Chapter 6

An Allotment Water Balance Model

> "This is not to say that western water technologies (and water related civil engineering) have not provided an important contribution to 'development' processes and human well being in both the developed and developing world. They have. But with these technologies come a form of cultural pervasiveness that did not often respect either the local environment or cultural context to which it was transferred. The whole process of technology transfer in the water industry is now a very relevant topic as there is growing awareness that the current form of technologies is neither economically or environmentally viable" [M.J. Mouritz and P. Koffel, 1996].

6.0 Introduction

As water resources become increasingly scarce it is important to make better use of available water supplies. Demand management is recognised within the urban water supply industry as an important tool for ensuring that water demand does not exceed available supply. Demand management methods include pricing and restriction policies, and education and advertising programs to encourage water conservation. In an urban system experiencing growth in water consumption demand management can only defer the inevitable need to augment the water supply headworks system [Kuczera and Ng, 1994].

In Chapters 2 and 3 it was found that significant demand reductions can be achieved by supplementing centralised water supply with rainwater collected from roofs and stored in tanks at the urban allotment for toilet flushing, hot water and outdoor uses. Roof runoff captured in rainwater tanks provided water of acceptable quality for hot water, toilet and outdoor uses (Chapters 2, 3 and 4). Although institutions such as water authorities and local governments have claimed that the use of rainwater tanks is a health hazard it was found that the use of rainwater for non-drinking purposes did not pose any health hazards. Indeed, in Chapter 5, it was argued that institutional resistance to the use of rainwater tanks resulted from government desire to maximise revenue from water sales and reluctance by water supply and stormwater engineers to accept the benefits that use of rainwater tanks will provide. The thinking of engineers has been constrained by adherence to the pipe paradigm.

The use of rainwater tanks on urban allotments will reduce stormwater peak and volumetric discharges, water consumption and peak mains water demand. The impact on existing water supply and stormwater infrastructure will be reduced. The provision of stormwater and water supply infrastructure is based on the probability of certain stormwater peak discharges and peak demands occurring. This requires analysis at short times scales. For example Stephens and Kuczera [1999] found that the time of concentration for stormwater runoff from an urban allotment was two minutes, while IEAust [1987] suggest a time of concentration of five minutes. A model to analyse the long-term performance of rainwater tanks on urban allotments at short time steps is required to determine the impact of rainwater tanks on the provision of water supply and stormwater infrastructure.

This Chapter describes the development of a water balance model to analyse stormwater discharges and water use on urban allotments or clusters. The performance of the model will be verified against data from the Maryville Experiment (Chapter 3) and then used to determine the long-term performance of the Maryville house (Chapter 3) and the Figtree Place clusters (Chapter 2). Finally, the impact of the use of rainwater tanks to supply domestic hot water, toilet and outdoor uses in the Lower Hunter region of NSW will be evaluated.

6.1 Outline of the Allotment Water Balance Model

A schematic of the Allotment Water Balance model is shown in Figure 6.1. The rainfall input to the model can be from pluviograph data, the DRIP (Disaggregated Rectangular Intensity Pulse) event rainfall model developed by Heneker et al. [2001] or a pluviograph rainfall generator. The pluviograph rainfall generator is used to create a rainfall pluviograph record from daily rainfall in locations where incomplete or no pluviograph data is available.

The rainfall falling on roof areas discharges to a first flush device and if the capacity of the roof gutter system is exceeded, rainfall also discharges to impervious areas. Rainwater is routed through the first flush device to a rainwater tank. Water is drawn from the rainwater tank for household uses (such as hot water, toilet and outdoor uses) and, if the water level in the rainwater tank is below a set minimum level, the tank is topped up with mains water at a nominated rate. Mains water is used to supply all household uses not sourced from the rainwater tank and to supplement the rainwater tank supply. The rainwater tank overflows can be directed to an infiltration trench, an on-site detention tank or the street drainage system.

Rain falling on impervious areas can be directed to pervious areas, an on-site detention tank or the street drainage system. Rain falling on pervious areas can infiltrate to the soil and can discharge to the atmosphere via evapotranspiration, an on-site detention tank or the street drainage system.

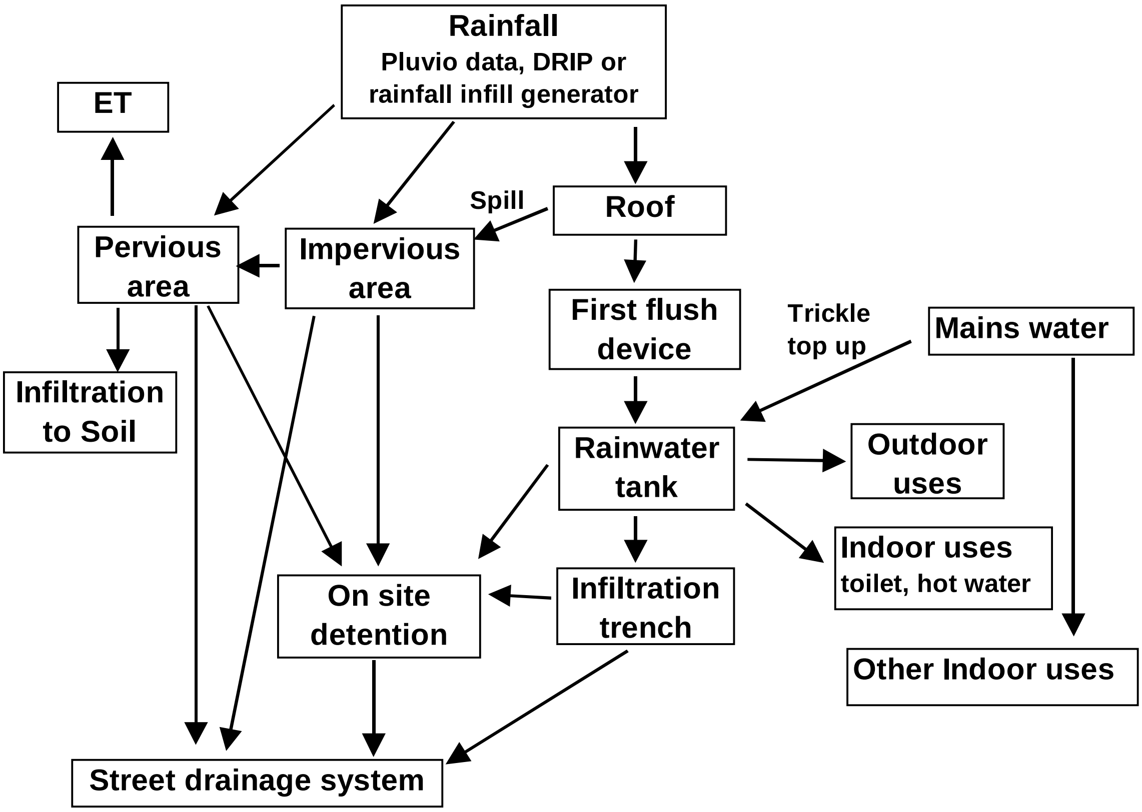

Figure 6.1: Schematic of the water balance model

The model provides data on daily water use from mains and rainwater tank supplies, storm event based information about performance of the system, annual maximum stormwater peak discharges and peak daily water demand.

6.2 Household Water Consumption

The household water consumption in the model has been divided into two categories: indoor and outdoor uses. Data from the Hunter Water Corporation (HWC) was used to develop indoor and outdoor water consumption patterns that are used in the Allotment

Water Balance Model.

6.2.1 Water Use in the Lower Hunter Region

The HWC [B. Berghout, personal communication, 1999] has monitored monthly indoor and outdoor water use in over 130 houses in the Lower Hunter region of New South Wales, Australia during the period 1986 to 1998 (Appendix G). Average monthly indoor and outdoor water consumption for the Lower Hunter region is shown in Figure 6.2. Indoor water use is shown to be reasonably constant and predictable. However, outdoor water use displays seasonal patterns and is highly variable. Outdoor water use varies from 17% to 38% (average: 25%) of total domestic water demand in the region.

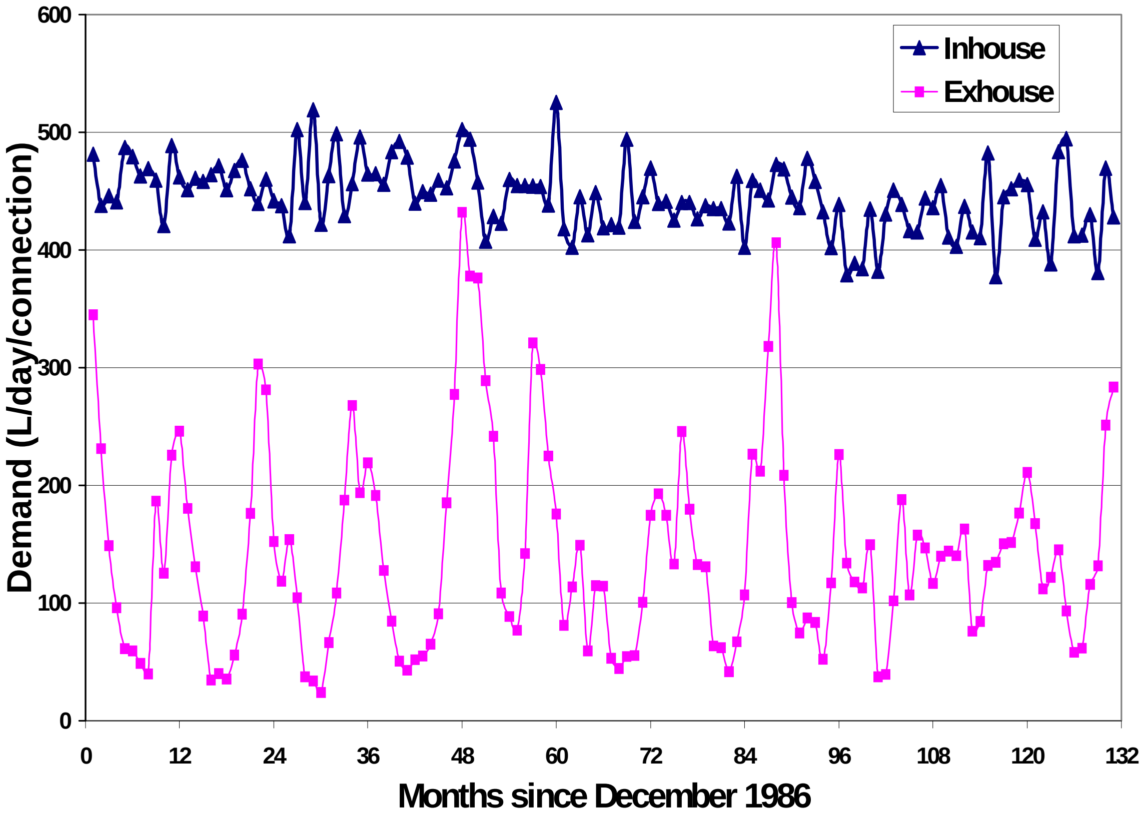

Figure 6.2: Average monthly indoor and outdoor water use for the Lower Hunter region

Water consumption and demographic data from nine zones within the Lower Hunter region is shown in Table 6.1. Average indoor and outdoor water use is seen to be highly variable across the region. Similarly demographic and climate information varies from zone to zone.

Table 6.1: Water consumption, climate and demographic data from the Lower Hunter region

Zone	Ave. indoor demand (L/day)	Ave. outdoor demand (L/day)	Ave. rain (mm/ day)	Ave. income ($/ person)	Ave. rain days per month	Population growth (%/year)	Ave. daily Temp (°C)
Inner Newcastle	337	105	3.07	429	11	1.03	22
Hamilton Mayfield	454	75	3.07	330	11	0.67	22
Lambton Jesmond	290	63	3.07	323	11	2.09	22
NW Wallsend	498	150	3.07	289	11	3.26	22
Lake Macquarie East	553	124	2.78	286	13	1.24	22
Lake Macquarie West	517	150	3.24	286	12	2.42	22
Maitland	351	213	2.45	284	7	1.16	24
Cessnock	426	133	2.05	270	7	0.38	24
Port Stephens	441	218	3.68	264	12	2.42	23

Many authors including Zhou et al. [2001], Kuczera and Ng [1994] and Heeps [1997] acknowledge that domestic water consumption is dependent on climatic and socio-economic factors. A linear regression was developed to estimate average daily household water use for the Lower Hunter region using climate, socio-economic and water use data from Table 6.1. The monthly daily average indoor water use (inDem) in L/day may be estimated from:

$$\begin{aligned}\mathrm{inDem} &= 27.79 + 145.69\,\mathrm{P} - 0.422\,\mathrm{M} - 10.579\,\mathrm{AveR} + 6.74\,\mathrm{Rdays} \\ &\quad - 0.162\,\mathrm{Inc} - 12.28\,\mathrm{G} + 0.49\,\mathrm{AveTemp}\end{aligned} \tag{6.1}$$

where P is the number of occupants in the dwelling, M is a seasonal index ranging from 1 to 6 (The index for January and December is 1, and the index for June and July is 6), Inc is average weekly income per person ($), AveR is monthly daily average rainfall, G is annual population growth (%), Rdays is the number of days with rainfall in a month and AveTemp is the mean of the monthly daily maximum temperature (°C). Equation (6.1) yielded a coefficient of determination (R^2) of 0.81. The monthly daily average outdoor water use (exDayDem) in L/day is:

$$\begin{aligned}\text{exDayDem} &= -251.5 + 7.53\,\text{M} - 11.3\,\text{AveR} - 0.025\,\text{Inc} \\ &\quad - 0.816\,\text{Rdays} + 24.44\,\text{G} + 19.08\,\text{AveTemp}\end{aligned} \tag{6.2}$$

Equation (6.2) yielded an R^2 value of 0.69. The indoor and outdoor water use in the Lower Hunter region is shown to be strongly dependent on the climatic and demographic parameters shown in Table 6.1. Equations 6.1 and 6.2 were used to calculate monthly daily average water demand for use in the Allotment Water Balance Model.

6.2.2 Diurnal Pattern of Indoor and Outdoor Water Use

In order to determine indoor and outdoor water use at short time steps in the water balance model a diurnal water use pattern was required. Cox and Cartwright [1998] reported on the performance of ten townhouses at the Stringybark Grove development in Sydney. The majority of the water use at the Stringybark Grove development was indoor water use. However the diurnal water use pattern was adopted for use in the Allotment Water Balance Model because it was assumed that outdoor water use would have similar patterns to indoor water use. The diurnal water use pattern for the entire development is shown in Figure 6.3.

