CITY WATER BALANCE
A New Scoping Tool For Integrated Urban Water Management Options

by

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Abstract

Urban water scoping modelling packages are used as tools to inform decision makers of the sustainability of different water management options for a city. Previous scoping models have not taken sufficient account of natural systems in the urban environment and are often limited in terms of the range of indicators used to measure sustainability and the choice of water management options offered. A new modelling package, named *City Water Balance*, has been developed to address these limitations. It has the capability to assess the sustainability of a variety of water management options, including sustainable urban drainage systems, in terms of water flow, water quality, whole life cost and life cycle energy for alternative scenarios of future urban land use, population and climate. Application of the modelling package to the City of Birmingham has demonstrated that the modelled components can describe adequately the existing system, giving confidence that it can be used for scoping strategic options for future water supply and wastewater management. The further application of the package to model alternative scenarios through to 2055 for Birmingham has also been undertaken to illustrate its application. The results from the different analyses have shown that medium scale rainwater harvesting and borehole abstraction are predicted to be more sustainable than the conventional centralised supply and that medium scale wastewater recycling would be more cost effective but less energy efficient. The most sustainable strategy was installation of water efficient appliances as there is the potential for large energy savings from reduced indoor usage and consequent water heating requirements.

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Acronyms

AWBM	Australian Water Balance Model
BLA	Birmingham Learning Alliance
BMP	Best Management Practices
BOD	Biological Oxygen Demand
CIRIA	Construction Industry Research and Information Association
COD	Chemical Oxygen Demand
CWB	City Water Balance
DAP	Drainage Area Plan
DEFRA	Department for Environment, Food and Rural Affairs
DMA	District Metered Area
DSS	Decision Support System
EE	Embodied Energy
EPSRC	Engineering and Physical Sciences Research Council
ESI	Environmental Sustainability Indicator
FF	First Flush
FM	Flow Monitor
GIS	Graphical Information System
GUI	Graphical User Interface
GWP	Global Warming Potential
GWR	Groundwater Recharge
IPCC	Intergovernmental Panel on Climate Change
IRUN	Impermeable area Runoff
ISO	International Organisation for Standardization
IUWM	Integrated Urban Water Management
LA	Learning Alliance
LCA	Life Cycle Assessment

MC	Minicluster
M ISTRA	Swedish Foundation for Strategic Environmental Research
NEAR	Non-Effective Area Runoff
NPV	Net Present Value
OFWAT	Office of Water Services
OS	Ordnance Survey
PA	Porous Asphalt
p.e.	Population equivalent
PET	Potential Evapo-Transpiration
POS	Public Open Space
PP	Porous Pavement
PRUN	Pervious area Runoff
PS	Pervious Store
RH	Rainwater Harvesting
RO	Reverse Osmosis
SI	Sustainability Indicator
SMD	Soil Moisture Deficit
SMURF	Sustainable Management of Urban Rivers and Floodplains
SS	Stormwater Store
STW	Severn Trent Water Company
SUDS	Sustainable Urban Drainage Systems
SUWM	Sustainable Urban Water Management
SWARD	Sustainable Water Industry Asset Resources Decisions
SWITCH	Sustainable Water Improves Tomorrow's Cities' Health
SWMP	Surface Water Management Plan
TDS	Total Dissolved Solids
TG	Trigger-to-Irrigate
TSS	Total Suspended Solids

TN	Total Nitrogen
TP	Total Phosphorus
UB	Unitblock
UKWIR	UK Water Industry Research
UVQ	Urban Volume and Quality
UWOT	Urban Water Optioneering Tool
VBA	Visual Basic for Applications
WaND	Water Cycle Management for New Developments
WCED	World Commission on Environment and Development
WLC	Whole Life Cost
WMS	Water Management Strategies
WRZ	Water Resource Zone
WWTP	Wastewater Treatment Plant

1 - Introduction

1.1 Water Issues

Urban water management faces a variety of challenges. Population growth, urbanisation and industrialisation can result in pollution, depletion of water resources and increased flooding. Climate change is predicted to add to these challenges with combinations of altered precipitation patterns, increasingly intense rainfall events and prolonged droughts. In the face of these global change pressures the development and adoption of sustainable water management strategies (WMS) is increasingly important and urban water models can be used to inform decision making in this regard.

In the last century global water use has increased by more than twice the rise in population, creating great pressure on water reserves in many areas (e.g. Australia, large parts of Africa, and the Mediterranean countries) (*Lundin*, 2004). Food production and economic growth place the greatest stress on the water supply of a growing population (*Lundin*, 2004). The pressing need to address current water management practices to reduce consumption and the related environmental impacts is highlighted in the STERN Report (*Stern et al.*, 2006), a report on the economics of climate change. The authors of this report estimate that by the 2080s more than one billion people will suffer water shortages. Summer rainfall volumes in the UK are expected to decrease by 35-50% by 2080 (*Hulme et al.*, 2002) which will result in a decreased supply concurrent with times of increased demand. Compounding this issue, past trends show that UK water consumption per person has increased from 146 l/p/d in 1994/5 to 154 l/c/d in 2003/4 (*Ofwat*, 2005). Severn Trent Water Company, which provides water and wastewater services to the West

Midlands area, currently does not have enough water resources to meet long term demand. They estimate that they will have an average deficit of 250 Ml/day over the next 25 years, or 14% of the current supply (1,900 Ml/day) (*STW*, 2007).

As well as placing great stress on water resources, population increase has resulted in flooding issues arising from urbanisation and consequent hardening of the landscape. Large areas of cityscape are effectively impervious, which impacts the water cycle; rainwater that would naturally have been retained on-site contributing to recharge and evaporation instead becomes runoff which flows off site much more rapidly than it would from greenfield land (*CIRIA*, 2005a). In addition, surface water systems in cities have traditionally been designed to remove stormwater from the city as quickly as possible to prevent local flooding. This has resulted in an unnaturally rapid-response and large peak in discharge to rivers that can lead to downstream flooding. In many cities the existing, ageing surface water sewers are struggling to cope with greater runoff volumes from an increasingly hardened cityscape, which can result in sewer surcharge, contributing to the flooding problem (*Smith & Ward*, 1998). Pollution is another issue related to stormwater discharge to rivers from urban areas. Discharge is often untreated and carries pollutants from road, roof and paved surfaces such as heavy metals and organic compounds (*Ellis and Mitchell*, 2006) that can damage the river's ecosystem and increase water treatment costs if the river is used for water supply.

In the UK, flooding problems are likely to be exacerbated by climate change (*Ekstrom et al.*, 2005) with changes predicted to be the greatest in urban areas, which may be subject to a 40% increase in rainfall intensity by 2080 (*Foresight*, 2004). Residential drains, which are typically designed for a one in 30 year flood occurrence (Ashley et al., 2005), are likely to have inadequate capacity for the predicted increased-intensity storms, resulting in more widespread flooding of properties. The cost of flooding in the UK is estimated to increase from 0.1% GDP to 0.2-0.4% GDP based on a rise of 3-4°C and the accompanying rainfall changes (*Stern et al.*, 2006). The need for better surface water management strategies was demonstrated during the summer floods of 2007, which cost the UK economy £3.2 billion (*EA*, 2009a). In response to these floods, Sir Michael Pitt was commissioned by the government to conduct an independent review (*Pitt*, 2007) and recommendations arising from the report have encouraged local councils to invest in the development and implementation of surface water management plans (SWMPs). The SWMPs build on ideas from the Government guidance document “Making Space for Water” (*Defra*, 2004) that sets out proposals for flood risk management, sustainability criteria, and land use policy under the pressures of climate change and increasing housing demand. The recent Flood and Water Management Act (2010) requires developers to include sustainable drainage (where practicable) in new developments, by making the right to connect to public surface water sewers conditional on meeting the new Sustainable Urban Drainage Systems (SUDS) standards.

The UK’s water system is still largely based on designs from the early 1800s when cost-optimised sanitation was the priority and environmental effects were given little importance. Health is still clearly the priority in water provision but technology has long been in-place to achieve this and so attention has turned to environmental sustainability (*Butler & Davies*, 2004). Sustainable development has been defined as “development that meets the needs of the present without compromising the ability of future generations to meet their own needs” (*WCED*, 1987). As part of the UK government’s commitment to sustainable development it has set priorities for water management in the future which include (*Woods-Ballard et al.*, 2007):

- Prudent use of water resources
- Reducing diffuse pollution of water
- Managing flood risks

Conventional water management has treated the urban water system as three separate parts: water supply, wastewater and stormwater. In doing so the water industry has fallen short of finding optimally sustainable solutions for the system as a whole (*UNESCO-IHE*, 2007) (Fig. 1.1). Integrated Urban Water Management (IUWM) seeks to find new solutions by breaking down the barriers between the three sections of the water industry (*UNESCO-IHE*, 2007), taking into account the entire urban water cycle including sludge disposal, materials and energy consumption and agriculture. The EU Water Framework Directive (WFD), passed in 2000, and the UK Water Act (*The Parliamentary Water Act*, 2003) are the most recent major legislation affecting the UK in this regard. By integrating previous directives the WFD provides a more holistic approach towards achieving its major objective of "good" status for European waters by 2015. Other areas of focus include (*van der Steen*, 2006):

- implementation of water pricing policies as an incentive for sustainable use of water resources and polluter pays principle
- integration of policies: agriculture, industry, consumers
- reduce emissions and enforce minimum discharge quality to receiving environment
- phase out discharge of priority contaminants in 20 years

The UK Water Act introduced new laws for water supply including:

- statutory requirement for water companies to produce drought and water resource management plans

- encouragement of competition in water supply services to give a “fairer deal to consumers”
- provisions for grants for flood defence work

The difference in perspective between traditional and sustainable urban water management is shown in Table 1.1.

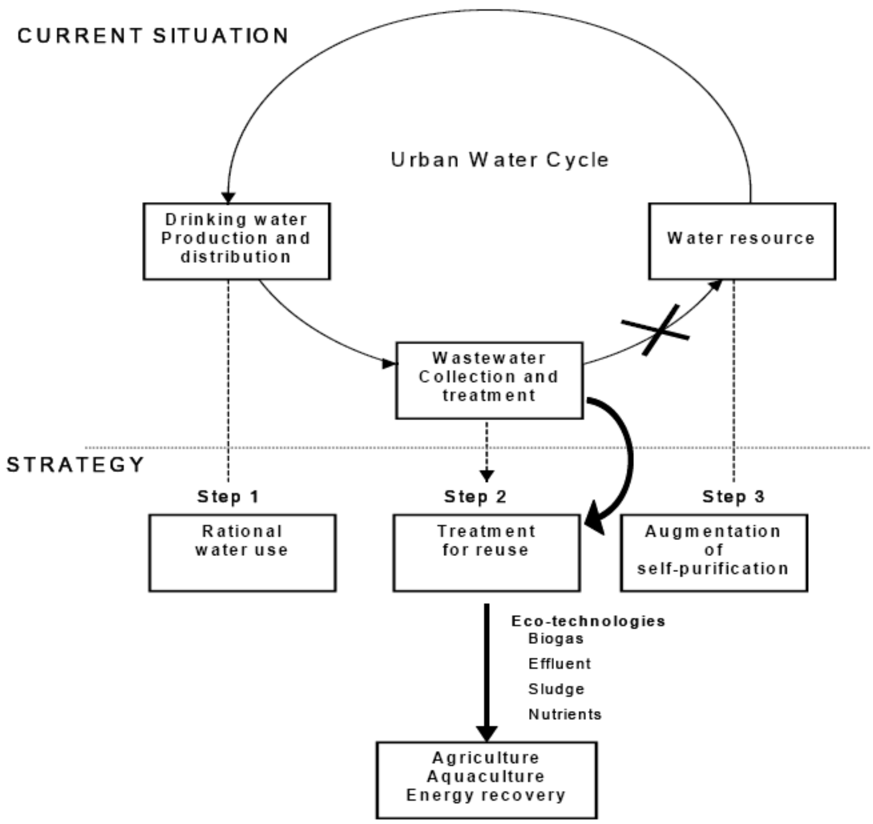


Figure 1.1 Demonstrating water management strategies for potential improvement on the current urban water cycle (*UNESCO-IHE*, 2007).

Table 1.1. Comparison of traditional and sustainable Urban Water Management (*Keith & Brown*, 2008)

Attributes	**Traditional**	**Sustainable**
System Boundary	Water supply, sewerage and flood control for economic and population growth and public health protection.	Multiple purposes for water considered over long-term timeframes including waterway health and other sectoral needs i.e. transport, recreation/amenity, micro-climate, energy etc.
Management Approach	Compartmentalisation and optimisation of single components of the water cycle	Adaptive, integrated, sustainable management of the total water cycle (including land-use)
Expertise	Narrow technical and economic focussed disciplines	Interdisciplinary, multi-stakeholder learning across social, technical, economic, design, ecological spheres etc
Service delivery	Centralised, linear and predominantly technologically and economically based	Diverse, flexible solutions at multiple scales via a suite of approaches (technical, social, economic, ecological etc)
Role of public	Water managed by government on behalf of communities	Co-management of water between government, business and communities
Risk	Risk regulated and controlled by government	Risk shared and diversified via private and public instruments

Urban water modelling can be used as a tool to facilitate movement towards IUWM, by generating indicator outputs, allowing quantitative comparison of conventional and unconventional water management strategies for supply, wastewater and stormwater in an urban setting. Using urban water models, with sustainability indicator outputs, provides structure to the sustainability assessment process, allows examination of its components and their interactions, and enhances stakeholder (including researchers) communication (*Jakeman et al.,* 2006). Scoping models with system boundaries that include all of the important processes within the urban water cycle are particularly suitable for use as decision-support tools for IUWM. The cornerstone of this thesis is the development of an urban water scoping modelling package that allows a more comprehensive assessment of sustainability than existing modelling tools.

1.2 SWITCH

The development of the scoping model presented in this thesis formed part of the research carried out within Theme 1 of the SWITCH (*Sustainable Water Management Improves Tomorrow's Cities' Health)* Project; a 5-year, 25 million Euro EU funded project whose goal is to "achieve a sustainable, healthy and safe urban water system" (*UNESCO-IHE*, 2007). The project, which began in February 2006, has focussed on developing sustainable urban water management approaches, technologies and financing (*UNESCO-IHE*, 2007) in response to pressures from climate change, urbanisation and industrialisation, population growth, urban sprawl, rural-urban migration and energy demand. Research has been carried out in the UK, Netherlands, Poland, Germany, Spain, Ghana, China, Brazil, Egypt, Colombia and Israel.

Ten cities worldwide, including Birmingham, are being used to demonstrate SWITCH results. In each city there is a SWITCH led Learning Alliance, whose purpose is to improve communication between the water stakeholders and in so doing encourage the development and implementation of more integrated water management strategies for the city.

The Project is divided into six themes (Appendix 3). The subject of Theme 1 is the IUWM paradigm shift, which is two-fold:

1) a shift to an integrated water management approach in order to optimise system efficiency

2) a shift of priorities; the traditional approach is to optimise the structural and hydraulic efficiency of the system whilst minimising cost whereas new approaches also consider the environmental effects and stakeholder interests

A primary output from Theme 1 has been a Knowledge Base System/Decision Support System called *City Water*, which builds on contemporary work in the field of decision support and scoping models for assessing the sustainability of alternative urban water management options. *City Water* stands out from its contemporaries in three key areas.

1) greater consideration is given to the sustainable exploitation of the natural environment.
2) attention is given to aspects of the regulatory environment and the existence and location of historical data that define past conditions, approaches and outcomes.
3) a wide range of time and space scales are covered that allow, for example, life cycle costing (financial and energy) over long times and local flooding over short times.

There are a range of modelling tools that have been developed under this Theme including tools for Economic modelling, Drainage modelling, Urban Evolution modelling and Risk modelling. The tool developed and tested during the course of this PhD, forming part of the *City Water* DSS, is the scoping model for urban water flow and quality modelling, called *City Water Balance* (*CWB*).

1.3 Research Aims

The aim of this research is the development and application of a scoping modelling code for the urban water system that has the capability of investigating water management strategies, at the city scale, to tackle the challenges that the cities of the future will face under pressure from climate change, population increase and rising energy prices.

A fundamental idea behind the model package is simplicity – it is intended to be a generic model that can be applied to any city in the world with relatively light data input requirements. The target audience are policy makers who may not have in-depth knowledge of water systems and so the model is designed to be comprehensible to the educated layman. Keeping data input to a minimum was vital in ensuring that the model appeals to a wide range of influential end-users, as it is the lack of data or the time consuming process of data collection that may result in the model not being used.

Scoping is another key aspect to the model – it is designed to be used for planning at the city-scale, quantitatively demonstrating possibilities for improvements to the water system and highlighting areas that may merit more in-depth investigation. Using simplified concepts to represent complex physical processes greatly reduces processing demand, enabling rapid simulation time, which is more likely to encourage multiple model simulations to assess the sustainability of different water management options.

The model is designed to look at future possibilities, supporting a proactive approach to tackling water management issues in contrast to the mainly reactive modelling stance taken by many water companies.

1.4 Methodology

The main areas of work were:

- Literature review and identifying areas for new research
- Development of modelling concepts
- Programming the modelling concepts
- Model testing
- Data collection for model application to the demonstration city, Birmingham
- Calibration and validation
- Simulations with different scenarios and strategies to represent Birmingham in 2055

Modelling concepts were developed, building on ideas from existing urban water scoping models. Primary areas of the existing scoping models identified for improvement were: more complete representation of the natural systems; expansion of water management options available; and, inclusion of energy and cost indicators.

An important additional layer in assessing the sustainability of the water system was the analysis of the energy used and economic cost during the lifetime of various water management strategies. Stand-alone Excel spreadsheets were developed that guide the user to enter data requirements for the estimation of total energy usage and cost. The process of identifying the key data requirements and keeping them to a minimum, to improve usability whilst retaining sufficient accuracy to give a meaningful result, was an important objective.

Many of the data inputs required for model simulations are spatially distributed and have been manipulated using a GIS. A GIS was also used to display spatially distributed results.

The model was initially developed in Visual Basic for Applications, which is ideal for model debugging and testing. It was then transferred to VB.NET because simulation time is greatly reduced; another important aspect of a scoping model.

The model was applied to the City of Birmingham. Data for Birmingham were collated from the literature, web searches, maps and the Birmingham Learning Alliance, in particular Severn Trent Water Company. Calibration and validation of the model was performed for water supply, wastewater and stormwater flows, water quality, whole life cost and life cycle energy usage. The model was then used to simulate various future scenarios to demonstrate some of its capabilities as a tool for water management planning at the city scale.

1.5 Structure of the Thesis

The thesis consists of the following chapters:

Chapter 1 – Introduction. A general introduction to current issues in water management is outlined. Development of the model is put into context within the *SWITCH* Project and the aims of research and methodology are detailed.

Chapter 2 – Literature review. A variety of urban water models are reviewed and their strengths and weaknesses highlighted. *City Water Balance* is introduced as the basis of this research and is shown to build on ideas from contemporary urban water models.

Chapter 3 – Model concepts. A detailed description of the model concepts behind *City Water Balance* is provided.

Chapter 4 – Case Study Birmingham. The City of Birmingham is introduced as a case study to test the model and to demonstrate its potential.

Chapter 5 – Data processing and calibration. In this section there is a description of how the data required for the Birmingham case study were prepared. Details of calibration undertaken are also outlined.

Chapter 6 – Validation. Details on the validation of data for water supply, wastewater and stormwater flows, water quality, cost and energy usage are discussed in this section.

Chapter 7 – Sensitivity analysis. The results are shown from a basic sensitivity analysis of several input parameters, highlighted during the calibration process as potentially difficult to constrain.

Chapter 8 – Scenarios. Three scenarios, proposed by the Birmingham Learning Alliance, for the City of Birmingham in 2055 are introduced. Calculation of climate data for 2055 is also detailed.

Chapter 9 – Results and Discussion. Results for various water management strategies are discussed for the three different scenarios. Results are displayed using indicators of water flow, contaminant loads, economic cost and energy usage. In addition, the effect of the use of efficient residential water appliances is demonstrated. Problems encountered during the course of the work are outlined.

Chapter 10 – Conclusion. A summary of the model, its innovative aspects and case study results is provided. Recommendations for improvements to the *CWB* model are made.

2 - Literature Review

In this chapter there is a review of sustainability indicators, urban water scoping models and life cycle analysis. The strengths and weaknesses of different methods and models used for sustainability assessment are outlined.

2.1 Sustainability Indicators

In this section there is an introduction to sustainability indicators and their use, in conjunction with urban water models, for decision support.

The use of sustainability indicators is a well established and documented method for assessing sustainable development (*Ashley et al.,* 2004; *Balkema et al.,* 2002), which can be defined as "development that meets the needs of the present without compromising the ability of future generations to meet their own needs" (*WCED*, 1987). In developed countries this is usually interpreted as "the provision of more effective and efficient services which maintain public health and welfare, whilst reducing harmful resource and environmental impacts" (*Foxon et al.,* 2002).

A popular definition of an indicator is "a piece of information that has a wider significance than its immediate meaning" (*Bakkes et al*., 1994). It can have the following functions:

- to assess current conditions and trends
- to anticipate future conditions and trends
- to provide information for spatial comparisons

- to provide early warning information

Consequently a sustainability indicator is a piece of information that indicates movement towards or away from sustainability. Incorporation of sustainability indicators into the decision making processes is a key task for water service providers (*Foxon et al.*, 2002).

Choosing appropriate sustainability indicators (SI) can be a complicated undertaking. The primary factor is determining the purpose of the indicators (*Lundin*, 2004), which depends, in large part, on the target audience; scientists may prefer a large set of indicators whereas the public and policy makers are usually only interested in headline indicators. A company internal survey may have a specific agenda or a narrow focus and indicators will be chosen on that premise, excluding possible harmful downstream effects. Ideally an SI should be:

- relevant to the end-users
- simple to understand
- based on reliable data that are relatively easy to collect
- predictive
- generic to allow comparison

Indicators can be classified under four categories: economic, environmental, social and technical. A comprehensive assessment of sustainability must consider all four categories. For example, a particular rainwater harvesting system may be employing proven technology, be financially viable and socially acceptable but be environmentally unsustainable. Table 2.1 shows a set of primary sustainability criteria developed during the UK-based SWARD (Sustainable Water Industry Asset Resources Decisions) Project (*Foxon et al.*, 2002). A criterion is a factor that is important in assessing the system in question (*Natural Resources Canada*, 2007). Each criterion may be associated with several indicators that can be used to evaluate change.

Table 2.1 Sustainability criteria developed by SWARD (*Foxon et al.*, 2002).

Economic	Environmental	Social	Technical
Life cycle costs	Resource utilization	Impact on risks to human health	System performance
Willingness to pay	Environmental impact	Acceptability to stakeholders	Reliability
Affordability	Service provision	Participation and responsibility	Durability
Financial risk exposure		Public awareness and understanding	Flexibility and adaptability
		Social inclusion	

Sustainability indicators generated by urban water models are an effective way to assess different water management options for a city. Indicators that are easily quantifiable such as system performance, life cycle costs, resource utilization and environmental impact are most suitable as outputs from such models. Other indicators like those falling under the social category are harder to quantify and so are less appropriate for output from an urban water model (*Makropoulos et al.*, 2008).

Examples of environmental sustainability indicators (ESI) are:

- Annual freshwater withdrawn / annual available volume
- Water use per capita per day
- Chemical and energy use for water supply
- Leakage (unaccounted water / produced water)

- Reused water
- Wastewater production per day
- Removal of biological oxygen demand (BOD), total phosphorus (TP) and total nitrogen (TN)
- Loads of BOD, TP and TN
- Chemical and energy use for wastewater treatment
- Amount of TP and TN recycled

These ESIs were used by *Lundin* (2004) in her case studies of the urban water systems of two towns (Goteburg, Sweden and King William's Town, South Africa) and can be categorised under the SWARD criteria of resource utilization, environmental impact and system performance. Alternatively, they could be classified as indicators for water flow, water quality, energy use and chemical use. With the inclusion of life cycle costs this list of indicators offers a strong quantitative assessment of sustainability. The author therefore suggests that the following sustainability indicators are important outputs from an urban water model:

- Water use
    - Freshwater abstraction
    - Per capita consumption
    - Water re-use and rainwater harvesting
    - Leakage
- Stormwater runoff
    - Stormwater harvesting

SUDS attenuation

Sewer flooding

- Wastewater discharge

  Water re-use

  Sewer exfiltration/infiltration

- Contaminant loads

  Loads discharged to the natural systems

  Loads to the WWTP

  Removal efficiency of treatment measures

  Nutrient recycling

- Energy use

  Embodied energy

  Operation and maintenance

  Decomissioning

- Chemical use

  For water supply

  For wastewater treatment

- CO_2 emissions

  Construction, operation and maintenance of water supply systems

  Construction, operation and maintenance of wastewater treatment systems

- Economic cost

  Capital cost

  Operation and maintenance

  Decomissioning

2.2 Urban Water Models

Urban water models can be used as decision-support tools that generate indicator outputs, allowing quantitative comparison of conventional and unconventional water management strategies for supply, wastewater and stormwater in an urban setting. Using urban water models, with sustainability indicator outputs, provides structure to the sustainability assessment process, allows examination of its components and their interactions, and enhances stakeholder (including researchers) communication (*Jakeman et al.*, 2006). In this section three types of urban water models are discussed: detailed, catchment scale, and urban scoping. Scoping models with system boundaries that include all of the important processes within the urban water cycle are particularly suitable for use as decision-support tools for integrated urban water management (IUWM).

There are a number of established models that address only sections of the water cycle in detail and do not provide the necessary information about other parts of the water system required for IUWM decision-making. For example, *Infoworks* (Wallingford Software, n.d.), developed by HR Wallingford (a company specialising in water management software), is widely used by the water industry. It consists of three models: *Infoworks RS*, *Infoworks CS* and *Infoworks WS*. These detailed models respectively cover the river system, the urban drainage system and water supply/distribution.

To address the need for the integration of several detailed models, covering different aspects of the urban water cycle, various linking software packages have been developed. However, most are not extended far enough to include the whole urban water cycle. For example, the US EPA's *BASINS* models runoff and stream quality by linking *HSPF, QUAL2E* and *TOXIROUTE* (*Whittemore et al.*, 2000), and several other examples exist for urban watersheds (*Rousseau, et al.*, 2005). Problems with differences in time scale and data type, and poor model accuracy are typical when linking several subsystem models (*Schmitt, et al.*, 2005), and research is now focussing on developing better 'integration' modules to improve linkages (eg. *Maheepala et al.*, 2005). In addition, the requirement for the use of several models, albeit linked, to gain the complete picture of the urban system may be data intensive and involve a steep learning curve for the user, because (s)he must learn how to use several different models. Several model packages are reviewed below.

SMURF (Sustainable management of Urban Rivers and Floodplains) was a three year EU Project completed in 2005. The main aim of the project was to "develop and disseminate a new methodology for improved land-use planning and water management in heavily urbanised and degraded environments" (*EU Life Programme,* 2005). It was tested in the River Tame catchment and used a combination of three established models: *SIMCAT*, *Infoworks RS* and *Infoworks CS*. The *SMURF* software outputs results for what-if scenarios in GIS format for ease of interpretation. However, because *SMURF* was designed to model river corridors, other aspects of the urban water cycle such as water usage and wastewater are outside its scope.

Hydro Planner is an Australian systems model under development by CSIRO (*Maheepala et al.*, 2005). The purpose of the software is to link existing water models to give urban water planners a better understanding of the interactions between water, wastewater and stormwater systems and with natural systems, in terms of water and contaminant flows, at city and regional scale. It consists of seven modules:

1) Catchment module – supports linking of models that can simulate contaminant and runoff generation from supply catchments

2) Water supply module – supports linking of models that simulate water supply system behaviour (The Resource Allocation Model (*REALM)* – represents the supply system as nodes (demand centres and supply sources) and links (trunk mains, waterways and rivers). It is designed to operate on a monthly timestep. Data requirements are monthly time series of inflow, rainfall and evaporation at source nodes, monthly time series of demand at demand nodes, capacities of links and operating rules as input data). Since a daily timestep was chosen for modeling water quality in *Hydro Planner* results from *REALM* are disaggregated to daily values.

3) Consumption module – supports linking of models that can simulate urban water consumption (Water Services Association's End Use Model, EUM). *Hydro Planner* is supplied with a monthly time series of water consumption from *EUM* for a specific area.

4) Stormwater module – supports linking of models that can simulate stormwater and associated contaminant generation and routing processes.

5) Wastewater Module - supports linking of models that can simulate wastewater and associated contaminant generation and routing processes.

6) Receiving water module – supports linking of models that can simulate flow and contaminant routing through a stream network.

7) Integration module – supports translation and computation of input/output data between modules and Graphical User Interface (GUI).

The strength of this software is its comprehensive coverage of urban water volume and contaminant flows. Weaknesses are that it requires very many input data, which is unsuitable for a scoping model, and energy and cost are not assessed.

The *eWater Co-operative Research Centre* (*Ewater,* 2009) is an Australian technology development initiative funded by the water resource management and research sector. Current research into urban water systems involves the development of a decision support tool which, integrating the use of eWater's *Water Cast* and *River Manager* tools, aims to simulate:

- Centralised and decentralised urban water technologies that evolve over time
- Risk, uncertainty, optimisation and multi-criteria analysis for urban and rural water

Advantages of the *eWater* approach include: a rigorous assessment of dynamics at different spatial scales within the urban water system, including the natural systems; a sophisticated demand module; risk and uncertainty; and an interface with receiving water modeling tools. Limitations of the *eWater* approach include the lack of quantification of energy consumption.

The purpose of Sweden's *Sustainable Urban Water Management (SUWM)* Programme, which ended in 2005, was to assess future urban water management options for water and wastewater. It was a six year project run by the Swedish Foundation for Strategic Environmental Research (*MISTRA*). The output is a sustainability assessment toolbox including *URWARE* (wastewater treatment plant model - *Jeppsson et al.*, 2005), *SEWSYS* (sewer system model - *Ahlman et al., 2005*), a cost estimation tool, and microbial and chemical risk assessment. Strengths of the *SUWM* Programme models include the consideration of the whole urban water system, the inclusion of organic household wastes, risk analysis, and energy and exergy quantification. Exergy is the maximum useful work possible during a process that brings the system into equilibrium with a heat reservoir and is often considerably less than the available energy as a result of inefficiencies in the process (*Yantovski*, 2004). However, although methodologies are provided for the integration of the results from the different models in *SUWM*, further work and a detailed understanding of the process is required to implement them.

The above review has highlighted the focussed nature of many of the developed linking tools and their limitations. Most are not extended far enough to include the whole urban water cycle and there are problems with differences in time scale and data type, and poor model accuracy. In addition, the requirement for the use of several models may be data intensive and involve a steep learning curve for the user, because (s)he must learn how to use several different models.

In recent years a number of urban water scoping models have been developed, each with a slightly different focus and consequently strengths and weaknesses. Key factors in assessing the worth of an urban water scoping model are: ease of use, complexity of input data requirements, difficulty in accessing input data, program run-time, accuracy of outputs, and scope of outputs. The ideal scoping model would: be straight-forward to use, have relatively low data input requirements with easily accessible data, have a rapid computer run-time, results that are accurate enough to inform decisions for further detailed modelling and output a wide range of sustainability indicators. Several examples of stand-alone urban water scoping models are reviewed below.

UWOT (Urban Water Optioneering Tool) is a daily-time step model that was developed under the Water Cycle Management for New Developments (WaND) Project (*Makropoulos et al.*, 2008) jointly funded by the Engineering and Physical Sciences Research Council (EPSRC) and industry. It outputs several sustainability indicators for urban residential areas, allowing exploration of indoor water efficiency options and a generic sustainable urban drainage system (SUDS) option. The model is Excel based and powered by MATLAB/Simulink. The main strengths of the model are its simplicity and the range of indicator outputs which include cost, energy, water use, wastewater flows and stormwater flows. The main weaknesses are that it is confined to residential landuse, is limited in the breadth of out-of-house water management options and the natural systems (soil stores, groundwater and surface flow) are outside the system boundary.

UrbanCycle (*Hardy et al*., 2003) is an Australian IUWM model that includes the main aspects of the urban cycle: water supply, wastewater, and stormwater. It was developed as a response to the need to revisit IUWM management tools in the light of technology advance and increased computational capacity. Using a hierarchical description of the cityscape it allows users to explore different alternative water management strategies (rainwater harvesting and wastewater recycling) and compare them with traditional, large-scale centralised systems. *UrbanCycle* uses simulated rainfall at a 6 minute time step and diurnal water consumption. The strengths of *UrbanCycle* are its description of the cityscape and ability to simulate peak flows. The main weaknesses are that it only addresses water flow, it has a limited number of water management options, and the short time step, which is good for simulating peaks, is less appropriate for application to large areas over longer time periods.

The Australian water balance program *Aquacycle* (*Mitchell et al.*, 2001) is freely available from < www.toolkit.net.au/tools>. It performs a daily water balance with various water recycling options available to test their effect on the primary indicators – mains water used, stormwater runoff and wastewater emissions. The alternative strategies are raintanks, cluster stormwater systems, catchment stormwater systems, subsurface direct greywater irrigation, aquifer storage and wastewater recycling at unit, cluster and catchment level. The basic model works along similar lines to *UrbanCycle*. Within the model catchment are various smaller areas called clusters whose boundaries are chosen based on land use and the sewer network. Each cluster consists of a number of identical unit blocks with a water demand profile and percentage of pervious space. *Aquacycle*'s successor, *UVQ* (Urban Volume and Quality) *(Mitchell & Diaper*, 2005) adds a contaminant balance to the water balance as well as improving the user interface and adding snow modelling capability. The main strengths of *Aquacycle/UVQ* are their simplicity, rapid runtime and description of the cityscape. The main weaknesses are that their primary focus is residential landuse, the narrow range of indicator outputs, limited number of water management options and limited consideration of the natural systems.

Urban Developer (*eWater*, 2009b) is an urban water scoping model, currently under development, that uses a hierarchical representation of the cityscape like *Aquacycle* but offers a more sophisticated demand module. Since this is a commercial tool, in its development phase, more details are not available.

The Australian model *WaterCress* (*Cresswell et al.*, 2002) is freely available from < http://www.waterselect.com.au/watercress/watercress.html>. It is primarily designed to assess the feasibility of a range of conventional and unconventional supply options. It works on very similar principles to *Aquacycle* but is intended to be used at river basin scale and not just as an urban model. This gives the model the flexibility to look at the water cycle on a broader scale. By including the whole catchment the city's boundary conditions will be better constrained and the natural systems are included. However, if the user-focus is urban water then the additional complexity in describing the catchment may be prohibitive. Principal indicators are reliability of water supply, water quality and average cost. There is no consideration of energy usage.

Fagan et al. (2007), based at the University of Melbourne, have developed an urban water scoping model, powered by MATLAB/Simulink (Mathworks Inc., 2007), that uses a combination of material/energy balance and kinetics to compare alternative water management strategies with a range of sustainability indicators:

- Water volume
- Water quality
- Other materials (reagents, infrastructure materials, greenhouse gases, sludge)
- Energy consumption (e.g. solar, electricity, embodied energy in infrastructure)
- Economic cost (capital and operating)
- Environmental impact (e.g. greenhouse gases, effluents, infrastructure materials)

The model's system boundary includes water supply, water consumption (residential, commercial and industrial), stormwater runoff and treatment, sewerage and wastewater treatment/recycling. As a case study, the model was applied to Aurora, an innovative Melbourne development consisting of 8500 houses, a local sewage treatment plant and recycled water treatment plant, with dual reticulation for potable and recycled water, and onsite stormwater treatment. The range of indicators covered by this model is extensive. However, there is a lack of consideration of the natural systems: rivers, lakes, canals and groundwater. One example of the importance of including natural systems within the scope of an urban water model is the siting of Sustainable Urban Drainage Systems (SUDS), which requires knowledge of groundwater level.

Jia et al. (2002) developed a spatially distributed model that calculates water and energy balance, using an hourly timestep, for catchments that include urban land use. The main focus of the model is the rainfall-runoff component at the expense of the anthropogenic components. The use of the hourly timestep allows more detailed analysis of stormwater peak flows than a daily timestep but still becomes demanding on computer memory for long time series. The energy balance is based on natural processes and is not used as a sustainability indicator for alternative water management options.

There are models that use a scoping approach at catchment scale such as *WaterStrategyMan* (*Assimacopoulos*, 2004), Aquatool (*Solera*, n.d.) and *Systems Modelling Rio Grande* (*Pasell et al.*, 2002). These usually treat the urban centres in a very simplified way as demand nodes and so are unsuitable for application at the urban scale.

WaterStrategyMan is a decision support system (DSS) (*Assimacopoulos*, 2004) designed to assess different water management plans for water scarce regions. It outputs cost, environmental, efficiency and resource use indicators. The basic structure of the program consists of nodes and links. The nodes are supply (surface, groundwater and storage), demand (consumptive and non-consumptive) and transhipment (junctions and treatment plant) and the links are man-made or natural conduits. Links introduce two constraints – maximum flow capacity and measured link monthly flow rates. The program operates on a monthly time step that finds a solution for different management priorities:

- minimises the storage
- demand constraints
- supply constraints
- flow conservation constraints
- capacity constraints

The program has been developed in Visual Basic and uses ArcGIS as its display software. The low data requirements, which is the primary strength of this scoping model for catchment analysis, means that it does not provide sufficient analysis at the urban scale.

Aquatool (*Solera*, n.d.), developed by the Universidad Politecnica de Valencia, Spain, is a GIS-based DSS for water resource planning and operational management at the watershed scale. It is used by several Spanish River Basin Agencies. The model consists of nodes with storage capacity (lakes, reservoirs), diversion and junctions, natural channels, aquifers, precipitation, evaporation and infiltration losses. It also covers water uses such as irrigation, municipal and industrial supply and hydroelectric plants. The module, SIMGES, simulates the system on a monthly timestep. OPTIGES, the optimisation module, minimises the weighted sum of water demand deficits and minimum flows using conservation of mass. There is also a tool for water quality assessment at each node and stream in the basin. The strengths of this model are the inclusion of the natural systems, relatively light data requirement and optimisation module. However, there is no consideration of energy and cost.

Systems Modelling RioGrande (*Pasell et al.,* 2002) is a system dynamics model based on mass balance of flows at the river catchment scale (including groundwater). Its purpose is to assess alternative water management strategies in terms of cost and water volume in the Rio Grande Basin, New Mexico. Operating on an annual timestep, modelled flows include precipitation, evaporation, surface flow, sewage return flows (assumed equivalent to total indoor water use for residential, commercial and industrial use), leakage, groundwater inter-basin flow and recharge. Municipal demand is based on per-capita demand for four categories of land use: residential, commercial, industrial and institutional. These are further split into indoor and outdoor demand. Account is also taken of leakage assumed as 10% of total demand. A simple annual population growth algorithm is used. The strength of the model is its consideration of the catchment and reduced data requirement since it operates on an annual timestep. It also gives stronger consideration to different aspects of the urban landscape than many catchment models which just treat it as a demand node. However, by modelling the catchment, the level of description of the urban area is still not detailed enough for the needs of an urban water scoping model. The annual timestep is too large to assess seasonal patterns and there is no consideration of energy use or contaminant flows.

In this section examples of three different types of models that can be applied to the urban environment have been discussed and are summarised in Table 2.2. It has been shown that the major limitations of detailed models are their limited scope and heavy data requirements. Linked detailed models have a wider scope but can suffer from integration problems arising from differences in time scale and data type and poor model accuracy.

Catchment scale scoping models have the advantage of wide system boundaries but do not provide sufficient analysis of the urban water system at the sub-city scale. Urban water scoping programs strike a balance by modelling city-scale dynamics that include all of the important processes within the urban water cycle but in less detail than more focussed models. This makes them particularly suitable for use as high level sustainability assessment tools. Key areas identified for improvement on existing urban water scoping models are:

- Better representation and sustainable exploitation of the natural systems
- Wider range of alternative water management options including sustainable urban drainage systems (SUDS)
- Calculation of life cycle energy use and whole life cost

Table 2.2 Summary of Urban Water models reviewed

Software	Type	System boundary	Strengths	Weaknesses
Infoworks	Detailed	Either supply, collection or river networks	Very accurate modelling	Data intensive, narrow focus
Basins	Linked	Urban water shed	Inclusive of main urban water processes	Problems with linking (differences time scale, data type and poor model accuracy). Steep user learning curve
SMURF	Detailed	River catchment (including urban)	Addresses natural systems in urban environment	Narrow focus (no water demand and wastewater model)
Hydro Planner	Linked	River catchment (including urban)	Comprehensive coverage of urban water volume and contaminant flows	Heavy data requirement. Lack of consideration of cost and energy
eWater's Water Cast and River Manager	Linked	Urban water shed	Addresses natural systems in urban environment. Includes major water flow processes	Possible problems with linking. User must learn to use two models. Energy consumption not assessed
SUWM's toolbox	Urban scoping	Urban water shed	Wide system boundaries. Assessment of wide range of indicators	Steep learning curve. Models are not linked.
UWOT	Urban scoping	Residential neighbourhood	Easy to use. Good range of indicators. Can assess water efficiency measures	Narrow system boundary. Very limited consideration of natural systems
UrbanCycle	Urban scoping	Urban water shed	Includes main urban water flow processes. Easy to use. Ability to simulate peak flow	Only models water flow. Limited number of water management options. 6 minute timestep less applicable at city scale
Aquacycle/ UVQ	Urban scoping	urban water shed	Easy to use. Rapid runtime. Description of cityscape	Primary focus residential landuse. No consideration of energy or cost. Limited consideration of natural systems and small range of WM options
Urban Developer	Urban scoping	Urban water shed	Builds on Aquacycle	Commercial model under development - no further details available.
WaterCress	Urban scoping	Urban focus but extending to river catchment	Wide system boundaries. Inclusion of natural systems	Additional complexity of modelling at river catchment scale. No consideration of energy use
Fagan's model	Urban scoping	Urban water shed	Extensive range of indicators	Very little consideration of the natural systems
Water StrategyMan	Catchment scoping	River catchment	Low data requirements. Rapid runtime. Range of indicators	Long timestep and resolution insufficient for modelling at the sub-city scale
Aquatool	Catchment scoping	River catchment	Inclusion of natural systems. Light data requirement	No consideration of energy or cost. Poor resolution at sub-city scale
Systems Modelling RioGrande	Catchment scoping	River catchment	Inclusion of natural systems. Light data requirement. More detail at urban scale than most catchment scoping models	Annual timestep too large to assess seasonal patterns. No assessment of energy or cost

2.3 Life Cycle Assessment

Life cycle assessment (LCA) is the "compilation and evaluation of the inputs, outputs and the potential environmental impacts of a product system throughout its life cycle" (*LCA Center*, n.d.). LCA using indicator output is one of the best techniques for assessing the environmental load of a system (*Lundin*, 2004). This is, in large part, a result of its comprehensive nature. In the past the tendency has been to concentrate on one aspect of the water system at a time which has lead to problem shifting rather than elimination. For example, the introduction of urban sewer systems lead to a significant improvement in public health but also a rapid degradation of local water bodies, the final destination for the water (*Butler & Memon*, 2006) LCA allows a wide system boundary to be defined and so all effects of a decision can be seen. Figure 2.1 illustrates three possible boundaries in the assessment of urban water systems (*Lundin*, 2004). Boundaries 1a and 1b are commonly used by engineers to compare different processes but are too narrow to assess the sustainability of an urban water system. Boundary 2 is often defined by an organisation for benchmarking and reporting purposes. It has much wider scope than level 1 but frequently drinking water is separated from sewage and stormwater and analysis may only be to satisfy legislation rather than a proactive search for sustainable solutions. At the third level the upstream/downstream effects of the urban water system are also considered, enabling a much more complete assessment of sustainability.

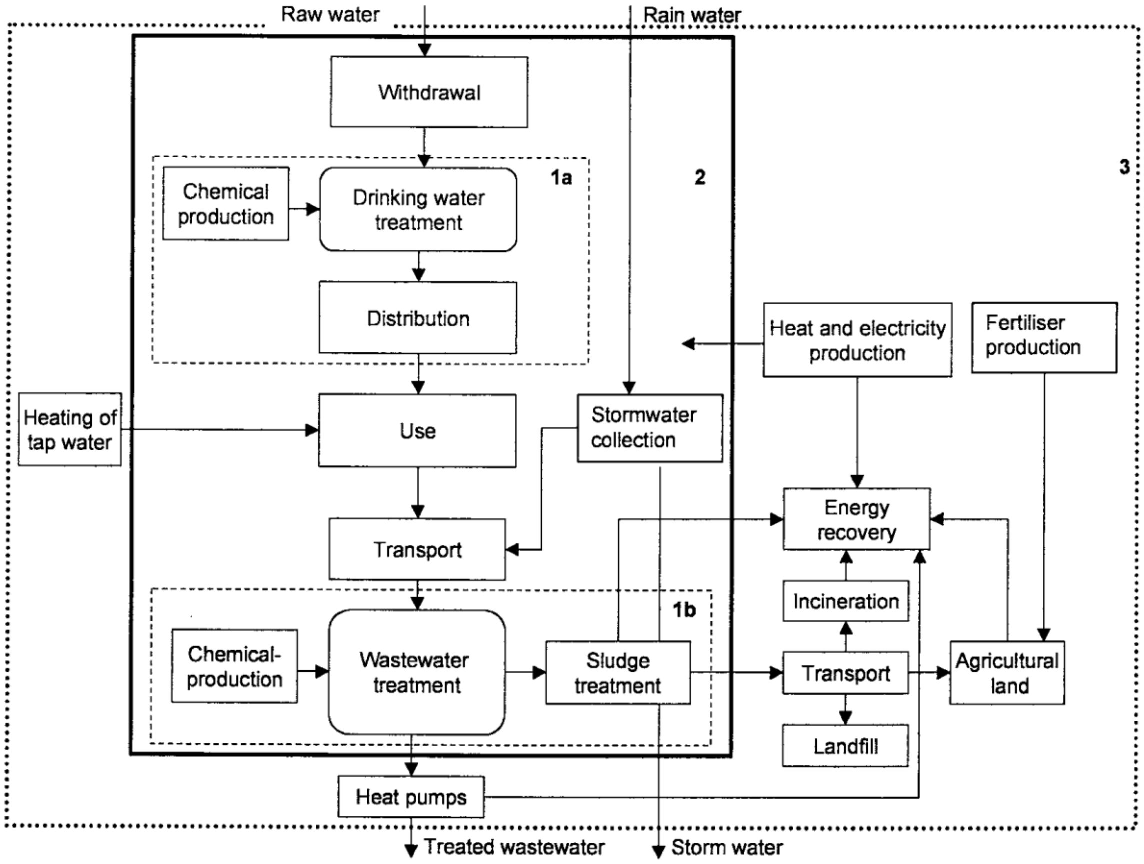


Figure 2.1 Overview of system boundaries for assessing sustainability of urban water systems. Level 1: process-defined, level 2: company-defined, level 3: extended including surrounding system (*Lundin*, 2004).

A number of detailed lifecycle cost and energy studies have been conducted in recent years investigating a range of conventional and unconventional technologies in urban water management. However, as a result of the large variety of technology options available and the data intensive nature of full life-cycle assessment they have generally focussed on particular aspects of the water cycle and have focussed solely on environmental indicators. A selection of these is reviewed below.

Recently there have been several studies to assess the environmental impact of the centralised urban water and wastewater systems. *Rihon et al. (*2002) conducted a LCA study of production and distribution of drinking water and the collection, treatment and disposal of wastewater for the hydrographic Basin of La Vesdre in Belgium. It was found that pumping and wastewater treatment are the main contributors to environmental impact. Of the wastewater treatment processes, secondary and sludge handling are the most damaging. *Lundie* (2004) carried out a prospective LCA to examine potential environmental impacts of Sydney Water's total operations in the year 2021. Strategies considered were demand management, energy efficiency, energy recovery from biosolids, seawater desalination and upgrading coastal sewage treatment plants to secondary and tertiary treatment. Costs were not considered. *Friedrich et al.* (2006) completed a South African case study of the environmental life cycle of water (abstraction, treatment, distribution, collection, treatment and disposal, including recycling) in the eThekwini Municipality. Key results were that the greatest environmental burden was placed by wastewater treatment, followed by distribution of potable water and then collection of wastewater. Recycled water for industry, located near the wastewater treatment plant (WWTP), had lower impacts than mains water. In this case study the process that carried the highest environmental burden was the use of electricity for pumping. The economic cost of water was not considered.

Several studies have been performed to examine the environmental impacts of decentralised technologies compared with centralised. *Van der Hoek et al.* (1999) considered three options for a new sustainable housing estate in Amsterdam: rainwater harvesting, greywater recycling and a dual supply system using surface water. Rainwater harvesting and greywater recycling had the highest life cycle impacts, principally as a result of energy use. *Tarantini and Ferri* (2001) applied the LCA methodology (under the *AQUASAVE* Project) to the domestic water cycle of Bologna. The aim was to assess water saving potential of water conservation measures, rainwater harvesting and greywater recycling in residential buildings. A key result was that 65% of energy used in the system was for water pumping, which is in agreement with *Friedrich et al.* (2006).

LCAs of centralised supply include those by *Friedrich* (2001), *Raluy* (2005) and *Landu and Brent* (2006). *Friedrich* (2001) investigated, in her South African study, the production of potable water using two different technologies: conventional and membrane filtration technology. For both methods electricity consumption was the most significant contributor to environmental impacts. *Raluy* (2005) assessed common desalination technologies: Reverse Osmosis (RO), Multi Effect Desalination (MED) and Multi Stage Flash (MSF) in the context of the Ebro River Water Transfer Project. It was found that the main contribution to the environmental impact of desalination technologies came from the operation phase, while, construction and disposal, are almost negligible by comparison. RO is the least energy intensive desalination technology (one order of magnitude lower than the thermal processes, MSF and MED). *Landu and Brent* (2006) performed a LCA of water supply to the Rosslyn industrial area, in the city of Tshwane, South Africa. The study included extraction from the River Vaal, treatment and pumping. The main conclusion of the study was that reducing water loss, and therefore extraction volume, was the best way to reduce environmental impact.

LCA of centralised wastewater treatment includes those by *Emerson* (1995), *Tillman et al.* (1998), *Zhang and Wilson* (2000), *Gaterell and Lester* (2000), *Dixon et al.* (2003) and *Hacker* (2007). *Emerson* (1995) found that biological filter plants use 56% less energy than activated sludge plants and produce 35% fewer airborne emissions, but are more land intensive. *Zhang & Wilson* (2000) performed a LCA for a large WWTP (164,000m^3/day) in southeast Asia. It was found that energy was the main contributor to environmental burdens and that over 70% of energy use was in the operational phase. In addition, economies of scale were highlighted: energy use of the plant investigated by *Zhang & Wilson* (2000), serving a population equivalent (p.e.) of 800,000, was about 25% of the usage of the small-scale plant, serving a p.e. of 1000, investigated by *Emerson* (1995). Both plants used activated sludge treatment. *Gaterell and Lester* (2000) carried out a full LCA (construction, operation and demolition) of three different configurations of a WWTP. Impacts considered included global warming potential (GWP), acidification, and ozone depletion. The operation stage was found to contribute 70-90% of impacts for all three treatment options. *Dixon et al.* (2003) performed an environmental assessment of two wastewater treatment processes: reedbed and aerated biological filter. CO_2 emissions calculated for the two processes included production and transportation of construction materials and operational energy. Results showed that the reedbed required less operational energy but that vehicle emissions for reedbed maintenance were significant (up to 20% of life cycle CO_2 emissions). Economies of scale were found for the aerated filter; *Dixon et al.* (2003) found that impacts per unit treated were reduced for larger filters. This was not observed for the reedbed treatment. *Hacker* (2007) created and applied a carbon accounting methodology (construction and operation only) to Blackminster Sewage Treatment Works operated by Severn Trent Water, serving a p.e. of 12,371. It was found that 77% of CO_2 emissions were associated with operation and 23% with construction. The largest contributor, to emissions in operation, was the generation of electricity, with blowers, for the oxidation ditches, using 80% of the plants electricity requirement. In contrast, the study by *Vidal et al.* (2002), comparing the environmental impacts of structural changes to an activated sludge WWTP, with the aim of reducing nitrogen emissions, found that the oxidation ditch was a better option than the Ludzack-Ettinger configuration. This highlights the subjective nature of sustainability assessment when indicator weighting is used because if greater weight is given to a particular indicator it can affect the overall result.

Various studies have been conducted that compare decentralised stormwater re-use and wastewater recycling. *Crettaz et al. (*1999) found that rainwater harvesting has a higher environmental impact and economic cost than centralized supply. It only became favourable when electricity costs for water treatment were high, which is corroborated by a LCA study conducted by *Hallmann & Grant* (2003). Use of a low-pressure, energy efficient pump is crucial in reducing the impact of a rainwater harvesting system. *James* (2003) used LCA to compare the use of raintanks with conventional supply. Two tanks sizes were used: one for toilet flushing and garden watering and the other solely for garden watering. Results showed that mains water consumption could be reduced by 30% and 8% respectively and that the sum of embodied and operational energy use, by raintanks, was greater than that used by the conventional system. However, it was considered that the scales of energy and material impacts were small in comparison to overall Australian energy use and that the water savings were considerable. Thus raintanks were advocated, in contrast to the result by *van der Hoek et al.* (1999) who weighted the energy impacts more strongly. *Mithraratne and Vale (2008)* simulated life cycle energy usage, CO_2 emissions and economic cost of various rainwater harvesting strategies applied to Auckland, New Zealand. They found that only if raintanks have long lifetimes (>50 years) is the life cycle energy less for raintanks than reticulated supply. Regardless of tank lifetime, the life cycle energy use of plastic tanks was considerably greater than that of reticulated supply. It was found that the use of concrete and plastic raintanks, in conjunction with demand management, results in economic savings, over the use of mains supply, after 12 and 20 years respectively.

Tillman et al. (1998) carried out a LCA of the environmental impact of two wastewater treatment plants (WWTP) serving population equivalents of 900 and 12,600. It was found that, for the smaller WWTP, urine separation, followed by anaerobic digestion and agricultural application, scored the best. For the larger WWTP the results did not significantly favour any technology over another. Similarly, *Lundin et al.* (2000) carried out a LCA to compare the environmental loads from different wastewater treatment processes. Conventional systems (sedimentation, chemical treatment, sludge handling) at different scales (72,000 p.e. and 200 p.e.) were compared with urine separation systems (72,000 p.e.) and liquid composting of black water (200 p.e.). Large economies of scale, in environmental terms, were found for both the construction and operation phase. In comparison to conventional treatment, separation systems showed lower pollutant emissions to water and more efficient nutrient recycling to agriculture. The implication of this would be significant reduction in the need for fertilizer production which would reduce the use of energy and phosphates. Environmental impacts of construction of the large scale treatments were assumed to be small compared to the operational impacts and so were not measured. However, for the small-scale treatments the impacts of construction were found to be significant. This is in agreement with the findings of *Tillman et al.* (1998). Electricity demand per functional unit is about four times greater for the small-scale systems compared to the large-scale systems in Lulea. Liquid composting has the highest electricity demand because of the vacuum transport and aeration of the liquid. *Memon et al.* (2007) examined the life cycle impact of four greywater treatment technologies: reed beds, membrane bioreactors (MBR), membrane chemical reactors (MCR) and a green roof recycling system (GROW). Materials and energy use (in construction and operation) were quantified for each technology at 20 development scales. Impact assessment was performed using the LCA software *SimaPro* (available from: <

http://www.pre.nl/simapro/>). Results show that technologies based on natural treatment processes (GROW and reed beds) have the least environmental impact. Transport was ignored since it was assumed to be the same for all four technologies and also to enable the results to be applicable elsewhere, regardless of location. The waste disposal phase was ignored since it was deemed to constitute a LCA in its own right. Importantly, *Memon et al.* (2007) found only a marginal decrease of environmental impact with a service increase from 500 to 1250 households.

There has been little research into the life cycle energy costs of SUDS. *Kirk* (2006) conducted a life cycle analysis of Best Management Practices (BMP) under evaluation at a BMP performance verification centre in New England, USA. The BMPs examined were sub-surface treatment and storage unit, retention pond, bioretention cell and subsurface-flow gravel wetland. Net present value (NPV) costs over a 30 year lifetime were respectively ($64,194; $43,263; $48,731; $43,383). Energy cost was not reported explicitly but as resource use (electricity, coal, natural gas etc.). Conversion of reported values yielded the following annual results, respectively: 5315 kWh, 3548 kWh, 3769 kWh and 3261 kWh.

There has been little work on the whole life cost (WLC) of rainwater harvesting systems (*Roebuck*, 2008) and greywater recycling WLC in the UK (*Memon et al.*, 2005). On the other hand life cycle costing of sustainable urban drainage systems (SUDS) is well documented (*Woods-Ballard et al.*, 2007).

Memon et al. (2005) calculated the WLC of a small-scale greywater recycling system in Maidenhead, UK. The initial capital cost was estimated to be £1625. If a conservative lifetime of 25 years is assumed (*McEvoy*, 2008), with annual supply of 31 m^3

(*Memon et al.,* 2005), this is equivalent to £2.1/m^3. The annual operation & maintenance cost was estimated to be £2.7/m^3, resulting in a total WLC of £4.8/m^3. The cost of rainwater harvesting systems were found to be in the range £0.29 – 2.32 /m^3, excluding capital costs (*Roebuck*, 2008 after *Brewer et al.,* 2001). *Baker* (2003) estimated that a borehole abstraction scheme abstracting 0.6 Ml/day from a minor aquifer in North Staffordshire would have NPV capital costs of £146,973 (adjusted at 4.5%/yr to 2010). The annual operation and maintenance cost and license fee from the Environment Agency (EA) was estimated to be £23,407(*Baker*, 2003). If the lifetime of the borehole is assumed to be 50 years (*Ecozi*, 2009) then over its lifetime the borehole will supply 10,950 Ml. This results in a NPV WLC of 0.07 £/m^3 supplied. If the lifetime of the borehole scheme is reduced to 30 years then the WLC increases to 0.09 £/m^3. It is clear that there are significant economies of scale and that groundwater abstraction is much more viable for commercial use. *Groundworks* (2004) estimated the capital cost of green roofs to be £74/m^2 (increased to 2010 value using 3.5% rate) or £7400 for a roof area of 100 m^2. Over a lifetime of 50 years with assumed 100 m^3/yr attenuation the capital cost is £1.48/m^3. Since maintenance for extensive green roofs is minimal this value is likely to be similar to the whole life cost.

The rigorous nature of life cycle assessment means that heavy data requirements and subsequent processing are limiting factors. Consequently, LCA studies often adopt a narrow focus, in terms of the number of technologies and the range of indicators considered. Many of the studies reviewed above have focussed on only one technology and have usually considered either environmental or cost indicators but not both. Further, it can be seen that there has been little work done assessing the life cycle energy use of SUDS. The opportunity therefore presents itself for the development of a scoping tool that is less data intensive, enabling a wider focus, which covers calculation of both life cycle energy and cost of a range of water management options, including SUDS, within the bounds of a single program. Simplified life cycle inventory for energy use and simplified whole life cost are suitable techniques for such a program. These derivative methods focus on key data only, identified through the use of basic sensitivity analysis for example, and make use of generic data where possible, such as for transportation and energy supply (*Friedrich et al.*, 2006). By so doing data requirements and processing can be greatly reduced, allowing an expansion of the study scope to include a broader range of water management options and a more complete set of sustainability indicator outputs. Table 2.3 summarises the LCA and WLC studies that have been reviewed.

Table 2.3 Summary of LCA and WLC review.

Authors	Date	Case study area	Scope	Key Findings
LCA				
Rihon et al.	2002	La Vesdre, Belgium	Centralised production and distribution of drinking water. Treatment and disposal of WW	Highest env. burden was from pumping and WW treatment
Lundie	2004	Sydney	Sydney Water's total operations	Alternative centralised strategies assessed. Demand management assessed. Costs not considered
Friedrich et al.	2006	eThekwini Municipality, South Africa	Centralised abstraction, treatment/distribution, collection/disposal, recycling	Highest env. burden from pumping. Cost not considered
Van der Hoek et al.	1999	Amsterdam, Holland	Housing estate	Considered RH, Greywater recycling and dual supply using surface water
Tarantini and Ferri	2001	Bologna, Italy	City scale	Investigated water conservation, RH, Greywater recycling in residential buildings. 65% of energy used for pumping
Friedrich	2001	South Africa	Potable water treatment	Compared conventional and membrane filter technology. Highest env. burden from electricity consumption
Raluy	2005	Ebro River, Spain	Desalination technology	Highest env. burden from operation phase. Reverse Osmosis less energy intensive than thermal processes
Landu and Brent	2006	Tshwane, South Africa	Extraction, treatment and supply to Rosslyn industrial area	Reduce leakage and therefore extraction volume best way to reduce env. impact
Emerson	1995	Hypothetical	Comparison of two WW treatment technologies	Biological filter plants use less energy and produce fewer emissions than activated sludge but are more land intensive.

Zhang and Wilson	2000	Southeast Asia	Large, centralised WWTP	Highest env. burden from operation phase. Significant economies of scale evident
Gaterell and Lester	2000	Hypothetical	Three configurations of centralised WWTP	Highest env. burden (70-90%) from operation phase.
Dixon et al.	2003	Hypothetical	Comparison of reedbed and aerated biological filter	Reedbed required less operational energy but had significant vehicle emissions for maintenance. Economies of scale were demonstrated
Hacker	2007	Blackminster Sewage Treatment Works, West Midlands	Carbon accounting of construction and operation of WWTP	77% of CO2 emissions from operation. 80% of operational electricity used by blowers for the oxidation ditches
Vidal et al.	2002	Hypothetical	Alterations to activated sludge WWTP to reduce TN emissions	Oxidation ditch found to perform better than Ludzack-Ettinger configuration
Crettaz et al.	1999	Hypothetical	Compare RH with centralised supply	RH higher env impact and economic cost. Only favourable when electricity costs for water treatment are high
Hallmann and Grant	2003	Hypothetical	Compare RH with centralised supply	RH higher env impact and economic cost. Use of energy efficient pump crucial for RH
James	2003	Hypothetical	Compare RH with centralised supply	Total energy use by RH greater. However in Australian context water savings considered more important
Mithraratne and Vale	2008	Auckland, New Zealand	Compare RH with centralised supply	Concrete RH more energy efficient than centralised only if tank life greater than 50 years. Plastic never. Water demand management more effective than RH
Tillman et al.	1998	Sweden	Compare two WWTPs (p.e. 900 and 12,600)	For smaller WWTP urine separation scored best. Larger WWTP no technology significantly better
Lundin et al.	2000	Lulea, Sweden	Compare conventional system with urine separation system (p.e. 72,000) and with liquid composting of black water (p.e. 200)	Separation systems showed lower emissions to water and more efficient nutrient recycling. Env impacts of construction only became important for the small WWTPs. Electricity demand per funcitonal unit 4 times greater for small WWTPs
Memon et al.	2007	Hypothetical	Comparison of four greywater treatment technologies	Natural treatment processes had lowest env impact (reed beds and GROW). Found only marginal decrease in env impact with increase of p.e. from 500 to 1250.
Kirk	2006	New England, USA	LCA and NPV of four BMPS	Subsurface treatment and storage was found to have the highest economic and energy cost. Retention ponds, bioretention cells and subsurface flow gravel wetland were found to have similar energy and economic cost
WLC				
Memon et al.	2005	Maidenhead, UK	Smal-scale greywater recycling	Total WLC of £4.8/m3 treated
Brewer et al.	2001	Hypothetical	Small-scale RH	Operational costs in range £0.29-2.32/m3
Baker	2003	North Staffordshire	Borehole abstraction (0.6 Ml/day)	WLC of £0.07/m3
Groundworks	2004	Hypothetical	Green roofs	Capital cost of attenuation is £1.48/m3.
Woods-Ballard et al.	2007	Hypothetical	SUDS	CIRIA Report 697 includes calculations of whole life cost of SUDS

2.4 Summary

Urban water scoping models, using sustainability indicator outputs, can be used as decision-support tools in moving towards the vision of integrated urban water management (IUWM), allowing the comparison of conventional and unconventional water management strategies for supply, wastewater and stormwater in an urban setting.

Based on reviews of sustainability indicators by *Lundin* (2004) and *Foxon et al.* (2002) the following headline sustainability indicators are suggested as important outputs from quantitative urban water models:

- Water use
- Stormwater runoff
- Wastewater discharge
- Contaminant loads
- Energy use
- Chemical use
- CO_2 emissions
- Economic cost

There are a number of established models that address sections of the urban water cycle in detail and in doing so do not provide the necessary information about other parts of the water system required for IUWM decision-making.

The difficulties involved in the development of software packages (*Hydroplanner, SUWM, SMURF, Basins*) that link the outputs from several detailed urban water models include problems with differences in time scale and data type, poor model accuracy, data intensive input requirement and a steep learning curve for the user.

A review of three catchment scale scoping models (*WaterStrategyMan*, Aquatool and *Systems Modelling Rio Grande)* demonstrated that they do not provide sufficient analysis of the urban water system at the sub-city scale.

Review of stand-alone urban water scoping models (*UWOT, UrbanCycle, Aquacycle*, *UVQ*, *WaterCress*) has identified key areas for improvement: better representation of natural systems in the urban environment, wider range of water management options and assessment of cost and energy indicators.

Case studies using full life cycle assessment have tended to focus on only parts of the urban water system as a result of the data intensive nature of LCA and it is concluded that the use of simplified LCA is more appropriate for incorporation into a scoping model because of its lighter data requirement.

3 - Model Concepts

City Water Balance (CWB) is a model that has been developed as the cornerstone of this research and as part of the SWITCH (Sustainable Water Improves Tomorrow's Cities' Health) Project. It has been designed to improve on contemporary urban water scoping models in three key areas that were identified in the literature search:

- Better representation and sustainable utilization of the natural systems
- Wider range of alternative water management options including sustainable urban drainage systems (SUDS)
- Calculation of life cycle energy use and whole life cost using simplified life cycle inventory and whole life costing.

CWB simulates, using simplified descriptions of a city's water system, the distribution of fluxes of water and water borne contaminants in space and time. It is specifically designed to explore alternative strategies for water management in a city under a range of alternative future scenarios. *CWB* builds on concepts from other urban water scoping models, in particular *Aquacycle*/*UVQ* (Mitchell *et al.*, 2001) and *UWOT* (*Makropoulos et al.*, 2008). It uses the idea of an extended collection of sustainability indicators from *UWOT* such as total energy consumed and whole life cost and applies many modelling concepts from *Aquacycle's* residential template to the broad range of landuses found in a cityscape. *CWB* builds a much larger collection of sustainable urban drainage systems (SUDS) than *UWOT* and combines the water efficiency options from *UWOT* with the re-use options in *Aquacycle*. In addition, *CWB* models the natural systems (groundwater, rivers, canals and lakes) in much greater detail than *Aquacycle*. Modelling groundwater, for example, is important when siting SUDS and according to *Thomas and*

Tellam (2006*):* "Urban aquifers are deemed to be of particular significance to the long term viability of many of the world's major cities". The model extends the basic analysis of cost and energy in *UWOT* to a more detailed, simplified life cycle assessment (energy and cost) of a number of alternative water management options. In the following sections there is a detailed description of the model concepts underpinning *CWB*. Although *CWB* applies a number of concepts from other models, the integration of these concepts to develop a new modelling structure and all the coding, model development and testing for *CWB* was undertaken by the author.

3.1 Core Concepts

3.1.1 Spatial Characterisation

Water demand and runoff patterns, two key aspects of the urban water cycle, are dependent on land use. In *CWB* land use is characterised in a spatial hierarchical fashion (Fig. 3.1). At the smallest spatial scale is the unit block (UB), which consists of pervious and impervious space and a water demand profile. An example of a unit block is a detached house that has roof, paved and garden area as well as an indoor water and possible irrigation demand. Larger parcels of land called "miniclusters" are constructed by defining land areas and assigning a number of unit blocks to them, all with identical attributes. The use of miniclusters allows rapid allocation of landuse to large areas of a cityscape. The concepts of unit block and minicluster were conceived in the Australian water balance model, *Aquacycle* (Mitchell *et al.*, 2001).

Flow of stormwater and wastewater though a city is generally governed by the layout of the piped, centralised network. In *CWB*, modelling is at a less detailed level than the pipe scale, focussing on broader flow patterns between subcatchments. A subcatchment is defined as an area of cityscape containing a network of foul or combined sewers that drain to a point at its downstream boundary. The urban area consists of a set of subcatchments, each containing one or more miniclusters.

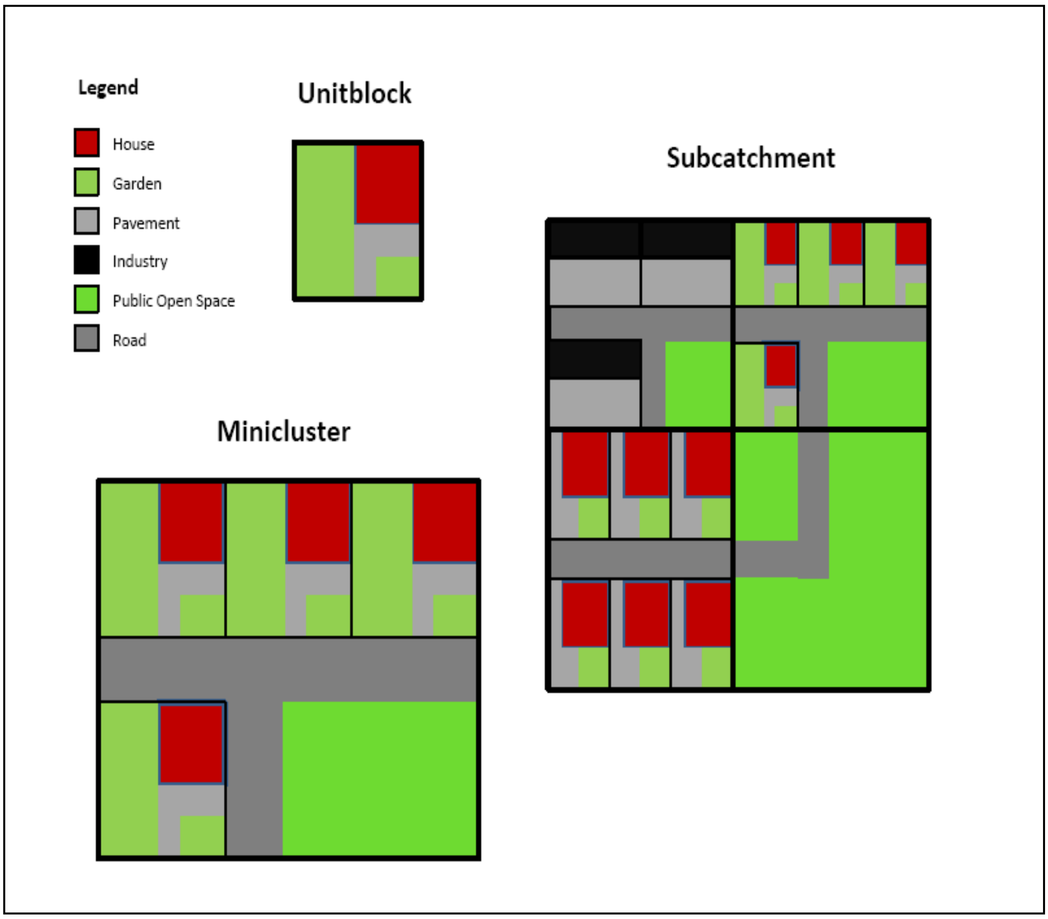


Figure 3.1 Schematic of unitblock, minicluster (of four unitblocks) and a subcatchment (of four miniclusters) to illustrate *CWB*'s descriptions of landuse configurations.

Each subcatchment can only receive wastewater from upstream subcatchments (i.e "1" can flow into "3" for example but "4" cannot flow into "3") (Figure 3.2). And wastewater/stormwater from each subcatchment can only flow into one other subcatchment (i.e. "3" can flow into "5" but not into both "5" and "6" for example). This is a modelling restriction that admits the calculation of flow through the urban area using a cascade from upstream to downstream, where downstream subcatchments can be affected by upstream flows but not vice-versa. This restriction has been found to be appropriate in the cities considered within SWITCH. It also permits the modelling to avoid the computational overhead of a more general connectivity.

In *Aquacycle* the wastewater network is defined by flows between miniclusters whereas in *CWB* it is defined by flows between subcatchments. The use of subcatchments allows simplification of the flow network without having to oversimplify the description of the cityscape. It allows minicluster areas to be allocated solely based on land use type (except at the subcatchment boundaries), enabling more rapid representation of the landuse pattern at the city scale while retaining the essential features of the wastewater network.

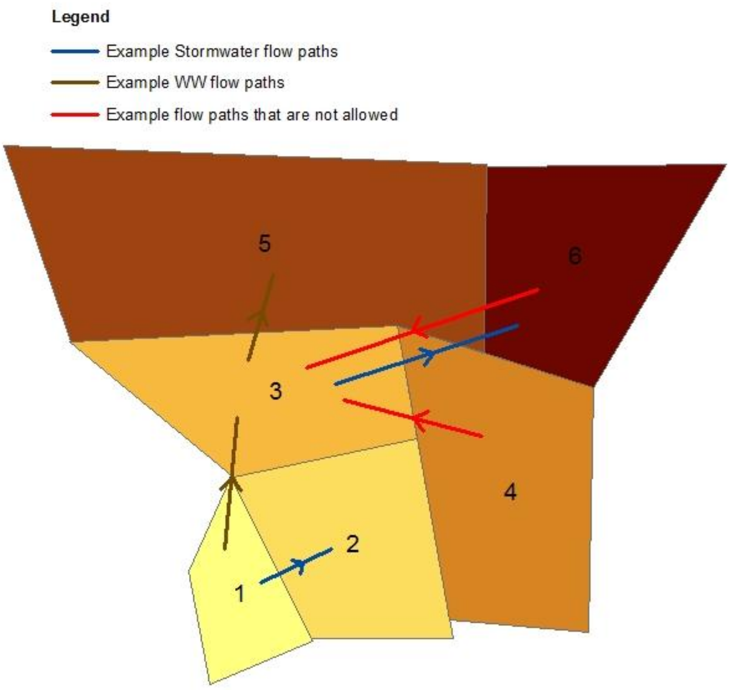


Figure 3.2 Example of flow pathways for stormwater and wastewater between subcatchments.

Although *Aquacycle* is capable of modelling landuses other than residential, it requires the user to gather many additional data and to make a number of pre-processing calculations to prepare the data for input into the models. In *CWB* some if this data gathering may be avoided by using pre-prepared generic land-uses that are available as a library for the user. These generic unit block profiles can also be edited, or added to, as necessary in order to meet the modelling requirements of different cities.

Table 3.1 shows the landuse categories in *CWB*. The choice was informed by water demand categories used in the *Plumbing Engineering Services Design Guide* (*The Institute of Plumbing*, 2002), *Thomas's* (2001) runoff-recharge model for Birmingham and detailed evaluation by the author of land use in Birmingham from Ordinance Survey Maps and Google Earth images.

Table 3.1 List of land uses in *CWB*.

Main category	Sub-category	Main category	Sub-category
City centre	None	Residential	Mansion
Fire station	None		Detached (Large garden)
Hospital	None		Detached (Small garden)
Hotel	Hotel		Semi-detached (Large garden)
	Hotel with grounds		Semi-detached (Small garden)
Industry	Industry		Terraced (Large garden)
	Industry disused		Terraced (Small garden)
	Depot		Flats
Garage	None		Home (Retirement & Nursing)
Office	None		High-rise
Place of Assembly	Bar, Pub or Nightclub	Retail	Supermarket
	Community Centre		With Canteen
	Library, Art Gallery, Museum		Without Canteen
	Restaurant	School	Further education
	Public Baths		Higher education
	Church		Nursery, Primary
	Health Centre		Primary through Secondary

Main category	Sub-category	Main category	Sub-category
Medical Centre	None		Secondary
Public Open Space	Golf course	Kennels	None
	Normal	Road	Main
	Wooded		Secondary
	Allotment	Parking	None
	Recreation ground	Railway	None
Prison	None	Water works	None
Army Centre	None		

Larger roads and public open space can be represented by large area unit blocks with appropriate impervious proportions and water demand profiles (e.g. a road would have impervious area set to 100% and no water demand). This facilitates more rapid representation of the landscape and allows large road systems, with different associated contaminant loads, to be addressed separately. Figure 3.3 shows the main flow paths that are modelled by *CWB*. The key influence that land use has is clearly demonstrated, as it affects water demand, runoff, infiltration and evaporation.

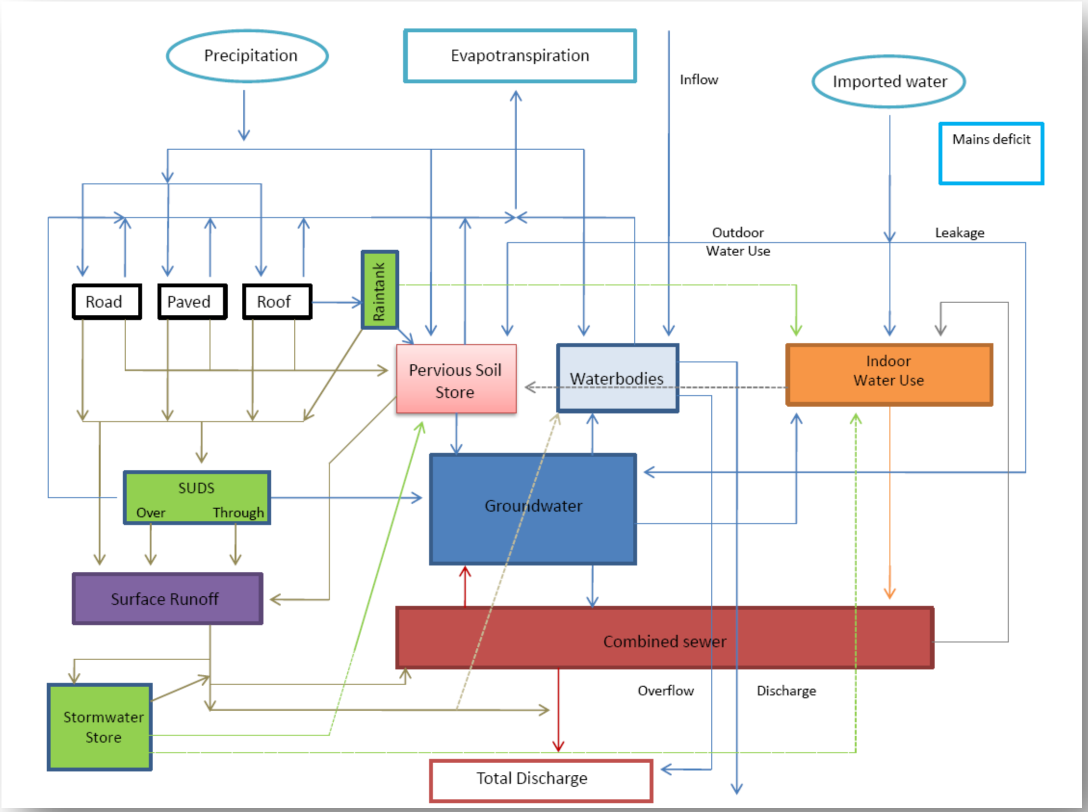


Figure 3.3 Main flow paths and stores modelled by *CWB*.

3.1.2 Temporal Resolution

The scope of *CWB* is to be a decision support tool for strategic planning and a preliminary evaluation of sustainable water management options, rather than as a detailed design tool. A daily timestep was thought to be appropriate for this type of model. Although this level of aggregation results in the loss of diurnal flow variations, it still provides sufficient information (*Makropoulos et al.*, 2008) for the comparison of different water management options, especially for the long-term simulations necessary for sustainability assessments.

3.1.3 Water Demand

Water demand input to *CWB* is based on land use. In *Aquacycle*, water demand is split into four generic uses for each land use type (*Mitchell et al.*, 2001) and for each of these four uses (labelled "Indoor uses" 1 to 4) the user must specify the demand per capita. The purpose of having the usage split is two-fold:

1) It allows creation of waste streams with different water quality

2) It provides a way of differentiating the required input water quality. For example, toilet flushing does not require water cleaned to potable standard.

The split usage demand can be used to describe the water demand for any landuse. However, the real challenge is to find good quality data to fill out the categories, especially for industrial demand whose water use varies dramatically and data are often confidential.

Unlike *Aquacycle, CWB* uses a parameter called the occupancy factor, allowing indoor unitblock demand to be expressed not only per capita but also per unit area. This is an effective way of quantifying water demand for many non-residential types of urban land use where the occupancy and floor area are strongly linked and the patterns of water use are restricted by the temporary occupancy of the building. For example, an office building may have similar water demand categories to residential properties: urinals, W.C., kitchen, and drinking water but in the office case the occupancy factor would be square metres of office space (*Hunt and Lombardi*, 2006), and the user would enter values for each demand category on this basis (litres/m^2).

CWB also introduces the idea of seasonal occupancy: this is an optional set of parameters with which the user can specify monthly occupancy factors, representing a fluctuating population. This is particularly important for a city like Athens, for example, where a significant proportion of the population leaves during the hot summer months (*Manoli*, 2009). During an application of *Aquacycle* to Athens it was not possible to simulate demand during the summer for this reason (*Karka et al.,* 2006).

When modelling residential areas, there is a choice of two water demand models in *CWB*: the split usage model or an appliance based model that has been adopted from *UWOT* (*Makropoulos et al.*, 2008). In the appliance model the demand profile is built from a selection of water using appliances (bath, W.C., kitchen tap etc.). This model has been introduced to enable more detailed exploration of the benefits of water efficient appliances. Input data required are the type and number of appliances in the unit block, water consumption/use, frequency of use/capita/day and occupancy. In addition, with each appliance use, there are associated water losses from the piped system, representing small amounts of water vaporisation and water consumption. A library of appliance options is available, to which the user can add their own appliances if required.

Demand is satisfied at the start of the daily timestep, which means that water accumulated during the current timestep is only available for re-use (by greywater recycling or rainwater harvesting) in the following timestep. In reality water demand varies throughout the day and so a situation can be envisaged whereby there is a 2mm summer rainfall event at 15.00 that fills an empty raintank that can then be used to satisfy toilet demand at 18.00. If the model satisfies demand at the start of the day then the toilet flush at 18.00 would have to rely on mains water, since the raintank does not receive the water from the rain event until after the demand calculations for that day. On the other hand it is equally possible to imagine the opposite scenario in which there is a toilet flush at 8.00 and the same rainfall event at 15.00. If the demand is satisfied at the end of the day then the water from the rainfall event can be used to supply a water demand that occurred before the actual rainfall happened. Since it can be seen that either satisfying demand at the start or the end of the day will lead to similar small inaccuracies in some situations it is not important which model is used. It is only important to note that this level of sophisticated analysis is most likely inappropriate for practical applications as well as being beyond the scope of the model.

There may be a number of sources of water available to satisfy a given demand. *CWB* incorporates and builds on the range of water management options offered in *Aquacycle* (Mitchell *et al.*, 2001). Within *CWB* the order of preferential use is as follows:

- Unitblock (UB) Wastewater store
- UB Raintank
- UB Borehole
- Minicluster (MC) Wastewater store
- MC Stormwater store
- Large Wastewater store
- Large Stormwater store
- Large Borehole
- Mains supply

This order is based on two principles. It is assumed that if a local store is in place (unit block) this should be used in preference to external sources. Water that is likely to be treated to a lower standard is used first as the order in which demands are satisfied is: Indoor use 1 (Toilet), Indoor use 4 (Laundry), Indoor use 3 (Bath), Indoor use 2 (Kitchen) and garden irrigation. It is assumed that water treated to a lower standard is more sustainable (requiring less energy, chemical use and cost for treatment) and therefore should be used in preference to better quality water for applications that do not require high quality water, such as toilet flushing.

Irrigation of public open space (POS), from minicluster and large stores, is supplied before unit block demands.

It is a model constraint that UB and MC sources of water can only supply demands in the MC in which they are situated. If a store is required that supplies several miniclusters a “Large” store must be used which is described later in this chapter. The UB wastewater store can provide water for Indoor use 1 (toilet in the case of residential) and garden irrigation. The UB raintank store and boreholes can provide water for all Indoor uses and garden irrigation. The MC wastewater and stormwater stores can provide water for the following uses, in order of priority: Public Open Space (POS) irrigation, Indoor Use 1 and garden irrigation. The large wastewater and stormwater stores can provide water for the following uses in user-specified miniclusters, in order of priority: Public Open Space (POS) irrigation, Indoor Use 1 and garden irrigation. Large scale boreholes can supply all water demands within specified MCs.

