capacity of the dam only.

Similarly, Mitchell (1982) found the following relationship between area and volume for large reservoirs in Zimbabwe

$$A = 0.53V^{0.66} \tag{B8}$$

where

A= surface area [km^2]
V=storage [10^6 m^3]

Adopting the same approach, the following relationships can be established

$$\begin{aligned} \frac{f}{c} &= \frac{2}{3} \\ a &= \frac{V_{\max}}{h_{\max}^{3}} \\ d &= 53.0 a^{\frac{f}{c}} \end{aligned} \tag{B9}$$

As before, the height of the farm dams are assumed equal to 10 m and from (B7), (B9) and (B10) the parameters needed by MIKE BASIN to calculate the HVA relationship for Zimbabwean farm dams can be calculated from knowledge of the total capacity of the dam only. It should be noted that Mitchell's area-volume relationship (B8) was derived for large reservoirs and not for smaller farm dams as in this study, but no general relationship for farm dams has been estimated in Zimbabwe.

If both volume and area are know at maximum height (full supply level), then the parameters a and d can be estimated as

$$\begin{aligned} a &= \frac{V_{\max}}{h_{\max}^{c}} \\ d &= \frac{A_{\max}}{h_{\max}^{f}} \end{aligned} \tag{B10}$$

Still, assumptions about the general shape of the HVA relationship have to be made through choices of parameters *c* and *f*, respectively.