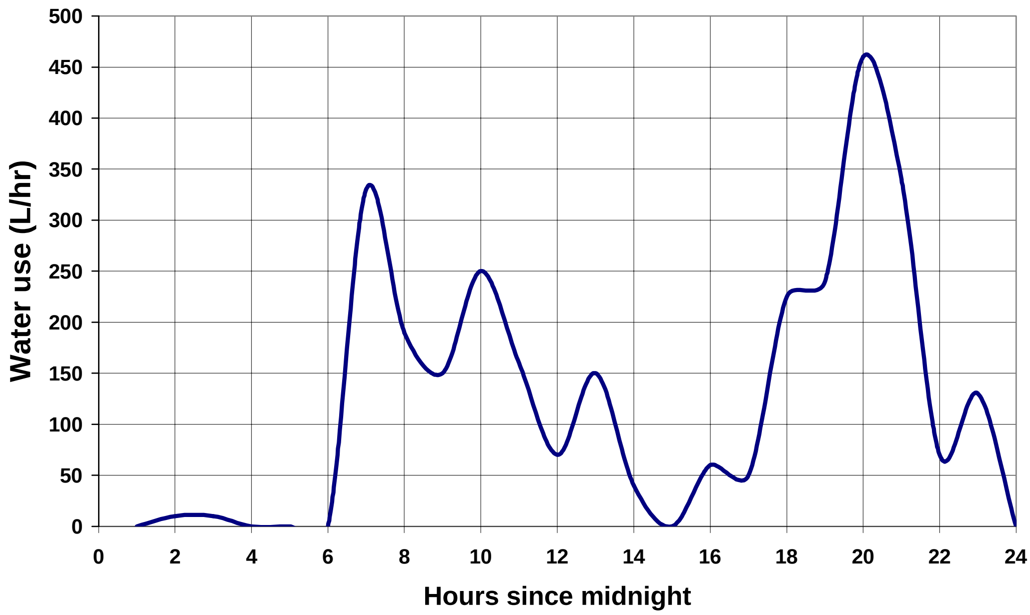

Figure 6.3: The diurnal water use pattern at the Stringy Bark Grove development

The water use patterns from Stringybark Grove (Figure 6.3) have been transformed into a normalised water use (cumulative use/daily use) versus normalised time relationship

(Figure 6.4) to enable the water balance model to determine diurnal indoor and outdoor water use patterns.

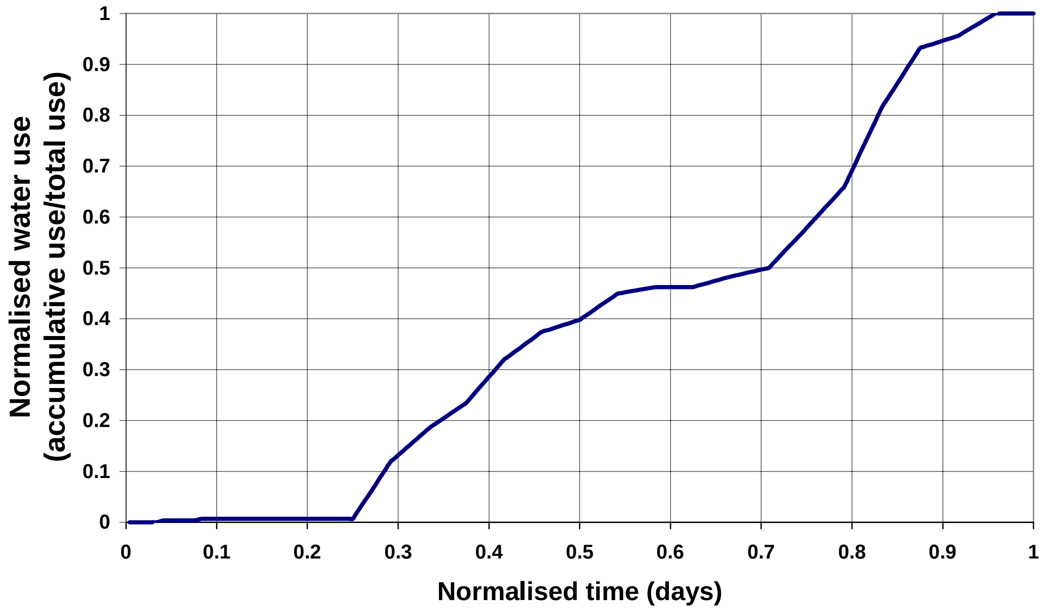

Figure 6.4: Normalised diurnal water use pattern used in the Allotment Water Balance Model

The water balance model calculates water consumption at six-minute intervals using the normalised diurnal water use pattern shown in Figure 6.4.

6.3 Rainfall Inputs to the Allotment Water Balance Model

The water balance model can use rainfall from pluviographs or tipping bucket rain gauges, the DRIP event based rainfall model by Heneker et al. [2001] or it can generate synthetic pluviograph rainfall from daily rainfall data. Rainfall input to the water balance model is analysed to determine event and inter-event characteristics. An inter-event or dry period is classified as having duration of greater than two hours and a rainfall event is classified as having a rainfall depth greater than the initial loss of the roof catchment and a dry period of greater than two hours on either side of the event.

6.3.1 Pluviograph Rainfall Generator

The pluviograph rainfall generator (schematised in Figure 6.5) requires a daily rainfall file with no missing data. This file is used as a reference file to find and infill missing storm events in a pluviograph rainfall record using normalised temporal patterns and average rainfall intensities derived from nearby pluviograph or tipping bucket rain gauge records. The average rainfall intensities and the normalised temporal patterns are randomly selected from files sorted by the month, which is used as a surrogate for season. A synthetic pluviograph record with rainfall depth totals at six-minute intervals is generated.

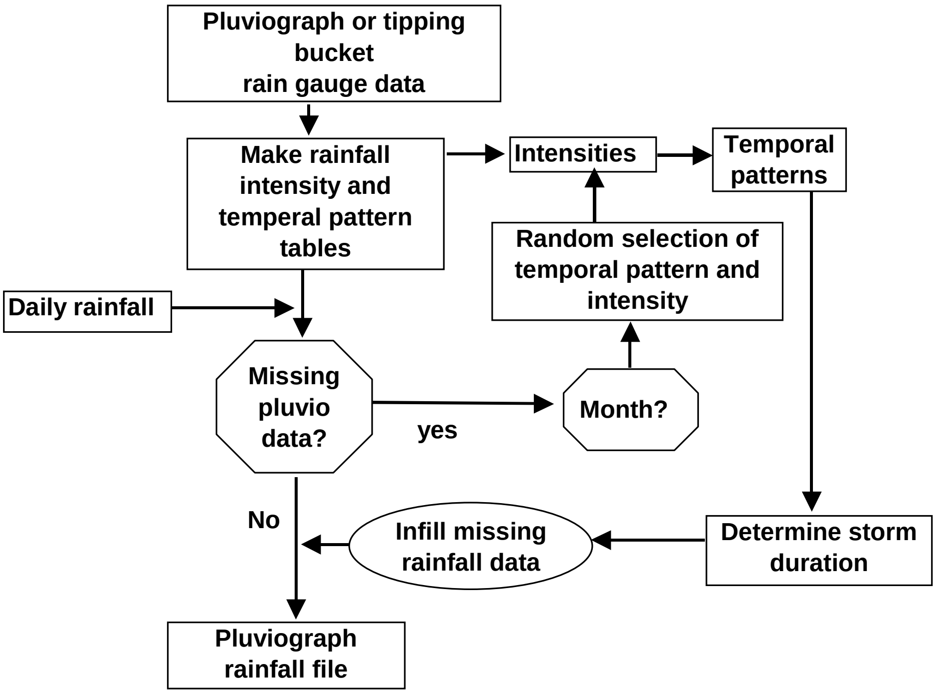

Figure 6.5: A schematic of the Pluviograph Rainfall Generator

The normalised temporal patterns used in the Pluviograph Rainfall Generator are developed using normalised rain depth (cumulative rain depth/total rain depth for an event) and normalised time (time/duration of rain event). Average rainfall intensity is derived as the average rainfall intensity of each rain event. On a day with missing pluviograph data the randomly selected average rainfall intensity is combined the daily rain depth to calculate the duration of the rainfall event. The daily rainfall depth and the calculated duration of the rainfall event are then disaggregated using the randomly selected normalised temporal pattern to produce a synthetic hyetograph for the missing rainfall event.

6.3.2 The DRIP Rainfall Event Model

The DRIP rainfall event model employs a dependence relationship between storm event, average rainfall intensity and time of year, and a temporal pattern generator using a conditional random walk through each rainfall event to disaggregate rainfall down to small intervals (such as six minutes) [Heneker et al., 2001]. A schematic of the rainfall generation process is shown in Figure 6.6.

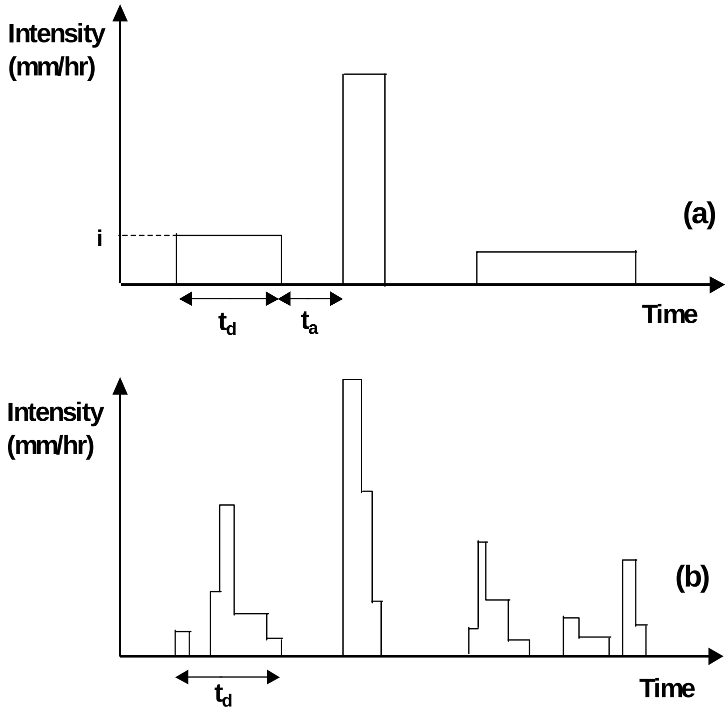

Figure 6.6: The rainfall generation process used in DRIP

In Figure 6.6 part (a) shows the first stage of the rainfall generation process. A time series of rectangular rainfall pulses or storm events is created with average intensities (i), event durations (t_d) and inter-event durations (t_a). Part (b) shows the result of the disaggregation process that generates randomly shaped hyetographs.

6.4 Outdoor Water Demand

Outdoor water use in the Lower Hunter region was shown in Figure 6.2 to be seasonal and highly variable. In Equation 6.2 monthly daily average outdoor water use was shown to be dependent on climatic and socio-economic parameters. Although Equation 6.2 provided a reasonable estimate of monthly average daily outdoor use it was believed that the performance of rainwater tanks is dependant on the day-to-day variation of water use. Given that outdoor water use forms a large portion of total domestic water use (25% in the Lower Hunter region) and the Allotment Water Balance Model uses a diurnal water use pattern, it was important to find a reliable method to simulate daily outdoor water use.

Dandy [1986] proposed two domestic water use models. The first used annual water allowance, number of residents and garden area watered and the second used annual water allowance, number of adults, number of children, number of infants and garden area watered to predict annual water use. Both models produced low values for the coefficient of determination (R^2) of 0.34 and 0.35 respectively for total domestic water use. Kuczera and Ng [1994] provided a model of monthly outdoor water demand that relied on price and climate. The climate variables employed were monthly rainfall and potential evaporation. The model also produced a low value for R^2 of 0.39 indicating it was a poor predictor of monthly outdoor water use.

Mitchell [1997, pp. 77] proposed a method to predict outdoor water use that was based on a soil water balance. The parameters, soil moisture storage at the previous time step, soil moisture storage capacity, precipitation, evapotranspiration and rainfall excess during the current time step were used to simulate outdoor water use. The homeowner's decision to water a garden was triggered in the model by a minimum soil moisture storage level.

Current demand models, as typified by those of Dandy [1986] and Kuczera and Ng [1994], are based on regression fitted to physical parameters or as shown by Mitchell [1997] are based on soil moisture conditions. However it is hypothesised that outdoor water use is a behavioural reaction to climate and social factors rather than a deterministic relationship dependent solely on soil moisture or climate data. This Section presents a new model of outdoor water use that has substantially better predictive ability than earlier models. The model operates on a daily time step and simulates consumer behaviour using a probabilistic framework. Following a description of the model, results are presented for the Lower Hunter region.

6.4.1 Development of the Outdoor Water Use Model

Domestic outdoor water use such as garden watering, car washing and filling of swimming pools is seen to be a recreational pastime that is dependant on human behaviour. It is hypothesized that outdoor water use behaviour is significantly modified by human reaction to daily temperature, days without rainfall and rainfall depth.

The probability of outdoor water use is expected to increase as the length of a period without rainfall increases and the volume of water used is a function of temperature and normal water use patterns (the monthly average daily demand defined by Equation 6.2). People are more likely to use water outside of the house when it is hot and dry, and in accordance with habits.

During a day with rainfall there is a smaller probability of water use and the volume of water used is dependent on the rainfall depth. There is a chance of outdoor water use when people perceive rainfall depth to be insignificant and, conversely, when rainfall depth is perceived to be large there will be no outdoor water use. When that rainfall depth is sufficiently high, people may not use water outside of the house for a number of days.

These behavioural considerations have been formalised into a probabilistic framework. The outdoor water use for day t, W_t, is conditioned on whether day t is wet or dry. Define R_t as the rainfall depth for day t. The model is presented in two parts:

6.4.2 Dry Day ($R_t = 0$)

The conditional probability of outdoor water use ($W_t > 0$) given a dry day t, is a function of days without rainfall DWR_t and the future outdoor water use flag F_t (defined in Section 6.4.3).

$$P(W_t > 0 \mid R_t = 0, F_t, DWR_t) = \begin{cases} \min(1,\ \alpha + \beta DWR_t) & F_t = 0 \\ 0 & F_t = 1 \end{cases} \tag{6.3}$$

where α and β are parameters.

A value U_t is randomly sampled from the uniform distribution spanning 0 to 1.

$$U_t \leftarrow U(0,1) \tag{6.4}$$

If the variate U_t is less than or equal to $P(W_t>0 \mid R_t=0, F_t, DWR_t)$ then outdoor water use is a function of monthly average daily outdoor water use (AD_t) and daily maximum temperature (T_t). Otherwise there is no outdoor water use.

$$W_t = \begin{cases} \lambda AD_t + kT_t & \text{if } U_t \le P(W_t > 0 \mid R_t = 0) \\ 0 & \text{Otherwise} \end{cases} \tag{6.5}$$

where λ is a multiplier of monthly average daily outdoor water use and k is the temperature multiplier.