Any remaining demand that has not yet been satisfied will be supplied by the mains in the following order of priority: Indoor use 1, Indoor Use 2, Indoor Use 3, Indoor Use 4, garden irrigation and POS irrigation. Water shortages can be modelled in *CWB* by setting the total volume of mains water available in each timestep to a percentage of the total daily study area demand. If a water shortage arises and there are no other supply options irrigation will suffer first, followed by Indoor Use 2, 3, 4 and 1. Since garden irrigation is highly dependent on the personality of the house owner and since irrigation of POS ranges from none to intensive, depending on its importance and use, it is optional in the model setup to have irrigation backed-up by mains supply.

Irrigation demand depends on the soil moisture content in the pervious store. For each unit block type a value called trigger-to-irrigate (TG) (*Mitchell et al.*, 2001) is set that is a proportion of the total soil moisture capacity. When the soil moisture content of the pervious stores falls below the trigger-to-irrigate threshold, irrigation is supplied to bring the level up to the threshold, provided sufficient supply is available. During dry periods this will result in frequent, small irrigation applications as, on a daily basis, water evaporates from the soil lowering the moisture to below the TG level, at which point irrigation is triggered to raise it back to the TG level. Whereas a pattern of less frequent but greater volume irrigation, representing hosepipe irrigation several times a week, is more likely to be observed in residential areas, for example. The results of one study showed that about 40% of households use hosepipes three times a week, on average, in hot dry weather (*Three Valleys*, 1991). However, it has been found that this model does satisfactorily fit observed irrigation patterns at weekly or greater timescales (*Mitchell,* 1999).

A proportion of Indoor use 2, 3 and 4 can require hot water. This has no impact on the water volume and quality calculations within *CWB* but affects the energy balance. It is assumed that the proportion of water used for hot water for each category is the same regardless of the amount supplied.

3.2 Supply

Mains water is imported from outside the system boundary. The mains supply demand is calculated as the sum of the minicluster demands that are not met by decentralised schemes. The user can specify a percentage of the demand that is satisfied, simulating a drought.

Water is supplied at fixed contaminant concentration and with a fixed energy and economic cost attached to it. A proportion of the supply is lost as leakage within the system boundary. Leakage can be made to vary on a weekly basis (this was found to be necessary during calibration).

Modelling of the mains supply is very simplified since cost, energy and contaminant data are readily available, and therefore do not need to be calculated again, by a scoping model. The aim of the model is to investigate the sustainability of alternative technologies that are less well documented and to compare them with the base case (mains supply).

3.2.1 Leakage

Following the concept in *Aquacycle* (Mitchell *et al.*, 2001), for each minicluster a proportion of the mains supply, is lost as leakage. Leakage flows to groundwater.

3.3 Runoff

3.3.1 Effective Areas

There are five types of surface that can generate runoff in *CWB*: roof, pavement, garden, road and Public Open Space (POS). In each minicluster, the degree to which these surfaces are connected to the piped stormwater system varies. To address the issue of overland flow outside the sewer network *CWB* uses the idea of effective areas, originally conceived and used in *Aquacycle* (*Mitchell et al.*, 2001). The effective area is defined as the proportion of the total area generating runoff that goes directly to the stormwater system. Runoff from non-effective areas (NEAR) flows onto nearby pervious space or sustainable urban drainage systems (SUDS), if available. In terms of water flow the concept of NEAR diverts more stormwater to infiltration, evaporation and recharge and as such is only applicable to impervious surfaces (road, roof and paved).

Non-effective roof area runoff is assumed to be split equally between the garden and paved area of that unitblock, and non-effective paved areas contribute runoff to the garden area of that unitblock. Non-effective road areas contribute runoff to POS within the minicluster. The POS is considered as one landmass within any minicluster even though it may, in reality, consist of several separate parts. If there is no pervious space available to receive the NEAR then it is treated as effective runoff and flows into the stormwater system.

(3.1) $$\text{NEAR\_roof (m}^3\text{)} = (1 - \text{ERA})*(\text{P} - \text{E})*(\text{Roof\_area (m}^2\text{)} - \text{Green\_roof\_area})$$

Where ERA = effective roof area (-)

P = precipitation (m)

E = evaporation (m)

Green_roof_area (m^2) = area of roof used as a living roof

(3.2) NEAR_paved = (1 – EPA)*((P-E)*Paved_area + (1 – PP_prop)*0.5*NEAR_roof)

Where EPA = effective paved area (-)

PP_prop = porous paved proportion (-)

Half of the NEAR_roof is redirected to the pavement to take advantage of any porous paving that may be present. If there is no porous paving (PP_prop = 0) then the NEAR_roof that flows onto the pavement will contribute fully to the NEAR_paved.

(3.3) NEAR_road = (1 – ERDA)*(P-E)*Road_area

Where ERDA = effective road area (-)

3.3.2 Initial Losses

The concept of initial loss, which represents all rainwater that is unavailable for runoff, is well documented (*Chow et al.*, 1988; *Dingman*, 1994). On pervious surfaces this can happen by interception loss, depression storage and infiltration. On impervious surfaces initial loss is by depression storage. Interception loss occurs when rainfall falls on vegetative surfaces and is subsequently evaporated. Depression storage is the volume of water retained in puddles and other small surface depressions, which is subject to evaporation. Infiltration is the process by which water on the soil surface enters the soil. Initial losses can vary depending on the characteristics of the roof, paved, road area or the type of vegetation.

In *CWB* the three impervious surface types (roof, paved, road) have an initial loss associated with them, representing depression storage and subsequent evaporation. In pervious areas (garden, POS and infiltration urban drainage systems) rainfall is subject to interception loss (if the area is wooded) and infiltration. The application of interception loss in wooded areas extends beyond concepts covered by *Aquacycle* (Mitchell *et al.*, 2001) and its inclusion is important because interception loss represents a significant proportion of total evapotranspiration in most vegetated areas (*Dingman*, 1994).

3.3.3 Porous Pavement (PP)

Porous pavements are a water management option not modelled in *Aquacycle* or *UVQ* (Mitchell *et al.*, 2001). Inputs to the porous pavement are direct precipitation, roof runoff and outputs are evaporation, overflow and infiltration. Evaporation from the porous pavement is at potential rate providing there is sufficient water available (sum of precipitation, NEAR_roof and storage level from the previous timestep). Overflow occurs when the storage volume from the previous timestep plus the inputs exceeds the capacity of the store. Overflow is assumed to flow to the stormwater system. A predefined infiltration rate (PP_inf_rate) for the porous paving in each unitblock type can be chosen. The infiltration volume (PP_inf) is the product of the porous pavement area (PP_area) and infiltration rate, provided there is sufficient supply (Eqs. 3.4 & 3.5). PP_storage is the current volume of water stored in the pavement.

(3.4) If [PP_area (m^2) * PP_inf_rate (m/day)] < PP_storage (m^3) then

PP_inf (m^3) = PP_area * PP_inf_rate

Else

(3.5) PP_inf = PP_storage

3.3.4 Porous Road (PA)

Porous roads are also not modelled in *Aquacycle* or *UVQ* (Mitchell *et al.*, 2001). Input to the porous road is direct precipitation and outputs are evaporation, overflow and infiltration.

Evaporation from the porous road is at potential rate providing there is sufficient water available (sum of precipitation and storage level from the previous timestep). Overflow occurs when the storage volume plus the precipitation exceeds the capacity of the store. Overflow is assumed to flow to the stormwater system. A predefined infiltration rate (PA_inf_rate) for the porous paving in each unitblock type can be chosen. The recharge volume (PA_inf) is then the product of PA area and infiltration rate provided there is sufficient supply (Eqs. 3.6 & 3.7). PA_storage is the current volume of water stored in the pavement.

(3.6) If [PA_area (m^2) * PA_inf_rate (m/day)] < PA_storage (m^3) then

PA_inf (m^3) = PA_area * PA_inf_rate

Else

(3.7) PA_inf = PA_storage

3.3.5 Pervious Store Models

Following the approach adopted in the *UVQ* model (*Mitchell & Diaper*, 2005), in *CWB* there are two soil store models that can be used: partial area and two-layer. The partial area model is more suited to semi-arid and arid climates that have high potential evaporation and intense rain events - no surface runoff or recharge occurs until the soil is fully wetted so that during dry periods the only mechanism for water loss from a partially filled store is evaporation. By contrast, the two-layer model allows a proportion of water above the field capacity to drain to groundwater, which better represents areas with lower potential evaporation. These soil models are applied to the garden area at unit block scale and POS at minicluster scale as well as to a number of the SUDS options.

Partial Area Model

The Partial Area Model is based on the Australian Water Balance Model (AWBM) by *Boughton* (2003). The pervious area is split into two (Fig. 3.4): pervious store 1 (PS1) and pervious store 2 (PS2). PS1 and PS2 can have different storage depths and different areas. Input to the pervious store area is split between the two stores proportional to their relative surface area. There are no flows between the stores. Evaporation from the stores is the lesser of:

(3.8) $E = E_p$

Or

(3.9) $E = (A1 * PS1_{cur} \ / \ PS1_{cap} + (1 - A1) * PS2_{cur} \ / \ PS2_{cap}) * E\_trans$

Where E_p = daily potential evaporation rate (m)

E_trans = capacity of vegetative cover to transpire (fixed) (m)

PS_{cur} = pervious store current level (m)

PS_{cap} = pervious store capacity (m)

A1 = proportion of total garden area underlain by PS1 (-)

The maximum evapotranspiration rate has a fixed value for the study area and is used as a second control on evaporation from pervious areas. When it is set to a high value, representing denser vegetation, actual evaporation will approach the potential. Evaporation cannot reduce the soil moisture content of the store below the residual level.

The only outflow from the stores is evaporation until the capacity of either store is exceeded. Excess water above the capacity is divided between groundwater recharge (GWR) and overland flow (PRUN) (Fig. 3.4b). The division is dependent on user defined proportions for each minicluster. It is assumed that overland flow from pervious stores is directed to the stormwater system.

Capillary drawup (C_d) is modelled using equation (3.10). When groundwater is deeper than a specified depth then there is no capillary draw up (Fig. 3.4a). If it rises above this level then drawup occurs up to a maximum of the daily potential evaporation (Fig. 3.4c). If the groundwater level rises above the base of the soil (note this is different from the effective soil depth) then it is assumed that the whole soil profile becomes saturated (3.4d). If groundwater rises above ground level then it contributes to PRUN.

$$C_d = E_p * (SMD/SMD\_max).(GWL - D_c)/(S_{base} - D_c) \quad (3.10)$$

Where SMD = soil moisture deficit ($PS_{cap} - PS_{cur}$)(m)

GWL = groundwater level AOD (m)

S_{base} = soil thickness (m)

D_c = Max capillary action depth

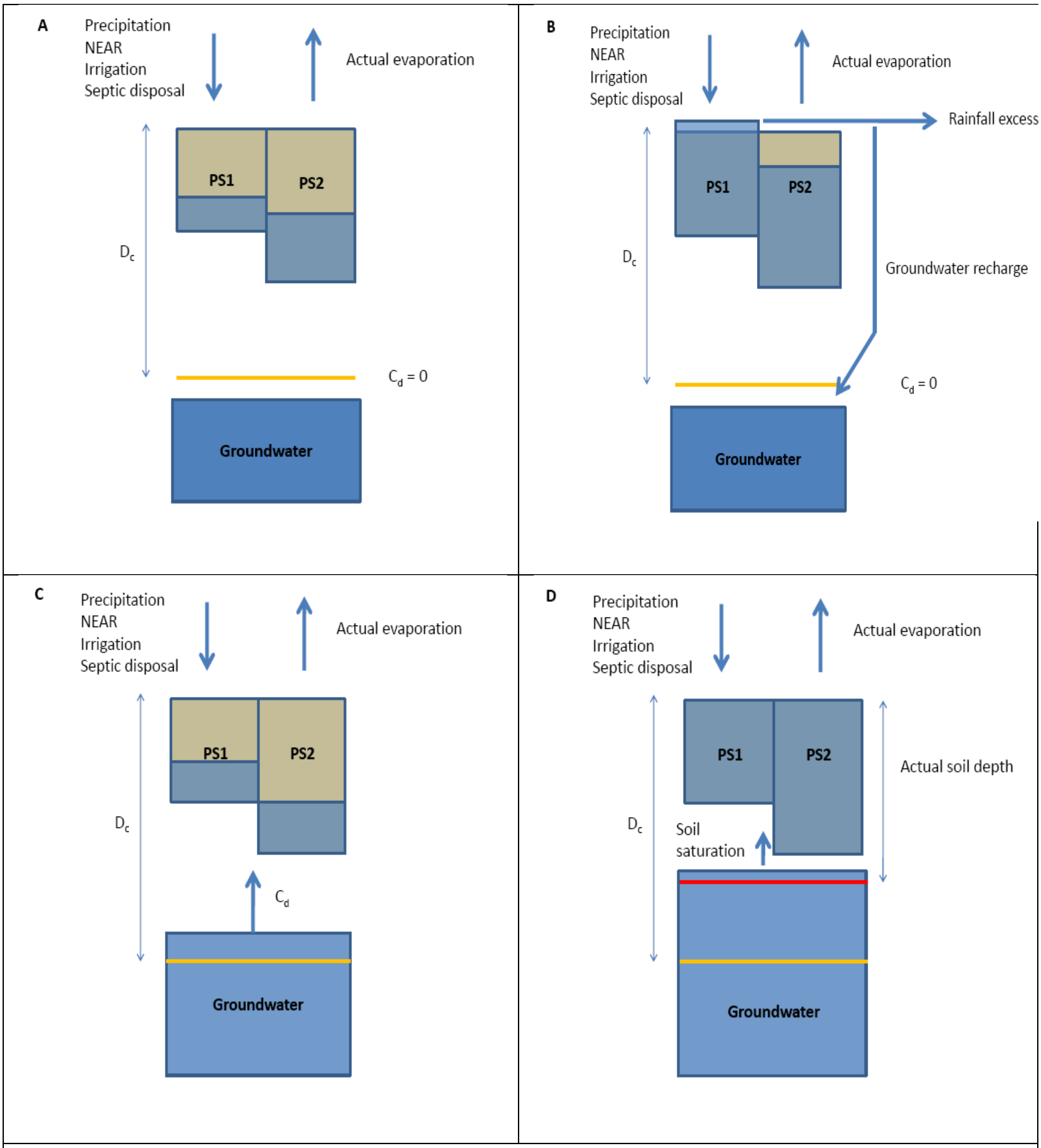


Figure 3.4 Partial Areas pervious store concept. NEAR is non-effective area runoff. (A) GWL below max Cd depth. (B) Excess soil moisture. (C) GWL between max Cd depth and base of soil store. (D) GWL above soil store base

Two-layer model

As in the Partial Area Model, the pervious store is split into two areas PS1 and PS2 (Fig. 3.5) (*Mitchell & Diaper*, 2005 after *Denmead &Shaw*, 1962). Both stores have the same areas and PS1 lies directly above PS2. Input to PS1 is rainfall and NEAR from adjacent impervious surfaces. Outputs are evaporation, draindown to PS2 and overflow. The input to PS2 is draindown from PS1 and outputs are evaporation and recharge to groundwater and the infiltration store.

If either soil store is greater than 75% full then evaporation is the lesser of potential evaporation (Ep) and potential evapotranspiration (E_trans) (*Mitchell & Diaper*, 2005 after *Denmead &Shaw*, 1962). Otherwise evaporation is a linear function of current volume, in excess of the residual water content (PS_{res}), over the total capacity up to a maximum of the potential evaporation rate (Eqs. 3.11 and 3.12). Total capacity is maximum volume of water that the soil can hold (PS_{cap}). Evaporation cannot reduce the soil moisture content of the store below the residual level.

(3.11) $$E\ (m) = E_p\ (m)$$

Or

(3.12) $$E = (PS_{current\_level} - PS1_{res}) / (0.75 * PS_{cap}) * E_{trans}$$

A choice of two models is offered for the calculation of draindown from the soil stores. One drains a linear proportion of soil moisture above the field capacity (*Mitchell & Diaper*, 2005 after *Denmead &Shaw*, 1962) and the other non-linear (*van Genuchten*, 1980).

In the linear model, also used in *UVQ*, if the level of PS1 store (after precipitation input and evaporation) is greater than the residual water content then draindown to PS2 is the lesser of:

(3.13) PS1_drain = (PS1_current_level – PS1_field_cap) * PS1_drain_prop

(3.14) PS1_drain = PS1_Drain_max

Where PS1_drain = drainage from PS1 (m)

PS1_drain_prop = user defined proportion for each MC (-)

PS1_Drain_max = maximum drainage from PS1 (m)

If the level of PS1 store exceeds the capacity even after draindown then the excess becomes surface runoff, which is assumed to flow to the stormwater system (Fig 3.5b). Drainage from PS2 follows the same rules as those for PS1. The drainage proportion (PS_drain_prop) determines the increase of drainage (or soil conductivity) with increasing water level in the store.

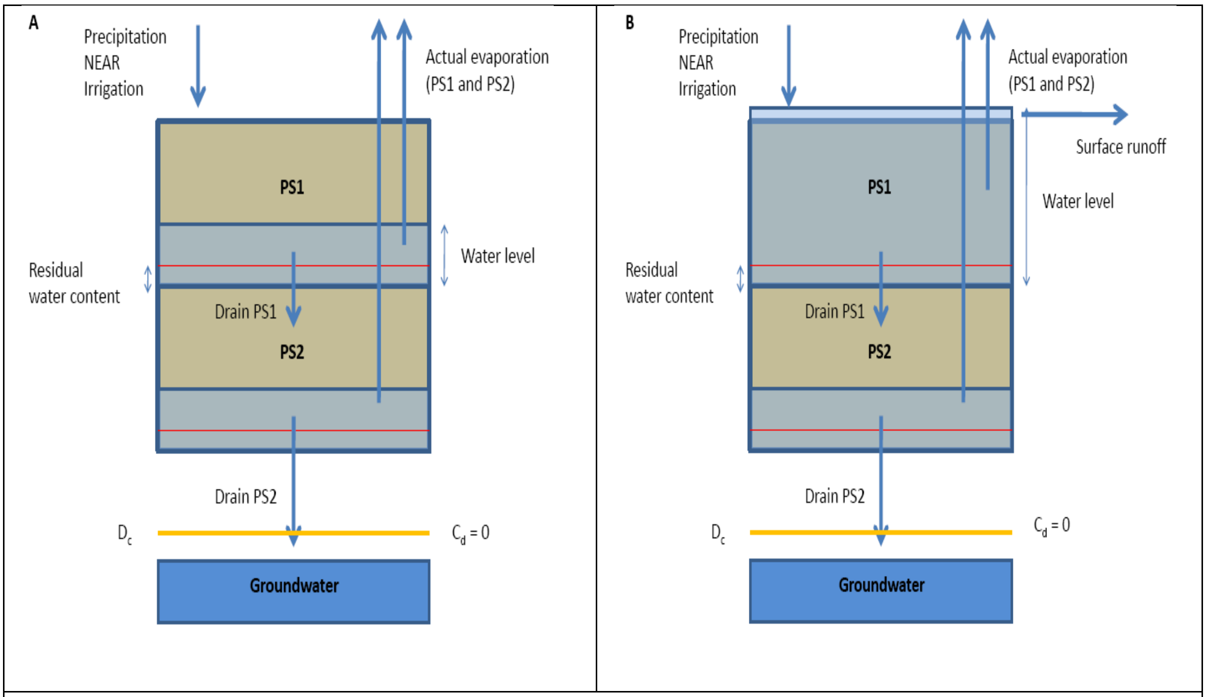


Figure 3.5 Two-layer pervious store concept. NEAR is non-effective area runoff. (A) Soil Moissture level greater than residual water content. GWL below max Cd level. (B) Soil moisture content of PS1 exceeds capacity resulting in surface runoff of the excess.

In the *van Genuchten* (1980) model the hydraulic conductivity, and therefore draindown rate, varies non-linearly with soil moisture content (Eq. 3.15).

(3.15) $K_r = S_c^{0.5} * [1 - (1 - S_c^{1/m})^m]^2$

Where m = empirical parameter ~ 0.5

$S_c = (\theta - \theta_r) / (\theta_s - \theta_r)$ = dimensionless water content

θ = soil water content

θ_s = saturated soil water content

θ_r = residual soil water content

K_r = relative hydraulic conductivity = K / K_s

K_s = saturated hydraulic conductivity

Figure 3.6 compares the classic soil curve from *van Genuchten* (1980) with three linear variations based on the *UVQ* model (equations 3.13 & 3.14).

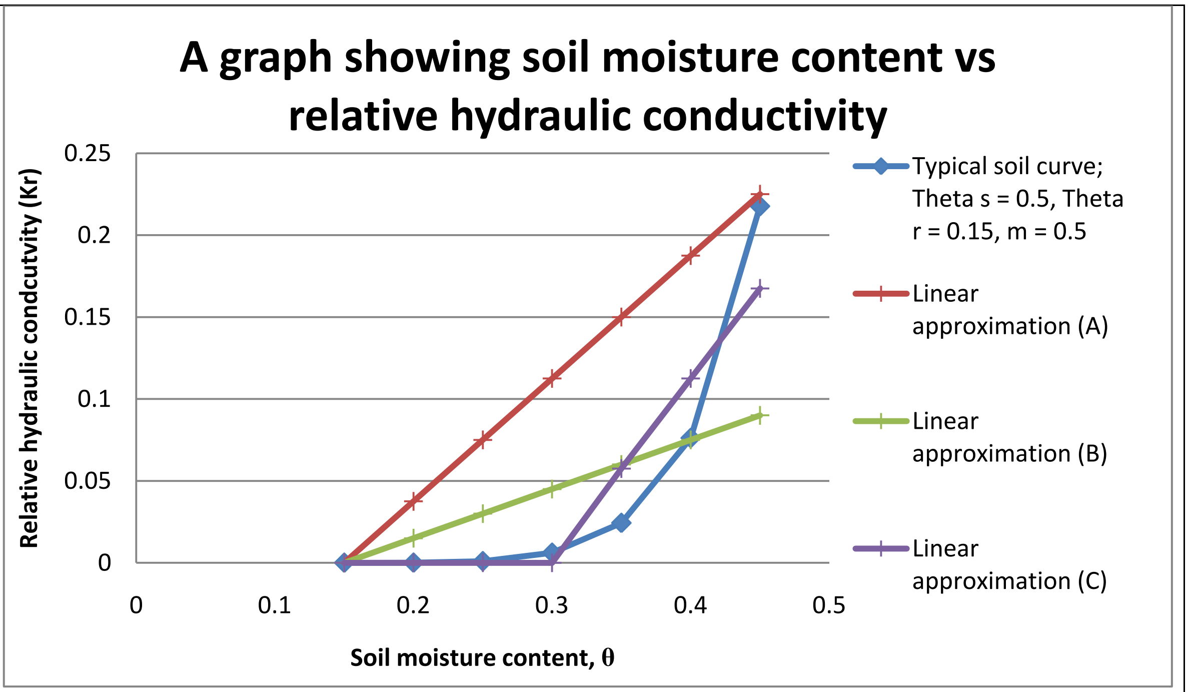


Figure 3.4 Comparison of the linear variation of hydraulic conductivity with soil moisture content (UVQ concept) vs non-linear increase of hydraulic conductivity with increasing soil moisture (*van Genuchten*, 1980).

3.3.6 Order of Action by Sustainable Urban Drainage Systems (SUDS)

Conventional urban piped drainage systems have the aim of removing water from the city as rapidly as possible. The problem with this is that unnaturally large peak volumes of stormwater are discharged to downstream water courses, which are then more susceptible to flooding. SUDS are used to attenuate stormwater flow, reducing the peak discharge to rivers and increasing the lag time from rainfall event to discharge to river. The SUDS treatment train is the idea that these drainage techniques can be used more effectively in series to change the water flow and water quality characteristics of runoff in stages (*CIRIA*, 2005b) (Fig. 3.7). It is based on the principle that prevention is better than cure and that it is generally better to attenuate stormwater as close to source as possible. The treatment train is now accepted as an often more sustainable way of managing urban stormwater runoff (*Woods-Ballard et al.*, 2007) than the conventional centralised piped system. In *CWB* a number of SUDS options are available at three different scales (Table 3.2).

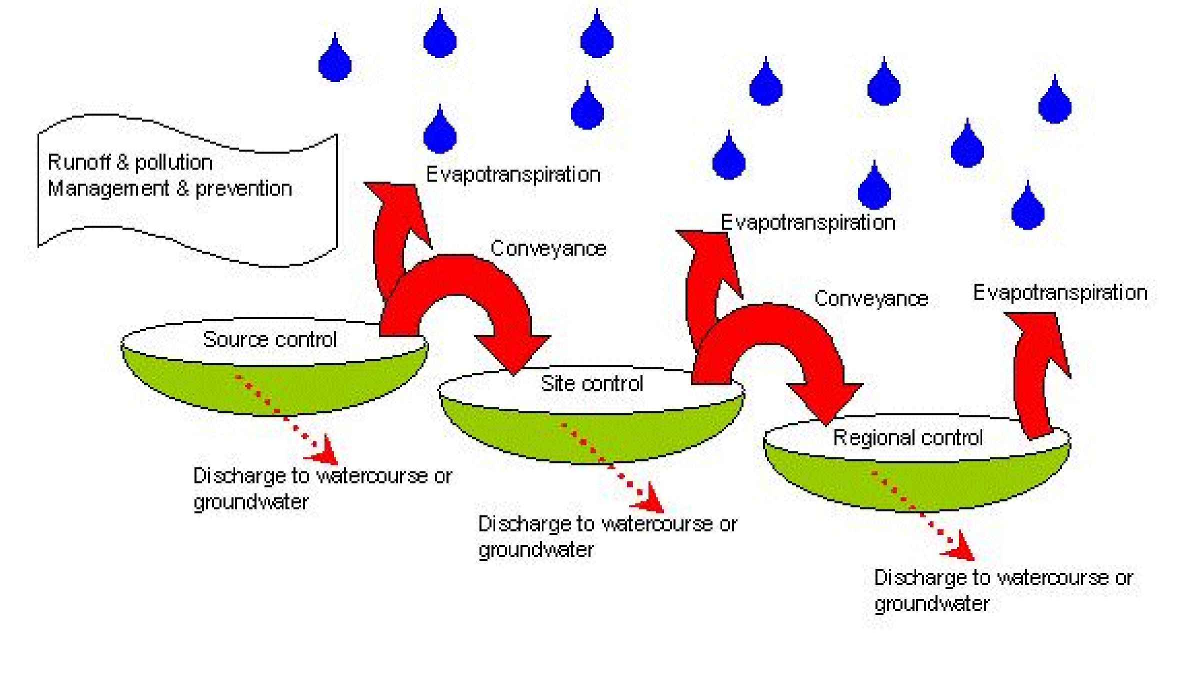


Figure 3.7 The SUDS treatment train (CIRIA, 2005).

Table 3.2 Range of SUDS that are modelled in CWB at different spatial scales.

Unitblock scale	Minicluster scale	Large Scale
Green roofs	Filter strips	Stormwater stores
Raintanks	Stormwater stores	Detention Basins
Swales	Swales	Retention Ponds
Soakaways	Soakaways	

3.3.7 Green Roofs

A green roof is an intentionally vegetated roof surface. They offer a number of environmental benefits, including improving air quality, providing habitats for insects and birds, contributing to the retention of humidity in urban areas and reducing roof runoff and associated contaminants (*Greenroofs.org*, 2009). Green roofs can be categorized into three groups: intensive, semi-intensive and extensive (*Livingroofs.org*, 2010). Extensive green roofs generally consist of a combination of moss, herbs and grass. They are light-weight (60-150kg/m^2) and low cost and maintenance. By contrast intensive roofs can consist of lawn, shrubs and trees. They weigh substantially more than extensive roofs (180-500kg/m2) and have a high capital cost with ongoing maintenance requirements (*Livingroofs.org*, 2010). Figure 3.8 shows a section through a typical green roof.

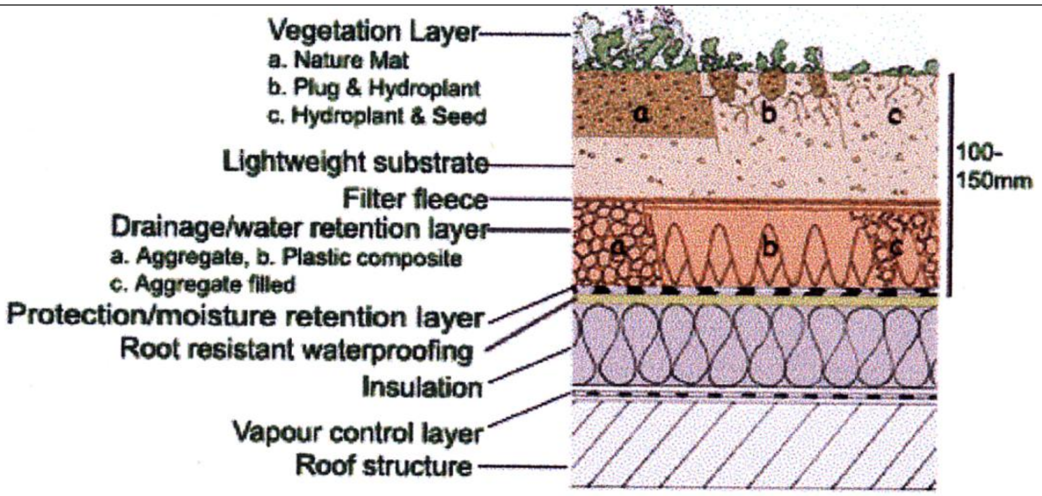


Figure 3.6 Section through a typical green roof (*Greenroofs.org*, 2009).

Bengtsson et al. (2005) modelled green roofs as a linear reservoir with no runoff until the store reaches field capacity. Since *CWB* generally uses simple reservoir models, this approach to modelling green roofs was appropriate.

Green roofs can be applied to a roof's effective area. Precipitation is the input and outputs are evaporation, drainage and overflow. If the sum of precipitation input and the current storage level is greater than the capacity then the excess is lost as overflow. After overflow has been deducted evaporation is at potential rate provided there is sufficient water for this. If the storage level in the green roof is greater than the residual storage level then drainage occurs at a fixed rate, from the drainage layer (Fig. 3.8), but never reducing the store level below its residual storage level (Eq. 3.16). Drainage is distinct from overflow because it is released gradually and so is less likely to contribute to flooding.

(3.16) Green_drain = DR * (Level_cur – Res_prop * Green_cap)

Where Green_drain = drainage from green roof (m)

DR = drainage rate (m/day)

Level_cur = Current level of water in store (m)

Res_prop = proportion of total capacity that is residual and cannot flow out (-)

Green_cap = capacity of green roof (m)

3.3.8 Raintanks

Raintanks can be used to collect runoff from impermeable surfaces (usually roof) that can then be used to supply either potable demand (with chemical or UV treatment) or non-potable use with little treatment (filtering). The degree of treatment required for non-potable use varies in different countries. In the USA, for example, even non-potable uses require disinfection in addition to filtering.

Inputs to the raintank are direct precipitation (if tank is open-air), effective roof area runoff, drainage and overflow from the greenroof. Outputs are overflow, evaporation (if open-air) and usage (indoor demand and/or irrigation). Evaporation is at the potential rate provided there is sufficient water.

During a dry period there is a build up of airborne contaminants and detritus on the roof surface. The initial runoff of a subsequent rainfall event washes these contaminants off the roof in a "first flush". Since this first flush is more contaminated (*Forster,* 1991) than the remainder of the runoff during the storm event, it can be diverted away from the raintank (Fig. 3.9). In *CWB* there is the option to remove a fixed volume of water from the flow to the raintank, representing first flush removal. If the roof runoff volume is less than the FF then no rainwater is harvested. The FF is assumed to be discharged to the stormwater system.

After precipitation and evaporation flows have been accounted for overflow is deducted for any volume in excess of the raintank capacity. Overflow is directed to the stormwater system.

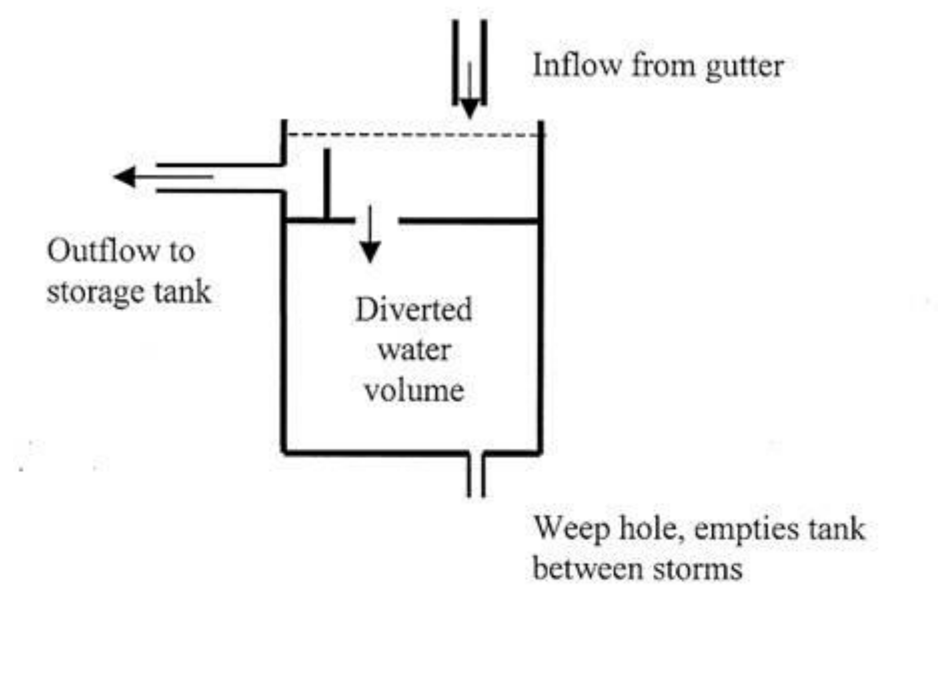


Figure 3.9 Example of a typical first flush system (*HarvestH20.com*, 2010)

3.3.9 Unitblock Swale

A swale is a vegetated open-air drainage channel. The increased roughness of the swale, in comparison with conventional pipes, acts to attenuate stormwater flow. Infiltration and evaporation also contribute to stormwater attenuation. There are two types of swales: conveyance (Fig. 3.10) and infiltration (Fig. 3.11). Conveyance swales are designed to transfer water and are more suitable in areas where infiltration is not required/desired. Infiltration swales have a number of mini-dams along their length creating a series of reservoirs during a storm event. Water stored in these reservoirs is subsequently infiltrated or evaporated.

Figure 3.10 (A) Drop curve from a car park to a conveyance swale. (B) Flush curb to a grass filter to a conveyance swale (*Lanarc Consultants*, 2005).

Inputs to the swale are runoff and precipitation. Any runoff from the unitblock that would otherwise go into the stormwater system is redirected though the swale. Outputs are evaporation, throughflow, infiltration and overflow. If there is ponding in the swale, evaporation occurs at the potential rate. After runoff and precipitation have been added to any remnant ponding, if the volume exceeds the swale's capacity then overflow occurs. Overflow is assumed to drain to the stormwater system. Subsequently the infiltration is deducted from the surface ponding. Equation (3.17) is proposed as a model for swale infiltration based on the common design criteria: time to half-empty storage (*Woods-Ballard et al.*, 2007).

(3.17) Swale_inf = (1 / drawdown) *0.5* D

Where drawdown = number of days to half-empty swale from full

Swale_inf = swale max daily infiltration potential (m)

D = max depth of swale (m)

Figure 3.11 Infiltration swale (A) under construction (B) in operation (*Lanarc Consultants*, 2005).

After infiltration has been estimated, throughflow is calculated. Throughflow from the unit block swale either goes to the conventional stormwater system or into another SUDS. There are two models for flow through the swale: Manning's flow and a reservoir model. These are used to represent conveyance and infiltration swales respectively.

Manning's flow

Representation of flow through a conveyance swale is approximated in *CWB* using a combination of mass balance and Manning's equation (Eq. 3.18). Manning's equation is applicable to longitudinal slopes of less than 0.1 (*Linsley and Franzini*, 1979).

(3.18) $V = 1/n * R_h^{2/3} * S^{0.5}$

Where v = average flow velocity (m/s)

S = longitudinal slope

R_h = hydraulic radius = X_a / P_w

X_a = X-sectional area of water in the swale

n = Manning coefficient of roughness

P_w = Wetted perimeter

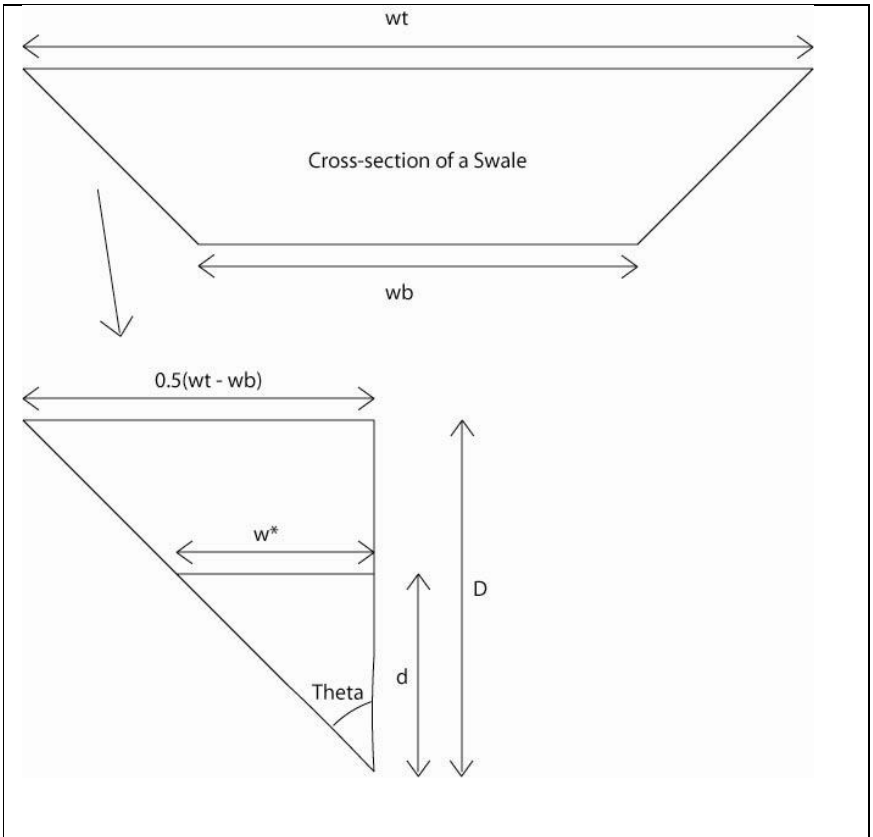


Figure 3.12 Labelling convention used in *CWB* for modelling of flow through a swale.

The mass balance for a swale is:

Storage $^{(t-1)}$ + Input volume$^{(t)}$ = Storage$^{(t)}$ +Output volume$^{(t)}$

(3.19) $Q_{in}^{(t)} = Q_{out}^{(t)} + \Delta Storage^{(t)}$

Where Q_{in} = input volume (m^3) (known)

Q_{out} = output volume (m^3) (unknown)

$\Delta Storage^{(t)} = (X_a^{\ t} - X_a^{(t-1)}) * L$ (m^3)

L = Swale length (m)

t = current timestep

t-1 = previous timestep

Calculate x-sectional area of flow:

(3.20) $\quad Tan\theta = 0.5(w_t - w_b)/D = w^* / d$

$\rightarrow \quad w^* = d(w_t - w_b)/2D$

$\rightarrow \quad X_a = d(w^* + w_b) = d(d(w_t - w_b)/2D + w_b)$

Where D = max depth of swale (m)

d = water depth in swale in current timestep (m) (constant over length)

w_b = width of swale base (m)

w_t = width of top of swale (m)

Output flows from the swale consist of infiltration, evaporation (E) and throughflow (Eq. 3.21). *Substitute Manning's Equation (3.18) for v:*

(3.21) $\quad Q_{out} = Swale\_inf + E + X_a * v = Swale\_inf + E + X_a * 1/n * R_h^{\ 2/3} * S^{\ 0.5}$

Calculate wetted perimeter:

(3.22) $\quad P_w = w_b + 2.[[d^2 + [d.(w_t - w_b) / 2D]^2]^{0.5}$

$= w_b + 2d[1 + ((w_t - w_b) / 2D)^2]^{0.5}$

Substitute wetted perimeter and cross-sectional area into Eq. 3.21:

$$(3.23) \qquad Q_{out} = Swale\_inf + E + 1/n^* \; d^2(w_t - w_b)/2D + d^*w_b) \;^*\_$$
$$[d^2(w_t - w_b)/2D + d^*w_b) \,/\, (w_b + 2d^*[1 + ((w_t - w_b) / 2D)^2]^{0.5})]^{2/3} * S^{\;0.5}$$

Outflow (Q_{out}) and change of storage ($\Delta Storage^{(t)}$) are functions of d. Equation (3.23) can be rewritten:

$$(3.24) \qquad Q_{in} = f(d) + g(d) = d * [\, f(d) / d + g(d) / d]$$

Re-arranging yields:

$$(3.25) \qquad d^{new} = Q_{in} / [\, f(d) / d + g(d) / d]$$

It is then an iterative procedure to find d; the program loops until the difference between d and d^{new} is negligible ($1 * 10^{-10}$). The initial value for d affects the number of iterations required before a solution is obtained. The further the initial value is from the solution the greater the number of iterations needed for convergence. A sensible starting value is one between zero and the maximum swale depth since this is the only range in which the solution can be found.

Therefore, once infiltration has been calculated, manning's equation can be used, in combination with mass balance, to find the depth of flow through the swale and consequently the volume of throughflow and ponded volume.

Reservoir Model

The reservoir model is used to represent infiltration swales. In this model there is no throughflow – once capacity is reached overflow occurs. In this way ponding and subsequent infiltration is encouraged.

The ground beneath the swale conforms to the pervious stores model, with the exception that evaporation from the stores only occurs if there is no surface ponding in the swale. Infiltration of ponded surface water goes into the underlying soil stores (PS1 and PS2 in the case of the partial stores model and PS1 for the two-layer model). Excess soil moisture in the pervious stores that does not contribute to recharge is assumed to be retained within the ponded surface water.

3.3.10 Unitblock Soakaway

A soakaway is a subsurface reservoir, filled with coarse aggregate, designed to store stormwater flows and subsequently infiltrate them (Fig. 3.13). They also provide some water treatment by settling and adsorption to the aggregate.

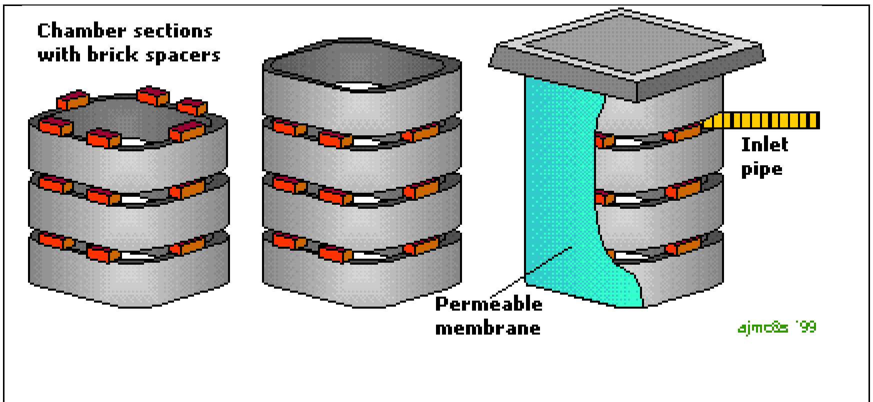


Figure 3.13 Example of a concrete soakaway (*McCormack*, 2010).

In *CWB*, soakaways are modelled as reservoirs. UB soakaways can receive flow from a UB septic tank (if present) and a proportion of runoff after the UB swale (if present). If this proportion is set to less than one then the remaining runoff is assumed to go to the centralised, stormwater system. Outputs are overflow and infiltration. Overflow occurs if the storage volume from the end of the previous timestep plus the new input exceeds the soakaway capacity. Overflow is assumed to drain to the stormwater system. Infiltration is calculated after overflow deductions and is a user-specified maximum volume per timestep, provided there is sufficient water. Infiltration goes to groundwater.

3.3.11 Minicluster Filter Strips

A filterstrip is a vegetated area over which stormwater flows as overland sheet flow (Fig. 3.14). They are designed to accept runoff from upstream developments and they treat runoff by vegetative filtering and promote settling of pollutants and infiltration (*Woods-Ballard et al.*, 2007).

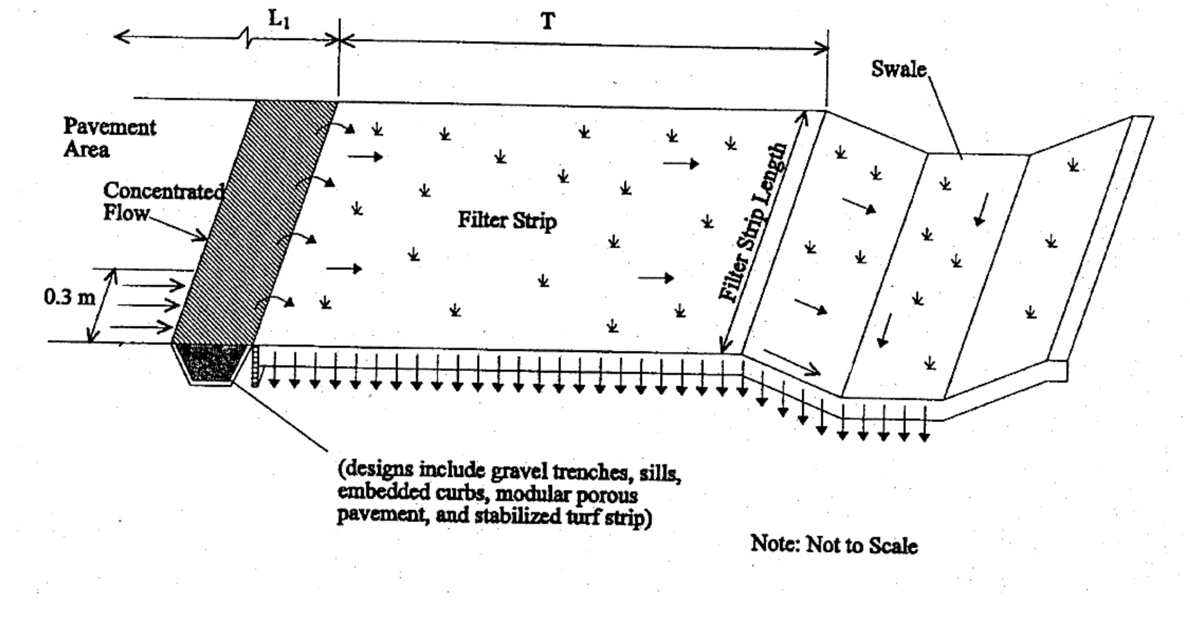


Figure 3.14 Example of a filter strip (*Daywater,* n.d.a)

Inputs to the filterstrip are direct precipitation and runoff from the unit blocks, POS and roads within the minicluster. Outputs are throughflow, infiltration and evaporation. In keeping with the simplified concepts employed by *CWB* infiltration is calculated as a fixed, user specified proportion of the input runoff. Remaining runoff is unaffected and passes as throughflow to the conventional stormwater system or to another SUDS. The filterstrip sub-surface is treated as a soil store with inputs of direct rainfall and infiltration from the filterstrip surface. Outputs from filterstrip soils are the same as the pervious store model.

3.3.12 Minicluster Swale

The MC swale receives throughflow from the filterstrip (if present) or runoff from the minicluster, in addition to direct precipitation. Evaporation occurs at potential rate if there is sufficient water available (sum of precipitation, runoff input and standing water).

Flow through the swale is either by Manning's equation or the reservoir model, described in the section "UB Swale". The swale sub-surface conforms to the pervious stores model, with the exception that evaporation from the stores only occurs if there is no surface ponding in the swale. Infiltration from the swale is the input to the underlying pervious stores (PS1 and PS2 in the case of the partial stores model and PS1 for the two-layer model).

3.3.13 Minicluster Soakaway

Input to the MC soakaway is a user specified proportion of the remaining runoff after the MC swale (if present). If this proportion is set to less than one then the remaining runoff is assumed to go to the centralised, stormwater system. As with the UB soakaway, outputs are overflow and infiltration. Overflow is assumed to drain to the stormwater system and infiltration contributes to groundwater recharge.

3.3.14 Inflow to the Wastewater System

A proportion (Inflow_prop) of any runoff after the MC soakaway (if present) is diverted to the wastewater system, representing cross-connections (*Mitchell et al.,* 2001) and leakage through manhole covers (*Metcalf & Eddy*, 1991), and the remainder enters the stormwater system. A cross-connection is the mistaken connection of a stormwater pipe to the foul sewer system. Combined sewer systems can be modelled by setting the value of Inflow_prop to one.

(3.26) Inflow (m^3) = Inflow_prop * Runoff (m^3)

3.3.15 Detention Basin (Pond)

A detention basin is an open-air basin that is used to store stormwater during a rainfall event (Fig. 3.15). It is designed to empty by a combination of evaporation, infiltration and draindown after the event (*Daywater*, n.d.) and to be empty during dry periods. Detention basins are assumed to be vertically sided reservoirs, since the actual shape of the basin is not important to the modelling process at the level of detail that *CWB* covers. Basins are sited within the bounds of a minicluster, and replace an equivalent area of landuse within that minicluster.

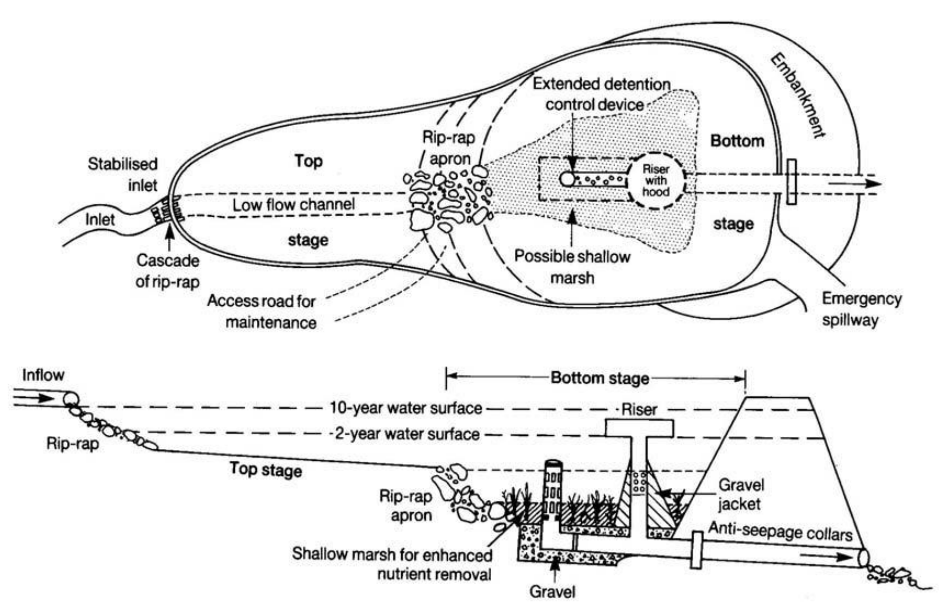


Figure 3.15 Example plan view and cross-section of an extended detention basin (*Daywater*, n.d.)

Similar to detention basins, a retention pond (Fig. 3.16) is an urban drainage system used to retain stormwater flows. They differ from basins in that they retain a permanent water volume between runoff events. Consequently, in *CWB*, retention ponds are modelled as detention basins with a permanent water volume.

Inputs to a basin are direct precipitation and stormwater runoff. A proportion of runoff after the MC soakaway (if present) may be diverted to a detention basin. Detention basins may receive flows from a number of miniclusters. Outputs are evaporation, overflow and infiltration. Evaporation is at potential rate provided there is sufficient supply (storage from end of previous timestep plus input flow). Overflow is any volume greater than the capacity of the basin after evaporation losses. Overflow is assumed to leave the study area in the stormwater system.

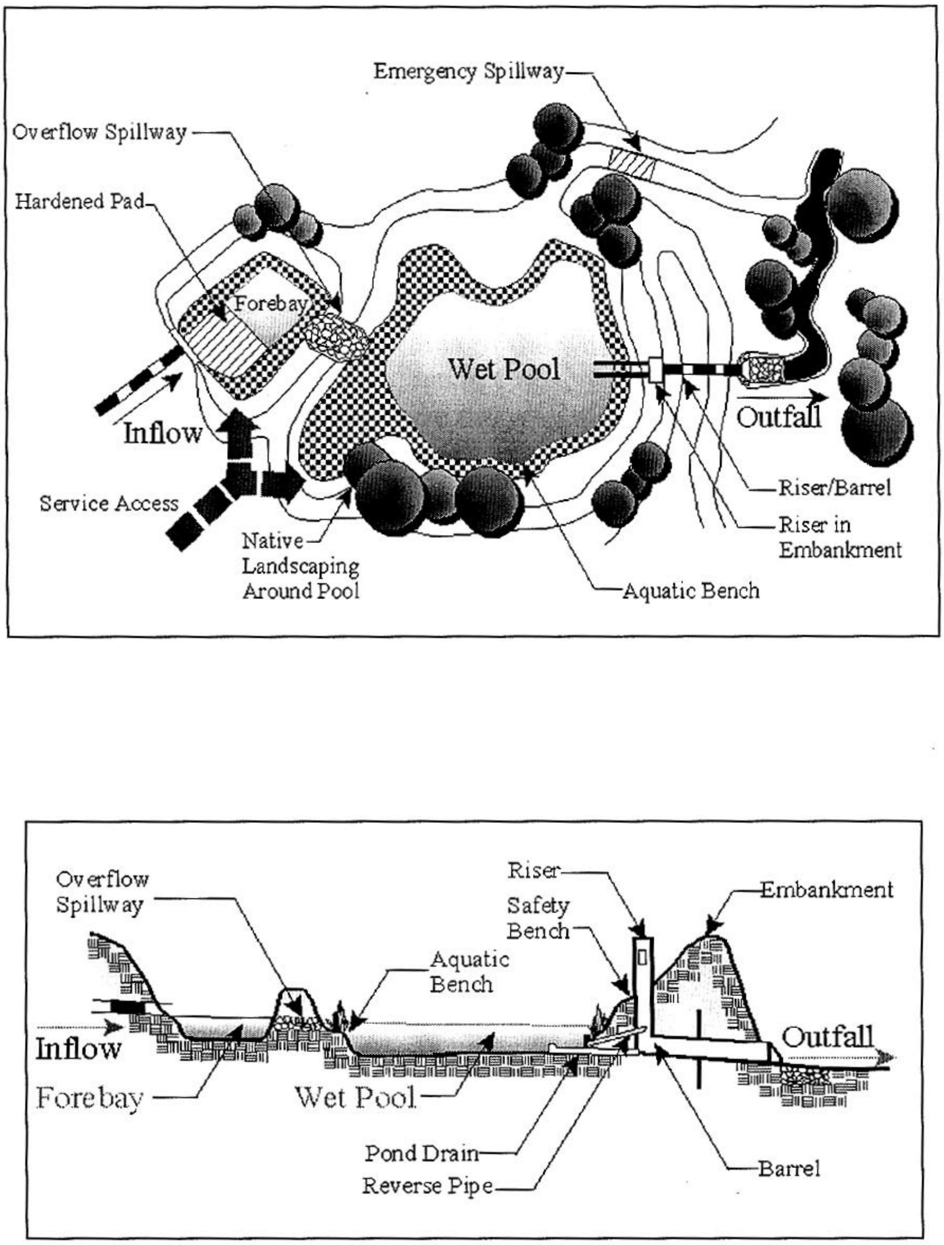


Figure 3.16 Example plan view and cross-section of a retention pond (*Daywater*, n.d.).

Infiltration is calculated based on Darcy's Law and depends on the groundwater level, the basin bed thickness and hydraulic conductance, and the surface water depth.

If the groundwater level is below the depth of the pond bed then:

(3.27) $$INF_{pond} = CONDUCT_{pond} * (DEPTH_{pond} + BED_{pond})$$

Otherwise:

(3.28) $$INF_{pond} = CONDUCT_{pond} * (DEPTH_{pond} + BED\_LEVEL - GW\_LEVEL)$$

Where INF_{pond} = infiltration from the pond (m^3)

$CONDUCT_{pond}$ = pond area (m^2) * hydraulic conductivity of the pond bed (m/day) / bed thickness (m) (*Knipe et al.*, 1993)

$DEPTH_{pond}$ = current pond water depth (m)

BED_{pond} = pond bed thickness (m)

BED_LEVEL = bed level relative to datum (m)

GW_LEVEL = groundwater level relative to datum (m)

3.3.16 Minicluster Stormwater Store (MC SS)

After any runoff has been diverted to a detention basin (if present) then the remainder is assumed to flow through stormwater drains to the MC stormwater store (if present). If there have been no swales, soakaway or detention basin earlier in the treatment train then there is the option to exclude road runoff and/or unitblock runoff from the MC stormwater store. Otherwise, the store will receive runoff from the upstream treatment measures.

Inputs to the MC stormwater store are direct precipitation (if store is open-air) and runoff from the minicluster in which the store is sited. Outputs are overflow, evaporation (if open-air) and usage. Evaporation is at the potential rate provided there is sufficient water. There is an option to remove a fixed volume of user-specified first flush (FF) from the runoff. If the runoff volume is less than the FF then no stormwater is harvested. The FF is assumed to be discharged to the stormwater system. Leakage from stormwater supply systems is not modelled but, in comparison to the aging, centralised supply systems in the UK, leakage from a newly installed system is assumed be insignificant.

After precipitation and evaporation flows have been accounted for, overflow is deducted as any volume in excess of the store's capacity. Overflow is directed to the stormwater system.

3.3.17 Waterbodies

A proportion of any remaining stormwater after the MC SS (if present) may now be diverted to a river section or a lake. Rivers are split into segments, where each segment represents a section of the river that is relatively homogeneous (depth, width, absence of tributaries). A lake is treated as a single water body (Fig. 3.17).

Each river segment or lake may receive flows either from an upstream segment within the same waterbody (in the case of rivers) or from another waterbody. When a lake or river is situated near the boundary of the study area a fixed input flow rate can be specified, representing flow from upstream catchments outside the study area. It is a model restriction in *CWB* that each river segment or lake can only receive flow from one other upstream waterbody.

In addition to natural surface water flows, waterbodies can receive stormwater discharge from the centralised stormwater system (upstream miniclusters). Direct precipitation and evaporation (at the potential rate) are also accounted for. Waterbodies are not part of the miniclusters but their area is accounted for at the subcatchment scale.

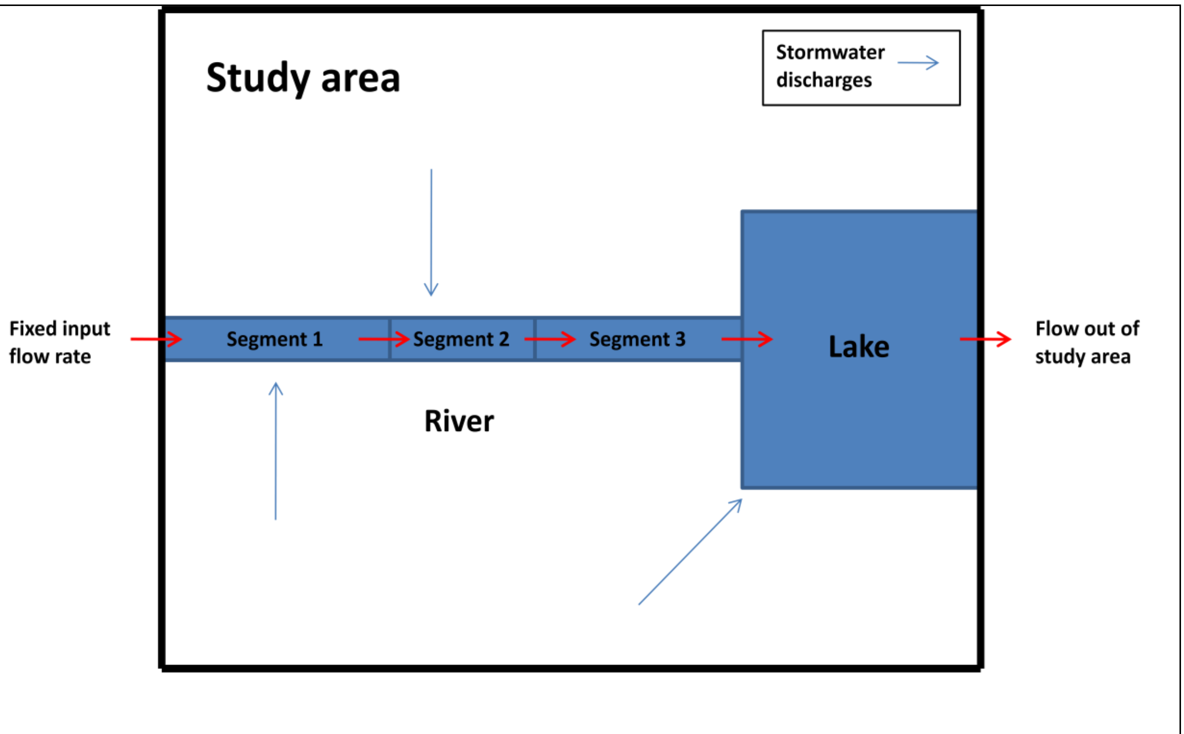


Figure 3.17 Example of waterbody flows.

Infiltration is calculated based on Darcy's Law and depends on the groundwater level, the waterbody bed level (set to 2m below ground level for that cell), the waterbody bed thickness and hydraulic conductance, and the water depth.

If the groundwater level is below the waterbody bed layer then:

(3.29) $$INF_{waterbody} = K_{waterbody} * (DEPTH_{waterbody} + BED_{waterbody})$$

Otherwise:

(3.30) $$INF_{waterbody} = K_{waterbody} * (DEPTH_{waterbody} + Ground\_LEVEL - GW\_LEVEL)$$

Where $INF_{waterbody}$ = infiltration from the waterbody segment (m3)

$K_{waterbody}$ = water area (m^2) * hydraulic conductivity of the waterbody bed (m/day) / bed thickness (m) (*Knipe et al.*, 1993)

$DEPTH_{waterbody}$ = current waterbody segment depth (m)

$BED_{waterbody}$ = waterbody bed thickness (m)

BED_LEVEL = bed level relative to datum (m)

GW_LEVEL = groundwater level relative to datum (m)

Flow in each river segment is calculated, similarly to flow through swales, by iterating to a new depth using a combination of Manning's equation and mass balance (see Eqs. 3.18 & 3.19).

It is assumed that outflow from lakes is over a broad-crested weir. If the water depth in the lake is less than the weir height then there is no outflow. If the water height is greater than the weir height then outflow is calculated based on equation 3.31 (*Linsley & Franzini*, 1979).

(3.31) $$Q = C_w * L * h^{3/2} * Seconds\_day$$

Where Q = flow over weir (m^3/s)

C_w = Weir co-efficient (user-specified)

Seconds_day = number of seconds in a day

L = length of the weir crest (m)

h = height of flow above weir crest (m)

Using the form of equation (3.24) the mass balance for lakes can be written:

(3.32) $$h^{new} = Q_{in} / [\ f(h^*) / h^* + g(h^*) / h^* + k(h^*) / h^*]$$

Where h initial value between zero and max water depth (m)

h^{new} = new water depth (m)

Q_{in} = input flow volume (storage from previous timestep + precipitation + inflow from other waterbodies) (m^3)

f(h) = discharge volume over the weir crest (m^3)

g(h) = final storage – initial storage = lake area * h

k(h) = infiltration (m^3)

It is then an iterative procedure to find h^{new}; the program loops until the difference between h and h^{new} is negligible ($1 * 10^{-10}$). Note that the evaporation outflow is not dependent on the water depth (provided there is sufficient water) and is deducted before the iteration.

Outflow from a river segment flows into the next segment of the same river. If it is the last segment then it will either flow into another waterbody or leave the study area. Similarly, outflow from lakes is either to a modelled downstream waterbody or it leaves the study area.

If the water level exceeds the capacity of the river channel in a segment then the excess water floods the riparian zone (if present). The capacity of the riparian zone is assumed to be infinite, so that all water is contained by it. The riparian zone extends either side of the river and is underlain by soil stores (Fig. 3.18).

Manning's flow through a flooded river segment uses the amended cross-sectional area and wetted perimeter and an increased Manning's coefficient to represent slower flow over the riparian zone. Lateral flow from the riparian soil store to the river is not modelled since it is assumed that the recharge from the soil store and subsequent rise of groundwater level will supply the river with a similar volume of water.

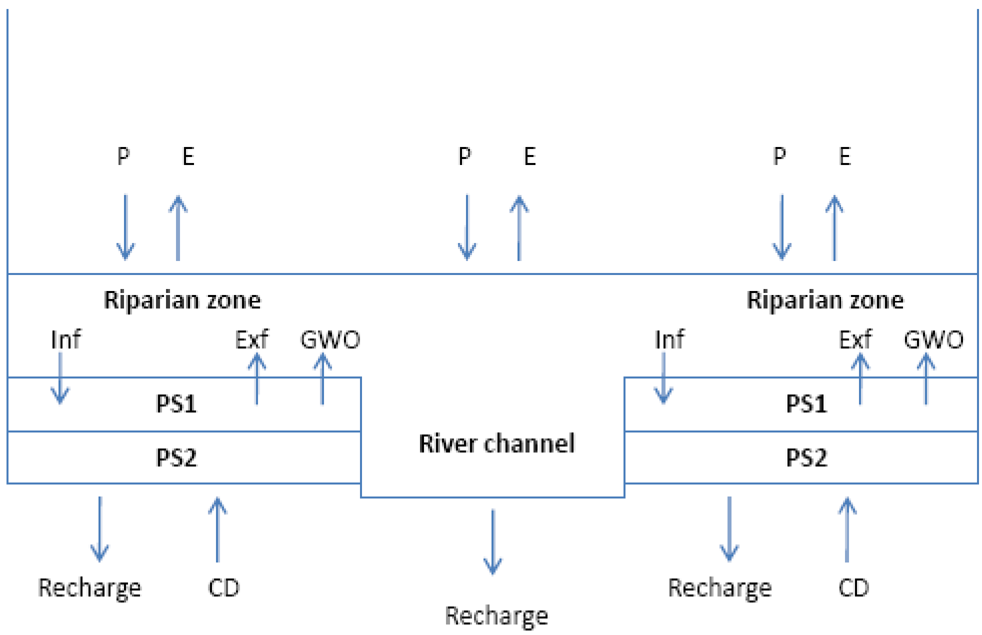


Figure 3.18 Cross-section of a river segment with riparian zone (using 2-layer soil model). Inf is infiltration from the surface water over the riparian zone to the soil store. Exf is exfiltration from the soil store to riparian surface water. GWO is groundwater overflow when the water table is higher than ground level. CD is capillary drawup.

3.3.18 Canals

In *CWB*, canals are modelled as water features with constant depth and constant leakage rate. Constant depth is a reasonable assumption since it is vital that canals are kept at operating depth of 1.5 m ± 150 mm (*Roberts*, 2009). If constant head in the canal is assumed then constant leakage rate is also reasonable. Infiltration is assumed to be zero from segments that do not overlie an unconfined aquifer.

If canals are assumed to maintain constant depth then inflow to the canal must equal outflow:

Demand = leakage + (evaporation – precipitation) + outflow from study area

The water demand for canals is satisfied by reservoirs and boreholes. Outflow from the study area as a result of the use of locks is not modelled. It is assumed that this unquantified volume of water is always matched by corresponding supply from reservoirs outside the study area in order to maintain constant operating depth in the canals. Therefore, the processes of importance that are modelled by *CWB* are canal recharge and direct precipitation/evaporation. In addition, borehole abstraction to supply canal water impacts groundwater level so is modelled when the borehole lies within the study area boundary.

Typically, much larger volumes of water are supplied by gravity from reservoirs to the canals, in preference to borehole abstraction, since it is cheaper (*Roberts*, 2009). The volume of leakage and (evaporation – precipitation) losses dictate the quantity of water abstracted from licensed boreholes and where this is insufficient, the remainder is supplied directly from reservoirs without further treatment.

3.3.19 Large Stormwater Store

MC stormwater may be diverted to a large stormwater store. The large store can receive stormwater from a number of miniclusters. Inputs to the store are direct precipitation (if tank is open-air) and runoff. Outputs are overflow, evaporation (if open-air) and usage. Evaporation is at the potential rate provided there is sufficient water. After precipitation and evaporation flows have been accounted for, overflow is deducted as any volume in excess of the store's capacity. Overflow is directed to the stormwater system.

Stormwater flows that are not directed to a large stormwater store form part of the subcatchment stormwater flow. The subcatchment stormwater flow is the sum of the flows leaving the miniclusters within it and this total flow can be directed into another subcatchment or leave the study area, depending on the drainage network. Stormwater (and wastewater) from upstream subcatchments is added to flow for the current subcatchment but it is currently modelled as unavailable for re-use.

3.3.20 Irrigation

Irrigation concepts are based on those used in *Aquacycle* (*Mitchell et al.,* 2001); irrigation is triggered when the soil moisture falls below a user-specified level - the trigger-to-irrigate level (TG). The TG is a fixed proportion of the soil store capacity. If at the end of the day the current storage is less than the TG level, irrigation (subject to availability) is applied to bring it up to the TG level. If the current storage exceeds the TG level then no irrigation is supplied.

For each unitblock type the user can specify a proportion of the garden area that is irrigated and similarly a proportion of POS in each minicluster that is irrigated. Irrigation may be supplied by stormwater or wastewater recycling stores or using mains water depending on the user preferences.

3.4 Wastewater

3.4.1 Septic Tank

Input to the septic tank (Fig. 3.19) is the wastewater from the indoor water use at unitblock scale. Outputs are throughflow and storage. Throughflow occurs when the level of fluid in the tank rises above the level of the outflow pipe. Throughflow from the tank is assumed to be either infiltrated into the unit block pervious store or routed to a UB soakaway. When routed to the garden it is split between PS1 and PS2 depending on their relative areas in the Partial Area Model (*Boughton*, 2003) and in the Two-Layer model (*Mitchell and Diaper*, 2005) drains entirely to the lower store (PS2).

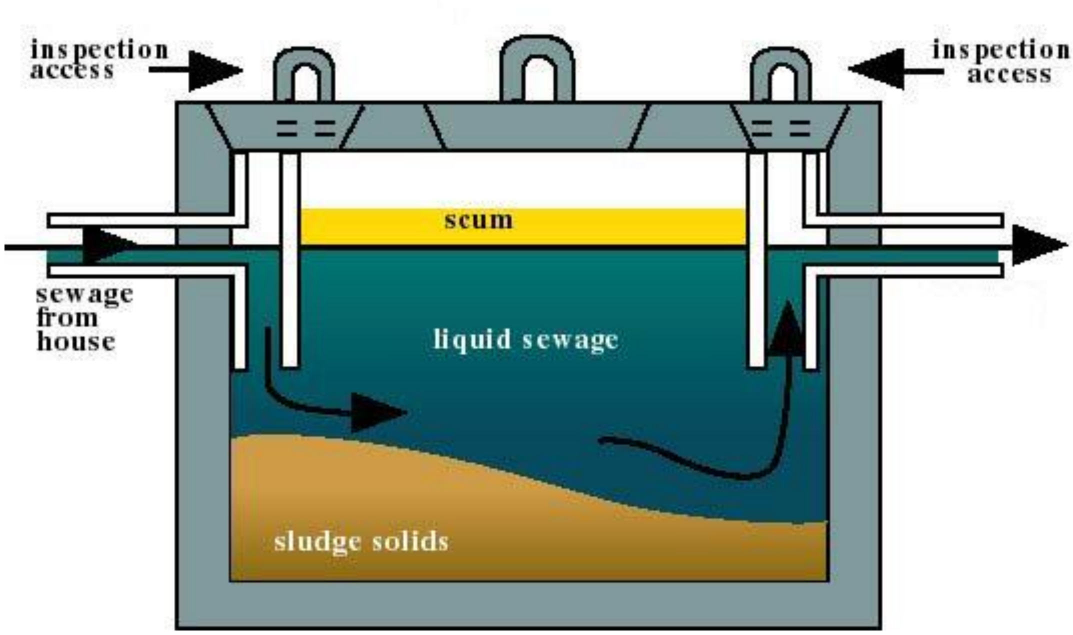


Figure 3.19 Example of a septic tank (*MTM Environmental*, 2010)

3.4.2 Unitblock Wastewater Store

The UB store can be set to receive waste flows from any or none of the four indoor uses at unitblock scale. Outputs are overflow and usage (indoor or irrigation). Overflow constitutes any volume greater than the capacity of the tank and is assumed to go to the wastewater system.

3.4.3 Minicluster Wastewater Store

MC wastewater stores are supplied by the sum of the wastewater from the unit blocks plus stormwater inflow and groundwater infiltration minus pipe leakage (exfiltration) to groundwater. Treated wastewater can be used to supply irrigation demands and Indoor use 1 within the minicluster. Outputs from the store are overflow and usage. Overflow is any volume greater than the capacity of the tank and is assumed to go to the wastewater system.

3.4.4 Large Wastewater Store

MC wastewater flows remaining after the MC Wastewater store (if present) may be diverted to a Large Wastewater Store. These stores can receive flows from a number of miniclusters. Outputs are overflow and usage. Overflow is any volume greater than the capacity of the tank and is assumed to go to the wastewater system.

Wastewater flows that are not directed to a Large Wastewater Store form part of the subcatchment wastewater flow. The subcatchment wastewater flow is the sum of the wastewater flows leaving the miniclusters within it. This total flow can be directed into another subcatchment or leave the study area, depending on the wastewater network.

3.4.5 Sewer Exfiltration and Groundwater Infiltration

The piped wastewater and stormwater systems are subject to leakage, which contributes to recharge, and infiltration in areas of high groundwater. Pipe leakage and infiltration occur at defective joints, cracks and if the pipe material is porous (e.g. porous concrete) (*Metcalf & Eddy*, 1991).

In *CWB* leakage from sewers is modelled as a fixed proportion of total flow. Infiltration of groundwater is modelled using Darcy's Law (with the hydraulic conductivity and pipe surface area assumed constant across the study area as a simplification).

When groundwater level > pipe elevation

(3.33) $Q = K.A.\Delta h/L$

Else

(3.34) $Q = 0$

Where Q = infiltration rate (m^3/day)

K = hydraulic conductivity (m/day)

A = surface area of pipe in contact with groundwater (m^2)

Δh = elevation difference (groundwater minus pipe) (m)

L = distance over which elevation difference occurs (m)

3.5 Urban Water Contamination

3.5.1 Introduction

Key parameters used to gauge pollution in urban water runoff are total suspended solids (TSS), biological oxygen demand (BOD), chemical oxygen demand (COD), Total nitrogen (TN), Total phosphorus (TP), heavy metals, hydrocarbons and bacteria (*Ellis & Mitchell*, 2006). Primary sources of contamination are animal faeces, plant detritus, dust, and vehicles (exhaust emissions, oil leakage and parts wear). Table 3.3 shows the typical composition of sewage. Sources of contamination in sewage are faeces, food particles, grease, oils, soap, salts, metals, detergents, plastic, sand and grit. It can be seen that BOD, COD, TSS, TN and TP are important measures of pollution in both runoff and wastewater flows.

Table 3.3 Typical compostion of sewage (*Gray*, 2005).

Parameter	US (mg/l)	UK (mg/l)
pH	7.0	7.0
BOD	250	350
COD	500	700
Suspended solids	250	400
Ammonia nitrogen	30	40
Nitrate nitrogen	<1	<1
Total phosphorus	10	15

In addition, Total Dissolved Solids (TDS) are also an important indicator of contamination in urban areas. TDS compliments the organic bulk measures by providing a measure of the total inorganic ion load, which includes heavy metals. Heavy metals pollution is primarily from vehicle emissions and industrial waste (*Gray*, 2005). Application of rock salt (sodium chloride) during winter, to ensure that roads do not become dangerously icy, introduces large quantities of inorganic ions, primarily sodium and chloride, into the water system. Sodium is highly soluble and reactive and may undergo ion exchange, releasing more toxic cations (such as heavy metals) into solution. Chloride is relatively unreactive and so is a much more persistant pollutant. Chloride accelerates corrosion, interferes with some industrial processes and can damage crops when applied in irrigation water (*Thomas*, 2001). High chloride levels render water unpotable. This is a particular problem in coastal areas, such as Dunedin (Florida), that face salt water intrusion as a result of groundwater abstraction for potable water supply.

3.5.2 Generic Concepts

Modelling of contaminants follows the concepts outlined in *UVQ* (*Mitchell and Diaper,* 2005). They are modelled conservatively between nodes, so there is no account taken of conversion or degradation in transport. A node is defined here as a point within the model where the concentration of a contaminant associated with a flow is altered, either by mixing of several flows, addition of a load or removal processes. An example of a node is a soil store where mixing of several water streams (e.g. infiltration, stored water, capillary drawup) and contaminant removal occur. Consequently, although modelling of contaminants in *CWB* uses generic calculations, so that in theory any species can be modelled, more accurate results will be obtained for urban water contaminants that are conservative in transport, such as chloride (which is a significant component of TDS).

Detailed modelling of contaminant interactions requires additional site specific data and increases the complexity of the model without necessarily improving accuracy *(Mitchell et al.,* 2003). These interactions are described in other models, such as *MUSIC* (*eWater Toolkit*, n.d.), which can be used in the next level of analysis.

A typical treatment node in *CWB* has various input streams (e.g. precipitation, runoff, wastewater, storage from previous timestep) each with associated contaminant concentrations. A node can be defined here as a point in the urban water cycle where the concentration of contaminant in the water (or wastewater) flow is altered, either by mixing of different flows, treatment or by addition of an external load (e.g. toilet). Treatment processes of waste and storm water are modelled as continuously stirred reactors (CTSRs), so the input streams are fully mixed at a node. Treatment is represented by a percentage removal of the total contaminant load (kg) as sludge. The term sludge is used, in this case, to represent all contaminant loads that are removed from the urban water system by treatment. It is assumed that this contaminant load is not remobilised in subsequent timesteps and has no further impact on the water system. The remaining load, after treatment, is carried from the node in the output streams (e.g. storage, overflow, water supply, infiltration, throughflow). The processes by which treatment occurs vary depending on the water management option and are not modelled explicitly but can be represented in the model by changing the proportion of contaminant that is removed.

In *CWB* treatment processes are modelled in the pervious stores, groundwater, and the water management options (borehole abstraction, wastewater recycling, rainwater harvesting, SUDS). At some nodes there is simple mixing of flows, without treatment (e.g. combining stormwater flows from roofs and paved areas).

The user is required to enter contaminant concentrations, which remain constant throughout the simulation, for the following flows:

- precipitation
- evaporation
- mains water
- pavement runoff
- roof runoff
- roof first flush
- road runoff

and input loads for:

- each of the four indoor water demand categories (e.g. bathroom, toilet, kitchen, and laundry in a house) for each minicluster
- fertiliser

In addition, initial contaminant concentrations need to be provided for surface water bodies (canals, rivers, and lakes) and for groundwater.

3.5.3 Example Treatment Node – Unitblock Raintank

The input load (kg) to the tank is from direct precipitation (if open-air), roof runoff from the ERA and overflow and drainage from the greenroof (if present):

(3.35) Roof_runoff_L = IRUN_roof_V*IRUN_roof_C + (Green_over + Green_drain) * Green_out_C

Where IRUN_roof_V = runoff from effective roof area (m^3)

IRUN_roof_C = contaminant conc. In IRUN_roof_V (kg/m^3)

Green_over = overflow volume from green roof (m^3)

Green_drain = drainage from green roof (m^3)

Green_out_C = contaminant conc. Of output flow from green roof (kg/m^3)

The concentration of first flush removal is user-defined. If the first flush load is greater than the load of contaminant in the roof runoff, in a particular timestep, then it is set equal to the roof runoff load for that timestep.

The concentration of contaminants in water supplied from the raintank is that of the tank output at the end of the previous timestep. A percentage of the input load, minus first flush, plus the load already in storage, minus usage, is removed as sludge. The remaining load, after treatment, is carried by the output flows.

The concentration of the output flows is the output load divided by the sum of the output volumes:

(3.36) RT_out_C = RT_out_L / (Storage + Overflow)

3.5.4 Water Usage Loads

Mains supply has a fixed, user-specified concentration for each contaminant species modelled. If a water demand is satisfied by another supply, the concentration of contaminant in that supply will be the concentration in the supplies' store at the end of the previous timestep (before new flows, with associated loads, have been added to the store, as waste from water applications, for example, in the current timestep).

Fixed contaminant loads (kg/unit of occupancy factor) are added to the waste streams from each of the four indoor uses during each timestep.

The irrigation load to a pervious store is the sum of the loads from the individual supplies. In the partial areas model the load is distributed proportionally with the irrigation volume to PS1 and PS2. For the two-layer model all the load is added to PS1.

3.5.5 Impervious Loads

Output flows from roof, paved and road areas have user-specified fixed concentrations, regardless of the concentration of the input load. This represents the wash-off of additional contaminants introduced onto the impervious surfaces by processes such as dry deposition and sources like animal faeces and vehicles.

Roof

The load that is retained on the roof in this timestep is the sum of the load present on the roof in the previous timestep and precipitation minus runoff (*Mitchell and Diaper*, 2005).

(3.37) $$\text{Roof\_L}^{(t)} = \text{Roof\_L}^{(t-1)} + (P * Pc) * (\text{Roof\_area} - \text{Green\_area}) - \text{IRUN\_roof\_C} * \text{IRUN\_roof} - \text{NEAR\_roof\_C} * \text{NEAR\_roof}$$

Where Roof_L $^{(t)}$ = Load of contaminant on non-green part of roof (kg)

IRUN_roof_C = Concentration of runoff from the effective roof area (kg/m^3)

IRUN_roof = Volume of runoff from ERA (m^3)

NEAR_roof_C = Concentration of runoff from non-ERA (kg/m^3)

NEAR_roof = Volume of runoff from non-ERA (m^3)

Note that IRUN_roof_C and NEAR_roof_C are user specified fixed concentrations, so the roof can act as a pollutant source.

Pavement

Similarly, the load that is retained on the pavement in this timestep is (*Mitchell and Diaper,* 2005):

(3.38) $$Pave\_L^{(t)} = Pave\_L^{(t-1)} + (P * Pc) * Pave\_area + 0.5 * (1 - PP\_prop) * NEAR\_roof\_C * NEAR\_roof - IRUN\_pave\_C * IRUN\_pave - NEAR\_pave\_C * NEAR\_pave$$

Where $Pave\_L^{(t)}$ = Load of contaminant on pavement (kg)

PP_prop = proportion of pavement that is porous (-)

IRUN_pave_C = Concentration of runoff from the effective paved area (kg/m^3)

IRUN_ pave = Volume of runoff from EPA (m^3)

NEAR_ pave _C = Concentration of runoff from non-EPA (kg/m^3)

NEAR_ pave = Volume of runoff from non-EPA (m^3)

Note that IRUN_pave_C and NEAR_pave_C are user-specified fixed inputs and consequently the paved area can act as a pollutant source.

Road

If the precipitation is less than 0.7mm (*eWater Toolkit*, n.d.) then it is assumed that no contaminants are mobilised in the road runoff (Road_runoff_C = 0). Otherwise Road_runoff_C is a fixed user-specified concentration (*Mitchell and Diaper,* 2005).

(3.39) $$Road\_L^{(t)} = Road\_L^{(t-1)} + (P * Pc) * Road\_area - Road\_runoff\_C * (IRUN\_road + NEAR\_road)$$

3.5.6 Pervious Stores

Input loads

Garden

Input load to the Partial Area garden model is storage ($PS\_L^{(t-1)}$), half of the NEAR load from the roof, the NEAR load from the paved area, throughflow from the septic tank (if present), precipitation and irrigation (Eq. 3.40). Input loads (apart from storage) are divided between PS1 and PS2 proportionally to their surface area.

$$PS\_input\_L = PS\_L^{(t-1)} + 0.5 * NEAR\_roof\_L + NEAR\_pave\_L + Septic\_out\_L + P * P_c * PS\_area + Irrigation\_L \quad (3.40)$$

Input load to PS1 of the Two-layer garden model is storage ($PS1\_L^{(t-1)}$), half of the NEAR load from the roof, the NEAR load from the paved area, precipitation and irrigation:

$$PS1\_input\_L = PS1\_L^{(t-1)} + 0.5*NEAR\_roof\_L + NEAR\_pave\_L + P*P_c*PS1\_area + Irrigation\_L \quad (3.41)$$

Input load to PS2 is storage, overflow and infiltration from the septic tank (if present), and drainage from PS1:

$$PS2\_input\_L = PS2\_L^{(t-1)} + Septic\_out\_L + Drainage\_PS1*PS1\_out\_C \quad (3.42)$$

POS

Input load to the Partial Area POS model is storage ($PS\_L^{(t-1)}$), NEAR from the road area within the MC, precipitation and irrigation (Eq. 3.43). Input loads (apart from storage) are divided between PS1 and PS2 proportionally to their surface area.