APPENDIX C
RESULTS FOR THE MGENI-MKOMAZI SYSTEM

	Scenario 1			Scenario 2			Scenario 3			Scenario 4			Scenario 5			Scenario 6		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
Midmar																		
Rel	0.89	0.73	0.58	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.97	0.87	1.00	1.00	1.00	1.00	1.00	1.00
Res	0.20	0.18	0.16	1.00	1.00	0.33	1.00	1.00	0.33	1.00	0.36	0.33	1.00	1.00	0.50	1.00	1.00	0.50
Vul	24.65	33.58	43.64	0.00	0.00	3.52	0.00	0.00	3.52	0.00	1.25	6.87	0.00	0.00	2.26	0.00	0.00	2.26
rVul	1.00	0.96	0.70	0.00	0.00	0.06	0.00	0.00	0.06	0.00	0.04	0.11	0.00	0.00	0.04	0.00	0.00	0.04
Sust	0.00	0.00	0.03	1.00	1.00	0.31	1.00	1.00	0.31	1.00	0.34	0.25	1.00	1.00	0.48	1.00	1.00	0.48
DHTS																		
Rel	0.91	0.73	0.55	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.78	1.00	1.00	1.00	1.00	1.00	1.00
Res	0.22	0.18	0.14	1.00	1.00	0.40	1.00	1.00	0.40	0.50	0.25	0.20	1.00	1.00	0.33	1.00	1.00	0.33
Vul	56.08	90.82	141.0	0.00	0.00	61.84	0.00	0.00	61.84	17.18	54.78	90.47	0.00	0.00	39.68	0.00	0.00	39.68
rVul	0.77	0.62	0.32	0.00	0.00	0.14	0.00	0.00	0.14	0.23	0.38	0.21	0.00	0.00	0.09	0.00	0.00	0.09
Sust	0.05	0.05	0.05	1.00	1.00	0.34	1.00	1.00	0.34	0.38	0.15	0.12	1.00	1.00	0.30	1.00	1.00	0.30
Wiggens																		
Rel	0.93	0.78	0.58	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.91	0.87	1.00	1.00	1.00	1.00	1.00	1.00
Res	0.32	0.28	0.24	1.00	1.00	0.42	1.00	1.00	0.42	0.43	0.34	0.29	1.00	1.00	0.43	1.00	1.00	0.43
Vul	15.52	23.12	31.01	0.00	0.00	18.69	0.00	0.00	18.69	5.39	12.14	18.79	0.00	0.00	12.44	0.00	0.00	12.44
rVul	0.74	0.66	0.28	0.00	0.00	0.17	0.00	0.00	0.17	0.26	0.34	0.17	0.00	0.00	0.11	0.00	0.00	0.11
Sust	0.08	0.08	0.10	1.00	1.00	0.35	1.00	1.00	0.35	0.31	0.20	0.21	1.00	1.00	0.38	1.00	1.00	0.38
Industry																		
Rel	0.90	0.90	0.90	1.00	1.00	1.00	1.00	1.00	1.00	0.90	0.90	0.90	1.00	1.00	1.00	1.00	1.00	1.00
Res	0.43	0.44	0.43	1.00	1.00	0.93	1.00	1.00	0.93	0.43	0.44	0.43	1.00	1.00	0.96	1.00	1.00	0.96