6.4.3 Wet Day ($R_t > 0$)

The conditional probability of outdoor water use (W_t >0) on a wet day (R_t >0) is μ if the future outdoor water use flag F_t equals 0. Otherwise there is no outdoor water use.

$$P(W_t > 0 \mid R_t > 0,\ F_t) = \begin{cases} \mu & \text{if } F_t = 0 \\ 0 & \text{if } F_t = 1 \end{cases} \tag{6.6}$$

If the variate U_t, sampled from U(0,1), is less than or equal to $P(W_t>0|R_t>0,F_t)$ then the volume of outdoor water use is a function of monthly average daily outdoor water use (AD_t), daily rainfall (R_t) and the average daily rainfall (ADR). Outdoor water use (W_t) on a wet day (R_t > 0) is assumed to be limited by the depth of rainfall (R_t) when the depth of rainfall is greater than ADR. People will reduce their outdoor water use when they perceive that the rainfall depth is significant. If U_t is greater than $P(W_t>0|R_t>0, F_t)$ there is no outdoor water use.

$$W_t = \begin{cases} \max\left(0, 1 + \varphi - \frac{\varphi R_t}{ADR}\right) AD_t & \text{If } U_t \le P(W_t>0|R_t>0,F_t) \\ 0 & \text{Otherwise} \end{cases} \tag{6.7}$$

where φ is a parameter

If rainfall multiplied by the parameter φ is greater than or equal to average daily rainfall (ADR) then the number of successive days (NR_t) when there is no outdoor water use is a function of rainfall and average daily rainfall. The maximum value for NR_t, NR_{max}, was set at 5 days.

$$NR_t = \begin{cases} \min\left(\text{int}\left[\dfrac{Y\varphi R_t}{ADR} \right], NR_{max} \right) & \text{if } \dfrac{\varphi R_t}{ADR} \geq 1 \\ 0 & \text{Otherwise} \end{cases} \qquad (6.8)$$

where Y is a factor used to calculate number of successive days of zero outdoor water use.

The future outdoor water use flag F_t is set to 1 for NR_t days following day t. Thereafter, the flag is set to 0.

$$F_{t+i} = \begin{cases} 1 & i = 1, \ldots, NR_t \\ 0 & i = NR_t + 1, \ldots, \infty \end{cases} \qquad (6.9)$$

A computer program Exhouse, implementing Equations (6.3) to (6.8) was developed to allow simulation of daily outdoor water use.

6.4.4 Outdoor Water Use Results

The Exhouse model was calibrated to HWC monthly outdoor water use totals for each of the 9 zones in Table 6.2 using the SCE-UA (shuffled complex evolution method developed at the University of Arizona) global optimisation method [Duan et al., 1994].

Table 6.2: Climate and water use data for zones in the Lower Hunter region

Zone	Ave. rain (mm/day)	Daily max. temp. (^{0}C)			Monthly Ave. Rain Days			Ave. monthly exhouse demand (L/day)		
		Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
Inner SE Newcastle	3.07	8.9	42	23	0	25	11	44	161	105
Hamilton Mayfield	3.07	8.9	42	23	0	25	11	28	126	75
Lambton Jesmond	3.07	8.9	42	23	0	25	11	19	126	63
NW Wallsend	3.07	8.9	42	23	0	25	11	56	271	150
Lake Macquarie East	2.78	8.9	42	23	0	27	13	32	245	124
Lake Macquarie West	3.24	8.9	42	23	0	27	12	57	236	150
Maitland	2.45	5.8	43	24	0	25	7	107	329	213
Cessnock	2.05	4.2	44	24	0	18	7	56	213	133
Port Stephens	3.68	9.2	41	23	0	26	11	54	368	218

The calibration process involved searching for the parameters α, μ, λ, β, φ, Y and k which minimised a goodness-of-fit objective function. Each set of trial parameters was used in

Equations 6.3 to 6.8 to simulate outdoor water use for each day that coincides with the observed data. The calculated daily outdoor water use volumes were summed to monthly totals (Pred) and compared to the observed monthly water use totals (Obs) using the least squares objective function:

$$OBJ = \sum_{i=1}^{n} (Pred_i - Obs_i)^2 \tag{6.10}$$

where i is the month index and n denotes the number of months in the observed data.

The SCE-UA method is continued until the value of the objective function is minimised. An example of calibration to outdoor demand data for the Mayfield area is shown in Figure 6.7. The outdoor demand model was able to reliably simulate the strong seasonal demand trends experienced in the Mayfield area. Strong seasonal outdoor demand trends are typical for all the zones in the Lower Hunter region.

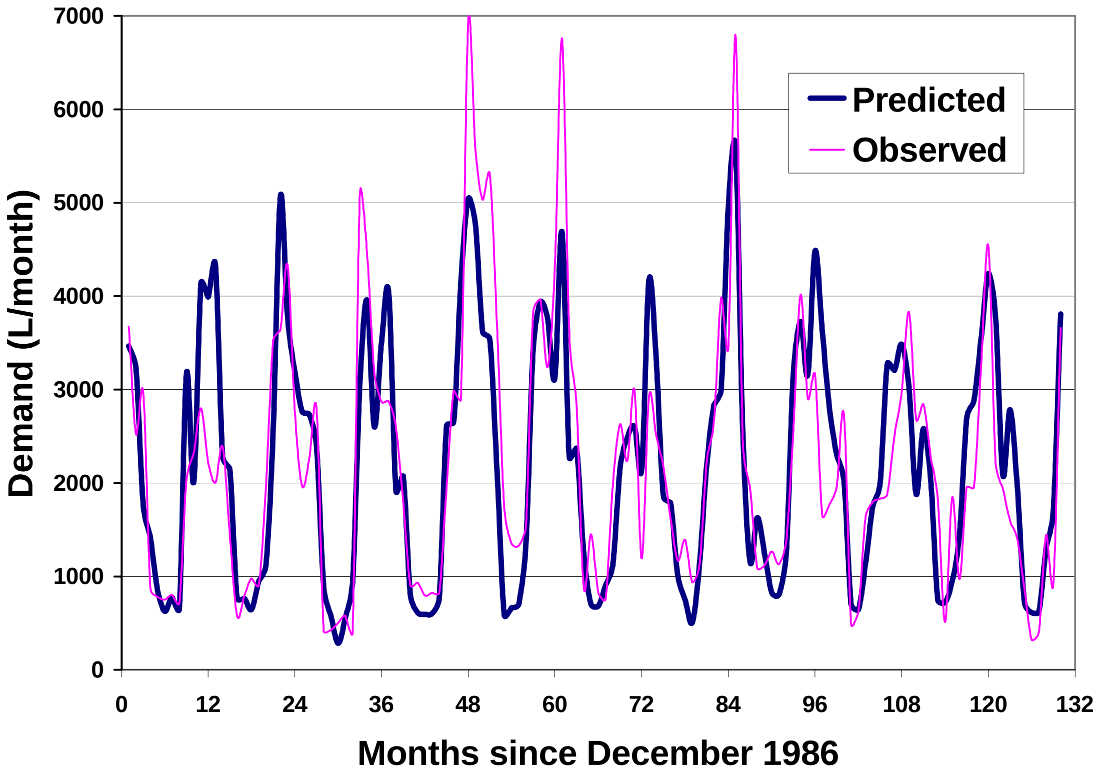

Figure 6.7: Time series of observed versus predicted monthly outdoor water use for the Hamilton Mayfield zone.

The predicted monthly outdoor demand is compared to observed outdoor demand in the scatter plot shown in Figure 6.8. The R^2 value was 0.67 and the scatter of points showed no evidence of systematic bias. The model was able to reliably predict the range of observed monthly outdoor demands.

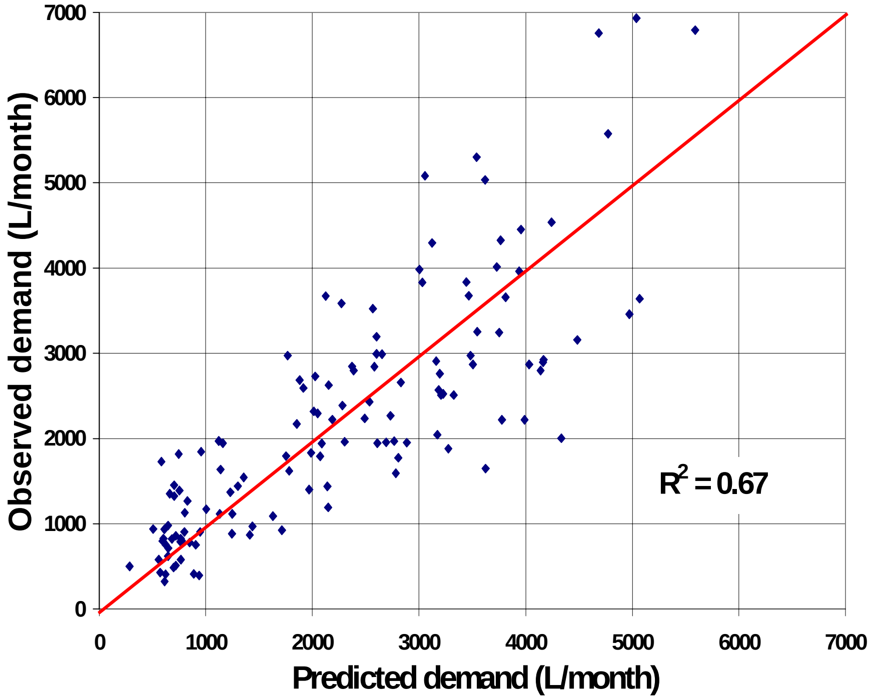

Figure 6.8: Observed versus predicted outdoor water use for Hamilton Mayfield zone.

The results of the outdoor water use model calibration to climate and demand data from all zones in the Lower Hunter region are shown in Table 6.3. The results of Table 6.3 demonstrate consistent performance of the model and provide insight into outdoor water use habits in the different zones.

Table 6.3: Model reliability and parameters from the Lower Hunter zones

Zone	R^2	α	μ	λ	β	φ	Y	k
Lambton Jesmond	0.59	0.97	0.06	0.013	0.0076	3.00	1.40	0.068
Hamilton Mayfield	0.67	0.96	0.05	0.008	0.0055	1.35	0.64	0.067
Inner SE Newcastle	0.59	0.97	0.06	0.021	0.0255	1.00	1.50	0.068
NW Wallsend	0.61	0.97	0.06	0.019	0.0133	0.35	1.65	0.069
Maitland	0.49	0.93	0.09	1.222	0.0300	1.45	1.95	0.003
Cessnock	0.51	0.95	0.1	1.268	0.0320	0.72	0.50	0.001
Lake Macquarie East	0.55	0.95	0.06	0.277	0.0048	1.00	3.00	0.055
Lake Macquarie West	0.58	0.93	0.08	1.035	0.0039	1.30	2.10	0.027
Port Stephens	0.57	0.94	0.07	1.307	0.0099	0.50	2.00	0.015

The results (Table 6.3) show that R^2 values ranged from 0.49 for the Maitland zone to 0.67

for the Hamilton Mayfield zone. This is a significant improvement when compared to the R^2 values from the relationships developed by Dandy [1986] of 0.34 and Kuczera and Ng [1994] of 0.39.

When the future water use flag F equals 0, the probability of outdoor water use on a day without rainfall (α) is close to 1 with results ranging from 0.93 to 0.97 for the region. The length of the period without rainfall at which outdoor water use is a certainty on any day is a function of β and days without rainfall. The period ranges from two days in the Cessnock zone to 15 days in the Lake Macquarie West zone. The frequency of outdoor water use in the Cessnock zone could be attributed to the higher temperatures, lower rainfall depths and less frequent rainfall experienced in that zone (Table 6.2). In contrast the lower frequency of outdoor water use in the Lake Macquarie West zone may have resulted from the greater rainfall depths, more frequent rainfall and lower temperature in that zone.

The volume of outdoor water use on a day without rain is a function of monthly average daily outdoor water use and daily maximum temperature. The parameters λ and k determine the influence of normal water use habits and temperature on the volume of outdoor water use. The volume of outdoor water use in the Cessnock zone, which is subject to higher temperatures, fewer days with rainfall and smaller daily average rainfall depth (Table 6.2) than the coastal zones shows less dependence on the daily maximum temperature than do zones closer to the coast.

When the future water use flag F equals 0, the probability of outdoor water use on a day with rainfall (μ) is small varying from 0.05 in the Hamilton Mayfield zone to 0.1 in the Cessnock zone. The greater probability of outdoor water use on days with rain at Cessnock could be attributed the zone's drier climate. The volume of water used will depend on the parameter φ, rainfall depth and daily average rainfall.

The maximum rainfall depth that will allow outdoor water use on a day with rainfall ranges from 3 mm in the Lambton Jesmond zone to 10 mm in the Port Stephens zone. The differences in volumes of water used on wet days between the Lambton Jesmond and the Port Stephens zones may be attributed to differences in soil types. The Port Stephens zone has sandy soils with low water retention and the Lambton Jesmond zone has clay soils that retain water.

The Exhouse model has shown improved prediction ability when compared to traditional models of outdoor water use but it is important to note that the Exhouse model was calibrated using monthly water use totals and has not been tested against actual daily water consumption. Future research will need to verify the Exhouse model using daily water consumption.

6.4.5 Comparison to the Outdoor Demand Model by Kuczera and Ng

Kuczera and Ng [1994] assumed that outdoor water use is largely dependant on a monthly water deficit defined as the difference between potential evapotranspiration and rainfall. The Exhouse model developed in this thesis has very different assumptions. The performance of the Kuczera and Ng [1994] model is compared to the Exhouse model in order to highlight the differences between the results of a traditional outdoor water use and the new Exhouse model.

Kuczera and Ng [1994] assumed that outdoor water demand is dependent on price and climate. They defined the climate dependency as a monthly water deficit Δ.

$$\Delta = \begin{cases} E - R & E > R \\ 0 & \text{Otherwise} \end{cases} \tag{6.11}$$

where E is monthly potential evapotranspiration (mm) and R is monthly rain depth (mm). The monthly water deficit (Δ) is included the relationship for outdoor water use.

$$Q_{ex} = \Delta D_{ex}\left[d_{ex} + \left(1 + d_{ex}\right)\exp\left(-\gamma_{ex} P_w\right)\right] \tag{6.12}$$

where Q_{ex} is the outdoor consumption (L/connection/day), P_w is the water price ($/kL), D_{ex} is the maximum consumption level per unit of water deficit (L/connection/day/Δ mm), d_{ex} is the minimum level of outdoor use per unit of water deficit and γ_{ex} is the price parameter (kL/$). The models are compared for the Hamilton Mayfield zone in Figure 6.9.