(3.43) $PS\_input\_L = PS\_L^{(t-1)} + NEAR\_Rd\_L + P*P_c*PS\_area + Irrigation\_L$

Input load to PS1 of the Two-layer POS model is storage ($PS1\_L^{(t-1)}$)), NEAR from the road area within the MC, precipitation and irrigation:

(3.44) $PS1\_input\_L = PS1\_L^{(t-1)} + NEAR\_Rd\_L + P*P_c*PS1\_area + Irrigation\_L$

Input load to PS2 is storage from the previous timestep and drainage from PS1:

(3.45) $PS2\_input\_L = PS2\_L^{(t-1)} + Drainage\_PS1 * PS1\_out\_C$

Output loads

A percentage of PS_input_L is removed as sludge in each store. The output load (PS_out_L) is the remainder. After the evaporation load has been removed the output concentrations are:

Partial Area

(3.46) PS_out_C = PS_out_L / (Excess + Storage)

The excess flow is split into surface runoff and groundwater recharge.

Two-Layer

(3.47) PS1_out_C = PS1_out_L / (Storage + Surface_runoff + Drainage_PS1)

(3.48) PS2_out_C = PS2_out_L / (Storage + Drainage_PS2)

Drainage from PS2 contributes to groundwater recharge.

3.5.7 Waterbodies

Minicluster flow after the MC stormwater store (if present) can be diverted to a waterbody (WB). In addition a lake or river segment may receive flow from an upstream waterbody:

(3.49) $\text{Input\_flow\_L (kg)} = \text{WB\_upstream\_out\_C (kg/m}^3\text{)} * \text{WB\_upstream\_out\_V (m}^3\text{)}$

There may also be a constant input stream defined for the first river segment or a lake, representing flow from outside the study area:

(3.50) $\text{Input\_flow\_L} = \text{Input\_flow\_L} + \text{Init\_C} * \text{Input\_fixed\_V}$

Where Init_C = fixed concentration of contaminant (kg/m^3)

Input_fixed_V = fixed input flow (m^3)

A river segment, other than the first, will receive flow from the previous segment:

(3.51) $\text{Input\_flow\_L} = \text{Input\_flow\_L} + \text{Water\_out\_L}_{(\text{segment-1})}$

In addition there is the load carried within the current volume and load added with direct precipitation:

(3.52) $\text{Input\_flow\_L} = \text{Input\_flow\_L} + \text{Storage\_L}^{(t-1)} + P * P_c * \text{Water\_area}$

Since loads are modelled conservatively the output load is equal to the input load. Output concentration is:

(3.53) $\text{Water\_out\_C} = \text{Water\_out\_L} / (\text{Storage} + \text{Infiltration} + \text{Throughflow} + \text{Overflow})$

3.6 Groundwater

Most programs for groundwater modelling are based on one of three methods: finite differences, finite elements or analytic elements. Of these, the finite difference method is the most widely used because it is quite versatile and relatively simple (*Fitts*, 2002) and for these reasons it has been adopted for use in *CWB*. The finite difference method calculates flows, in each timestep, using a series of equations based on Darcy's Law and conservation of mass.

In this model the groundwater system is represented by a 2D grid of cells (dimensions Δx and Δy) underlying the study area. The grid consists of Xmax * Ymax nodes with each x,y coordinate representing a cell centre. The structure of the grid is entirely independent of the surface landuse description (subcatchments and miniclusters).

Flows into a cell can be recharge from an overlying minicluster or flows from adjacent cells. Flows out of a cell can be into an adjacent cell, capilliary drawup to the soil store or exfiltration to the surface.

Assumptions

- Adoption of fully implicit modelling is reasonable since the accuracy of the Crank-Nicolson method is not required.

- Recharge from a minicluster is assumed to be uniformly distributed over its area. Recharge to a cell (m^3) = Total MC recharge (m^3) * Cell overlap area (m^2) / MC area (m^2)

- No flow across the unconfined aquifer boundary

- Contaminant removal efficiency is the same as for soil stores

Inputs

At the minicluster scale the input load to the groundwater store (GWS) is:

(3.54) GWS_input_L = BLOCK * (UB_L + PP_L + UB_Swale_L) + PA_L + Soak_L + POS_L + Filter_L + MC_Swale_L + Leakage_L + Exfiltration_L – Infiltration_L

The flows to each cell are from the miniclusters, waterbodies and ponds. Equations 3.55 and 3.56 are looped over all miniclusters, waterbodies and ponds:

(3.55) Cell_input_V = Cell_input_V + Pond_inf * Cell_prop / Pond_area + _
WB_inf * Cell_prop/WB_area + _
(MC_input– UB_Bore) * Cell_prop/MC_area

Where Cell_prop = the proportion of the cell covered by a landuse (-)

Pond_area = total number of cells that the pond overlies (-)

UB_Bore = unitblock borehole abstraction (m^3)

(3.56) Cell_input_L = Cell_input_L + Pond_inf * Cell_prop / Pond_area + _
WB_inf * Cell_prop/WB_area + _
(GWS_input_L – UB_Bore_L)
* Cell_prop/MC_area

In addition there are abstractions from Large Boreholes:

(3.57) Cell_input_V = Cell_input_V - Large_Bore

(3.58) Cell_input_L = Cell_input_L - Large_Bore_L

Sewer infiltration is modelled using Darcy's Law:

(3.59) $Q_{inf} = k * A_{sewer} * MC_{area} * (GWL – Sewer\_level\_Aod)$

Where $k \sim 0.2$ m/day (*Karpf & Krebs*, 2004)

A_{sewer} = Area of sewer / unit area of MC (m^2)

Data input requirements for each cell are:

a) Proportion of cell overlain by each MC

b) Proportion of cell overlain by waterbody

c) Proportion of cell that is unconfined aquifer (Fig. 3.20)

d) Transmissivity in the x-direction (note that transmissivities are assigned to cell walls not the nodes – Fig. 3.20)

e) Transmissivity in the y-direction

Darcy's law for flow from cell (i -1) to cell (i) can be written:

$$Q = K.A.(\mathbf{H}^{\mathbf{t}}_{\mathbf{i-1}} - \mathbf{H}^{\mathbf{t}}_{\mathbf{i}}) / \Delta x = K.b.\Delta y.(\mathbf{H}^{\mathbf{t}}_{\mathbf{i-1}} - \mathbf{H}^{\mathbf{t}}_{\mathbf{i}}) / \Delta x \quad (3.60)$$

The product Kb is called the transmissivity (T) and so:

$$Q = T_x.\Delta y/ \Delta x.(\mathbf{H}^{\mathbf{t}}_{\mathbf{i-1}} - \mathbf{H}^{\mathbf{t}}_{\mathbf{i}}) \quad (3.61)$$

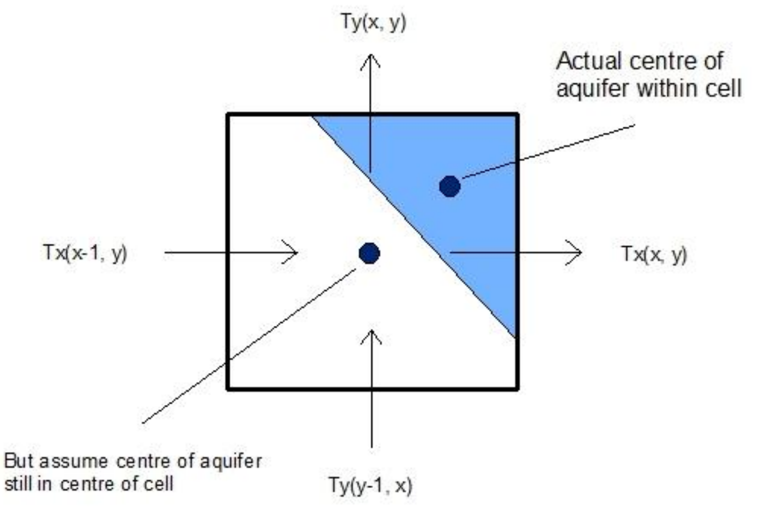


Figure 3.20 Diagram showing a cell at the unconfined aquifer boundary. For recharge calculation purposes the area is redefined as $\Delta x.\Delta y.\text{Factor}_{area.}$

The change in storage for each cell is the sum of flows from other cells (based on Eq. 3.61) plus recharge (*Fitts*, 2002):

$$(3.62)\quad (\mathbf{H^t_{i-1}} - \mathbf{H^t_i}).\,Tx_{i-1} + (\mathbf{H^t_{i+1}} - \mathbf{H^t_i}).\,Tx_i + (\mathbf{H^t_{j-1}} - \mathbf{H^t_i}).\,Ty_{j-1} + (\mathbf{H^t_{j+1}} - \mathbf{H^t_i}).\,Ty_j + R(i,j) = S_c.(\mathbf{H^t_i} - H^{t-1}_i)$$

Where H^t_i = height of water above datum in cell (i,j) at time t (m)

$S_c = S_y.\Delta x.\Delta y.\text{Factor}_{area}$ (m^2/day)

S_y = Specific yield, fixed at 0.15 (*Price and Reed*, 1989) (-)

$Tx = T_x.\Delta y/\ \Delta x$ (m^2/day)

$Ty = T_y.\Delta x/\ \Delta y$ (m^2/day)

R(i,j) = recharge volume to cell (i,j) (m^3)

Re-arranging Equation 3.62 to isolate the new head for the current node yields:

(3.63) $$\mathbf{H^t_{i-1}}.Tx_{i-1} + \mathbf{H^t_{j-1}}.Ty_{j-1} + \mathbf{H^t_{i+1}}.Tx_i + \mathbf{H^t_{j+1}}.Ty_j - \mathbf{H^t_{i.}}(Tx_{i-1} + Tx_i + Ty_{j-1} + Ty_j + S_{ci}) = -(H^{t-1}_i.S_c + R(i,j))$$

And:

(3.64) $$\mathbf{H^t_i} = [(H^{t-1}_i.S_c + R(i,j)) + (\mathbf{H^t_{i-1}}.Tx_{i-1} + \mathbf{H^t_{j-1}}.Ty_{j-1} + \mathbf{H^t_{i+1}}.Tx_i + \mathbf{H^t_{j+1}}.Ty_j)] / (Tx_{i-1} + Tx_i + Ty_{j-1} + Ty_j + S_c)$$

Where the recharge is:

(3.65) $$R(i,j) = R(\text{per unit area}) * \Delta x * \Delta y * \text{Factor}_{area}$$

And the storage change (ΔS) is:

(3.66) $$\Delta S = S_y * \Delta x * \Delta y * (H^t_{(i,j)} - H^{t-1}_{(i,j)})$$

Where $H^{t-1}_{(i,j)}$ is the head at the start of the timestep... i.e. the head in the cell before any calculation of the new heads has been carried out.

The calculation of the new central node head (Eq. 3.64), assuming that neighbouring heads are fixed, forces the local error to zero. Using an iterative scheme, over all the nodes, forces nodal imbalances across the grid towards zero.

In order to track the contaminant fluxes it is necessary to know the flow volumes between the cells. If the groundwater height is known across the grid (this has just been calculated) the base flows can be calculated as follows, based on Darcy's Law:

(3.67) $$Q_{(i-1)} = T_x *(H^t_{(i-1,j)} - H^t_{(i,j)})$$

(3.68) $$Q_{(i+1)} = T_x *(H^t_{(i,j)} - H^t_{(i+1,j)})$$

(3.69) $$Q_{(j-1)} = T_y *(H^t_{(i,j-1)} - H^{t(}_{i,j)})$$

(3.70) $$Q_{(j+1)} = T_y *(H^t_{(i,j)} - H^t_{(i,j+1)})$$

Note that flow in the positive x and y directions is considered positive. The mass balance for the water volume in a cell can be written:

(3.71) $\Delta S = Q_{(i-1)} - Q_{(i+1)} + Q_{(j-1)} - Q_{(j+1)} + R(i,j)$

Flows are assigned concentrations equal to concentration (from the previous timestep) of the groundwater in the source cell. The new cell load ($Cell\_L^{t}_{(i,j)}$) is:

(3.72) $Cell\_L^{t}_{(i,j)} = Cell\_L^{t-1}_{(i,j)} + C1 * Q_{(i-1)} + C2 * Q_{(j-1)} - C3 * Q_{(i+1)} - C4 * Q_{(j+1)}$

Where $Cell\_L^{t-1}_{(i,j)}$ = load in cell(i,j) at the end of the previous timestep (kg)

C1 = either concentration in cell(i-1, j) or cell (i,j) at the end of the previous timestep (kg/m^3)

C2 = either concentration in cell(i, j-1) or cell (i,j) at the end of the previous timestep (kg/m^3)

3.7 Energy and Cost

3.7.1 Introduction

Simplified life cycle energy use and whole life cost of decentralised water management options are calculated by *CWB*.

Full life cycle analysis has not been attempted, as the level of detail and data requirements are beyond the scope of this model. The four stages in a full life cycle analysis are documented in the ISO 14040 series of standards (*International Organisation for Standardization*, 1997): scope of study, inventory analysis, impact assessment and interpretation.

The scoping stage addresses the goal of the study (reasons for doing the study and intended audience) and describes the processes in the system, the system boundaries, data requirements and limitations. The inventory analysis involves data collection to quantify inputs and outputs from processes within the system boundary. Impact assessment evaluates the potential environmental impacts of the inputs and outputs calculated in the inventory analysis. Mandatory elements of this stage, according to ISO 14042, are: selection of impact categories, category indicators and characterization models (category definition); assignment of inventory results to impact category (classification) and calculation of category indicator results (characterisation). In the interpretation significant issues based on the results are identified in light of the goal of the study, sensitivity analysis may be conducted and recommendations are made.

A simplified life cycle analysis is the application of the LCA methodology for a screening assessment – using qualitative or generic quantitative data). Standard data for transportation and energy production are used, followed by simplified impact assessment that focuses on several key indicators. The rationale behind simplified life cycle assessment is to obtain similar results to a full LCA but more rapidly, with fewer data and less expense (*Friedrich et al.*, 2006).

In *CWB* a simplified life cycle inventory of energy use is calculated. Impact assessment is not performed but could be added in the future. Life cycle energy use can be categorized as follows (*Gaterell & Lester*, 2000): construction, operation, and decommissioning. The construction phase includes embodied energy (energy used to extract, process and manufacture a product) and transportation to site; and the operation phase includes maintenance. Decommissioning energy costs are assumed to be small in comparison to the other phases of the lifecycle. This is especially true for SUDS due to their landscaped nature and lack of hard infrastructure (Woods-Ballard et al., 2007). Further, *Gaterell & Lester* (2000) found in their study of centralised wastewater treatment plants that the decommissioning phase accounted for less than 5% of the total global warming potential. The most significant contributor to global warming potential is carbon dioxide, whose emissions are directly related to the use of energy sourced from fossil fuels.

The same categories of contruction and operation were applied in a whole life cost of the decentralised water management options. Whole life costing (WLC) is the identification of future costs and benefits and referring them back to present day costs using a standard accounting technique like present value (*Woods-Ballard et al.,* 2007). The WLC for several of the SUDS (porous pavement, porous asphalt, filterstrips, soakaways, ponds and swales), that are modelled in *CWB*, is well-documented (*Woods-Ballard et al.,* 2007) and so is not repeated. Instead, results from this study are used directly as input into *CWB*.

For each of the decentralised water management (WM) options in *CWB* simplified life cycle energy and cost stand-alone spreadsheet models have been developed, which serve as a technology library to *CWB.* A description of the lifetime cost and energy calculations for borehole abstraction is outlined below. Calculations for the other WM options are based on the same concepts. The cost and energy for supply and treatment using the conventional centralised piped system uses recent average energy and cost values per m^3 supplied and treated. Table 3.4 shows some key parameters that are used throughout the energy and cost calculations.

Table 3.4 List of key energy and cost input parameters.

Parameter	Value	Unit	Source
Density of topsoil	1600	kg/m^3	
Electricity cost	0.08	£/kWh	
Diesel cost	1.1	£/litre	Birmingham pump prices (2010)
Conversion (Diesel to kWh)	*38.4 / 3.6	NA	
Hours in working day	10	NA	(*Bridgeman*, 2009)

In the section that follows, an example of the calculation of life cycle energy use and WLC by *CWB* for one of the water management options is shown in detail.

3.7.2 Example – Borehole Abstraction

Key parameters specific to borehole abstraction are: Lifetime of longest lasting component of borehole (years), Main tank volume (m^3) and Annual supply (m^3). Annual supply is calculated by *CWB.*

Borehole Energy

Key parameters for the energy analysis are water lift energy costs (kWh/m^3), chemical dosage (mg/l) and embodied energy (kWh/kg).

Embodied energy (EE)

Information for the following parameters is required for all components:

- Mass of material (kg - based on density and volume)
- EE (kWh/kg)
- Lifetime of component (years)

The mass of the filters and pumps used is subject to multiplication by a factor of: (number toilets in UB) / 2. This is to take account of the fact that more filters and larger pumps will be required for unitblocks with more toilets.

Pipe volumes are calculated based on length, diameter and wall thickness, and cable volumes on diameter and length.

The total embodied energy of the borehole is the sum of the lifetime embodied energy of all the components:

(3.73) EE_Total = Comp_mass * Comp_EE*Borehole_life / Comp_life

Construction

Parameters used in the calculation of energy use during construction are:

- Tank density and wall thickness → Material_V = Tank_V$^{(1/3)}$ * 6*Wall_thick
- Tank clearance → Hole_V = (Tank_V$^{(1/3)}$ + Tank_clear)^{3}
- Trench dimensions for pipes (length, width, depth and consequently volume)

For each machine used, the following is specified:

- Distance travelled (km) and fuel economy (l diesel / km) loaded and unloaded
- Tonnage shifted (T) and fuel cost (l diesel / T)

CWB “recognises” several machine types in the calculation of tonnage shifted:

Digger: Mass shifted = (2*(Trench_V + Hole_V) – Tank_V) * Density_topsoil / 1000

Lorry: Mass shifted = Tank_V * Density_topsoil / 1000

If a machine name is not recognised then mass shifted is assumed to be zero.

The total construction energy usage is:

(3.74) Constr_E_Total (kWh) = (Dist_loaded*Fuel_econ_loaded + Dist_unloaded * Fuel_econ_unloaded + Mass_shifted * Fuel_econ_mass) * Conversion

Maintenance

Parameters used for the calculation of energy use of each maintenance operation are:

- Maintenance distance and frequency
- Fuel cost (litres diesel / km)

Operation

The lifetime energy usage for each type of chemical treatment used is:

(3.75) Consume_E_total (kWh) = Supply_annual * dosage * Borehole_life * Chemical_EE

The lifetime energy used in pumping is:

(3.76) Pump_E_total (kWh) = Supply_annual * Pump_rating * Borehole_life

CWB does not currently model varying pump energy use (kWh/m^3) with depth to water.

Total

The lifetime energy cost of the project is the sum of energy used during construction, embodied energy, maintenance energy costs and operation energy costs:

(3.77) Energy_Total = Constr_E_Total + EE_Total + Maint_E_Total + Operation_E_Total

Borehole Cost

Key parameters used in the calculation of borehole cost are:

- Number of toilets per unitblock. More toilets require more pipes and increase pumping costs.
- Number of unitblocks
- Subsidy proportion (proportion of total cost that is subsidised). Subsidy is optional and allows the cost to the individual or business (with government funding) to be investigated not just the cost to society.

Capital cost (CC)

For each component the following are specified for the calculation of capital cost:

- Number of units (where a unit is the most appropriate description of quantity for that component. For example, the unit of the main tank is cubic metres and the unit of a pipe is metres)
- Cost per unit (£)
- Lifetime of component (years)

(3.78) CC_Total = Num_Units * Unit_cost * Borehole_life / Comp_life

Construction

For each machine used in construction the following parameters are used in the cost calculation:

- Number of hire days and cost (£/day)
- Operator cost (£/hr)

Number of hire days is based on the size of the project, which is related to the size of the main tank (Tank_V). *CWB* uses the following rules:

If Tank_V <= $2m^3$ then Num_Hire_Days = user entered value in the spreadsheet

If $2m^3$ < Tank_V <= $6m^3$ then Num_Hire_Days = 4

If $6m^3$ < Tank_V <= $10m^3$ then Num_Hire_Days = 6

If $10m^3$ < Tank_V then Num_Hire_Days = 10

The number of labour hours is:

(3.79) Labour_hrs = Num_Hire_Days * 7

The total construction cost is the sum, over all the machines used:

(3.80) Constr_Cost_Total = Labour_hrs * Operator_cost + Num_Hire_Days*Hire_Cost + Diesel * Cost_Diesel

Where Diesel = total diesel used in construction (calculated in the energy section)

Cost_Diesel = cost per litre of diesel (£/l)

Maintenance

Parameters used in the cost calculation for each type of maintenance are:

- Frequency (per year)
- Cost per visit (£)

Net present value (NPV) maintenance cost (Maint_cost) for each type is calculated as follows:

For i = 1 to Borehole_life

Discount_factor = (1 + Discount_rate)$^{(i-1)}$

Maint_cost = Maint_cost + Cost_per_Visit * Freq / Discount_factor

Next i

Where Borehole_life = lifetime of the borehole (years)

Operation

Knowing the cost (£/kg) of consumables, and the annual supply, allows calculation of the NPV of the consumables over the lifetime of the borehole:

For i = 1 to Borehole_life

Discount_factor = (1 + Discount_rate)$^{(i-1)}$

Consume_cost = Consume _cost + Cost_per_kg *Dosage*Annual_supply / Discount_factor

Next i

The annual electricity usage (calculated in the energy section) multiplied by the cost of electricity allows the calculation of its NPV over the borehole lifetime.

For i = 1 to Borehole_life

Discount_factor = (1 + Discount_rate)$^{(i-1)}$

Elec_cost = Elec_cost + Annual_supply * Pump_rating* Cost_elec / Discount_factor

Next i

Total Cost

The lifetime cost of the project is the sum of the construction, capital, maintenance and operation costs:

(3.81) Cost_Total = Constr_Cost_Total + CC_Total + Maint_Cost_Total + Operation_Cost_Total

The total cost minus the subsidy is:

(3.82) Cost_Total_Subsidy = Cost_Total * (1 – Subsidy_prop)

Annual averaged costs are calculated by dividing by the lifetime of the borehole. Cost per cubic metre supplied is calculated by dividing the cost by the lifetime supply.

3.8 Summary

In this chapter model concepts for *CWB* have been described. This began with key concepts related to spatial characterisation and water demand. Subsequently water supply, runoff and wastewater flows were explained. Rules for pollutant pathways were then outlined followed by a description of groundwater dynamics in the model. Finally, energy and cost calculations were detailed.

In the following chapters there is a description of the case study background and preparation and calibration of the model. This is followed by validation and a basic sensitivity analysis. Three future scenarios are modelled and the results are presented and discussed. After the conclusion, recommendations are given for future improvements to *CWB*.

4 – The Birmingham Case Study

Birmingham was used as a case study in order to test the model, to establish data requirements for a city, and to investigate water management strategies against future scenarios. The city is appropriate as a model subject for two main reasons: firstly it contains the major attributes that are relevant to the design of *CWB* (diverse land use, range of natural systems, developed groundwater system) and secondly there was good access to data required to run the model (primarily: land use, groundwater, geologic, topographic, climate, water demand, sewer network). Close proximity to the local water company, Severn Trent Water, facilitated the acquisition of important data. In particular, historic supply and stormwater/wastewater flow data enabled calibration and validation of water flow in the model.

Birmingham is located in central England (Fig. 4.1) and is the second most populated British city after London. It has an elevation of 75 to 350 m above sea level and a population of approximately one million people (*Office for National Statistics*, 2001a). The City of Birmingham, the focus of this case study (Fig. 4.2), is part of the larger West Midlands conurbation which includes several neighbouring towns and cities; such as Solihull, Wolverhampton and the towns of the Black Country.

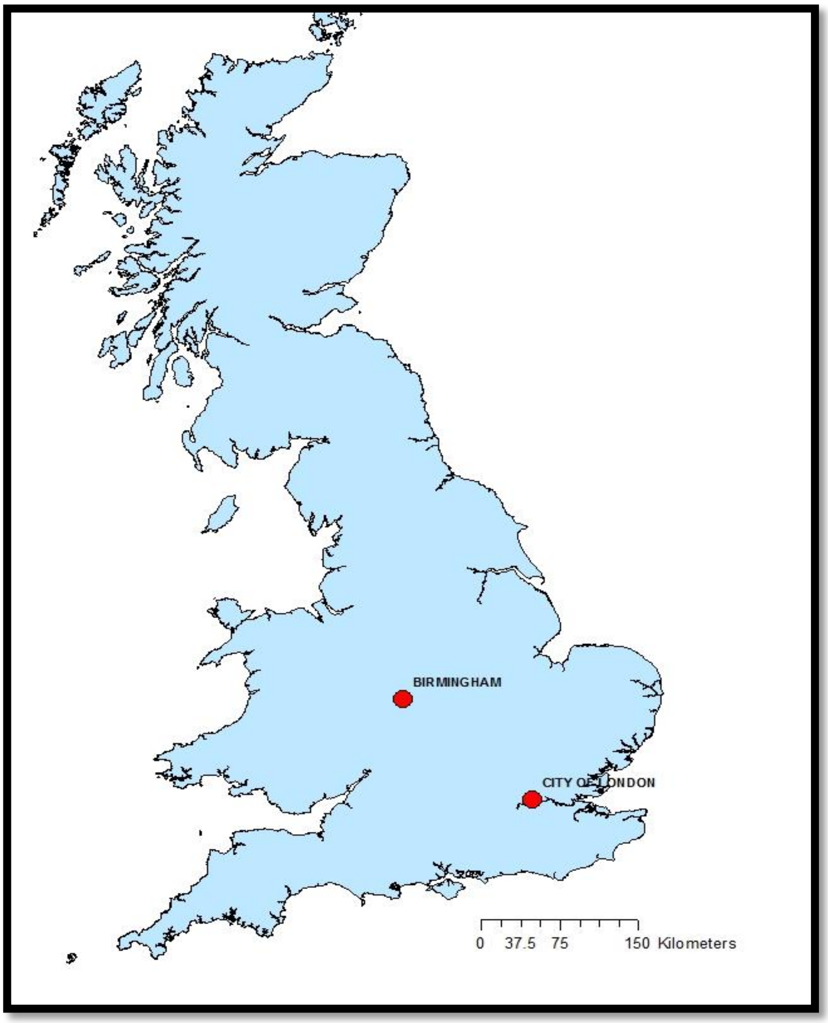


Figure 4.1 Location of Birmingham within the UK.

Although Birmingham's industrial importance has declined since the post-war economic boom during the period 1950-1970 (*Lerner*, 2004), it has developed into a national commercial centre, being named as the second best place in the UK to locate a business, the 14th best in Europe (*Birmingham Post*, 2009) and the fourth most visited city by foreign visitors in the UK (*Khatri & Vairavamoorthy*, 2009). In 2003 the service sector accounted for 78% of the city's economic output and 97% of its economic growth (*National Office for Statistics*, 2003). The Birmingham area accounts for 42% of the UK's conference and exhibition revenue (*NEC Group*, n.d.). It has three universities and two university colleges which have over 65,000 students and employ approximately 15,000 staff. It also has the country's busiest shopping centre (*icBirmingham*, 2004), the Bullring, and the largest department store outside of London, House of Fraser (*PropertyMall*, 1998).

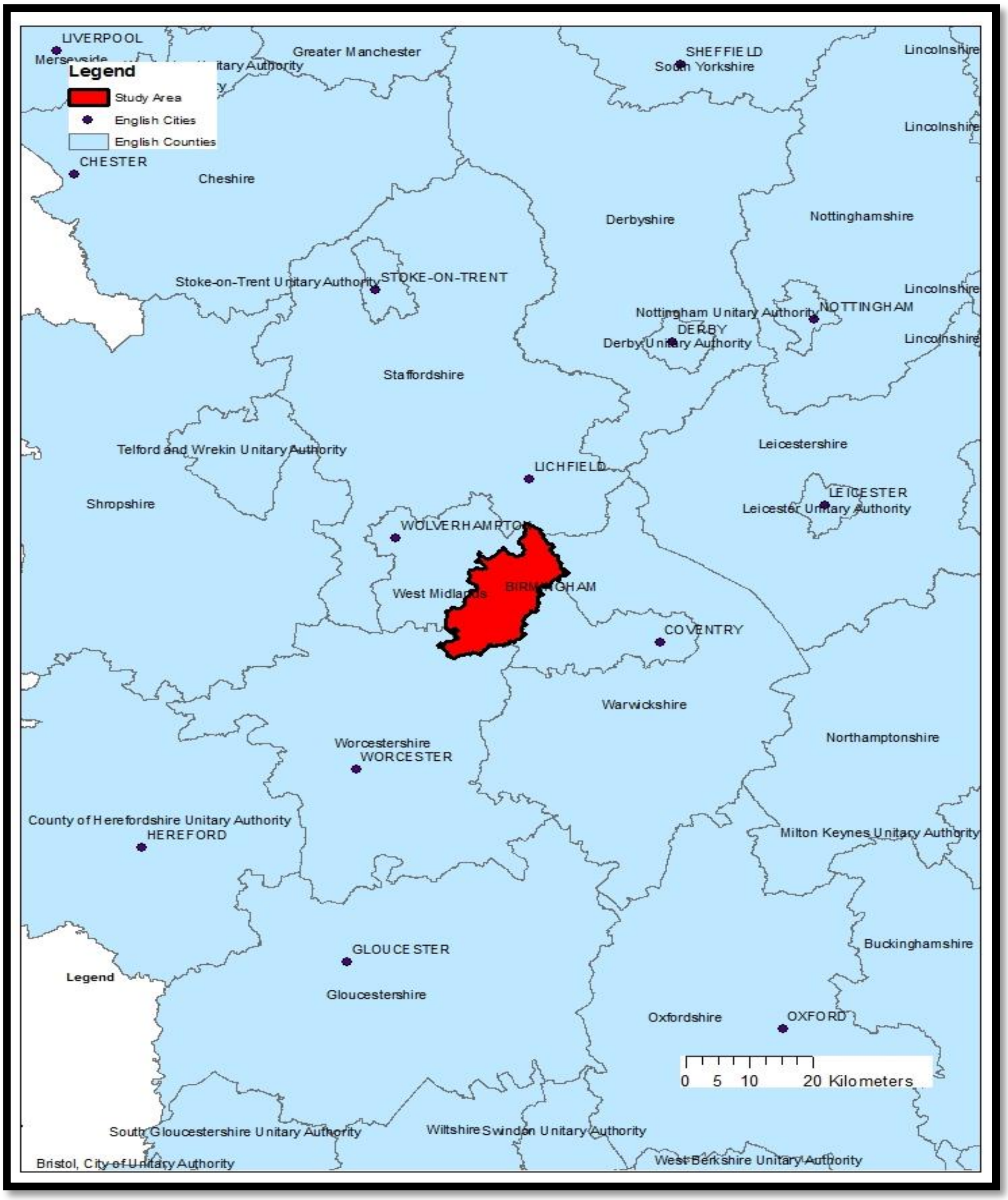


Figure 4.2 The Birmingham Study Area and surrounding area.

Birmingham has a temperate maritime climate with average maximum temperatures in summer (July) of 20°C and in winter 4.5°C (*Wikipedia*, 2010). Annual average precipitation from 2000-2009 was 877 mm and annual average potential evaporation was 616 mm (*University of Birmingham Climate and Atmospheric Research Group*, 2007).

The City of Birmingham is underlain by quaternary glacial deposits (0 – 40m) beneath which two rock formations dominate: Mercia Mudstones and Sherwood Sandstones (*Ellis & Rivett*, 2007). The Sherwood Sandstones form an unconfined aquifer bounded in the southeast by the Birmingham Fault which downthrows the sandstone layer to the east confining the aquifer beneath the Mercia Mudstone Formations. At the west and south-west boundaries the Sherwood Sandstones thin out over older Carboniferous formations (Warwickshire mudstones and sandstones).

The Triassic sandstone aquifer was used extensively for public supply until approximately 1900 at which time reservoirs were established in Wales and since then the majority of groundwater abstraction has been for industrial use (*Hughes et al.,* 1999). Groundwater levels have risen significantly since 1960 as a result of industrial decline and consequent reduced abstraction (*Lerner*, 2004).

The dominant river in Birmingham is the Tame whose tributaries include the Rea and the Cole as well as numerous brook and streams. It is the main river in the West Midlands and the most important tributary of the River Trent (*EA*, 2004). The Tame flows in an easterly direction and stretches about 40 km from its source in Oldbury to its confluence with the Trent. The catchment of the Tame up to its confluence with the river Blythe, just to the East of the study area, is ~54,500 ha (*EA*, 2010).

The canal network in Birmingham is one of the densest in the country (*Greswell*, 1992). Within the study area there are four main canals: the Tame Valley, the Worcester & Birmingham, the Grand Union and the Birmingham & Fazeley (*Turner*, 2009).

The privatised water company Severn Trent Water Limited (STW) is responsible for most of the water supply and all of the drainage and sewage treatment within the Birmingham area. STW supplies 1,900 Ml/day to 7.4 million people, and sewerage services, treating 2,500 Ml/day, to 8.5 million people in an area covering 21,000 square kilometres in the Midlands and mid-Wales. Severn Trent's assets include 46,000 km of water mains, 54,000 km of sewers, 181 groundwater treatment works and 1,017 sewage treatment works. Water is sourced from river abstraction (40%), groundwater (30%) and reservoirs (30%) (*Severn Trent Water*, 2007). The Birmingham Water Resource Zone (WRZ), one of 6 that STW supplies, includes most of the City of Birmingham. The Elan Valley reservoir in Wales is the principle source of water for this WRZ, supplying in excess of 300,000 Ml/day (*Khatri & Vairavamoorthy*, 2009). Minworth Wastewater Treatment Works is STW's largest plant, treating sewage from a population equivalent of 1.75 million (*BiWater*, n.d.). The catchment for Minworth includes the entire city of Birmingham.

Water issues and challenges that the city faces are population increase, climate change, allocation of responsibility for water management, affordability and source control of pollution. The Regional Spatial Strategy is looking at development options to meet the government's national housing targets. There are currently 400,000 dwellings (2001 Census) in Birmingham and the three options being considered for Birmingham are the addition of a further 70,000, 90,000, or 105,000 new houses by 2026 (*SWITCH*, 2007). This would increase the population to 1.3 million in the next 15 years and place more pressure on the existing, aging Victorian centralised supply systems. Demand management and the consideration of stormwater harvesting and wastewater re-use are strategies that are likely to be increasingly employed in response to the rising demand. Climate change, resulting in greater intensity rainfall events, is likely to increase the frequency of river flooding and sewer overflows. A further significant challenge that faces UK cities, including Birmingham, is the difficulty in adopting strong policies and Best Practice when there are multiple authorities involved in urban water management: primarily the Environment Agency, the City Corporation and the Water Company.

5 - Data Preparation and Calibration

5.1 Introduction

This section describes the data preparation and calibration stages for the Birmingham case study. A discussion of the validation of the model is covered in the next chapter.

Initially the study area was divided into subcatchments based on the sewer/stormwater network and land use/cover was defined, including main water networks and roads. Then, parameters associated with the different land uses were chosen (e.g. water demand profiles and unit block areas). Climate data were obtained and processed to obtain daily potential evaporation. Soil parameters were calibrated based on recharge estimates, then irrigation demand was calibrated to seasonal fluctuations in the observed data, provided by Severn Trent Water (STW). Having calibrated water supply, calibration of wastewater flows, using data from STW was performed. After data preparation for the groundwater modelling and abstractions was complete, contaminant input loads and concentrations were calibrated. Finally there is a description of data collection for energy and cost.

5.2 Characterizing Land Use

The following methodology is proposed in the preparation of a new case study:

1) Define subcatchments

2) Outline the main water networks

3) Outline miniclusters

The use of GIS software is recommended for the task and in this case study ArcMap was used. A detailed description of each stage in the preparation of subcatchments and land use data is provided below.

5.2.1 Subcatchments

Flow of stormwater and wastewater though a city is generally governed by the layout of the piped, centralised network. In *CWB*, modelling is at a less detailed level than the pipe scale, focussing on broader flow patterns between subcatchments, which are defined as areas of cityscape containing networks of foul or combined sewers that drain to a point at their downstream boundaries. The urban area consists of a set of subcatchments, each containing one or more miniclusters.

a) Figure 5.1 shows the sewer/stormwater network for the Minworth catchment area (*STW*, 2009). The task of identifying general flow patterns was made easier by filtering out sewers of less than 500 mm diameter (Fig. 5.2).

b) Severn Trent divides the drainage and sewer system in Birmingham into catchments called “Drainage Areas”, of which there are twenty seven in the study area. The boundaries of the Drainage Areas used by Severn Trent are suitable for use as subcatchment boundaries in the Birmingham model since they drain to single downstream points at their boundaries.

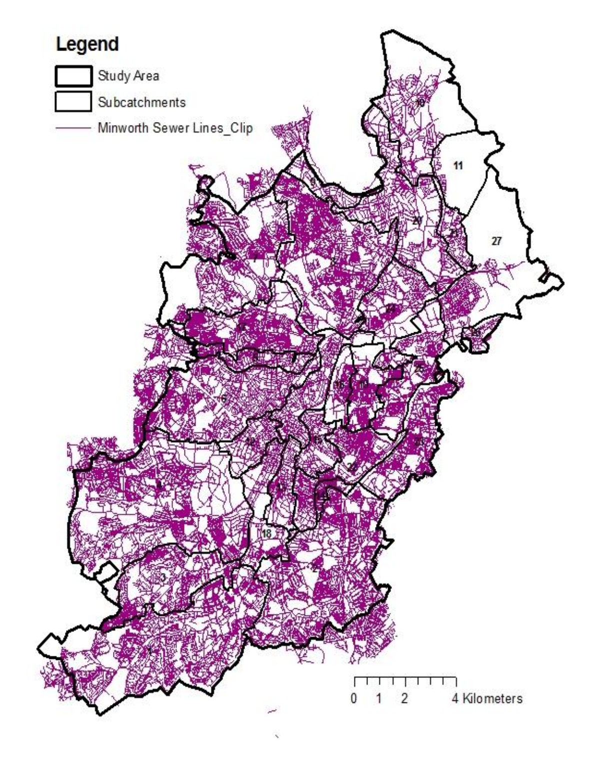


Figure 5.1 Sewer/stormwater network for the Minworth catchment area.

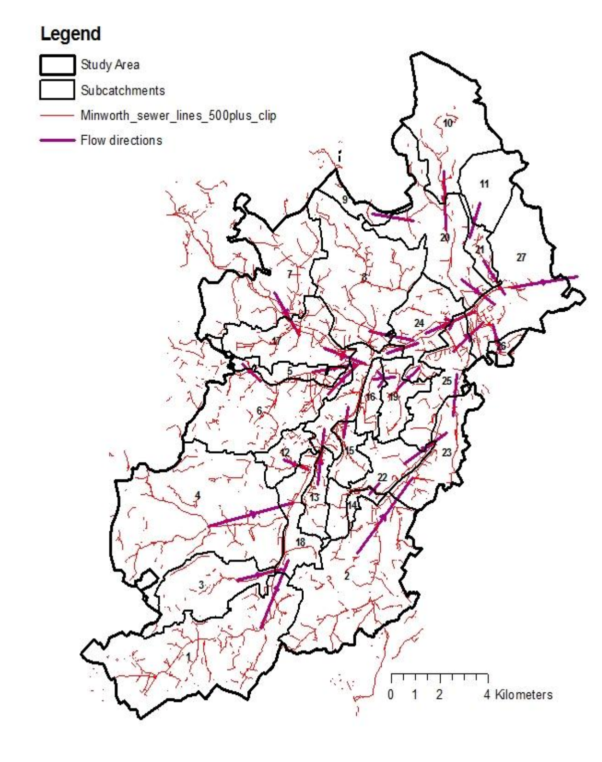


Figure 5.2 Sewer/stormwater (500mm diameter or greater) network for the Minworth catchment area.

5.2.2 Main water networks

5.2.2.1 Rivers

Background

There are three significant rivers in the study area: the Tame, the Rea and the Cole (Fig. 5.3). The Rea and the Cole are tributaries to the Tame, which is the main river in the West Midlands. The Tame is the most important tributary of the River Trent (*EA*, 2004). It flows in an easterly direction and stretches about 40 km from its source in Oldbury to its confluence with the Trent. The catchment of the Tame up to its confluence with the River Blythe, just to the east of the study area, is ~ 54,500 ha (*EA*, 2010). Within the bounds of the study area, the River Tame's main tributaries are the River Rea, River Cole, Hockley Brook, Witton Brook and Perry Brook (*Greswell*, 1992).

The River Rea flows to the east of the Birmingham Fault on the Mercia Mudstone formation and converges with the Tame to the north of the city centre. The Rea's main tributary is the Bourn Brook. The Cole, running further to the east of the Fault, also forms a significant drainage feature (*Greswell,* 1992) and joins with the Tame to the east of the study area.

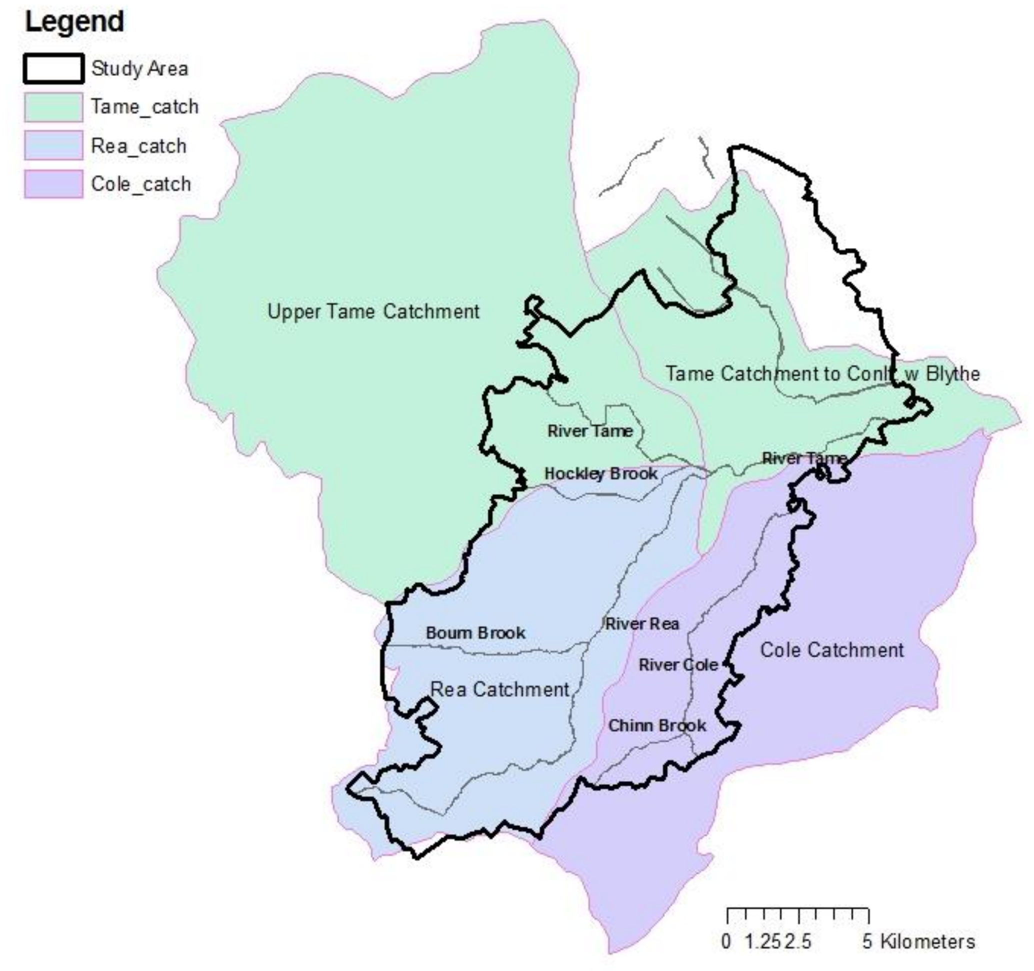


Figure 5.3 Catchments of the Tame, Rea and Cole (*EA*, 2010).

Data preparation

A new layer was created in ArcMap for each river. The layer was type "polygon" and not "line" to allow subsequent calculation of the overlap area with the groundwater grid. Mapping for each river began at the most upstream point (study area boundary usually). The following guidelines were followed in the definition of segment boundaries:

- Each segment should end when the waterbody is intersected by an inflow feature such as a sewer outfall or tributary on the Ordnance Survey 1:10000 scale map.

- Each segment should end at a subcatchment boundary since the segment area is included in the subcatchment area.

The degree of lining of urban rivers can significantly affect interaction with groundwater and so it was important to take this into account in the model. The Tame has been significantly channelized with Gabion bank support along much of its length (*Ellis et al.,* 2007). However, the bed permeability is largely unaltered since the reinforcement is only along the banks, except for a short section beneath the M6 motorway (*Greswell*, 1992). Therefore, in the model it was assumed that the Tame bed is permeable over its entire length. By contrast, significant sections of the Rea are impermeably lined; from Cannon Hill Park, south of the city centre, to its confluence with the Tame (*Knipe et al.,* 1993). However, since the Rea flows over the confined aquifer for most of its length, there would be no recharge modelled by *CWB*, even if it were not lined. Similarly, the River Cole flows over the confined aquifer to the east of the Birmingham fault, so no recharge is modelled.

Culverted sections were treated as open air, and consequently subject to direct precipitation and evaporation, since this simplified the task of defining their area and because the culverted area is small in comparison to the open-air sections.

A number of key parameters and their sources are listed in Table 5.1. The hydraulic conductivity of the river bed (1 m/day) was chosen as that typical for unconsolidated sandy deposits (*Knipe et al.*, 1993). Widths of Bourn Brook and Hockley Brook were set to 1m by *Turner* (2009) in his groundwater modelling (Groundwater Vistas) of the Birmingham catchment to investigate borehole catchment areas.

For each of the rivers and brooks, Manning's Equation (Eq. 5.1) was used to calculate the magnitude of fixed input flow from outside the study area to the first segment. The smallest slope over all segments in each watercourse was used in this calculation to ensure that flooding did not occur as a result of typical flows (Table 5.2).

$$V = 1/n * R_h^{2/3} * S^{0.5} \qquad (5.1)$$

Where v = average flow velocity (m/s)
S = longitudinal slope (-)
R_h = hydraulic radius = X_a / P_w (m)
X_a = X-sectional area (m^2)
n = Manning coefficient of roughness ($sm^{-1/3}$)
P_w = Wetted perimeter (m)

Table 5.1 Key river parameters and their data sources.

Parameter	Value	Units	Source
Number of segments	Variable	NA	ArcMap
Area of each segment	Variable	m^2	ArcMap
Width	10	m	*Thomas & Tellam* (2006)
Length	Area/Width	m	Calculation
Maximum depth	2	m	Estimate
Initial depth	0.6	m	Estimate
Bed thickness	0.5	m	*Greswell* (1992)
Hydraulic conductivity of bed	1	m/day	*Knipe et al.*, 1993; *Binley et al.*, 2002; *Powell* (2000)
Segment elevation	Variable	m asl	Edina Digimap
Slope	Based on elevation differences	NA	Calculation

River bed hydraulic conductance (K) of 1 m/day was used, since saturated hydraulic conductivities for sandy-loam soil can be in excess of this (*Binley et al.*, 2002). *Powell* (2000) used hydraulic conductivity values for the sandstone of 1 - 3.5m/day. The K values for a number of substrates, used by *Turner* (2009) in his Groundwater Vistas model of Birmingham, are shown in Table 5.2.

Table 5.2 Hydraulic conductivity for the predominant subsurface types in the Birmingham City area (*Turner*, 2009).

Sub-surface type	Kx (m/d)	Ky (m/d)	Kz (m/d)
Sandy Drift	50	50	50
Clayey Drift	0.001	0.001	0.001
Eastern Sandstone	2.2	2.2	1
Western Sandstone	1.7	1.7	1

Ellis (2003) estimated that, at mean flows, non-groundwater inputs to the Tame are approximately 40 to 50 ML/day. The Tame was estimated to have an average input flow volume of 0.99 m^3/s or 85.5 Ml/day (Table 5.3). A value of 50 Ml/day in accordance with the estimate by *Ellis* (2003) can be achieved by reducing the river flow depth to 0.44m from 0.6m.

It was found, during calibration, that high groundwater levels were encountered in the northern part of the study area. In order to reduce this, tributaries to Footherley Brook and Plant Brook (Fig. 5.4) were included, thereby creating additional points of discharge from the aquifer which locally lowered the groundwater levels.

Table 5.3 Calculation of input flow to each river or brook from outside the study area.

Name	Depth (m)	n	Width (m)	Xa (m2)	Pw (m)	Min (S) (-)	Manning vel min (m/s)	Flow min (m3/s)
Hockley	0.4	0.04	1	0.4	1.8	0.0018	0.39	0.16
Tame1	0.6	0.04	10	6	11.2	0.0001	0.16	0.99
Bourn	0.4	0.04	1	0.4	1.8	0.0009	0.28	0.11
Rea	0.6	0.04	5	3	6.2	0.0004	0.31	0.92
Tame2	0.6	0.04	10	6	11.2	0.0001	0.16	0.99
Chinn	0.4	0.04	1	0.4	1.8	0.0018	0.39	0.16
Cole	0.5	0.04	5	2.5	6	0.0003	0.24	0.60
Footh	0.4	0.04	2	0.8	2.8	0.0005	0.24	0.19
Footh2	0.4	0.04	2	0.8	2.8	0.005	0.77	0.61
Plant trib	0.4	0.04	1	0.4	1.8	0.0007	0.24	0.10
Plant	0.4	0.04	2	0.8	2.8	0.0001	0.11	0.09

5.2.2.2 Canals

Background

The canal network in Birmingham is one of the densest in the country (*Greswell*, 1992). They were originally lined with clay, though this does not preclude subsequent leakage through wear. Within the study area there are four main canals: the Tame Valley, the Worcester & Birmingham, the Grand Union and the Birmingham & Fazeley (*Turner*, 2009) (Fig. 5.4).

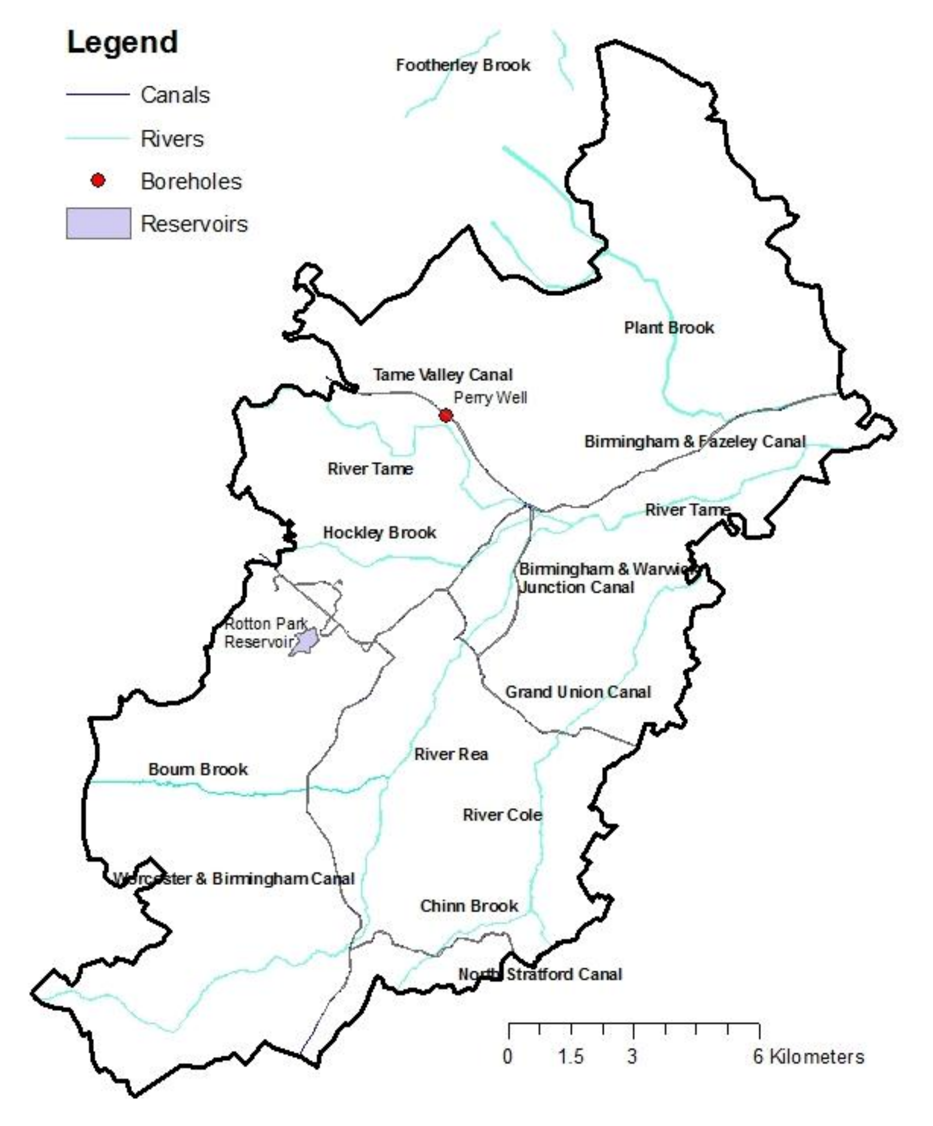


Figure 5.4 Map showing the main rivers and canals in the Birmingham study area.

Canals in the Birmingham area are supplied by reservoirs and boreholes. The main boreholes are located at Perry Well (Fig. 5.4), which is in the northern part of the study area, and Bradley, which is to the west of the study area (*Roberts*, 2009). Within the study area Rotton Park Reservoir supplies the canals and is fed from the Titford level to the west of Birmingham. Other reservoirs to the south of the study area are Earlswood lakes and Bittel. The most important water supply to the Birmingham canals is from Chasewater Reservoir (*Roberts*, 2009) to the north of the study area.

Data preparation

In *CWB*, canals are modelled as water features with constant depth and constant leakage rate. Constant depth is a reasonable assumption since it is vital that canals are kept at operating depth of 1.5 m ± 150 mm (*Roberts*, 2009). If constant head in the canal is assumed then the assumption of constant leakage rate is also valid. The standard figure used by British Water Ways for unaccounted water is 1.75 Ml/km/week (21 mm/day) (*Roberts*, 2009). However, the term "unaccounted-for water" includes evaporation and discharge from waste weirs. Waste weirs are not monitored by British Waterways but it is estimated that at least half of unaccounted-for water is lost to this overflow (*Roberts*, 2009). Therefore, British Waterways estimate leakage to be less than 0.88 Ml/km/week (10 mm/day). In their steady-state groundwater model of Birmingham, *Knipe et al.* (1993) found the average leakage from canals to be 0.4 Ml/km/week (5 mm/day). *Thomas and Tellam* (2006) used a leakage value of 12 mm/day in their recharge model for Birmingham. For the purposes of this case study the more recent estimate by *Thomas and Tellam* (2006) has been adopted. Leakage is assumed to be zero from canal segments that do not overlie unconfined aquifer.

If canals are assumed to maintain constant depth (change in storage is zero) then inflow to the canal must equal outflow:

(5.2) Demand = leakage + (evaporation – precipitation) + outflow from study area

Outflow from the study area, as a result of the use of locks, is not modelled. It is assumed that this unquantified volume of water is matched by supply from the reservoirs outside the study area (e.g. Chasewater) in order to maintain constant operating depth in the canals.

Demand is satisfied by water from reservoirs and boreholes. The only supply features within the study area bounds are Perry Well and Rotton Park Reservoir (Fig. 5.4). In *CWB*, it is assumed that inflow to reservoirs equals outflow. Therefore, the only processes of importance that are modelled by *CWB* are the reservoirs' interaction with groundwater and direct precipitation/evaporation. Perry Well is modelled as a "large borehole". The level of borehole abstraction is up to ¾ of the license (1900 Ml/yr) and most of this is taken during the summer period (*Barsley*, 2009). Any demand not satisfied by Perry Well is satisfied by reservoirs and boreholes outside the study area and it is assumed that demand never exceeds supply potential. Water used to supply canals, from outside the study area, is assumed to have the same concentration of contaminants as the initial concentrations specified in the canals, at the start of simulation. Various key parameters for the description of canals in *CWB* are shown in Table 5.4.

Table 5.4 Key canal parameters and their data sources.

Parameter	Data	Units	Source
Width	12	m	(*Roberts*, 2009)
Operating depth	1.5	m	(*Roberts*, 2009)
Leakage	12	mm/day	*Thomas & Tellam* (2006)
Length	Area/Width	m	*CWB* calculation
Maximum depth	2	m	Estimate

5.2.2.3 Lakes

A "lakes" shapefile was created in ArcGIS and their vertices were extracted for use by *CWB*. Weir height above the lake bed was initially set to 3 m and initial water depth to 2 m. Bed sediments were assumed to be clayey and so hydraulic conductivities were set to 0.001 m/day. Lakes are treated as waterbodies and are subject to the same rules of interconnection between waterbodies as rivers. Sixty-eight ponds, lakes and reservoirs were modelled in this case study and it was assumed that the flows in streams feeding and discharging from the ponds and lakes were insignificant in comparison to the flows in the rivers and so they were not included in the model. Instead, outflows from lakes were assumed to leave the study area.

5.2.3 Outline Miniclusters - Land Use Description

Land use data were obtained using a combination of Google Earth images and Ordnance Survey (OS) maps (*Edina Digimap Collections*, 2009). OS maps were used as follows: the area of interest was located at “Local” scale (1 km^2), then at “Detailed Zoom” scale (2500 m^2), to get the most information possible about landuse. Navigating the study area at “Detailed Zoom” scale was a time consuming process because it is focused on a small area and there was a delay time of several seconds when loading adjacent maps. In cases where it was unclear what function a named building on the OS map served, Google searches were employed. Google Earth images were useful for rapidly identifying wooded and green areas, as well as large areas of similar residential housing.

Small pockets of land-use, different from that prevailing in the area, were absorbed into the prevailing land-use. For example, a public house in a large industrial area was not isolated as an individual minicluster but was classified as part of the industrial area. Road area on the boundary between two miniclusters was divided equally between them.

In addition to being included within the bounds of a minicluster with a different land-use, roads can also be modelled as separate miniclusters. Roads are considered as MCs with no unit blocks (water demand) and no POS (Public Open Space). As such, they follow the same mapping constraints as miniclusters - polygons cannot cross subcatchment boundaries. They were categorised into major and secondary, where “Major roads” are considered to be A-roads or larger and "secondary" all other roads. This distinction was made because major roads have heavier traffic and so are more polluting. Railways were modelled in the same way as roads but no distinction was made between different railway types.

The list of UB types used to describe land use in the Birmingham study area is shown in Table 5.5. The choice was informed by water demand categories used in the *Plumbing Engineering Services Design Guide* (*The Institute of Plumbing*, 2002), *Thomas's* (2001) runoff-recharge model for Birmingham and detailed observation, by the author, of land use in Birmingham from Ordnance Survey Maps and Google Earth images.

Table 5.5 Unitblock Types and ID numbers.

UB_Type	ID	UB_Type	ID
Residential – Terraced Small Garden	1	Place of Assembly – Restaurant	26
Residential – Terraced Large Garden	2	Prison	27
Residential – Semi-detached small garden	3	Office	28
Residential – Semi-detached large garden	4	Industry	29
Residential – Detached small garden	5	Industry_disused	30
Residential – Detached large garden	6	Hotel	31
Residential – Flats	7	Hotel with grounds	32
Residential – Home	8	Hospital	33
Residential – High Rise	9	Fire Station	34
POS – POS	10	City Centre	35
POS – wooded	11	Garage	36
POS – Golf course	12	Public Baths	37
POS – Allotment	13	Medical Centre	38
Recreation ground	14	Health Centre	39
Retail – supermarket	15	Church	40
Retail with canteen	16	TA Centre	41
Retail without canteen	17	Depot	42
School – Nursery/Primary	18	Mansion	43
School – Primary through Secondary	19	Kennels	44
School – Secondary	20	Road Main	45
School – Further Education	21	Road Secondary	46
School – Higher Education	22	Railway	47
Place of Assembly – Bar, Pub or Club	23	Parking	48
Place of Assembly – Community Centre	24	Waterworks	49
Place of Assembly – Library, Museum	25		

A number of other land uses were included within the categories shown in Table 5.5 (Table 5.6).

Table 5.6 Allocation of various landuses to generalized UB types.

UB Type	Land use	UB Type	Land use
Community Centre	Indoor Tennis Centre	POS – allotment	Farm
	Sports Hall		Garden Centre
	Bowling Alley	Recreation Ground	Outdoor Bowls
	Bingo Hall		Sports Ground
	Scout Hut	Prison	Young Offender's Institute
Depot	Electricity Sub-Station	Residential – Flats	Convent
	Builder's Yard		Nursing Home
	Warehouse	Residential – Detached Large Garden	Farmhouse
Hotel	Conference Centre	Restaurant	Botanical Gardens
Industry	Works	Retail – without canteen	Banks
Industry Disused	Building Site	Retail – with canteen	Mixed Retail Areas
Medical Centre	Surgery		Shopping Centre
Office	Telephone Exchange	School – Nursery/Primary	Special Needs School
	Police Station		Day Centre
Parking	Stations		Day Care Centre
	Garages (private storages)		

In addition to the 3 months spent by research assistant Liam Mackay categorising landuse in Birmingham, a further six weeks was spent by the author (Fig. 5.5).

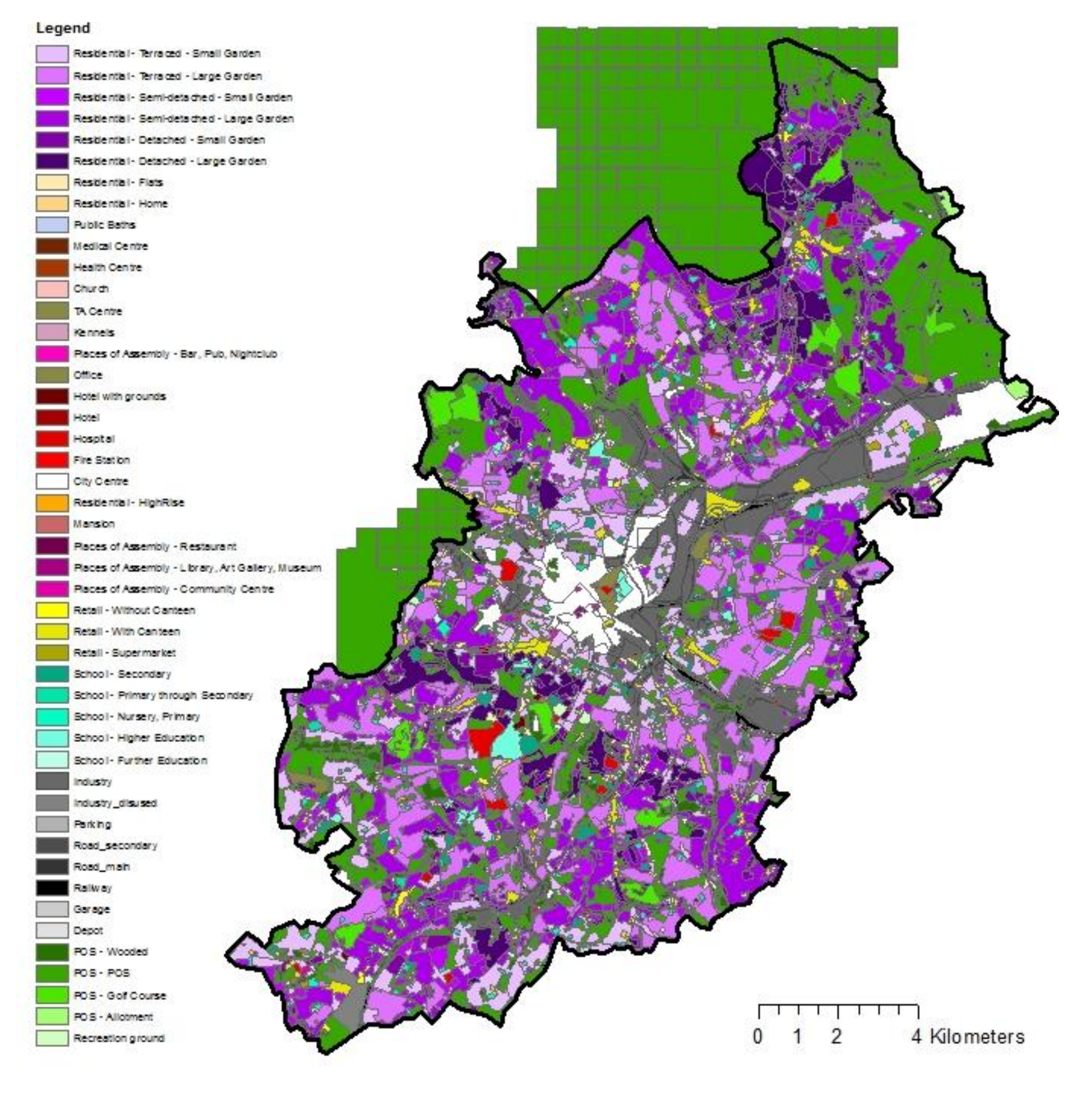


Figure 5.5 Landuse in the study area categorized into miniclusters.

5.3 Minicluster Attributes

Initially the polygons of each UB type were stored in separate layers in ArcMap, since this allows land use to be displayed in different colours (Fig. 5.5). When all land use had been defined it was necessary to assign attributes (e.g. subcatchment number, MC number) to sub-sets of MCs with different UB types. This was achieved by copying all miniclusters within a particular subcatchment to a new layer and subsequently labelling them (Appendix 1.1). The numbering order of miniclusters within a subcatchment is not important since *CWB* is only concerned with flows at the subcatchment scale. However, the minicluster numbers must be sequential between subcatchments – i.e. if subcatchment 1 contains ten miniclusters then these are labelled 1 to 10, in any order, and the first MC in subcatchment 2 must be labelled 11.

5.4 Unitblock Attributes

5.4.1 Unitblock Areas

For each UB type, the MC with the smallest area was identified, in the attribute table, by sorting. It was then located on the map and the underlying Google Earth image was used to estimate % roof, % paved and % PS for the MC area (Fig. 5.6). The number of significantly-sized buildings in the MC was counted and the UB area was calculated as the MC area (in the attribute table) divided by the number of buildings. Then the UB area was divided into roof, paved and PS areas according to the estimated percentages. The smallest MC was used for this exercise so that there would not be a problem with defining a unitblock area larger than a minicluster comprised of that unitblock type. If this does occur *CWB* automatically assigns the land use in the MC as POS.

Figure 5.6 Example of estimation of percent roof, paved and garden area for a unit block type (church). Aerial photograph from Google Earth.

After definition of the UB types was complete, areas of POS and Road were estimated from the GIS view by eye (Table 5.7). This involved looking at various miniclusters for each unitblock type and estimating an average.

Table 5.7 Percent POS and Road for different UB types.

UB category	% POS	% Road
Residential	15	15
POS	100	0
Roads/Railways	0	100
Prison	0	5
Church	10	10
Waterworks	5	10
City Centre	5	15
Industry	5	10
Industry disused	10	10
Other	0	10

Wooded areas, which are clearly visible on the Google Earth images, were given their own unitblock type and "proportion wooded" was set to 1. For other areas of POS this parameter was set to zero.

5.4.2 Water Demand Profiles

Water demand profiles for the different UB types were researched from literature and are shown in Table 5.8.

Table 5.8 Land uses and associated water demand data (c is capita).

Land use	UB Type	Demand data	Source
Residential		150l/c/day	(*Waterwise*, 2009)
		Occupancy = 2.5	Estimate
		Split: 35% toilet, 15% kitchen, 30% bathroom, 20% laundry	*Rueedi & Cronin* (2005) after EA (2001)
		Split: 33% toilet, 15% kitchen, 36% bathroom, 16% laundry	(*Waterwise*, 2009)
	Mansion	Twice residential (300l/c/day); Occupancy = 10	Estimate
Commercial		0.056 l/ m^2/day	*(Rural Utilities Service and U.S. Forest Service,* 1999)
Industry		Low = 0.11 l/ m^2/day Mid = 0.22 l/ m^2/day High = 1.1 l/ m^2/day	*(Rural Utilities Service and U.S. Forest Service,* 1999)
		20 l/m^2/day (roof area only)	(*Pitt & Clark*, 2009)
	Waterworks	Industry demand	Estimate
	Depot	Half industry demand	Estimate
Office		2.4 l/m^2/day	(*Hunt & Lombardi*, 2006)
		65% toilet, 25% washing, 10% kitchen	(*Leggett et al.*, 2001)
		60% toilet, 30 washing, 10% kitchen	*Rueedi & Cronin* (2005)
		49 l/c/day	*Rueedi & Cronin* (2005) after Adams (1999)
		50 l/c/day	*Vesilind* (2003) after *Metcalf & Eddy* (1991)
	City centre	Same as office. Characterized by 4-6 storey offices, public buildings and hotels	Estimate. *Knipe et al.* (1993)
	Fire station	4 x office demand	Estimate
	Medical centre	2 x office demand	Estimate
	Health Centre	2 x office demand	Estimate
Hotel		150 l/bedroom	(*The Institute of Plumbing*, 2002)
		182 l/c/day	*Rueedi & Cronin* (2005) after Adams (1999)
		190 l/c/day	*Vesilind* (2003) after *Metcalf & Eddy* (1991)
		6 floors with 10 bedroom/floor	Estimate
		30% toilet, 30% bathroom, 30% kitchen, 10% laundry	*Rueedi & Cronin* (2005)
Hospital		300 l/bed	(*The Institute of Plumbing*, 2002)
		625 l/c/day	*Rueedi & Cronin* (2005) after Adams (1999)
		6 floors with 100 beds/floor	Estimate

		30% toilet, 55% bathroom, 10% kitchen, 5% laundry	*Rueedi & Cronin* (2005)
TA Centre		150 l/c/day (same as residential). Occupancy = 50	Estimate
School	Nursery/Primary	15 l/pupil/day	(*The Institute of Plumbing*, 2002)
	Secondary/ Further/Higher	20 l/pupil/day	(*The Institute of Plumbing*, 2002)
		Same split as office (65% toilet, 25% washing, 10% kitchen)	Estimate
		Same as office: 65% toilet, 30% bathroom, 10% kitchen	*Rueedi & Cronin* (2005)
		57 l/pupil/day	*Rueedi & Cronin* (2005) after Adams (1999)
		100 pupils / 500m^2	Estimate
Prison		Same as residential flats. Since assumed occ. of 30 for 500m^2 flats apply this to prison	Estimate
Public Baths		20 l/c/day	(*The Institute of Plumbing*, 2002)
		Assume 500 visits/day for Baths of 500m2. Assume 50% toilet, 50% shower	Estimate
Church		Occupancy of 100	Estimate
		Assume same demand as library – 6 l/c/day	(*The Institute of Plumbing*, 2002)
Restaurant		4 l/c/day	(*The Institute of Plumbing*, 2002)
		35-40 l/customer/day	*Vesilind* (2003) after *Metcalf & Eddy* (1991)
Bar		80 l/seat/day; 50 l/c/day	*Vesilind* (2003)
Recreation ground		35 l/c/day. Assume occupancy of 500	(*The Institute of Plumbing*, 2002)
Retail with canteen		45 l/c/day	(*The Institute of Plumbing*, 2002)
		40 l/employee/day	*Vesilind* (2003) after *Metcalf & Eddy* (1991)
		40 l/m^2/day (roof area only)	(*Pitt & Clark*, 2009)
		Assume same split as office (65% toilet, 25% washing, 10% kitchen)	Estimate
Retail without canteen		40 l/c/day. Assume 70% toilet, 30% wash	(*The Institute of Plumbing*, 2002)
Kennels		Base on office usage: 3 l/c/day; 90% washing, 10% kitchen	Estimate

5.4.3 Residential Appliances

Data for residential appliances were taken from the technology library for the *UWOT* model (*Makropoulos et al.*, 2008). A sample is shown in Table 5.9.

Table 5.9 Various parameters associated with indoor water using appliances.

Washing machine	Water Usage (l/use)	Water loss (proportion)	Energy use (kWh/use)	Capital cost (£)	Operational cost (£/use)	Water loss (l/use)
Hotpoint WF320G	30	0.25	1	220	0.2	0.4
Zanussi ZWF1430	49	0.3	1.02	300	0.3	0.4
Siemens WF320G	58	0.25	1.1	360	0.2	0.4
Miele Premier 520	39	0.2	0.57	460	0.2	0.4
Bosch WFX1485	54	0.35	1.14	500	0.4	0.4
Bosch WVTI2840	47	0.2	0.85	750	0.2	0.4

5.4.4 Other Unitblock Parameters

Other unitblock parameters were set the same for all unitblocks and are outlined in Table 5.10.

Table 5.10 Various UB parameters and their data sources.

UB Parameter	Value	Units	Source
Effective roof area	0.95	none	*Rueedi & Cronin* (2005) after Fewkes (1999)
Effective paved area	0.4	none	*Rueedi & Cronin* (2005) after Berthier et al. (2004)
Roof initial loss	0.5	mm	*Rueedi & Cronin* (2005) after Berthier et al., 2004)
	0.5	mm	*Villareal* (2004)
	2.5 – 7.5 (flat)	mm	*Thomas* (2001)
	1.3-2.5	mm	*Thomas* (2001)
Paved initial loss	3.5	m	*Rueedi & Cronin* (2005)
	1.3 – 3	mm	*Thomas* (2001)
Prop use2 hot	0.3	none	Estimate
Prop use3 hot	0.4	none	Estimate
Prop use4 hot	0.3	none	Estimate

5.5 Minicluster Data

Figure 5.7 shows that the study area is largely served by separate sewer systems, except in the city centre. Even when separate pipes are used to transport wastewater and stormwater, a proportion of stormwater runoff will enter the foul sewers through cross-connections or leakage through manholes (*Mitchell et al.*, 2001; *Metcalf & Eddy*, 1991). In their application of *UVQ* to Doncaster, *Rueedi & Cronin* (2005) found that approximately 1% of surface runoff was entering the foul sewers, so this value, assumed applicable to UK cities, was adopted for use in the Birmingham model (Table 5.11) in areas with separate surface water drains.

A combined sewer is a pipe that transports both wastewater and stormwater. In *CWB* combined sewers are modelled as foul sewers with 100% of surface runoff draining to them, in addition to the usual sewage volume. Based on Figure 5.8, the inflow proportions proposed for the subcatchments are shown in Table 5.11. Table 5.12 shows a list of other MC-specific data and their sources.

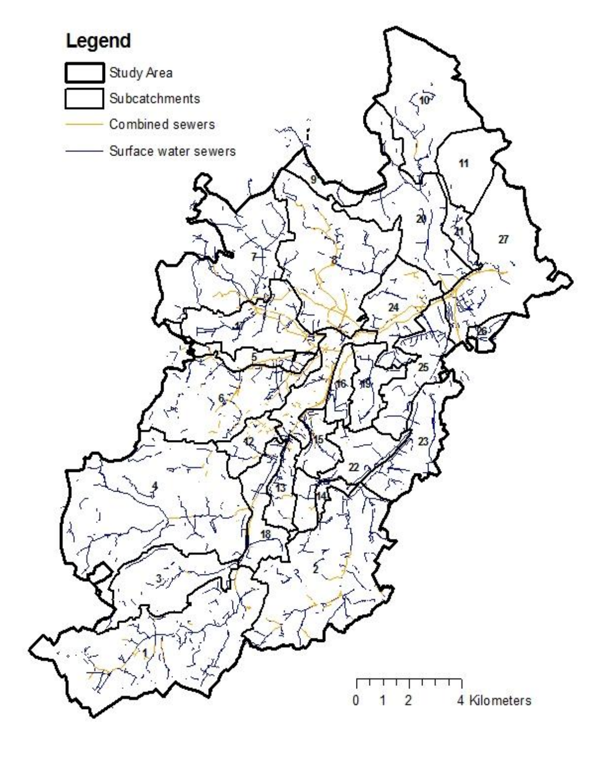


Figure 5.7. Map showing distribution of combined sewers and separate stormwater systems (diameter ≥ 500mm) in the Birmingham study area.

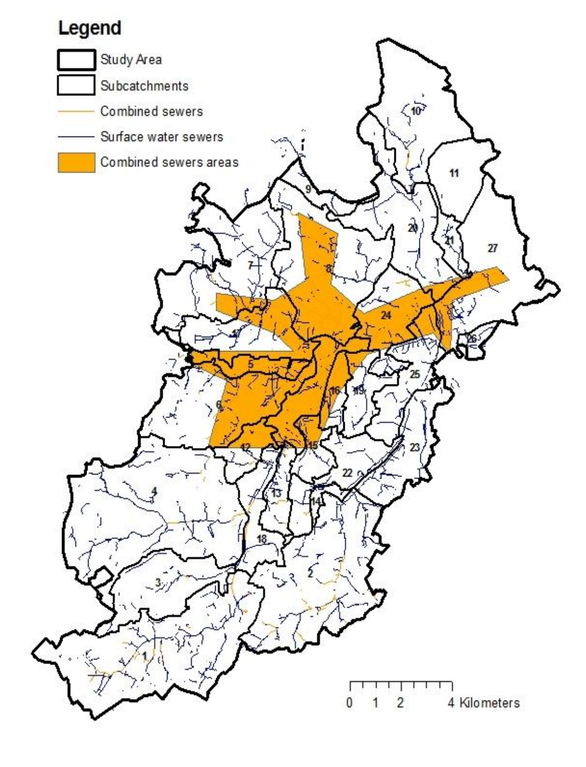


Figure 5.8. Map showing distribution of combined sewers and separate stormwater systems (diameter ≥ 500mm) in the Birmingham study area with combined sewer area outlined in orange.

Based on weekly averaged daily gross supply and mains leakage data from Severn Trent, the average annual leakage for April 2008-March 2009 was 25%, which is in exact agreement with the average figure reported in Severn Trent's *Strategic Direction Statement* (*STW*, 2007). Since there were considerable gaps in the data provided, a simple *Visual Basic for Applications* (VBA) routine was written to process this result from the data.

Table 5.11 Proportion of stormwater runoff flowing into the subcatchment foul sewer systems (Inflow proportion).

Subcatchment number	Proportion combined	Inflow proportion	Subcatchment number	Proportion combined	Inflow proportion
1	0	0.01	**15**	0.3	0.3
2	0	0.01	**16**	0.6	0.6
3	0	0.01	**17**	0.45	0.45
4	0	0.01	**18**	0.5	0.5
5	1	1	**19**	0.1	0.1
6	0.55	0.55	**20**	0.1	0.1
7	0.1	0.1	**21**	0	0.01
8	0.35	0.35	**22**	0	0.01
9	0	0.01	**23**	0	0.01
10	0	0.01	**24**	0.5	0.5
11	0	0.01	**25**	0.1	0.1
12	0.5	0.5	**26**	0	0.01
13	0	0.01	**27**	0.2	0.2
14	0	0.01			

Table 5.12 Other minicluster data and their sources.

Parameter	Data	Units	Source
Proportion of surface runoff as inflow to foul	See Table 5.11	None	*Rueedi & Cronin* (2005)
Sewer exfiltration proportion	0.05	None	Estimation – mains leakage often 20% but sewers are not pressurized – confirmed by *Yang et al.* (1999)
Woods intercept	5	mm	*Thomas and Tellam* (2006)
Woods potential evapotranspiration rate	3	mm/day	*Thomas and Tellam* (2006)
Mains leakage proportion	0.25	None	Severn Trent data
Road initial loss	3.5	mm	*Rueedi & Cronin* (2005) after Berthier et al. (2004)
	2	mm	*Thomas and Tellam* (2006)
	0.7	mm	*Villareal* (2004)
Effective road area	0.75	None	*Rueedi & Cronin* (2005) after Berthier et al. (2004)

5.6 Groups

In *CWB* a group of miniclusters can be created to allow rapid assignment of the same parameter set (e.g. soil type) to more than one minicluster. Initially, twenty eight groups were assigned, representing the twenty seven subcatchments and the study area. The subcatchment groups are a convenient way of dividing the study area into smaller sections.

5.7 Natural Systems

5.7.1 Climate Data

5.7.1.1 Background

Daily rainfall and potential evaporation time series are required as input to *CWB*. Climate data were taken from the Winterbourne Climate Station, which is situated on the University of Birmingham campus (*University of Birmingham Climate and Atmospheric Research Group*, 2007). It is assumed that the data are applicable to the whole of the study area since there is little areal variation of precipitation in the City of Birmingham (*Greswell*, 1992). Hourly climate data are available for download in ASCII format (http://kermit.bham.ac.uk/~kidd/zmetdata/TWF_hourly.html), monthly from 1999 to the present day, including:

- Hourly rainfall (mm)
- Hourly averaged dry and wet bulb temperatures (°C)
- Hourly averaged solar radiation (KJ)
- Max., min., and T_{100} (100cm off ground) hourly temperatures (°C)

5.7.1.2 Data Processing

Textfiles containing hourly data for each month were loaded into an Excel worksheet and missing data were linearly interpolated - typically several weeks of data in a year. Headers were added to the data and the relevant data columns were extracted to another spreadsheet. In the new spreadsheet the hourly values were summed or averaged, as appropriate, to daily values using VBA. A description of the calculation (including VBA code) of the daily potential evaporation using the Penman-Monteith equation is shown in Appendix 1.2.

Figures 5.9 and 5.10 show the processed times series of daily rainfall and potential evaporation (PET) for 2008. PET is the amount of evaporation that would occur providing there is a sufficient water source. Table 5.13 shows the variation of annual rainfall and PET for several years. It can be seen that 2008 was a wet year and that potential evaporation does not have a wide range of variation.

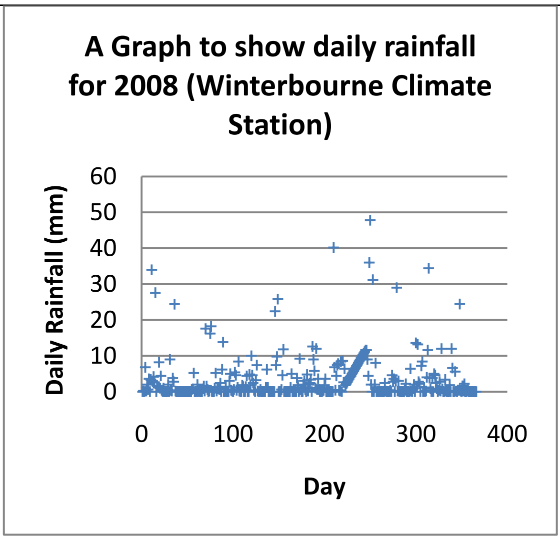
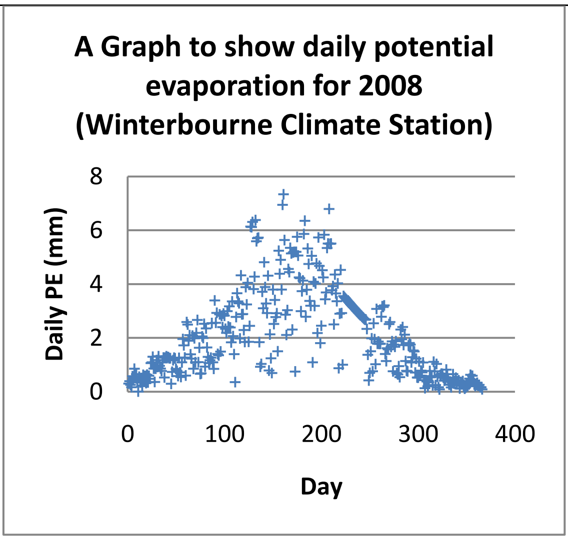
	

Figure 5.9 Daily rainfall for 2008, based on hourly recorded data from the Winterbourne Climate Station. Missing data for days 224-244 were linearly interpolated.	**Figure 5.10 PET for 2008, based on hourly recorded data from the Winterbourne Climate Station. Data for the days 224-244 were calculated based on linearly interpolated values.**

Table 5.13. Annual precipitation and evaporation for several years based on climate data from the Winterbourne Climate Station.