Vul	4.15	4.20	4.10	0.08	0.09	0.10	0.08	0.09	0.10	4.15	4.20	4.10	0.07	0.09	0.09	0.07	0.09	0.09
rVul	0.48	0.48	0.48	0.01	0.01	0.01	0.01	0.01	0.01	0.48	0.48	0.48	0.01	0.01	0.01	0.01	0.01	0.01
Sust	0.20	0.21	0.20	0.99	0.99	0.92	0.99	0.99	0.92	0.20	0.21	0.20	0.99	0.99	0.95	0.99	0.99	0.95
Irr Mko4																		
Rel	0.99	0.99	0.99	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.99	1.00	1.00	1.00	1.00	1.00	1.00
Res	0.60	0.65	0.72	1.00	1.00	1.00	1.00	1.00	1.00	0.60	0.65	0.72	1.00	1.00	1.00	1.00	1.00	1.00
Vul	1.35	1.32	0.96	0.00	0.00	0.00	0.00	0.00	0.00	1.35	1.32	0.96	0.00	0.00	0.00	0.00	0.00	0.00
rVul	0.50	0.50	0.50	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.50	0.50	0.00	0.00	0.00	0.00	0.00	0.00
Sust	0.30	0.32	0.36	1.00	1.00	1.00	1.00	1.00	1.00	0.30	0.32	0.36	1.00	1.00	1.00	1.00	1.00	1.00
Irr Mko5																		
Rel	0.99	0.99	0.99	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.99	1.00	1.00	1.00	1.00	1.00	1.00
Res	0.74	0.83	0.71	1.00	1.00	1.00	1.00	1.00	1.00	0.74	0.83	0.71	1.00	1.00	1.00	1.00	1.00	1.00
Vul	0.58	0.46	0.55	0.00	0.00	0.06	0.00	0.00	0.06	0.58	0.46	0.55	0.00	0.00	0.06	0.00	0.00	0.06
rVul	0.50	0.50	0.41	0.00	0.00	0.04	0.00	0.00	0.04	0.50	0.50	0.41	0.00	0.00	0.04	0.00	0.00	0.04
Sust	0.37	0.41	0.41	1.00	1.00	0.96	1.00	1.00	0.96	0.37	0.41	0.41	1.00	1.00	0.96	1.00	1.00	0.96
IFR 4																		
Rel	0.81	0.81	0.81	0.95	0.93	0.90	0.95	0.93	0.90	0.81	0.81	0.81	0.95	0.94	0.91	0.95	0.94	0.91
Res	0.44	0.45	0.45	0.61	0.60	0.57	0.62	0.60	0.57	0.44	0.45	0.45	0.63	0.61	0.56	0.63	0.61	0.57
Vul	10.04	10.17	9.95	4.43	3.86	3.72	4.43	3.80	3.72	10.04	10.17	9.95	4.44	3.86	3.78	4.44	3.89	3.74
rVul	0.27	0.28	0.29	0.12	0.11	0.11	0.12	0.11	0.11	0.27	0.28	0.29	0.12	0.11	0.11	0.12	0.11	0.11
Sust	0.26	0.26	0.26	0.51	0.50	0.46	0.51	0.50	0.46	0.26	0.26	0.26	0.53	0.51	0.46	0.53	0.51	0.46
rS_{WDC}	**0.18**	**0.19**	**0.20**	**0.93**	**0.93**	**0.62**	**0.93**	**0.93**	**0.62**	**0.40**	**0.27**	**0.26**	**0.93**	**0.93**	**0.65**	**0.93**	**0.93**	**0.65**