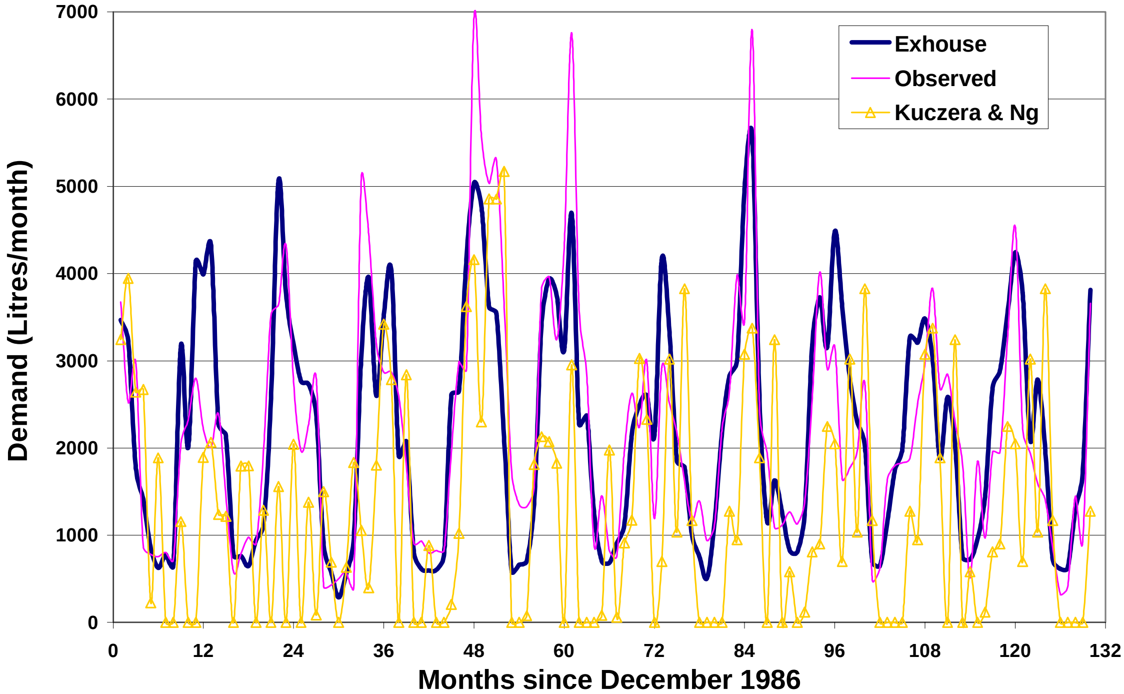

Figure 6.9: Time series of monthly outdoor water use predicted using the Exhouse model and the model by Kuczera and Ng and observed outdoor water use at the Hamilton Mayfield zone

In Figure 6.9 the Exhouse model is shown to have better ability to simulate the seasonal variation in outdoor water demand and the winter outdoor water demand. A time series of residuals, defined as the difference between observed outdoor water use and outdoor water use predicted using the Exhouse and the Kuczera and Ng [1994] models is shown in Figure 6.10.

The outdoor water use model by Kuczera and Ng [1994] is shown in Figure 6.10 to produce greater variation in residuals than the Exhouse model. The new Exhouse model shows improved ability in comparison to the model by Kuczera and Ng [1994] to simulate seasonal trends in outdoor water use. The improved performance is attributed to more realistic behavioural rules for outdoor water use and the use of daily rather than monthly input data.

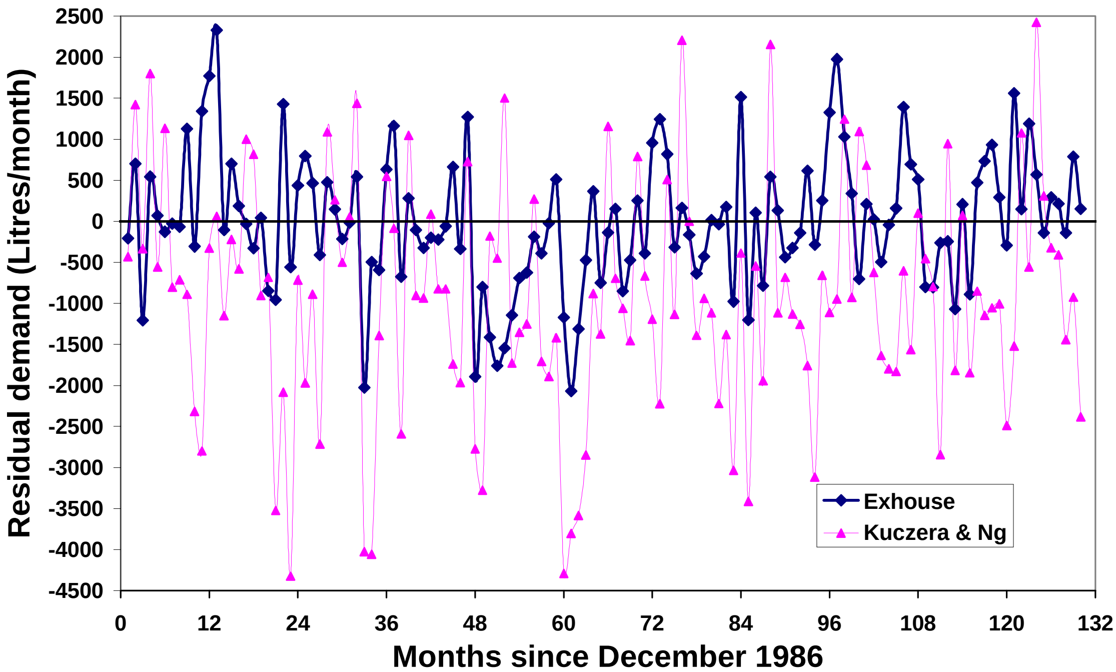

Figure 6.10: Time series of residuals obtained as the difference between observed and predicted outdoor water use for the Hamilton Mayfield zone.

6.5 Indoor Water Demand

The indoor water use data from the HWC [Berghout, personal communication, 1999], the Figtree Place development (Chapter 2) and the Stringybark Grove development [Cox and Cartwright, 1998] was used to establish water use categories and patterns. The water use categories are shown in Figure 6.11. The total indoor water demand is based on indoor water use in the Lower Hunter region (Appendix G) and is calculated using Equation 6.1. The water use categories and proportions shown in Figure 6.11 are indicative of those used to allocate indoor water use from the rainwater tank in the Allotment Water Balance model. The monthly variation of water use for hot water and toilet flushing as a proportion of total indoor water use from the Figtree Place experiment (Table 2.11) is used in the Allotment Water Balance model to define hot water and toilet flushing demand in each month.

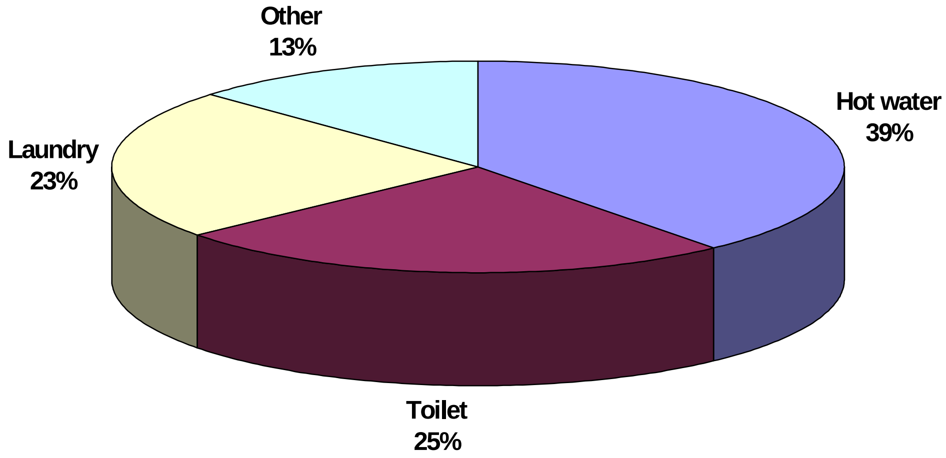

Figure 6.11: Household indoor water use categories

6.6 The First Flush Device

The first portion of rainwater that discharges from a roof is often reported as the first flush [Yaziz et al., 1989; Jenkins and Pearson, 1978 and Cunliffe, 1998]. Yaziz et al. [1989], Jenkins and Pearson [1978] and Cunliffe [1998] determine that the first flush of rainfall from a roof catchment may contain higher than average amounts of accumulated dust, bird and animal droppings, leaves and debris. This description of the first flush is confirmed by the results of the roofwater monitoring program at Figtree Place (Chapter 2). The performance of first flush devices was discussed in Section 4.3.3.

The quantity of first flush separation producing acceptable water quality in rainwater tanks is unknown. However Jenkins and Pearson [1978] suggest separating the first 0.25 mm of rainfall for a typical sized household roof. Yaziz et al. [1989] found that separation of the first 0.33 mm of roof runoff improved the bacterial quality of runoff from a small roof catchment.

Many authors [including Jenkins and Pearson; 1978, Mitchell; 1997 and Cunliffe, 1998] describe the first flush as a fixed amount of roof runoff requiring separation. Design and modelling of the performance of first flush devices have been dominated by this belief. However the number of dry days preceding a rainfall event, rainfall intensity and rainfall depth is an indicator of roofwater quality [Yaziz et al., 1989] (Chapter 2). Design of first flush separation devices and simulation of their performance will therefore need to reflect the dynamic nature of roofwater quality and quantity.

The first flush pits (Figure 6.12) at Figtree Place were designed to separate the first 2 mm of roofwater from inflow to the rainwater tanks. The design of these first flush pits was based on rainfall volumes from design storms described in Australian Rainfall and Runoff [IEAust, 1987]. The first flush pits proved to be so efficient that no inflow to the rainwater tanks resulted. Design of first flush separation devices needs to maximise conservation of roofwater and minimise contaminant transport to the rainwater tank whilst accounting for variation of roofwater quality and quantity from all storms. A conceptual design of a typical first flush device is shown in Figure 6.13.

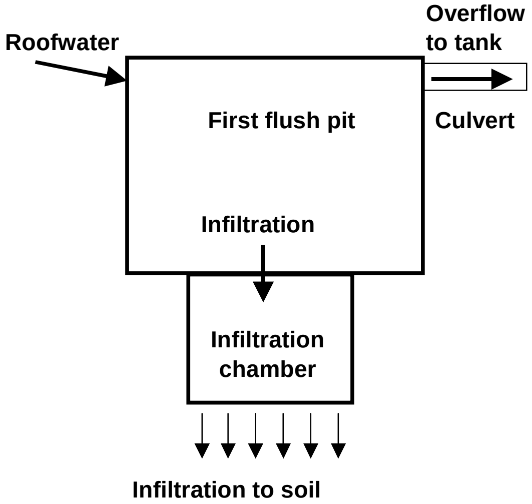

Figure 6.12: First flush pit buried in the ground at Figtree Place.

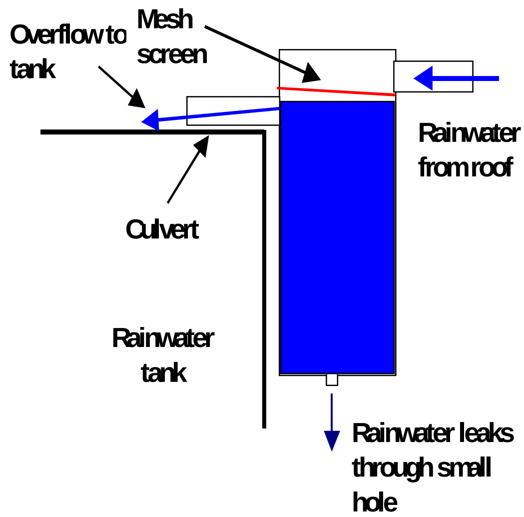

Figure 6.13: A typical above ground first flush separation device.

The design of the first flush device (Figures 6.12 and 6.13) in the Allotment Water Balance model includes an inlet from the roof, a storage area to capture the first flush of rainwater allowing it to leak through a small hole in the base of the chamber, a mesh screen to separate debris and an overflow to the rainwater tank.

6.6.1 Development of a Model to Simulate the Performance of the First Flush Device

A model to simulate the performance of the first flush device was developed below for inclusion in the Allotment Water Balance model to allow assessment of different design options. Overflow from the first flush pit to the rainwater tank OF_t (m^3/s) is a function of roofwater inflow Rin_t (m^3/s), separation of roofwater inflow via infiltration through the base of the pit In_t (m^3/s) and volume of water stored in the first flush device FFs_t (m^3) at time t and the volume of roofwater stored in the device from the previous time step FFs_{t-1} (m^3).

$$FFs_t = FFs_{t-1} + (Rin_t - In_t - OF_t)\Delta t \tag{6.13}$$

where Δt is the time interval (s) between the time step t-1 and t.

The first flush pit discharges to the rainwater tank via a circular pipe with a square edge entrance. The rainwater tank is assumed to be full when its water level is at the invert level of the inlet culvert and will overflow via an outlet culvert. Inlet control will govern the hydraulic behaviour displayed by the inlet culvert in this situation. Boyd [1985] determined that two distinct hydraulic regimes exist for culverts subject to inlet control dependent on whether the inlet is submerged or not submerged. The relationships are as follows:

$$OF = \begin{cases} 1.32D^{0.87}HW^{1.63} & \dfrac{HW}{D} < 1.2 \\ 1.62D^{1.87}HW^{0.63} & \dfrac{HW}{D} > 1.2 \end{cases} \tag{6.14}$$

where the discharge OF (m^3/s) is a function of the pipe diameter D (m) and the depth of water above the culvert invert HW (m). These relationships (Equation 6.13) were adapted to describe discharge from the first flush pit to the rainwater tank.

A relationship for infiltration through the hole in the base of the first flush device can be determined using the Bernoulli's equation [see Streeter and Wylie, 1983]. The relationship for discharge through the infiltration hole In_t (m^3/s) is:

$$In_t = C_c\left(\sqrt{2g}H\right)A \tag{6.15}$$

where C_c is the contraction coefficient ($C_c \approx 0.6$), g is the gravitational acceleration constant (9.806 m/s^2), H is the depth of water in the first flush device (m) and A is the area of the infiltration hole (m^2).