Year	Annual precipitation (mm)	Annual PET (mm)
2000	1055	589
2001	758	585
2002	903	623
2003	719	639
2004	840	669
2005	740	627
2006	711	646
2007	973	630
2008	1161	568
2009	913	587

5.8 Soil Data

5.8.1 Background

Two rock formations are encountered immediately below the surface drift layer under the majority of the City of Birmingham: Mercia Mudstone and Sherwood Sandstone (Fig. 5.11). The Sherwood Sandstone forms an unconfined aquifer bounded in the southeast by the Birmingham Fault which downthrows the sandstone layer to the east confining the aquifer beneath the Mercia Mudstone Formation (Fig. 5.12). At the west and south-west boundaries the Sherwood Sandstone thins out over older Carboniferous formations (Warwickshire Mudstone and Sandstone).

Most of the unconfined aquifer is covered with drift, of glacial origin, that varies in thickness from 0-40 m. A simplified map of the drift deposits over the Birmingham aquifer is shown in Figure 5.13. The deepest drift deposits are present in the areas of the proto Tame and proto Rea depressions where the early river channels ran. Shallow drift of about 1-5 m thickness exists in the Tame Valley (*Ellis & Rivett*, 2007). The northern part of the Sherwood Sandstone is covered by Quaternary deposits of glacial, lacustrine, Aeolian and fluvial origin. These deposits are up to 20-30 m thick and can seriously impede local recharge (*Thomas and Tellam*, 2006). The map clearly indicates the dominance of sandy drift deposits over the aquifer with relatively small areas of clay-dominated, quaternary deposits (*Turner*, 2009).

As Birmingham has developed, depressions have been in-filled with made ground, which is generally permeable material, and can be treated in a similar manner to the sandy drift (*Turner*, 2009). Consequently, these regions are considered to have the same hydrological properties as natural, sandy drift areas.

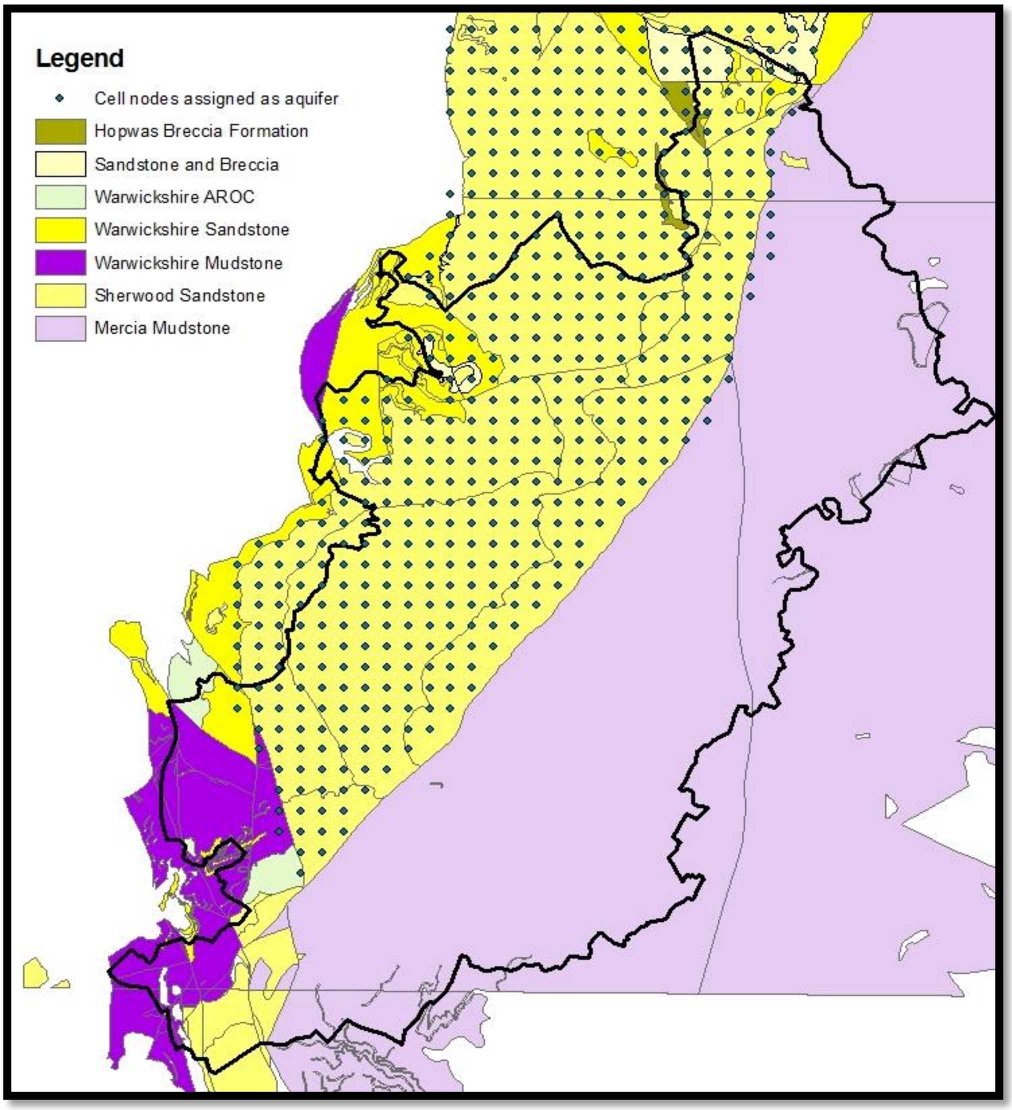


Figure 5.11 The major rock formations underlying Birmingham City (*Edina Digimap*, 2009).

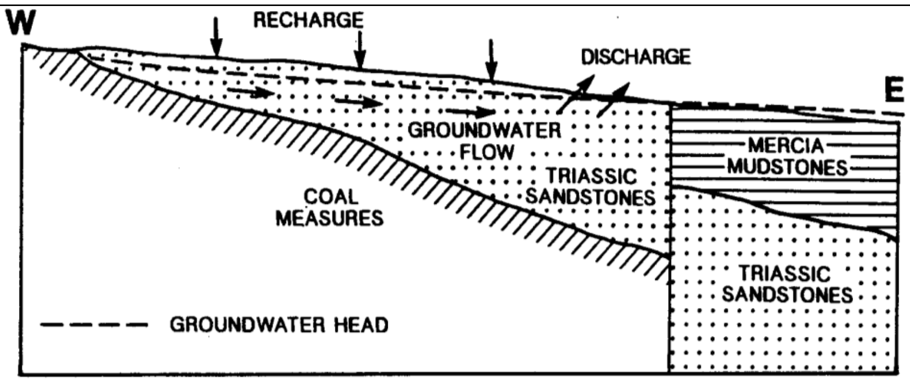


Figure 5.12 Cross-section of Birmingham aquifer showing the main formations and general direction of groundwater flow (*Greswell*, 1992 after *Jackson*, 1981).

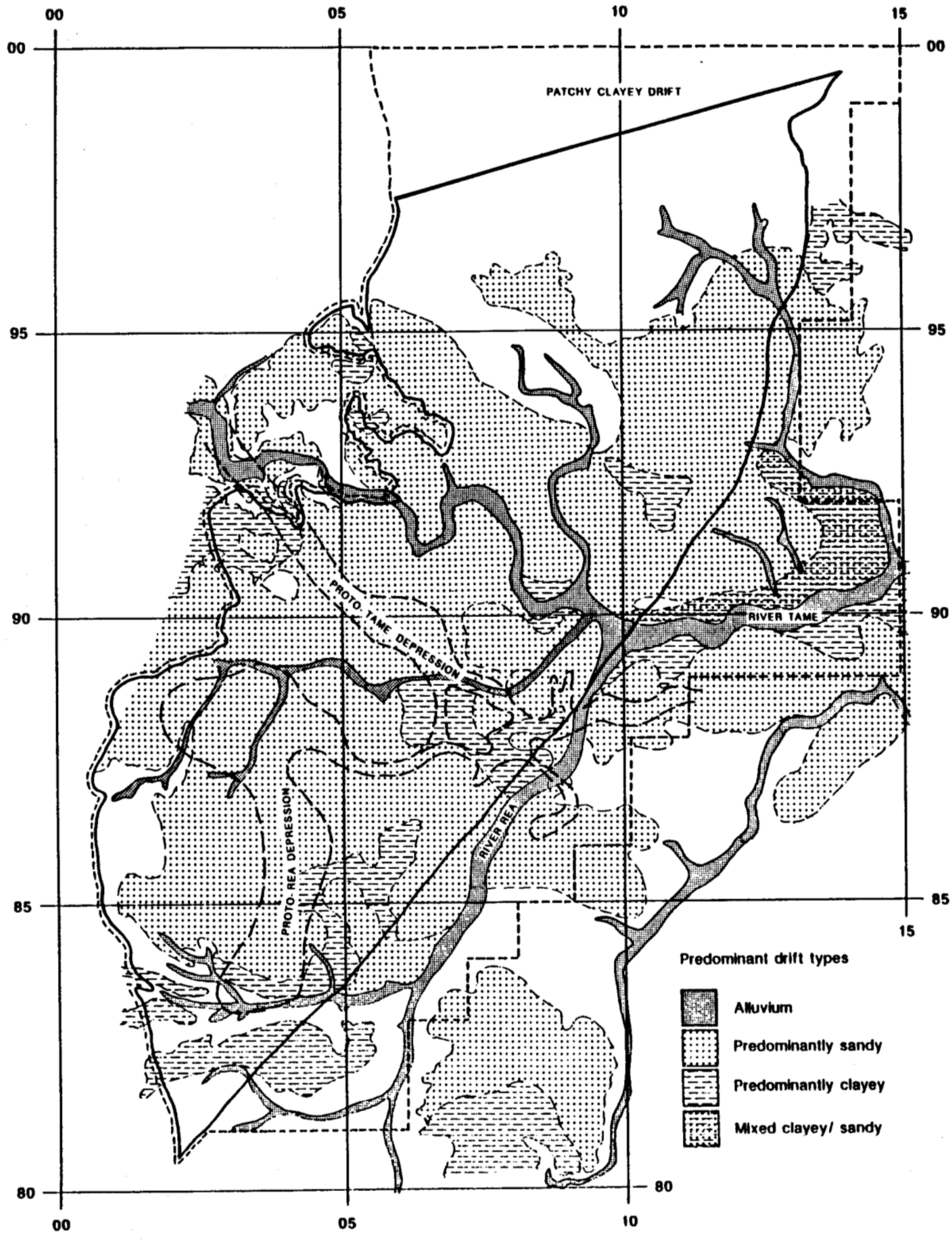


Figure 5.13 Simplified drift extent overlying the Birmigham aquifer (*Knipe et al*., 1993).

5.8.2 Calibration

The two-layer soil model was used in the Birmingham study since it was considered more appropriate for the UK climate than the partial area model. Based on data in Table 5.14, Table 5.15 shows estimates of soil parameters that were used as initial guesses for the Birmingham model. Calibration of the soil parameters was based on comparison of simulated recharge rates with rates from previous UK studies.

The target recharge rates for model calibration lie in the range 100-300 mm/yr. *Thomas and Tellam* (2006) found recharge rates for Birmingham of 90-100 mm/yr (averaged over a 20 year period with maximum annual rainfall of 820 mm) over the Sherwood Sandstone. *Ragab et al.* (1997) calculated recharge rates of 200-300 mm/yr for Sherwood sandstone. Using the results of a baseflow analysis, *Ellis* (2003) estimated that recharge within the Tame catchment is ~28% of the rainfall, i.e. 315 mm for rainfall of 1124 mm/year (April 2008- March 2009). This figure includes mains supply leakage, and contributions from areas other than the Sherwood Sandstone Formation. For the soil calibration, rainfall data from April 2008 to March 2009 were used since this is the period for which mains leakage data from Severn Trent were provided; mains leakage is an important source of recharge to the aquifer.

Table 5.14 Initial estimates of sandy soil parameters.

Parameter	Value	Units	Source
Soil column	0.5	m	*Rueedi & Cronin* (2005) after Berthier et al. (2004)
Total porosity	31	%	*Rueedi & Cronin* (2005) after Binley et al. (2002)
	24	%	*Turner* (2009) *after Allen et al.* (1997)
Residual porosity	5	%	*Rueedi & Cronin* (2005) after Binley et al. (2002)
Maximum daily drainage rate	20	mm/day	*Rueedi & Cronin* (2005) after Binley et al. (2002)

If PS1 and PS2 capacity are set to 78 mm, soil residual capacity to 5%, soil field capacity to 10% (Table 5.15) and max daily drainage set to 10 mm, annual recharge of 388 mm is simulated (based on annual rainfall of 1124 mm) (Table 5.16A). This is more than the literature values, even for a wet year. Table 5.16B shows that reducing the "maximum drainage rate" to 2 mm/day reduces recharge to 325 mm/yr which is closer to estimates proposed by *Ragab et al.* (1997) of 200-300 mm/yr and *Ellis* (2003) ~ 315 mm.

Table 5.15 Initial estimates of model soil parameters (Two layer model).

Parameter	Value	Units
PS1 soil thickness	0.25	m
PS2 soil thickness	0.25	m
PS1 capacity	0.31*0.25 = 0.0775	m
PS2 capacity	0.31*0.25 = 0.0775	m
PS1 field capacity	0.1*0.25 = 0.025	m
PS1 field capacity	0.025	m
PS1 residual water content	0.05*0.25 = 0.0125	m
PS2 residual water content	0.0125	m

The simulations used steady state initial conditions for soil moisture deficit. Initial levels for the pervious stores were set to 10 mm. However, after one year of simulation there was considerable change in storage, indicating that the system was not in steady state. After running the 2-layer soil store model for several years, with the same climate data, the average soil depths and infiltration store depth achieved steady state and had the following values: 75 mm (PS1), 48 mm (PS2).

Table 5.16. Annual recharge depths (averaged over the aquifer) for various sources with field capacity set to 0.1 and residual soil moisture to 0.05. (A) Max drainage rate = 10 mm/day (B) Max drainage rate = 2 mm/day.

	Aquifer averaged recharge (mm/yr)	
Recharge Parameter	**A**	**B**
Change in storage	-5.7	-6.3
Gardens	65	45
POS	231	189
Mains leakage	66	66
Sewer exfiltration	26	25
Waterbodies	-152	-125
Sewer infiltration	-247	- 212

The final soil parameters chosen, based on the recharge calibration, are shown in Table 5.17. This table also shows the initial estimates for the soils overlying the Mercia Mudstones, for which some parameters were assumed the same as for sandy soils (soil depth, porosity and drain factor). The residual soil moisture was based on data in Table 5.18 and the field capacity was assumed to be twice the residual soil moisture. The maximum drainage from clay soils was set to half that of the sandy soils.

Subcatchments that are underlain by Mercia Mudstones (Fig. 5.14) for the majority of their area were assigned clay soil type and remaining subcatchments were assigned sandy soil type. Total recharge is unaffected by the introduction of the clay soil class because they do not overlie the unconfined aquifer and therefore their contribution to recharge is relatively insignificant.

Table 5.17 Calibrated values for model soil parameters. Soil depth is 0.5m.

Parameter	Sandy soils	Clay soils	Units
Capacity of PS1	0.1	0.1	m
Capacity of PS2	0.1	0.1	m
Porosity	40	40	%
Field capacity	10	14	%
Residual moisture	5	7	%
Field capacity factor (prop. of soil capacity)	0.25	0.35	None
Residual factor (prop. of soil capacity)	0.13	0.18	None
PS1 Drain max	2	1	mm
PS2 Drain max	2	1	mm
PS1 Drain factor	1	1	None
PS2 Drain factor	1	1	None

Table 5.18. Soil characteristic parameters for Richard's equation (*Carsel & Parrish,* 1988).

Soil	θ_s (%)	θ_r (%)	Ks (mm/day)	n
Sand	43	4.5	7128	2.69
Loamy Sand	41	5.7	3502	2.28
Sandy Loam	41	6.5	1061	1.89
Loam	43	7.8	249	1.56
Silt	46	3.4	60	1.37
Sily Loam	45	6.7	108	1.41
Sandy Clay Loam	39	10	314	1.48
Clay Loam	41	9.5	62	1.31
Silty Clay Loam	43	8.9	16.8	0.01
Sandy Clay	38	10	28.8	0.027
Silty Clay	36	7	4.8	0.005
Clay	38	6.8	4.8	0.008

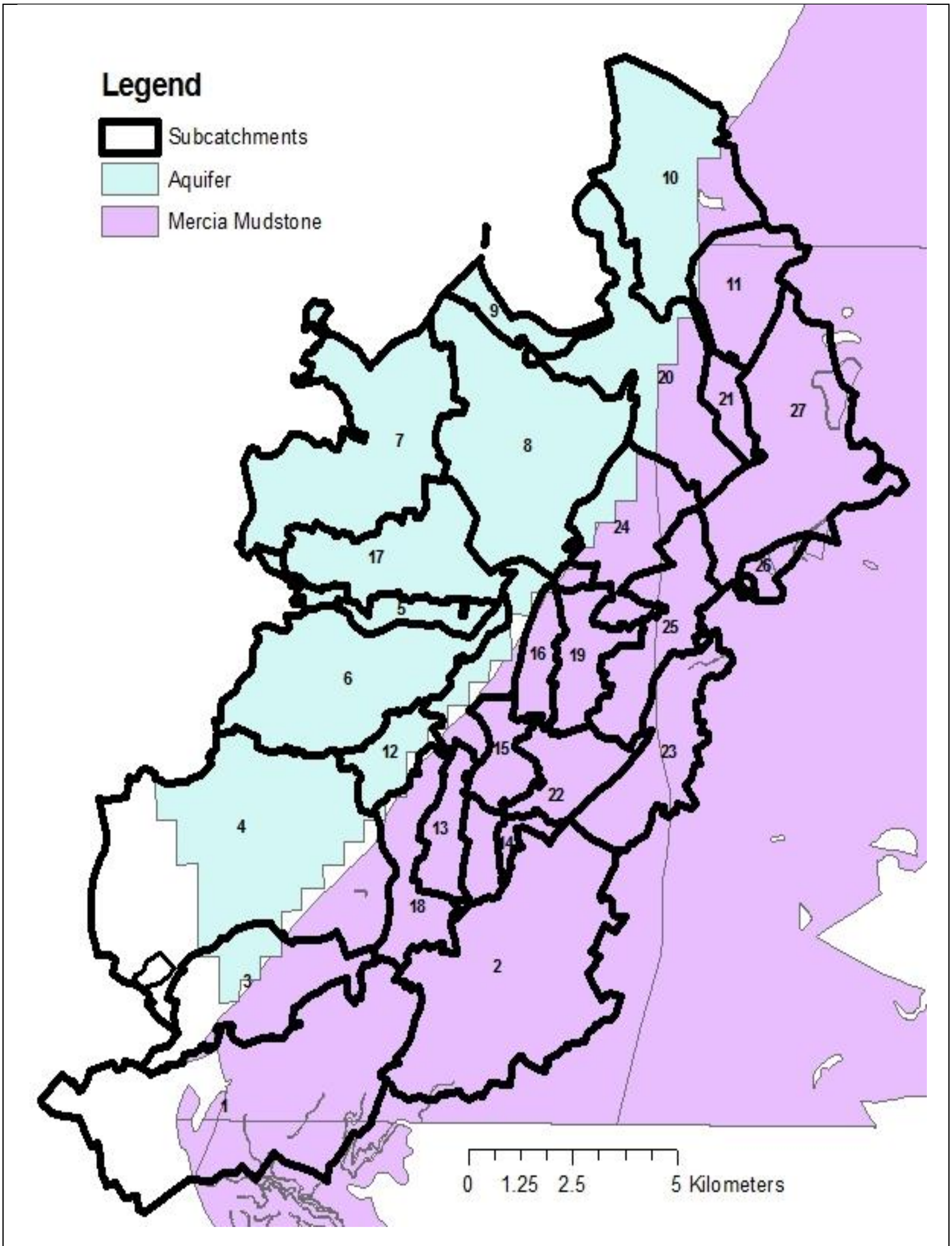


Figure 5.14 Figure showing the main sub-surface formations in the study area. The following subcatchments were assigned clay soils: **1,2,3,11,13,14,15,16,18,19,21,22, 23, 24, 25, 26, 27.**

5.9 Infiltration from Groundwater to Sewers

Infiltration of groundwater into sewers is modelled using Darcy's Law (with the hydraulic conductivity and pipe surface area assumed constant across the study area as a simplification). Therefore, in order to calculate the infiltration it was necessary to know the area of pipe per m^2 of ground surface area. Using data from Severn Trent Water (STW), the total length of sewers and stormwater pipes within the study area was calculated to be 5,116 km. The study area is 280 Mm^2, so the average pipe length per m^2 ground is 0.0183 m/m^2. Average pipe diameter was calculated to be 299 mm, based on sewer records from STW (1.88 m circumference, assuming circular pipes). Consequently, average pipe surface area per m^2 surface area is 0.034.

5.10 Water Demand

Calibration of water demand was only applied to the area for which supply and leakage data were provided by Severn Trent (Fig. 5.15). The area in the northern part of the Study Area is the responsibility of the South Staffordshire Water Works Company and data were not obtained since communication was only with Severn Trent Water. It was assumed results of the calibration for water demand were also applicable to the South Staffs area. Water supply monitoring by Severn Trent is split into areas called District Metered Areas (DMAs) and data supplied by Severn Trent for this case study were categorised by DMA. There are approximately 270 DMAs, managed by Severn Trent, within the bounds of the case study.

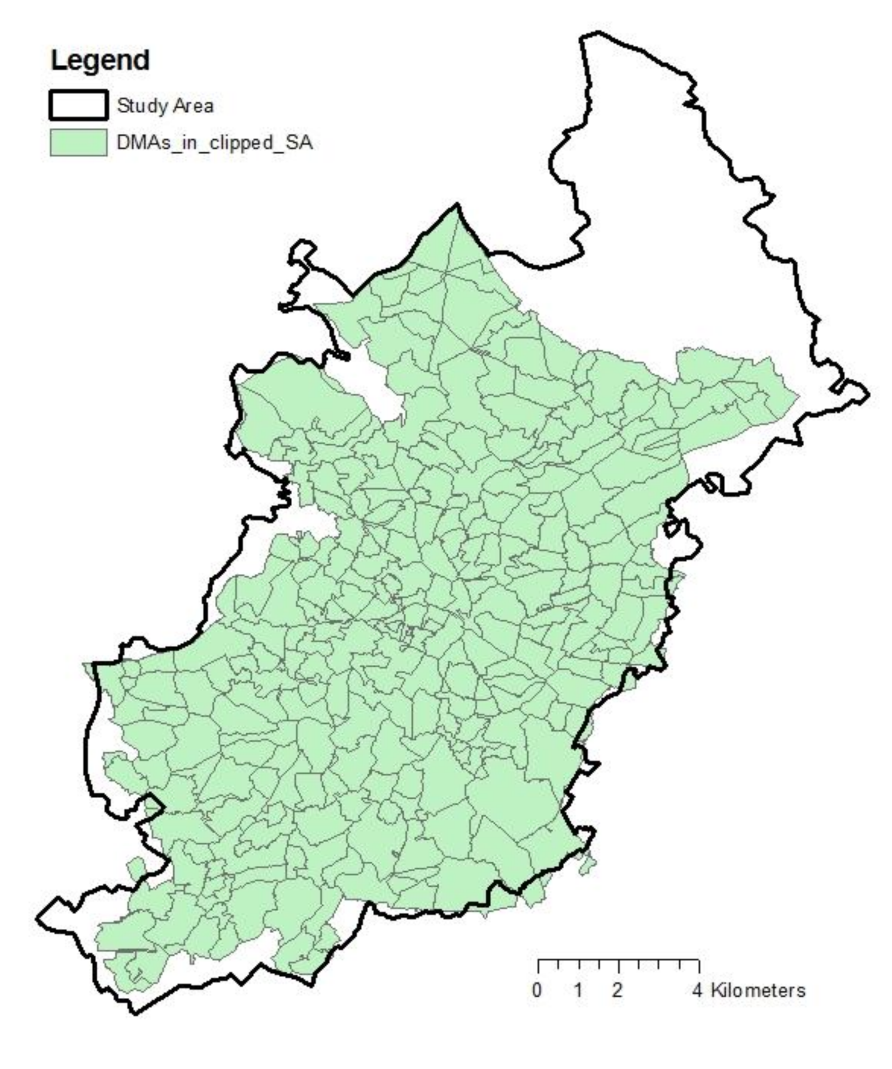


Figure 5.15 Figure showing the area for which supply and leakage data was provided from Severn Trent. DMAs that straddle the Birmingham Study Area boundary were omitted for ease of calculation.

5.10.1 Methodology

This sub-section summarises the methodology used to calibrate model demand to supply records from STW. Calibration began by investigating the water demand for a single DMA. In order to isolate specific unitblock types, DMAs were selected that have almost entirely uniform land use.

In the attribute table of the target DMA its number, name and area are listed, as supplied by Severn Trent Water. From the daily data supplied by STW (April 2008 - March 2009) annual average gross supply, leakage and number of properties were calculated (data gaps were filled with annual averaged values). Percentage leakage was calculated as:

(5.3) % leakage = 100 * leakage / gross supply

In ArcMap, the layer holding the DMA boundaries was set to 55% transparent so that it was possible to simultaneously see the google earth image beneath. A "typical" unitblock was identified by eye and polygons were drawn, in an edit session, over the roof, garden and paved areas.

In the attribute table, of the calibration DMA, a new field "Areas" was created and the areas were calculated using "Calculate geometry". These values were exported to Excel, where the UB area and the total UB area, for the number of properties in the DMA, were calculated. Then areas of POS and road were identified and polygons were drawn over them. Their areas were calculated in the attribute table.

The sum of the UB, POS and road areas should have equalled the DMA area but it was usually less as a result of underestimation. The main problem was the wide variation in garden size, even within a small area. For example, houses on street corners or end-terraced often have larger gardens. Consequently, based on the assumption that much of the error between the sum of the UB areas (plus POS and roads) and the actual DMA size was in the estimation of the garden area, a typical garden area was re-estimated and the remainder was assigned to POS.

Once a suitable geometry for the DMA was identified a simulation was run using *CWB*. The results from the simulation were then compared with the STW data using percentage differences.

5.10.2 Calibration

5.10.2.1 Residential

Table 5.19 shows typical indoor residential usages for the UK based on the UK average consumption of 150 l/c/day (*Waterwise*, 2009b). These values were used in the calibration of residential miniclusters to DMA supply data.

Table 5.19. Residential indoor use expressed as a percentage and volume (*Waterwise*, 2009a*; *Waterwise*, 2009b).**

Water use	Indoor use (%) *	Indoor use (l/c/day)**
Toilet	33	49.5
Kitchen	15	22.5
Laundry	16	24
Bathroom	36	54

Tables 5.20 to 5.22 show the results of the calibration. Roof, pave and garden areas were estimated by drawing polygons over Google Earth images. Where there was a discrepancy between the estimated areas and the actual area of the DMA, the difference was assigned to POS. Figure 5.16 shows the location of the DMAs for which calibration was performed. Although two are situated outside the study area this was unimportant in the determination of water demand since it is the selection of a representative land use type for the city that is significant.

Occupancy of 2.5 was assumed (*Khatri & Vairavamoorthy*, 2009) and did not change during calibration with the exception of DMA 4027, which is a student area, so occupancy of 4 was found to better fit observed data. The time period used was April 2008 to March 2009, which is the most current period of supply data available from Severn Trent.

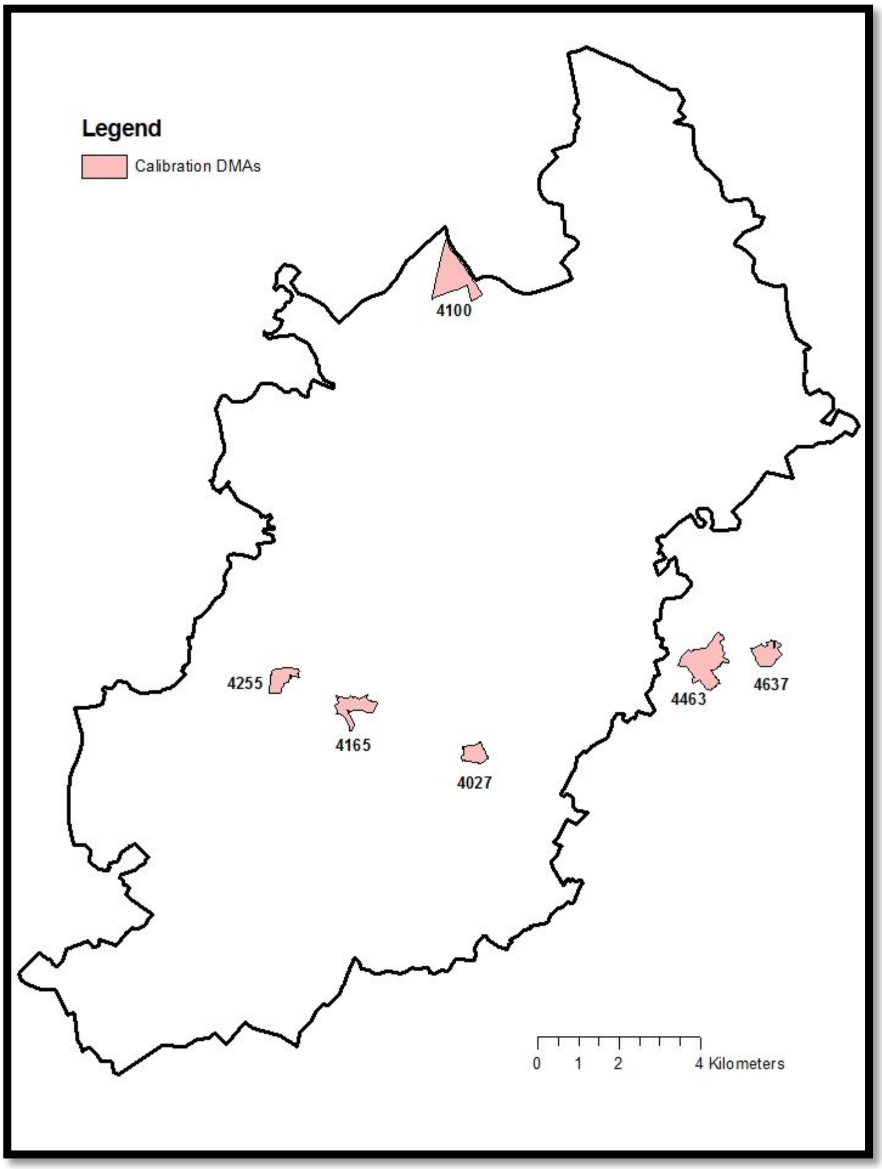


Figure 5.16 Location of DMAs used for residential supply calibration.

Table 5.20 Properties of calibration DMAs.

DMA	Area (m^2)	Location	Land use	Properties	Type	Roof (m^2)	Paved (m^2)	Garden (m^2)	UB area (m^2)
4463	831,084	Sheldon Heath	Terraced, small garden	1653	1	67	37	110	214
4027	231,650	Selly Park	Terraced, large garden	839	1	76	20	87	183
4637	314,191	Tile Cross Rd	Semi-detached, small garden	523	1	53	25	57	135
				523	2	67	59	224	350
4100	786,072	Kingstanding Rd	Semi-detached, large garden	1215	1	50	30	245	325
				1215	2	84	59	370	513
4255	278,345	Norfolk Rd	Detached, small garden	323	1	93	25	210	328
				323	2	252	270	280	802
4165	380,248	Church Rd	Detached, large garden	277	1	197	86	335	618

Table 5.21 Summary of properties of calibration DMAs.

DMA	Type	Total UB area (m^2)	POS (m^2)	Road (m^2)
4463	1	353,742	106,286	121,880
4027	1	153,537	0	37,213
4637	1	70,605	94,735	33,124
	2	183,050	94,735	33,124
4100	1	394,875	93,000	115,000
	2	623,295	93,000	115,000
4255	1	105,944	23,230	24,824
	2	259,046	23,230	24,824
4165	1	171,186	64,311	37,740

Table 5.22 Results of residential DMA calibration, where net flow is gross minus leakage.

	Severn Trent data				CWB Results		
DMA	**Area (m^2)**	**Gross (m^3)**	**Net (m^3)**	**Type**	**Gross (m^3)**	**Net (m^3)**	**% Diff (m^3)**
4463	831,084	726	607	2	749	629	**3**
4027	231,650	843	586	1	455	318	**46**
	Occ. 4			1	725	508	**13**
	Occ. 5			1	905	634	**8.2**
	Occ. 4.5			1	815	570	**3**
4637	314,191	273	172	2	326	205	**16**
4100	786,072	634	502	2	600	475	**5.4**
4255	278,345	160	131	2	136	111	**15**
4165	380,248	128	92	1	151	108	**15**

5.10.2.2 Unitblock Areas for Non-residential Land Use

Table 5.23 summarises estimates of UB areas for various land uses. The smallest minicluster for each UB type was used to prevent the creation of a UB that is larger than other MCs of the same type (e.g. Mansion). If this does occur *CWB* automatically assigns the area to POS, in the absence of better information.

Table 5.23 Estimation of UB areas for different land uses.

UB Type	**Area (m^2)**	**% Roof**	**% Paved**	**% Garden**
Kennels	12100	35	25	40
Mansion	6453	5	10	85
Depot	276	50	50	0
TA Centre	4569	40	10	50
Church	435	40	10	50
Health centre	1270	40	30	30
Medical Centre	530	35	60	5
Public Baths	686	85	15	0
Garage	239	50	50	0
City centre	500 (guess)	60	40	0
Fire Station	883	45	50	5

5.10.2.3 Calibration of Supply to the Study Area

After the model was calibrated to supply data for several DMAs it was then calibrated to the total supply to the Severn Trent metered area within the bounds of the study area (henceforth called the "clipped study area"). Since Severn Trent supplies a significantly larger area than the study area it was necessary to exclude DMAs outside the study area for the purposes of calibration. In order to examine mixed land-use demand at the DMA scale it was necessary to know which miniclusters were within the bounds of each DMA. Since there are over 6000 miniclusters this task was performed using a Visual Basic program (Appendix 1.3).

Total annual observed gross water supplied for all DMAs in the clipped study area was 62 Mm^3 for the 2008-2009 period (STW supply data). So the model initially over-estimated the demand by a factor of at least 3.5 (219 Mm^3/yr).

First it was important to establish whether the over-estimation was a result of the quality of the input data or the model itself. An Excel spreadsheet was used to approximately estimate what the demand values should be based on:

(5.1) Demand = Number of UBs * Occupancy factor * UB indoor demand

CWB yielded the same answers (with no irrigation) as the Excel sheet, so it was concluded that the problem was with the input data. It is possible to reduce Indoor demand in the following ways:

1) Reduce the UB demand / occupancy factor
2) Reduce Occupancy factor
3) Increase size of the UB and consequently reduce the number of UBs in a MC

Since the demand for residential DMAs had already been calibrated to within 20% of observed data, this discrepancy was a result of the inaccuracies in estimating non-residential demand. It was found that the unitblock types with the poorest initial estimates were "Hospital" and "Retail".

Figure 5.17 shows the results of the supply calibration at this stage. The annual volume is a good match but the seasonal variation is not. Simulation of seasonal variation in supply required calibration of the irrigation factors, which is covered in the next section.

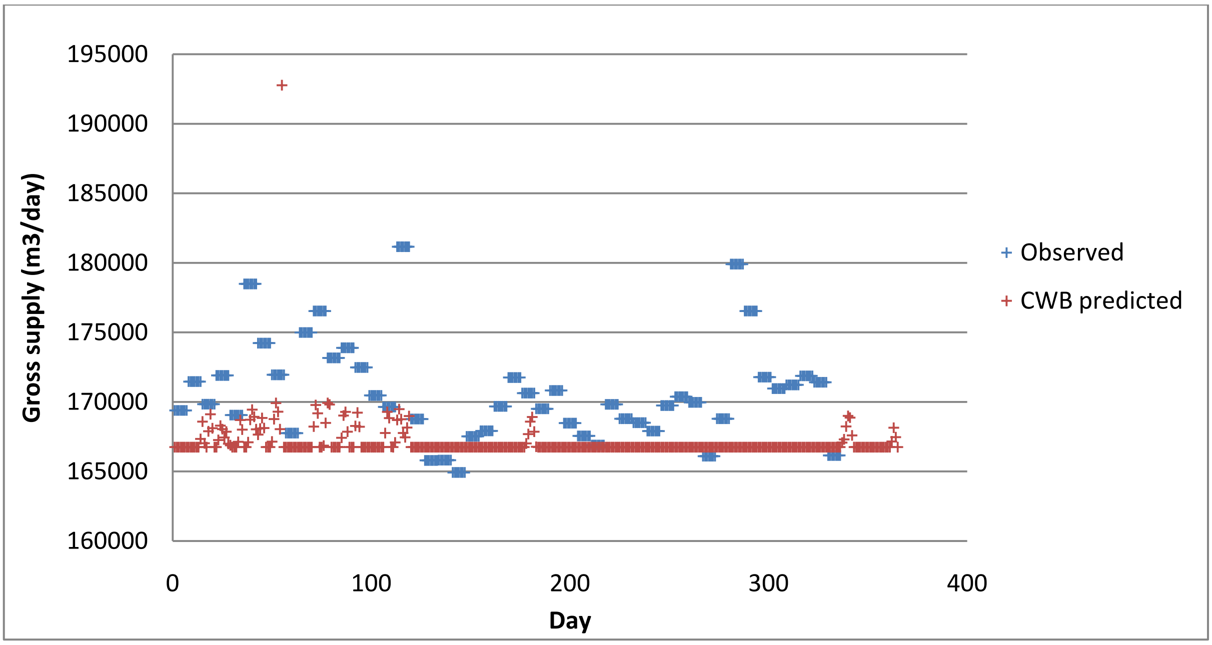


Figure 5.17 Predicted and observed gross mains supply (April 2008 to March 2009). Simulated uses TG = 0.09 and residential occupancy = 2.5.

5.11 Irrigation

There are two primary parameters that control irrigation in *CWB*. They are trigger-to-irrigate (TG) and proportion of area irrigated (Prop_I). When soil moisture falls below the TG value then irrigation is applied to raise it back to the TG level. Irrigation demand is linearly related to the "proportion of area irrigated". Changing the TG value affects not only the volume of irrigation applied but also the seasonal profile, whereas the proportion irrigated is effectively only a scaling factor.

Table 5.24 shows some initial estimates for irrigation parameters. *Rueedi & Cronin* (2005) used a TG of 0.06 in their application of UVQ to Doncaster. Since this is less than the residual soil moisture of the clay soils (0.07) and only slightly greater than that of the sandy soils (0.05) (Table 5.17) there would be almost no irrigation if this value was adopted. A higher initial estimate of 0.15 was therefore selected to begin calibration.

Table 5.24 Initial estimates for irrigation parameters.

Parameter	**Value**	**Source**
Proportion garden irrigated	Varies	Estimation and *Rueedi & Cronin* (2005)
	0.5	Estimate
Garden trigger-to-irrigate	0.15	Estimate
	0.2 (Golf courses & allotments)	Estimate
Proportion POS irrigated	0	Estimate
POS trigger-to-irrigate	0	Estimate
Supply garden w mains water?	Yes	Estimate
Supply POS with mains water?	No	Estimate

5.11.1 Calibration

The selection of a DMA, for calibration of the irrigation parameters, was based on two criteria:

1) The unitblock type should have garden area. Since the irrigation of POS is much more varied and therefore difficult to estimate.
2) There should be uniform landuse within the DMA to reduce the number of contributing variables that affect irrigation volumes.

Consequently, DMA 4463 was selected which is a terraced, residential area with small gardens (Sheldon Heath, Yardley), located over the Mercia Mudstones formation (Fig. 5.14). Total metered irrigation volume for the year April 2008 to March 2009 was 6,676 m^3, assuming a base supply of 590 m^3/day (Figs. 5.18 & 5.19).

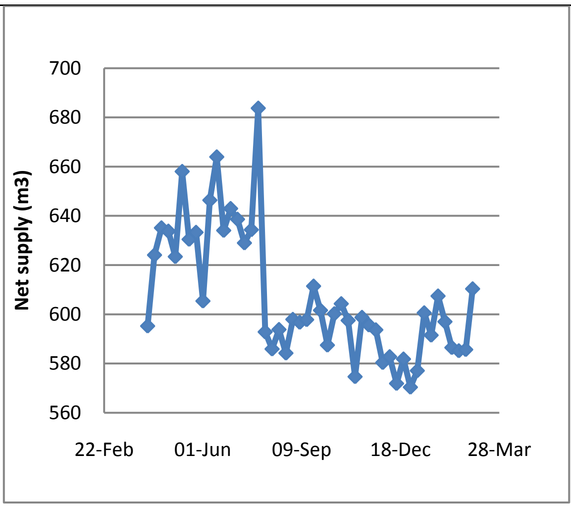
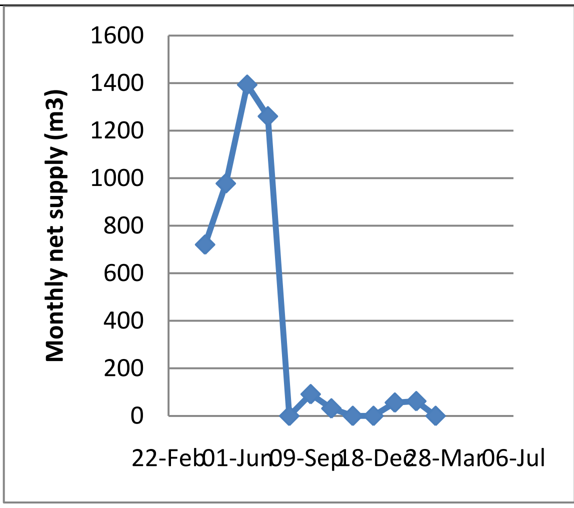
	

Figure 5.18 Weekly averaged daily net supply to DMA 4463.	**Figure 5.19 Monthly irrigation supplied to DMA 4463.**

The irrigation ratio between the observed annual demand and the maximum monthly irrigation demand is 3.9. It was found that for the clay soils, underlying DMA 4463, that a TG of around 0.4 was more suitable which is just larger than the field capacity of 0.35. The soils require this high value for TG to match the observed because they do not overlie the aquifer and so the only modelled means of losing soil moisture is by evaporation. Table 5.25 shows the effect of varying the TG on the irrigation ratio for *CWB* simulations and Figure 5.20 shows the results graphically.

Table 5.25 Variation of irrigation patterns with trigger-to-irrigate proportion. Prop_I is set to 50% unless otherwise stated.

TG	Annual irrigation (m3)	Max monthly irrigation (m3)	Irrigation ratio
0.34	1203	1202	1.0
0.38	3136	2299	1.36
0.4	4627	3031	1.53
0.4 (Prop_I = 0.2)	2700	1914	1.41
0.45 (Prop_I = 0.1)	3320	2007	1.65
0.5 (Prop_I = 0.2)	7684	3561	2.16

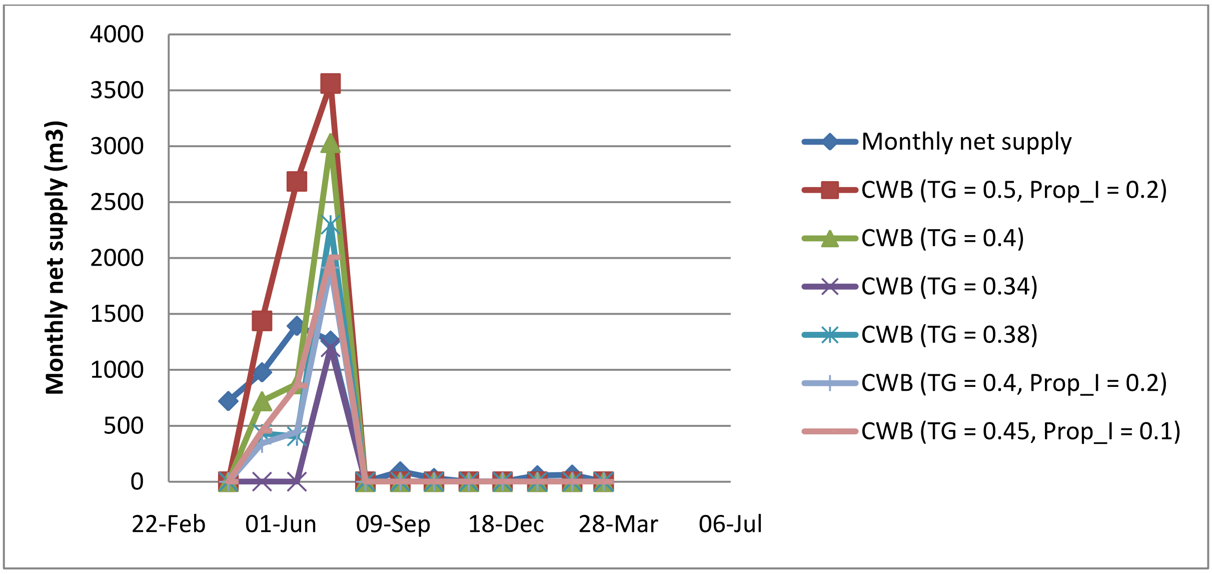
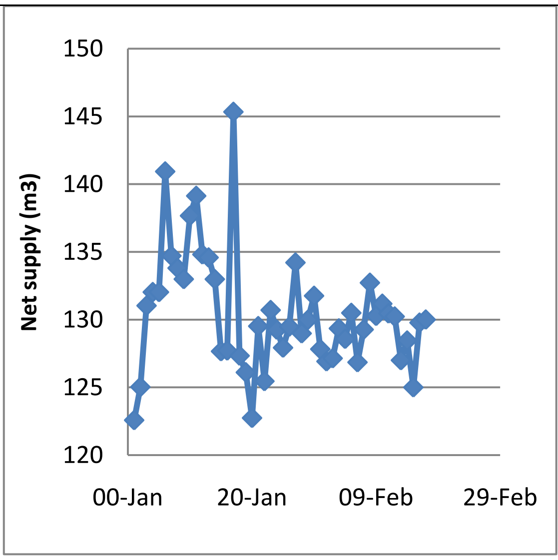


Figure 5.20 Irrigation supply to DMA 4463 varying with TG and proportion irrigated. Prop_I = 0.5 where not stated.

When the trigger-to-irrigate value is increased much above 0.45, the summer irrigation peak is predicted to be much larger than the observed. The total annual irrigation predicted by the model for this TG was 3,320 m^3 which is less than the observed (4,593 m^3). However, ~1,000 m^3 of observed "irrigation" is applied in the period October to February and so it is likely to be attributable to other factors. This discrepancy is resolved in the calibration of the study area supply in the next section. In Figure 5.20 it can be seen that there is a trade-off between matching the annual irrigation volume and the seasonal pattern (summer peak). An appropriate choice of irrigation parameters for clay soils, not overlying the unconfined aquifer, is TG of 0.45 and Prop_I of 0.1.

Since DMA 4463 has clay soils, overlying the confined aquifer, it was necessary to do a second calibration for a DMA with sandy soils overlying the aquifer, since they were likely to have very different irrigation parameters. For this purpose DMA 4255 was selected; it is a detached, residential area with small gardens (Fig. 5.16). Total metered irrigation volume for the year April 2008 to March 2009 was 888 m^3, assuming a base supply of 131 m^3/day (Figs. 5.21 & 5.22).



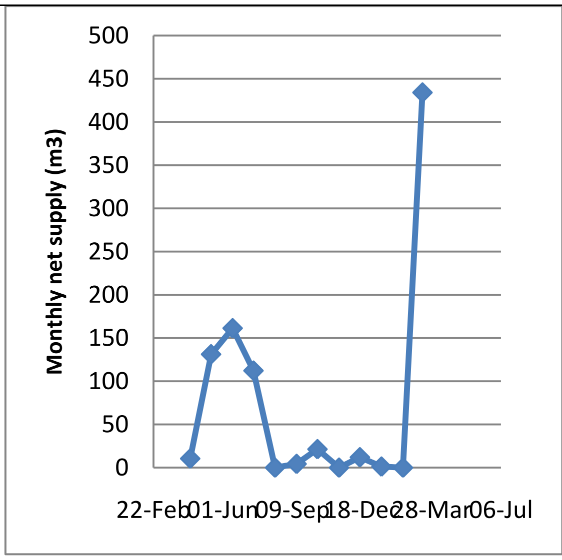


Figure 5.21 Weekly averaged daily net supply to DMA 4255.	Figure 5.22 Monthly irrigation supplied to DMA 4255.

Simulated results are shown in Figure 5.23. The best fit to the summer peak uses a TG of 0.22 and Prop_I of 0.5. The simulated annual irrigation volume of 606 m^3 for these parameters is also an acceptable match to the observed (888 m^3) for the purposes of a scoping model.

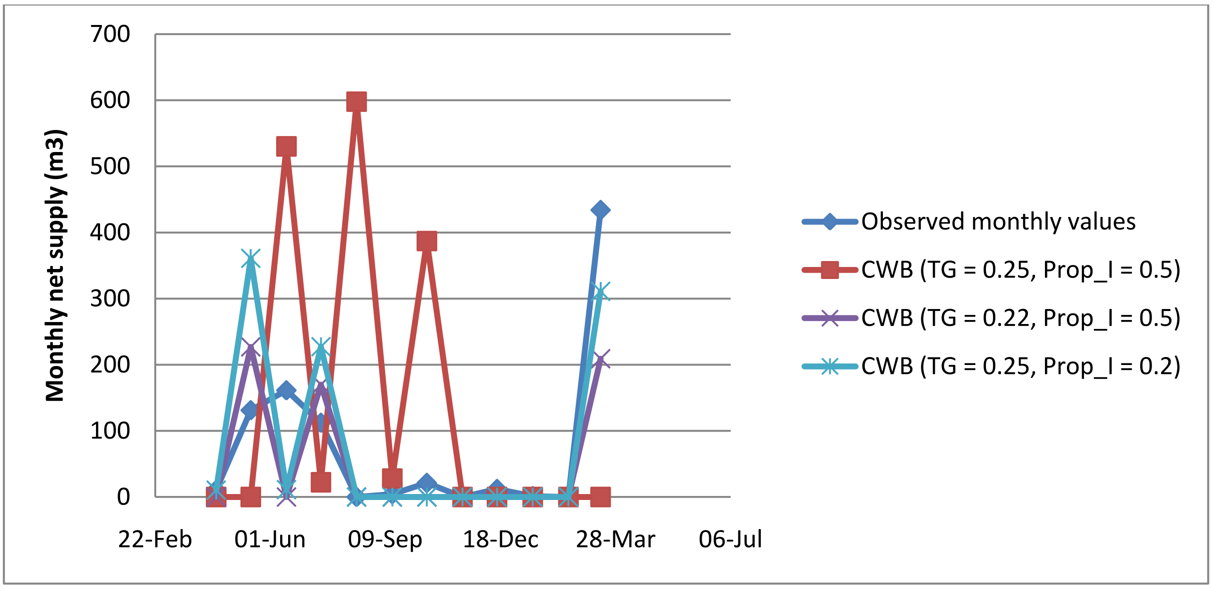


Figure 5.23 Irrigation supply to DMA 4255 varying with TG and proportion irrigated.

Having calibrated the principal soil parameters for the study area based on recharge and irrigation parameters for DMAs 4463 and 4255 it was then necessary to validate the supply to the study area.

5.12 Calibration of the Study Area Water Supply

Figure 5.24 shows the results of a study area simulation using the calibrated soil and irrigation parameters. The simulated irrigation peaks are larger than the observed but the total volume supplied during the summer period is similar.

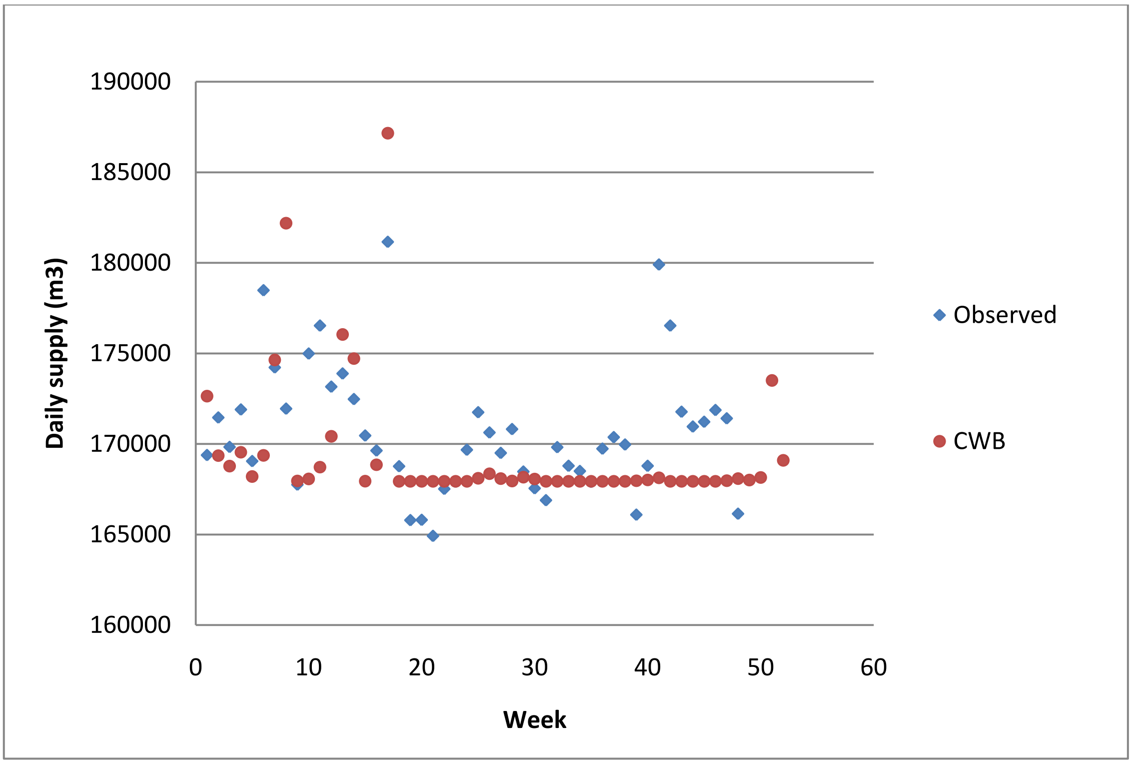


Figure 5.24 Weekly averaged gross mains supply to the study area (April 2008 to March 2009). Sand soils (TG = 0.22, Prop_I = 0.5); Clay soils (TG = 0.45, Prop_I = 0.1). Drain max = 2mm.

There is a failure to predict the large peak in autumn/winter around week 41. However, this is unlikely to be an irrigation peak at that time of year so there must be additional factors causing the fluctuations in water use. Factors affecting water demand are (*Wurbes*, 1997):

- resident and seasonal population
- leakage
- personal income
- climate
- weather conditions
- number, market value and type of housing units
- employment in service industries

- manufacturing employment and output
- water and wastewater prices and rate structures
- irrigated acreage in residential, commercial and public use
- types of lawn and watering practices
- water using appliances
- demand management activities

Figure 5.25 shows the weekly averaged daily leakage for the study area. It can be seen that there is a peak around 08/01/09 which is week 41. Increased pipe bursts due to thermal expansion/contraction of the pipes in the cold weather would explain this peak. The peak is concurrent with the winter peaks observed in the demand data, so this would suggest that incorporation of seasonal leakage variation, into the model, is required to accurately simulate mains water demand.

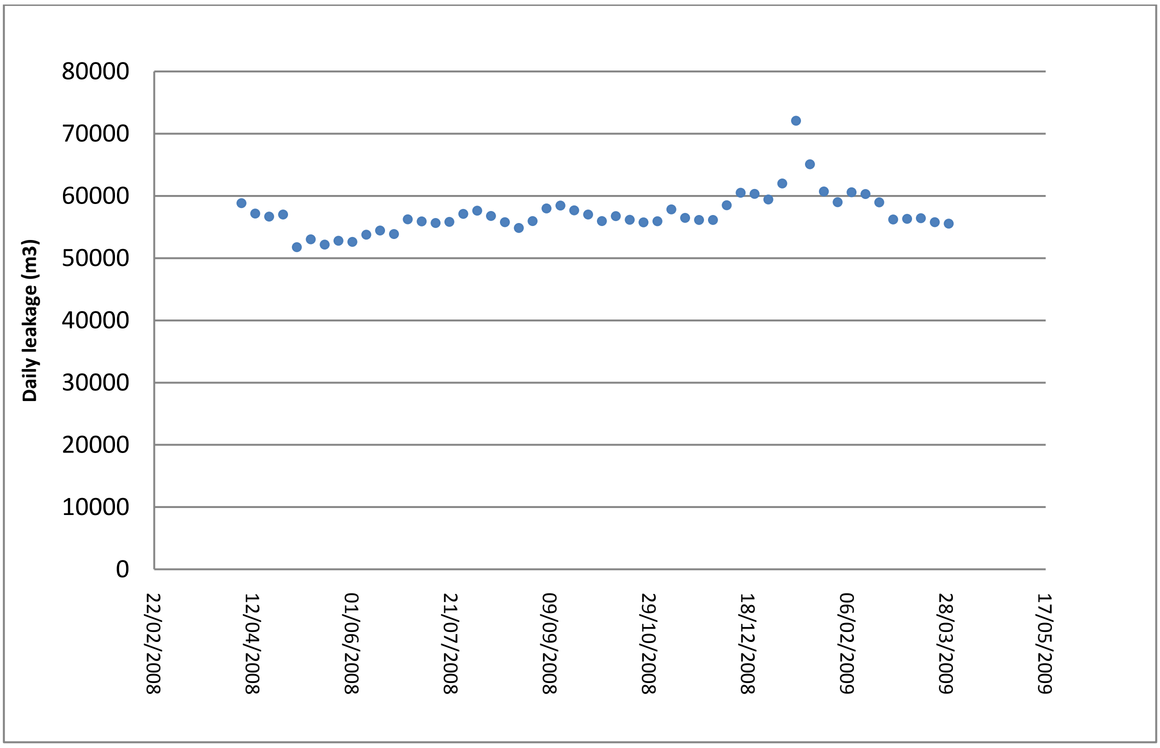


Figure 5.25 Weekly average daily leakage for all DMAs.

The average daily leakage for the period April 2008 to March 2009 was 57,151 m^3. If *CWB* uses weekly leakage correction factors, based on deviation from the average for 08-09, then a much better fit is obtained at the peak around week 41 (Fig. 5.26).

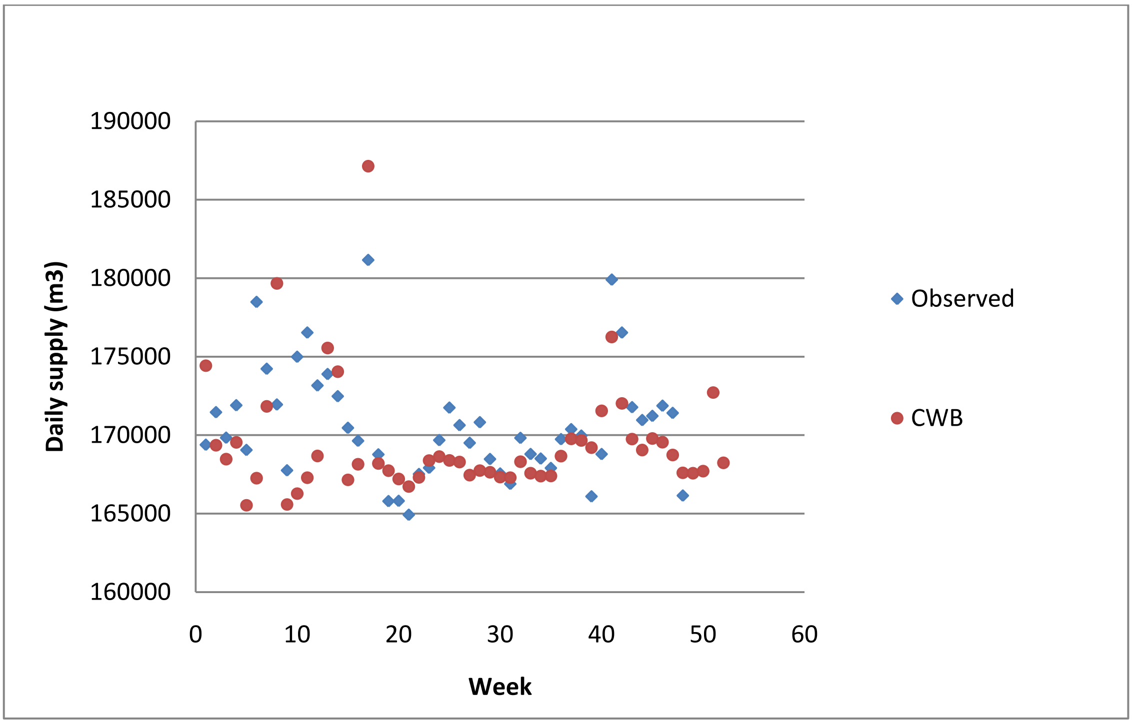


Figure 5.26 Weekly averaged gross mains supply to the study area with seasonal leakage factor applied (April 2008 to March 2009).

5.13 Calibration of Wastewater Flows

After calibration of the water supply to the 2008-2009 water records from Severn Trent, calibration of stormwater and wastewater flows was undertaken.

Figure 5.27 shows that the drainage areas, of priority interest for flow calibration, are those situated on the boundary of the Minworth Catchment (that are also within the study area). They are preferred because there is no flow into them from upstream drainage areas.

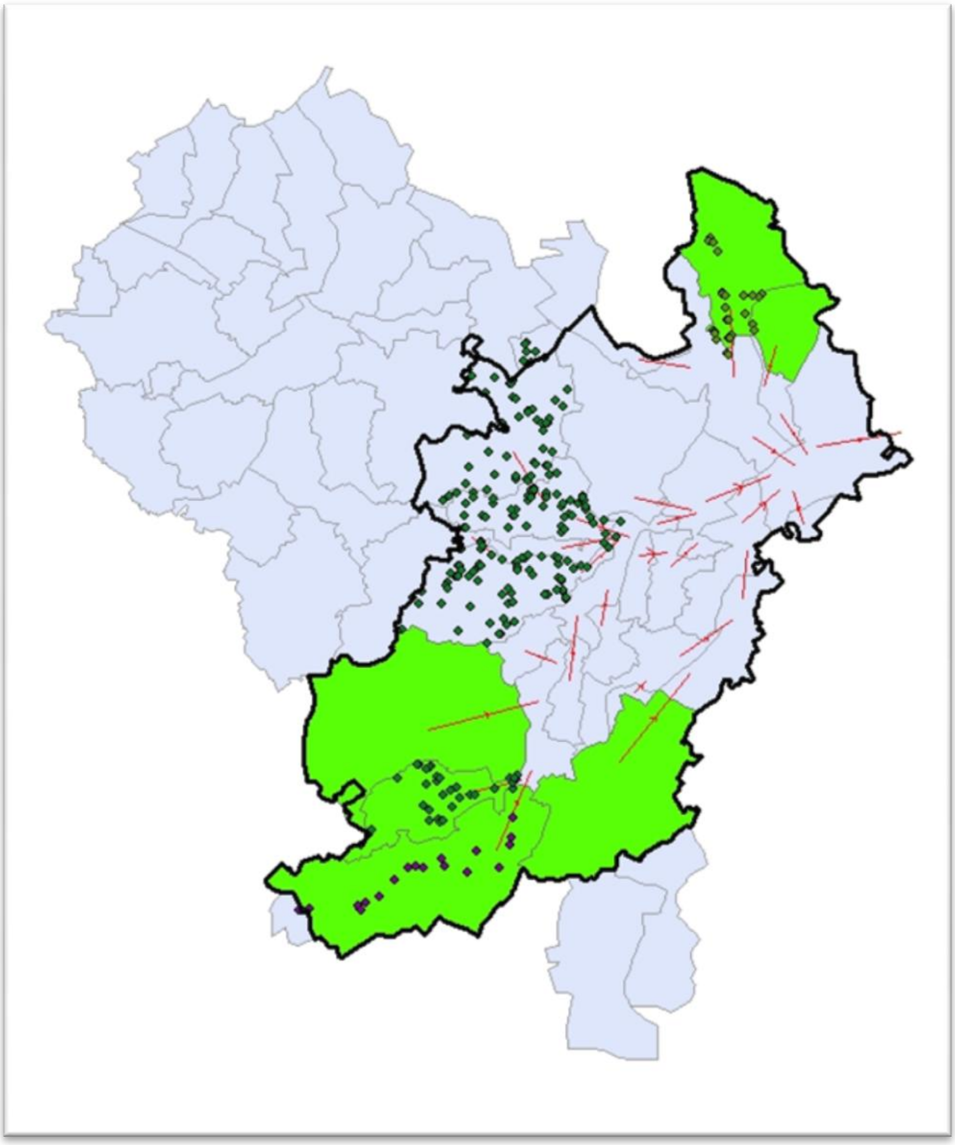


Figure 5.27 Map showing drainage areas to Minworth (grey), drainage areas of priority interest (green), flows (red arrows) between drainage areas within the study area (outlined black). Dots are flow monitoring points for select DAPs.

Table 5.26 shows the requests for flow information to Severn Trent. Data for Drainage Area Plans (DAPs) later than 2005 were much easier for Severn Trent to access and so only data for Griffin Brook, of the priority drainage areas, was procured. Aston & Handsworth and Hockley Main DAPs were carried out in 2006 so data were available but they receive flow from upstream drainage areas so the results were more difficult to analyze accurately. Figure 5.28 shows the locations of the flow monitoring points (FMs) of interest.

Table 5.26. Drainage areas of interest for calibration. Drainage areas of priority interest are highlighted in green.

DAP name	DAP Year	FM ref number	N classification	DAP_code
Aston & Handsworth	2006	FM72	FM206	F-922-19-AMP4
		FM83	FM222	F-922-19-AMP4
		FM11	FM254	F-922-19-AMP4
		FM113	FM238	F-922-19-AMP4
Birmingham Smethick	2001	FM114	FM379	F-922-19-AMP4
Hockley Main	2006	FM258	FM330	F-922-19-AMP4
Upper cole Valley West		FID 130		F-922-03
		FID 179		
Griffin Brook	2006	FM136	FM412	F-922-19-AMP4
		FM137	FM414	F-922-19-AMP4
Bournbrook	no date	FM03		F-922-09
		FM04		F-922-09
Upper Rea Main	2001	M21		F-922-13-AMP3
Upper cole Valley East	2001			
Mere Green	2004	FM30		F-922-10-AMP3
		FM31		F-922-10-AMP3
Langley Mill	2004	FM2 FM8 FM27 FM31 FM66		F-922-22 F-922-22 F-922-22 F-922-22 F-922-22

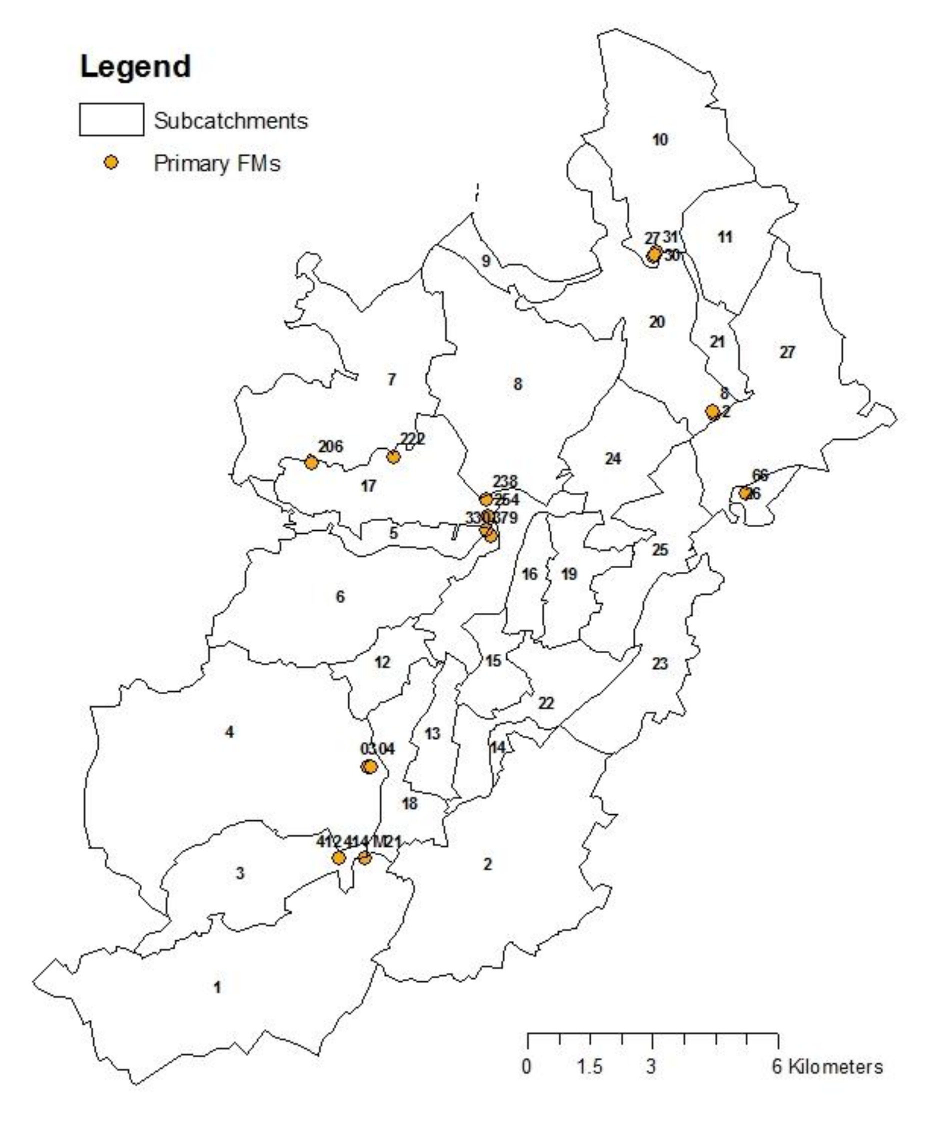


Figure 5.28. Location of flow monitoring points of interest.

FM 412 is a 750 mm diameter foul sewer that was monitored during a two month period (February to April 2007) (*STW*, 2009). It is located at the outflow point from Subcatchment 3. When the percentage of stormwater runoff flowing into the foul network was set to 1% then *CWB* failed to match the fluctuations in the observed data (Fig. 5.29). By increasing the inflow proportion to 10% a better match was seen but the inflow proportion needed to be increased further.

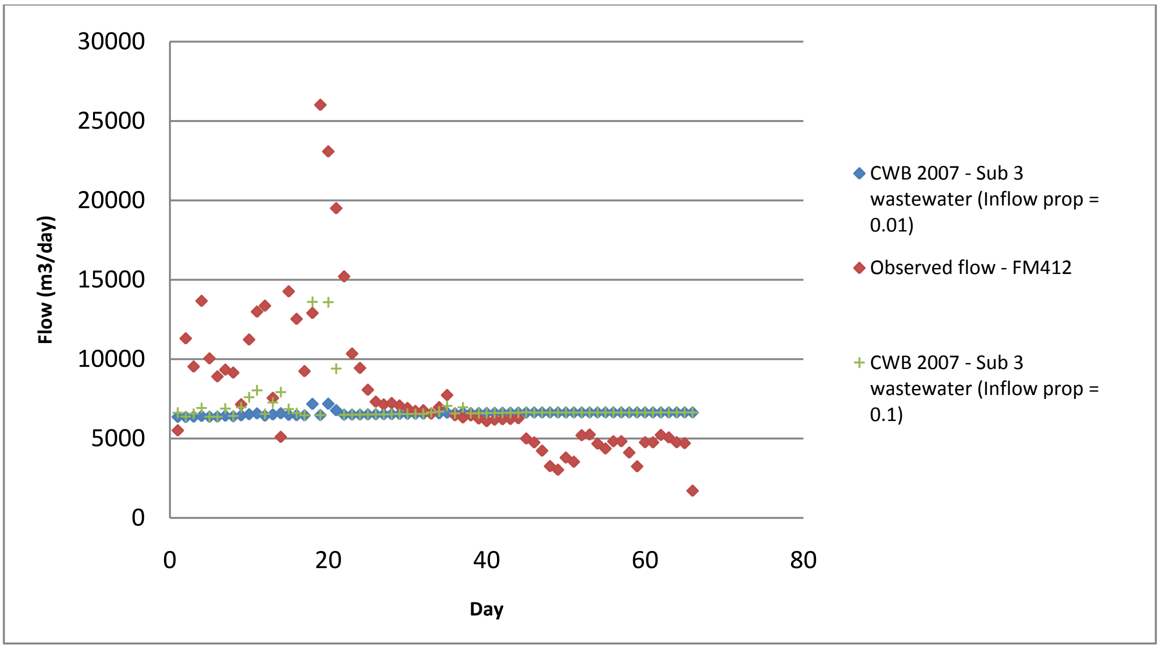


Figure 5.29. Observed daily flow in FM 412 and simulated wastewater flows for subcatchment 3 (Feb – Apr 2007).

Figure 5.30 shows the result of increasing the inflow proportion to 25%, which is a significantly improved fit. The correlation coefficient (R_{xy}) is 0.5 and was calculated using equation 5.5 (*Swift and Piff*, 2005).

$$R_{xy} = S_{xy} / (S_{xx}S_{yy})^{1/2} \qquad (5.5)$$

Where $S_{xx} = \sum x^2 - (\sum x)^2 / N$
$S_{yy} = \sum y^2 - (\sum y)^2 / N$
$S_{xy} = \sum xy - \sum x \sum y / N$
N = number of data

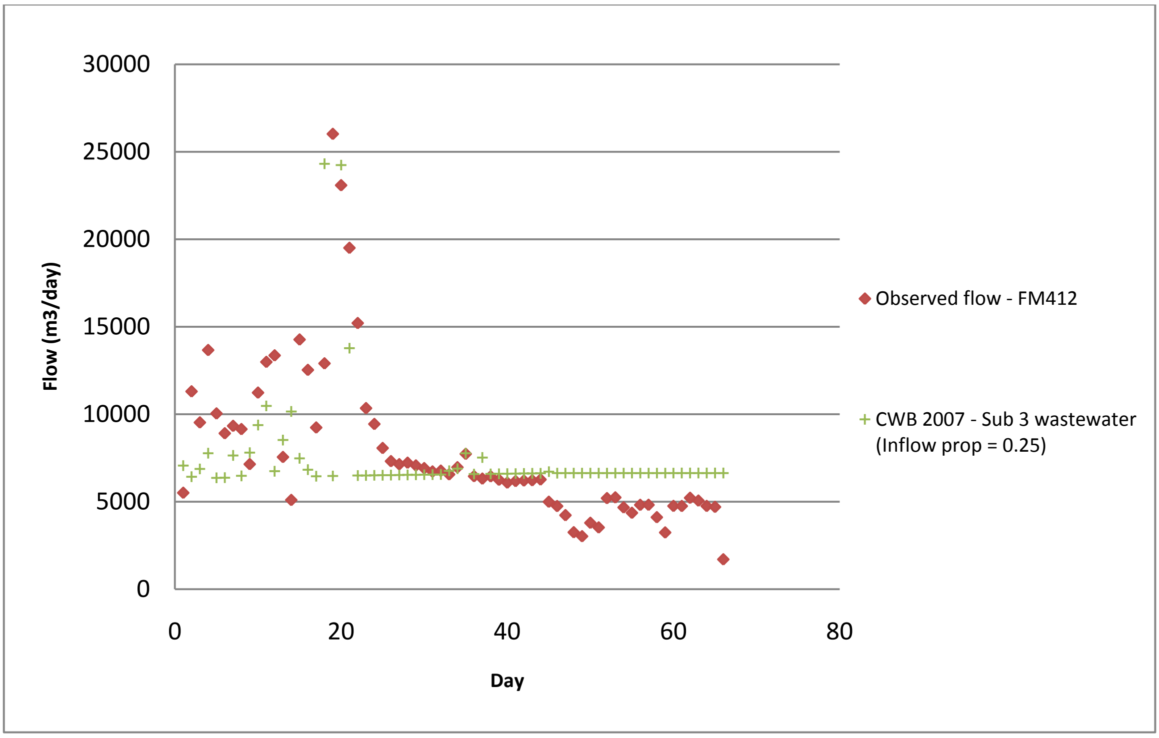


Figure 5.30 Observed daily flow in FM 412 and simulated wastewater flows for subcatchment 3 (Feb – Apr 2007).

5.14 Groundwater Data

The background groundwater grid was created using the ArcMap toolbox (Data Management Tools →Feature Class→Create Fishnet). The grid bounds are shown in Table 5.27. The 500m spacing resulted in a grid of 42 (east-west) by 52 (north-south) cells.

Table 5.27 Grid bounds and spacing in Ordnance Survey coordinates.

Left	397500	**Right**	418500	**Grid spacing**	500
Bottom	275500	**Top**	301500	**Grid spacing**	500

5.14.1 Ground Elevation Data

Ordnance Survey (OS) Landform Profile 1:10000 DTM maps were taken from the "Data download" section of *Edina Digimap Collections* (2009). Tiles downloaded were:

SK10sw, SK00se, SO98se, SO97ne, SP07ne, SP07nw, SP08, SP09, SP17nw, SP18, SP19. These were subsequently processed in *Global Mapper Software, LCC (2009)* to an ASCII file of format (x,y,z) with a 10 m resolution, which was then read into *CWB*. The ground elevation values were bin averaged to get a single elevation for the centre of each 500 by 500 m cell. The grid bounds used for downloading the ground elevation data were the same as the groundwater grid.

The elevation data, processed by *Global Mapper*, were also viewed in ArcMap (Fig. 31). In order to view the data clearly the colour scale had to be adjusted (Right-click layer→properties→symbology; under "Type" choose minimum-maximum and under "Color ramp" choose the preferred colour scale; "Apply" the changes).

As can be seen in Figure 5.31 the topography is gently undulating, ranging from about 230 m in the southwest to 70 m AOD in the eastern part of the Tame Valley.

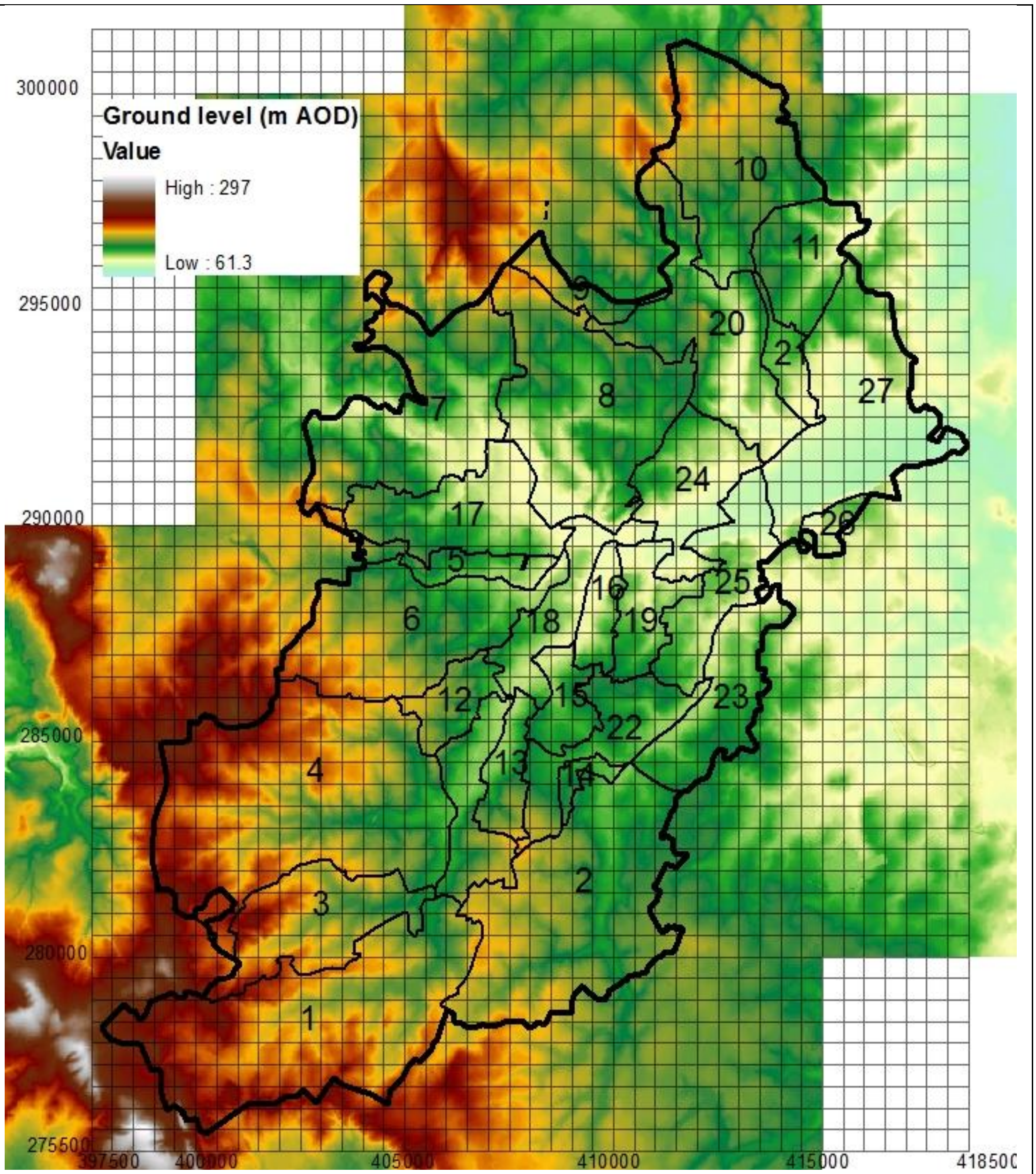


Figure 5.31 Ground elevation map for the Birmingham Study Area.

5.14.2 Birmingham Aquifer Data

Initial water heads and transmissivities (x and y), for each cell, were based on the results from the Groundwater Vistas model for Birmingham, developed by *Daly* (2005) and *Turner* (2009). Conversion of the cell-centred transmissivities to wall centred, required for *CWB*’s groundwater calculations, was performed within *CWB*. Two methods were considered: arithmetic and harmonic (equations 5.6 & 5.7) (*Fitts*, 2002).

(5.6) $K_{arith} = (K_1 + K_2) / 2$
(5.7) $K_{harm} = 2.K_1.K_2 / (K_1 + K_2)$

The harmonic method was selected because it forces the transmissivity to zero at aquifer boundaries, as long as non-aquifer transmissivities are zero.

Each cell was assigned a value of 1 or 0, representing unconfined aquifer or non-aquifer/confined aquifer, respectively. Alternatively, this could have been obtained by the following method:

1) Write a program to get the overlap of aquifer and the grid.

2) Assign a value of 1 to any cell with aquifer coverage ≥ 0.5, otherwise assign 0.

Groundwater flow is in a general north easterly direction from the high ground in the south west toward the river Tame (*Turner*, 2009) (Figs. 5.12 & 5.31). The depth to the base of the aquifer ranges from zero in the west to 125m near the fault (*Greswell*, 1992). The sloping base of the aquifer has two impacts on the model: it reduces the transmissivity across cells in the west (since T = Kb; where T is transmissivity, K is hydraulic conductivity, and b is thickness of aquifer layer) and it reduces the volume of water available for abstraction in boreholes over the western part of the aquifer.

5.14.2.1 Boundary conditions

All boundary conditions at edge of the aquifer are modelled as no-flow. In the Birmingham model this is approximately correct since the aquifer is underlain by low permeability carboniferous coal measures, it thins out over the coal measures in the west and south-west and is bounded by The Birmingham Fault, (Fig. 5.32) to the east and south-east, across which flows are very limited (*Daly*, 2005).

5.14.2.2 Groundwater Abstractions

Abstractions from the Birmingham aquifers amounted to 75 Ml/day in the 1940s but this had dropped significantly to 15 Ml/day in 1993 as a result of industrial decline (*McKay*, 2003). With the reduction in abstraction, groundwater levels have risen and by 2020 it is predicted that they will reach 2 m below ground level in some areas (*Knipe et al,* 1993). Rising groundwater is a major concern for Eastside, a 170 ha site to the east of the city centre currently undergoing physical regeneration; for example, it has caused problems with underground parking for developers of the Masshouse scheme (*Hunt & Lombardi*, 2006).