APPENDIX D
RCCC FOR TIME SERIES WITH MISSING DATA

RCCC: residual cross correlation coefficient

In section 7.4 a procedure was developed for stochastic generation of annual runoff at multiple sites in a region where no runoff is likely to occur due to exceptional dry conditions. The method considered annual runoff $q(i,t)$ at any given site i in year t to be a product of two stochastic processes as

$$q(i,t) = v(i,t)z(i,t) \tag{D1}$$

where $v(i,t)$ is a binary process indicating the occurrence of a zero flow event ($v(i,t) = 0$) or not ($v(i,t) = 1$), and $z(i,t)$ is the non-zero part modelled as a CARMA(p,q) model estimated based on the observed non-zero annual runoff events and subsequently adjusted for the time series generated using the model in Eq. (D1) to have a mean value equal to mean value of the observed time series incl. events of zero runoff. To estimate the residual cross correlation coefficient between the non-zero events in the multiple time series necessary for the generation of $z(i,t)$ the following method was developed based on the use of an indicator function as recommended by Madsen (1995). Given M time series of equal extension T of annual runoff $q(i,t)$, where $t = 1, \ldots, T$ and $i = 1, \ldots, M$. The indicator function $\alpha(i,t)$ is defined as

$$\alpha(i,t) = \begin{cases} 1, & q(i,t) > 0 \\ 0, & q(i,t) = 0 \end{cases} \tag{D2}$$

First, the non-zero time series of annual runoff are transformed into time series of normally distributed event $x(i,t)$. Next, the residuals $\varepsilon(i,t)$ are calculated by pre-whitening of the observed time series of non-zero annual runoff according to the univariate ARMA(p,q) model specified at each site as

$$\varepsilon(i,t) = \left(x(i,t) - \bar{x}(i)\right) - \sum_{j=1}^{p} \phi(j)\left(x(i,t-j) - \bar{x}(i)\right) - \sum_{j=1}^{q} \theta(j)\varepsilon(i,t-j) \tag{D3}$$

where

$$\bar{x}(i) = \frac{\sum_{t=1}^{T} \alpha(i,t)x(i,t)}{\sum_{t=1}^{T} \alpha(i,t)} \tag{D4}$$

The ARMA(p,q) model for each site should be estimated also under consideration of missing events as recommended by, for example, Madsen (1995). However, in this study no serial correlation was assumed between annual runoff.

The adjusted residual cross correlation $\rho^*(i,j,l)$ between site i and j and at lag time l is estimated as

$$\rho^*(i,j,l) = \frac{1}{T}\sum_{t=1}^{T-l}\alpha(i,t)\alpha(j,t+l)\varepsilon(i,t)\varepsilon(j,t+l) \tag{D5}$$

Next a counting function *N* is defined as

$$N(i,j,l) = \sum_{t=1}^{T-l}\alpha(i,t)\alpha(j+l,t) \tag{D6}$$

Finally, the residual cross correlation coefficient $\rho(i,j,t)$ is estimated as