Simulation of infiltration through the hole in the base of the first flush pit is adapted from equation 6.14. Routing of roofwater through the first flush pit is described below. The water level WL_t (m) in the first flush pit is a function of raindepth R_t (m), roof area RA (m^2), infiltration from the pit In_t, overflow to the rainwater tank OF_t (m^3/s), area of the pit PA (m^2) at time t and water level at the previous time step WL_{t-1} (m).

$$WL_t = \begin{cases} WL_{t-1} + \dfrac{(R_t - IL)RA - (OF_t + In_t + OL_t)\Delta t}{PA} & R_t > IL \\ WL_{t-1} - \dfrac{(OF_t + In_t)\Delta t}{PA} & R_t < IL \end{cases} \qquad (6.16)$$

where the initial rainfall losses from the roof are defined as IL (m), PA is the surface area of the first flush device (m^2) and overflow from the roof when the capacity of the roof gutter system GC (m^3) is exceeded at time t is OL_t (m^3/s).

$$OL_t = \begin{cases} \dfrac{((R_t - IL).RA - GC)}{\Delta t} & (R_t - IL).RA > GC \\ 0 & (R_t - IL).RA < GC \end{cases} \qquad (6.17)$$

The volume of infiltration Vf_t (m^3) from the first flush device at time t is defined as a function of the area of the infiltration hole (A), the time interval (Δt), the capacity of the infiltration chamber CHS (m^3) below the device, the first flush water level WL_t and the volume of water $CHEA_t$ (m^3) in the infiltration chamber at time t. The infiltration chamber (Figure 6.12) can be used in below ground first flush devices.

$$Vf_\tau = \begin{cases} (0.6\sqrt{19.612\,WL_t})A.\Delta t & Vf_t \le CHS - CHEA_t \\ CHS - CHEA_t & Vf_t > CHS - CHEA_t \\ 0 & WL_t = 0 \end{cases} \qquad (6.18)$$

The volume of water stored in the infiltration chamber ($CHEA_t$) is a function of the capacity of the infiltration chamber (CHS), the infiltration volume from the first flush device (Vf_t), the surface area of the chamber CA (m^2), the infiltration rate of the soil θs (m/s) and the volume of water stored in the infiltration chamber at the previous time step $CHEA_{t-1}$.

$$CHEA_t = \begin{cases} CHEA_{t-1} + Vf_t - \theta_s.CA.\Delta t & \text{if } 0 \le CHEA_t \le CHS \text{ and } \theta s.\Delta t \le CHEA_{t-1} + Vf_t \\ CHS & \text{if } CHEA_{t-1} + Vf_t - \theta s.\Delta t \ge CHS \\ 0 & \text{if } \theta s.\Delta t \ge CHEA_{t-1} + Vf_t \end{cases} \tag{6.19}$$

The water level above the pipe invert (HW_t) is the difference between the height of the invert of first flush overflow pipe Dpit (m) and the water level (WL_t) at time t.

$$HW_t = WL_t - Dpit \tag{6.20}$$

Overflow to the rainwater tank (OF_t) is a function of the first flush overflow pipe diameter (D) and the water level above the pipe invert (HW_t).

$$OF_t = \begin{cases} 1.32D^{0.87}HW_t^{1.63} & 0 < \frac{HW_t}{D} \le 1.2 \\ 1.62D^{1.87}HW_t^{0.63} & \frac{HW_t}{D} > 1.2 \\ 0 & HW_t < 0 \end{cases} \tag{6.21}$$

A computer program FFpit, implementing equations (6.13) to (6.21) was developed to allow continuous simulation of the performance of a first flush pit incorporated in the Allotment Water Balance model. A time interval of 1 second was used during storm events to ensure accurate simulation of first flush dynamics.

6.6.2 Results

The performance of the FFpit model was verified using a comparison with the monitored data from a first flush pit at Figtree Place. The first flush pit has an area of 0.72 m^2, a depth of 0.5 m to the invert of the outlet pipe with a diameter of 0.1 m, an infiltration hole with a diameter of 0.008 m and a contributing roof area of 175 m^2. The comparison is shown in Figure 6.14.

Figure 6.14 shows the predicted and observed water levels to be consistent. The FFpit model was able to predict water levels in the first flush pit with good accuracy using a 0.85

mm initial loss of roofwater from the roof and a time of concentration of 150 seconds for roofwater reaching the pit.

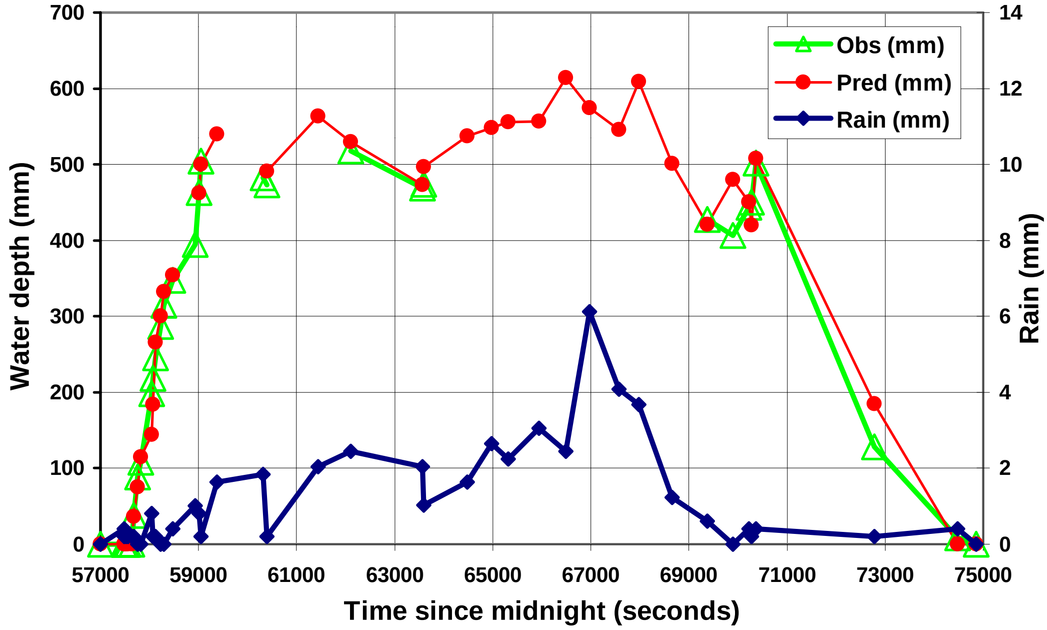

Figure 6.14: Comparison between observed and predicted water levels in a first flush pit at Figtree Place

The FFpit model was combined with the SCE-UA global optimisation method [Duan et al., 1994] (Section 6.4.4) to determine the area and depth required to separate the first 1 mm of roofwater from the inlet to the rainwater tank (Table 6.4). An optimisation scheme was developed to maximise water conservation whilst penalising the supply of roofwater to the rainwater tank within the first one millimetre of rainfall within a storm event. The objective function OBJff was developed to maximise transfer of rainwater from the first flush device to the rainwater tank $Volff_s$ (m^3) whilst heavily penalising transfer of rainwater to the rainwater tank when the first flush separation depth $ffSep_s$ (m) is less than 0.001 m for event s. The objective function was evaluated over n storm events as follows:

$$OBJff = \sum_{s=1}^{n} \begin{cases} Volff_s & \text{if } ffSep_s > 0.001 \\ Volff_s - 10000(0.001 - ffSep_s)^3 & \text{if } ffSep_s < 0.001 \end{cases} \qquad (6.22)$$

The SCE-UA method was continued until the value of the objective function was maximised. The area and depth of the first device that produced the maximum value for OBJff were accepted as the optimum dimensions.

An above ground (Figure 6.13) installation of first flush devices was optimised. Therefore the volume of the infiltration chamber (Figure 6.12) below the first flush device was assumed to be infinite. Performance of the first flush devices was continuously simulated using a synthetic pluviograph record of 64 years for the Maryville zone in the Lower Hunter region. This record was generated using the pluviograph rainfall generator described in Section 6.3.1. Table 6.4 reports the optimal dimensions for different roof areas assuming an outlet pipe diameter of 0.1 m and an infiltration hole with a 5 mm diameter.

Table 6.4: Optimum first flush device dimensions to separate 1 mm of roofwater in the Maryville area.

Roof area (m^2)	Pit area (m)	Pit depth (m)
100	0.26	0.3
135	0.39	0.3
175	0.3	0.39
215	0.4	0.38
250	0.4	0.48

The values for the roof areas and associated first flush device dimensions will be used in analysis throughout the remainder of the thesis.

6.7 The Rainwater Tank

A schematic of the rainwater tank used in the Allotment Water Balance model is shown in Figure 6.15. Rainwater overflows from the first flush device into the rainwater tank, water is drawn from the tank for household use and the tank is topped up with mains water if the water level is less than the prescribed minimum water level. Overflow from the rainwater tank is routed via a pipe system to an OSD tank, infiltration trench or the street drainage system. Spills from the rainwater tank are directed to impervious areas when the tank is full. The design of the tank also includes an airspace above the invert of the overflow pipe for backflow prevention and stormwater detention.

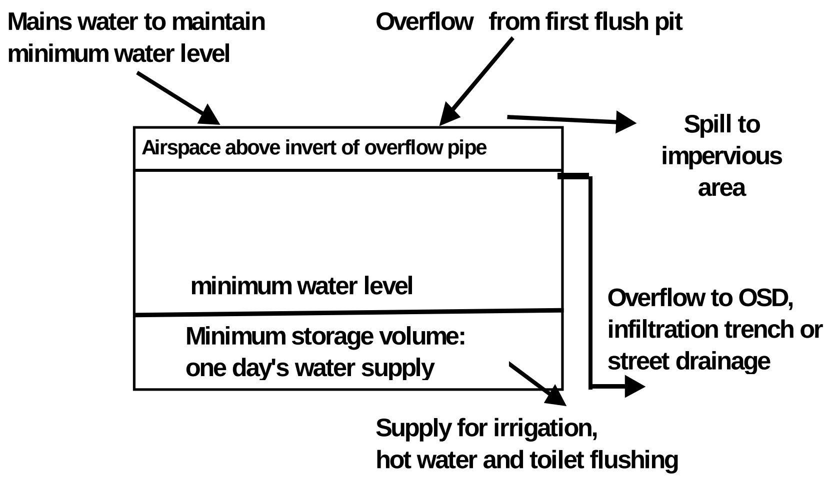

Figure 6.15: Schematic of the rainwater tank used in the Allotment Water Balance model

The volume of water stored in the rainwater tank Vol_t (m^3) at time t is a function of inflow volume from the first flush device $OF_t\Delta t$ (m^3), mains water trickle top up volume $TopUp_t$ (m^3), the volume of water use from the tank Use_t (m^3), overflow volume from the tank OT_t (m^3), the volume of spills from the tank $Spill_t$ (m^3) and the water volume in the tank at the previous time step Vol_{t-1} (m^3) as shown in Equation 6.23.

$$Vol_t = OF_t\Delta t + TopUp_t + Use_t - OT_t - Spill_t + Vol_{t-1} \tag{6.23}$$

Water use from the tank (Use_t) has been defined in Sections 6.4 and 6.5. The inflow to the rainwater tank from the first flush device (OF_t) has been defined in Section 6.6 and overflow from the rainwater tank (OT_t) uses Equation 6.20. Evaporation has not been included in the equation because it is assumed that evaporation from the rainwater tank will be negligible.

The mains water trickle top up volume $TopUp_t$ of the rainwater tank is a function of the minimum water level MinWL (m) in the tank and the mains water top up rate MRate (m^3/s).

$$TopUp_t = \begin{cases} MRate\Delta t & TWL_t + \dfrac{MRate\Delta t}{TA} \le MinWL \\ (MinWL - TWL_t)TA & \dfrac{MRate\Delta t}{TA} + TWL_t > MinWL \\ 0 & TWL_t > MinWL \end{cases} \tag{6.24}$$

where TA (m^2) is the plan area of the rainwater tank and TWL_t is the water level in the rainwater tank at time t.

When the volume of water in the rainwater tank exceeds the capacity of the tank excess water spills ($Spill_t$) from the tank to impervious and pervious areas.

6.8 On Site Detention

The design of the on-site detention (OSD) tank used in the Allotment Water Balance model is shown in Figure 6.16. Stormwater from roofs or rainwater tanks or infiltration trenches, impervious and pervious areas can be directed to the discharge control pit in the OSD tank. The OSD tank has two chambers: a discharge control pit and site storage chamber.

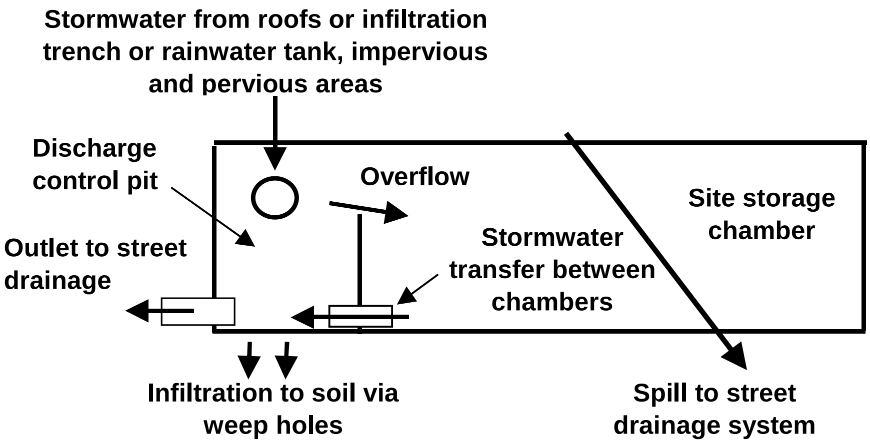

Figure 6.16: Diagram of OSD tank used in the Allotment Water Balance model

Stormwater stored in the discharge control pit is discharged via an orifice to the street drainage system. If the pit is full stormwater overflows into the site storage chamber. The site storage chamber discharges to the discharge control pit as the discharge control pit empties. Stormwater infiltrates via weep holes to the soil below the discharge control pit and spills to the street drainage system when the chamber and the pit are full.