Groundwater abstraction data was inherited from the work by *Turner* (2009) in his Groundwater Vistas simulation of borehole abstraction scenarios for the Birmingham Aquifer. The origin of these data is the Environment Agency. Daily abstraction volumes from the period 2007-2008 were assumed to be approximately correct for the present day. The locations of each active borehole were plotted in ArcMap (Fig. 5.32) and the information exported to textfile, to be read by *CWB*. *CWB* calculates the grid cell in which each borehole is located and assumes that they are cell centred. However, five of the boreholes draw water from the confined aquifer, which is not modelled by *CWB* so these were not included in the simulation (Jaguar Cars, LDV Ltd., Tyseley, Kappa SSK Ltd, and Irving Carter).

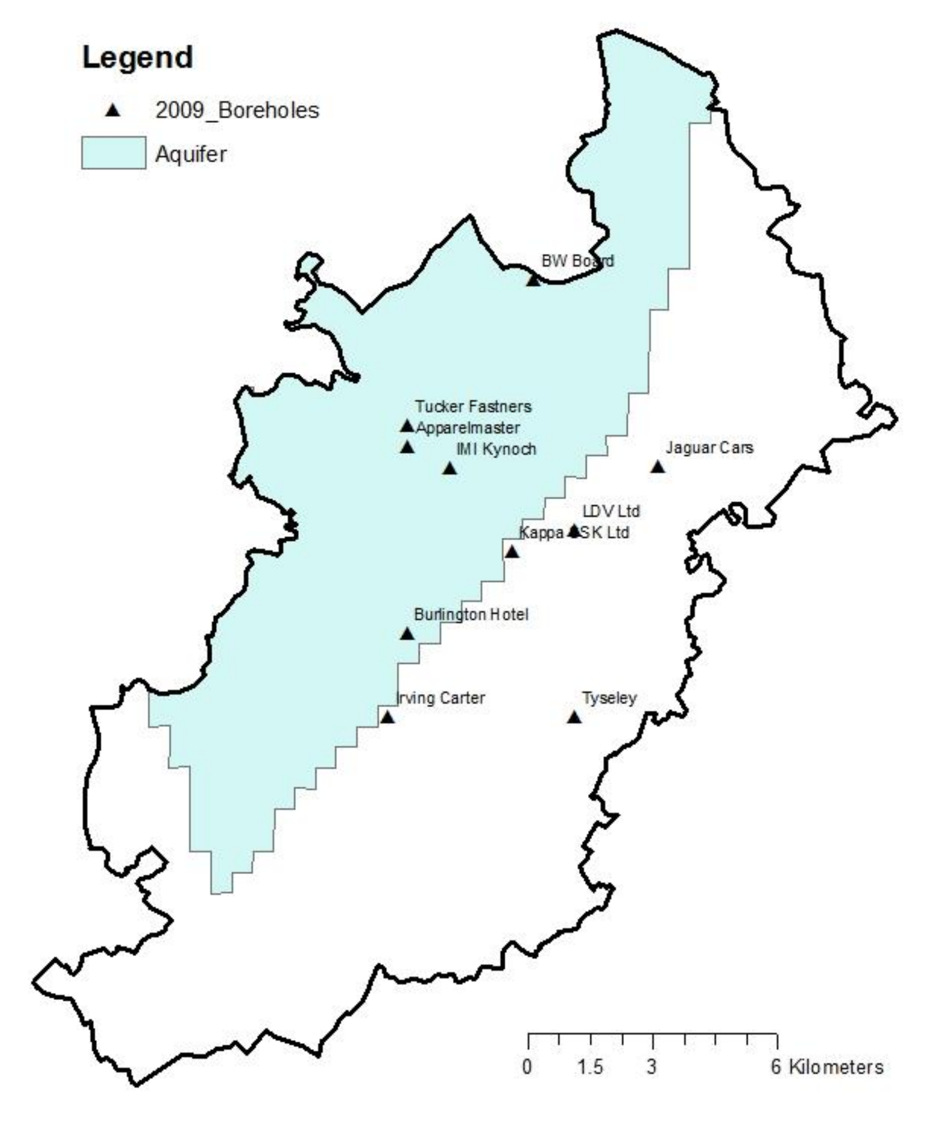


Figure 5.32 Location of licensed abstraction wells in the Study Area (*Daly*, 2005).

5.14.3 Processing the Link between the Groundwater Grid and Surface Recharge Sources

The creation of the link between the groundwater grid and overlying miniclusters required the calculation of the overlap area between each cell and minicluster. This was done in the following way:

- The *Shapedump Program* (*Warmerdam & Weidauer*, 2003) was used to extract vertices from the ArcMap minicluster shapefiles, outputting them in ASCII format. As preparation for this all MCs from the subcatchment layers were combined to create a new "Study Area" layer. Shapefiles for this layer were copied to the "Shapedump" folder. A command window was opened and from the "Shapedump" directory the following command executed the program: *Shapedump filename*
- The textfile containing the MC vertices, output from *Shapedump*, was then processed using a program, internal to *CWB*, based on the Sutherland-Hodgman algorithm, detailed below, to get the overlap data between each MC and the underlying groundwater grid.

The Sutherland-Hodgman Algorithm

In order to calculate the overlap areas between the MCs and the groundwater grid, it was decided that it would be quicker to use a program that had already been tested than to develop code from first principles. One such program uses the Sutherland-Hodgman algorithm which clips a polygon against the four edges of a rectangle sequentially, according to the rules shown in Figure 5.33. The source code for the algorithm is available in Java (*Sunshine*, 2007). It was converted into VB.NET using an automatic language converter in conjunction with manual fine-tuning and then code was written to call the subroutines.

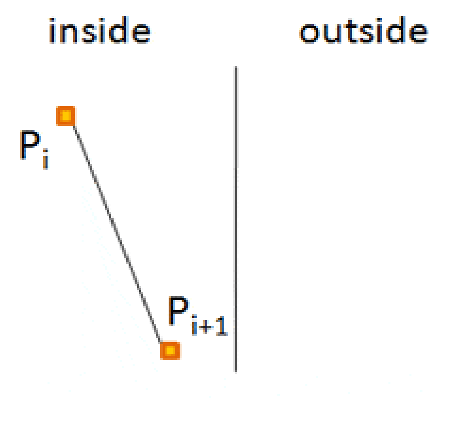
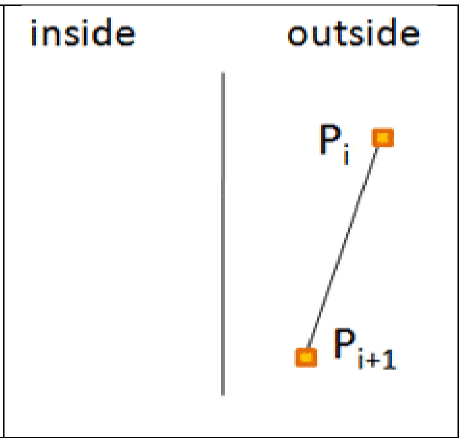
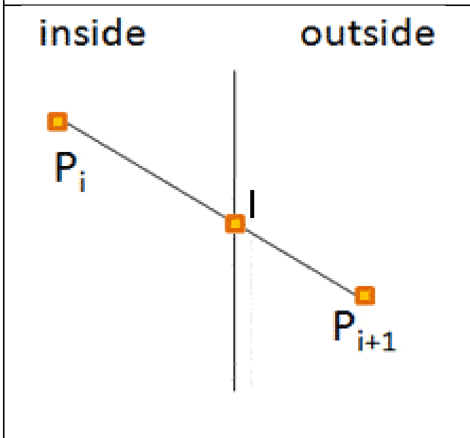
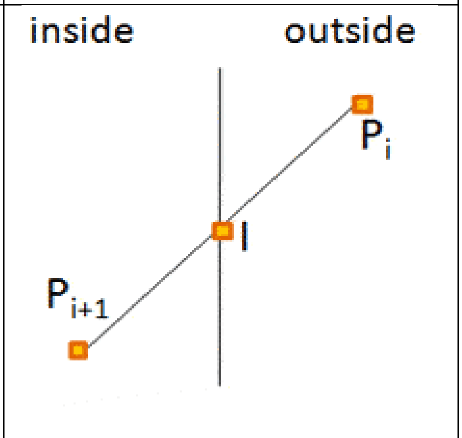
	
Both points are inside the clipping region => add the second one to the result polygon, here it's P_{i+1}.	Both points are outside the clipping region => insert none of them to the result polygon.
	
The current point is inside the clipping region but the next one lies outside => add the intersection point I to the result polygon (note that P_i was already inserted in the step before).	The current point is outside the clipping region but the next one lies inside => we add the intersection point I and the next point P_{i+1} to the result polygon.

Figure 5.33 Rules for application of the Sutherland-Hodgman algorithm (*Sunshine*, 2007).

5.14.4 Pond-Groundwater Connection

A new polygon layer called "Pond" was created. For each pond a MC was chosen in which to site it. The pond boundary was drawn within that minicluster. Once all ponds were drawn the "Pond" shapefile was processed by *Shapedump*. Overlap area between the pond and groundwater grid and the area of each pond is calculated by *CWB* based on the vertices data.

5.15 Contaminant Data

Key contaminants in terms of water pollution potential are total suspended solids (TSS), biological oxygen demand (BOD), bacteria, heavy metals, herbicides, polyaromatic hydrocarbons (PAH) and nutrients (*Ellis & Mitchell*, 2006). Three contaminant species were chosen for modelling: TSS, total nitrogen (TN) and total phosphorus (TP). Total nitrogen includes organic nitrogen, ammonia, nitrate and nitrite (*Metcalf & Eddy*, 1991). In the absence of observed data from the study area, data were taken from the literature (Tables 5.28 & 5.29). In the case of significant discrepancies *Rueedi & Cronin's* (2005) data were used, in preference, because their case study was UK based.

Table 5.28 Indoor loads (Black - *Mitchell & Diaper*, 2005; Green - *Rueedi & Cronin*, 2005). TSS – 90,000 mg/c/day (*Vesilind*, 2003); TP – 1800 mg/c/day (Alexander & Stevens, 1976), 3000mg/c/day *(Vesilind*, 2003); TN – 20,000mg/c/day (*Vesilind*, 2003)

Indoor loads	TSS	TN	TP
	mg / capita / day		
Kitchen	3990	238, 50	42, 30
Bathroom	8303	462, 100	22, 30
Toilet	36240	13709, 2500	1568, 1200
Laundry	4858	327, 70	152, 100

Table 5.29 Other concentrations (Black - *Mitchell & Diaper*, 2005; Green - *Rueedi & Cronin*, 2005; Blue – *WROCS*, 2000, Purple – *Ellis & Mitchell*, 2006, Dark Blue – *Lerner* (n.d.)).

Indoor loads	TSS	TN	TP
	mg / L		
Road	75, 11-400	1.6, 1.5, 0.18-0.98	0.21, 0.2
Pavement	75	1.6, 1.5	0.21, 0.2
Roof	75, 3-281	1.6, 1.5	0.21, 0.2
Roof FF	150	3.2, 3	0.42, 0.4
Rain	17	1.33, 2.5	0.87, 0
Water supply	0.26	0.11, 18.9, 10	0.007
Groundwater	0.26	0.11	0.007
Evaporation	0	0	0

5.15.1 Calibration

Figure 5.34 shows the concentration of TSS in daily runoff from the study area. The average concentration is 54.6 mg/l, which is comparable to values in the literature: 80 mg/l (*Wyoming Department of Environmental Quality*, 1999); 85.1 mg/l residential areas, 50.4 mg/l commercial and industrial areas (*Ellis & Mitchell*, 2006). It should be noted that the values quoted from literature are averages that have been calculated from data with a large range. For example, the range for the average 85.1 mg/l in residential areas is 21 – 1104 (*Ellis & Mitchell*, 2006).

Figure 5.35 shows the concentration of TSS in daily wastewater emissions from the study area. The average concentration is 3286 mg/l. This value is considerably larger than those found in the literature: 300-800 mg/l (*Gray*, 2005), 240 mg/l domestic (*Vesilind*, 2003).

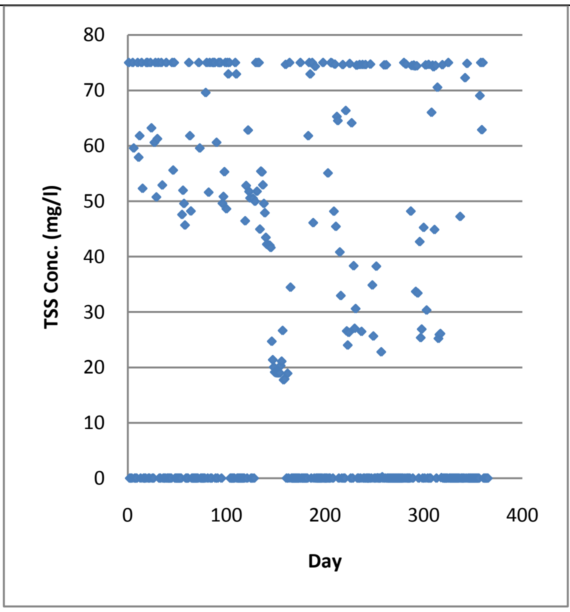


Figure 5.34 Daily concentration of TSS in runoff from the Study Area (April 2008 – March 2009).

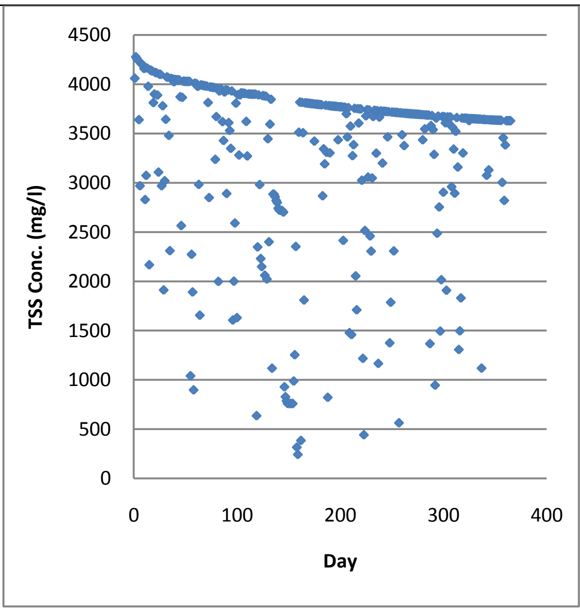


Figure 5.35 Daily concentration of TSS in wastewater from the Study Area (April 2008 – March 2009).

Figure 5.36 shows the concentration of TN in daily wastewater emissions from the study area. The average concentration is 182 mg/l, which is also significantly greater than typical nitrate concentration in sewage (*Lerner*, 2003: 30 mg/l; *Metcalf & Eddy*, 1991: 40 mg/l (range 20-85)).

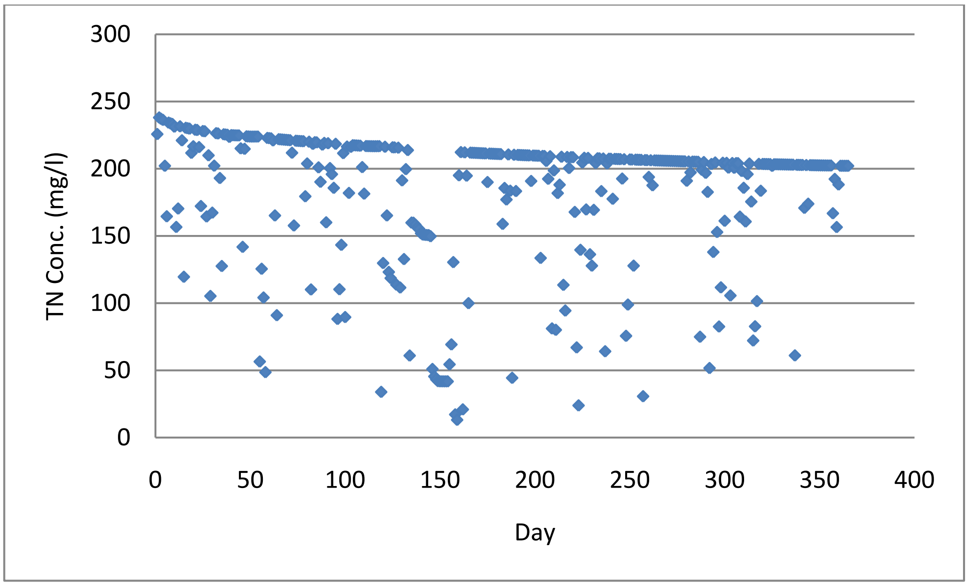


Figure 5.36 Daily concentration of TN in wastewater from the Study Area (April 2008 – March 2009).

The reason for the over-estimation of the wastewater contaminant concentrations is that the indoor per capita domestic load was applied to all UB types. However, some UB types have their daily demand defined per m^2. Table 5.30 shows estimations of loading correction factors for all the UB types in the model.

Table 5.30 Estimation of loading correction factors for each UB type (TSS)

UB type	UB number	Demand/unit (litres)	Desired WW concentration (mg/l)	Input load (mg/unit)	Reduction factor from domestic load
Residential – Terraced Small Garden	1	150	240	36000	1.000
Residential – Terraced Large Garden	2	150	240	36000	1.000
Residential – Semi-detached small garden	3	150	240	36000	1.000
Residential – Semi-detached large garden	4	150	240	36000	1.000
Residential – Detached small garden	5	150	240	36000	1.000
Residential – Detached large garden	6	150	240	36000	1.000
Residential – Flats	7	150	240	36000	1.000
Residential – Home	8	400	240	96000	2.667
Residential – High Rise	9	150	240	36000	1.000
POS – POS	10	0	0	0	0.000
POS – wooded	11	0	0	0	0.000
POS – Golf course	12	0	0	0	0.000
POS – Allotment	13	0	0	0	0.000
Recreation ground	14	35	500	17500	0.486
Retail – supermarket	15	45	500	22500	0.625
Retail with canteen	16	30	500	15000	0.417
Retail without canteen	17	30	500	15000	0.417
School – Nursery/Primary	18	15	500	7500	0.208
School – Primary through Secondary	19	15	500	7500	0.208
School – Secondary	20	20	500	10000	0.278
School – Further Education	21	20	500	10000	0.278
School – Higher Education	22	20	500	10000	0.278
Place of Assembly – Bar, Pub or Club	23	4	500	2000	0.056
Place of Assembly – Community Centre	24	6	500	3000	0.083
Place of Assembly – Library, Museum	25	6	500	3000	0.083
Place of Assembly – Restaurant	26	7	500	3500	0.097
Prison	27	150	500	75000	2.083
Office	28	2.4	500	1200	0.033
Industry	29	0.4	500	200	0.006
Industry_disused	30	0	500	0	0.000
Hotel	31	150	500	75000	2.083
Hotel with grounds	32	150	500	75000	2.083
Hospital	33	300	500	150000	4.167
Fire Station	34	10	500	5000	0.139
City Centre	35	2.4	500	1200	0.033
Garage	36	0.11	500	55	0.002
Public Baths	37	20	500	10000	0.278

UB type	UB number	Demand/unit (litres)	Desired WW concentration (mg/l)	Input load (mg/unit)	Reduction factor from domestic load
Medical Centre	38	4.5	500	2250	0.063
Health Centre	39	4.5	500	2250	0.063
Church	40	6	500	3000	0.083
TA Centre	41	150	500	75000	2.083
Depot	42	0.11	500	55	0.002
Mansion	43	350	500	175000	4.861
Kennels	44	3	500	1500	0.042
Road Main	45	0	0	0	0.000
Road Secondary	46	0	0	0	0.000
Railway	47	0	0	0	0.000
Parking	48	0	0	0	0.000
Waterworks	49	0.225	500	112.5	0.003

The correction factors can be roughly summarised as shown in Table 5.31.

Table 5.31 Coefficients to allow application of domestic indoor loads to all UB types.

UB types	UB numbers	Correction factor
Residential, Prison, Mansion	1-9, 27, 43	1
Retail, Schools, Recreation ground, Public Baths, Fire station	14-22, 34, 37	0.3
Place of Assembly, Medical/Health Centre, Church	23-26, 38-40	0.07
Office, City Centre,	28,35	0.03
Industry, Garage, Depot, WaterWorks	29,36,42,49	0.005

Figures 5.37 and 5.38 show the results of simulation with the correction factor applied. The average TSS concentration, for the study area, is 375 mg/l and the average TN concentration is 34 mg/l. These values are both within the bounds suggested by the literature (*Gray*, 1999; *Vesilind*, 2003). The days with low concentrations are a result of the dilution of wastewater with stormwater inflow. It can be seen from the gradual flattening of the concentration curves that the system takes about a year of simulation to approach dynamic steady state.

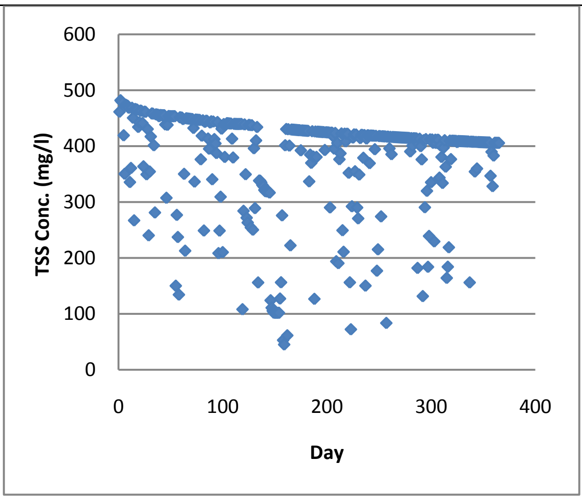
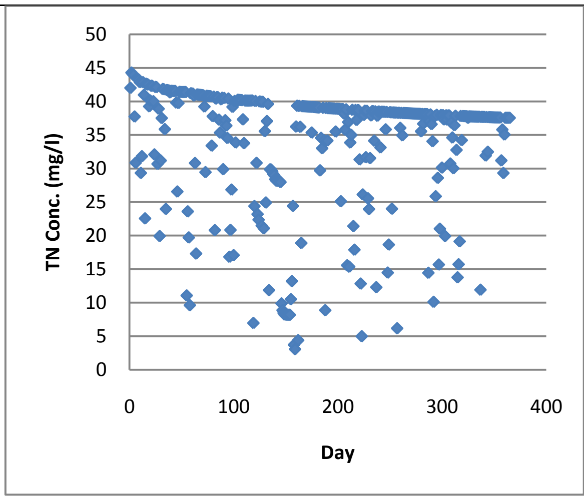
	

Figure 5.37 Daily concentration of TSS in wastewater from the Study Area (April 2008 – March 2009).	**Figure 5.38 Daily concentration of TN in wastewater from the Study Area (April 2008 – March 2009).**

5.16 Energy and Cost Data

5.16.1 Data Collection

Energy and cost data for mains supply and centralised wastewater treatment were taken from industry reports. In the Water UK Sustainability Indicators Report (08-09), operational energy in the water industry was 8650 GWh and water abstraction was 2,009,549 Ml (after leakage deduction) which is equivalent to 4.3 kWh/m^3 (*Water UK*, 2009). The cost of water supply to the consumer is £1.3/m^3 and wastewater treatment is £0.84/m^3 (Severn Trent Bill).

Energy and cost data for decentralised water management options were obtained from the following main sources:

- Specification manuals from suppliers
- Literature

- Materials embodied energy (*Hammond & Jones*, 2008)
- Diesel consumption (*DEFRA*, 2007; *SPON*, 2005)
- SUDS dimensions (*Woods-Ballard et al.*, 2007)
- Pump operation energy (*Hunt & Lombardi*, 2006; *Woods-Ballard et al.,* 2007)
- Cost (*SPON*, 2005; *Woods-Ballard et al.*, 2007)

The data were compiled into stand-alone spreadsheets for each WM option, allowing estimation of life cycle energy and NPV cost for each.

5.17 Summary

5.17.1 Recharge

Annual recharge rates to unconfined Sherwood Sandstones lie in the range 100 – 350 mm (*Thomas and Tellam*, 2006; *Ragab et al.*, 1997; *Ellis*, 2003). After calibration, *CWB* calculated an annual recharge of 330 mm to the aquifer. Even taking into account that this was a wet year (2009) (914 mm precipitation) compared with the average for the ten year period 2000-2009 (877 mm) (*University of Birmingham Climate and Atmospheric Research Group*, 2007), this estimate is still at the high end of the range. However, other data related to the water balance were adequately modelled and therefore it is difficult to reduce the recharge without distorting these calibrated values.

Recharge from leakage of 66 mm/yr calculated by *CWB* is equivalent to a rate of 20% which compares with observed leakage levels of 25%. *Price and Reed* (1989) estimated a figure of 20-30% for unaccounted water, most of which was assumed to be leakage. It is interesting to note that leakage levels have not been reduced in the Birmingham area during the last 20 years, despite OFWAT, the water regulating authority, trying to enforce stricter targets.

Knipe et al. (1993) found discharge to rivers over the aquifer to be 8.8Ml/day (3.2Mm^3/yr) and to streams 13.3Ml/day (4.7Mm^3/yr). *CWB* calculated annual discharge to rivers and streams of 12.4 Mm^3. Since *CWB* predicts recharge of more than twice the figure given by *Knipe et al.* (1993) this is likely to be the dominant cause of the discrepancy. In addition, no-flow boundary conditions across the perimeter of the aquifer mean that groundwater can only leave the aquifer by discharge to rivers and groundwater overflow.

5.17.2 Water supply

The calibration procedure for water demand began with single land-use, residential DMAs, in order to isolate one land-use and because residential demand is well-documented and less variable than commercial or industrial demand. The process continued by examining other land uses. In particular, the demand for "hospital" and "retail" required significant alteration from the original estimates. Calibration of the irrigation parameters and leakage factors was required to match the seasonal variation in demand. It was necessary to introduce a seasonal variation in leakage in order to simulate the effects of increased pipe bursts during winter.

5.17.3 Wastewater and Stormwater

Data from key flow monitoring points were supplied by Severn Trent. Flow from subcatchment 3 was calibrated to observed data with a correlation coefficient of 0.5.

5.17.4 Contaminants

Two contaminant species were successfully calibrated to data from literature: total suspended solids (TSS) and total nitrogen (TN). These species were chosen for demonstration purposes because they are dominant loads in wastewater flows and their typical concentrations in waste and stormwater are well-documented.

6 - Validation

Validation was performed for water supply to several DMAs and the study area, mains leakage, river recharge, effective rainfall, sewer flows in two drainage areas (DA), energy and cost. In addition, there is a comparison of results, using test data, from *CWB* with those from *Aquacycle*.

6.1 Comparison with *Aquacycle*

As a first stage in the verification process, results from *CWB* were compared with those from *Aquacycle* for a number of simulations. Since the basic concepts for water flow in *CWB* are based on those used in *Aquacycle*, it was anticipated that the results would be similar. Further, since *Aquacycle* has already been verified in various case studies (*Karka et al.*, 2006; *Mitchell et al.*, 2007) then if *CWB* matches the results produced by *Aquacycle*, this would provide the necessary verification that *CWB* is producing meaningful results.

A simple test case was simulated using two miniclusters (with no roads or POS) of 10 residential UBs, each comprising of a roof and garden only with no leakage and no additional water management options. A one year time series of constant daily 1mm precipitation and 0.5mm evaporation was used. The Partial Areas soil store model (*Boughton*, 2003) was used since this is the only option available in *Aquacycle*.

Evaporation from the partial stores is taken as the lesser of:

(6.1) $E = Ep$

(6.2) $E = PS\_current\_level / PS\_capacity * E\_trans$

Where Ep = daily potential evaporation rate (m)

E_trans = maximum evapotranspiration rate (set to 7mm)

PS_current_level = pervious store current level (m)

PS_capacity = pervious store capacity (m)

Since precipitation is always greater than evaporation in this scenario, the level of the stores will never fall sufficiently for equation 6.2 to apply and the evaporation will be constant at 0.5 mm/day.

A simplified garden spreadsheet model was also constructed to model this scenario. It yielded total runoff for MC 1 of 443.75 m^3/yr, which is the same as the result from *CWB*. *Aquacycle* gives a total runoff value of 364 m^3 which is not correct.

In addition, since there is a fixed evaporation rate of 0.5 mm/day for 365 days from a 1000m^2 area (10 blocks * 100m^2 garden – with no initial loss from roof areas) the annual evaporation volume is 182.5 m^3 but *Aquacycle* calculates it to be 180 m^3. To investigate if this is a rounding error, a simulation was run with 500m^2 of POS in one cluster. By comparison with manual calculations it was found that *Aquacycle* rounds decimal places down.

Further calculations were carried out using the tutorial data supplied with the *Aquacycle* model, comprising of two residential MCs, where MC 2 has occupancy of 3 and 15 UBs of 770 m^2. Indoor water demand for MC 2 is 9.585 m^3/day (based on 213 l/c/day) or 3500 m^3 / yr (Table 6.1). *Aquacycle* calculates the annual irrigation supplied to MC 2 as 363mm, spread over an area of 15 * 770 m^2. This equates to 4190 m^3 / yr. Thus the total water supplied is (4190 + 3500) x 1.03 (leakage) = 7920 m^3 / yr. *Aquacycle* calculates it to be 6624 m^3/yr (19.6% difference) and *CWB* yields 7899 m^3 / yr (0.3% difference).

Table 6.1 Comparison of water flow results from *CWB* and *Aquacycle* for two miniclusters. MC 1 has a total area of 11000m^2 including 7200m^2 of POS. MC 2 has a total area of 18000m^2 including 11550m^2 of POS.

	City Water Balance		***Aquacycle***		**% difference**	
Minicluster:	**1**	**2**	**1**	**2**	**1**	**2**
Imported water (m3)	4913	7901	4290	6624	-12.69	-16.16
Stormwater (m3)	3082	5143	2783	4626	-9.69	-10.05
Wastewater (m3)	2291	3763	2211	3654	-3.51	-2.90
Evaporation (m3)	6210	9916	5984	9306	-3.64	-6.15
GI & POS irrigation (m3)	2636	4163	2614	4193	-0.83	0.72
Annual indoor usage (m3)	2130	3501	2130	3501	-0.02	-0.01

Examination of the ‘household water use tab’, in the *Aquacycle* results section, shows figures that are more self-consistent; for MC 2 the indoor use is 3495 m^3/yr and the irrigation is 2940 m^3/yr giving a total of 6435 m^3/yr. Taking into account leakage this becomes 6634 m^3/yr, which is only 10 m^3/yr different from the total imported water amount of 6624 m^3/yr.

Variation in the results from the two models is also seen in other aspects of the water flow balance. When PS1 is set to be a high proportion of the total garden area (e.g. 0.9) there is a substantial difference in total evaporation (25% lower in *City Water Balance*) and runoff (25% higher in *City Water Balance*) predicted by the two programs. Large differences arise when mains irrigation back up is turned on. These differences increase with increasing trigger-to irrigate ratio (TG) (e.g. 22% difference for imported water when TG is set to 0.7). Changing the TG has an effect in *Aquacycle* even when the mains back-up is off and there are no alternative WM options available, which should not be the case. CWB does not suffer from this problem.

As expected there is general agreement in the water flow results calculated by *Aquacycle* and *CWB*, but there are several areas where differences were found. Some were as a result of small conceptual differences but other results produced by *Aquacycle* are unexplained and are linked to soil and irrigation parameters.

6.2 Climate Data

Knipe et al. (1993) suggest an average effective rainfall of 250 mm/yr for the area overlying the unconfined Birmingham aquifer. The average effective rainfall for ten years of data from the Winterbourne Climate station, between 2000 and 2009 (Table 6.2), is 261 mm which compares well with the estimate by *Knipe et al.* (1993).

Table 6.2 Calculation of annual effective rainfall.

Year	Annual precipitation (mm)	Annual potential evaporation (mm)	Annual effective rainfall (mm)
2000	1055	589	466
2001	758	585	173
2002	903	623	280
2003	545	639	80
2004	840	669	171
2005	740	627	113
2006	711	646	65
2007	973	630	343
2008	1161	568	593
2009	913	587	326

6.3 Supply

6.3.1 District Metered Area Scale

6.3.1.1 DMA 4592

The purpose of this simulation was to validate the demand for a DMA with mixed land-use. DMA 4592 consists of 858 domestic properties (*STW*, 2009) and has an area of 180,433 m^2. The landuse is shown in Table 6.3. For the year 06-07, Severn Trent Water reports annual gross supply of 536,004 m^3, with average daily flow of 1469 m^3/day. Leakage data are unavailable.

CWB predicts gross supply of 504,076 m^3/day, including 20% leakage rate, and indoor water demand of 342,770 m^3/day. The breakdown of the annual results by UB type is shown in Table 6.3. It can be seen that the majority of the demand is for the retail area even though it is less than a third of the "City Centre" area. The school has no demand because the UB area is larger than the MC area and so it has been automatically allocated as POS by *CWB*.

Table 6.3 Land use and annual demand in DMA 4592.

UB Type	Area (m^2)	Annual Demand (m^3)
City Centre	131,303	135,265
Office	1,529	1,664
Retail with canteen	41,724	362,316
Community Centre	5,137	4,831
School – higher education	652	0

The difference between the *CWB* predicted demand and the observed value is 6%, which is acceptable.

6.3.1.2 DMA 4660

The high retail demand, observed in simulation results from DMA4592, was investigated further using another mixed land-use DMA. DMA 4460 consists of 468 domestic properties (*STW*, 2009) and has an area of 598,688 m^2. The landuse is shown in Table 6.4. For the year 08-09, Severn Trent Water reports average gross supply of 391 m^3/day, with average leakage of 150 m^3/day (38% leakage).

CWB predicted gross supply of 4993 m^3/day, including 38% leakage rate. The breakdown of the annual results by UB type is shown in Table 6.4. "Retail with canteen" accounted for a large proportion of the demand. The difference between the *CWB* predicted demand and the observed data is 1100% which is clearly excessive.

In a second simulation the occupancy factor for retail was reduced to 20 (from 50) and the demand to 20 l/occupant/day (from 40 l/occupant/day). *CWB* now predicted gross supply of 514 m^3/day, including 38% leakage. The breakdown of the annual results by UB type is shown in Table 6.4. The difference between the *CWB* predicted demand and the observed value improved to 31%.

Table 6.4 Land use and annual demand in DMA 4660.

UB Type	Area (m^2)	Annual Demand (m^3) (1)	Annual Demand (m^3) (2)
Retail with canteen	174,836	1,663,694	73,942
Industry	294,890	58,749	29,452
Terraced small garden	123,788	100,007	84,112

Verification was undertaken for the water supply to two mixed land-use DMAs. The calculated supply was within 6% of the observed for DMA 4592, which is an excellent level of accuracy for a scoping model. The calculated supply for DMA 4660 was far in excess of the observed but this was reduced to 31% by reducing the supply to the "Retail with canteen" UB type.

6.3.2 Study Area Scale

A description of the calibration of study demand using Severn Trent supply and leakage data from the period April 2008 to March 2009 was described in the previous chapter. The results of a subsequent validation simulation, for the study area, using observed data from 2006-2008 are shown in Figure 6.1. Although the correlation coefficient is quite low (0.4), it can be seen that the general seasonal pattern and volume is an acceptable match for a scoping model applied at the city scale. Since leakage data were not provided pre-2008, the leakage factors for 2008 were applied to all years. This introduces an error that explains some of the deviation between observed and calculated flows over the longer time scale.

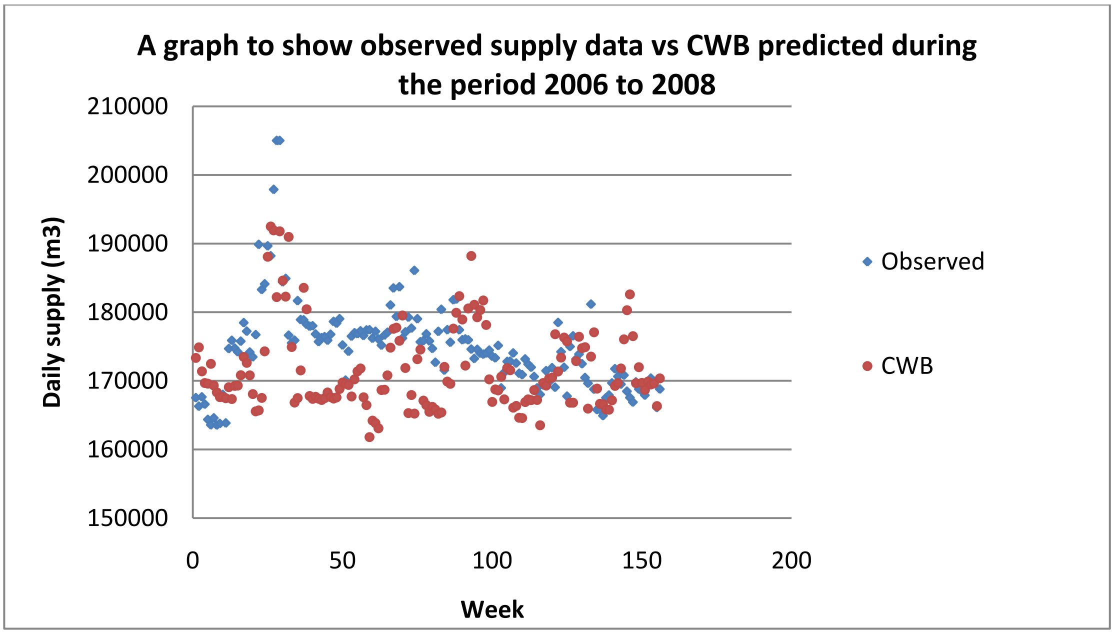


Figure 6.1 Weekly averaged gross supply to the study area. Validation of the results from *CWB* with supply data from Severn Trent (2006-2008).

6.4 Wastewater Flows

After calibrating the wastewater flows from Subcatchment 3, using FM 412, validation simulations were run for flows from Subcatchments 1 and 4. FM 414 is a 1650 mm diameter foul sewer that was monitored during a two month period (February to April 2007) (*STW*, 2009). It is the main sewer from the Upper Rea and is near the outflow point from Subcatchment 1. The reason for monitoring this site was because previous flooding had been reported in this area (*STW*, 2009). Figure 6.2 shows the observed data versus *CWB* wastewater flows with 25% stormwater inflow. Although the correlation coefficient is quite low (0.5) it can be seen that the general shape of the observed wastewater flows is predicted by the model. A significant contributor to the weak correlation is that, in this subcatchment, *CWB* predicts a base wastewater level that is below observed levels, predicting approximately 20,000 m^3/day compared with the observed average of 30,000 m^3/day.

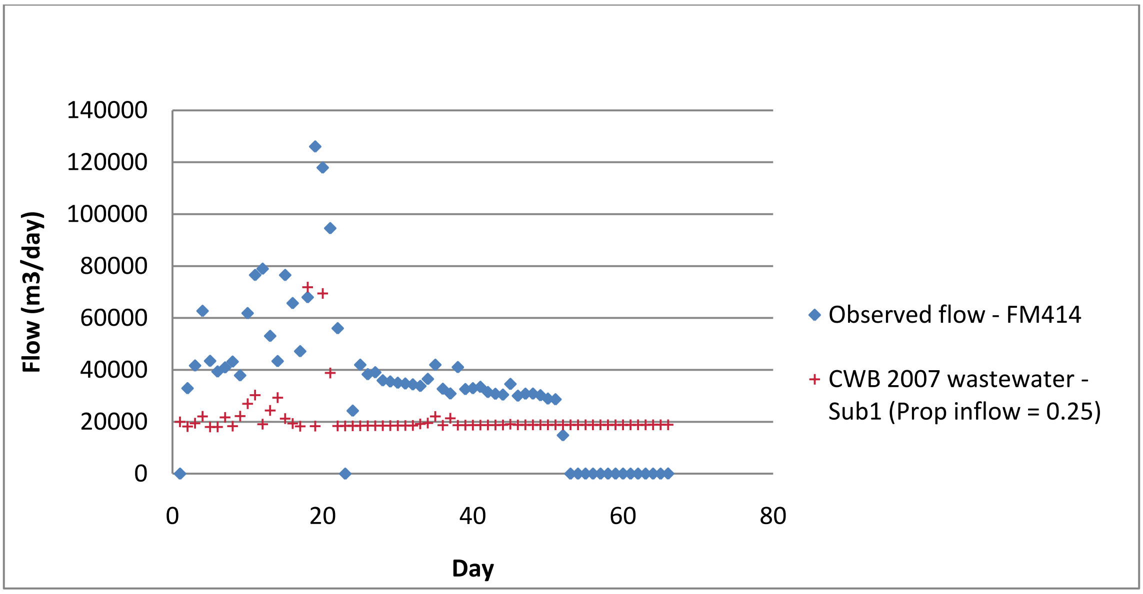


Figure 6.2 Validation of predicted wastewater flows from Subcatchment 1 with observed data by FM 414 during the period February to April 2007.

Figure 6.3 shows the observed flow (FM 03) versus simulated wastewater flow (with 20% inflow and 25% inflow) for Subcatchment 4. The correlation coefficient is 0.8. There is a much better fit between the observed and predicted flows which is acceptable for the purposes of a scoping model.

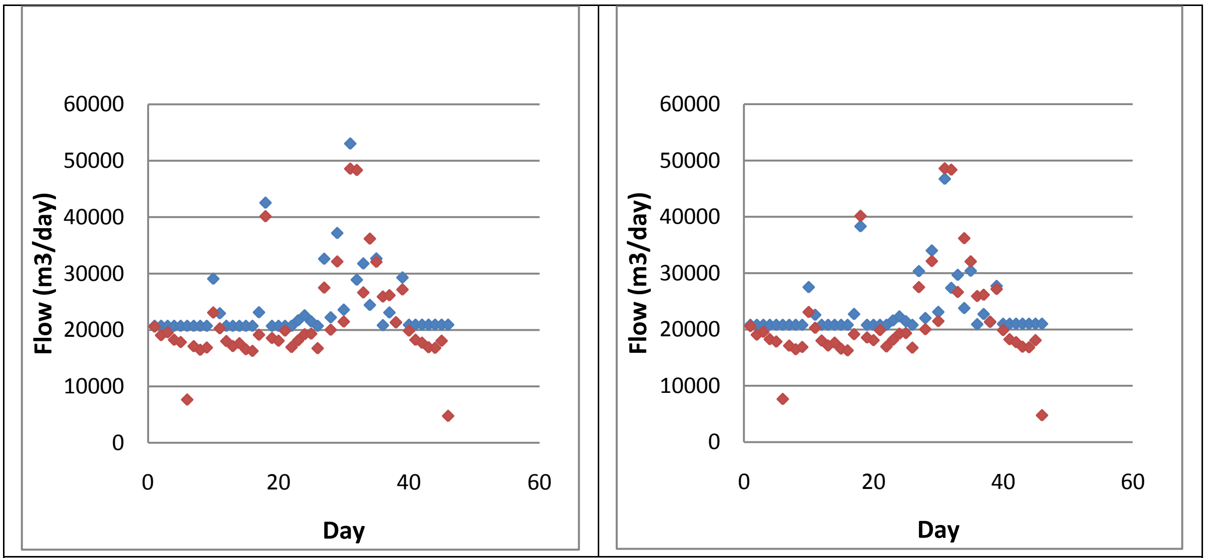


Figure 6.3 Validation of predicted wastewater flows (Left - 20% inflow, Right – 25% inflow) from Subcatchment 4 with observed data by FM 03 during the period April to June 2006. Blue – observed, Red – simulated.

6.5 Energy and Cost

Energy and cost validation was carried out by comparing values calculated by *CWB* for each water management option with equivalent, published studies.

6.5.1 Rainwater Harvesting

For a typical domestic raintank system, with a HDPE tank of 2m^3, supplying 60m^3/yr for toilet and garden use, *CWB* calculates energy use of 8.67 kWh/m^3 and net present value (NPV) whole life cost (WLC) of £1.7/m^3. 60% of the energy use is a result of the high embodied energy of the HDPE tank (wall thickness 5cm). If a concrete tank (wall thickness 7 cm) is used instead, the energy use is 3.8 kWh/m^3 and the NPV cost is £1.9/m^3. *BlueScope Water* (n.d.) states that the mass of a 5m^3 HDPE tank is 100kg. This is equivalent to a cubic tank with 1cm walls. With a reduced wall thickness of the HDPE main tank, *CWB* calculates the energy use to be 4.6 kWh/m^3.

The costs of rainwater harvesting systems were found to be in the range £0.29 – 2.32 /m^3, excluding capital costs (*Roebuck*, 2008 after *Brewer et al.*, 2001), which concurs with the results from *CWB*.

Mithraratne & Vale (2008) conducted a study into the life cycle energy use, CO_2 emissions and economic cost of residential rainwater harvesting in Auckland, New Zealand. In the simulation 25m^3 plastic or concrete tanks supplied all household needs of 180 m^3/yr. For a useful life of 50 years energy use was estimated to be 1.6 kWh/m^3 (plastic) and 0.8 kWh/m^3 (concrete). This is significantly less than predicted by *CWB*. *BlueScope Water* (n.d.) estimate the life cycle (LC) embodied energy and pump energy to be 1.3 kWh/m^3 (Concrete tank) and 1.4 kWh/m^3 (HDPE tank). However, since pumping energy costs alone are of the order of 1-3 kWh/m^3 then these estimates are very low (*Clarke et al.*, 2009).

6.5.2 Wastewater Recycling

For a typical domestic greywater system, with a HDPE tank of 2m^3, supplying 60m^3 for toilet and garden use, *CWB* calculates energy use of 7.7 kWh/m^3 and NPV, WLC of £2.7/m^3. If a concrete tank (wall thickness 7cm) is used instead, the energy use is reduced to 6.9 kWh/m^3 and the NPV cost is £2.9/m^3.

Lundin et al. (2000) calculated energy usage of a small-scale WWTP, for a village of 200 inhabitants, as 154 kWh/c/yr (electricity and fossil fuels). If household occupancy of 2.1 is assumed, this is equivalent to 323 kWh/yr or 5.4 kWh/m^3 for an annual supply of 60 m^3/household/yr. The discrepancy between the value estimated by *CWB* and that of *Lundin et al.* (2000) could be explained by economies of scale or variation in the annual supply volume.

Memon et al. (2005) calculated the WLC of a small-scale greywater recycling system in Maidenhead, UK. The initial capital cost was estimated to be £1625. If a conservative lifetime of 25 years is assumed (*McEvoy*, 2008), with annual supply of 31 m^3 (*Memon et al.,* 2005), this is equivalent to £2.1/m^3. The annual operation and maintenance cost was estimated to be £2.7/m^3, resulting in a total WLC of £4.8/m^3. If *CWB* uses the reduced supply estimate of 31m^3/yr the WLC is £4.5/m^3 which is in excellent agreement.

6.5.3 Borehole Abstraction

For a borehole abstraction system with a HDPE surface storage tank of 2 m^3, supplying 100 m^3 for all uses, *CWB* calculates energy use of 10.3 kWh/m^3 and NPV, WLC of £1.6/m^3. If a concrete tank (wall thickness 10cm) is used instead, the energy use is reduced to 8.1 kWh/m^3 and the NPV cost remains the same at £1.6/m^3.

Baker (2003) estimated that a borehole abstraction scheme abstracting 0.6 Ml/day from a minor aquifer in North Staffordshire would have NPV capital costs of £146,973 (adjusted at 4.5%/yr to 2010). The annual operation and maintenance cost and license fee from the Environment Agency (EA) was estimated to be £23,407(*Baker*, 2003). If the lifetime of the borehole is assumed to be 50 years (*Ecozi*, 2009) then over its lifetime the borehole will supply 10,950 Ml. This results in a NPV WLC of 0.11 £/m^3 supplied. If the lifetime of the borehole scheme is reduced to 30 years then the WLC increases to 0.13 £/m^3. It is clear that there are significant economies of scale and that groundwater abstraction is much more viable for commercial use.

At the time of writing, literature searches did not show any studies investigating the life cycle energy use of borehole abstraction.

6.5.4 Septic Tanks

For a domestic septic tank (1m^3), with 100 m^3/yr of infiltration, *CWB* calculates energy use of 4.7 kWh/m^3 and NPV WLC of £0.9/m^3. 69% of the life cycle energy usage is from the use of sodium hypochlorite as a disinfectant.

At the time of writing, literature searches did not show any studies investigating the life cycle energy use or whole life cost of septic tanks.

6.5.5 Sustainable Urban Drainage Systems

In general the author found that there are very few reports on the life cycle energy use of sustainable urban drainage systems and so there is much scope for conducting new research in this area.

For a domestic brown roof system (roof area $100m^2$) with a lifetime of 50 years (*Banting et al.*, 2005) and 100 m^3/yr of attenuation, *CWB* calculates energy use of 1.4 kWh/m^3 and NPV, WLC of £1.1/m^3.

Groundworks (2004) estimated the capital cost of green roofs to be £74/m^2 (increased to 2010 value using 3.5% rate) or £7400 for a roof area of $100m^2$. Over a lifetime of 50 years with $100m^3$/yr attenuation the capital cost is £1.48/m^3. However, since maintenance for extensive green roofs is minimal this value is unlikely to be a significant underestimate. The value is similar to that predicted by *CWB*.

Kirk (2006) conducted a life cycle analysis of Best Management Practices (BMP) under evaluation at a BMP performance verification centre in New England, USA. The BMPs examined were sub-surface treatment and storage unit, retention pond, bioretention cell and subsurface-flow gravel wetland. Energy cost was not reported explicitly but as resource use (electricity, coal, natural gas etc.). Conversion of reported values yields the following annual results, respectively: 5315 kWh, 3548 kWh, 3769 kWh and 3261 kWh. By comparison, *CWB* predicts the annual energy use of a $294m^2$ retention pond as 3145 kWh, which is similar to the result produced by *Kirk* (2006), through detailed LCA, of a pond with the same surface area: 3548 kWh.

The costs of filterstrips, porous roads and paving, soakaways, swales and ponds are well documented in the CIRIA best practice guide (*Woods-Ballard et al.*, 2007) and this work was not repeated here.

6.6 Summary

Initially some test case studies were simulated in *Aquacycl*e and *CWB* to compare the water flow results. As expected there was general agreement, but also several notable differences. Some of these were a result of small conceptual differences but other results produced by *Aquacycle* seemed to be wrong and are linked to irrigation and soil store parameters.

The processed climate data from the Winterbourne Climate station were found to agree with the effective rainfall off 250 mm/yr, suggested by *Knipe et al.* (1993).

Supply data were validated at the DMA and study area scales against historical data from Severn Trent. Two mixed land use DMAs were used. The first DMA predicted water demand to within 6% of the observed. For the second DMA, demand for "retail" had to be reduced and the final result was 31% higher than the observed. Study area demand was validated against the observed data for a three year period with a correlation coefficient of 0.4. Since leakage data were not provided pre-2008, the leakage factors for 2008 were applied to all years. This introduced an error that explains some of the deviation between observed and calculated flows.

Simulated flows from subcatchments 1 and 4 were validated, with correlation coefficients of 0.5 and 0.8 respectively, against flow monitoring data from Severn Trent.

Energy and cost validation was carried out by comparing values calculated by *CWB* for each water management option with equivalent, published studies. Rainwater harvesting and wastewater recycling were validated for cost and energy. The cost of borehole abstraction was predicted to be significantly greater than the cost proposed by *Baker* (2003). However, his proposed cost of £0.09/m^3 is unrealistically low. No studies were found on the life cycle energy use of borehole abstraction nor for either the cost or energy use of septic tanks. Green roof WLC was validated against an estimate by *Groundworks* (2004). The costs of filterstrips, porous roads and paving, soakaways, swales and ponds are well documented in the CIRIA best practice guide (*Woods-Ballard et al.*, 2007) and these values were adopted by *CWB*. However, there have been very few studies into the life cycle energy use of SUDS. A study by *Kirk* (2006) used full life cycle analysis to calculate energy use of a retention pond and *CWB* predicted a result within 12%.

7 - Sensitivity analysis

The model has been successfully calibrated and validated against data supplied by Severn Trent and the literature. *City Water Balance* has been developed to demonstrate quantified possibilities for improved water management at the city scale to decision makers. For this remit, the model has not been designed to produce highly accurate results at small spatial scales and sub-daily timesteps but only to predict general patterns at larger scales (time and space). However, it is important to understand which input parameters have the most influence on the results and therefore contribute most to uncertainty. To this end a basic sensitivity analysis of key water flow, energy and cost parameters has been undertaken.

Important parameters that govern runoff volumes are the proportions of impervious and pervious space. Increasing the roof area of all MCs (where garden area is sufficient) by 10% resulted in a 7.1% (Fig. 7.1) increase in roof runoff. However, total study area runoff only increased by 2.0% and evaporation decreased by 1.0%.

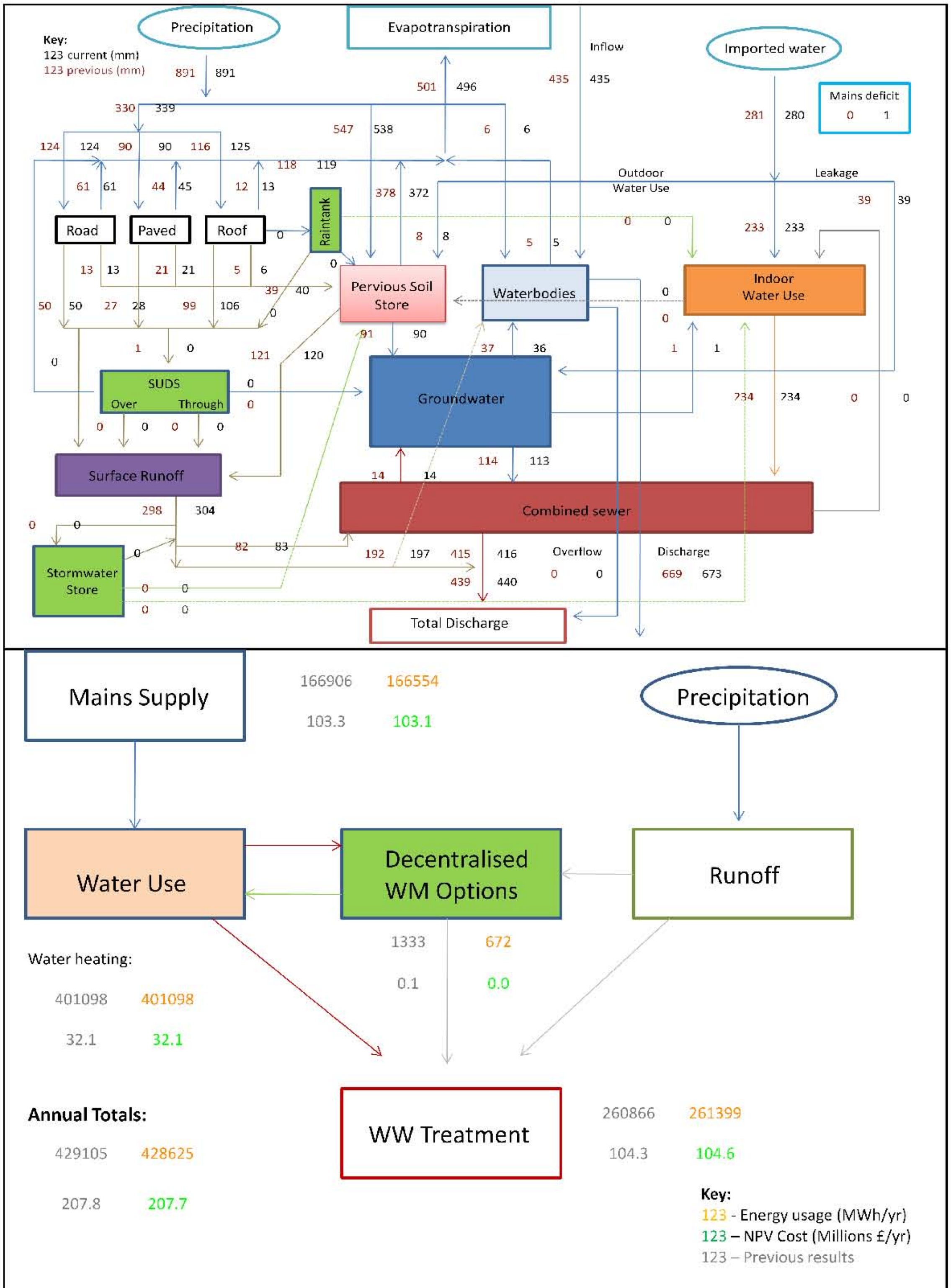


Figure 7.1 Base case 2055 (red – top; bottom – grey). Roof area increased by 10% and area subtracted from garden where possible (top – black; bottom – orange and green).

Water demand for different types of land use needed considerable calibration and it was found that it is quite sensitive to changes in occupancy. Increasing occupancy factor by 10% resulted in a 9.6% in water demand.

Testing of "proportion inflow" of stormwater into the wastewater sewers showed that total volume is not sensitive to it: when the proportion inflow from subcatchment 3 was increased from 0.1 to 0.25 (150%) the output volume increased from 457,600 m^3 to 497,300 m^3 (8.7%). However, the magnitude of the peak is sensitive, increasing from 13,600 m^3 to 24,300 m^3 (79%).

The effect of varying trigger-to-irrigate (TG) and proportion irrigated (Prop_I) on irrigation volumes was investigated. It was found that irrigation volume is very sensitive to changes in the value of TG. For example, an increase of 12% (from 0.34 to 0.38) resulted in a 161% increase in annual irrigation volume (from 1203 m^3 to 3136 m^3). Irrigation volume is also sensitive to the proportion irrigated but less than to TG. Increasing Prop_I by 150% (from 0.2 to 0.5) resulted in a 71% increase in annual irrigation volume (from 2700 m^3 to 4627 m^3).

The sensitivity of the results for various parameters related to sewer depth was tested (Table 7.1). Sewer depth was increased by 10% from 2 to 2.2 metres which resulted in a decrease in groundwater overflow of 2.4%, increase in recharge of 4.4%, increase in infiltration to sewers of 8.8% and a decrease in discharge to surface water bodies of 24.3%. Increasing the hydraulic conductivity of the sewers by 100% from 0.22 to 0.44 resulted in a decrease in groundwater overflow of 9.1%, increase in recharge of 13.0%, increase in infiltration to sewers of 41.6% and a decrease in discharge to surface water bodies of 75.0% (Table 7.2). Decreasing the sewer K by 100% had the reverse effect with less water infiltrating into the sewers and instead leaving the aquifer by other flow paths (groundwater overflow and discharge to waterbodies). Clearly, discharge to rivers is sensitive to sewer depth and sewer hydraulic conductivity.

Recharge was found to be quite sensitive to the field and residual capacity and not particularly sensitive to maximum daily drainage rate. An 80% reduction of the maximum daily drainage rate from 10mm/day to 2mm/day led to only a 16% reduction in annual recharge (from 388 mm to 325 mm). A 10% increase in field capacity resulted in a 3.3% reduction in recharge, 5.7% reduction in discharge to waterbodies and a 0.9% reduction in groundwater infiltration to sewers (Table 7.1). A 10% increase in residual soil moisture resulted in a 2.4% increase in recharge, negligible reduction in discharge to waterbodies and a 0.2% reduction in groundwater infiltration to sewers.

Table 7.1 Sensitivity of results to a 10% increase in various input parameters.

	Groundwater overflow (% change)	**Recharge from soils (% change)**	**Infiltration to sewers (% change)**	**Discharge to surface water bodies (% change)**
K sewers	-1.7	3.3	5.3	-10.8
Sewer depth	-2.4	4.4	8.8	-24.3
Field capacity	negligible	-3.3	-0.9	-5.7
Residual soil moisture	negligible	2.4	-0.2	negligible
K river bed	negligible	0.6	-0.7	negligible

Results were found to be sensitive to river bed hydraulic conductivity (Table 7.2). K ranges from 0.1m/day for unconsolidated silty deposits (*Freeze and Cherry*, 1979) to in excess of 1m/day for sandy-loam soil (*Binley et al.*, 2002). *Powell* (2000) used values of K between 1 and 3.5 m/day for sandstones. It was found that increasing K by 100% from 1 to 2m/day resulted in an increase in recharge to the aquifer from losing river sections that was greater than the increased base flow from gaining sections. The net result was a decrease in the total volume passing from the aquifer to waterbodies. When K was decreased by 100% from 1 to 0.5 m/day the opposite was the case: recharge from losing sections was reduced more than the base flow to gaining sections resulting in a net increase of flow from the aquifer, even though gross flows were significantly reduced.

Table 7.2 **Sensitivity of results to a 100% change in river bed and sewer pipe hydraulic conductivities.**

K (m/day)	**Groundwater overflow (% change)**	**Recharge from soils (% change)**	**Infiltration to sewers (% change)**	**Discharge to surface water bodies (% change)**
River bed				
From 1 to 0.5	-2.5	+3.2	-4.4	+22.2
From 1 to 2	+6.6	-8.7	+4.4	-30.1
Sewer				
From 0.22 to 0.11	+14.9	-19.6	-37.2	+52.8
From 0.22 to 0.44	-9.1	+13.0	+41.6	-75.0%

The contaminant concentrations in municipal sewage are well constrained so sensitivity analysis of input loads (from toilets for example) was not carried out for these. Water quality data for stormwater and combined sewers in the study area were not obtained so no sensitivity testing was possible for this.

The sensitivity of several parameters affecting life cycle energy use and cost were also tested using a two cubic metre, domestic, HDPE raintank supplying 60 m^3/yr for toilet and irrigation requirements, with no chemical or UV treatment. Increasing the pump efficiency by 10% from 0.3 to 0.33 resulted in a 0.7% increase in life cycle energy cost (Table 7.3). Increasing the tank volume by 10% resulted in a 1.3% increase in life cycle energy use. It can be seen that life cycle cost and energy use are not very sensitive to tank volume, pump efficiency and wall thickness.

Table 7.3 Sensitivity of life cycle energy and cost to a 10% increase in various input parameters.

Parameter	Change WLC (%)	Change life cycle energy use(%)
Tank volume	1.6	1.3
Pump eff	0.1	0.7
Wall thickness	NA (prices calculated per m^3 tank capacity)	2.2

In summary, runoff volumes were found to be relatively insensitive to changes in impervious and pervious proportions. Total runoff volume is not sensitive to "proportion inflow" of stormwater into wastewater sewers but peak runoff is sensitive. Water demand is sensitive to changes in occupancy. Irrigation results are sensitive to both TG and proportion irrigated. Recharge is moderately sensitive to the hydraulic conductivity of the sewers (K_{sew}), sewer depth, soil field capacity (F_{cap}) and residual soil moisture (R_{soil}). It is insensitive to the hydraulic conductivity of the river beds (K_{riv}). Groundwater overflow volume is mildly sensitive to K_{sew} and sewer depth but insensitive to F_{cap}, R_{soil} and K_{riv}. Infiltration to sewers is sensitive to K_{sew} and sewer depth but almost insensitive to F_{cap}, R_{soil} and K_{riv}. Discharge to surface water bodies is very sensitive to K_{sew} and sewer depth and moderately sensitive to F_{cap}. It is insensitive to changes in R_{soil} and K_{riv}. WLC and LC energy use are mildly sensitive to changes in tank volume but much less sensitive to changes in pump efficiency. WLC is insensitive to main tank wall thickness because calculations are not based on it.

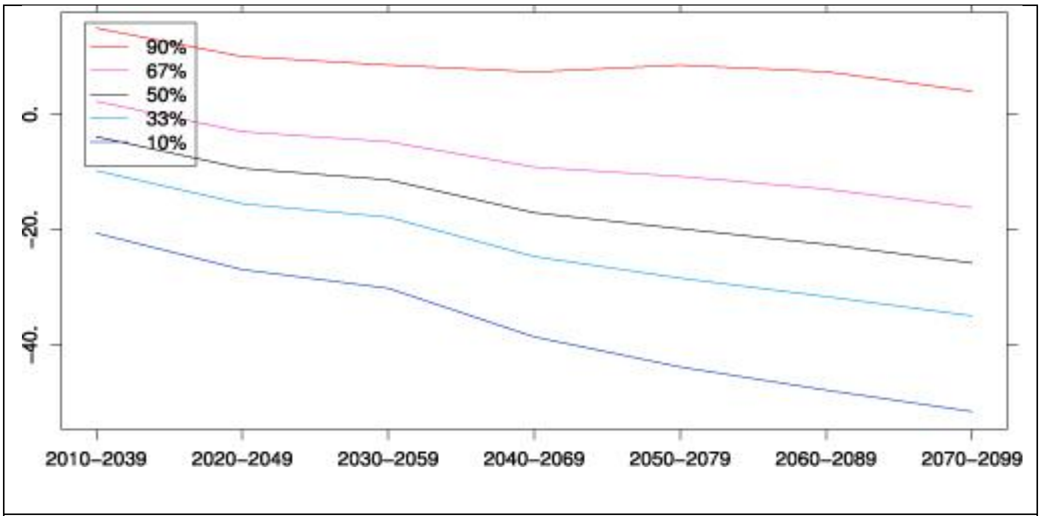
8 - Scenarios

This section includes a discussion of the calculation of daily precipitation and potential evaporation (PET) for 2055, projected demand change in Birmingham as well as three future scenarios proposed by the Birmingham Learning Alliance (BLA).

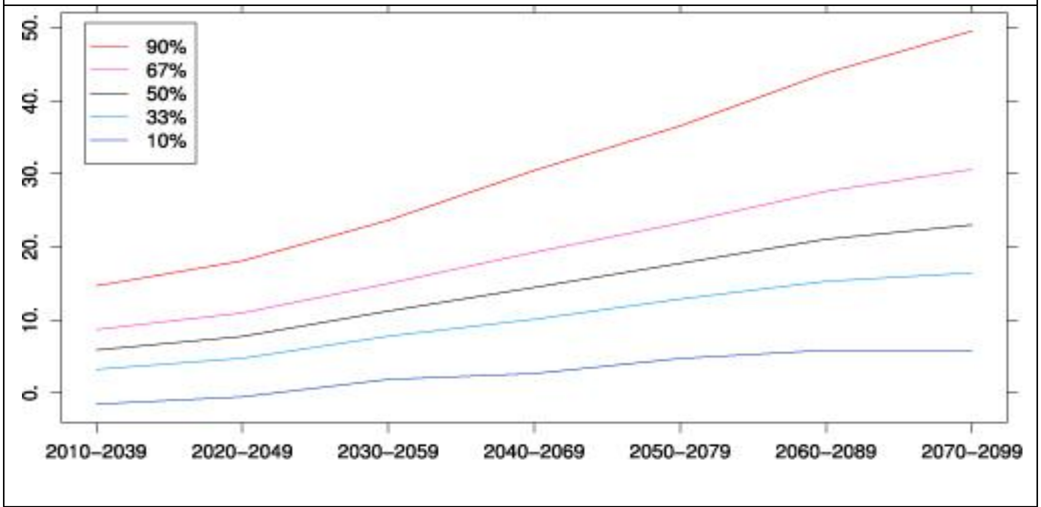
8.1 Climate change

Statistical downscaling was used to calculate the future climate for Birmingham. This technique uses change factors from global climate models and applies them to present day local climate to calculate possible future climates. The advantage of statistical downscaling is that it provides high resolution (spatial and temporal). However, it assumes that the statistics of the current climate will remain the same in the future which is unlikely to be the case (*UKCIP*, 2009a).

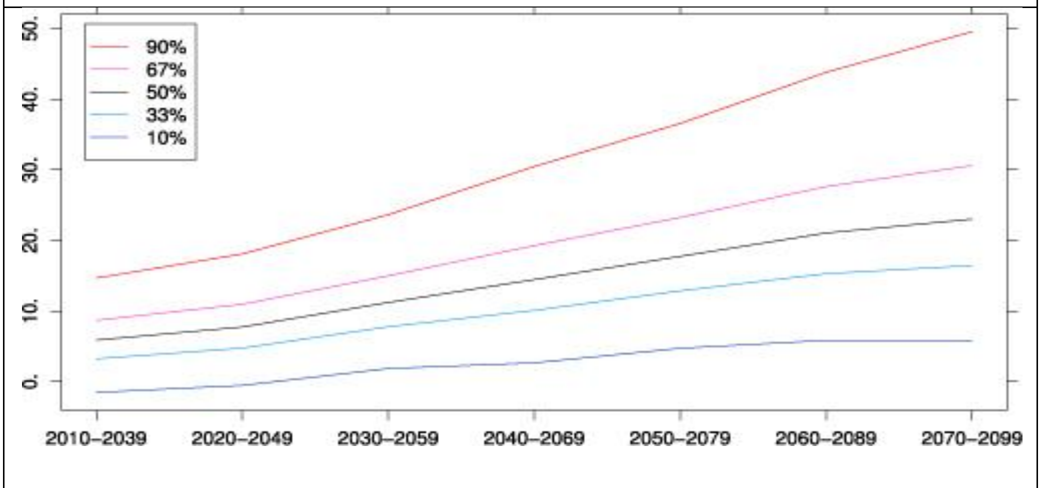
Figure 8.1 shows the percentage precipitation and mean temperature changes expected for the *IPCC*'s high emission scenario (A1FI) in the West Midlands, calculated by the *UKCP09* tool (*Met Office*, 2010). A description of the IPCC scenarios is summarised in Table 8.1. In these case studies the worst case scenario is used, so that the more extreme possible future climates could be modelled.



Estimate of % change in summer precipitation for the West Midlands by UKCP09



Estimate of % change in winter precipitation for the West Midlands by UKCP09



Estimate of % change in annual mean temperature for the West Midlands by UKCP09

Figure 8.1 Probability distribution graphs for high emissions scenarios. The curves show the maximum change in the climate parameter for a range of percent probabilities (*UKCP09,* 2010).

Table 8.1 IPCC Scenarios (*IPCC*, 2000).

A1 - **Rapid economic growth** - **Global population peaks mid-century and then declines** - **Rapid introduction of new, efficient technologies** - **Convergence among regions** - **Capacity building** - **Increased cultural and social interactions** - **Significant reduction in regional differences in per capita income** - **A1FI (fossil intensive), A1 T (non-fossil energy sources, A1B (balanced mix)**	**A2** - **Heterogeneous world** - **Self-reliance and preservation of local identities** - **Continuously increasing global population** - **Economic development is primarily regional** - **Slow, fragmented changes in per capita income and technology**
B1 - **Convergent world** - **Rapid change in economy towards service and information** - **Global population peaks mid-century and then declines (same as A1)** - **Introduction of clean, resource-efficient technologies** - **Emphasis on solutions for global sustainability, but without additional climate initiatives**	**B2** - **Emphasis on local solutions** - **Continuously increasing global population (at rate lower than A2)** - **Intermediate economic development** - **Less rapid, more diverse technology change than A1 and B1.**

The year 2055 was chosen for use in the scenario simulations. This represents the medium to long term future in which sufficient time has passed to allow significant changes to water management, climate and population to occur but not so far into the future that accuracy of the predicted changes suffers. The year 2055 is the mid-point of the period 2040-2069 output from the *UKCP09* model. Potential evaporation is dependent on the following primary variables: mean air (Tmean), maximum (Tmax), minimum (Tmin), wet and dry-bulb temperatures, wind speed and solar radiation. Consequently, the calculation of future PET requires knowledge of how each of these parameters will change from present-day. The *UKCP09* online tool provides data for future changes in precipitation, Tmean, Tmax, Tmin and Net downward solar radiation (R) monthly, averaged over thirty year periods. No data on changes in wind speed are available from *UKCP09* but *Entec* (2003) reports that the mean daily wind speed is expected to change by less than 1% by the 2050s. In the south eastern part of the West Midlands mean winter wind speed is expected to increase by up to 4% but summer mean wind speed is predicted to stay the same (± 1%). However, confidence levels in these predictions are low (*Entec*, 2003). No data on changes in wet/dry-bulb temperatures are available. Therefore, for the purposes of this calculation they are assumed to remain constant.

The changes in precipitation, Tmean, Tmax, Tmin and R were applied to 2009 climate data from the Winterbourne Climate Station. *CWB* is a daily timestep model so changes in rainfall intensities and frequencies are not considered, since they occur on a sub-daily timescale. Figure 8.2 shows the results of the climate change calculations.

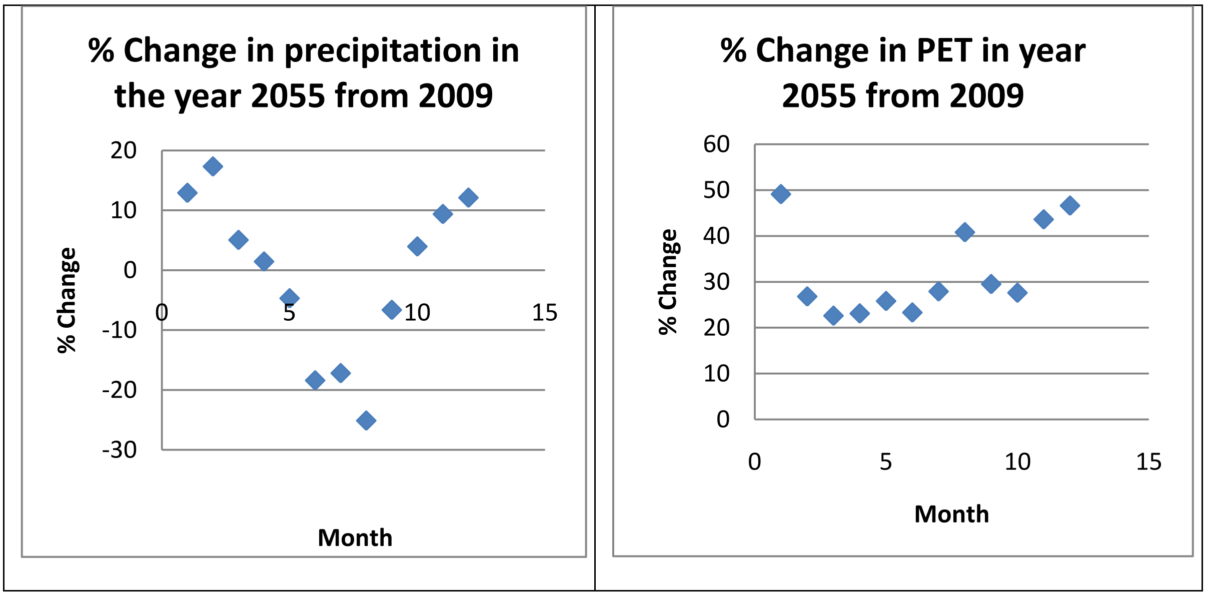


Figure 8.2 Expected change in precipitation and PET as a result of climate change using data from UKCP09 (period 2040 – 2069 – referred to 2055 in this case study) and the Winterbourne Climate Station, Birmingham.

These results are similar to those used by *Rueedi & Cronin* (2005) in their application of *UVQ* to Doncaster (Figure 8.3). Their data were sourced from *UKCIP* and *CEH*, but no further details on the assumptions involved were provided.

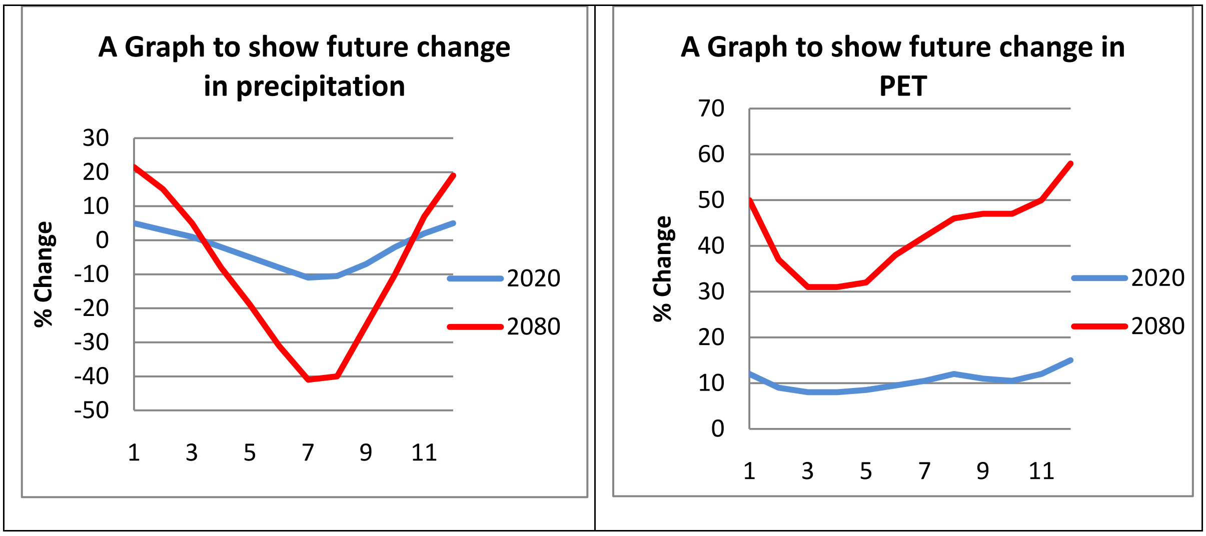


Figure 8.3 Expected change in precipitation and PET as a result of climate change (*Rueedi & Cronin*, 2005).

8.2 Demand change

Future demand changes in the UK have been estimated by the *Environment Agency* (2009b) in a report that outlines the EA's water management strategy to 2050. Key objectives of the strategy include:

- Reduce pressure on the environment caused by water taken for human use
- Encourage options resilient to climate change
- Reduce life cycle greenhouse gas emissions from people using water
- Safeguard water resources through effective catchment management, considering the interaction between quality and quantity
- Reduce treatment and energy costs for users

- Ensure water is used efficiently in homes and buildings, and by industry and agriculture
- Further reduce leakage
- Promote incentives to reduce demand

Forecasts of residential per capita demand are shown in Table 8.2.

Table 8.2 Forecasts of per capita consumption with different metering and water efficiency policies (*EA*, 2009)

Scenario	ID	Demand in 2030 (l/c/day)
Present day	-	150
Only new houses and those who opt to be metered	A	151
Near universal metering in seriously water stressed areas by 2030	B	146
Water company projected metering rates	C	143.5
Near universal metering in seriously water stressed areas by 2020	D	142.5
D + Internal water efficiency measures	E	132
D + E + External water efficiency measures	F	128
D + E + F + New homes built to a standard of 110 l/c/d	G	121
D + E + F + New homes built to a standard of 85 l/c/d	H	116

Classes E and F agree with DEFRA's Future Water Strategy Report, which aims to reduce consumption to 130 l/c/day by 2030 (*Water UK*, 2009).

8.3 Scenarios from the Birmingham Learning Alliance

The Birmingham Learning Alliance (BLA) was set up by SWITCH to encourage communication between stakeholders in the water industry and has been facilitated by meetings organised by a SWITCH appointed coordinator. Three future scenarios for Birmingham in 2050 were proposed by the BLA, with input by representatives from Arup, UKWIR, the University of Birmingham, the University of Middlesex, the Environment

Agency, British Waterways, and the Consumer Council for Water. The three scenarios are outlined below.

8.3.1 Scenario 1

The urban heat island effect has caused the maximum annual temperature to rise by 10 °C, resulting in air conditioning in nearly every building. The energy crisis has hit and there is limited availability of fossil fuels. Increasing temperatures across the UK has caused a rapid increase in the population of Birmingham, which has doubled to approx. 2 million by 2050, due to migration from the South-East. As a result of the large population rise, there is increasing densification/urbanisation of the city. Building controls have been lax, and legislation surrounding water and environmental management is not cohesive. Many of the developments are inefficient in terms of water and energy use, but approximately 30% have brown roofs. The water requirements of an increased population have not been met by improved capacity of the existing Victorian infrastructure, which is now close to collapse. Water and wastewater is pumped large distances but, due to energy rationing, there is often interrupted supply (no water supply for four hours per day on a rota system between districts).

Modelling implications:

- Increased occupancy factor in city centre from 300 to 600. Double residential occupancy factors from 2.1 to 4.2. Since much of central Birmingham has recently been regenerated (*Birmingham City Council*, 2008) it may reasonably be assumed that there will not be significant further work undertaken in this area before 2050. If it is also assumed that it is unlikely that large areas of existing housing stock will be regenerated then the most probable consequence of increased population is greater household occupancy, forced by rising house prices.

- 30% of study area has brown roofs. The total roof area in the study area is 3,680 hectares and 30% of this is 1104 ha. Application of green roofs is more economical the larger the roof (*Groundwork*, 2004) and so is more likely to be applied to industrial and commercial landuses than residential. In addition it is required that roofs have a slope of less than 15 degrees (*Daywater*, n.d.b) which renders many residential properties unsuitable. The sum of all non-residential roof areas is 1,390 ha. It is assumed that brown roofs have been applied to this sector.
- Increase leakage to 35% from 25%.
- Increase energy cost of centralised supply by 50% from 2.1 (*Water UK*, 2009) to 3.25 kWh. Increase cost by 50% from £1.3/m^3 to £1.95/m^3 at net present value.
- Reduce mains supply to 90% of demand, representing interrupted supply. Many commercial and industrial developments have some degree of water storage capacity and so are unlikely to be affected by supply interruption. In addition, many of the regeneration developments have on-site water storage capacity because the current network is unable to meet projected peak demand (*Birmingham City Council*, 2008). However, new build residential stock outside the city centre have been built without water tanks and so are reliant on mains pressure. These will be affected by the interruptions and it is assumed that a conservative 10% of total demand will not be met.

8.3.2 Scenario 2

The population has doubled to nearly 2 million inhabitants resulting in high density housing and increased urbanisation. The supply of fossil fuels is running low. After the recession of 2008/2009 the economy boomed, but there was no planning or investment in water management or energy supplies. Much of the water network is still based around the Victorian infrastructure and the city relies on conventional energy supplies. Water has become increasingly scarce, potable water is imported from Scotland and Wales allowing 80 l/p/d, but there are rumours of the supply being restricted to 50 l/p/d. Therefore the city is now looking at recycled water to provide 50 l/p/d of additional non-potable supplies. Flash flooding has caused storm tanks to fill up too quickly and overflows are allowed to discharge freely to receiving watercourses adversely affecting aquatic systems.

Modelling implications:

- Increased occupancy factor in city centre from 300 to 600. Double residential occupancy factors from 2.1 to 4.2.
- Increase leakage to 35% from 25%.
- Increase energy cost of centralised supply by 50% from 2.1 (*Water UK*, 2009) to 3.25 kWh. Increase cost by 50% from £1.3/m^3 to £1.95/m^3 at net present value.
- Reduce mains supply to equivalent of 80l/c/day and 50l/c/day (reduce available mains supply to 80/150 of total demand, where 150 is the present day per capita daily demand. Reduce residential demand to 130 l/c/day in accordance with DEFRA's Future Water strategy (*Water UK*, 2009).

- Supply deficit with combination of rainwater harvesting, greywater re-use and borehole abstraction.

8.3.3 Scenario 3

After the 2008/2009 recession, the economy never fully recovered and Britain is still paying off the debt incurred. There was an energy and water crisis in 2015, but the radical shake up over the course of the following 5 years resulted in the UK becoming more self sufficient. The population of Birmingham has not significantly increased above the 2010 level of approximately 1 million. This is in agreement with the report produced by *Entec* (2003) on “The Potential Impacts of Climate Change in the West Midlands” which estimates the population of Birmingham will not increase by more than 100,000 by 2025. In 2020 Birmingham went through a green revolution and became a water sensitive city. Potable water was removed from the supply chain and is now delivered to offices and private dwellings in a similar way to "water at work". The piped supply is provided to a "good" standard and is suitable for washing, clothes washing etc but not drinking. As a result of a widespread education programme to increase awareness of individual responsibility with regard to sustainability, communities have become more proactive in their approach to environmental management. There is a single agency responsible for water management in the city and it is adequately funded. Privatised water companies have been dissolved and there is now a government agency responsible for water/wastewater management in the UK. The UK now produces 80% of its own food, and Birmingham meets its targets through a mixture of rooftop gardens, allotments and common green space planted with fruits and vegetables which residents maintain and use.

Modelling implications:

- Occupancy factors stay constant
- Leakage stays constant
- If 60% of the energy cost of centralised water supply is due to pumping (*Tarantini and Ferri*, 2001; *Friedrich et al.*, 2006) then the remaining 40% will be reduced as a consequence of lowered treatment standards for centralised supply. If this is assumed to be halved, a reduction in the energy cost of centralised supply from 2.15 kWh/m^3 (*Water UK*, 2009c) to 1.72 kWh/m^3 is achieved. Similarly the economic cost is reduced from £1.3/m^3 to £1.04/m^3 NPV. Cost of wastewater treatment remains the same at 2.15 kWh (*Water UK*, 2009).
- All commercial and industrial areas have brown roofs. 30% of residential areas have green roofs.
- Increase irrigation of POS to 70% but not supplied with mains water. Irrigation demand satisfied with MC-scale greywater re-use.