$$\rho(i,j,l) = \frac{\rho^*(i,j,l)}{N(i,j,t)} \tag{D7}$$

APPENDIX E
RESULTS FOR THE MUPFURE SYSTEM

	Scenario 1			Scenario 2			Scenario 3			Scenario 4			Scenario 5		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
Chegutu															
Rel	0.76	0.71	0.62	0.77	0.72	0.68	0.76	0.72	0.68	0.76	0.74	0.74	0.83	0.79	0.75
Res	0.38	0.32	0.24	0.37	0.33	0.29	0.37	0.32	0.29	0.37	0.35	0.34	0.50	0.42	0.36
Vul	1.38	2.00	3.21	1.34	2.06	3.01	1.46	1.69	1.79	1.46	1.60	1.67	0.95	1.59	2.51
rVul	0.11	0.11	0.13	0.11	0.12	0.13	0.11	0.10	0.07	0.11	0.09	0.07	0.07	0.09	0.10
Sust	0.26	0.20	0.13	0.26	0.21	0.17	0.25	0.21	0.18	0.25	0.24	0.23	0.39	0.30	0.24
IRR1															
Rel	0.57	0.57	0.53	0.59	0.56	0.57	0.59	0.55	0.54	0.59	0.56	0.57	0.79	0.74	0.71
Res	0.22	0.22	0.20	0.23	0.22	0.22	0.23	0.21	0.20	0.23	0.22	0.22	0.23	0.22	0.21
Vul	0.99	1.10	1.28	0.98	1.02	0.99	0.98	1.15	1.24	0.98	1.02	0.99	0.99	1.19	1.27
rVul	0.10	0.10	0.11	0.10	0.09	0.09	0.10	0.10	0.11	0.10	0.09	0.09	0.10	0.11	0.11
Sust	0.12	0.11	0.09	0.12	0.11	0.12	0.12	0.10	0.10	0.12	0.11	0.12	0.16	0.14	0.13
IRR2															
Rel	0.56	0.54	0.50	0.57	0.55	0.55	0.57	0.53	0.50	0.57	0.55	0.55	0.79	0.75	0.69
Res	0.19	0.18	0.16	0.19	0.18	0.18	0.19	0.17	0.16	0.19	0.18	0.18	0.20	0.20	0.18
Vul	12.30	13.72	16.66	11.62	12.59	12.57	11.62	14.28	16.00	11.62	12.59	12.57	12.27	14.41	16.55
rVul	0.10	0.10	0.11	0.09	0.09	0.08	0.09	0.10	0.10	0.09	0.09	0.08	0.10	0.10	0.11
Sust	0.09	0.09	0.07	0.10	0.09	0.09	0.10	0.08	0.07	0.10	0.09	0.09	0.14	0.13	0.11
IRR3															
Rel	0.43	0.37	0.31	0.44	0.42	0.41	0.44	0.36	0.32	0.44	0.42	0.41	0.68	0.63	0.57
Res	0.28	0.18	0.14	0.29	0.27	0.26	0.29	0.17	0.14	0.29	0.27	0.26	0.12	0.11	0.10
Vul	2.72	4.68	5.79	2.49	3.41	3.42	2.49	4.95	5.95	2.49	3.41	3.42	7.39	9.15	10.93
rVul	0.06	0.08	0.09	0.06	0.06	0.05	0.06	0.09	0.09	0.06	0.06	0.05	0.16	0.16	0.17
Sust	0.11	0.06	0.04	0.12	0.11	0.10	0.12	0.06	0.04	0.12	0.11	0.10	0.07	0.06	0.05
IRR4															
Rel	0.43	0.40	0.36	0.43	0.41	0.39	0.43	0.39	0.36	0.43	0.42	0.40	0.71	0.66	0.60
Res	0.12	0.11	0.10	0.12	0.11	0.10	0.12	0.11	0.10	0.12	0.11	0.11	0.12	0.12	0.10
Vul	39.96	47.31	55.16	40.44	41.56	43.01	40.74	46.82	53.29	40.74	41.01	41.71	38.71	48.07	59.02
rVul	0.11	0.11	0.11	0.11	0.10	0.09	0.11	0.11	0.11	0.11	0.10	0.09	0.11	0.12	0.12
Sust	0.05	0.04	0.03	0.04	0.04	0.04	0.04	0.04	0.03	0.04	0.04	0.04	0.07	0.07	0.05
DWT															
Rel	0.81	0.74	0.64	0.81	0.76	0.71	0.80	0.76	0.70	0.80	0.79	0.78	0.89	0.85	0.79
Res	0.25	0.25	0.21	0.24	0.22	0.21	0.25	0.24	0.24	0.25	0.24	0.23	0.24	0.22	0.20
Vul	0.21	0.21	0.26	0.21	0.26	0.28	0.21	0.21	0.21	0.21	0.21	0.23	0.18	0.21	0.26
rVul	0.10	0.09	0.10	0.10	0.12	0.11	0.10	0.09	0.08	0.10	0.10	0.09	0.09	0.09	0.10
Sust	0.18	0.17	0.12	0.18	0.15	0.13	0.18	0.16	0.15	0.18	0.17	0.16	0.20	0.17	0.14
IFR site															
Rel	0.94	0.93	0.94	0.94	0.93	0.94	0.94	0.93	0.92	0.94	0.93	0.94	0.97	0.96	0.95
Res	0.35	0.35	0.37	0.35	0.35	0.38	0.35	0.35	0.36	0.35	0.35	0.38	0.39	0.38	0.39
Vul	1.16	1.18	1.15	1.16	1.18	1.15	1.17	1.17	1.13	1.16	1.18	1.15	0.95	1.10	1.09
rVul	0.11	0.10	0.10	0.11	0.10	0.10	0.11	0.10	0.10	0.11	0.10	0.10	0.09	0.10	0.10
Sust	0.29	0.29	0.31	0.29	0.30	0.32	0.29	0.29	0.30	0.29	0.30	0.32	0.35	0.33	0.33
rS_{WDC}	**0.16**	**0.14**	**0.11**	**0.16**	**0.14**	**0.14**	**0.16**	**0.13**	**0.13**	**0.16**	**0.15**	**0.15**	**0.20**	**0.17**	**0.15**