The volume of water in the discharge control pit $Vol1_t$ (m^3) at time t is a function of the volume in the pit at the previous time step $Vol1_{t-1}$ (m^3), inflow volume from the site storage chamber $Vol2in_t$ (m^3), inflow volumes from impervious and pervious areas, rainwater tanks and infiltration trenches $Inflow_t$ (m^3), overflow volume into the site storage chamber $Oflow1_t$ (m^3), discharge volume via an orifice to the street drainage system Otf_t (m^3), infiltration volume to the soil Inf_t (m^3) and the volume of spills from the pit when the volume of water in the pit exceeds the pit's storage capacity $Spill1_t$ (m^3).

$$Vol1_t = Vol1_{t-1} + Vol2in_t + Inflow_t - Oflow1_t - Orf_t - Inf_t - Spill1_t \tag{6.25}$$

The volume of water in the site storage chamber $Vol2_t$ (m^3) at time t is a function of the volume of water in the site storage chamber at the previous time step $Vol2_{t-1}$ (m^3), inflow volume from the discharge control pit $Oflow2_t$ (m^3), volume of discharge from the site storage chamber to the discharge control pit $Vol2in_t$ (m^3) and the volume of spills from the chamber when the volume of water in the chamber exceeds the chamber's storage capacity $Spill2_t$ (m^3).

$$Vol2_t = Vol2_{t-1} + Oflow2_t - Vol2in_t - Spill2_t \tag{6.26}$$

The performance of the OSD tank is simulated in the water balance model using Equations 6.24 and 6.25.

6.9 Infiltration Trench

A diagram of the infiltration trench used in the Allotment Water Balance model is shown in Figure 6.17. Stormwater from roofs or rainwater tanks can be directed to the infiltration trench where it is stored in the void spaces in the gravel for infiltration to the surrounding soil. When the infiltration trench is full it overflows to the street drainage system or to an OSD tank.

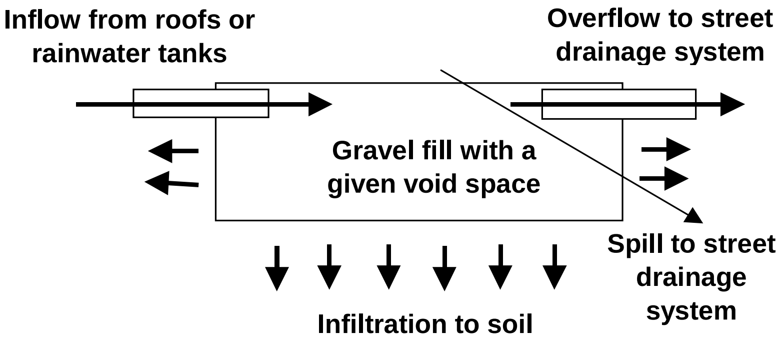

Figure 6.17: Diagram of the infiltration trench used in the Allotment Water Balance model

The volume of water stored in the infiltration trench $Volt_t$ (m^3) at time t is a function of the volume of water in the infiltration trench at the previous time step $Volt_{t-1}$ (m^3), inflow volume from roofs or rainwater tanks $InRR_t$ (m^3), overflow volume to the street drainage system Oft_t (m^3), infiltration volume to the soil $Inft_t$ (m^3) and spill volume to the street drainage system when the volume of water in the infiltration trench exceeds the capacity of the trench $Spillt_t$ (m^3).

$$Volt_t = Volt_{t-1} + InRR_t - Oft_t - Inft_t - Spillt_t \quad (6.27)$$

The volume of infiltration to the soil (Oft_t) is dependent on the infiltration capacity of the soil θs_t (m/s) at time t. The infiltration capacity of the soil is defined in Section 6.10. The storage capacity of the infiltration trench is a function of the void spaces available in the material used to fill the infiltration trench. Argue [1998] determines the capacity of infiltration trenches as follows:

- Trenches filled with gravel: $e_s = 0.35*W*Len*Dt$
- Trenches filled with plastic cells: $e_s = 0.95*W*Len*Dt$

where e_s (m^3) is the effective storage volume, W (m) is the width, Len (m) is the length and Dt (m) is the depth of the trench.

6.10 Pervious Area

A schematic of the pervious area routing used in the Allotment Water Balance model is shown in Figure 6.18. A soil type can be chosen from four categories: 1 sand, 2 silty sand, 3 sandy clay and 4 clay. Each soil type has a dry and saturated infiltration rate, and porosity that has been estimated from Rawls and Brakensiek [1983] (Table 6.5).

Table 6.5: Porosity, dry and saturated infiltration rates for various soil types used in the water balance model

Soil type	Class	Porosity (%)	Dry infiltration rate (mm/hr)	Saturated infiltration rate (mm/hr)
Sand	1	44	250	25
Silty sand	2	44	200	13
Sandy clay	3	43	125	6
Clay	4	48	75	3

The pervious area routine has two storages: surface or depression storage and soil moisture storage in the effective root zone of chosen vegetation (effective root zone assumed to be a depth of 500 mm). Rainfall falls on the soil surface, is stored in the depression storage, infiltrates into the soil moisture store and is transferred to the atmosphere via evapotranspiration. When the depression storage is full it overflows directly to the OSD tank or the street drainage system. The Allotment Water Balance model does not include routing across pervious or non-roof impervious areas. Thus discharges from the pervious surfaces (for example) move instantly to an OSD storage or the street drainage system. This method can produce high peak stormwater discharges when the rainfall interval is small. To remedy this situation rainfall intervals are chosen that are equal to the expected time of concentration from an allotment. The inclusion of time-area routing in the Allotment Water Balance model will eliminate the requirement to use rainfall intervals that are similar to times of concentration.

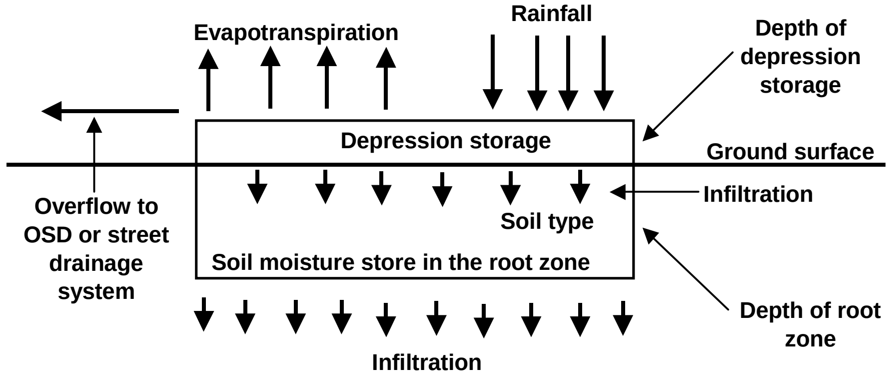

Figure 6.18: Schematic of the pervious area routing in the Allotment Water Balance model

Stormwater stored in the soil moisture store infiltrates to the soil below and is transferred to the atmosphere via evapotranspiration at a rate dependant on the depth of storage. The infiltration capacity of the soil θs_t (m/s) at time t is calculated using the dry infiltration rate Idry (m/s), the saturated infiltration rate Isat (m/s), the soil moisture store $Smstore_t$ (m^3) and the total storage available in the soil Tstore (m^3). The total storage available in the soil is dependent on the value of the soil porosity Por.

$$\theta s_t = Idry + (Idry - Isat)(1\text{-}Smstore_t/Tstore) \tag{6.28}$$

$$Tstore = Por.PervA.Sdepth \tag{6.29}$$

where PervA (m^2) is the area of pervious surfaces on the allotment and Sdepth (≈ 0.5 m) is the depth of soil in the root zone of vegetation on the urban allotment.

The volume of water in the depression storage $Dpstore_t$ (m^3) at time t is a function of rainfall depth falling on the pervious surface $Rperv_t$ (m), overflow from the rainwater tank $Spill_t$ (m^3), volume of garden watering GW_t (m^3), the depression storage volume at the previous time step $Dpstore_{t-1}$ (m^3), evapotranspiration Ep_t (m), infiltration volume to the soil moisture store $DpInf_t$ (m^3), area of pervious surfaces PervA (m^2) and the volume of overflow to the street drainage system or an OSD tank $Dpof_t$ (m^3). Stormwater overflows to the street drainage system or an OSD tank when the capacity of pervious area depression storage Dpmax (m^3) is exceeded.

$$Dpstore_t = Rperv_t.PervA + GW_t + Dpstore_{t-1} - Ep_t.PervA - DpInf_t + Spill_t \tag{6.30}$$

$$\text{If } Dpstore_t > Dpmax\text{, then } Dpof_t = Dpstore_t - Dpmax \text{ and } Dpstore_t = Dpmax \tag{6.31}$$

The volume of water in the soil moisture store $Smstore_t$ (m^3) at time t is a function of the volume of the soil moisture storage at the previous time step $Smstore_{t-1}$ (m^3), infiltration volume from the soil moisture storage to the surrounding soil $Sminf_t$ (m^3), infiltration volume from the depression storage to the soil moisture store $DpInf_t$ and evapotranspiration from the soil moisture store $smEp_t$ (m).

$$Smstore_t = Smstore_{t-1} - Sminf_t + DpInf_t - smEp_tPervA \tag{6.32}$$

Evapotranspiration from the soil moisture store $smEp_t$ is dependant on the volume of water in the depression storage. In the event that evapotranspiration demand is greater than the depression storage the value of $Dpstore_t$ is set to zero. Thus:

$$smEp_t = \begin{cases} Ep_t - Dpstore_t & \text{if } Smstore_{t-1} \geq Ep_t\text{-}Dpstore_t \text{ and } Ep_t > Dpstore_t \\ 0 & \text{Otherwise} \end{cases} \tag{6.33}$$

Infiltration from the depression store to the soil moisture store $DpInf_t$ will be also be dependant on the volume of water in the depression storage. Similarly infiltration from the soil moisture store to the surrounding soil $Sminf_t$ is dependant on the volume of water in the soil moisture store.

Daily potential evapotranspiration Ep_d (mm/day) or evaporation from a well watered reference crop was derived from the method described by Hargreaves and Samani [1985]. The Hargreaves and Samani [1985] method was chosen in preference to other methods of estimating potential evaporation (such as the Penman-Monteith method) because the method required considerably less data. The method uses daily average minimum temperatures Tmin (ºC), daily average maximum temperatures Tmax (ºC) and an empirical variable S_0.

$$Ep_d = 0.0023 S_0 (T\max - T\min)^{0.5} \left(\frac{T\max + T\min}{2} - 17.8 \right) \tag{6.34}$$

The empirical constant S_0 is a function of the relative distance between the earth and the sun d_r, the sunset hour angle ωs, the latitude of the site Lat and the solar declination δ.

$$S_0 = d_r \left[\omega s \sin(Lat) \sin(\delta) + \cos(Lat) \cos(\delta) \sin(\omega s) \right] \tag{6.35}$$

$$\omega s = \arccos\left(-\tan(Lat) \tan \delta \right) \tag{6.36}$$

The solar declination δ of the site and the sun-earth distance d_r are function of the Julian day number J.

$$\delta = 0.4093\sin\left(\frac{2\pi}{365}J - 1.405\right) \tag{6.37}$$

$$d_r = 1 + 0.033\cos\left(\frac{2\pi}{365}J\right) \tag{6.38}$$

The potential evapotranspiration Ep_t (m) at time t is estimated to be function of the daily evapotranspiration Ep_d (mm/day) and the time interval Δt (s).

$$Ep_t = 1000Ep_d\Delta t/(24*60*60) \tag{6.39}$$

6.10.1 Verification of the Hargreaves Model

The Hargreaves model was evaluated using daily minimum and maximum temperatures, and Class A pan evaporation data from 1974 to 1999 from Williamtown Airport near Newcastle. Potential evapotranspiration was generated using the minimum and maximum daily temperatures and compared to the values for pan evaporation at the Williamtown site in Appendix G. Potential evapotranspiration is related to pan evaporation as follows:

$$Ep_d = k_{pan}E_{pan} \tag{6.40}$$

The average value for k_{pan} was found to be 0.548 was calculated using daily values of Ep_d and E_{pan}. Doorenbos and Pruitt [1977] suggest that the value of k_{pan} for a well-watered short green reference crop should be in the range of 0.4 to 0.85 depending on wind strength and humidity. It is assumed that domestic lawns and gardens will have a similar potential evapotranspiration to a well-watered short green reference crop and the average value for k_{pan} is within the expected range. Therefore the Hargreaves equation provides a reasonable estimate for potential evapotranspiration from an urban allotment. The seasonal variation of pan evaporation and evapotranspiration during a typical year is shown in Figure 6.19.

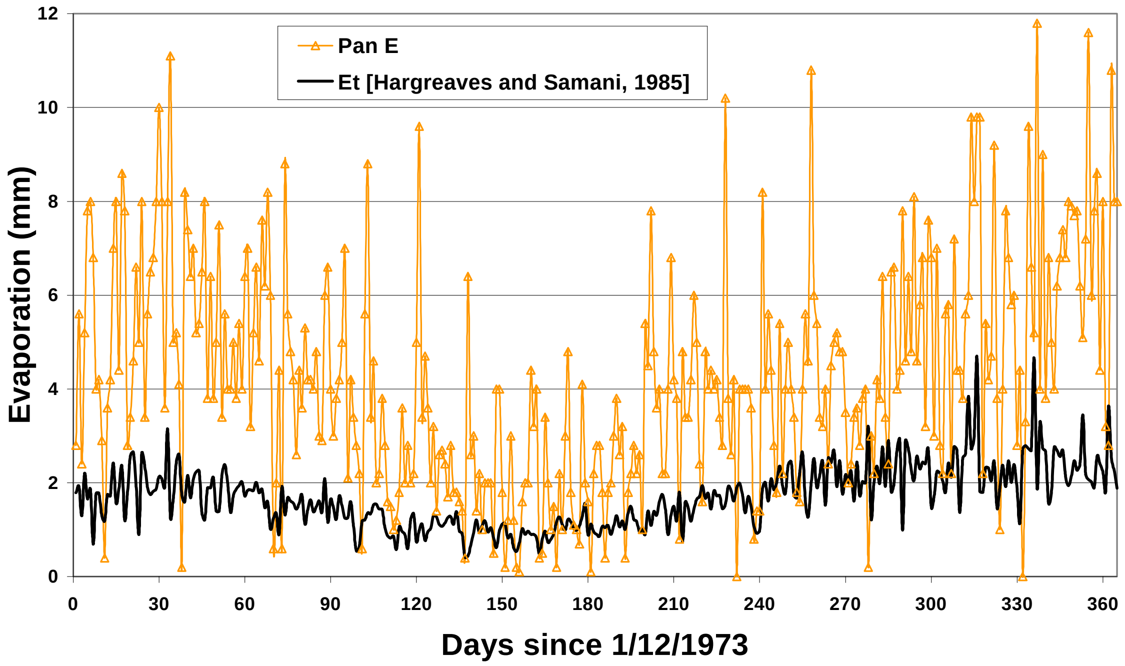

Figure 6.19: Seasonal variation for pan evaporation and potential evapotranspiration

It is shown in Figure 6.19 that the seasonal trends in the estimated potential evapotranspiration are consistent with the observed trends in pan evaporation. It is also clear that the value for the k_{pan} is subject to substantial seasonal variation. The statistics for Ep_d and potential evapotranspiration PE (mm/day) calculated for the nearby Seaham catchment by Wooldridge [2001] using the Priestly-Taylor method are compared in Table 6.6.