9 – Results and Discussion

Various strategies were applied, using *CWB*, to the three scenarios proposed by the Birmingham Learning Alliance. Key sustainability indicators of water supply, wastewater emissions, energy use and cost are reported and discussed for each strategy (water quality results are shown in Appendix 2). This is followed by a discussion of the main problems encountered in the model development.

9.1 Base Cases

9.1.1 2009 Base Case

For comparative purposes business-as-usual (centralised supply and wastewater treatment) was simulated for the year 2009 (Fig. 9.1). Note that decentralised alternative water management (WM) energy use and cost is non-zero because borehole abstraction is currently (2010) used to supply a number of commercial establishments in the study area.

9.1.2 2055 Base Case

Climate data for 2055 were calculated using percentage changes of temperature and precipitation, from current climate, predicted by the UKCP09 model (*Met Office,* 2010). Data from 2009 were used as the base year for extrapolation to 2055 since this is the most recent full year of climate data. This year was slightly wetter (914 mm precipitation) than the average for the period 2000-2008 (877 mm) (*University of Birmingham Climate and Atmospheric Research Group*, 2007). If a drier year had been used as a basis for calculation of the 2055 climate then less runoff and recharge would be expected and the effectiveness of rainwater harvesting strategies would be reduced for example. In addition irrigation demand would increase. Apart from the change in climate no other data were altered from the 2009 simulation.

The climate change scenario affected several key parameters (Fig. 9.1). Annual recharge depths from the pervious stores were 216 mm (2009) and 182 mm (2055), which is a 16 % reduction. Note that this is approximately double the values for recharge seen in the figure, which are depths averaged over the study area rather than the aquifer. The Environment Agency predicts long term average recharge to fall by 9% by 2025 (*EA*, 2009). In their application of *UVQ* to Doncaster, *Rueedi and Cronin* (2005) found that recharge was reduced by 20 mm/yr or 10% by 2020 and by 70 mm/yr or 30% by 2080. The results from *CWB* compare well with these predictions. To the nearest 0.1 metre, steady-state aquifer-averaged groundwater levels are predicted to fall by 0.5 m between 2009 and 2055 as a result of the reduction in recharge. Figure 2 shows the distribution of the change in groundwater level (GWL) across the aquifer. It can be seen that the groundwater level drops more in areas away from the river systems. The reason for this is that the reduced recharge lowers the flows to the rivers and reduces the hydraulic gradients. However, the reduction in gradient to the rivers has only a minor impact on water levels near the rivers because the river water levels are largely unchanged. The effect of reduced hydraulic gradients on groundwater levels is larger at distance from the discharge point; the river acts as the pivot point.

Surface runoff decreased by 8.3% (325mm to 298mm) as a result of decreased average precipitation, increased evapotranspiration (7.4%) and a decrease in groundwater overflow as a result of slightly lowered groundwater levels. Irrigation demand doubled from 4 mm/yr to 8 mm/yr as a result of increased evaporation from the soil stores. In reality garden irrigation is usually not applied based solely on soil moisture deficit, as modelled in *CWB*, but depends on the behaviour of the gardener, which is less easily quantifiable. Their decision to irrigate is affected by perceived plant water requirements, desired garden condition and affluence (*Mitchell et al.,* 2001). The increased temperatures predicted for 2055 may result in irrigation in excess of that predicted by the soil moisture deficit, as gardeners may perceive a greater need for irrigation based on increased temperatures.

The base case climate change scenario shows changes in urban water flows of the order of 10-20% from current climate, with reduction in runoff, recharge and precipitation and increased evaporation and irrigation demand. The sections that follow examine the effects on the water system of various future scenarios that include climate change, and investigate the sustainability of various water management strategies applied to the scenarios.

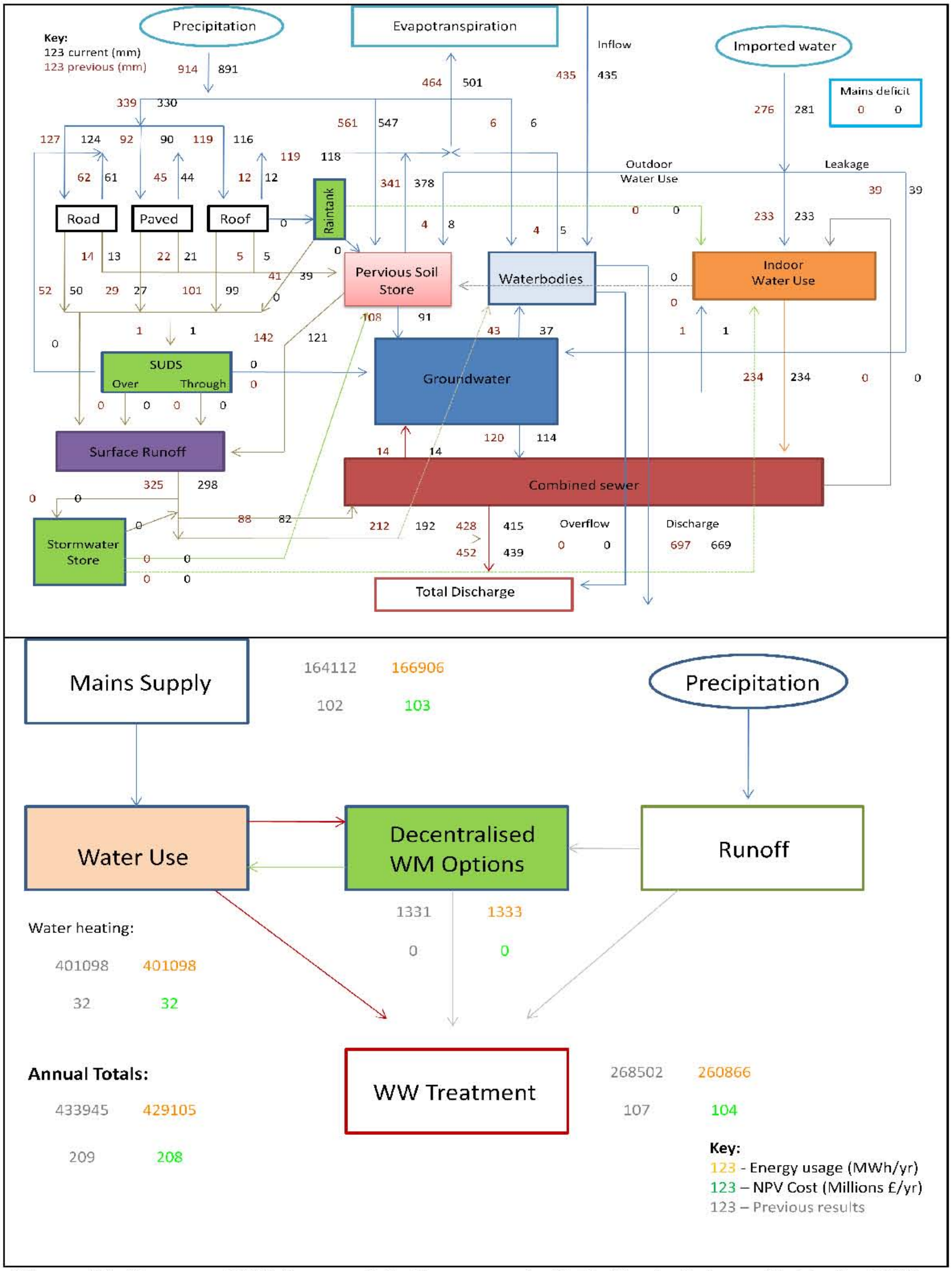


Figure 9.1 Base case 2009 (top – red; bottom – grey). 2055 climate data applied to the 2009 Birmingham model (top – black; bottom – orange and green).

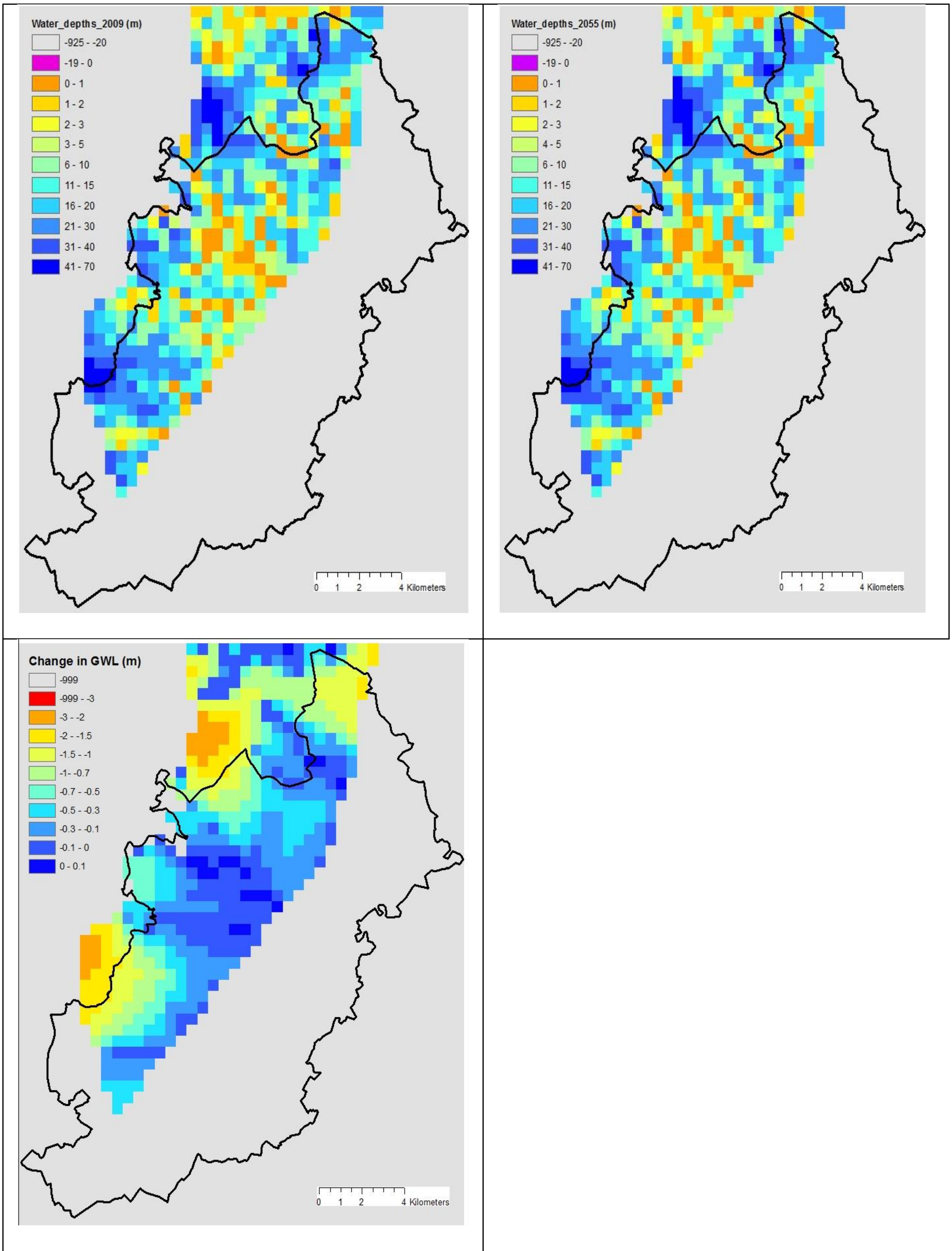


Figure 9.2 Steady state groundwater depths for the 2009 base case (top left) and 2055 base case (top right). Change in GWL from 2009 base case to 2055 base case (bottom).

9.2 Scenario 1

City centre and residential occupancy doubled since 2009. Mains leakage increased to 35%. The cost (energy and economic) of centralised supply increased by 50% and only 90% of demand could be satisfied by mains supply. Residential demand was assumed to be 130 l/c/day (*Water UK*, 2009b; *EA*, 2009). Climate data for 2055 were used.

9.2.1 Business-as-Usual

Supply and drainage remain centralised. Figure 9.3 compares the results for the 2055 base case with Scenario 1 business as usual for water flow, cost and energy. The predicted doubling of population to 2 million, for this scenario, results in a large increase in mains water demand, with consequent leakage to the aquifer and increased wastewater volumes. The additional recharge from leakage results in only a small increase in the average groundwater levels (0.7m) because it is accommodated by increased groundwater overflow and infiltration to the sewer system in cells that already had high groundwater levels. The greatest rise in GWL occurs in the southwest part of the aquifer (Fig. 9.4).

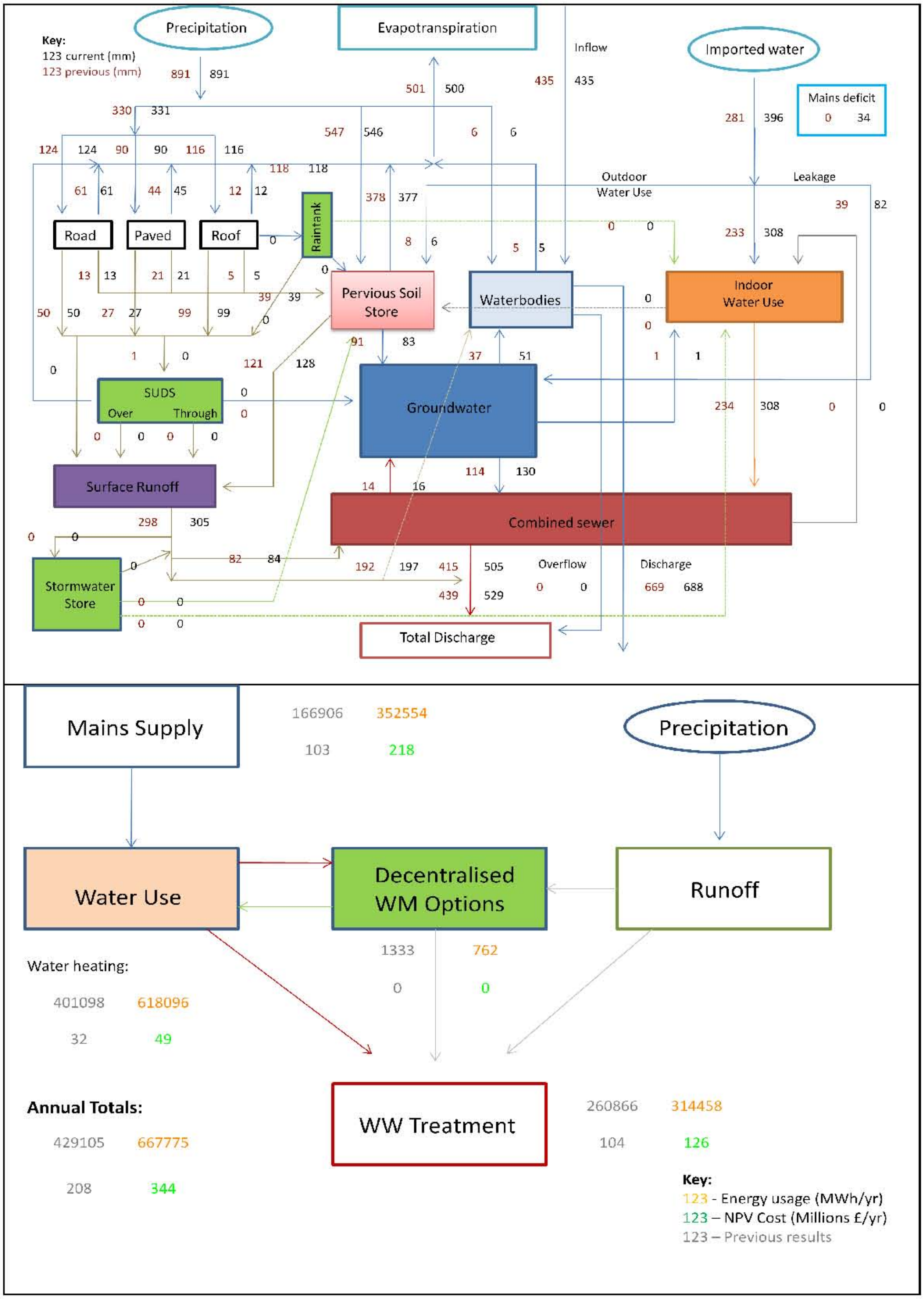


Figure 9.3 Base case 2055 (top – red; bottom – grey). Scenario 1 –Business as usual (top – black; bottom – orange and green).

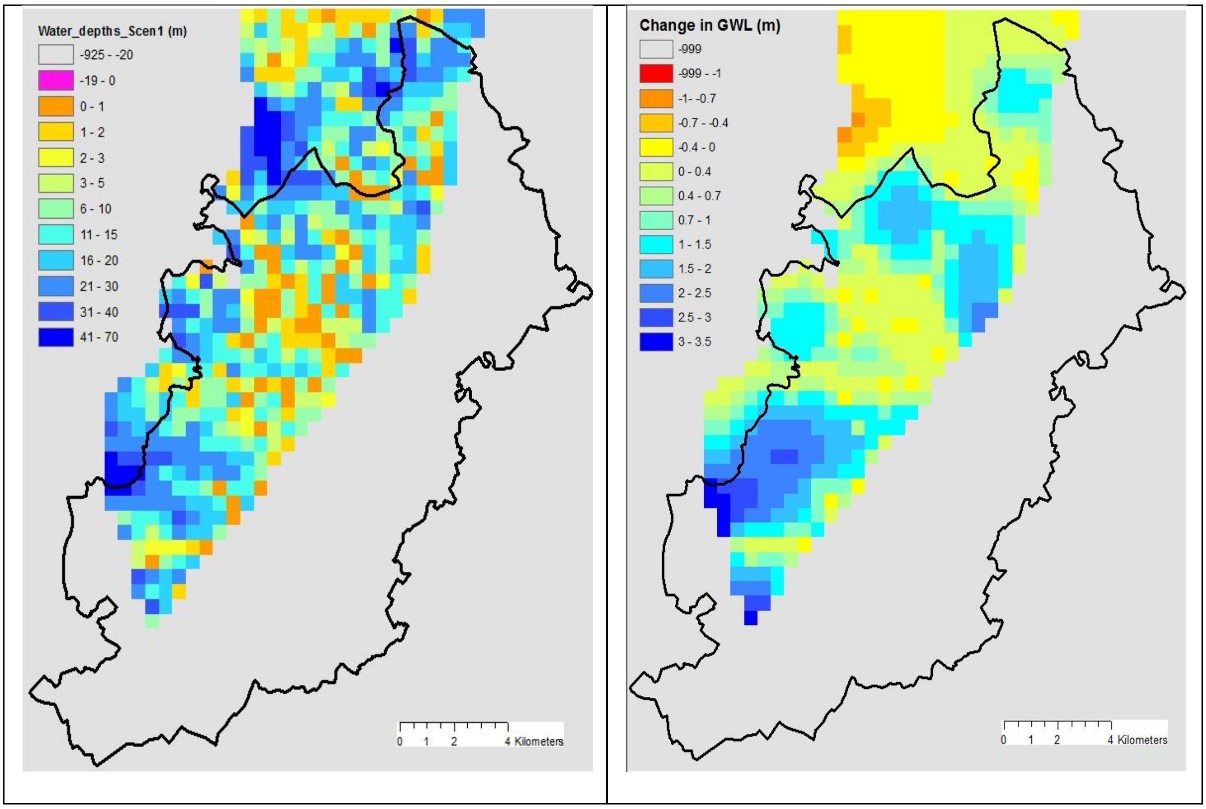


Figure 9.4 Steady state groundwater depths for the 2055 Scenario 1 business as usual (left). Change in GWL from 2055 base case to Scenario 1 business as usual (right).

9.2.2 Strategy - Non-residential Brown Roofs

Introduction of brown roofs (50 year lifetime) to all non-residential buildings increases the annual energy usage by 10,000 MWh (1.5%) and the annual cost by £7 million (2.0%) (Fig. 9.5). The cost of the brown roofs is partially offset by savings in stormwater treatment since the annual runoff decreased from 304 mm to 287 mm and runoff loads are reduced by 24 kg Total Suspended Solids (TSS)/Ha (Appendix 2). Groundwater depths for the brown roof strategy are very similar to the base case, as the recharge is unaffected by the introduction of brown roofs, and are therefore not shown here.

Slow release draindown from the brown roofs is 16 mm, which is a similar quantity to the reduction in surface water runoff. Actual savings for avoided conventional wastewater treatment, as a result of stormwater retention by the brown roofs, are estimated to be £2 million and 4,000 MWh. If it is assumed that slow release draindown could achieve similar savings, in terms of flood mitigation, then brown roofs become more attractive as a water management option. In addition, there are various other important benefits of brown roofs that are more difficult to quantify and are not modelled by *CWB*. They provide habitats for insects and birds, have aesthetic value, provide additional insulation during winter and have a cooling effect during summer which helps to mitigate the urban heat island phenomenon.

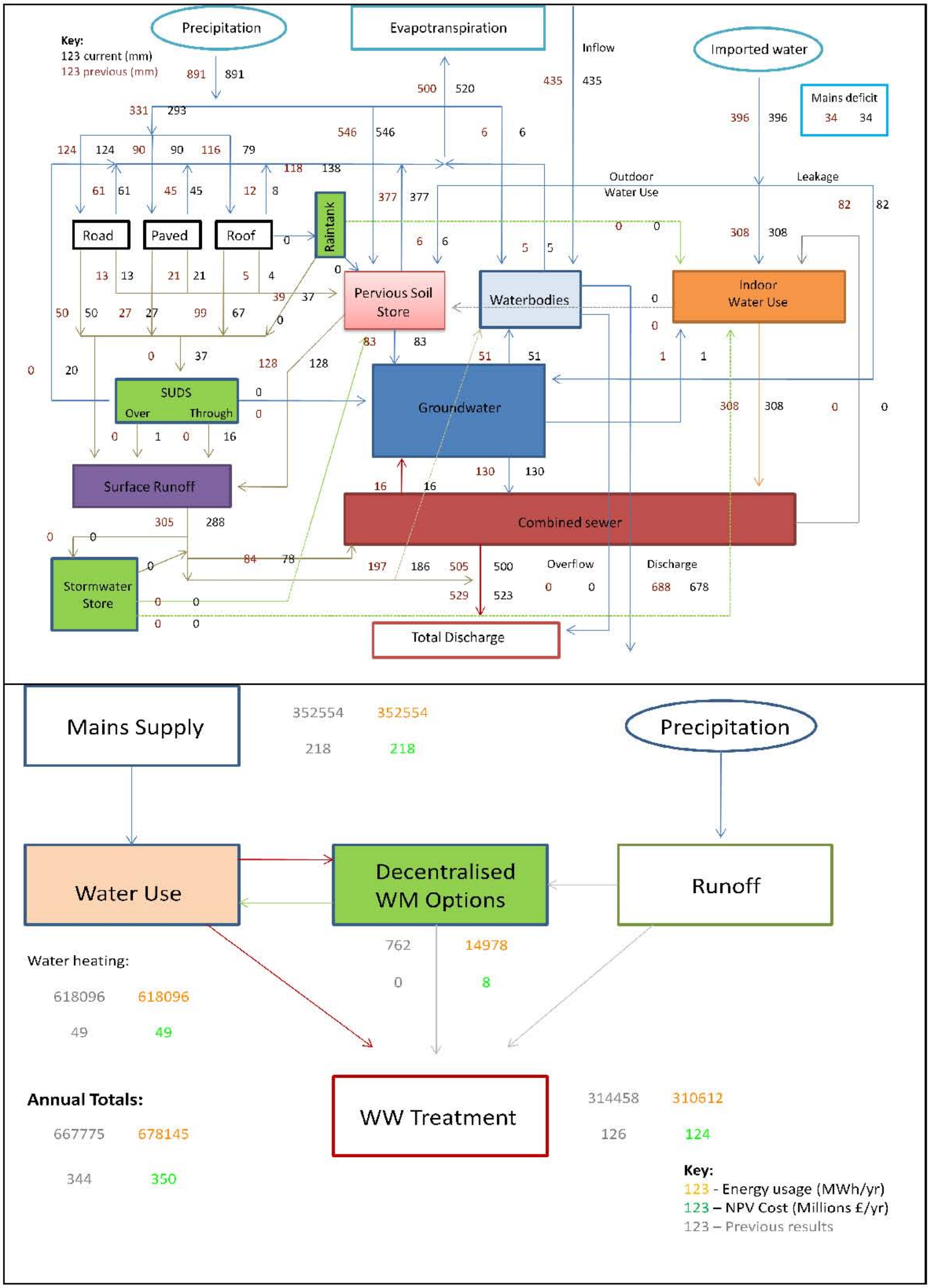


Figure 9.5 Scenario 1 –Business as usual (top – red; bottom – grey). Brown roofs (50 year lifetime) applied to all non-residential buildings (top – black; bottom – orange and green).

9.2.3 Strategy – Non-residential Brown Roofs and Minicluster Rainwater Harvesting

Brown roofs were applied to all non-residential buildings and rainwater harvesting (RH) (40m^3 concrete tanks with lifetime of 50 years; 5 toilets / unitblock) was installed for all industrial and commercial buildings (Fig. 9.6).

The use of raintanks to supply commercial and industrial miniclusters reduces the mains deficit by 9 mm. The life cycle annual cost for this supply, after deducting the cost of the brown roofs, is £3.3 million and 8,000 MWh, which is equivalent to £1.31/m^3 and 3.15 kWh/m^3.

Flow volumes for various stormwater store performance indicators are shown in Table 9.1. There is significant potential for greater rainwater use as 75% is lost to overflow and the demand is greater than the supply to the tank. Because miniclusters are diverse in size, uniformly increasing the tank capacity may mean that for smaller MCs the tank is oversized, resulting in poor lifetime cost and energy use results. It is expected that, by optimising store size, MC RH applied extensively across Birmingham would be a sustainable option. To address this issue a simple form of automatic optimisation was programmed; stormwater tanks were sized based on the assumption of 2 m^3 of storage per 100 m^2 of contributing impervious area.

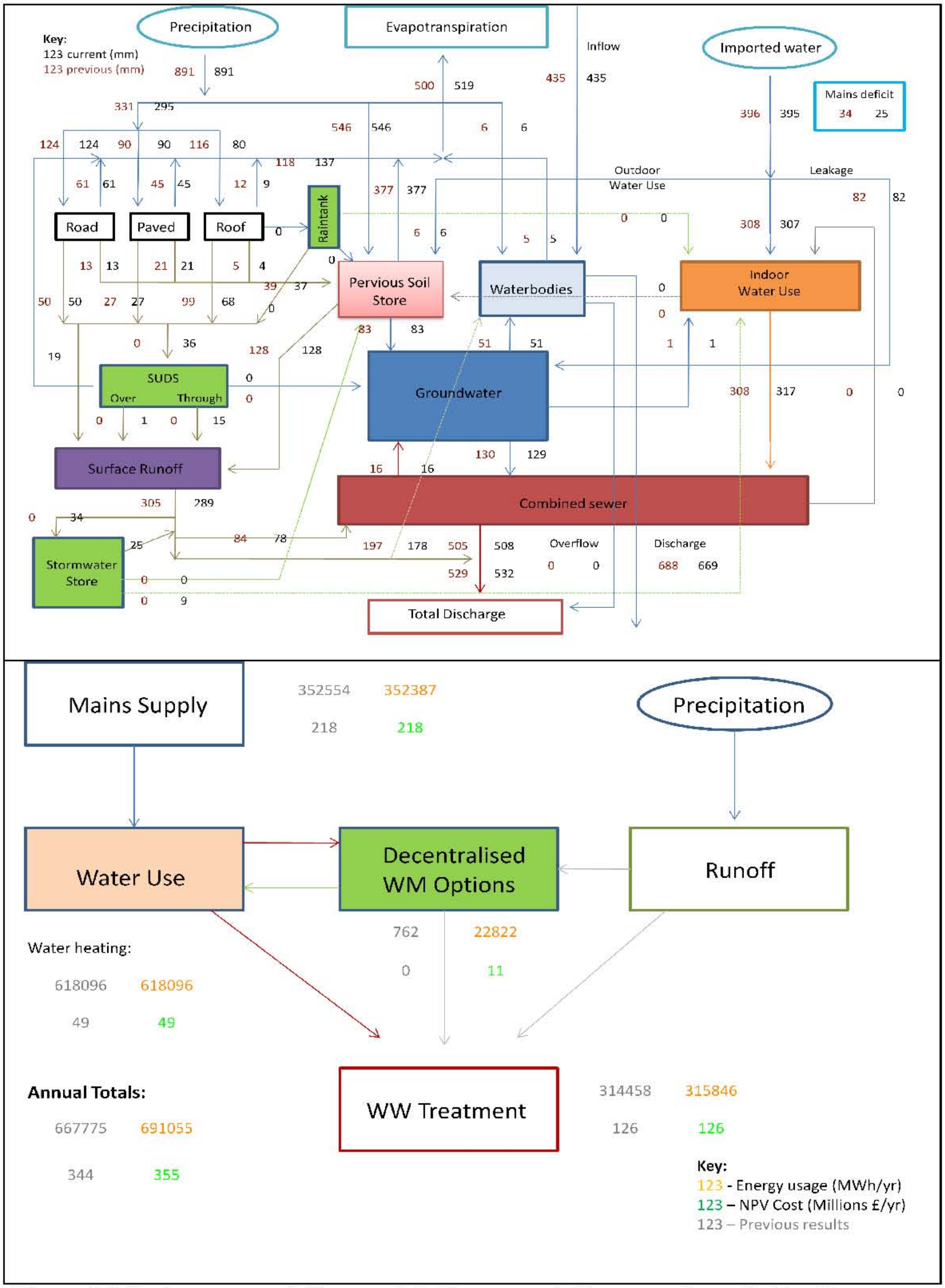


Figure 9.6 Business as usual (Top – red; bottom – grey). Brown roofs applied to all non-residential buildings and RH using 40m³ concrete tanks with lifetime of 50 years (top – black; bottom – orange and green).

The importance of sizing stormwater tanks appropriately is convincingly demonstrated in Table 9.1 and Figure 9.7 which show the improved water volume performance achieved and energy benefits. Supply volume is increased by 50% and energy use improves to 2.37 kWh/m^3. However, economic cost increases slightly by 3% to £1.35/m^3. After more detailed analysis of the results it is seen that this price increase, per unit volume supplied, is occurring for the larger tank sizes, which is an unexpected result since it is in contradiction to economies of scale. This is because the optimisation routine only takes into account contributing impervious area and not demand. So, for some miniclusters with large impervious areas and relatively small demand, such as depots, the tank was oversized, resulting in increased cost.

The assumption of 2 m^3 of storage per 100 m^2 of contributing impervious area was based on a residential configuration for the future scenario of occupancy 5 and 133 l/c/day demand, using the rainwater for toilet flushing and garden irrigation (approximately 35% of total demand), which is equivalent to 233 litres/household/day. For non-residential miniclusters, that have lower demands, there therefore needs to be a factor to reduce the tank size proportionally. By introducing a linear factor into the optimisation the results are improved to 2.32 kWh/m^3 and £1.15/m^3 which is significantly better than the fixed tank sizes for both energy and cost.

Table 9.1 Stormwater store performance for the study area (values in Mm^3).

	Supply to tank	**Overflow**	**Demand**	**Use**	**Deficit**
Fixed tank sizes	9.8	7.1	18.4	2.6	15.8
Tanks sized by CWB	9.8	5.5	18.4	4.0	14.3

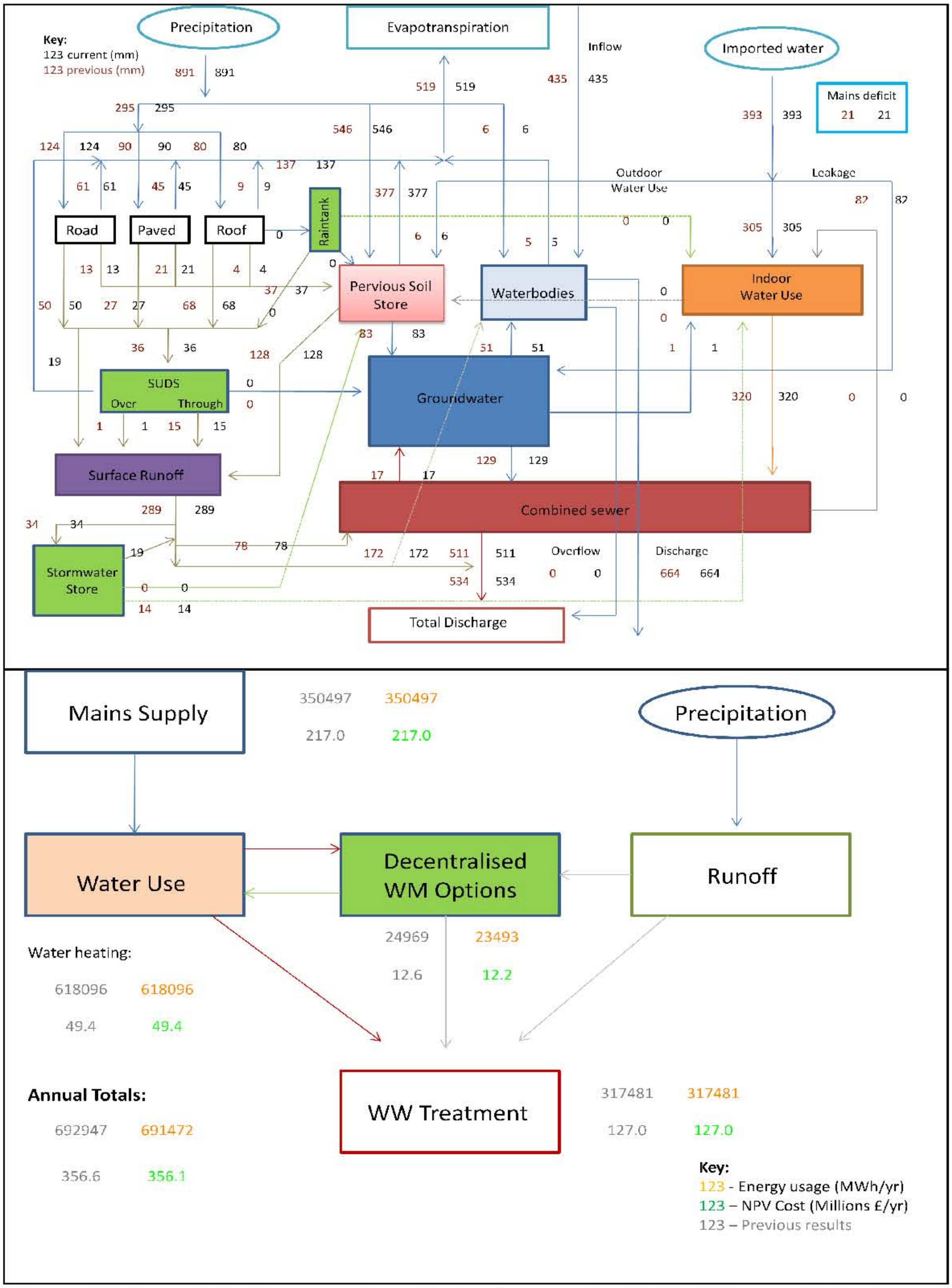


Figure 9.7 Scenario 1 – Brown roofs applied to all non-residential buildings and RH using concrete tanks of varying size with lifetimes of 25 years (top – red; bottom - grey) and 50 years (top – black; bottom - orange and green).

Figure 9.7 also shows the effect of varying the lifetime of the main tank on economic cost and energy use. The annual energy cost is reduced by 1,476 MWh (14.8%) and the economic cost by £0.46 million (7.9%). The differences are significant but perhaps not as large as expected. This is because the embodied energy of the extensive pipe systems required and unitblock cisterns are more dominant contributors, and these remain the same regardless of the size of the main tank.

Figure 9.8 shows the results using HDPE for the main tank material instead of concrete. It was found that, for this configuration, both materials have similar lifecycle cost and energy use. Although HDPE has a much higher embodied energy per unit mass than concrete (23,100 kWh/kg versus 300 kWh/kg) significantly less mass is required to make the tank, since HDPE is less dense (950 kg/m^3 versus 2000 kg/m^3) and can be used in thinner walls (0.01 m versus 0.07 m). For the HDPE tanks the effect of doubling the tank lifetime is similar to that for the concrete tanks: energy costs are reduced by 14.7% and economic cost by 9.3%.

Figure 9.9 shows the distribution of the stormwater stores and their tank capacities. If required, for decision-making purposes, further analysis of the performance of individual minicluster stores (supply to store, overflow, demand, usage, and deficit) could also be spatially displayed in *ArcGIS*.

In the calculation of the whole life cost and lifecycle energy use provision is made for the additional new pipework necessary for larger scale harvesting systems. So, it can be seen that, even with the requirement for extensive pipework, the installation of stormwater stores at MC scale is predicted to be more sustainable than future centralised supply, in terms of energy and cost.

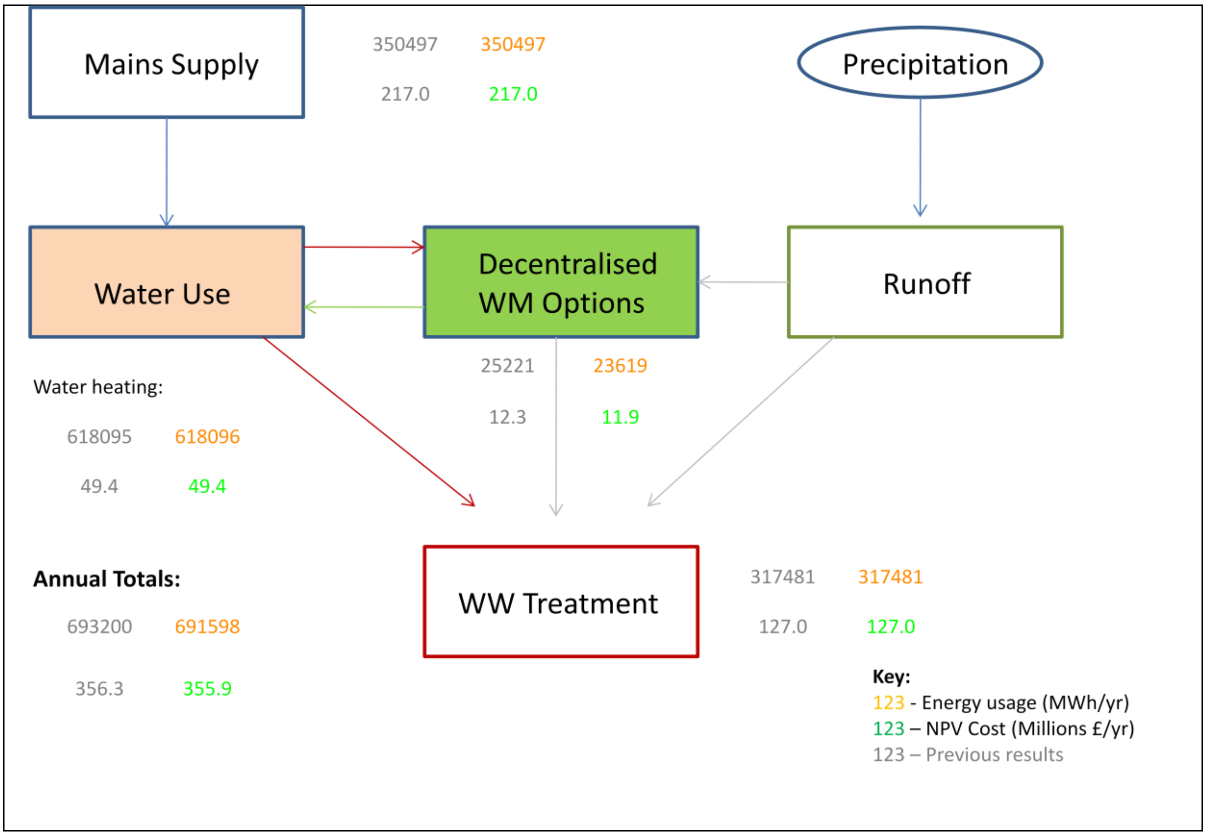


Figure 9.8 Scenario 1 – Brown roofs applied to all non-residential buildings and RH using HDPE tanks of varying size with lifetimes of 25 years (grey) and 50 years (orange and green).

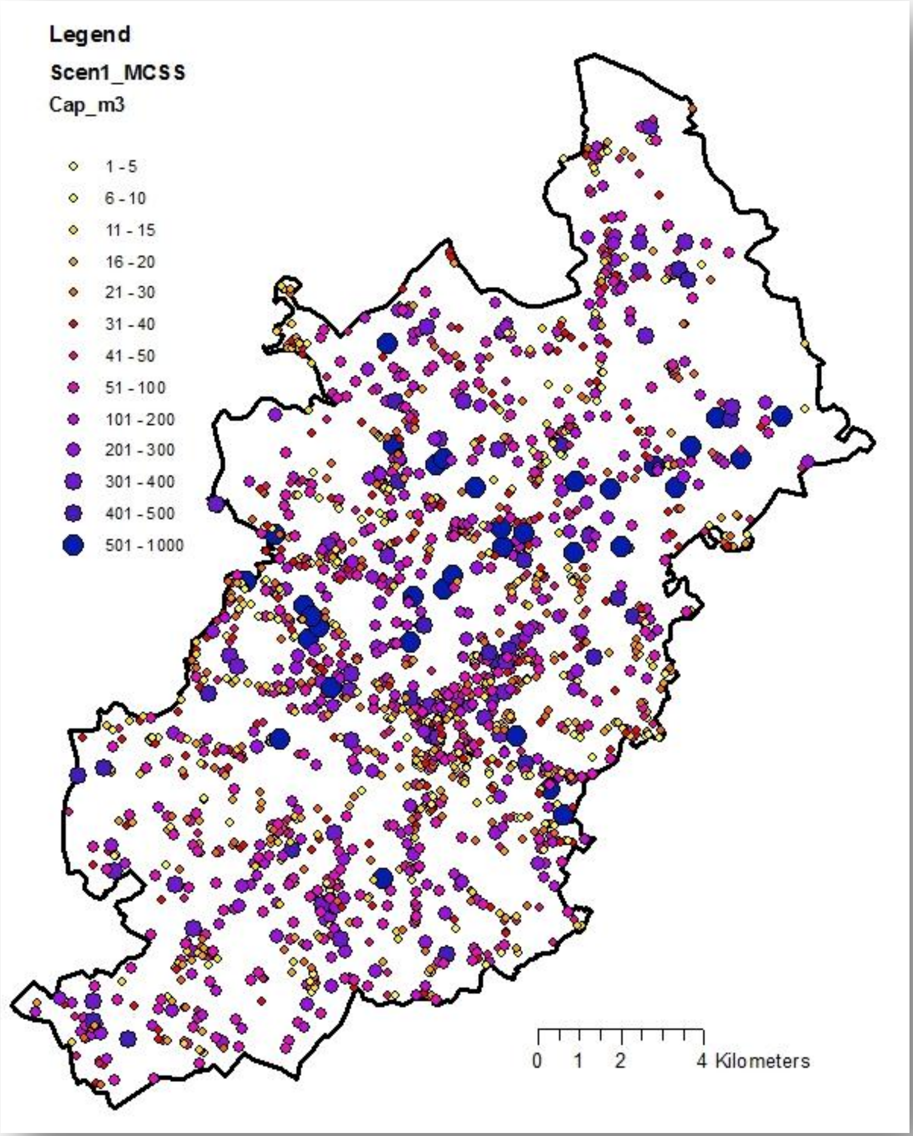


Figure 9.9 Scenario 1 MC stormwater tank capacities, sized within *CWB*.

9.3 Scenario 2

This scenario is the same as Scenario 1 but only 54% of water demand can be supplied by the mains instead of 90%. Strategies investigated in this scenario were supplementing the mains supply with combinations of rainwater harvesting, greywater re-use and borehole abstraction.

9.3.1 Business-as-Usual

Supply and sewerage remain centralised. Figure 9.10 compares the results for the 2055 base case with Scenario 2 business as usual for water flow, cost and energy. The business as usual for this scenario has a mains supply deficit of 162 mm as a result of failure of the conventional supply system to cope with the increased population. Although Scenario 2 business as usual has less imported water than the base case (the effect of population increase is outweighed by the increased water shortages), the increased cost of centralised supply resulted in an increase of 5.5% in annual lifecycle energy use and 9.6% in WL annual cost for the system. Various strategies were investigated to reduce the mains deficit.

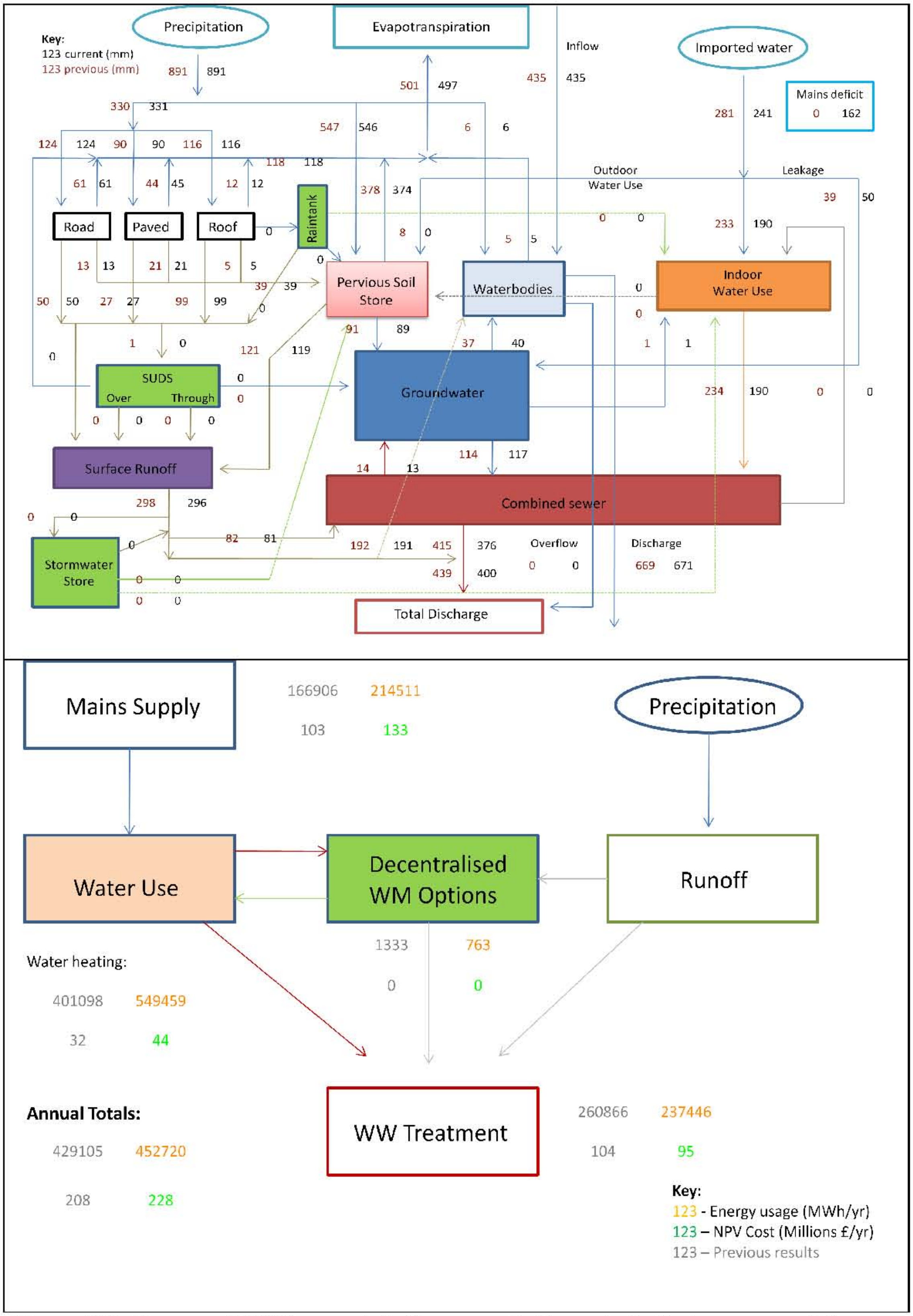


Figure 9.10 Base case 2055 (top – red; bottom – grey). Scenario 2 –Business as usual (top – black; bottom – orange and green).

9.3.2 Strategy – Minicluster Rainwater Harvesting

Using MC rainwater harvesting (tank life 50 years, size optimised), to supply indoor use 1, garden and POS irrigation demands, reduced the mains deficit to 101 mm from 162 mm (Fig. 9.11). The cost of this alternative supply is 53,900 MWh and £20.3 million. This is equivalent to supply costs of £1.18/m^3 and 3.13 kWh/m^3 which compares very favourably with the mains supply in terms of WLC (£1.95/m^3) and has similar life cycle energy use (3.15 kWh/m^3). Note that these costs are slightly more than those for Scenario 1 because in this scenario stormwater harvesting has been applied to the entire study area and not just commercial and industrial landuses, which have large roof areas and so are often better suited for RH applications. As can be seen from Table 9.2 there is significant potential to reduce the mains deficit by optimizing the tank size.

Rainwater harvesting at minicluster scale is predicted to be a viable alternative supply option, with the capability of reducing the mains deficit by at least 40%. The energy and cost figures have been calculated based on an efficient pump with a rating of 0.3 kWh/m^3 (*Hunt and Lombardi*, 2006). The simulation is repeated with a poorer pump rating of 1kWh/m^3 and the supply costs are found to be 3.83 kWh/m^3 and £1.20/m^3 which are still more economical than mains supply but slightly less energy efficient.

It is interesting to note the influence of economies of scale on the viability of stormwater harvesting. For a domestic system, with a 2m^3 concrete tank and 0.3 kWh/m^3 pump rating, the energy use and NPV cost, calculated by *CWB*, are 3.8 kWh/m^3 and £1.9/m^3 respectively (assuming annual use of 60m^3) and *Roebuck* (2008) estimated the cost of small scale rainwater harvesting systems to be in the range £0.29 – 2.32 /m^3, excluding capital costs. Example estimates of life cycle energy use for 25m^3 concrete tanks are 0.8 kWh/m^3 (*Mithraratne & Vale*, 2007) and 1.3 kWh/m^3 (*BlueScope Water*, n.d.). A wide range of pump ratings are used in literature, ranging from 0.3 – 3 kWh/m^3 (*Clarke et al.*, 2009; *CIRIA*, 2001; *Hunt & Lombardi*, 2006). However, even using the most efficient pumps from the range these values are quite low and it appears that life cycle averaged energy use of 2-3 kWh/m^3 is likely to be more achievable.

Table 9.2 Stormwater store performance for the study area under Scenario 2 (values in Mm^3). Note that the supply to the stormwater stores are much greater in this scenario than in Scenario 1 (Table 1) because all MCs have RH in this scenario, compared with only business and commercial MCs for Scenario 1.

	Supply to tank	Overflow	Demand	Use	Deficit
Fixed tank sizes	40.9	33.4	46.5	7.4	39.1
Tanks sized by CWB	40.9	23.0	46.4	17.2	29.2

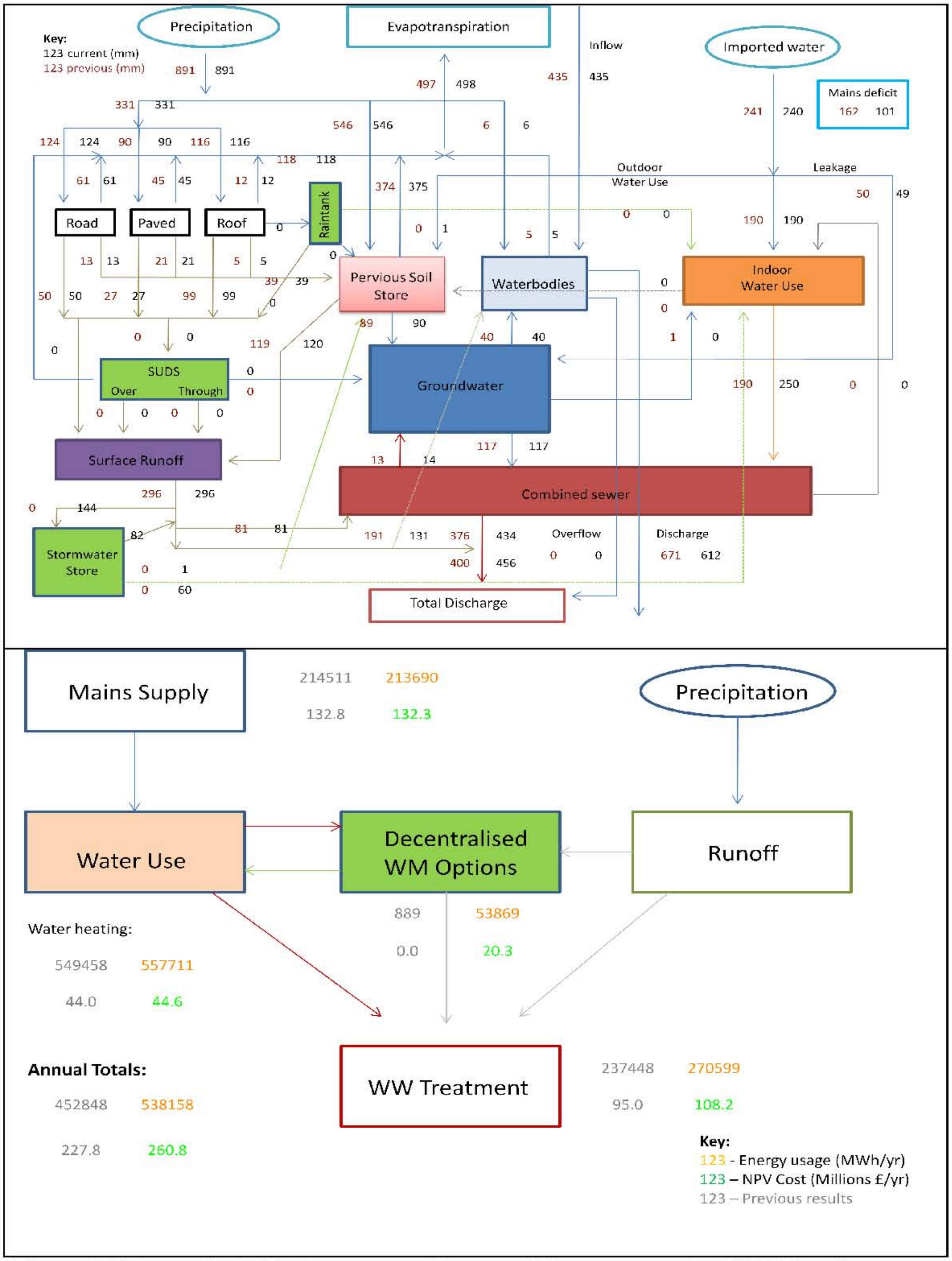


Figure 9.11 Scenario 2 – Business as usual (top – red; bottom – grey). MC RH in all miniclusters supplying indoor use 1, garden and POS irrigation demands. Tanks sized by CWB (top – black; bottom – orange and green). Note that the indoor waste volumes are greater than the mains supply volume because the supply is augmented by harvested stormwater.

9.3.3 Strategy – Minicluster Wastewater Recycling

Using fixed tank sizes MC WW recycling (tank life 50 years) supplied 55 mm for indoor 1 use and irrigation at an annual cost of 90,300 MWh and £19.6 million (Fig. 9.12). This is equivalent to supply costs of 5.62 kWh/m^3 and £1.22/m^3.

When tank sizes were subject to basic optimisation the supply from WW recycling increased to 147 mm (34.7 Mm^3) for indoor 1 use and irrigation at an annual cost of 181,300 MWh and £30.7 million (Fig. 9.13). This is equivalent to supply costs of 5.22 kWh/m^3 and £0.88/m^3, which is a significant improvement in economic performance. If savings made in avoided centralised wastewater treatment are included, the cost of MC WW recycling is reduced to 5.02 kWh/m^3 and £0.81/m^3. Clearly, there are considerable potential cost savings but energy use by the wastewater recycling strategy is 59% more than the conventional supply (3.15 kWh/m^3). Figure 9.14 shows the location and tank capacities of the MC WW stores.

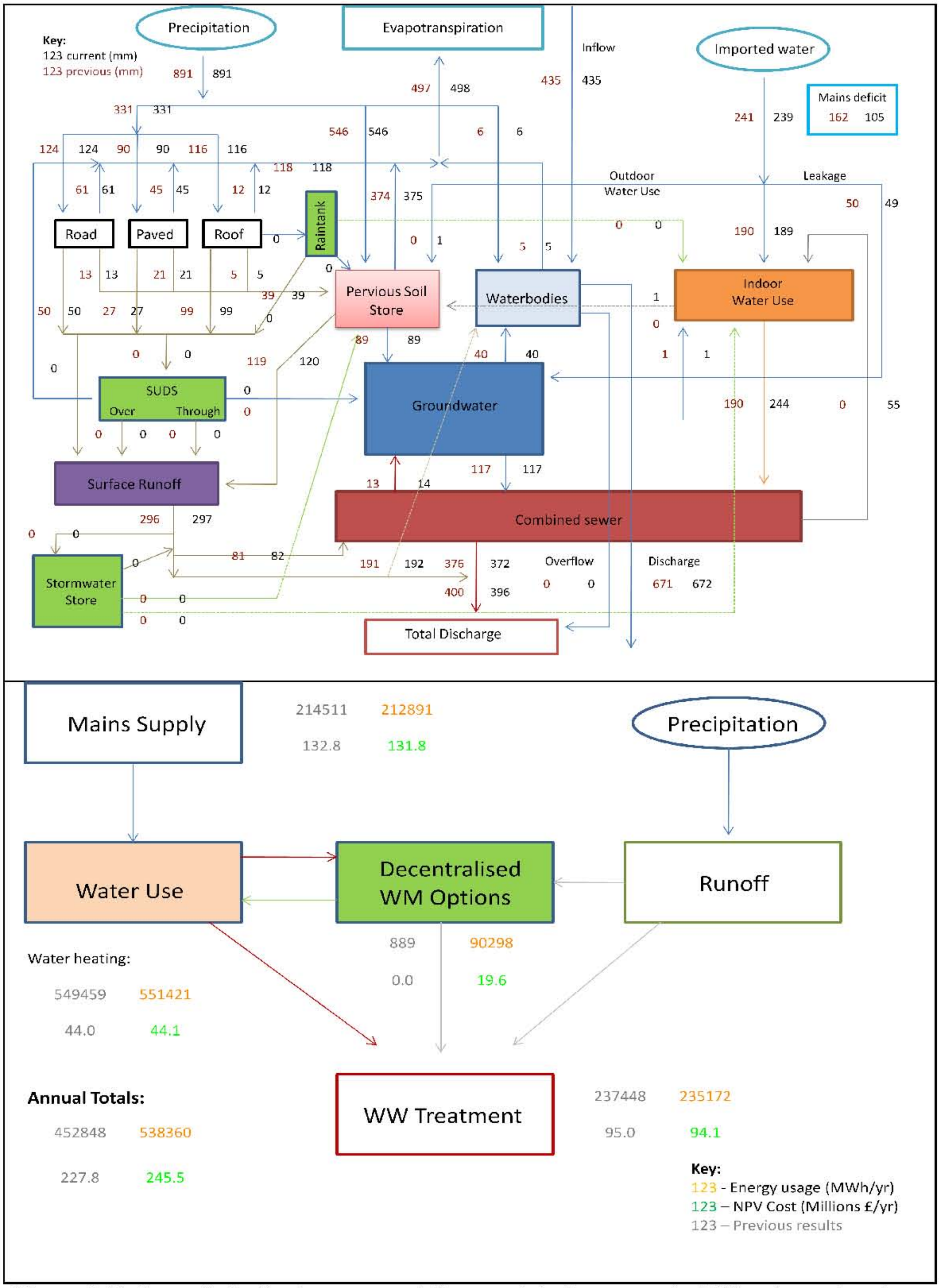


Figure 9.12 Scenario 2 - Business as usual (top – red; bottom – grey). MC scale wastewater recycling with fixed tank sizes across all miniclusters (top – black; bottom – orange and green).

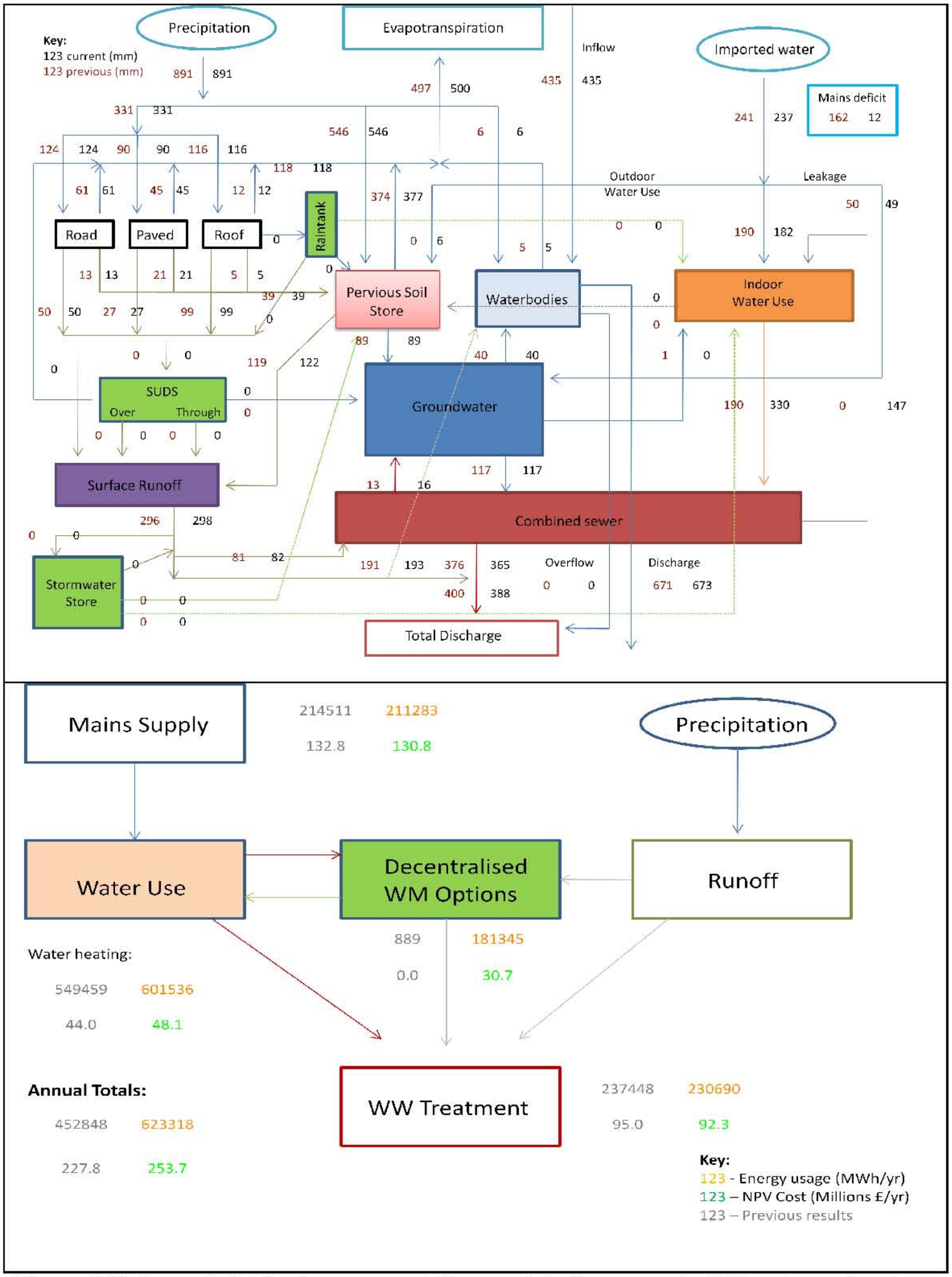


Figure 9.13 Scenario 2 - Business as usual (top – red; bottom – grey). MC scale wastewater recycling (WW) with varying tanks sizes estimated by *CWB* (top – black; bottom – orange and green).

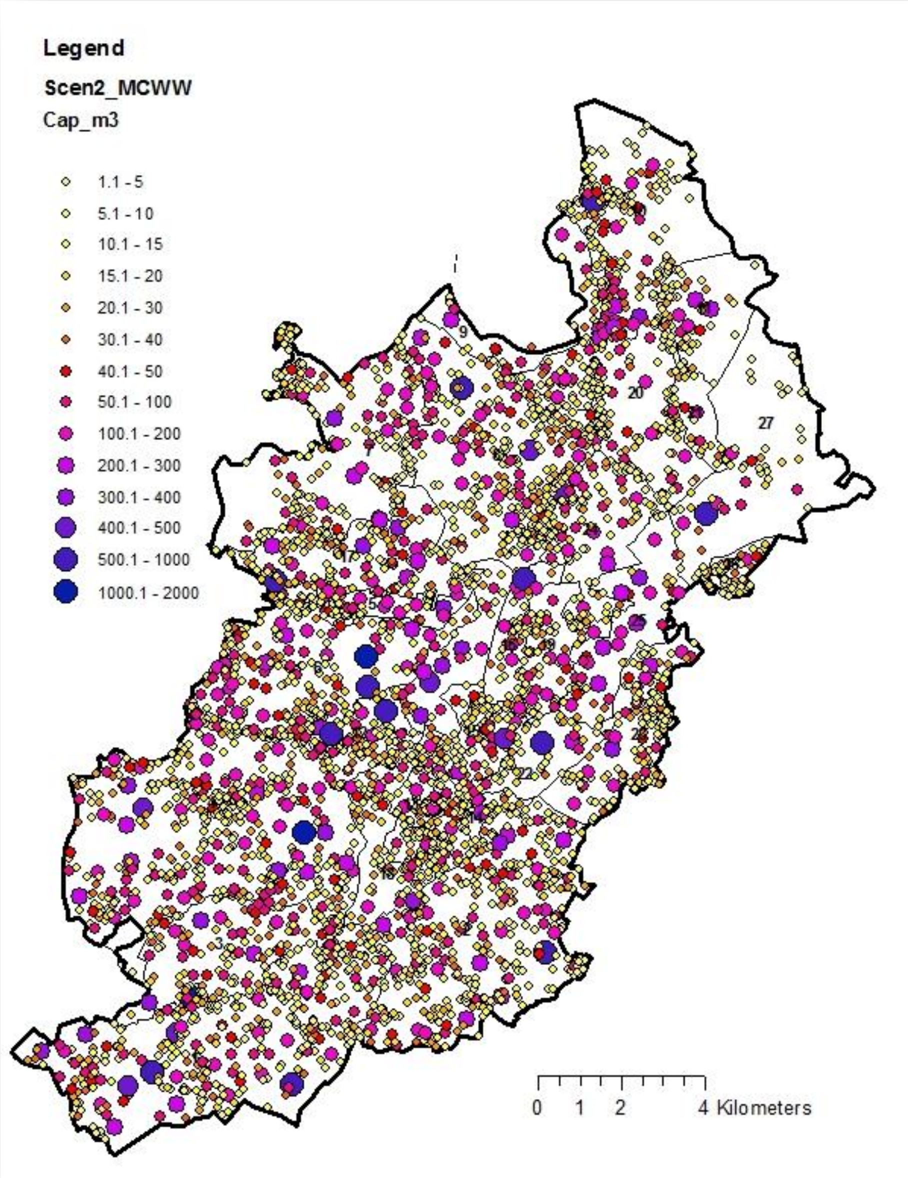


Figure 9.14 Scenario 2 MC Wastewater recycling store capacities, sized within *CWB.*

Table 9.3 demonstrates, using a breakdown of the water flows, the improved efficiency achieved by optimising tank sizes. Notice the greater supply to the tanks as a result of the increased levels of recycling.

Table 9.3 Wastewater recycling store performance for the study area under Scenario 2 (values in Mm^3).

	Supply to tank	Overflow	Demand	Use	Deficit
Fixed tank sizes	102.1	86.2	46.1	15.8	30.3
Tanks sized by CWB	121.6	79.8	45.9	41.7	4.2

Within *CWB* the demand is satisfied in the following order:

- Indoor use 2 (kitchen if residential)
- Indoor use 3 (bathroom if residential)
- Indoor use 4 (laundry if residential)
- Indoor use 1 (toilet if residential)
- Garden irrigation
- POS irrigation

This is why Indoor Use 2 has no deficit and Indoor Use 3 has less deficit than Use 4 etc (Table 9.4). The deficit for Indoor Use 1, which is top of the supply sequence for this stormwater strategy, is reduced to nearly zero with the introduction of varied tank sizes.

Table 9.4. Annual water supply deficit by usage category (values in Mm^3).

	Indoor Use 1	Indoor Use 2	Indoor Use 3	Indoor Use 4	Garden Irrigation	POS irrigation
Fixed tank sizes	20.3	0	0.004	7.1	3.7	0.3
Tanks sized by CWB	0.09	0	0.004	7.1	3.5	0.3

Although, wastewater recycling uses more energy than stormwater harvesting it is more economical and has much greater potential to reduce the mains deficit. In addition it is a more secure supply source since it is unaffected by climate change or by weather fluctuations. As with the stormwater harvesting, economies of scale are evident. For a domestic greywater system with a 2m^3 concrete tank (supplying 60m^3/yr) *CWB* calculates the whole life cost to be 6.9 kWh/m^3 and £2.9/m^3. *Lundin et al.* (2000) calculated energy usage of a small-scale WWTP, for a village of 200 inhabitants, as the equivalent of 5.4 kWh/m^3 for an annual supply of 60m^3/household/yr. This result is in excellent agreement with the predictions by *CWB*. Cost was not calculated in that study.

9.3.4 Strategy – Minicluster Rainwater Harvesting and Wastewater Recycling

Using a combination of rainwater harvesting and WW recycling (tank life 50 years) at MC scale to supply indoor use 1 and irrigation demands reduces the mains deficit from 162 mm to 6 mm (Fig. 9.15). However, this is only 6 mm less than using minicluster wastewater recycling by itself. So, the additional 6 mm is from the use of stormwater harvesting supplying irrigation demand. The introduction of MC scale RH with its associated extensive pipework to supply a relatively small irrigation demand increases the economic cost of this strategy significantly (from £0.88 to £1.16/m^3) and the energy use marginally (from 5.22 kWh/m^3 to 5.23 kWh/m^3) above that of MC WW recycling alone.

This raises the question of indicator importance. Although the combined strategy has diminished economic returns, if reduction in the supply deficit is the primary concern, even at higher economic or energy cost, then it needs to be considered as a viable option, especially for water scarce areas such as the Middle East and Australia.

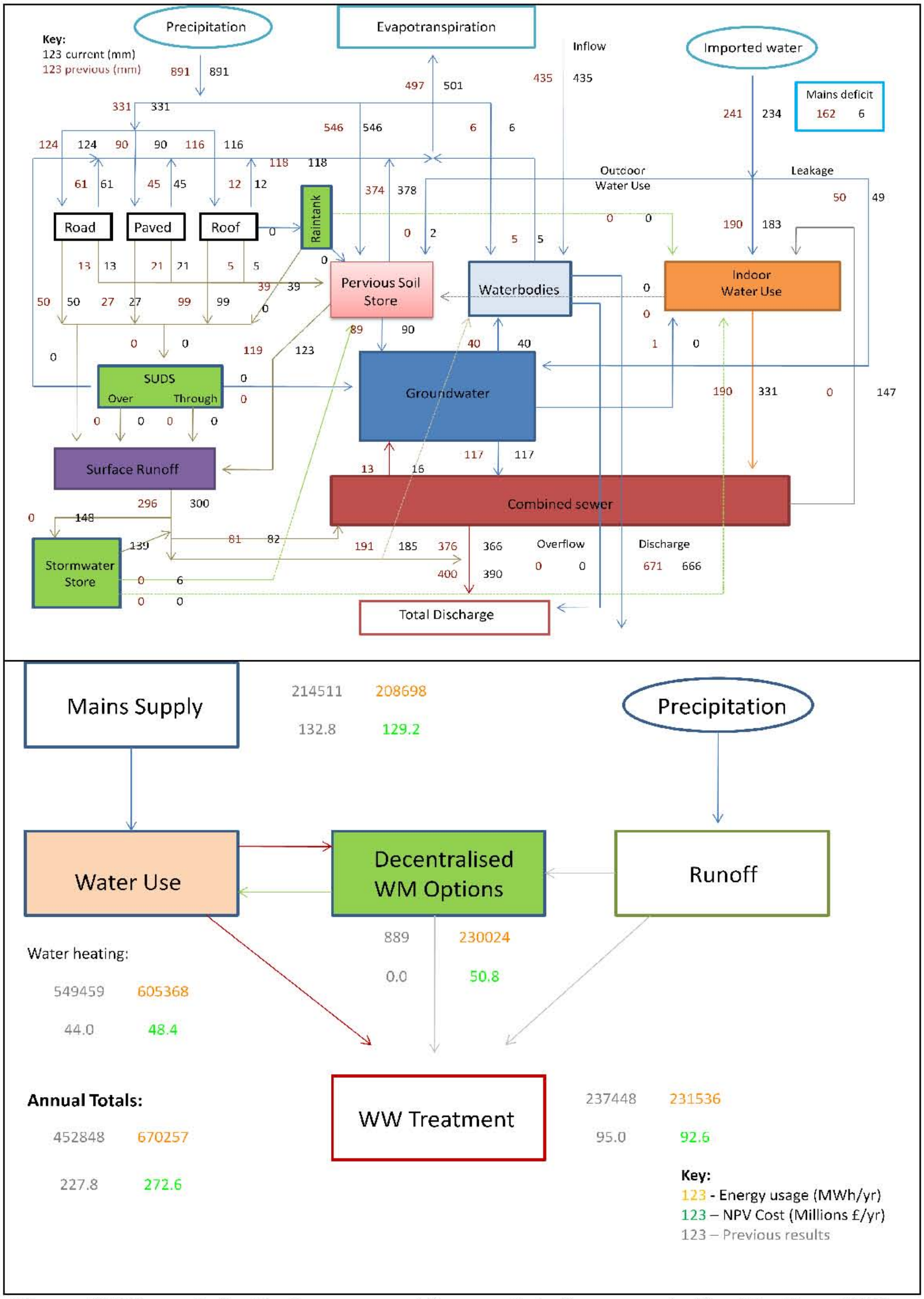


Figure 9.15 Scenario 2 – Business as usual (top – red; bottom – grey). Combination of MC scale WW recycling and RH applied across the study area, supplying Indoor Use 1 and garden and POS irrigation (top – black; bottom – orange and green).

9.3.5 Strategy - Borehole abstraction and Minicluster Wastewater Recycling

Large boreholes were situated in each subcatchment that overlies the unconfined aquifer (Fig. 9.16). They were sited away from the west boundary, where the Sandstone thins out over the Carboniferous formations, and at reasonable distances apart from other boreholes. Transmissivities range from 150-300 m^2/day (*Knipe et al.*, 1993) in the aquifer and so using Logan's rule (Eq. 9.1), and assuming that drawdown of 20 - 30 m would be acceptable, an estimate of sustainable abstraction from each borehole is 2.5 Ml/day.

(9.1) $Q = H_d * T / 1.32$ Where H_d = daily drawdown (m)
T = transmissivity (m^2/day)

This level of abstraction is in agreement with The Environment Agency Groundwater Position Statement (2005) which states that it is in receipt (2005) of applications for additional abstraction of 25 Ml/day. If all applications are accepted the licensed volume would rise to 63.45 Ml/day. However, it should be noted that although abstraction of up to 38.45 Ml/day was licensed in 2004, many of the licenses have been consistently underused for two decades, during which time the average abstraction has been 15 Ml. *Knipe et al.* (1993) show that historic groundwater levels in the Constitution Hill observation well fell significantly once total abstraction from the aquifer exceeded 45 Ml/day. Allowing a safety factor of 10 Ml/day, 35 Ml/day could be abstracted without impacting the groundwater level. Since current abstraction (2005) is 15 Ml/day (licensed for 38 Ml/day) the EA states that it will license an additional 20 Ml/day increasing the licensed total to 58 Ml/day.

In addition, under the Birmingham Groundwater Scheme, Severn Trent Water Company has been licensed to extract 33 Ml/day from the Sherwood Sandstone (4 boreholes in the confined and 1 in the unconfined) for 150-200 days/yr, in dry years (no more than 2 in 5), and discharge it to the Tame to reduce low flows on the River Trent and enhance supplies in the Nottingham area (*Winstanley*, 2011). At the time of the Environment Agency Statement, two boreholes in the unconfined aquifer and one in the confined were licensed to abstract 15 Ml/day and 5 ML/day respectively. Therefore, since the statement the EA has granted an additional 13Ml/day to STW. These additional licenses are equivalent to an average daily abstraction of only 2.85 Ml/day, taking into account the aforementioned restrictions. STW also intends to add a further borehole in the unconfined aquifer at Edgbaston, with 10Ml/day capacity subject to the same restrictions, within the next five years (*Winstanley*, 2011). The project is currently at the pilot borehole stage. This would increase STW's average additional abstraction since 2005 to 5.04 Ml/day.

It was assumed that groundwater is a suitable source for all water uses, given appropriate treatment, since the Birmingham aquifer is not severely contaminated, with only NO_3 and Ba^{2+} exceeding European Community Maximum Admissible Concentrations for Class A water (*Ford & Tellam*, 1994). Although contamination with Trichloroethylene (TCE), an industrial solvent, is currently quite widespread and was detected at 87% of abstraction sites in 1987, this was reduced to 56% by 1998 (*Rivett et al.*, 2005) and since the sources of TCE to the aquifer (heavy industry) are now gone it is expected that, by 2050, most of the existing TCE will have degraded. Some treatment would be required to improve abstracted groundwater quality to Class A standard, but little or no treatment would be required for Class B use (*Hunt & Lombardi*, 2006).

The results of a simulation using MC WW recycling and subcatchment borehole abstraction are shown in Figure 9.17. The addition of borehole supply to MC wastewater recycling reduces the mains deficit by a further 11 mm to 1 mm/yr. Since there are 11 additional boreholes capable of supplying 2.5 Ml/day, the potential annual supply is 36 mm (10,038 Ml) and the actual supply is 33 mm (9240 Ml) (including the 4 boreholes already established). The WW recycling system supply remains the same as in the previous MC WW strategy (147 mm).

It can be seen in Figure 9.13 that the annual cost for the MC WW strategy is £253.7 million and 623,300 MWh. When large boreholes are used, in addition, the energy usage rises to 636,400 MWh and the cost is reduced to £235.3 million (Fig. 9.17). Since the quantity of wastewater recycled in this combined strategy (147 mm) is the same as that in the WW recycling strategy (Fig. 9.12), and therefore the cost and energy, the annual cost of the borehole system is 55,100 MWh (236,400 -181,300 MWh) and £5.7 million (£36.4 – 30.7 million). This is equivalent to 5.92 kWh/m^3 and £0.61/m^3.

Borehole abstraction in combination with wastewater recycling proves to be a sustainable strategy. The economic cost is less than that of the business as usual case, partly because of the savings made in reducing the use of the high cost centralised wastewater systems. The reduction in wastewater flows is a result of lowered groundwater levels leading to reduced infiltration into the sewers and less groundwater overflow. Although this strategy uses more energy than the conventional supply, once the reduced wastewater treatment flows are taken into account the energy costs are much improved; with the introduction of boreholes, in addition to MC WW recycling, the total annual cost of the system rises from 623,300 MWh to 636,400 MWh (an increase of 13,100 MWh), for a reduction in deficit of 11 mm. Therefore the net energy cost of the borehole supply, including consequent additional wastewater treatment, is 4.23 kWh / m^3 (13.1 / (0.011 * 281.8)). Since the energy cost of conventional wastewater treatment in this scenario is 2.1 kWh/m^3, the cost of the borehole supply, by subtraction, is 2.13 kWh/m^3, which is significantly better than the business as usual cost of supply for this scenario (3.15 kWh/m^3).

The borehole abstraction strategy causes significant local drawdown of 4 to 9 metres, which is displayed clearly in Figure 9.16. These abstractions result in an overall lowering of the water table by 1.3 m from the business as usual levels.

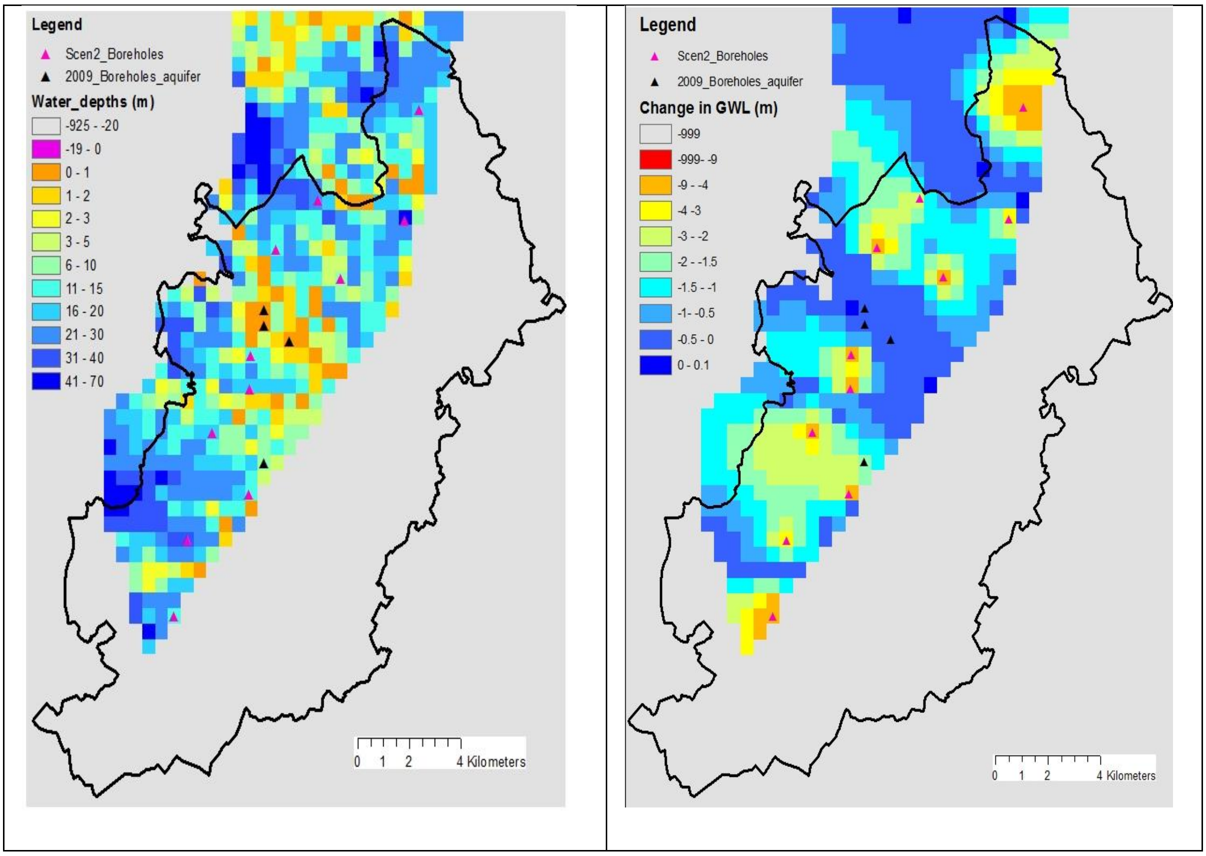


Figure 9.16 Steady state groundwater depths for the 2055 Scenario 2 borehole abstraction with WW strategy (left). Change in GWL from 2055 base case to Scenario 2 borehole abstraction with WW strategy (right).