	Scenario 6			Scenario 7			Scenario 8			Scenario 9			Scenario 10		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
Chegutu															
Rel	0.79	0.74	0.70	0.83	0.81	0.78	0.78	0.75	0.72	0.83	0.82	0.80	0.83	0.79	0.74
Res	0.39	0.33	0.29	0.50	0.45	0.39	0.39	0.34	0.31	0.49	0.46	0.44	0.49	0.40	0.33
Vul	1.41	2.25	3.32	0.95	1.55	2.22	1.56	1.82	2.07	1.08	1.24	1.29	1.15	1.93	2.95
rVul	0.11	0.13	0.14	0.07	0.09	0.09	0.12	0.10	0.09	0.08	0.07	0.05	0.09	0.11	0.12
Sust	0.27	0.21	0.17	0.39	0.33	0.27	0.27	0.23	0.20	0.38	0.35	0.33	0.37	0.28	0.22
IRR1															
Rel	0.74	0.69	0.66	0.79	0.77	0.75	0.74	0.69	0.66	0.79	0.77	0.76	0.83	0.79	0.75
Res	0.24	0.23	0.23	0.23	0.22	0.22	0.24	0.23	0.23	0.23	0.22	0.22	0.25	0.23	0.22
Vul	0.99	1.16	1.25	0.99	1.07	1.02	0.99	1.16	1.25	0.99	1.07	1.02	0.97	1.15	1.25
rVul	0.10	0.11	0.11	0.10	0.10	0.09	0.10	0.11	0.11	0.10	0.10	0.09	0.10	0.10	0.11
Sust	0.16	0.14	0.14	0.16	0.15	0.15	0.16	0.14	0.14	0.16	0.15	0.15	0.19	0.16	0.15
IRR2															
Rel	0.66	0.60	0.56	0.79	0.77	0.75	0.66	0.61	0.56	0.79	0.77	0.75	0.78	0.74	0.68
Res	0.19	0.17	0.18	0.20	0.20	0.19	0.19	0.17	0.18	0.20	0.20	0.19	0.20	0.18	0.17
Vul	13.95	16.43	18.12	12.27	12.81	13.44	13.96	16.35	18.27	12.31	12.74	13.40	13.64	16.05	18.35
rVul	0.11	0.12	0.12	0.10	0.09	0.09	0.11	0.12	0.12	0.10	0.09	0.09	0.11	0.11	0.12
Sust	0.11	0.09	0.09	0.14	0.14	0.13	0.11	0.09	0.09	0.14	0.14	0.13	0.14	0.12	0.10
IRR3															
Rel	0.66	0.60	0.55	0.68	0.66	0.63	0.66	0.60	0.55	0.68	0.67	0.63	0.76	0.72	0.66
Res	0.19	0.18	0.17	0.12	0.11	0.10	0.19	0.18	0.17	0.12	0.11	0.10	0.18	0.17	0.15
Vul	4.24	4.94	5.42	7.39	8.24	8.61	4.25	4.94	5.42	7.39	8.24	8.70	4.33	4.93	5.57
rVul	0.09	0.09	0.09	0.16	0.14	0.14	0.09	0.09	0.09	0.16	0.14	0.14	0.10	0.09	0.09
Sust	0.12	0.10	0.09	0.07	0.06	0.06	0.12	0.10	0.09	0.07	0.06	0.06	0.13	0.11	0.09
IRR4															
Rel	0.65	0.59	0.53	0.71	0.69	0.66	0.65	0.60	0.54	0.71	0.70	0.67	0.77	0.72	0.66
Res	0.15	0.14	0.14	0.12	0.12	0.10	0.15	0.14	0.14	0.12	0.12	0.11	0.16	0.14	0.13
Vul	28.70	33.96	45.60	38.71	43.26	46.58	28.52	33.72	44.73	38.94	42.66	44.97	26.29	33.35	50.60
rVul	0.08	0.08	0.09	0.11	0.11	0.10	0.08	0.08	0.09	0.11	0.10	0.09	0.07	0.08	0.10
Sust	0.09	0.08	0.07	0.07	0.07	0.06	0.09	0.08	0.07	0.07	0.07	0.06	0.11	0.09	0.07
DWT															
Rel	0.84	0.79	0.74	0.89	0.86	0.83	0.83	0.80	0.76	0.89	0.87	0.86	0.89	0.85	0.79
Res	0.21	0.19	0.18	0.24	0.23	0.20	0.21	0.19	0.19	0.24	0.23	0.22	0.20	0.18	0.18
Vul	0.21	0.23	0.26	0.18	0.21	0.26	0.21	0.23	0.23	0.18	0.18	0.21	0.21	0.26	0.28
rVul	0.10	0.11	0.10	0.09	0.09	0.10	0.10	0.11	0.09	0.09	0.08	0.08	0.10	0.12	0.11
Sust	0.16	0.13	0.12	0.20	0.18	0.15	0.16	0.14	0.13	0.19	0.18	0.17	0.16	0.14	0.12
IFR site															
Rel	0.97	0.95	0.95	0.97	0.96	0.96	0.97	0.95	0.94	0.97	0.96	0.96	0.98	0.97	0.96
Res	0.42	0.41	0.42	0.42	0.43	0.45	0.42	0.41	0.41	0.42	0.43	0.43	0.42	0.43	0.42
Vul	1.08	1.12	1.11	1.08	1.12	1.07	1.07	1.15	1.13	1.07	1.10	1.09	1.09	1.18	1.16
rVul	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
Sust	0.37	0.36	0.36	0.37	0.37	0.39	0.36	0.36	0.35	0.36	0.37	0.37	0.37	0.38	0.36
rS_{WDC}	**0.18**	**0.16**	**0.15**	**0.20**	**0.19**	**0.17**	**0.18**	**0.16**	**0.15**	**0.20**	**0.19**	**0.18**	**0.21**	**0.18**	**0.16**