Table 6.6: Statistics for potential evapotranspiration and reference crop evapotranspiration

Statistic	Seaham PE (mm/day)	Ep_d (mm/day)
Minimum	2.5	0.2
Maximum	4.7	5.3
Average	3.9	1.7
Standard deviation	2.2	0.7

The results in Table 6.6 suggest that the estimate of reference crop evapotranspiration using the method by Hargreaves and Samani [1985] has under-estimated the values of daily potential evapotranspiration. About 99.9% of the values for Ep_d were found to be less than the maximum value for potential evaporation at Seaham (4.7 mm/day). The Hargreaves and Samani [1985] method does not appear to over estimate the value for reference crop evapotranspiration.

The average value for EP at Seaham is 3.9 mm/day, which is 2.2 mm/day less than the average value for Ep_d. The values for Ep_d appear to underestimate the values for EP. Hargreaves and Samani [1985] suggest that errors of up to 1 mm/day can be expected in the values for Ep_d. In order to correct the underestimation of Ep_d a constant of 2.2 mm/day was added to Equation 6.34 as follows:

$$Ep_d = 2.2 + 0.0023 S_0 (T\max - T\min)^{0.5} \left(\frac{T\max + T\min}{2} - 17.8 \right) \tag{6.41}$$

Equation 6.41 was used to calculate new values of Ep_d for Williamtown Airport that were compared to the statistics for the Seaham catchment. The average value for EP at Seaham and Ep_d were now estimated to be 3.9 mm. About 87% of the values for Ep_d were less than the maximum value for PE at Seaham (4.7 mm) and only 0.1% of the values for Ep_d were less than the estimated minimum value for EP at Seaham. Equation 6.41 was considered to provide reasonable estimates for Ep_d and was included in the Allotment Water Balance model.

6.11 The Non-Roof Impervious area

A schematic of the non-roof impervious area routing used in the water balance model is shown in Figure 6.20. Rainfall falls on the non-roof impervious surface and is stored in the depression storage, which overflows directly to an OSD tank, the street drainage system or a pervious area when the capacity of the storage is exceeded. Overflows from the roof gutter system are also directed to the impervious area.

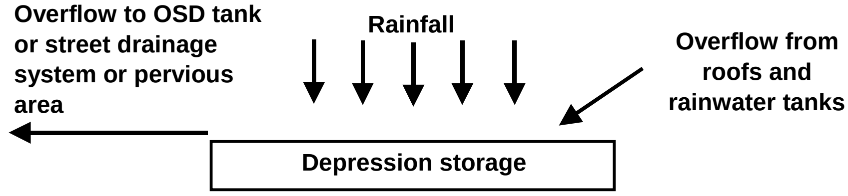

Figure 6.20: Schematic of non-roof impervious area routing in the water balance model

The volume of water in the non-roof impervious area storage $Istore_t$ (m^3) at time t is a function of the volume of water in the non-roof impervious area storage at the previous time step $Istore_{t-1}$ (m^3), rainfall depth falling on the non-roof impervious area $Irin_t$ (m), overflow from the roof gutter system OL_t (m^3), overflow from the rainwater tank $Spill_t$ (m^3), the area of the impervious surfaces IpervA (m^2) and discharge to the street drainage system or an OSD tank Iof_t (m^3). Stormwater overflows to the street drainage system or to an OSD tank occur when the capacity of impervious area depression storage IDpmax (m^3) is exceeded.

$$Istore_t = Istore_{t-1} + Irin_t.IpervA + OL_t + Spill_t \quad (6.42)$$

$$\text{If } Istore_t > Idpmax\text{, then } Iof_t = Istore_t - Idpmax \text{ and } Istore_t = Idmax \quad (6.43)$$

6.12 Analysis of the Maryville Experiment

In this Section the performance of the Allotment Water Balance model was verified against monitoring data from the Maryville Experiment and the model was then used to estimate the long-term performance of the Maryville Experiment.

6.12.1 Verification of the Allotment Water Balance Model

Monitoring results from the Maryville Experiment (Chapter 3) from the period 5/3/2001 to 21/08/2001 were used to verify the performance of the water balance model. The estimated monthly average water demand from the rainwater tank and the mains supply at the Maryville house from Tables F2 and F3 (Appendix F) is shown in Table 6.7. It is assumed that there was no outdoor water use from the rainwater tank. These estimates were used in the water balance model to simulate performance of the dual water supply system at the Maryville house.

Table 6.7: Estimated average demands from the rainwater tank and mains supply

Month	Demand from tank (L/day)	Demand from mains (L/day)
3	320	300
4	180	166
5	160	146
6	130	120
7	155	134
8	178	168

The water levels in the rainwater tank at the end of each day were simulated using the water balance model and compared to the observed water levels from the rainwater tank at the Maryville house in Figure 6.21. The water balance model was able to reliably estimate (R^2 = 0.97) the observed water levels in the rainwater tank as shown in Figure 6.21.

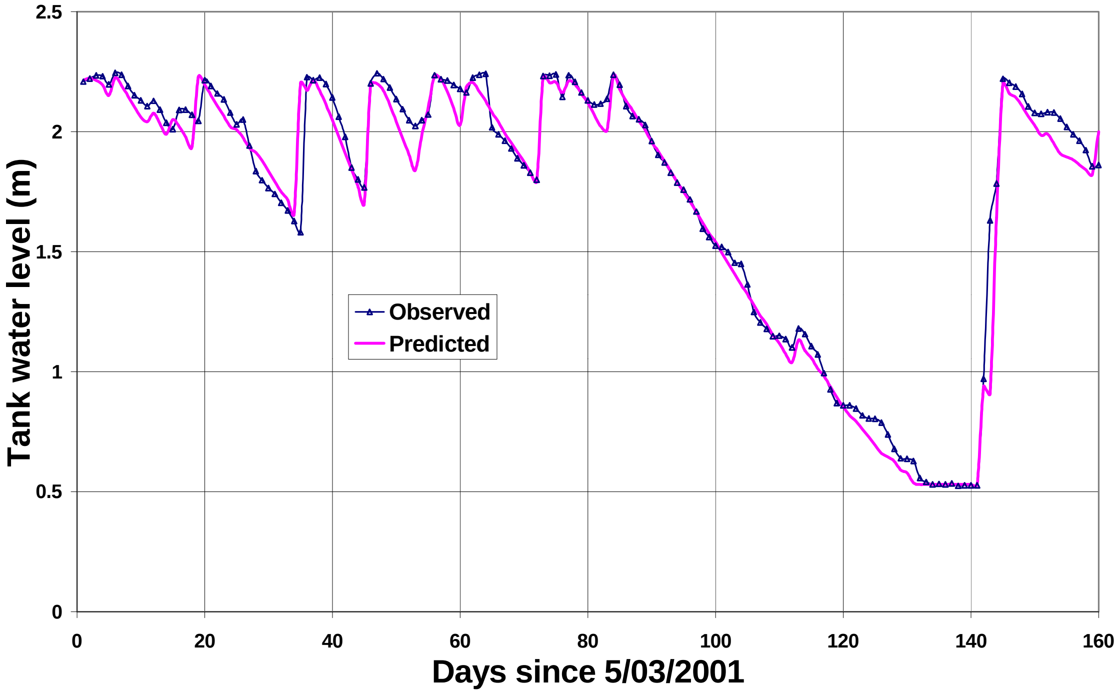

Figure 6.21: Comparison between predicted and observed water levels in the rainwater tank at the Maryville house

The simulated volume of stormwater discharge from the tank was 43.4 m^3 compared very well with the observed value of 42.7 m^3 for the monitoring period. The Allotment Water Balance model was able to reliably simulate the water balance at the Maryville tank.

It is important to note that the outdoor and indoor water demand models in the Allotment Water balance model have not been tested against actual daily outdoor and indoor water consumption. Nonetheless the model was able to reliably simulate daily water levels in the rainwater tank at the Maryville house indicating that knowledge of monthly daily average water use can allow the Allotment Water Balance model to reliably simulate the performance of rainwater tanks. Furthermore it is noted that the pervious and non-roof impervious area routing components of the model were not tested.

6.12.2 Long-Term Performance of the Maryville Experiment

The long-term performance of the dual water supply scheme at the Maryville house was estimated using 64 years of synthetic pluviograph data. The synthetic pluviograph data was generated by the pluviograph rainfall generator described in Section 6.3.1 using the Maryville pluviograph record (1964 – 1993, average annual rainfall = 1112 mm), the Newcastle daily rainfall record (1862 – 1997, average annual rainfall = 1141 mm) and data from the nearby Figtree Place (Chapter 2) tipping bucket rain gauge (Appendix G). The Figtree Place rainfall data was employed to create temporal patterns and storm durations used to generate random rainfall events in the pluviograph rainfall generator. The Figtree Place tipping bucket rain gauge data was chosen to disaggregate daily rainfall into storm events for days with missing pluviograph data because the tipping bucket rain gauge data was at a smaller time intervals (<1 minute) than the Maryville pluviograph data (6 minutes).

The water use from the rainwater tank and mains supply used in the analysis from Tables F2 and F3 (Appendix F) is shown in Table 6.8. It is assumed that there is no outdoor water use from the rainwater tank. The rainwater tank was trickle topped up with mains water to a minimum water level of 530 mm when the tank water level falls below the minimum water level. The trickle top up rate was assumed to be at a rate of 18 Litres per hour.

Table 6.8: Estimated average demands from the rainwater tank and mains supply

Month	Demand from tank (L/day)	Demand from mains (L/day)
1	250	240
2	250	240
3	320	300
4	180	166
5	160	146
6	130	120
7	155	134
8	178	168
9	210	192
10	160	145
11	250	240
12	250	240

The results from the long-term analysis of the performance of the dual water supply system at the Maryville house are shown in Table 6.9.

Table 6.9: Results from the analysis of the water supply systems at the Maryville house

Item	Dual water supply	No dual water supply	Reduction
Average annual mains water use (kL)	45.4	121.1	63%
Average annual stormwater discharge (kL)	50.3	126.0	60%
1-year ARI peak daily mains water demand (L)	330	421	22%
1-year ARI peak instantaneous mains water demand (L/s)	0.01	0.018	44%

The results from Table 6.9 show that the dual water supply system at the Maryville house is expected to reduce mains water use by 63%, stormwater discharges from the site by 60%, the 1-year ARI daily mains water demand by 22% and the 1-year ARI instantaneous mains water demand by 44% over a period of 64 years. The 1-year ARI daily mains water demand is defined as the 64th largest annual maximum daily mains water demand in the 64-year record. Similarly the 1-year ARI instantaneous mains water demand is defined as the 64th largest annual maximum instantaneous mains water demand in the 64-year record.

The peak stormwater discharges from the Maryville house with and without the rainwater tank are compared for various average recurrence intervals (ARI) in Table 6.10. The ARI is defined as the average value of the time period between exceedances of a given stormwater discharge [IEAust, 1987]. It is traditional practice to determine ARIs for stormwater discharges generated by design storm bursts – in this approach the selection of antecedent conditions (for example: how full is the rainwater tank) is problematic. In this analysis the ARI was established from stormwater discharges produced using continuous simulation of storm events over a 64-year period. This ensures that antecedent conditions are properly accounted for.

The dual water supply system at the Maryville house is shown (Table 6.10) to provide considerable reductions in peak stormwater discharges for ARIs between 2 and 0.25 years (18% - 95% reduction). Stormwater quality in urban catchments is influenced by peak discharges from more frequent storm events (ARIs 0.25 – 2 years). The reduction in peak discharges resulting from the use of dual water supply systems for domestic housing is expected to significantly improve stormwater quality in urban catchments.

Table 6.10: Peak stormwater discharges from the Maryville house

ARI	Peak stormwater discharges (m^3/s)		Reduction (%)
	No tank	Tank	
10	0.0063	0.006	5
5	0.0054	0.0051	6
2	0.004	0.0033	18
1	0.0027	0.0021	22
0.5	0.0016	0.001	38
0.25	0.0004	0.0002	95

Stormwater drainage infrastructure in urban catchments is designed to manage stormwater peak discharges from 2 –10 year ARIs. The reductions in peak stormwater discharges from houses with dual water supply systems for the 2 – 10 year ARIs will decrease the requirement for stormwater infrastructure or improve the performance of stormwater infrastructure in urban catchments.

6.13 Long-Term Analysis of the Figtree Place Experiment

Operation of dual water supply systems at Figtree Place was intermittent due to flaws in the design/construction of the project. The dual water supply systems (mains and tank water) could not operate consistently due to faulty solenoid valves. If the solenoid valves were repaired allowing continuous operation of the dual water supply systems at the four unit clusters what would be the long-term reductions in mains water use and stormwater discharges?

The Allotment Water Balance model was utilised to analyse the performance of the unit clusters at Figtree Place using the synthetic Maryville pluviograph rainfall (see Section 6.12). The average monthly total indoor water consumption and the hot water and toilet uses (HT) for different numbers of occupants from Table 2.7 was combined with occupation rates from Appendix C to produce the water use for each unit cluster shown in Table 6.11.

The roof areas and rainwater tank volumes from Table 2.7 and the average monthly indoor water use results from Table 6.11 were used in the water balance model. The ability of the rainwater tanks to capture rainwater at Figtree Place was severely compromised by the poor design of the first flush pits. The first flush pits were oversized resulting in the separation of large volumes of volumes of roofwater from entry to the rainwater tanks. In the model each unit cluster was assigned a first flush pit with a storage volume of 0.35 m^3 and an infiltration hole with a diameter of 5 mm. It was also assumed that there was no outdoor water use from the unit clusters. The results from the analysis are shown in Table 6.12.