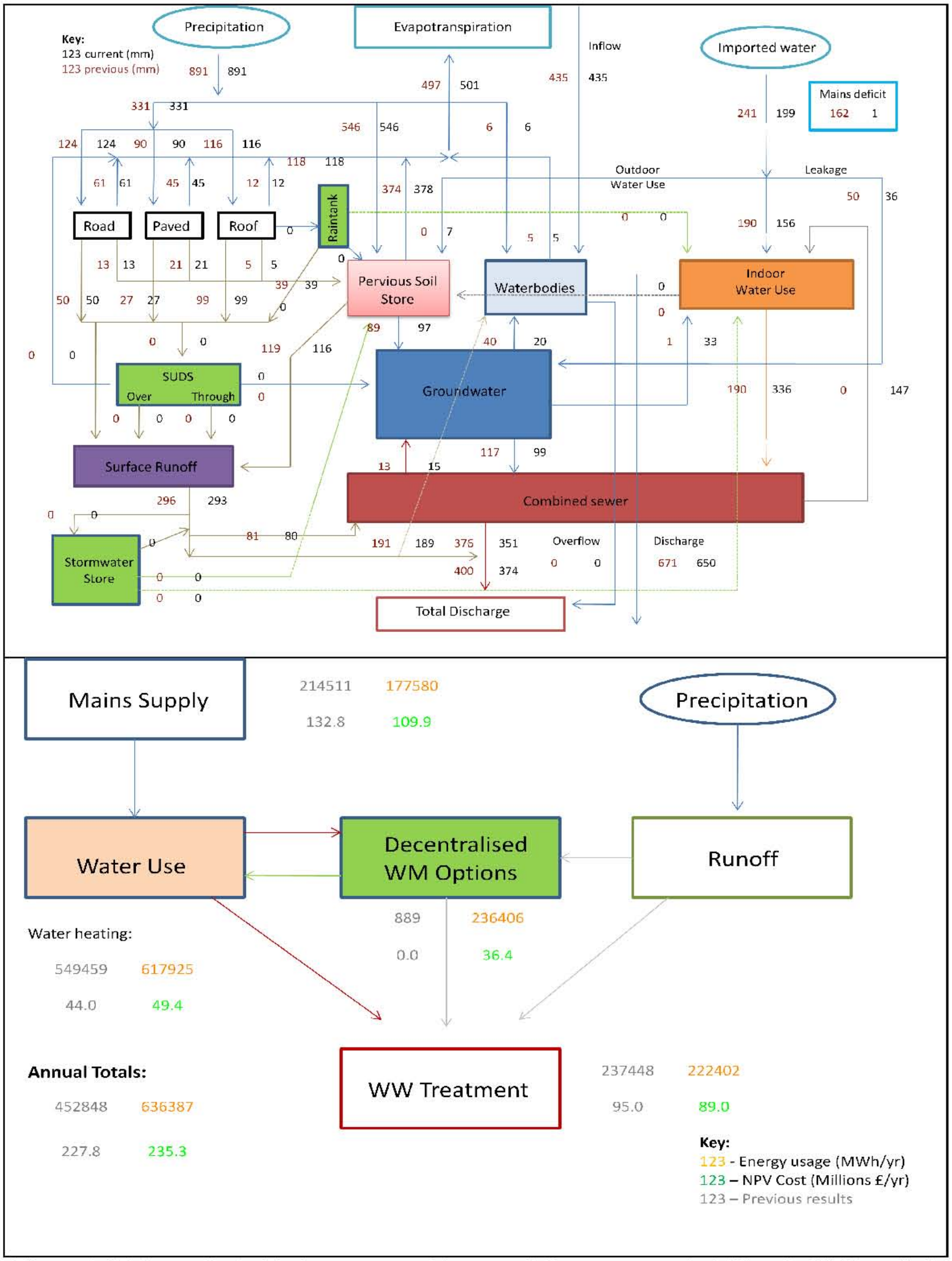


Figure 9.17 Scenario 2 – Business as usual (top – red; bottom – grey). MC scale WW applied across the study area, supplying Indoor Use 1 and garden and POS irrigation. Mains supply is supplemented by large-scale borehole abstraction (top – black; bottom – orange and green).

9.4 Scenario 3

In this scenario alternative water management strategies were investigated to supply the irrigation demand of widespread urban agriculture. Climate data for 2055 were used. Occupancy levels remain unchanged, as does mains leakage. Residential demand is assumed to be 130 l/c/day. The present value cost (energy and economic) of centralised supply decreases from 2009 values because it is treated to sub-potable standard.

Seventy percent of POS is used to grow food and is irrigated from mains supply. In order to simulate the increased irrigation demand for urban agriculture, all areas of POS are designated as wooded, which increases the evapotranspiration potential.

Since the mains supply is treated to sub-potable standard in this scenario, drinking water must be supplied separately. Annual study-area indoor water use is 54 Mm^3 and with the assumption of 130 l/c/day the population equivalent is:

(9.2) $54 * 10^6 / 365 / 0.13 = 1.1$ million

If each person requires 2 litres of drinking water per day (*BBC*, 2010) then 2200 m^3/day of bottled water are needed, in addition to other water requirements. Cotswold Spring 19 litre water bottles cost £6 each (2010 prices), including delivery (*WaterCoolersDirect*, 2010). That is equivalent to £0.32/litre. Therefore the total cost for 1.1 million people drinking two litres per day is £695,000/day or £254 million/yr! Clearly, this is unsustainable. Although, economies of scale are not taken into account, even if this estimate could be reduced by two thirds it would still be less economical than conventional potable-standard supply.

9.4.1 Business-as-Usual

Supply and drainage remain centralised. Figure 9.18 compares the results for the 2055 base case with Scenario 3 business as usual for water flow, cost and energy. It can be seen that there is considerably more evaporation as a result of the urban agriculture. Consequently, there is less recharge and less groundwater overflow. Mains supply is slightly less than the base case because it is assumed that demand falls from 150 l/c/day to 130 l/c/day by 2055. The total energy and cost results are less than those for the base case by 25% and 20% respectively because the mains water is treated to sub-potable standard.

9.4.2 Strategy – Brown Roofs and Minicluster Wastewater Recycling

Brown roofs were applied to all non-residential buildings, high-rise residential, apartments, old people's homes and detached residential buildings. Irrigation was supplied to POS using 800 m^3 capacity subcatchment-scale wastewater reuse plants and mains back-up (Fig. 9.19).

Depths to groundwater are shown in Figure 9.20. This scenario has the same population and leakage as the 2055 base case but has increased evapotranspiration as a result of widespread urban agriculture and the application of brown roofs. A general lowering of the average water table (by 2.3 m) across the aquifer can be seen as a result of reduced recharge which is most evident in the western part, where the water table drops by up to 12 metres.

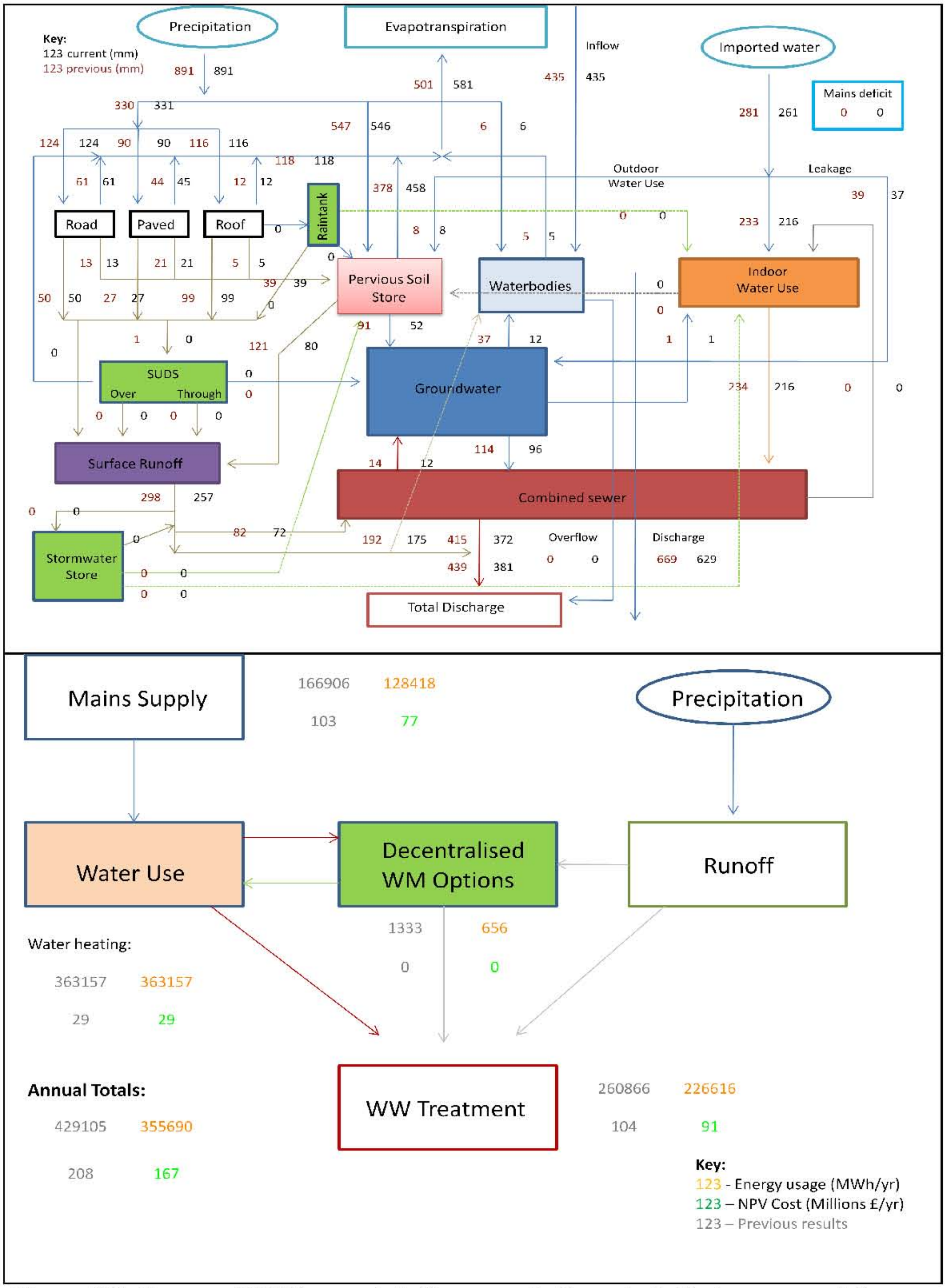


Figure 9.18 Base case 2055 (top – red; bottom – grey). Scenario 3 –Business as usual (top – black; bottom – orange and green).

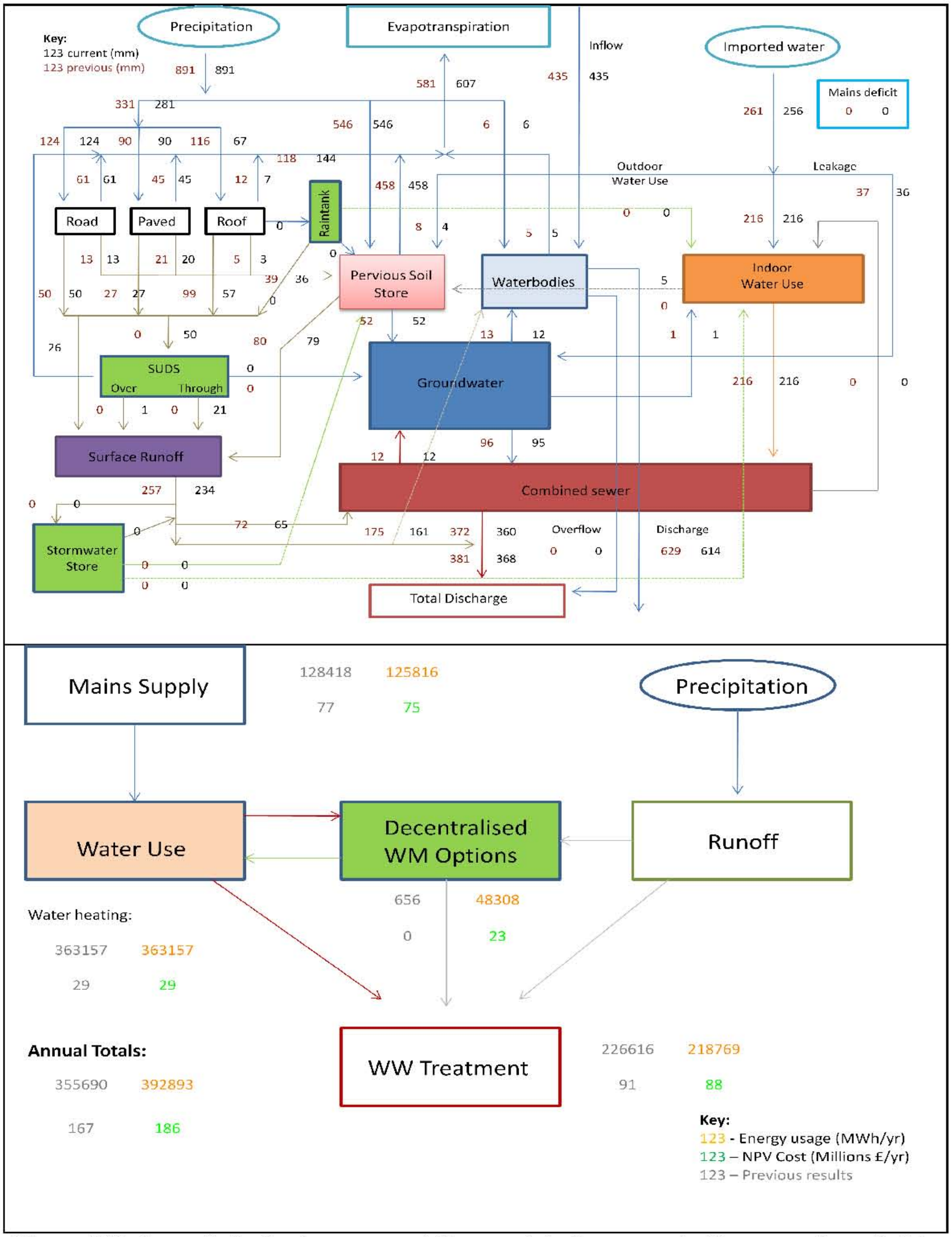


Figure 9.19 Scenario 3 - Business as usual (top – red; bottom – grey). Brown roofs applied to all non-residential buildings and to high-rise, apartments, old people's homes and detached residential buildings. Supply irrigation to pervious stores with large -scale treated wastewater reuse (top – black; bottom – orange and green).

Table 9.5 Water flow (Mm^3) performance indicators for brown roofs and wastewater recycling plants.

	Supply to green roof	Overflow	Draindown	Evaporation	Storage
UB Green	14.0	0.3	5.9	7.3	0.5
	Supply to tank	**Overflow**	**Demand**	**Use**	**Deficit**
Subcatchment WW	98.0	96.6	2.6	1.4	1.2

The application of subcatchment scale wastewater recycling to supply irrigation needs is found to be more expensive than the mains supply both in terms of energy and cost; 5 mm (1,400 Ml) is supplied at a cost of 27,000 MWh (19.3 kWh/m^3) and £12 million (£8.6/m^3). This is primarily a result of the relatively small irrigation demand compared with the cost of installing and operating large wastewater plants. In addition, mains water is already being supplied at reduced quality, and therefore at less cost (energy and economic). Investigation of direct greywater re-use (without chemical treatment) shows annual savings of 4,500 MWh and £1 million over the strategy with treatment, but is still far more expensive than mains supply (Fig. 9.21). However, when the wastewater stores are sized depending on demand and used to supply indoor use 1, in addition to irrigation, they became more sustainable than conventional supply in terms of cost (£0.99/m^3) but not in terms of energy (5.11 kWh/m^3) (Fig. 9.22 & Table 9.6). The locations and capacities of the stores are shown in Figure 9.23.

For subcatchment scale wastewater recycling it would be necessary to install extensive sub-potable supply systems, which would be subject to leakage like the conventional system. It is not currently modelled in *CWB* but it is expected that leakage from a new system will be much lower than the conventional supply pipes, which can be over 100 years old (*Mouchel*, 2010).

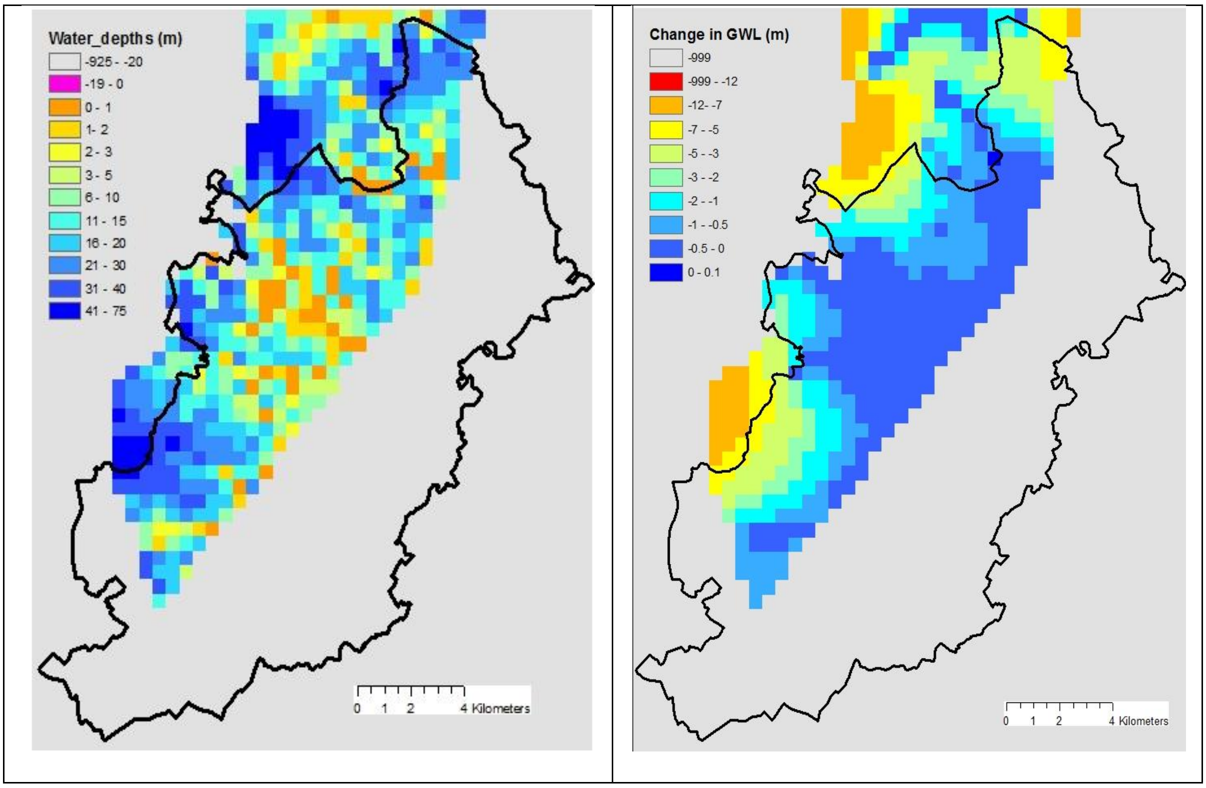


Figure 9.20 Water depths to groundwater for base case Scenario 3. Change in GWL from 2055 base case to Scenario 3 business as usual (right).

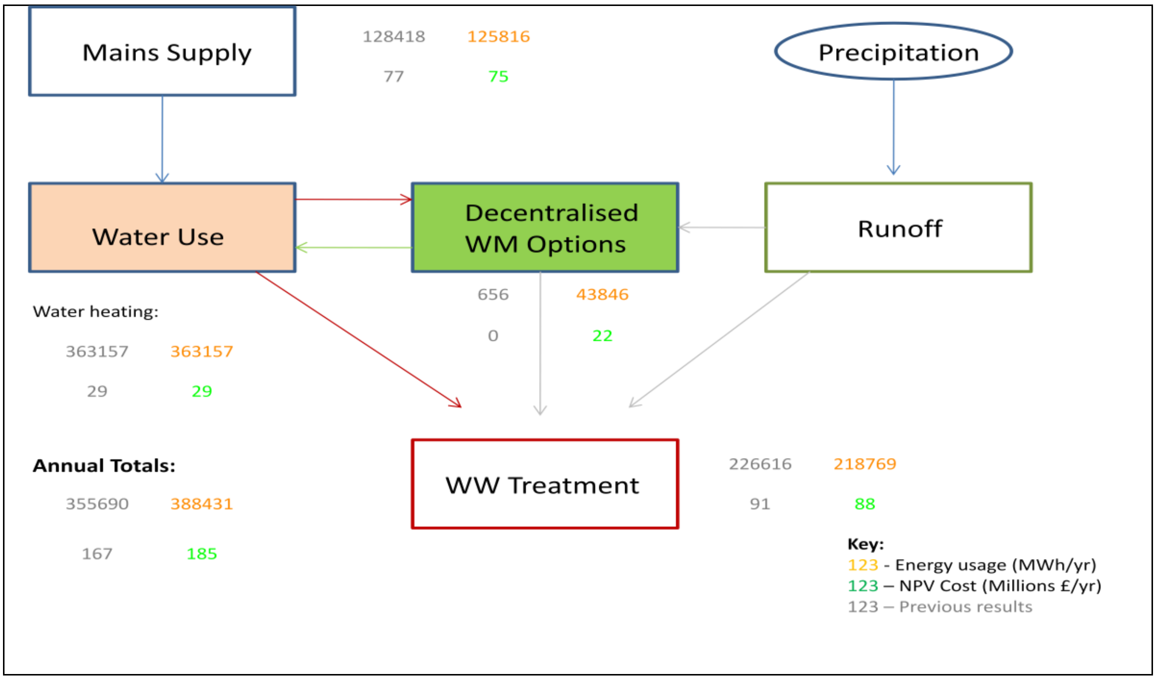


Figure 9.21 Scenario 3 - Business as usual (grey). Brown roofs applied to all non-residential buildings and to high-rise, apartments, old people's homes and detached residential buildings. Supply irrigation to pervious stores with large -scale greywater reuse (no treatment) (orange and green).

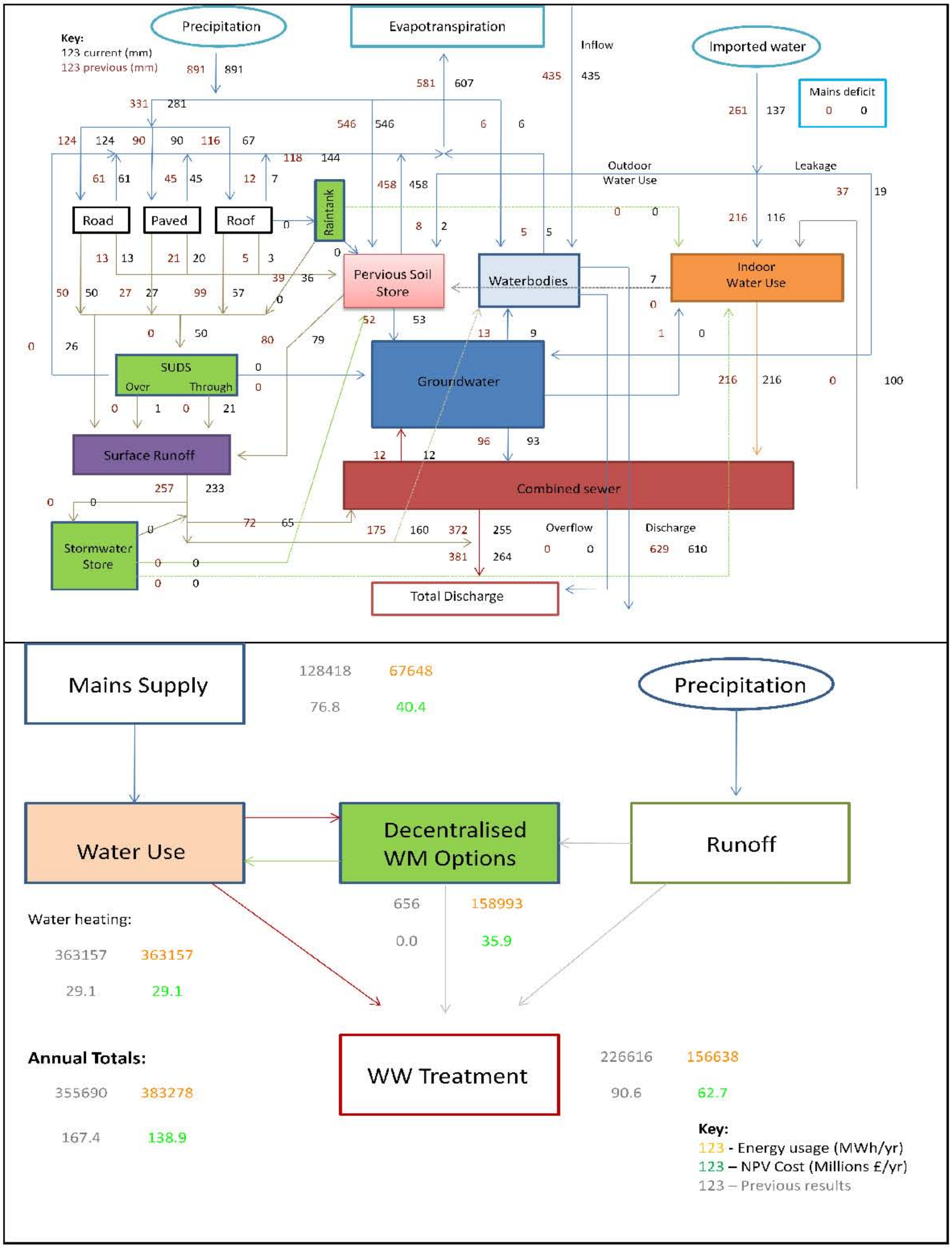


Figure 9.22 Scenario 3 - Business as usual (top – red; bottom – grey). Brown roofs applied to all non-residential buildings and to high-rise, apartments, old people's homes and detached residential buildings. Supply irrigation to pervious stores and Indoor Use 1 with Large-scale wastewater reuse (top – black; bottom – orange and green).

Table 9.6 Water flow (Mm^3) performance indicators for brown roofs and wastewater recycling plants.

	Supply to green roof	Overflow	Draindown	Evaporation	Storage
UB Green	14	0.3	5.9	7.3	0.5
	Supply to tank	**Overflow**	**Demand**	**Use**	**Deficit**
Subcatchment WW	97.2	67.0	32.7	30.1	2.6

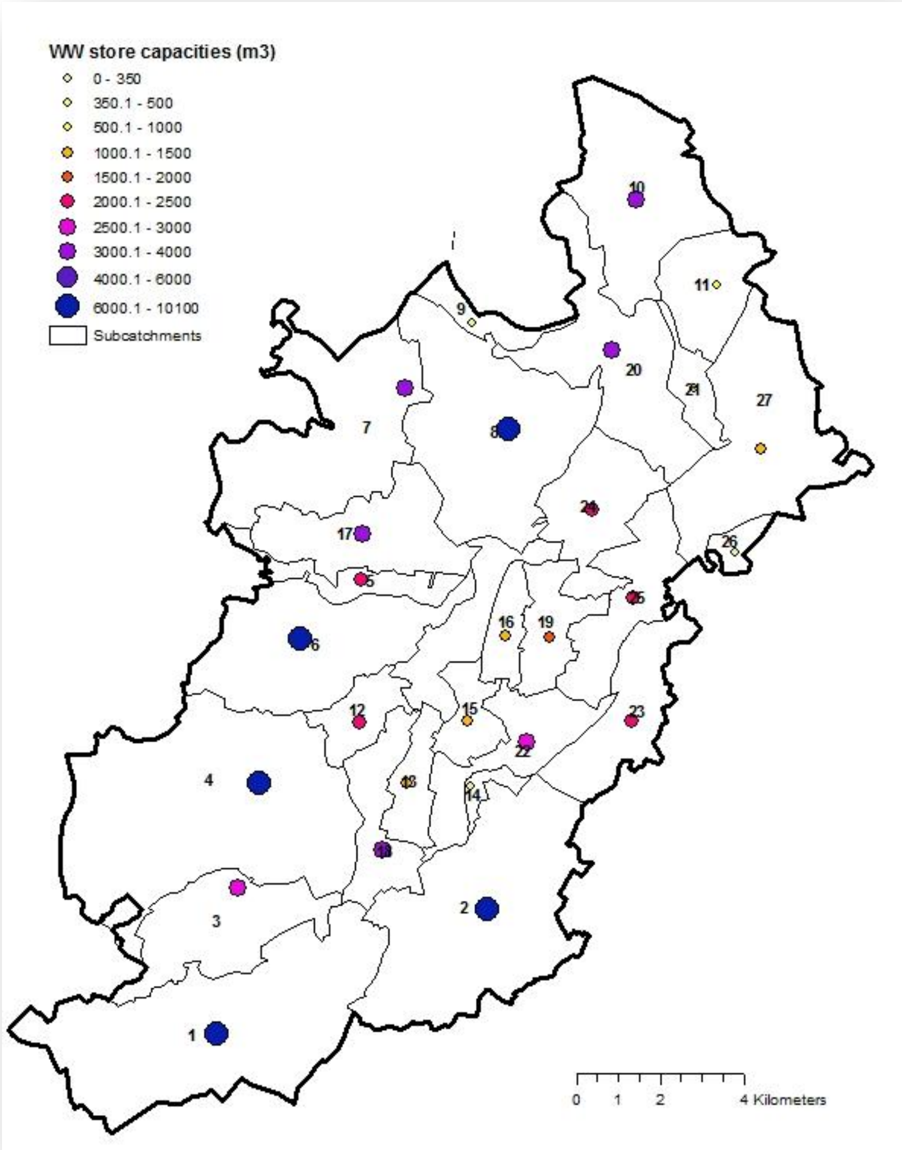


Figure 9.23 Scenario 3 large wastewater recycling stores locations and capacities, calculated by *CWB*.

9.5 Summary of Scenario Results

Table 9.7 summarises the energy and cost results for the three scenarios. It can be seen that the most sustainable strategy investigated, in terms of energy and cost, is large-scale borehole abstraction. Rainwater harvesting at minicluster scale for commercial and industrial landuse is also predicted to be more sustainable than centralised supply. Wastewater recycling at both MC and large scales is predicted to be quite economical but has a high energy usage.

Table 9.7 Life cycle energy and cost indicators for the three scenarios.

	Cost (£/m3)	Energy (kWh/m3)
Scenario 1		
Business as usual	1.95	3.15
Brown roofs	0.84	0.88
MC SS	1.15	2.32
Scenario 2		
Business as usual	1.95	3.15
MC SS	1.18	3.13
MC WW	0.81	5.02
Large Boreholes	0.61	2.13
Scenario 3		
Business as usual	1.74	2.1
Large WW	0.99	5.11

9.5.1 Groundwater

Aquifer averaged groundwater levels are shown in Table 9.8. As can be seen groundwater levels are slightly lowered as a result of climate change between the 2009 and 2055 base cases. The water table stays at about the 2009 average level for Scenario 1 and 2 as a result of the increased leakage volumes from a deteriorating, centralised supply system struggling to cope with the greater demand of an increased population. The borehole abstraction strategy caused significant local drawdown (from 4 to 9 metres – Fig.

9.16) and resulted in a lowering of the average water table by 1.3 m. Scenario 3 has the same population and leakage as the 2055 base case but has increased evapotranspiration as a result of widespread urban agriculture. This leads to a lowering of the average water table by 2.3 m across the aquifer as a result of reduced recharge.

Table 9.8 Water table heights averaged over the aquifer.

Scenario and strategy	Average groundwater height (m above datum)
2009 base case	125.6
2055 base case	125.1
Scenario 1 base	125.8
Scenario 2 base	125.2
Scenario 2 boreholes	123.9
Scenario 3	122.8

Significant volumes of groundwater discharge to overland flow were evident in areas where the groundwater is near the surface, generally around the rivers, in all scenarios. This is partly a result of the terrain smoothing that *CWB* uses and the coarse grid size (500m by 500m). Further, *CWB* is a scoping model, designed to give general indications, and not detailed groundwater dynamics. In addition, there are known areas of very shallow groundwater in Birmingham (e.g. springs on the University of Birmingham campus), particularly near the Birmingham Fault and the River Tame.

9.6 Water efficiency

Additional simulations were carried out to quantify the sustainability of using water efficient residential appliances. Table 9.9 shows the appliances that were used in the business as usual and water efficient cases.

Table 9.9 List of appliances and their water usage for the business as usual and the water efficient strategies.

	Business as usual		Water efficient	
Appliance type	**Description**	**Water usage (l)**	**Description**	**Water usage (l)**
Washing machine	Bosch WFX1485	54	Hotpoint WF320G	30
Toilet	Conventional	14	Dual flush 6/4	4.4
Shower	Conventional	110	Low flow shower-heads	70
Bath	Small	80	Small	80
Handbasin	Conventional	8	Aerator	7
Kitchen sink	25mm sink tab	36	15mm sink tab	12
Dishwasher	None	0	None	0

In order to get a fair comparison between the two residential demand models (split usage and appliance based), occupancy had to be increased by 56% from 2.5 (split usage) to 3.9 (appliances). A comparison of the results for the residential appliance model and the split usage model is shown in Figure 9.24.

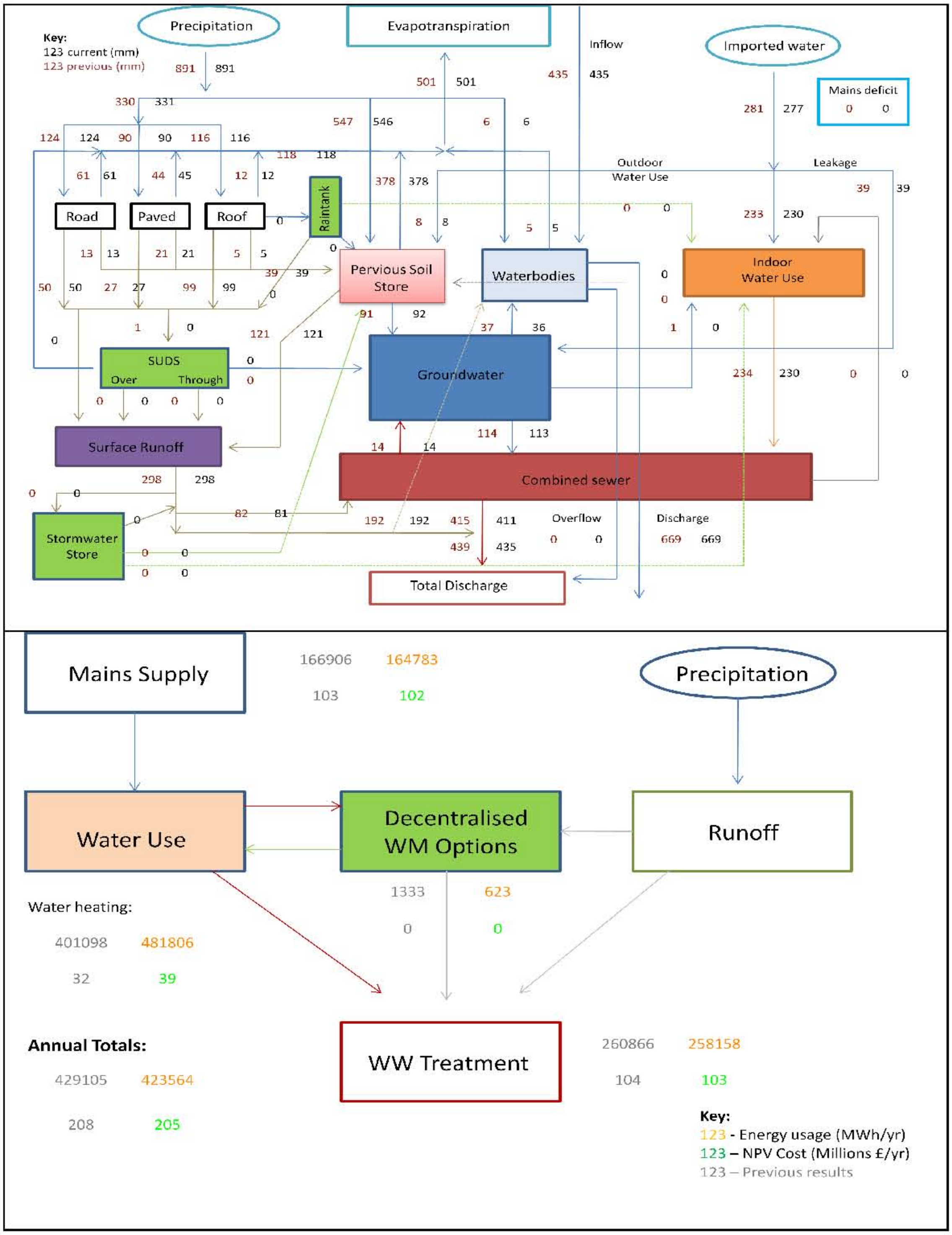


Figure 9.24 Split use model (top – red; bottom – grey). Residential appliance model (top – black; bottom – orange and green).

Figure 9.25 compares the results of a simulation with water efficient appliances in residential houses with the results from a simulation with standard appliances (Table 9.9). Mains demand was reduced by 15 mm/yr (4200 Ml/yr) and wastewater discharge by 14 mm/yr (3900 Ml/yr). Consequently, the total annual cost of the system was reduced by 18,500 MWh and £9 million. Savings on domestic water heating were 61,000 MWh and £5 million. In the calculation of these estimates no account is taken of the cost of retrofitting. It is assumed that, between now and 2055, households will gradually replace old appliances with new, more efficient ones. The volume of water that is designated "hot" in a household is calculated as a fixed proportion of the total use for each appliance. Consequently, a reduction in water use by an appliance results in a proportional reduction in hot water required. This assumption probably over-estimates the reduction in "hot" water requirement but there are very little data available to quantify this (*Clarke et al.*, 2009).

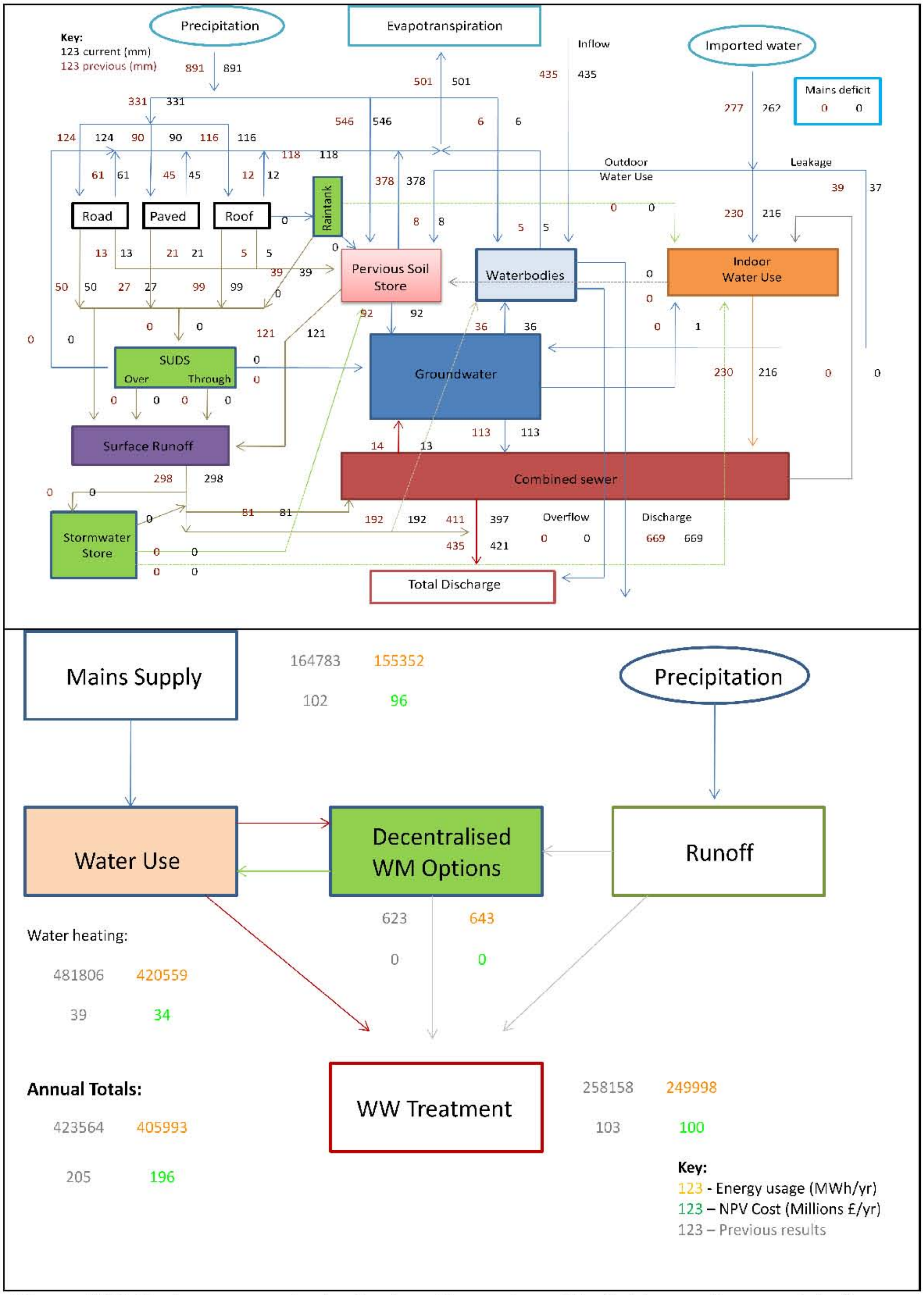


Figure 9.25 Business as usual: standard appliances in residential houses (top – red; bottom – grey). Water efficient residential appliances (top – black; bottom – orange and green).

It is clear that the use of water efficient appliances is the most sustainable water management option by a significant margin, especially if water heating is taken into account. This finding is in agreement with current thinking on water management (*EA*, 2009; *Clarke et al.*, 2009). Figure 9.26 demonstrates the dominant impact that domestic water heating has on CO_2 emissions from, and therefore energy use in, the water industry. A caveat is that some appliances that reduce water consumption but use electricity to heat the water, such as electric showers or dishwashers, may represent false economy in terms of CO_2 emissions (*Clarke et al.*, 2009). However, in this demonstration scenario electric water efficient appliances were not used.

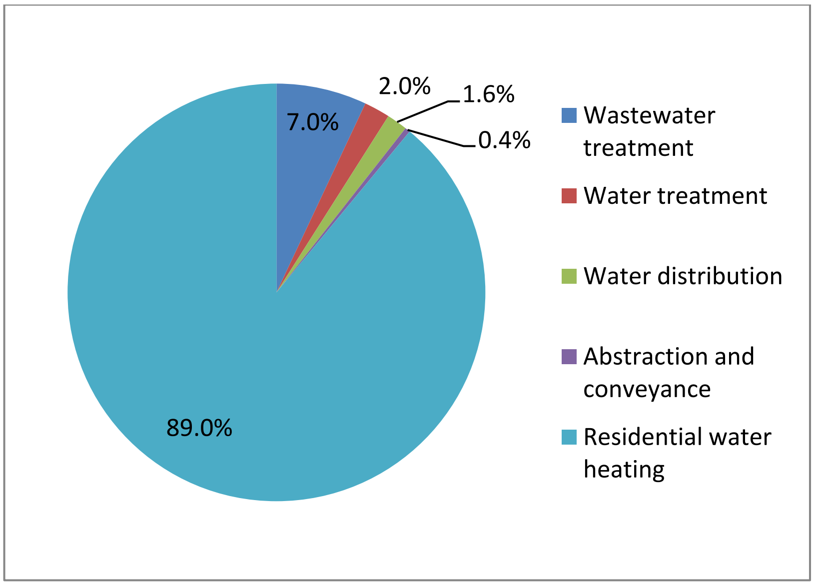


Figure 9.26 Carbon emissions associated with the water industry and household use in England and Wales (*EA*, 2009).

A selection of strategies have been applied to the various scenarios as part of the demonstration case study for Birmingham. *CWB* also has the capability to examine the performance of other water management options not demonstrated here, such as filter strips, swales, soakaways, porous paving, and detention basins. In addition, much greater detail on performance data can be provided, if required, ranging from daily results at the unitblock scale to total time period catchment results.

9.7 Problems Encountered

The main problems encountered during the course of this research were gaining access to the necessary data for the Birmingham case study and the time-consuming nature of processing that data. Table 9.10 shows key data that were supplied by Severn Trent. Since they were obtained late in the project the completion of the case study was delayed.

Table 9.10 Data supplied by Severn Trent and their application.

Data description	Use
DMA supply data	Calibration and validation of supply; leakage estimates
Flow monitoring data	Calibration and validation of stormwater and wastewater flows
Sewer/stormwater network in GIS	Defined subcatchment boundaries

The process of describing land use for such a large, diverse area as the City of Birmingham was very time consuming. Miniclusters are designed to rapidly delineate large areas of uniform land-use such as residential areas. However, in Birmingham there are many areas that have very mixed land use, which often made allocation of miniclusters less clear-cut. There was then a trade-off between accuracy of land-use description and time spent either creating smaller clusters or deciding on estimated boundaries.

Approximately 6,000 miniclusters were used in this case study and this significantly impacted the runtime of simulations. For a typical case study a one-year simulation took about ten to fifteen minutes. By comparison a one-year test case study simulation with 4 miniclusters takes only several seconds on a 2.4 GHz PC.

Several weeks were spent correcting conceptual problems with groundwater simulations. In areas where there is a sharp contrast in ground level between adjacent cells (Table 9.11) the groundwater table was rising above ground level. In *CWB* each cell of the aquifer grid is assigned a water height above datum and a ground level above datum and each minicluster is assigned a ground level and groundwater height based on the areal average contribution of the cells which it overlies. The groundwater model acts to reduce the difference in groundwater heights between cells, so if there is a sharp difference in terrain height the average groundwater level of a MC that overlies both will be above ground over the terrain low. Once the source of the problem was found it was solved by dividing MCs in problem "hotspots" into parts of approximately grid cell size.

Table 9.11 Example ground level in metres above datum for cell centred on (408250, 288750) and surrounding cells.

117	110	98.4
109	98.1	108
110	108	105

9.8 Application of CWB to other cities

In addition to the Birmingham case study *CWB* has been used, by three MSc students at the University of Birmingham, to model Alexandria (Egypt), Accra (Ghana) and Dunedin (Florida). The results of this work gave rise to three successful dissertations. Table 9.12 shows key information about the three cities for comparison with Birmingham.

Table 9.12 Key information about the case study cities (*Simister*, 2010; *Jaweesh*, 2010; *Spencer*, 2010).

	Birmingham	**Alexandria**	**Accra**	**Dunedin**
Location	UK, central	Egypt, coastal	Ghana, coastal	Florida, coastal
Area (km^2)	280	180	185	27
Landuse	Mixed industry, commercial and residential houses	Mixed industry, commercial and high rise residential	Mixed industry, commercial and residential (shanty and high rise)	Mainly residential
Population (millions)	1	4 (increasing by 30% in summer)	2	0.037
Climate	Temperate marine. T_{max} 20°C. T_{min} 4.5°C. 877mm annual precip.	Arid coastal. T_{max} 26.4°C. T_{min} 13.5°C. 190mm annual precip.	Tropical. T in range 20-35°C throughout yr. 1200mm in 2008 w peak of 400mm in May	Sub-tropical. ~1400 mm in 2009
Hydrology	Extensive canal system. Three rivers: Tame, Rea and Cole	Two major water supply canals. Lake Meriout	Three river catchments. Only Korle modelled in case study	No signifcant surface water courses within city bounds
Hydrogeology	Unconfined aquifer to west of Birmingham Fault	Several unconfined aquifers. Groundwater too saline for abstraction	Majority of Korle catchment is relatively impervious Dahomeyan schists	Unconfined (2-16m) overlies confined aquifer (30-80m) separated by aquitard
Water systems	Majority supply from Elan Valley reservoir, Wales. Wastewater treated at Minworth WWTP (largest in Europe)	Supply by aqueduct from the Nile. WWTP discharges to Lake Mariout	Supply from two reservoirs outside the city. Large areas not connected to central network. Central WWTP in James Town serves city centre and discharges to Gulf of Guinea. Waste from private septic tanks is disposed of in land fill with sludge from the WWTP in James Town.	Supply from groundwater in confined aquifer within city bounds. Recycled water used for irrigation. WW discharge to Tampa Bay.
Water issues	Climate change, population increase, high groundwater levels, source control of pollution	Climate change and population increase. Increased abstraction further upstream on the Nile. Unaccounted for water is high (36%). High water consumption (230l/c/day). Much of WW only given primary treatment	Rapid population increase. Development of slum area with poor access to water supply or sanitation. High levels of water loss and low cost recovery. Pollution. Flooding	Groundwater quality including salt intrusion and sustainable abstraction

Several new modelling issues arose during the application of *CWB* to cities with such widely varying size, climate, hydrogeology, demographics and affluence.

In Alexandria wastewater is discharged, after treatment, to Mariout Lake, which was included in the model. The current version of *CWB* only allows up to ten MCs to discharge to a lake, but the model had more than 100. This was circumvented by dividing Lake Mariout into 13 smaller lakes. It is a simple coding task to increase the maximum number of MCs that can discharge to a lake but the code was not available to the MSc student. The model was successfully calibrated to water supply, wastewater and recharge volumes.

In Accra the main problem encountered in the application of *CWB* was that only 55% of people have a mains connection. These people still use water but get it from either tankers or standpipes. If a mains connection is modelled to represent this supply then it will be subject to leakage and that leakage will be unrealistic. It would be possible to modify the leakage rate to take account of this but there would still be recharge from leakage occurring where in reality it does not. The model was calibrated to water supply and was tested using three population scenarios.

It was found that a one-layer unconfined aquifer was insufficient to model groundwater in Dunedin. Groundwater is the sole source of supply to the city and it is abstracted from the confined aquifer within the city bounds. To adequately model this it is necessary to expand *CWB*'s capability to include a multiple layer subsurface. It was also suggested that modelling of a constant head boundary, instead of no flow, to allow sub-surface flow out of the aquifer would be more realistic. All of the input data were collected but simulations were not completed by the MSc student as a result of insufficient time to implement the necessary changes to the groundwater model.

9.9 Summary

In this chapter the sustainability of various water management strategies applied to three future scenarios, proposed by the Birmingham Learning Alliance, for the City of Birmingham have been discussed. In addition to demonstrating possible impacts of climate change on the urban water system, it was shown that rainwater harvesting and borehole abstraction are likely to be more sustainable options than wastewater recycling and the use of brown roofs. Water efficiency measures were the most sustainable strategy, which is in agreement with current thinking in the field of water management.

10 - Conclusion

A scoping model for urban water sustainability has been developed called *City Water Balance* (*CWB*) as the main output of this research. *CWB* has the capability to assess the sustainability of a variety of water management options against future scenarios, in terms of water flow, contaminant loads, whole life cost and life cycle energy. It has built on concepts from other contemporary urban water modelling programs, in particular *Aquacycle* and *UVQ* (Urban Volume and Quality) *(Mitchell & Diaper,* 2005). Innovative aspects of the model include better representation of the natural systems, expansion of water management options available and inclusion of energy and cost indicators.

A library of life cycle cost and energy data has been collated for alternative water management options including rainwater harvesting, wastewater recycling, borehole abstraction, green roofs and a variety of SUDS. Costing of SUDS such as filter strips and retention basins is well documented by CIRIA (*Woods-Ballard et al.*, 2007) and so was not repeated. There has been some work into the whole life cost of small scale wastewater recycling and rainwater harvesting as well as green roofs. There have also been various studies that have investigated the life cycle energy use of raintanks and wastewater recycling. However, very little work has been undetaken on life cycle energy costing of SUDS such as filter strips, porous pavements, retention basins, soakaways and swales. Moreover, most work that has been completed in this area has concentrated on only one or two water management options. The novel aspect of *CWB* is that not only does it break into new areas of research with life cycle energy use of SUDS but it brings a wide range of water management options together into one model.

CWB has improved on existing urban water scoping models with its more inclusive treatment of the natural systems. In other models the interaction between groundwater, surface water bodies and soil stores and the consequent implications for quantity of surface runoff, soil storage and evaporation is either omitted (e.g. *UWOT*) or modelled very simplistically (e.g. *Aquacycle* and *UVQ*). Adequate modelling of these flows is particularly pertinent when siting infiltration SUDS for which knowledge of groundwater levels is required. For example, in the US Environmental Protection Agency's guidance for Best Management Practices it is recommended that most infiltration SUDS are unsuitable for use in areas where the groundwater is within two metres of the surface (*USEPA*, 2004). The simulation results for Birmingham not only show that there are significant areas bordering rivers that are likely to be unsuitable but also clearly show good potential sites for infiltration SUDS above the southwestern and northeastern parts of the aquifer. In addition, the effect of borehole abstraction on groundwater levels both locally and at the city-scale was clearly demonstrated and was predicted to be within the bounds of acceptability. It is these types of results that models that do not sufficiently account for groundwater dynamics cannot comment on.

The choice of two indoor demand models for residential land use is another novel aspect of *CWB*. There is the split-usage model adopted from *Aquacycle* and the water appliance model adopted from *UWOT*. The split-usage model is generic and flexible, enabling rapid allocation of water demand to widely varying land-use types. On the other hand, the water appliance model enables more detailed exploration of the benefits of water efficiency measures.

Application of *CWB* to the City of Birmingham in 2055 has shown that medium scale rainwater harvesting is predicted to be more sustainable than the conventional centralised supply. It was calculated that medium scale wastewater recycling would be more cost effective but less energy efficient than conventional supply. Borehole abstraction was predicted to be the most sustainable strategy, in terms of energy and cost, in the three scenarios. The sustainability of brown roofs was not clear but it was estimated that, if attenuation benefits are considered, they are marginally more sustainable than centralised drainage, in terms of the indicators output by *CWB*. Ecological and aesthetic benefits would further tip the balance in favour of brown roofs.

The most sustainable strategy, in terms of whole life cost and life cycle energy use, was installation of water efficient appliances. There is the potential for large energy savings as a result of reduced indoor usage and consequently water heating requirements. This result is in agreement with current thinking on water management (*EA*, 2009; *Clarke et al.*, 2009).

10.1 Model Limitations

City Water Balance has been developed to demonstrate quantified possibilities for improved water management at the city scale to decision makers. Under this remit, the model has not been designed to and does not produce highly accurate results at small spatial scales and sub-daily timesteps but rather predicts general patterns at larger scales (time and space).

In its current version *CWB* is capable of modelling one unconfined aquifer, so cities, such as Dunedin (Florida), for which it is necessary to model several aquifers (since the confined aquifer is used for water supply in this instance), cannot be modelled with sufficient accuracy to yield meaningful results for gourndwater dynamics.

In developing countries the model cannot take account of water supplied by tanker since if this is assumed to be supplied by the mains it is subject to leakage and consequent recharge.

CWB does not calculate the energy and cost of centralised water supply and wastewater systems, but uses an average user selected value for a city since it was designed to investigate decentralised technologies.

Although it was considered that the indicator list output by *CWB* was quite broad it does not calculate CO_2 emissions, nor does it calculate environmental impact indicators such as those output from a Life Cycle Analysis.

There is currently no optimisation tool or measurement of uncertainty in the software.

CWB does not account for removal processes of pollutants in transport (i.e. in the sewers). The removal processes are set as fixed percentages of the total input load and so are sufficiently accurate for the purposes of this scoping model but should be used with caution.

10.2 Future Work

The following ideas for future improvement of *CWB* and expansion of its capabilities are recommended:

Integration of CWB into ArcGIS

Since many of the input data are spatially distributed *CWB* could be integrated into the ArcGIS environment. This would facilitate display of spatial results, reduce preparation time for the model and make it simpler to use.

Enhancement of Display of Results

The range of display tools for results could be widened. The current flow diagram format that was used to demonstrate the results of the Birmingham case study may be too detailed for the user who is more interested in a few key indicators. In this case, the use of a spider plot could be a very visually effective way to rapidly communicate these (Fig. 10.1). In addition the use of an aggregated indicator that is the user-weighted average of a range of indicators may also be useful.

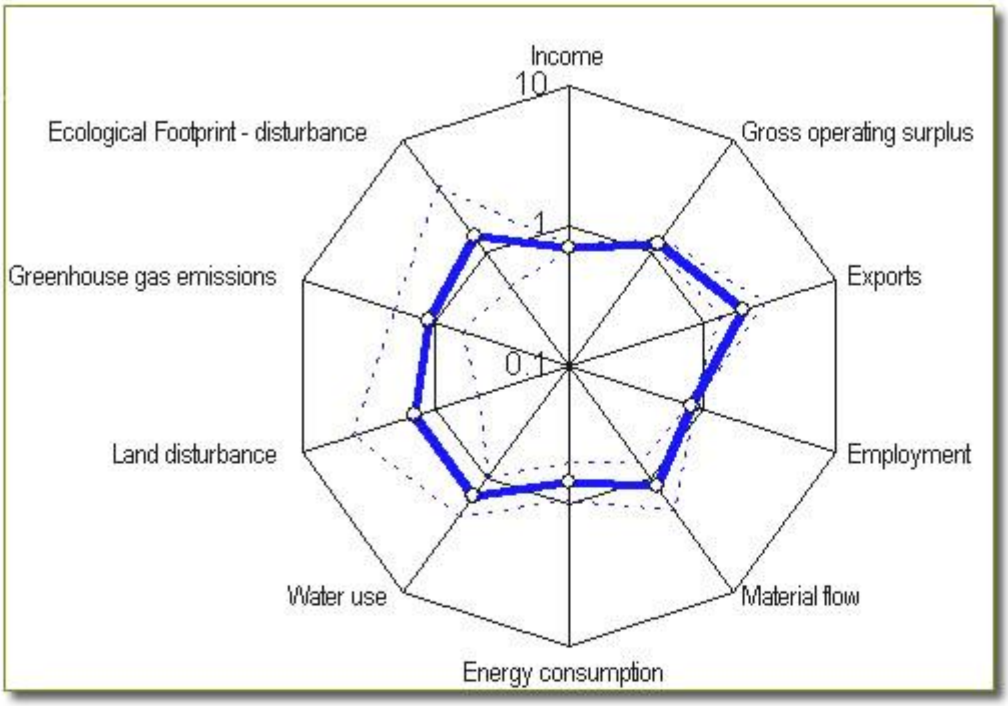


Figure 10.1 Example of a spider diagram measuring the sustainability of two business strategies (dashed blue lines) against the benchmark (bold blue line). Poor scores are given a high relative value and fall towards the periphery of the plot, outside the benchmark (*Integrated Sustainability Analysis*, 2005).

Inclusion of sub-daily timestep

The optional inclusion of a sub-daily timestep would allow more detailed exploration of peak flows. Separate climate data would have to be used but in theory much of the existing subroutines could be called, without alteration, during the shorter timestep simulation. Ideally, the user would be able to choose from the graphical user interface (GUI) which timestep he/she wishes to use for a particular simulation.

Energy cost analysis of larger centralised supply

The energy and cost analysis could be extended to include centralised supply options instead of using the very simplified approach currently adopted. This would allow more accurate calculation of the cost and energy use of centralised supply to a particular city. For example, a city largely supplied with groundwater will have different treatment costs from a city supplied from reservoirs.

Expand the sustainability indicator list

The indicator output from *CWB* could be expanded to include carbon costing with relative ease since energy use and carbon cost can often be related by a simple factor. An impact assessment module could be developed that converts the results from the sustainability indicators into impacts, as done in LCA, such as global warming potential, ozone depletion potential and acidification potential.

Model confined aquifer

The subsurface model could be extended to include confined aquifers. In the Birmingham case study this would have allowed some of the groundwater to flow across the fault into the confined aquifer and reduced the quantity of groundwater discharge to the surface.

Aquifer storage and recovery

Include an aquifer storage and recovery module which would essentially work the same as a borehole for the recovery part and storage would be modelled as direct recharge of the treated wastewater into the aquifer. A surface treatment plant and store would have to be included as part of the model as well.

Street sweeping

This could be modelled as a simple reduction factor applied to the fixed runoff concentration from roads.

CSO thresholds

The inclusion of combined sewer overflow thresholds and discharge points to the rivers would be relatively straight-forward. The overflow threshold could be set at the minicluster or subcatchment scale and overflows could be directed to surface water bodies in the same fashion as stormwater discharges in *CWB*.

Green Space

Differentiate between Public Open Space and Green Space, when defining land use, since there are important ownership, and therefore cost, issues for retrofitting SUDS.

Optimisation

Develop a more sophisticated optimisation module for rainwater harvesting and wastewater recycling tank sizes.

Monte Carlo risk based approach

The Monte Carlo approach could be integrated into the software as an additional option to calculate uncertainty in the results. The Monte Carlo simulations would be an additional option and not the default as the simulations should only need to be run once and would be quite time consuming as numerous runs would be required. The use of the

Monte Carlo approach would also require additional data about the uncertainty of the input data which would further increase the work of data gathering.

Automatic calculation of centroids

By automatically calculating the centroids from the input vertices files, the data input load would be reduced. A subroutine was written for this but requires some debugging.

Historical river flow data input

It would be interesting to see the effect of real flow data on the system rather than average flows estimated using Manning's Equation. These data are available from the Environment Agency for a small fee.

10.3 Final Statement

City Water Balance is an important contribution to the research of Theme 1 within the *SWITCH* Project, forming part of the *City Water* decision support system which is a comprehensive toolkit that allows decision makers to investigate the sustainability of a range of future possibilities for urban water systems, so that they can make informed decisions about managing the systems in a more integrated manner.

11 – Glossary

Black water – wastewater that contains faecal matter and urine

Detention Basin - open-air basin that is used to store stormwater during a rainfall event

District Metered Area – water company designated area where water supply is metered.

Embodied Energy - the quantity of energy required by all of the activities associated with a production process, including all activities upstream such as the acquisition of natural resources and the share of energy used in making equipment and in other supporting functions i.e. direct energy plus indirect energy

Exergy – maximum useful work possible during a process that brings the system into equilibrium with a heat reservoir. This is often considerably less than the available energy as a result of inefficiencies in the process

Field Capacity –soil moisture content above which water will drain by gravity

Filter Strip - vegetated area over which stormwater flows as overland sheet flow, normally adjacent to a drainage channel. A filter strip is designed to accept runoff from upstream developments and to treat runoff by vegetative filtering and promote settling of pollutants and infiltration

Green Roof - roof that has a covering of vegetation and soil over an impermeable membrane. Designed to provide water attenuation and wildlife habitat.

Greywater - wastewater that is not highly contaminated e.g. from bathing or laundry

Hydraulic Conductivity – measure of the ease with which a medium transmits water; the greater the hydraulic conductivity the more easily water is transmitted.

Life Cycle Assessment – analytical tool for modelling environmental impacts associated with a product or service over its whole life cycle, i.e. "from cradle to grave"

Life Cycle Inventory – phase of Life Cycle Assessment involving compilation of inputs and outputs for a product/service throughout its life cycle

Mains supply – potable water supply

Minicluster – *CWB* model concept: a land area or neighbourhood with one or more identical unitblocks assigned to it. In addition there may be road area and public open space.

Monte-Carlo Simulation – a technique involving many model simulations, using parameter values selected randomly given defined probability distributions for each value, to establish the range of results possible based on the uncertainty of the model parameters.

Net Present Value – the present value of predicted future cash flows minus the cost.

Present value - the current value of one or more future cash payments, discounted at an appropriate interest rate

Residual Soil Moisture – minimum water content of the soil. Water is held by attraction to soil particles.

Retention Pond – a best management practice (BMP) that is used to manage stormwater runoff to prevent flooding downstream. It is designed as a permanent water body.

Soakaway - subsurface reservoir, filled with coarse aggregate, designed to store stormwater flows and subsequently infiltrate them

Soil Moisture Deficit – difference between the capacity of the soil to hold water and the current moisture level, expressed as a depth

Subcatchment – *CWB* model concept: area of cityscape containing a network of foul or combined sewers that drain to a point at the downstream boundary

Sustainable Development – defined in the Brundtland Report as "development that meets the needs of the present without compromising the ability of future generations to meet their own needs"

Sustainability indicator – a piece of information that indicates that a system is moving towards or away from sustainability

Sustainable Urban Drainage Systems - Surface water drainage methods that take account of quantity, quality and amenity issues

Swale - vegetated open-air drainage channel. The increased roughness of the swale, in comparison to conventional pipes, acts to attenuate stormwater flow. Infiltration and evaporation also contribute to stormwater attenuation.

Unitblock – *CWB* model concept: smallest spatial scale in *City Water Balance*, consisting of pervious and impervious area and a water demand profile

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13 - Appendices

Appendix 1 – Modelling Information

A1.1 Assigning Miniclusters attributes in ArcMap

After the landuse description, by minclusters, for the Birmingham case study was completed the miniclusters were transferred from layers of distinct land uses into separate subcatchment layers. The following fields were added to the attribute tables of each land use type layer: Area, UB_Type, X_centroid, Y_centroid, Subcatch and MC_number. Within an “Edit session” the area and co-ordinates of the centroid of each polygon were calculated (with relevant rows and columns highlighted, right-click→ “Calculate Geometry”). In each layer the UB type is the same, so the appropriate ID number (Table 2) was added to all the polygons in the “UB_Type” Field, using “Field calculator”.

Subcatchment numbers were assigned to each MC in the following way:

a) A new layer was created for each subcatchment.

b) Fields were added in the attribute tables of the new subcatchment layers so that they identically matched those of the UB layers. On the map, all of the miniclusters in subcatchment 1 were highlighted (working round the edges first so that none were missed) and copy/pasted into “Subcatchment 1” layer. This was repeated for all subcatchments. An improvement to this method would be to write a program that takes the centroid of each MC and locates which subcatchment it is in.

c) Within each subcatchment layer, the subcatchment number was then assigned in the attribute table.

d) MC numbers were assigned by exporting the attribute table as a textfile and then importing the textfile into Excel. In Excel the MC numbers were assigned (the order does not matter within a subcatchment) and then directly copy-pasted back into the attribute table in ArcMap.

A1.2a – Calculation of daily potential evaporation

T_{max} and T_{min} (°C) values from the climate station were used to calculate the saturation vapour pressure in kPa (*FAO*, 1998a):

(1) $e° = 0.6108 * Exp(17.27 * T / (T + 237.3))$

The average saturation vapour pressure is:

(2) $e_{s\_av} = [e°(T_{max}) – e°(T_{min})] / 2$

Actual vapour pressure (kPa) at mean daily air temperature is (*FAO*, 1998a):

(3) $e_a = 0.6108 * Exp(17.27 * T_{wet} / (T_{wet} + 237.3)) – 0.0008 * (T_{dry} – T_{wet})$

The dry bulb temperature is the temperature of air measured by a thermometer freely exposed to air but shielded from radiation and moisture (*The Engineering Toolbox,* 2010). The wet bulb temperature is the lowest temperature a wetted body will attain when exposed to an air current *(Sensor Glossary*, n.d.).

After converting wind speed (U) from knots to m/s (1 knot = 0.514 m/s), the total potential evaporation (mm/day) is (*FAO,* 1998a):

(4) $PE = 0.408 * \Delta * (1 - \alpha) * R_n / 1000 + \gamma * 900 / (T_{mean} + 273) * U * (e_s - e_a) /$

$(\Delta + \gamma * (1 + 0.34*U)$

R_n = net radiation received at the surface

α = surface albedo = 0.23 (*FAO*, 1998a)

U = wind speed at 2m in m/s

The psychometric constant, Psi, was read from Table 2.2 of the *FAO Crop Evapotranspiration document* (1998b). At 100m (height of Winterbourne Station is 131m AOD) γ = 0.067 kPa/°C. Delta is the slope of the saturation curve for water at mean air temperature (T_{100}) (*FAO*, 1998b):

(5) Δ (kPa/°C) $= 4098 * 0.6108 * Exp(17.21 * T_{100} / (T_{100} + 237.3)) / (T_{100} + 237.3)^2$

A1.2b VBA code

```
Option Explicit

'Declare variables:
Dim Day_current As Integer, Day_start As Integer, Day_counter As Integer, i As Integer, j As Integer,
marker As Integer, intDry As Integer, intDrywet As Integer, Counter As Integer, Difference As Integer
Dim wind As Double, drybulb As Double, wetbulb As Double, rainfall As Double, solar_rad As Double
Dim T100 As Double, dblDelta As Double, dblRH As Double, dblT_drywet As Double, dblSatvap As
Double, dblVap As Double, dblTdew As Double, dblPsi As Double, dblEvap As Double, dblTotal_evap As
Double, dblTotal_Evap_albedo As Double, dblU As Double, dblTdry As Double, dblTwet As Double
Dim dblRa As Double, dblGsc As Double, dblDr As Double, dblWs As Double, dblLat As Double,
dblSolardec As Double, dblKr As Double, dblTmax As Double, dblTmin As Double, Pi As Double, dblBoltz
As Double, dblOut As Double, albedo As Double

' dblRa - extraterrestrial radiation
' dblGsc - solar constant, MJ m-2d-1
' dblDr - inverse relative distance Earth-Sun
' dblWs - sunset hour angle, radians
' dblLat - latitude, radians
' dblSolardec - Solar decimation, radians
' dblKr - adjustment coefficient (from 0.16 on coast to 0.19 interior)

Public Sub Calc_daily_values_Click()

'Initialize variables:
        Day_current = 0 ' Day of year
        wind = 0 ' Daily average windspeed / knots
        drybulb = 0 ' Daily average drybulb temperature / C
        wetbulb = 0 ' Daily average wetbulb temperature / C
        rainfall = 0 ' Total daily rainfall / mm
        solar_rad = 0 ' Net solar radiation / KJ
```

```
        T100 = 0 ' Daily average temperature at 1m / C
        dblTmin = 50
        dblTmax = 0
        Day_start = Cells(2, 1)
        Day_current = Day_start
        Counter = 0
        Day_counter = 1

' Get daily average from hourly data:
        For i = 2 To 8547
                If Day_current = Cells(i, 1) Then
                        Counter = Counter + 1
                        Call Calc_daily(i)
                Else
                        'reset variables to zero at start of each twenty four hour run:
                        Cells(Day_counter + 1, 13) = Day_current
                        Cells(Day_counter + 1, 14) = wind / Counter
                        Cells(Day_counter + 1, 15) = drybulb / Counter
                        Cells(Day_counter + 1, 16) = wetbulb / Counter
                        Cells(Day_counter + 1, 17) = rainfall
                         Cells(Day_counter + 1, 18) = solar_rad
                        Cells(Day_counter + 1, 19) = T100 / Counter
                        Cells(Day_counter + 1, 20) = dblTmax
                        Cells(Day_counter + 1, 21) = dblTmin

                        If Cells(i, 1) > Day_current + 1 Then 'in case of data gaps
                                Difference = Cells(i, 1) - Day_current - 1
                                For j = 1 To Difference
                                        Day_counter = Day_counter + 1
                                        Cells(Day_counter + 1, 13) = Day_counter
                                        Cells(Day_counter + 1, 14) = wind / Counter
                                        Cells(Day_counter + 1, 15) = drybulb / Counter
                                        Cells(Day_counter + 1, 16) = wetbulb / Counter
                                        Cells(Day_counter + 1, 17) = rainfall
                                        Cells(Day_counter + 1, 18) = solar_rad
                                        Cells(Day_counter + 1, 19) = T100 / Counter
                                        Cells(Day_counter + 1, 20) = dblTmax
                                        Cells(Day_counter + 1, 21) = dblTmin
                                Next j
                        End If
                        Day_current = Cells(i, 1)
                        Counter = 1
                         Day_counter = Day_counter + 1
                        dblTmin = 50
                        dblTmax = 0
                        wind = 0
                        drybulb = 0
                        wetbulb = 0
                        rainfall = 0
                         solar_rad = 0
                        T100 = 0

                         Call Calc_daily(i)
                End If
        Next i
End Sub

Public Sub Calc_daily(i)
       wind = Cells(i, 3) + wind
       drybulb = Cells(i, 4) + drybulb
```

```
            wetbulb = Cells(i, 5) + wetbulb
            rainfall = Cells(i, 6) + rainfall
            solar_rad = Cells(i, 7) + solar_rad
            T100 = Cells(i, 10) + T100
            ' get daily max/min temperatures:
            If dblTmin > Cells(i, 9) Then dblTmin = Cells(i, 9)
            If dblTmax < Cells(i, 8) Then dblTmax = Cells(i, 8)
End Sub

Function Acos(X As Double) As Double
             Acos = Atn(-X / Sqr(-X * X + 1)) + 2 * Atn(1)
End Function

Private Sub cmdEvaporation_Click()
Dim L As Double 'Latent heat of vapourisation for water
Dim Tmean As Double 'average daily temperature

L = 2260 'kJ/kg (Wikipedia)
Pi = 3.14159265358979
dblPsi = 0.067 ' at 100m elevation
albedo = Cells(13, 34)
Day_current = 0
wind = 0
drybulb = 0
wetbulb = 0
rainfall = 0
solar_rad = 0
T100 = 0
i = 0
j = 0
dblDelta = 0 ' Slope of the saturation curve for water at mean air temperature kPa/C
dblGsc = 0.082
dblLat = 0.92
dblKr = 0.17
dblBoltz = 4.903 * 10 ^ (-9)

For i = 2 To 367
             'set parameters to zero:
             dblTotal_evap = 0
             dblEvap = 0
             dblVap = 0
             dblSatvap = 0
             dblTdew = 0
             intDry = 0
             dblT_drywet = 0
             dblRH = 0
             T100 = 0
             dblDelta = 0
             dblTmin = Cells(i, 21)
             dblTmax = Cells(i, 20)
             Tmean = (dblTmin + dblTmax) / 2
              T100 = Cells(i, 19)
              dblDelta = 4098 * 0.6108 * Exp(17.21 * Tmean / (Tmean + 237.3)) / (Tmean + 237.3) ^ 2

             'Calculate relative humidity dblRH --- dblT_drywet
              dblT_drywet = Cells(i, 15) - Cells(i, 16)
             ‘separate decimal and integer part
             'get dry bulb temperature into integer multiples of 2:
             intDry = Cells(i, 15) / 2
             intDry = Round(intDry) * 2
```

```
        If intDry < 2 Then intDry = 2
        If dblT_drywet < 0 Then
                dblRH = 100
        Else
                If dblT_drywet <= 1 Then
                        dblRH = Worksheets("RH_table").Cells((intDry / 2 + 1), 2)
                Else
                        If dblT_drywet <= 2 Then
                                dblRH = Worksheets("RH_table").Cells((intDry / 2 + 1), 2) +
                                        (Worksheets("RH table").Cells((intDry/2+1), 3) –
                                        Worksheets("RH table").Cells((intDry/2+1), 4))* _
                                        (dblT_drywet – 1)
                        Else
                                dblRH = Worksheets("RH_table").Cells((intDry / 2 + 1), 3) +
                                        (Worksheets("RH table").Cells((intDry/2+1), 3) –
                                        Worksheets("RH table").Cells((intDry/2+1), 4))* _
                                        (dblT_drywet – 2)
                        End If
                End If
        End If
        If dblRH > 100 Then dblRH = 100
        Cells(i, 25) = dblRH / 100

        'Mean daily saturation vapour pressure:
        dblSatvap = ((0.6108 * Exp(17.27 * dblTmin / (dblTmin + 237.3))) + (0.6108 * Exp(17.27 * _
                        dblTmax / (dblTmax + 237.3)))) / 2
        dblTwet = Cells(i, 16)
        dblTdry = Cells(i, 15)

        'FAO actual vapour pressure:
        dblVap = 0.6108 * Exp(17.27 * dblTwet / (dblTwet + 237.3)) - 0.0008 * (dblTdry - dblTwet)

        'to get dewpoint temperature put dblVap as new saturation vapour pressure:
        dblTdew = 237.3 * Log(dblVap / 0.6108) / (17.27 - Log(dblVap / 0.6108))

        'Calculate evaporation:
        'Convert windspeed in knots to m/s; 1 knots = 0.514444444 meters per second:
        dblU = Cells(i, 14) * 0.514444444

        'Calculate the evaporation as a result of convection and vapour pressure defecit
        dblEvap = 0.35 * (1 + dblU / 100) * (dblSatvap - dblVap)

        'FAO Penman-Monteith equation (divide the radiation 'cells(i,18)' by 1000 to convert to MJ):
        dblTotal_evap = (0.408 * dblDelta * Cells(i, 18) / 1000 + dblPsi * 900 / (Tmean + 273) * dblU * _
                        (dblSatvap - dblVap)) / (dblDelta + dblPsi * (1 + 0.34 * dblU))
        dblTotal_Evap_albedo = (0.408 * dblDelta * (1 - albedo) * Cells(i, 18) / 1000 + dblPsi * 900 / _
                (Tmean + 273) * dblU * (dblSatvap - dblVap)) / (dblDelta + dblPsi * (1 + 0.34 * dblU))

        Cells(i, 27) = dblTotal_evap
        Cells(i, 28) = dblTotal_Evap_albedo
Next i
End Sub
```

A1.3 – Isolation of DMAs within the study area bounds and assignment of miniclusters to DMAs

Two new layers were created: one to hold the DMAs that are within the study area and one to hold the MCs in the study area. “The layers were “joined” to their appropriate source layer in order to create the right headers in the attribute table of the target layers. All DMAs outside the study area were removed from the target DMA layer.

A program was written that identifies which DMA the centroid of each MC lies in. A value of 0 was assigned to MCs not within the bounds of any DMA. The results were copy pasted into the attribute table in ArcMap and all the “0s” were filtered out. Since the MC boundaries are not clipped to the DMA boundaries there were still gaps.

The MC numbers were no longer sequential and had to be re-labelled in the textfiles and in the attribute table.

Appendix 2 - Contaminant Load Results

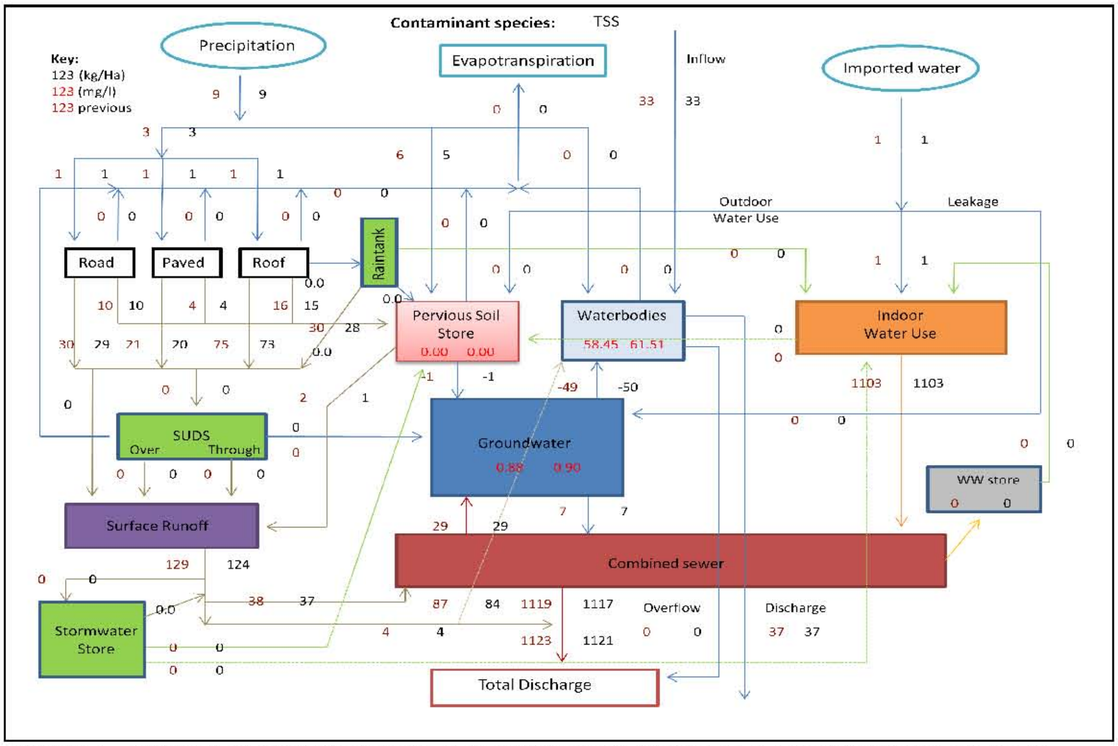

Figure 1. Business-as-usual 2009 (red). 2055 climate data applied to the 2009 Birmingham model (black).

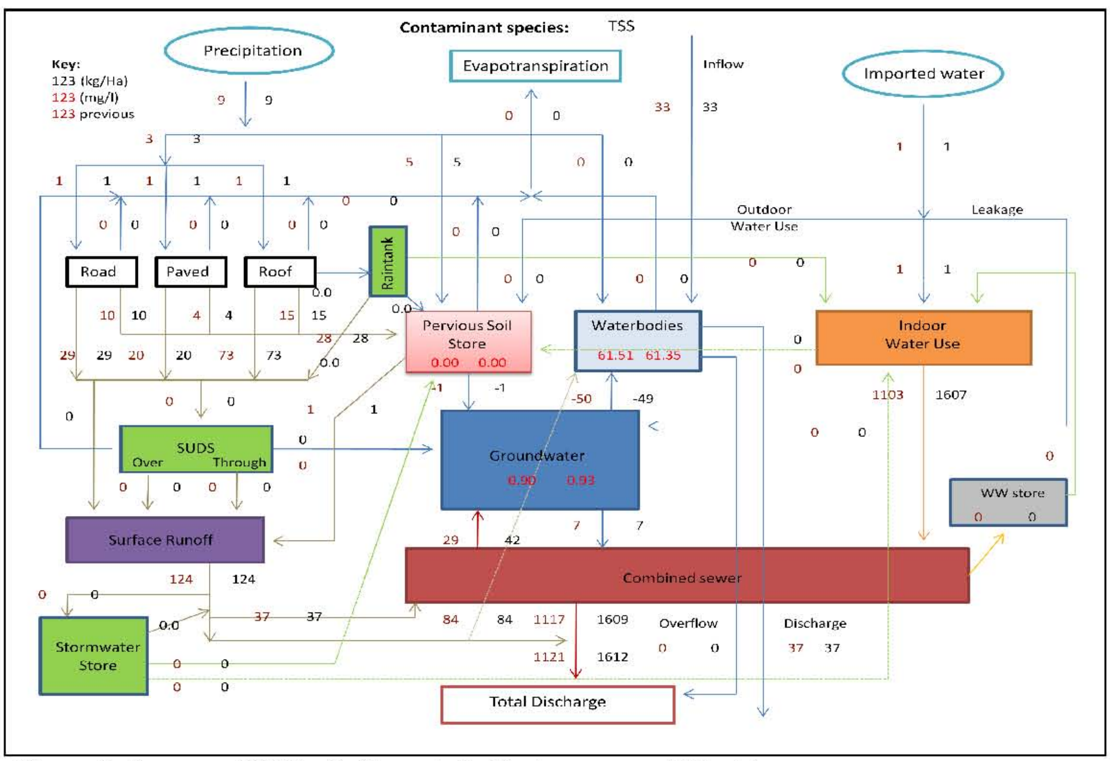

Figure 2. Base case 2055 (red). Scenario 1 –Business as usual (black).