APPENDIX F

RIAM SCORE CARD FOR MUPFURE SYSTEM

This appendix contains the scores attached to each of the four reservoir options in the Mupfure water resources system. The results originate from Makoni (2000) but are presented here in a slightly modified version.

1. No reservoir
2. Construction of the Muda Dam
3. Construction of the Mhondoro Dam
4. Construction of the Muda and the Mhondoro Dam

Table F1 RIAM results for the construction of the no reservoir option.

No reservoir option	FAS	A_1	A_2	B_1	B_2	B_3
Physical/chemical components (P/C)						
Impact on soils	0	2	0	3	3	1
Impact on visual/aesthetic character of the area	0	2	0	3	3	1
Biological/Ecological components (B/E)						
Loss of habitat for terrestrial and avifauna	0	3	0	3	3	3
Impact on flora/vegetation	0	3	0	3	3	3
Social/cultural components (S/C)						
Impact on human settlement	0	2	0	3	3	3
Loss of historical, archaeological and cultural sites	0	4	0	3	3	1
Impact on human health	0	3	0	3	2	1
Impact on recreational activities	0	2	0	3	2	1
Impact on migration away from area	-24	3	-1	3	2	3
Economic/operational components (E/C)						
Impact on local economy	0	3	0	3	2	3
Impact on food security	0	3	0	3	2	1
Impact on infrastructure	0	3	0	3	2	1
Impact on economy of external user	0	3	0	3	2	3

Table F2 RIAM results for the construction of the Muda dam.

Construction of the Muda Dam	FAS	A_1	A_2	B_1	B_2	B_3
Physical/chemical components (P/C)						
Impact on soils	-14	2	-1	3	3	1
Impact on visual/aesthetic character of the area	-14	2	-1	3	3	1
Biological/Ecological components (B/E)						
Loss of habitat for terrestrial and avifauna	-27	3	-1	3	3	3
Impact on flora/vegetation	-27	3	-1	3	3	3
Social/cultural components (S/C)						
Impact on human settlement	-18	2	-1	3	3	3
Loss of historical, archaeological and cultural sites	0	4	0	3	3	1
Impact on human health	-18	3	-1	3	2	1
Impact on recreational activities	12	2	1	3	2	1
Impact on migration away from area	24	3	1	3	2	3
Economic/operational components (E/C)						
Impact on local economy	48	3	2	3	2	3
Impact on food security	18	3	1	3	2	1
Impact on infrastructure	36	3	2	3	2	1
Impact on economy of external user	0	3	0	3	2	3

Table F3 RIAM results for the construction of the Mhondoro Dam.

Construction of the Mhondoro Dam	FAS	A_1	A_2	B_1	B_2	B_3
Physical/chemical components (P/C)						
Impact on soils	-28	2	-2	3	3	1
Impact on visual/aesthetic character of the area	-28	2	-2	3	3	1
Biological/Ecological components (B/E)						
Loss of habitat for terrestrial and avifauna	-54	3	-2	3	3	3
Impact on flora/vegetation	-54	3	-2	3	3	3
Social/cultural components (S/C)						
Impact on human settlement	-54	2	-3	3	3	3
Loss of historical, archaeological and cultural sites	-84	4	-3	3	3	1
Impact on human health	-36	3	-2	3	2	1
Impact on recreational activities	12	2	1	3	2	1
Impact on migration away from area	48	3	2	3	2	3
Economic/operational components (E/C)						
Impact on local economy	72	3	3	3	2	3
Impact on food security	18	3	1	3	2	1
Impact on infrastructure	54	3	3	3	2	1
Impact on economy of external user	72	3	3	3	2	3

Table F4 RIAM results for the construction of the Muda and Mhondoro Dam.

Construction of the Muda and Mhondoro Dam	FAS	A_1	A_2	B_1	B_2	B_3
Physical/chemical components (P/C)						
Impact on soils	-42	2	-3	3	3	1
Impact on visual/aesthetic character of the area	-42	2	-3	3	3	1
Biological/Ecological components (B/E)						
Loss of habitat for terrestrial and avifauna	-54	3	-2	3	3	3
Impact on flora/vegetation	-54	3	-2	3	3	3
Social/cultural components (S/C)						
Impact on human settlement	-54	2	-3	3	3	3
Loss of historical, archaeological and cultural sites	-84	4	-3	3	3	1
Impact on human health	-54	3	-3	3	2	1
Impact on recreational activities	24	2	2	3	2	1
Impact on migration away from area	72	3	3	3	2	3
Economic/operational components (E/C)						
Impact on local economy	72	3	3	3	2	3
Impact on food security	36	3	2	3	2	1
Impact on infrastructure	54	3	3	3	2	1
Impact on economy of external user	72	3	3	3	2	3

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