Table 6.11: Water use at each unit cluster used in the water balance model

Month	Average monthly indoor water use (L/day)							
	Unit cluster A		Unit cluster B		Unit cluster C		Unit cluster D	
	HT	Total	HT	Total	HT	Total	HT	Total
January	1,466	2,737	1,269	2,623	2,121	4,373	991	1,978
February	1,449	2,598	1,234	2,448	2,036	3,990	941	1,770
March	1,500	2,715	1,354	2,660	2,164	4,246	959	1,828
April	1,335	2,233	1,473	2,488	2,454	4,042	1,074	1,714
May	1,521	2,449	1,722	2,712	2,851	4,452	1,230	1,910
June	1,521	2,449	1,722	2,712	2,851	4,452	1,230	1,910
July	1,521	2,449	1,722	2,712	2,851	4,452	1,230	1,910
August	1,294	2,133	1,536	2,221	2,151	3,470	922	1,433
September	1,552	2,511	1,671	2,411	2,468	4,010	1,154	1,813
October	1,504	2,451	1,739	2,474	2,470	4,155	1,123	1,867
November	1,507	2,449	1,779	2,489	2,510	4,170	1,122	1,866
December	1,466	2,737	1,623	2,623	2,121	4,373	991	1,978

The average reduction in mains water use for the Figtree Place unit clusters was found to be 28% and the average reduction in stormwater discharges was 68% (Table 6.12). In Section 2.6 the average reduction in mains water use was observed to be about 40% from a short monitoring period with significantly higher than the average rainfall. The long-term average reduction in mains water use was expected to be lower than the observed 40%.

Table 6.12: Results from the analysis of the unit clusters at Figtree Place

Item	Unit Clusters				Total
	A	B	C	D	
Annual rainwater use (kL/year)	253	293	378	189	1,113
Annual total water use (kL/year)	911	915	1,528	669	4,023
Reduced mains water use (%)	28	32	25	28	28
Annual stormwater discharge (kL/year)	76	311	160	85	632
Annual roofwater discharge (kL/year)	375	684	587	316	1,962
Reduced stormwater discharge (%)	80	55	73	73	68
1-year ARI daily mains water use (L/day)	2,680	2,465	4,341	1,939	11,425
1-year ARI daily demand (L/day)	2,737	2,712	4,452	1,978	11,879
Reduced annual maximum daily demand (%)	2	9	2	2	4

The Figtree Place dual water supply system (Chapter 2) does not use a mains water trickle top up arrangement (Chapter 3). Therefore significant reductions in 1-year ARI daily mains water demand were not expected. In Table 6.12 it was shown that the maximum daily mains water demand was reduced by an average of 4%.

The results show that the rainwater tanks can provide a substantial average reduction in stormwater discharge volumes (68%). The combination of infiltration trenches, a central infiltration basin and rainwater tanks at Figtree Place ensured that there was no stormwater discharges to the street drainage system (Chapter 2). The impact of the rainwater tanks on reducing peak stormwater discharges is shown in Table 6.13.

Table 6.13: Reductions in stormwater peak discharges from the unit clusters at Figtree Place

ARI	**Reductions in peak stormwater discharges (%)**			
	Cluster A	**Cluster B**	**Cluster C**	**Cluster D**
10	32	15	20	24
5	36	25	31	30
2	50	24	37	34
1	40	29	47	43
0.5	78	31	65	60
0.25	100	100	100	100

The rainwater tanks very significantly reduced the peak stormwater discharge from the unit clusters for ARIs from 10 to 0.25 years. The use of rainwater tanks on sites with unit cluster developments will significantly reduce the requirement for stormwater and water quality infrastructure, and reduce the mains water demand.

The long-term results for the Maryville house showed that the use of the rainwater tank could reduce mains water use by 63% whilst the long-term results from Figtree Place revealed that rainwater tanks reduced mains water use by an average of 28%. The difference in mains water savings is a result of the different daily water demands at the Maryville and Figtree Place sites. The daily water use from the rainwater tank at Maryville (Table 6.8) was considerably less than the daily water use from the Figtree Place rainwater tanks (Table 6.11) even though the rainwater tanks at clusters B and D at Figtree Place and the rainwater tank at the Maryville site had similar effective capacities. There is considerable scope for the optimisation of tank dimensions to maximise mains water savings that has not been explored in this thesis.

6.14 Lower Hunter Case Study

It has been shown that the use of rainwater tanks with mains water trickle top up to supply domestic hot water, toilet and outdoor uses in houses and unit clusters will produce substantial reductions in mains water use and stormwater discharges. In the previous Sections and Chapters individual sites have been analysed. We now pose the question what is the likely impact of installing rainwater tanks to all domestic dwellings in a region such as the Lower Hunter region?

The Lower Hunter region spans five local government areas, namely Newcastle, Lake Macquarie, Maitland, Cessnock and Port Stephens and has a population of 455,000 people with an overall growth rate of 0.9%. The impact of installing a rainwater tank on every domestic dwelling in the region to supply hot water, toilet and outdoor uses is evaluated in this Section.

6.14.1 Demographic and Climate Statistics for the Lower Hunter Region and Assumptions

The mains water demand of a dwelling that also uses water from rainwater tanks will depend on local climatic conditions, water use patterns, dwelling size and the number of occupants. The Lower Hunter region has been divided into nine demographic and climate zones to facilitate modelling of the dual water supply scenarios (Table 6.1). The Australian Bureau of Statistics [1996] provides population, the number of dwellings and annual population growth for these zones (Table 6.14).

Table 6.14: Population statistics for zones in the Lower Hunter region

Zone	Population	Dwellings	Growth (%/year)
Inner SE Newcastle	38,085	15,484	0.24
Hamilton Mayfield	33,513	15,079	-0.77
Lambton Jesmond	35,463	14,369	1.35
NW Wallsend	27,681	10,372	1.64
Lake Macquarie East	113,260	42,104	0.29
Lake Macquarie West	57,080	21,219	1.4
Maitland	49,951	17,528	1.27
Cessnock	44,368	15,775	0.23
Port Stephens	51,189	18,771	2.99

The dwellings in the five local government areas were categorised as either a detached house or a housing unit (Table 6.15). The housing unit category includes flats and apartments. All dwellings in the five areas were also categorised by number of occupants allowing water use modelling for different numbers of occupants and different dwelling types.

Table 6.15: Occupation rates and housing categories for local government areas in the Lower Hunter region

Area	Houses (%)	Units (%)	Occupants per dwelling by area population (%)				
			1	2	3	4	5+
Newcastle	91.5	8.5	20	33.8	16.9	18.1	11.1
Cessnock	95.6	4.4	21.8	31.6	17	17.5	12.1
Port Stephens	95.6	4.4	19.9	35.9	15.6	17.1	11.5
Maitland	95.6	4.4	19.9	30.2	17.9	19.5	12.6
Lake Macquarie	91.5	8.5	20	33.8	16.9	18.1	11.1

Five housing (H1-H5) and two unit cluster scenarios (C1 and C2) (Table 6.16) were considered for each zone. The unit clusters C1 and C2 were based on the unit clusters D and C, respectively, from the Figtree Place experiment (Chapter 2). Roof areas were assumed to increase with the number of occupants in a dwelling and tank sizes were estimated as a function of available land area in typical housing allotments.

Table 6.16: Dwelling characteristics

Item	Occupants	Dwellings	Roof area (m^2)	Tank size (kL)
H1	1	1	100	10
H2	2	1	135	10
H3	3	1	175	10
H4	4	1	215	10
H5	5+	1	250	10
C1	11	4	340	20
C2	24	9	632	30

The climate data used to generate indoor and outdoor water use for the Lower Hunter region is summarised in Tables 6.1 and 6.2. Rainfall data used to make rainfall records for each of the zones in the Lower Hunter region is shown in Table 6.17.

Table 6.17: Availability of rainfall data for the Lower Hunter region

Location	Period	Average rainfall (mm/year)	Average rain days per month
Cessnock Post Office	1903 –1992	748	7
Cessnock (Nulkaba)	1966 – date	763	9
Charlestown	1968 – 1972	1012.9	13
East Maitland	1902 – 1994	894.9	7
Maryville	1964 – 1993	1111.7	11
Newcastle Lighthouse	1862 – date	1141.9	11
Nelson Bay	1881 – date	1342.4	11
Raymond Terrace	1938 – date	1145.7	9
Toronto	1965 – date	1181.9	12
Williamtown RAAF	1942 – date	1126.6	11

Missing data from the Cessnock Post Office was filled with data from Cessnock (Nulkaba) to make a continuous rainfall record for the Cessnock zone and missing data from Maryville was filled using data from Newcastle Lighthouse to make a continuous rainfall record. Gaps in the rainfall records from the Nelson Bay, Maitland, Toronto and Charlestown rain gauges were filled using the daily regression relationships shown in Table 6.18.

Table 6.18: Daily rainfall regression relationships for the Lower Hunter

Location	Relationship	R^2
Charlestown	0.663Newcastle + 0.262Raymond Terrace	0.68
Maitland	0.125Newcastle + 0.682Raymond Terrace	0.65
Nelson Bay	0.402Newcastle + 0.514Raymond Terrace	0.58
Toronto	0.54Newcastle + 0.262Raymond Terrace	0.61

The daily maximum temperature data used to make temperature records for each of the Lower Hunter zones is shown in Table 6.19.

Table 6.19: Availability of daily maximum temperatures for the Lower Hunter region

Location	Period	Daily maximum temperatures (°C)		
		Maximum	Minimum	Average
Cessnock	1973 – date	44	8	24
Newcastle	1957 – date	42	8.9	22
Sydney	1859 – date	45	9.3	21.9
Williamtown	1942 – date	44	8.9	23

Gaps in the daily maximum temperature records from the Newcastle, Cessnock and Williamtown were filled using the daily regression relationships shown in Table 6.20.

Table 6.20: Daily maximum temperature regression relationships for the Lower Hunter

Location	Relationship	R^2
Cessnock	-2.204 + 1.18Sydney	0.83
Newcastle	1.816 + 0.87Sydney	0.86
Williamtown	-1.843 + 1.12Sydney	0.87

The daily rainfall and daily maximum temperature records used in each of the Lower Hunter zones is shown in Table 6.21. A synthetic rainfall pluviograph record was made for each zone using the Pluviograph Rainfall Generator described in Section 6.3.1 with temporal patterns and rainfall intensities from the Figtree Place tipping bucket rain gauge.

Table 6.21: Daily rainfall and maximum temperature records used in the Lower Hunter zones

Zone	Rainfall	Temperature
Inner SE Newcastle	Maryville	Newcastle
Hamilton Mayfield	Maryville	Newcastle
Lambton Jesmond	Maryville	Newcastle
NW Wallsend	Maryville	Newcastle
Lake Macquarie East	Charlestown	Newcastle
Lake Macquarie West	Toronto	Newcastle
Maitland	East Maitland	Cessnock
Cessnock	Cessnock Post Office	Cessnock
Port Stephens	Nelson Bay	Williamtown

6.14.2 Results of the Regional Analysis

The performance of the seven housing scenarios (Table 6.16) was simulated using the Allotment Water Balance model for each zone in the Lower Hunter region. In the model the minimum storage volume in each rainwater tank was assumed to be the average daily indoor water use for each zone and a mains water trickle top up rate of 18 L/hr/dwelling was used.

The water use for each of the seven housing scenarios in the different zones was derived from the HWC monitoring data (see Section 6.3 and Appendix G). These water use patterns were used in the analysis for the period 1931 to 1994.

The results from the simulations were combined (assuming that all domestic dwellings install rainwater tanks) using the proportion of different household types and occupation rates from Table 6.15, and population (Table 6.14) to produce average reductions in mains water use, 1-year ARI daily mains water use and stormwater discharges (Table 6.22) for each zone.

Table 6.22: Average reductions in mains water use, stormwater discharges and 1-year ARI daily mains water demands each zone in the Lower Hunter region.

Zone	Average reduction (%)		
	Mains water use	**Stormwater discharge**	**1-year ARI daily mains water use**
Inner SE Newcastle	50.3	52.1	25.6
Hamilton Mayfield	54.3	43.1	27.3
Lambton Jesmond	54.6	38.6	35.8
NW Wallsend	44.7	55.4	18.3
Lake Macquarie East	50.4	50.8	22.2
Lake Macquarie West	50.0	46.6	16.8
Maitland	40.9	52.9	17.2
Cessnock	40.3	59.0	24.2
Port Stephens	49.1	45.1	12.9

The average values for each zone were determined from each of the 7 housing scenarios for all storm events in the period 1931 to 1994. The 1-year ARI daily mains water use was the 64th largest daily water demand for each housing scenario. The results (Table 6.22) show that substantial average reductions in domestic mains water use, stormwater discharges and 1-year ARI daily mains water use can be derived from the installation of rainwater tanks in the Lower Hunter region.

6.15 Summary

An Allotment Water Balance model has been developed that will allow long-term simulation at short time steps to analyse the performance of dual water supply systems and source control measures on allotment or cluster developments. The performance of the water balance model was partially verified against monitoring data from the Maryville Experiment (Chapter 3) providing excellent agreement with the observed data.

The water balance model includes new and improved methods for the simulation of outdoor water use and the performance of first flush devices. The new outdoor water use model is dependent on human behavioural reaction to climatic conditions rather than the traditionally assumed deterministic relationship to climatic variables. The outdoor water use model showed significantly improved predictive abilities.

The first flush model continuously simulates the performance of a first flush device over a long time period. This enables the rational design of a first flush device to account for the variable nature of rainfall events. This is a significant improvement over the traditional assumption that the first flush device will separate a fixed quantity of rainfall.

The Allotment Water Balance model was used to predict the long-term performance of the Figtree Place unit clusters (Chapter 2) and the Maryville house (Chapter 3). In both cases the analysis revealed that the use of rainwater tanks in a dual water supply system provided significant reductions in mains water use, stormwater discharges and stormwater peak discharges. The use of the mains water trickle top up scheme at the Maryville house also showed that substantial reductions in peak daily and instantaneous water use are possible. The analysis also suggests that the use of rainwater tanks was likely to significantly reduce the impact of urbanisation on stormwater quality.

The installation of rainwater tanks with mains water trickle top up to all domestic dwellings in the Lower Hunter region was examined using the water balance model. Very significant reductions in average mains water use, peak daily mains water use and stormwater discharges were demonstrated for each zone in the region.

It was shown that the installation of dual water supply systems with rainwater tanks on domestic housing has the potential to provide substantial benefits to the community that are derived from reduced mains water use and stormwater discharges. In Chapter 7 the Allotment Water Balance model is used with a stormwater catchment model to determine the impact of installation of rainwater tanks on domestic allotments in an urban catchment.

In Chapter 8 a new regional demand model will be described that allows the long-term simulation of mains water demand from regions where rainwater tanks are used to supplement mains water supplies for domestic water demand. This model will provide the input to Monte Carlo simulation models of water supply headworks systems to be evaluated in Chapter 9. Specifically, the impact of rainwater tanks on deferring water supply dams will be evaluated for the Lower Hunter and Central Coast regions. Finally in Chapter 10 the whole of water cycle benefits of the installation of rainwater tanks will be evaluated.