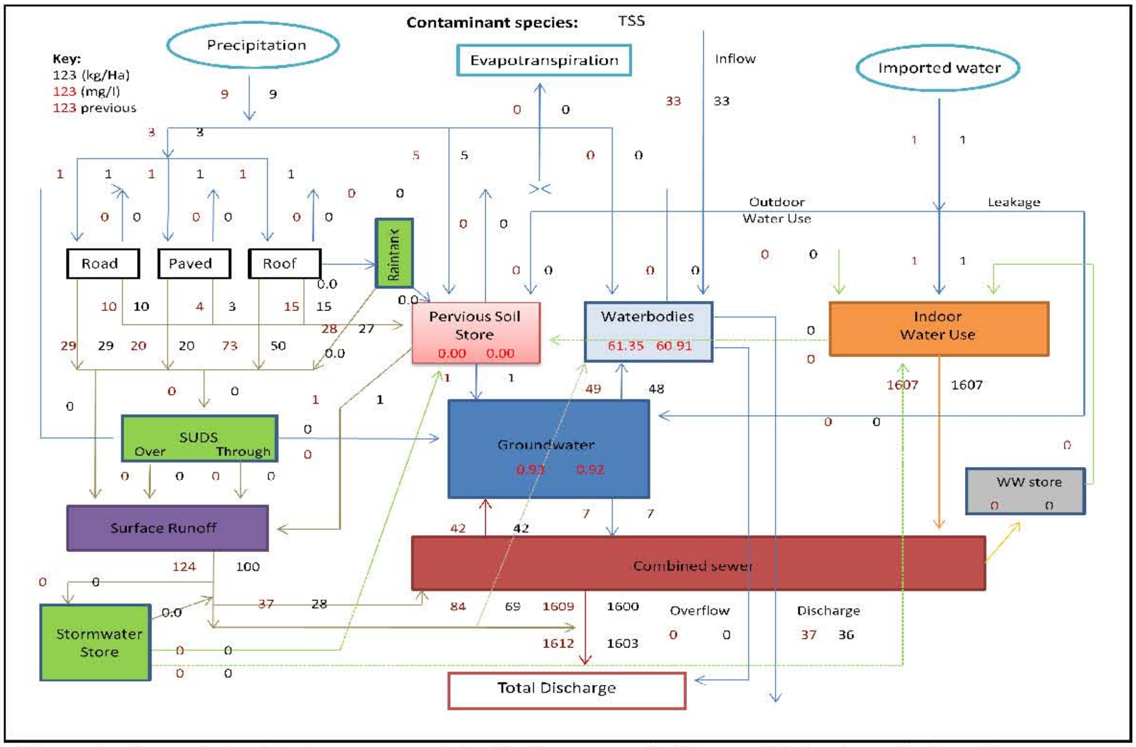

Figure 3. Scenario 1 – Business-as-usual (red). Brown roofs (50 year lifetime) applied to all non-residential buildings (black)

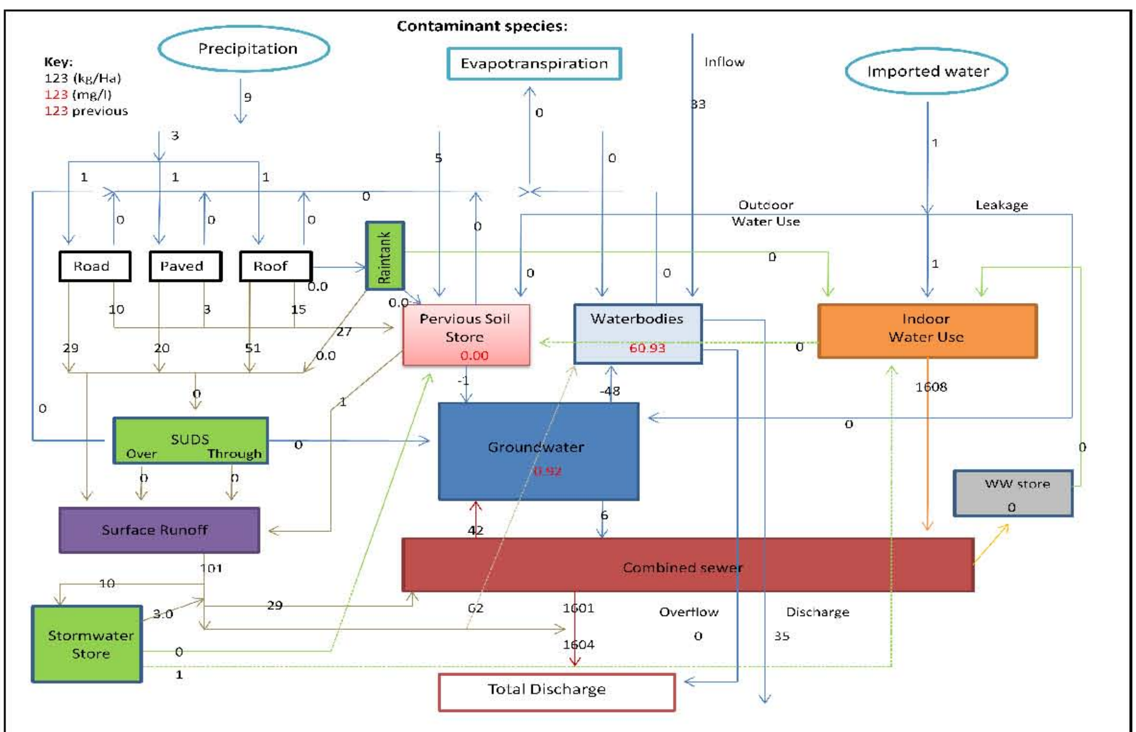

Figure 4. Scenario 1 – Brown roofs applied to all non-residential buildings and RH using 40m^3 concrete tanks with lifetimes of 25 and 50 years; 5 toilets/ unitblock.

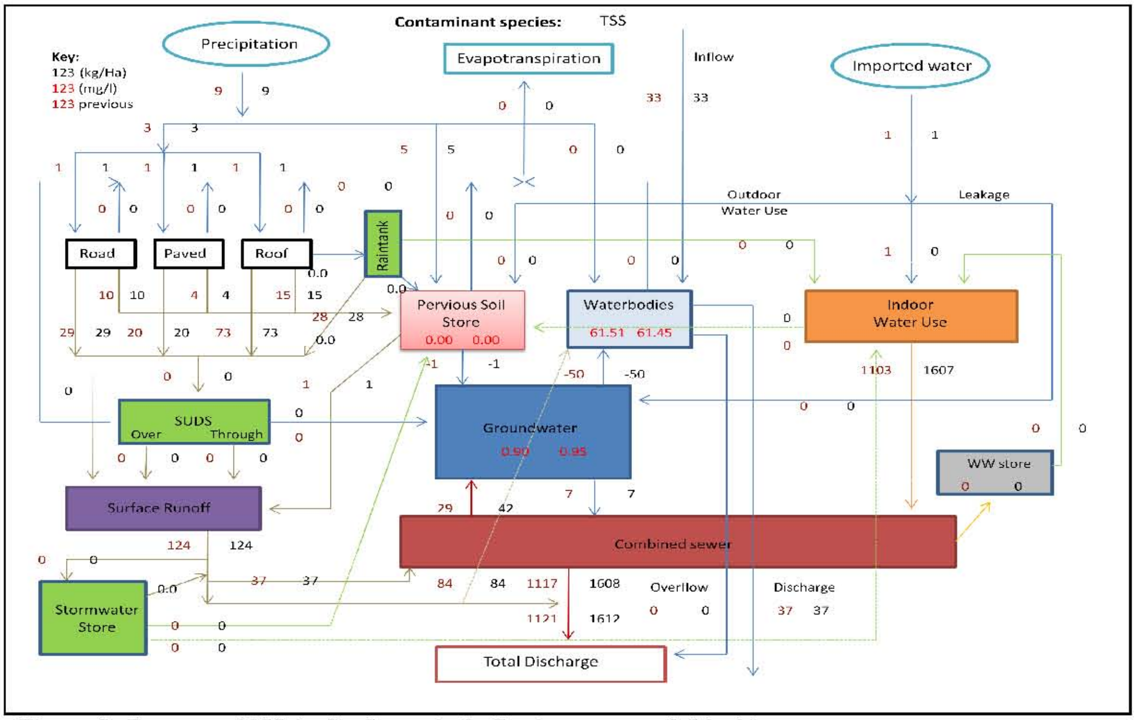

Figure 5. Base case 2055 (red). Scenario 2 –Business as usual (black).

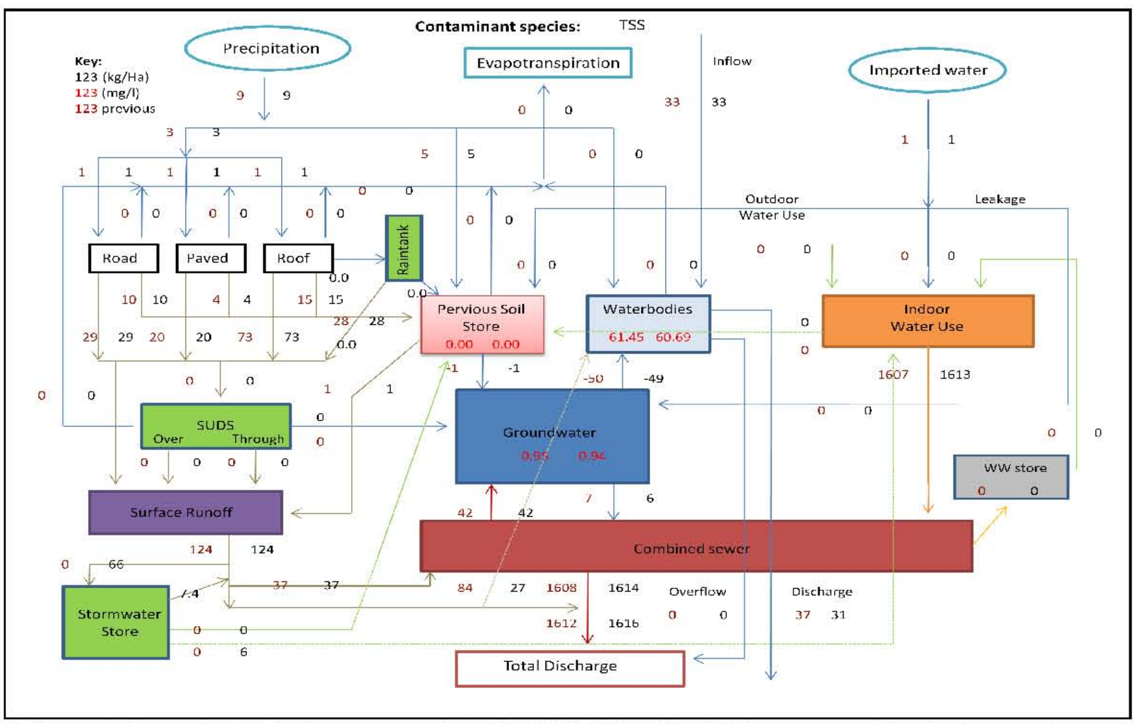

Figure 6. Scenario 2 – Business-as-usual (red). MC RH in all miniclusters supplying indoor use 1, garden and POS irrigation demands (black).

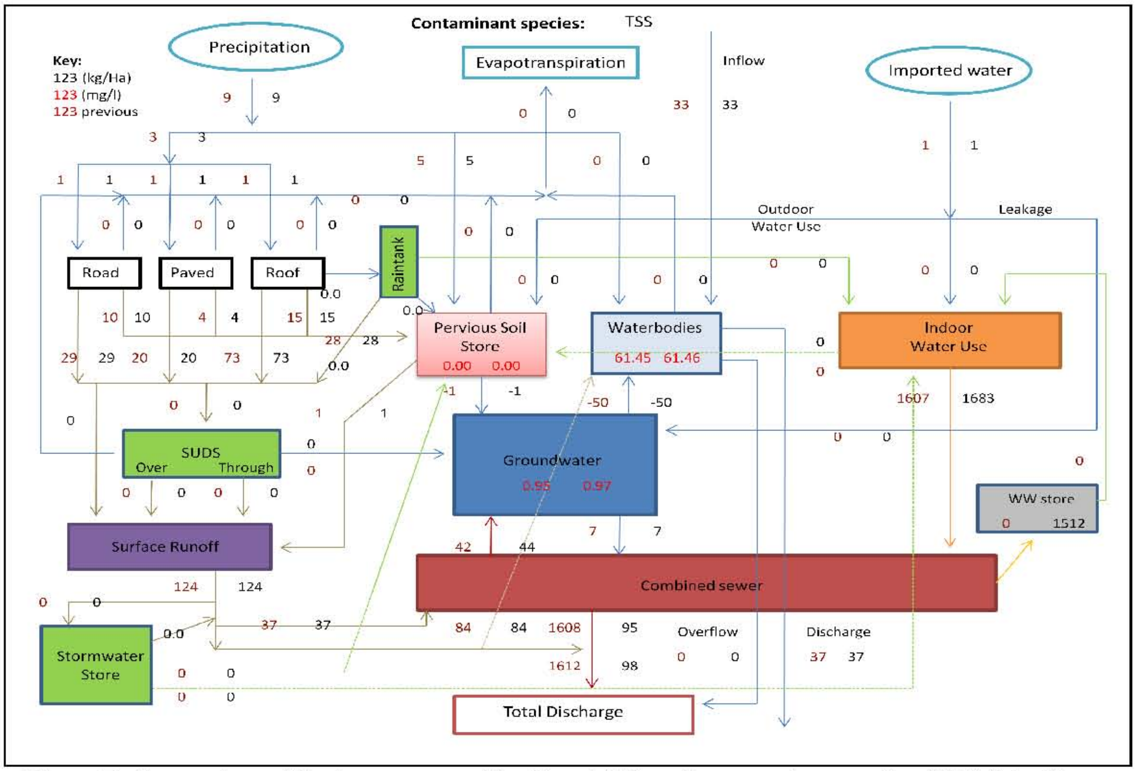

Figure 7. Comparison of business-as -usual (red) and MC scale greywater recycling (GR) (black).

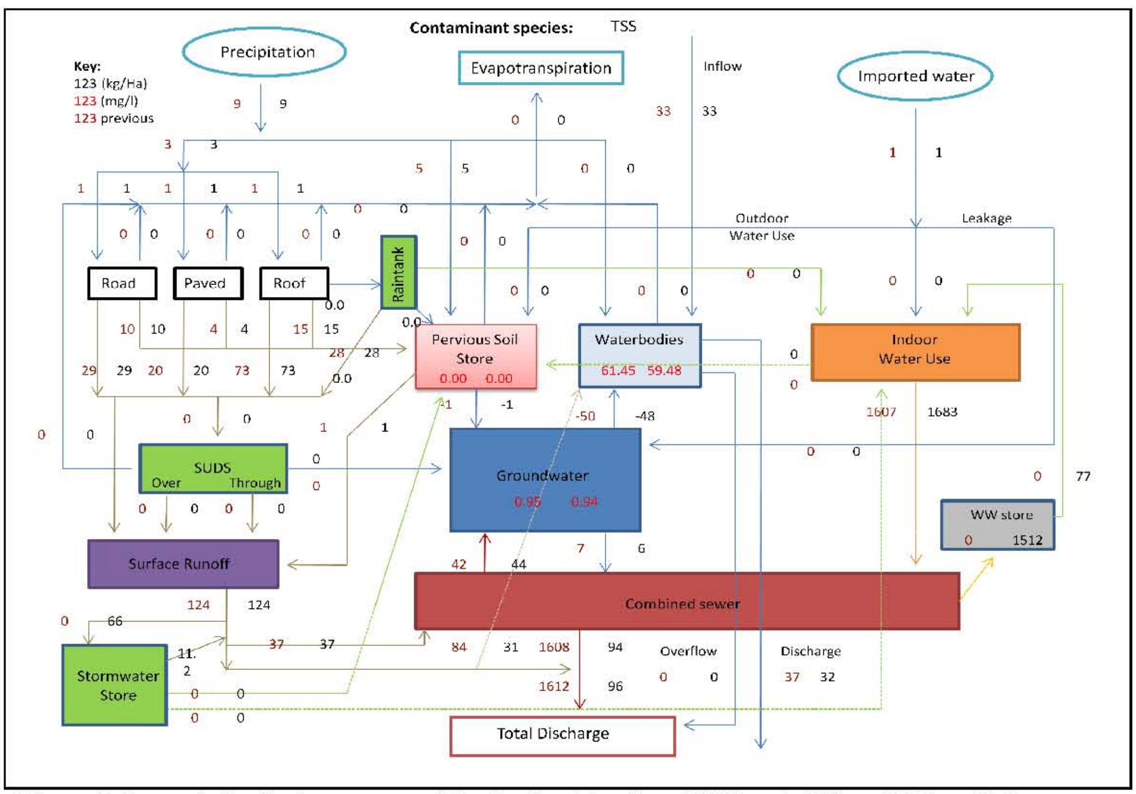

Figure 8. Scenario 2 – Business-as-usual (red). Combination of MC scale GR and RH applied across the study area, supplying Indoor Use 1 and garden and POS irrigation (black).

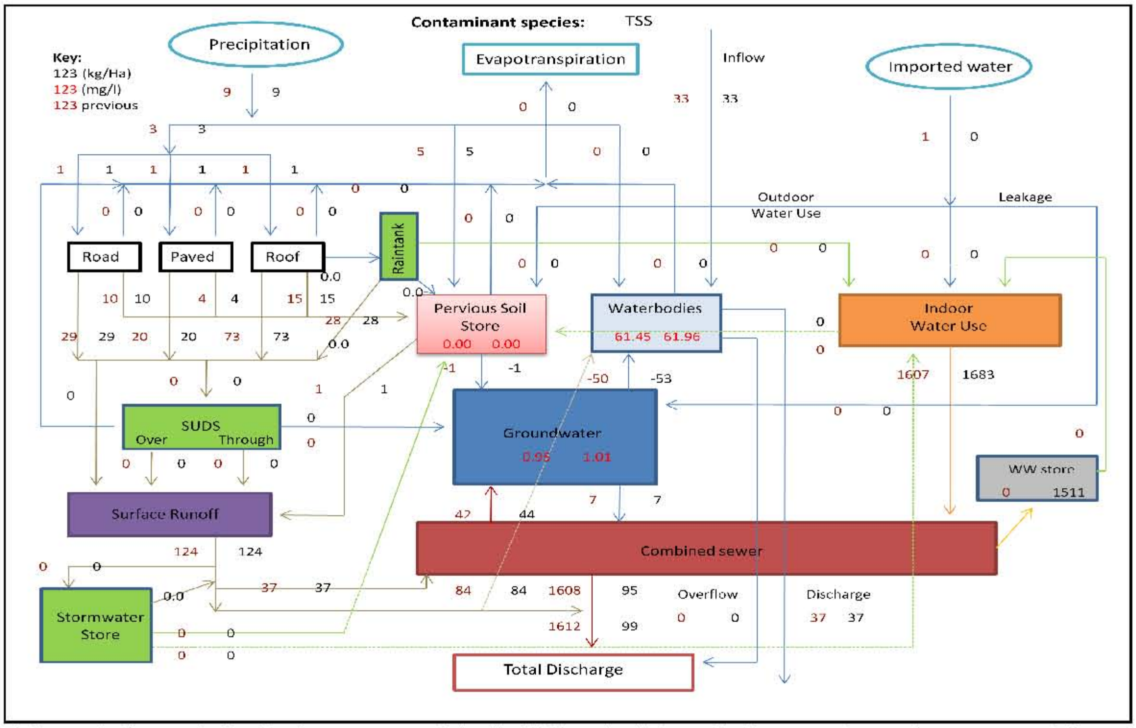

Figure 9. Scenario 2 – Business-as-usual (red). MC scale GR applied across the study area, supplying Indoor Use 1 and garden and POS irrigation. Mains supply is supplemented by large-scale borehole abstraction (black).

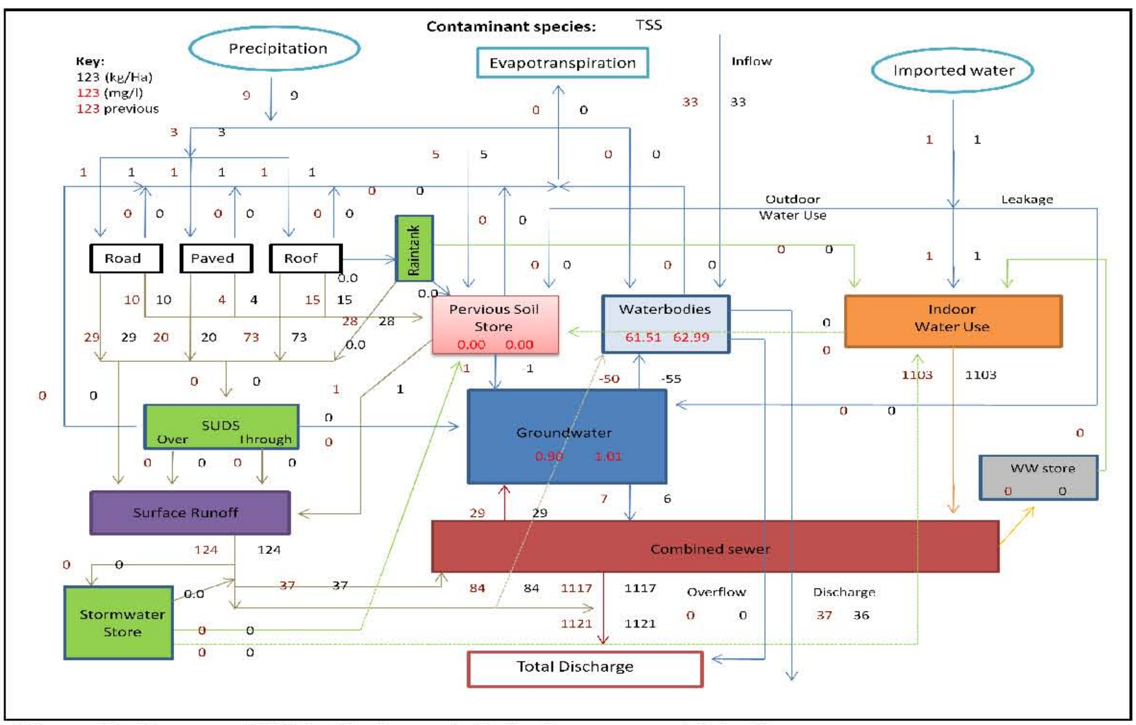

Figure 10. Base case 2055 (red). Scenario 3 –Business as usual (black).

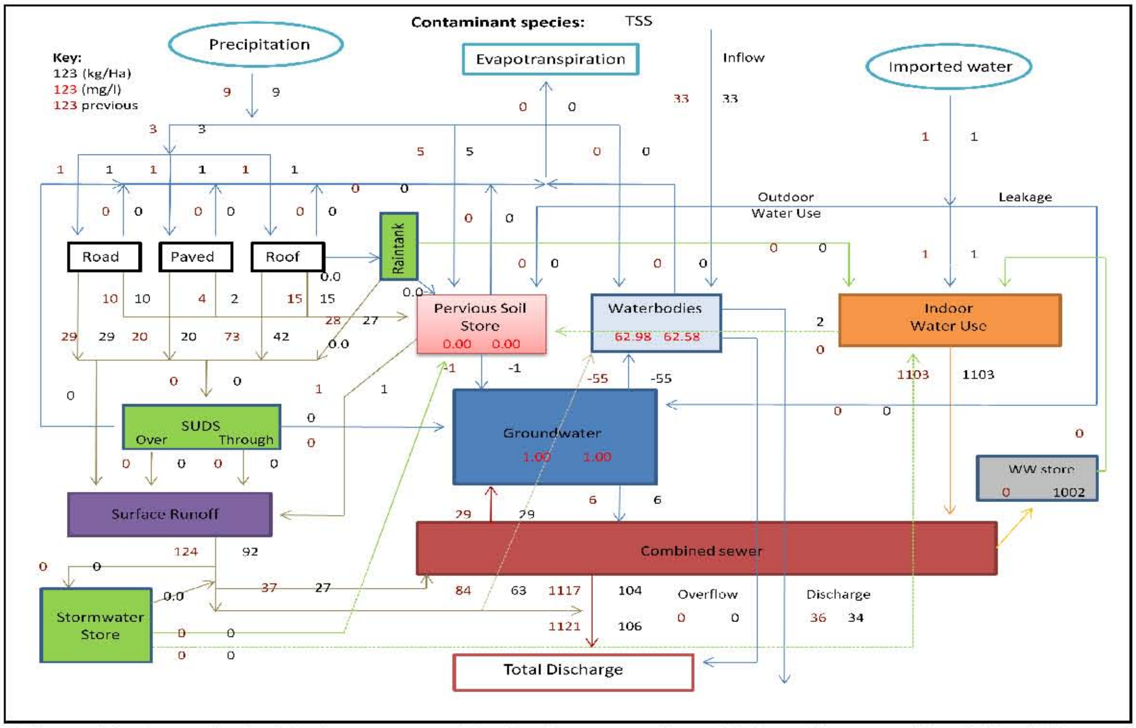

Figure 11. Scenario 3 - Business-as-usual (red). Brown roofs applied to all non-residential buildings and to high-rise, apartments, old people's homes and detached residential buildings. Supply irrigation to pervious stores with Large -scale greywater reuse (black).

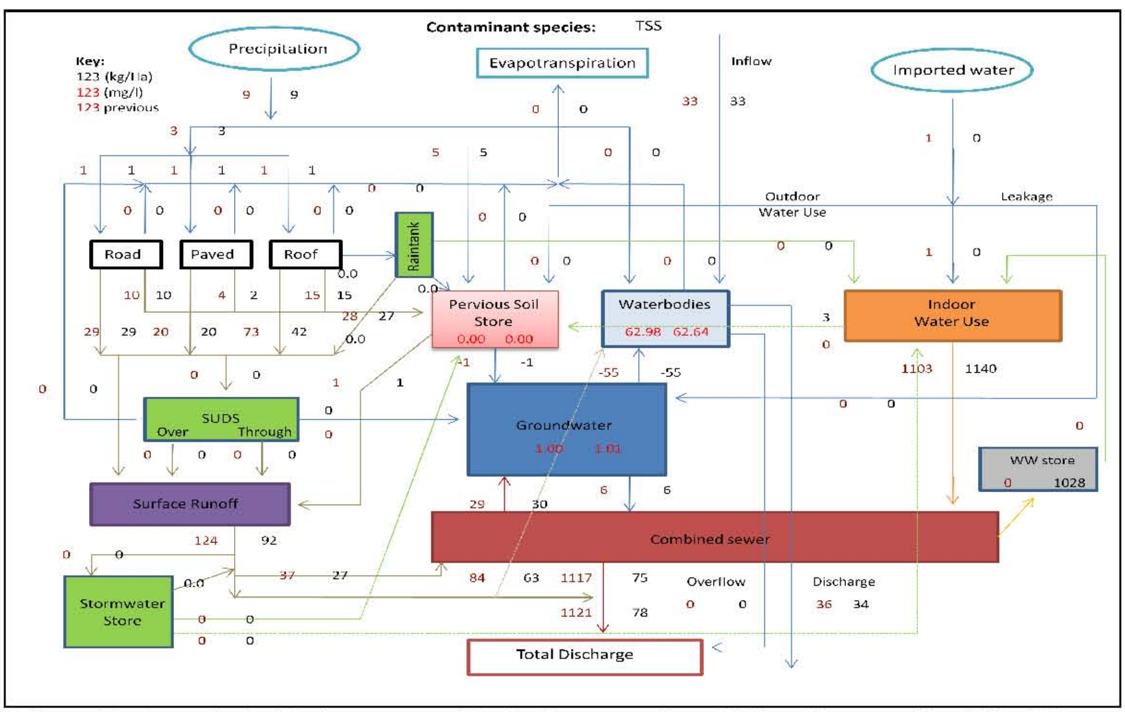

Figure 12. Scenario 3 - Business-as-usual (red). Brown roofs applied to all non-residential buildings and to high-rise, apartments, old people's homes and detached residential buildings. Supply irrigation to pervious stores and Indoor Use 1 with Large-scale greywater reuse (black).

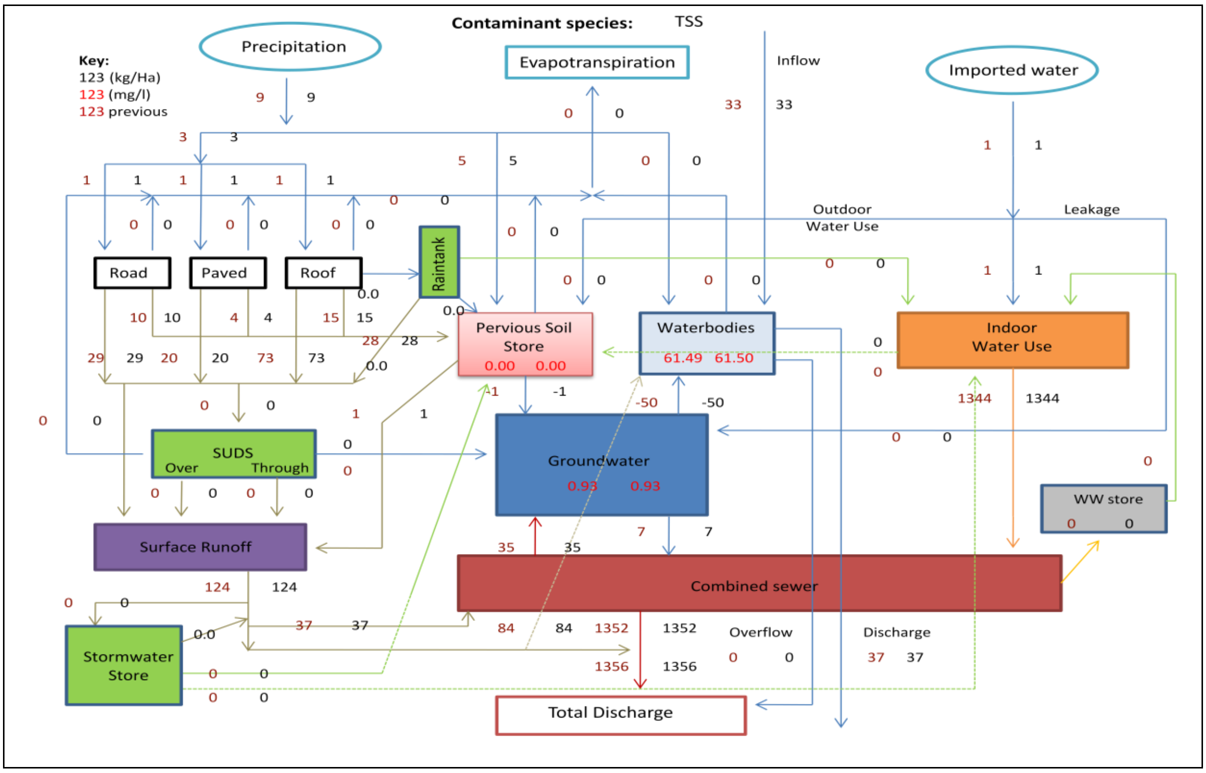

Figure 13. Standard appliances in residential houses (red). Water efficient residential appliances (black).

Appendix 3 – SWITCH Project Research Themes

Urban Paradigm Shift

1.1 Development of a strategic approach and of indicators for sustainability and risk assessment.
1.2 Modeling of urban water systems and the development of a decision support system.
1.3 Integration of existing infrastructure.
1.4 Strategic planning, implementation and performance assessment.

Storm Water Management

2.1 Technological options for storm water control under conditions of uncertainty.
2.2 Decision-making processes for effective urban stormwater management.
2.3 Environmental change studies for stormwater control and reuse options.

Efficient Water Supply and Use

3.1 Demand management for optimization of urban water services.
3.2 Safe water reuse.
3.3 Urban water supply and use - other productive reuses.

Waste Water

4.1 Eco-sanitation and decentralized waste water management in an urban context.
4.2 Management of industrial emissions.

Urban Water Planning

5.1 Urban Waterscapes - Planning and development in urban transformation processes.
5.2 Use of urban water (fresh and wastewater) for urban agriculture and other livelihood opportunities.
5.3 Maximising the use of natural systems in all aspects of the municipal water cycle.

Governance and Institutions

6.1 Governance for integrated urban water management
6.2 Learning alliances
6.3 Optimising social inclusion
6.4 Financing, cost recovery and institutional models

(*UNESCO-IHE*, 2007)

Appendix 4 - *City Water* Technical Outline

A4.1 *City Water* Tools

Web Interface

Operation:

The Web Interface is a web based platform that provides access to the *CITY WATER* tools and models. It is connected to the GIS database and has the capability to gather the basic information required from stakeholders and personnel involved using an online questionnaire in order to identify the problem and translate it into a workable outline. When all objectives are listed and verified, the model of the existing system will be analysed. The results will be visualized using an automated problem tree analysis diagram which also enables simulation of the existing situation. This information forms the basic input for setting up scenarios and strategies. This web interface also provides an on-line form of voting that can be used by stakeholders to analyse all relevant strategies by comparing their benefits with those of the existing situation and then selecting the preferred strategies. The web interface also enables the translation of all model results into an understandable form using graphs, GIS maps and animations. Furthermore, the web interface also provides stakeholders and others, an interactive platform for discussion and negotiation such as the Internet, a short message service, instant messenger, electronic meeting, e-voting, e-brainstorming and video conferencing, all of which assist the decision-making process.

Input:

The input for the web interface is basically the answers from stakeholders and other personnel involved on the online questionnaire and online voting form. In addition, topological data is also required such as city area, river network, water bodies etc., and hydrological data such as precipitation, temperature etc. The data enable the simulation of the existing situation and also simulations of futures scenarios to be carried out.

Output:

The output from the web interface take the form of various graphs, reports, GIS maps, simulations and animations.

The Common Database

Operation

The different tools and models included in the City Water platform require data including: inputs, outputs and parameters for models, spatial locations for the GIS viewer, group and interrelations information for the information system on the water system, opinions and other participatory contributions for the knowledge management system, indicators values, multimedia documents and, general information and metadata about the various elements. Some of these tools and models make use of the same information, and they produce results that should be in similar formats in order to be compared and easily readable by users. The common geo-database provides a unified common means of storing all the data needed for City Water.

The common database will not provide direct outputs to end users, as data is to be provided through the different tools, such as GIS Viewer. New information, such as legal documents, opinions or model results can be added and stored into the database through the use of different tools. Administrative tasks, such as importing data from other sources, managing users' rights, modifying tables' structures, generating backups or performing other maintenance operations, are to be performed by specialists.

The data are organised along three main axes: system elements, information and uncertainties:

- System elements are objects, groups of objects or interrelations that play a role in the water-related system, for instance: a river, an industry, a navigable zone, a wetland, a swimming pool, a monitoring station, a group of water stakeholders, available funds, tools like database or models, administrative boards, water quality standards, policies regarding ecosystems, etc.
- Information about system elements is separate from the former. Such information may indeed be abundant or absent, and take different forms such as textual descriptions, multimedia document, detailed numeric time series values, aggregated values (indicators), probability distribution functions, etc.
- Uncertainties documentation may refer to several system elements (e.g. models used, experts involved in assessment, monitoring stations which produced the data, etc.) and/or values (e.g. models parameters, kind of probability distribution function, confidence intervals, fuzzy numbers, etc.) to create background uncertainty information, called "uncertainty pedigree". These pedigrees can also include meta-information such as the degree of consensus, or the subjectivity part in a choice, or the level of confidence in the data. It applies to values (e.g. indicator value) or to system elements (e.g. model, causal relation between elements).

GIS Viewer

Operation:

The main function of a GIS viewer is to provide a visual access to spatial data and spatial indicator stored in the geo-database. It allows the user to explore the various features of the city's area and acquire an intimate knowledge of it. Editing of spatial data is also important if the tool is to be used to capture all kinds of inputs from the end users (additional data, subjective preferences, weighting factors, etc.).

Exploring new spatial indicators on the basis of existing data is another function that will be added to the GIS viewer. Finally, the GIS viewer will also provide some tools to link the spatial data layers to the modelling tools (City Water Balance, City Water Drain, etc.) as input data.

Input:

Spatial data layers in the geo-database

Output:

Spatial data layers in the geo-database, Maps.

Indicators Viewer

Indicators Viewer Operation:

The Indicators viewer displays indicators and data which are stored in the geo-database. However, unlike the GIS viewer, it does not allow users to consult spatial information. The indicators viewer is designed to provide graphs, charts and tables which may represent data such as performance assessment results, time series values/indicators or confidence intervals. Furthermore, the tool has several additional functions:

- Creation of new indicators by combining existing data (e.g. time aggregation or combined indicators)
- Data export under excel and image formats.

Indicators Viewer Input:

The Indicator viewer does not require any additional inputs besides the selection of data or combined data to display. It uses the data stored in the geodatabase (except geographic information): e.g. city data, scenarios data, strategies data, model outputs, indicators results.

Indicators Viewer Output:

The outputs of the indicator viewer are:

- Graphs / charts (export in “jpg” or excel format)
- Tables and values (export in excel format)

A4.2 *City Water* Models

City Water Vision

Operation:

City Water Vision is an interactive tool to assist stakeholders in exploring urban water issues and scenarios. In this module, stakeholders are guided to define the problem in the city using an online questionnaire. The results from this questionnaire are presented in an online report and also in a problem tree analysis diagram. Using these outputs, stakeholders will define the vision for the city and then try to set up scenarios and strategies to overcome the problems. All relevant strategies will be analysed by stakeholders by comparing their benefits with existing situation using an online voting and weighting form. The result will show which strategies get the most votes using various types of graphs and this result can be used to help stakeholders selecting the preferred strategies. All the manual process in recent visioning exercises such as voting and weighting scenarios and strategies are automated in this module.

Input:

The input for City Water Vision is the answer from the online questionnaire and online voting and weighting form from different stakeholders.

Output:

The output will be the online report and problem tree analysis diagram of problems in the city, the visions of different stakeholders and also various types of graph to present the strategies and their weights after the voting process.

City Water Balance

Operation:

City Water Balance is a scoping model that assesses the sustainability of a city's water system using indicators. Central to the model is the water balance which is calculated using meteorological data on a daily time-step. The study area is divided into clusters based on common land use and watersheds. Within each cluster, a generic unit is defined that represents the consumption patterns corresponding to the land use of that cluster. Each cluster comprises of one or more unit blocks. The user has the option to model various water management strategies to investigate their effect on the sustainability indicators. The strategies include direct greywater irrigation, stormwater re-use, wastewater recycling, green roofs, sustainable urban drainage systems, canal/river abstraction and borehole abstraction. The model can also compute a daily water quality balance which provides annual, monthly or daily water quality data for any part or the entirety of the system. For each water management strategy, life cycle energy use and life cycle cost (net present value) will be calculated. The user will also be provided with a clear breakdown of the energy use and cost.

Input:

The inputs requirements for City Water Balance are meteorological and land use data. For each land use, water usage profiles and average occupancy need to be supplied. The study area is characterised in terms of impermeable and permeable surfaces. Other input parameters include soil infiltration capacity, water quality data (water supply, precipitation, evaporation, roof first flush, input from bathroom, laundry, toilet and kitchen, and roof, road and paved area runoff), net present value costs and embodied energy values.

Output:

The model will output a suite of sustainability indicators to aid decision makers to choose the best strategy to address proposed future scenarios. The headline indicators from the model are: mains water used, stormwater runoff volume, waste volume, water quality, life cycle energy and life cycle cost at net present value.

City Water Drain

Operation:

City Water Drain is a model for the integrated assessment of interactions between elements of the urban drainage system; catchment runoff, sewers, treatment plants and receiving waters. The model enables block-wise modelling of different parts of the urban drainage system. Each block represents a system element (subsystem) with different underlying modelling approaches for hydraulic and mass transport. The study area is divided in to sub-catchments based on the urban drainage watersheds. Each sub-catchment includes part of the sewer system of the study area, any combined sewer overflow structures, and pumping stations. The model can determine frequency and volume of over flow from the combined overflow structures, flood volume at a sub-catchment level, and evaluate existing wastewater and stormwater drainage systems and their effect on the receiving water.

Input:

The input requirements for the City Water Drain model are a set of simple descriptors of the urban drainage system. For each sub-catchment; the sub-catchment area, the number of inhabitants, dry weather flow, percentage of imperviousness and pollutant types and flow concentrations are required. Other input data required include location, basin volume and maximum effluent flow of each combined over flow structures; location, basin volume, mean pumping rate, treatment plants treatment efficiency for each pollutant and their capacity. Rainfall records and flow time series of the receiving water river section are also required.

Output:

The model will output overflow volume and frequency of overflow from combined overflow structures, flood volume on sub- catchment level, concentration of pollutants discharging form the urban drainage system to the receiving water.

City Water Risk

Operation:

The first part of city water risk will be the analysis of global change pressures, and associated uncertainties. It will provide the drivers of city water changes and their distributions (e.g. urban water demand and supply, proportion of urban pervious and impervious area, breakage and failure rates of pipes etc.) in the form of probability or membership function. These results will be used as an input for the integrated urban water model to analyse water supply, waste water, drainage systems. The second part of the city water risk tool will use the output of this integrated model after processing the parameters provided from first part to determine the measure of the systems performance. These performance values will be processed and compared with the proposed or target performance level of the systems, and finally the risks of not meeting these objectives will be identified.

Input:

The inputs for city water risk will be the data and information related to the existing urban water systems and their elements; demographic features; demand patterns; hydrological information; water resources data; performance criterion of urban water systems and output from the integrated urban water model.

Output:

The output from city water risk will be

- the identification of the major uncertainties and their distributions due to the global change pressures in the form of graphs and tables; and
- risks of failures in systems performance (water deficits, water pressures, urban flooding, water quality etc) in the form of graphs.

City Water Futures

City Water Futures Operation:

City Water Futures is a tool to help stakeholders get a clear picture about each strategy before they choose the best and/or preferable one. This module has two coupled models, which are territorial model and social model. The territorial model will represent the physical and urban space and the social model will represent the agents. There are four central processes; climate data; hydrological data; the agents' decision and; the update of information in the end of time step. This processes is repeated for each strategy and for every scenario identified by stakeholders. The result will be shown graphically.

Input:

The inputs for City Water Futures are climate data such as temperature and hydrological data such as water level. This system also needs the decision from stakeholders for the scenarios identified and the set of parameters for each strategy.

Output:

The output for City Water Futures is basically several graphs that represent the final result.

City Water Strategy

Operation:

City Water Strategy is a tool designed for the creation and evaluation of strategies to cope with environmental, demographic and societal changes. It helps decision makers carry out rapid assessments of a range of technical and non-technical options and therefore assists the conception of relevant strategies to face the issues coming out of the scenarios. For this purpose, the tool has three main components:

1. A *strategy builder* to develop strategies and options in time and space.
2. A *performance assessment* tool for comparing strategies. It is based on cost-effectiveness and multi-criteria analysis using sustainability and performance indicators.
3. To close the loop, a *solution explorer and optimiser* exploits the performance assessment results to provide users with alternative options for strategy building.

Input:

The *City Water Strategy* tool requires several inputs to operate adequately:

- A set of indicators for benchmarking, with some values of references such as target or limit values. These indicators should match the objectives defined with the *City Water Vision*. The indicators are selected by the stakeholders.
- A list of options with their potential influence on each indicator is stored in the geo-database. The users may have to edit and add new options.
- The data that characterises the current situation of the city and the scenarios (obtained through the *City Water Vision* tool).

Uncertainty is taken into account at every stage of the process. However, to be consistent, users need to spend time to document any information uploaded into the geodatabase, according to the guideline provided with the tool.

The tool *City Water Strategy* takes automatically into account the results obtained through the *City Water models*.

Output:

The *performance assessment* tool provides users with the results of the sustainability and performance indicators (Multi-Criteria Assessment) and the results of the cost effectiveness analysis for the considered strategies/scenarios. Both are visualised with the *Indicators Viewer* as graphs, charts or tables. No direct ranking of strategies is provided.

Through a semi-automatic process, the *solution explorer and optimiser* searches the geo-database for alternative options to enhance the performance of each strategy.

City Water System

Operation:

City Water System provides water stakeholders with an application for consulting and managing water system information, through a system view including components (e.g. rivers, energy resources, administrative boards, standards) and interrelations (e.g. water consumption or monetary fluxes, any kind of influences). Its detailed capabilities are:

- Management:
    - Management of the water system view: add / organise / connect / delete water-related components and interrelations.
    - Management of the water system data: add / edit / delete data linked to components and interrelations.
- Consultation:
    - System view (complementary to the GIS view, addressing the three points mentioned above) including:
        - (i) Synthetic view, through the possible expansion of components groups (e.g. ecosystem) to display sub-groups, (e.g. lakes, wetlands), sub-sub-groups (e.g. white lake, fishy lake, muddy pond) and so on.
        - (ii) Interrelations, in the form of influences network (textual descriptions) or fluxes (e.g. water, pollutant, energy fluxes, monetary flows, data flows) with proportional arrows; possibility to highlight in colours causal networks (cascading influences).
        - (iii) Both geographic (e.g. rivers, wastewater treatment plant) and non-geographic elements (e.g. funds, stakeholders, policies and standards) in a schematic way.
    - Data browsing, related to components and interrelations, using:
        - (i) A pop-up information box for short information display, including hyperlinks to easily access related information.

(ii) A multimedia document visualiser.

City Water System extracts its information from a database. Therefore, relevant inputs into the database are crucial to provide users with useful outputs: complete and detailed system view and information for their water-related system.

City Water Economics

Operation:

The City water economics model aims at analysing alternative scenarios for the distribution and recovery of future strategy costs among households, water service providers, municipal authorities and society at large. Cost categories covered by the model include; investment costs for user equipment; water supply and treatment; wastewater collection and treatment; and stormwater management.

In a second stage, the model will offer the possibility to define, analyse and evaluate pricing schemes for financial cost recovery and to calculate subsidies required to determine whether on-site systems are an economically viable option. The analysis can be undertaken for the entire urban water supply system, or for subsystems such as stormwater management or wastewater collection and treatment, for the entire urban catchment or for specific geographic areas (clusters).

Input:

The model will use the detailed life cycle cost assessments performed by the City Water Balance Model, and the representation of the urban water system in terms of unit blocks, clusters and catchment. Additional user-input data requirements are water consumption and household income patterns, and existing cost recovery mechanisms and tariff structures for mains water supply, sewerage and stormwater management.

Output:

The main model output will be a set of indicators derived from the allocation of strategy costs, such as maximum and average share of household income spent on water services, and recovery of financial water service costs. Furthermore, the scenario analysis functionalities will provide a rough estimate of subsidies and tariff reforms required to support the implementation of the analysed strategies.

Appendix 5 - Publications

A5.1 Proceedings from the SWITCH Annual conference in Tel Aviv (2007)

Developing a New Scoping Model for Urban Water Sustainability

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Abstract

It was an early SWITCH decision to use the water and transport balance concepts underpinning the integrated urban water management scoping model Aquacycle and its successor UVQ as the basis for the development of a rapid urban water sustainability assessment tool. An Excel-based version of the new tool, provisionally referred to as Aquacycle plus, has been implemented and verified against aquacycle at each of the three scales of calculation – dwelling, cluster and city. Currently, a range of options are being investigated and implemented in the new model that reflect the need for better spatial articulation of the natural water systems across a city and the need for additional indicator outputs.

Improved flow and transport representations include the addition of cluster scale groundwater abstraction and groundwater flow, the introduction of canals and rivers and an enhanced sustainable urban drainage (SUDS) capability. A range of SUDS options based on a common model algorithm are being implemented and will include the evolving results from the extensive green roof trials currently taking place in Birmingham UK.

Two additional tools are being added to enable a more complete assessment of sustainability. They will provide indicator values based on whole life cycle energy use and life cycle cost predictions. Initial developments are being based on information collected for the Eastside regeneration scheme, Birmingham, but the model is being formulated to be generally applicable to all cities and all scales.

Keywords: urban water model, scoping model, sustainable

Introduction

The SWITCH paradigm shift in water management is from the conventional to the integrated in which all aspects of the urban water system are considered simultaneously by the stakeholder groups to develop a strategy that should be more sustainable. The demand for a more sustainable water system is now growing in the face of several factors including climate change, rising energy costs and rising population: all creating large adjustments to and pressures on the management of the available water reserves globally (notably in Australia and large parts of Africa) (*Lundin*, 2004). Food production and economic growth place the greatest stress on the water supply of a growing population (*Lundin*, 2004). The pressing need to address current practices to reduce their environmental impacts is also highlighted in the STERN Report (*Stern, N. et al.*, 2006; which addresses the economics of climate change). One estimate is that by the 2080's more than one billion people will suffer water shortages. Several climate models predict an annual runoff reduction of 30% for a temperature increase of 2ºC and 40-50% for a rise of 4 ºC (*Arnell*, 2006). In addition to water scarcity are the effects of increased intensity rainfall events: with a rise of 3-4 ºC the cost of flooding in the UK is estimated to increase from 0.1% GDP to 0.2-0.4% GDP.

The UK's water system is still largely based on designs from the early 1800s when cost-optimised sanitation was the priority and environmental effects were not considered. Health is clearly still the priority in water provision but technology has long been in place to achieve this and so attention has turned to the environmental effects. In order to address these, new designs need to be considered for the water system.

Under Theme 1.2 a new scoping model is being developed for the urban water system that will have the capability of addressing different scenarios (e.g. climate change, population increase) with a variety of sustainable technologies, outputting a suite of sustainability indicators. Fundamental indicators include mains water saved, wastewater emitted, runoff quantity, various contaminant emissions, life cycle energy and life cycle economic cost.

Birmingham, UK, will be used as a demonstration city for the model. Particular focus will be on a 170 hectare site east of the city centre, Eastside. It is currently undergoing physical regeneration which has sustainability as one of it's driving principles.

Aquacycle and UVQ

It was a SWITCH decision to use the concepts underpinning the Australian water balance program Aquacycle as a basis for developing the scoping model on a daily time-step (*Mitchell*, 2004). Aquacycle is a good example of a Life Cycle Analysis (LCA) predictive model that outputs a suite of sustainability indicators. It's successor, UVQ (Urban Volume and Quality) adds a contaminant balance to the water balance in addition to improving the interface and adding snow modelling capability.

Aquacycle performs a daily water balance with various water recycling options available to test their effect on the primary indicators – mains water used, stormwater runoff and wastewater emissions. The alternative strategies are raintanks, cluster stormwater systems, catchment stormwater systems, subsurface direct greywater irrigation, aquifer storage and wastewater recycling at unit, cluster and catchment level.

Spatial representation is as follows: the entire study area is called the catchment. Within the catchment are various smaller areas called clusters that are primarily delimited based on land use. Each cluster consists of a number of unit blocks –unit block characteristics are homogeneous within a cluster. The model outputs results at the three spatial levels (catchment, cluster and block) in terms of quantity and, in the case of UVQ, quality.

Aquacycle does not have the capability to address events that require modelling on a sub-daily timestep. For example, it cannot adequately describe flash flooding as it works in daily averages.

The new Scoping Model

Aquacycle' and UVQ' concepts and algorithms, based on information in their manuals, have been embedded, initially, in an Excel Workbook using the programming language Visual Basic for Applications (VBA). This programming environment is efficient and user friendly such that additional model capabilities can be developed, tested and demonstrated quickly in the initial stages of the model's construction.

Various steps have been taken to ensure that the new code is correct. Internal consistency has been assured by using mass balance checks for each component of the code (e.g. raintanks, garden area, public open space) and for the total water cycle.. Errors are of the order 10^{-9} m^3 for the whole model for simulations for a small catchment area of 100 ha for one year. Verification of the results against those for Aquacycle using the same input data has also been undertaken. Discrepancies between the models have been noted but these problems cannot be traced to errors in the implementation of the algorithms in the new code.

To find the cause for the discrepancies both models were run with simple test input data whose result could be checked manually. By way of an example a test calculation was performed for a one year time series of constant daily 1mm precipitation and 0.5mm evaporation for a garden area. The values adopted for the required input parameters were set to simplify the manual calculations (Table 1).

Table 1: Input parameters for code verification – Test 1 (Garden area runoff)

Days	365
PS1 capacity (m)	0.05
PS2 capacity (m)	0.5
PS1 proportion	0.5
Garden area (m^2)	100
Roof area (m^2)	100
No. blocks in cluster	10

The garden allows infiltration up to the capacity of its two stores PS1 (Pervious Store) and PS2, with no interaction between the two. Once the capacity is exceeded there is surface runoff. For the purpose of this verification recharge was set to zero and all roof runoff flows onto the garden without loss. The evaporation algorithm is shown in Figure 1. Since Precipitation is always greater than evaporation the level of the stores will never fall below half and so condition (2) will always apply and the evaporation will be a constant 0.5mm per day.

The simplified garden model yields a total runoff for cluster 1 of 443.75 m^3 (365 m^3 from the roof and 78.75 m^3 from garden runoff once the PS are full) which is the same as the result for the new code. Aquacycle gives a total runoff value of 364 m^3. This suggests either that Aquacycle implements slightly different algorithms to those described in the manual or that there is a possible bug in Aquacycle.

A further example illustrates possible additional discrepancies in the evaporation calculations in Aquacycle. For a fixed 0.5mm evaporation per day for 365 days from a (10 blocks * $100m^2$ garden) $1000m^2$ area, the resulting evaporation should equal 182.5 m^3/yr but Aquacycle gives 180 m^3/yr unless the soil stores are emptied. For the chosen example this does not arise. A possible explanation is that a fraction of the water cannot return to the atmosphere in aquacycle, but this is not explained in its user manual. While this discrepancy is not particularly large, it is inappropriate for a model to produce results that cannot be explained by the equations that have been apparently employed for its construction. The results for the Scoping model code produce the expected outcome for a fixed evaporation rate.

```
(1)      E_actualPS1(j)  =  PS1_unit(j) / PS1_cap(j) * E_trans  (set to 0.007 m)

         E_actualPS2(j)  =  PS2_unit(j) / PS2_cap(j) * E_trans


   Maximum evaporation for day i is the potential, Ep(i):


(2)      If  E_actualPS1(j)  >  Ep(i)  Then

                  E_actualPS1(j) = Ep(i) 'units m

         End If

         If  E_actualPS2(j ) >  Ep(i) Then

                  E_actualPS2(j) = Ep(i)

         End If


         If  PS1_unit(j)  <  E_actualPS1(j) Then
```

Figure 1: Summary of the algorithms used to calculate evaporation from garden areas in the new model, as reproduced from the Aquacycle manual.

Developing additional model components

There is considerable scope to develop new facets to the scoping model's water features that are applicable to the conditions observed in Birmingham, as well as to other cities. Initial additions have incorporated:

- Sustainable Urban Drainage Systems (SUDS) options including green roofs
- Borehole extraction
- Intercluster groundwater flow
- Canal/river option with extraction capability
- spatially heterogeneous aquifer storage
- a more user-friendly indoor usage profile

A generic SUDS option has been programmed at unit, cluster and catchment level. All components are based on a simplified transformed conceptual representation of input, storage compartment(s) and output. The description and functioning of the storage compartments and the outflow behaviour is component dependent The unit level includes options for small swales and infiltration ditches, the cluster level larger swales, ditches and gully pots and the catchment scale infiltration basins, detention basins, lagoons etc. These options allow the user to create a SUDS "treatment train" (*Butler et al*., 2006).

For a basic Swale, a user specified drain period is applied with the default set as 2 days. The store has a capacity which if exceeded goes to the drains and each day up to (1- 1 / drain period) * capacity can be infiltrated into the pervious store (Figure 2). Once the pervious store is saturated infiltration will cease until space is created by evaporation or drainage. Excess water, that cannot infiltrate, flows to the drains. Work is in progress testing the suitability of Manning's equation for channel flow to better represent storage and flow on a daily time-step.

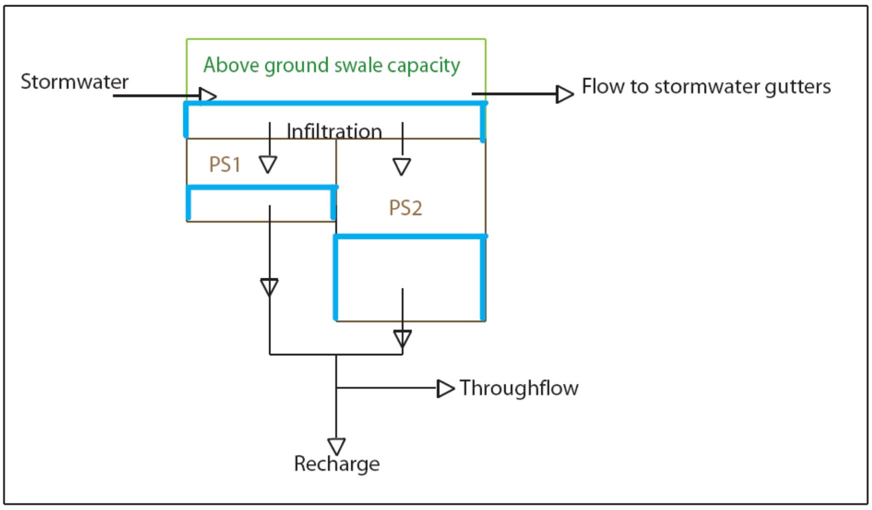

Figure 2 : Flow diagram for the new swale unit in the scoping model

Groundwater storage is represented at the cluster scale with input from recharge and intercluster flow (baseflow) and loss by baseflow. By modelling at cluster level there is some degree of spatial heterogeneity which is important when assessing groundwater level issues (e.g Birmingham, UK has shallow groundwater which has implications for infrastructure planning). The user can specify which cluster aquifer unit to use for borehole extraction. This can supply other clusters as well. There is the option to use borehole water at the unit level for irrigating the garden or flushing the toilet and at cluster level to irrigate the public open space.

There is the option to model open water (rivers/canals/lakes). The generic water body has a capacity (dimensions – width, depth and length), infiltration rate, interaction with groundwater (gaining or losing river) and flow rate (with the option of setting flow to zero to model static water bodies such as ponds).

The user can choose from five generic indoor usage profiles when running the program. These are domestic, office, hotel, retail and industrial. Default UK values are currently provided. This differs from Aquacycle which has a single usage input profile based on domestic usage split (kitchen, toilet, bathroom, laundry) that the user has to adjust according to his modelling requirements.

The completion of the water cycle components is important and essential for the model's application for quality and energy assessments as well as quantity, the primary goal. However, it is the extension of the aquacycle approach to address life cycle energy usage for a city water system that will be the focus of the research effort in the coming year.

Using Eastside as a Demonstrator

Eastside is a 170 hectare site to the east of Birmingham (UK) city centre that is currently undergoing physical regeneration. It is a £6 billion joint public and private venture with best practice sustainability as one of its core principles.

Figure 3. Masshouse in the Eastside - a very different building from the typical unit proposed in Aquacycle and UVQ (*Birmingham City Council*, 2007).

This area will be a pilot study with the aim of applying the model to Birmingham as a whole eventually. It is important to note that the model's design means that it will be generic and applicable to any urban area - not just Birmingham. Unfortunately, the model will not be finished in time to have an effect on the plans for the development of Eastside but application to Eastside provides a framework for testing and refining the model and the underpinning concepts.

Map layers have been prepared with ArcMap commencing with the base ordinance survey layer from Edina Digimap, the proposed building layer from the council website for Eastside, and then additional layers of polygons for canal areas, road area, building areas and general land use areas (Fig. 4). The general land use divisions are based on the proposed land uses. The canal, road and building areas were outlined manually from the proposed map – this was very time consuming and it is likely that supervised pixel selections for the different land uses is a more efficient technique. In this technique a pixel on the map is selected representing a particular land use and the software then identifies all other areas on the map with the same pixel (or to a specified difference in pixel shade e.g. plus or minus 2). Once the selection has been made shadows or poor map definition may require manual correction. The area calculations from the polygons are a dominant input to the water balance model.

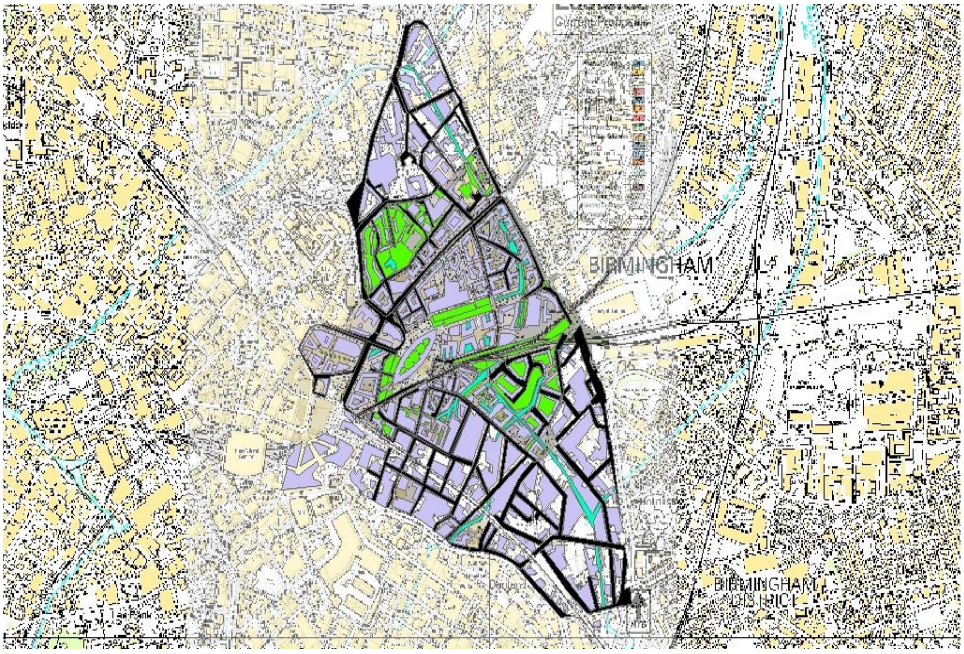

Figure 4: ArcGIS map of Eastside. Green areas are coloured green, buildings are grey, roads are black and canal sections are blue.

Preliminary Results

A limited number of alternative water management options have been simulated for Eastside using the water balance model using the best available data available in early 2007. The results are shown in Table 2 and are given for illustration only. The column titled “Mixture with cluster level” shows results for a model run with a mixture of unit block rainwater and wastewater recycling options with the addition of a cluster scale stormwater and wastewater system. Cluster scale alternative water management options can collect water or waste from user specified upstream clusters including the one in which they are situated. They can then supply any cluster with recycled water. All recycling systems use at least 20% less mains water than the conventional system. Reduction in stormwater runoff and wastewater runoff varies from 12-35% but there is some tradeoff between the two. The systems with a mixture of wastewater and rainwater recycling yield the best results.

It is interesting to note that evaporation remains unaltered by the implementation of the different strategies. Simulations were set so that mains water was used to satisfy any outstanding irrigation demand so for all strategies the same volume of water is applied to pervious areas. Consequently the effect of the alternative strategies is not seen as increased evaporation as a result of greater irrigation supplied but rather as a further reduction in the mains water used. Exfiltration from wastewater/stormwater pipes is not currently modeled (it is intended to investigate the inclusion of these in the ongoing model development) and so a reduction in flows has no effect on subsurface moisture levels and as a result no effect on the evaporation volume.

Table 2: Annual catchment equivalent areal depths for precipitation and various indicators for a range of development options.

Annual Areal depth:	Conventional system	Unit block raintanks (RH)	Unit block wastewater system (WW)	Mixture RH and WW	Mixture with cluster level
Annual precipitation (mm)	1055.4	1055.4	1055.4	1055.4	1055.4
Catchment mains water used (mm)	499.6	390.8	332.7	362.6	364.0
Catchment stormwater (mm)	859.8	740.1	849.6	744.0	787.5
Catchment wastewater (mm)	480.1	476.6	310.6	443.7	421.6
Catchment evaporation (mm)	183.9	183.9	183.9	183.9	183.9

Since this is only a water balance it shows an unrepresentative selection of sustainability indicators. A saving of 20% on "mains water used" for example may cost more in terms of energy use than the conventional or may not be economical. In order to make an informed decision the user needs a suite of indicators covering the "three pillars of sustainability" (Hunt, 2006): environmental, economic and social.

In response to this need it is planned to perform a detailed analysis of energy requirements for the various developments within Eastside. This will inform model development of life cycle energy with the objective of developing a set of generic values for use within the model. Data collection for this is underway. A final additional layer of life-cycle cost will be added using UK data (however the user will have the option to change values for different countries).

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A5.2 Proceedings from the 2010 IAHR conference, Edinburgh

CITY WATER BALANCE – A NEW TOOL FOR SCOPING INTEGRATED URBAN WATER MANAGEMENT OPTIONS

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ABSTRACT

Global pressures (climate, environment, pollution, energy and population dynamics) are forcing a need to change the way water is managed in future in cities. To meet this need, the SWITCH European Framework VI project is developing simulation tools to assist stakeholder communities to scope future options for integrated urban water management (IUWM). One of the tools being developed is City Water Balance (CWB). This tool allows users to explore a broad range of strategies for future city-level change scenarios and to output indicator information on water demand, quality, energy consumption, and life-cycle cost. CWB is designed as an efficient scoping model.

CWB is a daily-timestep conceptual water, energy and quality balance model operating over annual to decadal periods that uses simplified representations of the flows and storages within representative land use areas mapped across a city. Data requirements allow a model to be established relatively quickly from extant spatial mapping. Default descriptions and datasets for individual land uses and water management technologies further facilitate model development. CWB advances previous work of IUWM scoping models such as AQUACYCLE and UVQ in two ways. The first concerns the integration of natural systems more fully into the urban water cycle description (groundwater and surface water bodies are spatially articulated). The second concerns the extension to address energy consumption and production as well as life cycle costs.

Application of the model to Birmingham using historical data for verification and validation of the model has been undertaken; the results provide a valuable illustration of the effectiveness of the model.

INTRODUCTION

City Water Balance has been developed as a part of the EU VIth Framework funded integrated project – SWITCH (Sustainable urban Water management Improves Tomorrow's City's Health) [http://switchurbanwater.lboro.ac.uk/index.php] . The main goal of SWITCH is the development of sustainable and effective water management systems for the 'city of the future'. To achieve this SWITCH aims to improve the scientific basis for the development and management of urban water systems, to ensure that they are robust, flexible and adaptable to a range of future global change pressures. One component of the SWITCH toolset is City Water, a Knowledge Base System/Decision Support System. City Water builds on past and contemporary work in the field of Decision Support and Scoping models for assessing alternative management options for urban water (AQUACYCLE, UVQ, UWOT, MULINO, HARMONIT, DAYWATER) but departs from these in several ways that are specific to the SWITCH project and to the learning alliance environment for participatory planning of IUWM strategies. There are three key areas where it is strikingly different to the existing tools that have been applied for long term water planning.

- significant weight is given to the integrated exploitation of the natural environment as one possible contribution to the development of a sustainable future.

- contributions are made that deal with aspects of the regulatory environment, the legal frameworks for decision making and with the existence and location of historical data that define past conditions, approaches and outcomes.
- a broad range of time and space scales are covered that permit, for example, life cycle costing (either energy or financial) over long times and local flooding over short times. It is also applicable to the exploration of the evolution of the landscape of a city as opposed to only addressing the end-point(s) of a process of development.

City Water Balance forms part of the City Water KBS/DSS and is a scoping model for assessing the dynamic balances of water, energy and pollutants at the city scale. It works on a daily time step over decadal periods and can explore water and wastewater stresses in response to climate change and changing urban populations under a range of strategic technical options for improved IUWM. Five indicators, characterizing neighbourhood or city-scale conditions, are derived for output: water demand/supply ratios; wastewater production; water quality; life cycle energy and life cycle cost. This paper describes some concepts underpinning City Water Balance and presents early results from an application to the City of Birmingham, UK.

CITY WATER BALANCE

In this section the concepts underpinning City Water Balance (*CWB*) are explored. A primary goal for the design of *CWB* was that it should be capable of application to both cities where mapping and monitoring are underdeveloped and cities possessing a high level of detailed knowledge of the water and wastewater system.

Spatial characterisation

To facilitate rapid characterisation of land use within the cityscape the unit block concept adopted for the water balance model, *Aquacycle* (Mitchell *et al.*, 2001) has been adopted within *CWB*. The unit block describes the water storage, use and transmission of water for easily recognized land uses (e.g houses with gardens, hospitals or golf courses). Forty-nine standard unit block types have been designed based on data from the United Kingdom. Neighbourhoods of a city containing areas of a given unit block type can be generally identified using a combination of available aerial photography and road maps, with some additional ground truthing, if needed. The neighbourhoods, termed “miniclusters” are comprised of one to many essentially identical unit blocks. In this way a city can be characterized by a mosaic of miniclusters. At this point, the City Water Balance approach departs from that used by *Aquacycle.* A further aggregation level has been introduced within *CWB.* In addition to the mosaic of miniclusters, the city plan area is divided into subcatchments formed around the wastewater and natural drainage network. Subcatchments may contain any number of complete miniclusters: a minicluster may not span two subcatchments. Different miniclusters in a subcatchment can possess different unit block characteristics. To simplify the connections between subcatchments and the flow calculations, each subcatchment can receive wastewater/stormwater from several upstream subcatchments but wastewater/stormwater from each subcatchment can only flow into one downstream subcatchment. This modelling restriction allows flow calculations to proceed through the urban area using a cascade from upstream to downstream: downstream subcatchments can be affected by upstream flows but not vice-versa. The use of subcatchments means that minicluster areas can be based solely on land use type and enables more rapid representation of the wastewater network at the city scale.

Fundamental attributes of the unit block are pervious/impervious proportions, water demand profiles and pollutant input loads (Mitchell *et al.*, 2001; *Mitchell and Diaper*, 2005). For example, a house could be a unit block with user defined proportions of its area assigned to garden, pavement and roof and an indoor water demand profile. *CWB* has been developed so that there are a range of landuses with generic profiles that the user can choose from, which greatly reduces pre-processing and makes the model easier to use. Flexibility is achieved by retaining the option to create new unit block types, if required. Unit block types are derived based on the primary values of their attributes. For example, "Retail with canteen" and "Retail without canteen" may have similar building and land use characteristics defined by the pervious/impervious proportions but different water demand profiles.

Temporal resolution

CWB is a decision support tool for strategic planning and the assessment of sustainable water management options, rather than a detailed design tool. Consequently, a daily timestep was selected as appropriate for modeling the transient behavior of the city water system under changing land use, water management strategies and climate scenarios. Although this level of aggregation results in the loss of diurnal flow variations, it provides sufficient information (*Makropoulos et al.*, 2008) for the comparison of different water management options, especially for long-term simulations which are necessary for sustainability assessments.

Water supply

Mains water is imported from outside the system boundary. The mains supply demand is calculated as the sum of the minicluster demands that are not met by local or decentralised supply schemes. The user can specify a percentage of the demand that is satisfied by the supply, allowing drought simulation. Water is supplied at fixed contaminant concentration and with a fixed energy and economic cost attached to it. A proportion of the supply is lost as leakage within the system boundary. Time dependent leakage from the supply network is included in the model. This simplified approach to the modelling of the mains supply is adopted as aggregate cost, energy and contaminant data are typically more easily determined and the need to model the specific characteristics of the water sources, the transfers and treatment is avoided in the model setup. Water demand input to *CWB* is based on land use. *CWB* expresses demand in terms of per capita demand and also per unit area for different water use requirements. The unit area approach provides an effective way of describing water demand for many types of urban land use such as office space (*Hunt and Lombardi*, 2006). *CWB* also introduces the idea of the seasonal occupancy: this is an optional set of parameters for which the user can specify monthly occupancy factors, representing a fluctuating population. Athens, Greece provides an example of a city where a significant proportion of the population leaves during the hot summer months (*Manoli*, 2009). The application of *Aquacycle* to Athens proved to be problematic as it was unable to simulate demand during the summer for this reason (*Karka et al.*, 2006). For residential blocks an alternative model for the estimation of indoor demand is available. These have been based on ideas adopted in *UWOT* (*Makropoulos et al.*, 2008): the demand profile is built from a selection of water using appliances (bath, W.C., kitchen tap etc.). In this case, the input data required are the types and number of appliances in the unit block, water consumption/use, frequency of use/capita/day and occupancy. A library of default options is available with *CWB*, to which the user can add their own appliances, if required.

Runoff

There are five types of surface that can generate runoff in *CWB*: roof, pavement, garden, road and Public Open Space (POS). In each minicluster, the degree to which these surfaces are connected to the piped stormwater system varies. To address the issue of overland flow outside the sewer network *CWB* uses the idea of effective areas, originally introduced in *Aquacycle* (*Mitchell et al.*, 2001). The effective area is defined as the proportion of the total area of an impermeable surface (roof, paved, road) generating runoff that goes directly to the stormwater system. Runoff from non-effective areas (NEAR) flows onto nearby pervious space or sustainable urban drainage systems (SUDS), if available.

Local water management options

CWB allows exploration of the sustainability of a variety of water management options at different spatial scales. These are green roofs, rainwater harvesting, wastewater recycling, septic tanks, borehole abstraction, porous paving, porous asphalt, swales, filter strips, soakaways, retention ponds and detention basins. Inflows to SUDS are direct precipitation and runoff. Outflows are evaporation, infiltration, throughflow, and overflow. Flow along conveyance swales is governed by a normal depth approximation using the swale cross-section and a manning's resistance coefficient.

Groundwater and surface water

A single layer two dimensional areal groundwater system can be modeled beneath a city. The areal extent and resolution of the groundwater system are independent of the land use mapping and user defined. Connection between miniclusters and the groundwater system allows water exchanges to and from the groundwater to the surface soil and water stores. The groundwater is also fully coupled to the surface water features. Ground elevation and aquifer base elevation data are required to model the spatial patterns of the surface – subsurface interactions.

Rivers, lakes and ponds can all be represented as discrete water bodies. Rivers are split into reaches. Connectivity between water bodies is achieved by a connection matrix. Connections are currently one-way: reverse flows in a river cannot be simulated. External water supplies to surface water bodies are simulated as source terms. Subcatchments may discharge to the surface water network.

Contaminants

Modeling of contaminants follows the concepts outlined in *UVQ* (*Mitchell and Diaper*, 2005). In any simulation up to four pollutant species can be modeled concurrently. Contaminant inputs are represented as concentrations in input streams and as loads added during indoor water use. Contaminants are modeled conservatively, with no conversion or degradation. Detailed modeling of contaminant interactions requires additional site specific data and increases the complexity of the model without necessarily improving the accuracy *(Mitchell et al.,* 2003). Such interactions are described in other models, which can be used in the next level of analysis. It is assumed that when water flows, with different contaminant concentrations, combine they become well mixed. A proportion of dissolved contaminant load is removed as sludge in the soil stores, representing filtering and adhesion, and in various water management options, representing treatment.

Energy and Cost

For each of the decentralised water management (WM) options in *CWB*, simplified life cycle energy and cost stand-alone spreadsheet models have been developed, which serve as a technology library to *CWB.* The cost and energy for supply and treatment using the conventional, centralised piped systems uses published average energy and cost values per cubic metre supplied and treated.

Lifetime energy use is calculated as the sum of the embodied energy of all component parts, diesel used in transportation and by construction plant, diesel used in maintenance and operational costs (e.g. electricity for pumps and embodied energy of chemical treatment).

Whole life cost is calculated as the sum of the capital, construction, maintenance and operation costs. Costs incurred throughout the lifetime of the asset are reduced to net present value. For a number of SUDS (porous pavement, porous asphalt, filterstrips, soakaways, ponds and swales) only the lifetime energy use is calculated since the lifetime cost for these options is well-documented (*Woods-Ballard et al.,* 2007).

Energy and cost data were obtained from suppliers' specification manuals and literature (e.g. Materials embodied energy (Hammond & Jones (2008)); Diesel consumption (*DEFRA*, 2007; *SPON*, 2005); SUDS dimensions (*Woods-Ballard et al.*, 2007); Pump operation energy (*Hunt & Lombardi*, 2006; *Woods-Ballard et al.,* 2007); Cost (*SPON*, 2005))

BIRMINGHAM STUDY AREA

Birmingham, United Kingdom is one of three areas that are presently being modelled using City Water Balance. Birmingham has a population of approximately 1 million. It is part of the West Midlands conurbation which accounts of almost 5 million inhabitants. Other municipal areas within the West Midlands include Sandwell & Dudley, Wolverhampton, Walsall and Solihull. Birmingham rose from being described as a small village only worth 20 shillings in the Domesday Book (1086) to a powerhouse during the industrial revolution and became known as "the workshop of the world" or the "city of a thousand trades". Heavy industrial activity has markedly declined over fifty years and today the city is building a reputation also as a commercial centre and tourist destination. In the future, most new developments will be built at much higher density than previously experienced. This will cause a "hardening" of the urban landscape with potentially 70 dwellings per hectare (mainly flats and terraced housing). This will therefore increase the need to control runoff from rainfall events and attenuate localised flood risk. The major water company in the Midlands, Severn Trent Water, estimates water usage at 132l/d/household (estimated average household consumption 2004-2005). If new housing developments are to meet sustainability targets on water supply reduction and efficiency of 20% on current demand equates to a reduction of 25l/d/household. In the UK, less than 26% of household have metered supplied and Birmingham follows a similar pattern. The main pressures on water in Birmingham, and the West Midlands as a whole, are similar to many other major Western European cities and can be divided into three distinct areas:

- Surface water management and quality
- Groundwater protection and management
- Flood risk

Increasing population (it is predicted that the West Midlands population will increase by 6.6% between 2003 and 2023) is leading to an increased demand for potable water and is increasing flows and loads in the sewerage system and wastewater treatment plants. This is potentially a major problem for Birmingham as much of its area is drained by combined (surface and foul water) collector systems.

To gain insights into alternative futures for Birmingham, CWB has been used to model the wastewater drainage area for the major wastewater treatment plant at Minworth, which covers most of the city. Six thousand miniclusters have been mapped across the Minworth catchment area comprised of 49 different unit block types (Fig. 1). These miniclusters fall within 27 subcatchments that are matched to the Drainage Study Areas used by Severn Trent Water for its management of the wastewater network (Fig. 2). In addition, 271 district metering areas are identified that define the water demands across the city. The majority of the water supplied to the city is imported from the Elan valley, 120 kilometres from Birmingham. To permit verification of City Water Balance, and to develop a baseline for the analysis of future water management strategies, detailed data characterising the water consumption for all district metered areas and wastewater/stormwater flows for selected short periods spanning the years 2004 to 2009. An iterative process of model calibration and validation was adopted both to verify the attributes of the model components and to learn more about the behaviour of the water supply and waste water collector systems. Certain features of the system assumed *a priori* for the setting up of the model, particularly in relation to changes in leakage from the water supply network and surface and foul sewer mixing were revised during this process. To illustrate the model capability, results from the calibration of the storm water and wastewater flows are reported in the following section.

CALIBRATION/VALIDATION

Figure 2 shows the drainage areas used for flow calibration. They are situated on the boundary of the Minworth Catchment and were preferred for the initial calibrations because there is no flow into them from upstream drainage areas. After calibration of the water supply to the 2008-2009 water records from Severn Trent (Fig. 3), calibration of stormwater and wastewater flows was undertaken.

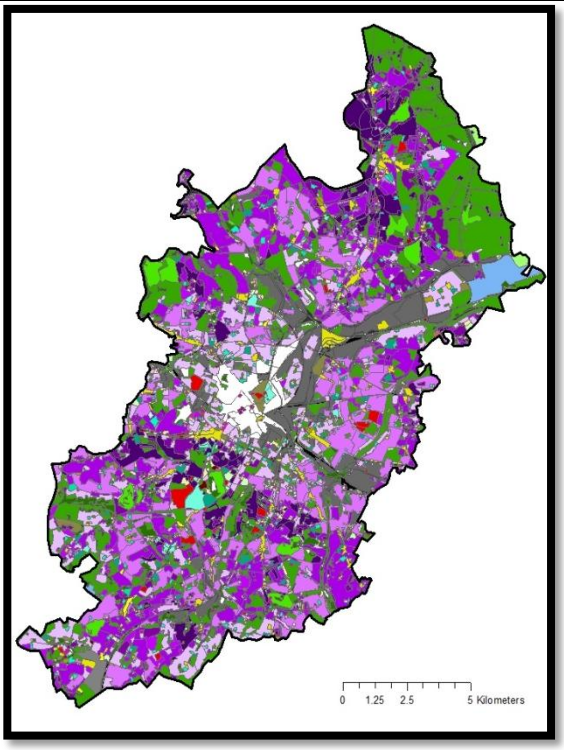

Figure 1. Landuse map of the Birmingham Study Area. Pink/purple is residential; grey includes industry, roads and railways; green is vegetated area; white is city centre; and brown/yellow is commercial

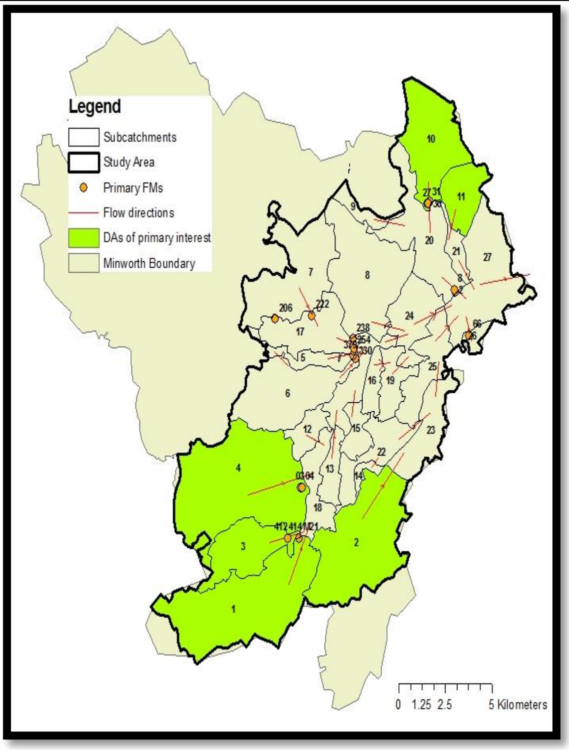

Figure 2. Minworth WWTP catchment split into 27 subcatchments in the Study Area. FMs of key interest are shown in orange.

FM412 (Fig. 2) is a 750mm diameter foul sewer that was monitored during a two month period (February to April 2007) (*STW*, 2009). It is located at the outflow point from Subcatchment 3. In their application of the Australian model UVQ to Doncaster, UK, *Rueedi & Cronin* (2005) calculated that 1% of stormwater runoff inflows into the foul network. If this value is applied to the Birmingham model then *CWB* fails to match the fluctuations in the observed data (Fig. 4). By increasing the inflow to 25% a much better match is obtained (Fig. 5).

As a validation exercise, these parameters were applied to flows observed by FM 03 near the downstream edge of Subcatchment 4 (Fig. 6). There is an acceptable fit between the observed and predicted flows for scoping the impact of changes to the urban water management in Birmingham.

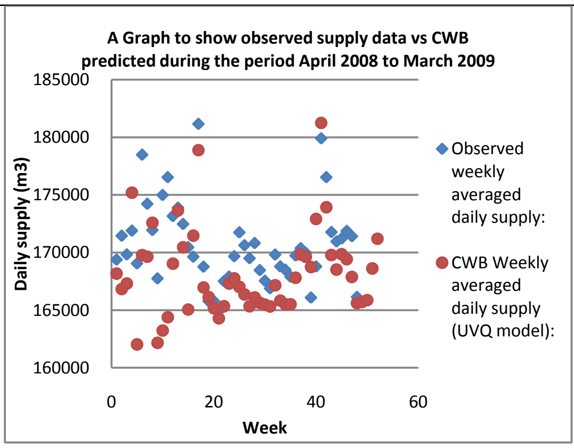

Figure 3. Gross mains supply to the study area (April 2008 to March 2009).

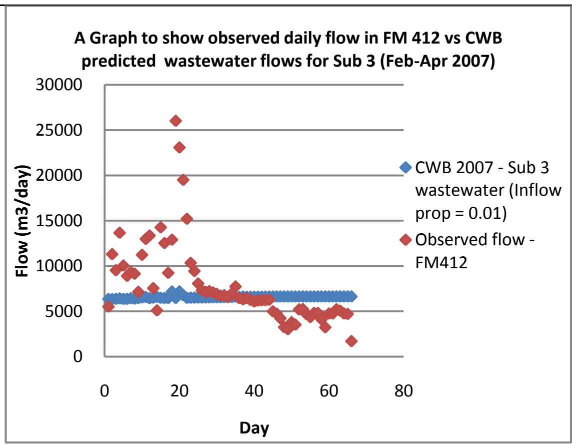

Figure 4. Wastewater flows from Subcatchment 3

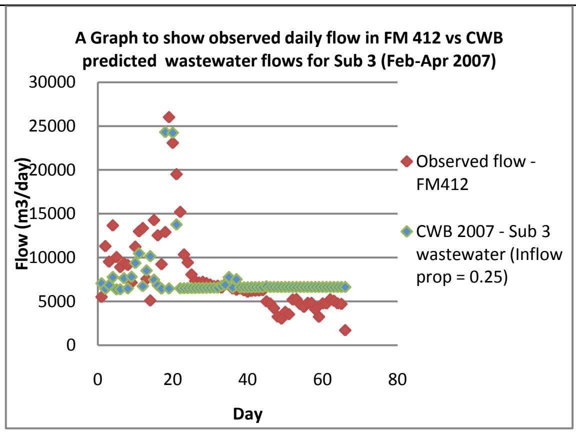

Figure 5. Wastewater flows from Subcatchment 3

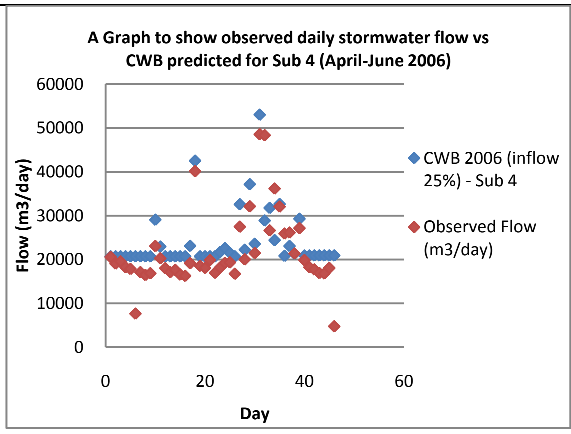

Figure 6. Wastewater flows from Subcatchment 4

CONCLUSIONS

City Water Balance has been developed to permit scoping calculations to be undertaken to assess the impact of different future scenarios and future urban water management strategies on a range of flow, quality, cost and energy indicators for a city. The model has been designed to be simple to establish and operate. Initial testing, using data from Birmingham, has demonstrated that the model is capable of reproducing the spatial and temporal characteristics of the flow systems for an urban area. It has also demonstrated that some aspects of water distribution have significant uncertainties and account may need to be taken of these for exploration of future strategies prior to more detailed modeling with other tools.

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Appendix 6 – Contents of Electronic Appendix

1) *CWB* executable with read.me file
2) *Shapedump* executable (supporting software)
3) Birmingham case study files
    a) Base case
    b) Scenario 1
    c) Scenario 2
    d) Scenario 3