Chapter 7

The Impact of the Use of Rainwater Tanks in Subdivisions

> "The impact of man on the water cycle is greatest per unit area in urban places. Man is capable of transforming his local environment in an almost endless variety of ways, over a matter of a few years, whereas nature moves largely on a timescale of eons. Thus, urban hydrology contends with the dimension of dynamic change because urban development everywhere has been in continuous states of expansion and flux" M. B. McPherson [1974].

7.0 Introduction

Urban development and its constructed hydraulic systems cause profound changes to the natural water cycle. The area of impervious surfaces is increased whilst natural watercourses are replaced with hydraulically efficient gutters, pipes and channels that increase the volume and velocity of stormwater runoff, and prevent infiltration of rainfall into the soil. Water demand resulting from urban development is met by importing large volumes of treated water, across large distances and at considerable cost, from neighbouring catchments. At the same time similar volumes of stormwater from roofs are discharged unused from urban developments via expensive stormwater drainage systems.

The majority of local governments in Australia stipulate strict adherence to the pipe paradigm (Chapter 5) requiring the provision of pipe drainage systems in all new developments in accordance with the methods outlined in Chapter 14 of Australian Rainfall and Runoff [IEAust, 1987]. Many authors including Whipple [1981], Anderson [1970] and Espey et al. [1969] report that the construction of stormwater drainage systems in urban developments significantly reduces the time of concentration of discharges to the downstream environment and, in combination with impervious areas, increases peak discharges by a factor that ranges from 2 to 9. The largest changes in stormwater runoff due to urbanisation occur for the frequent storm events. Whipple [1981] found that urban development increased peak discharges by a factor of 9 for a two-year ARI (average recurrence interval) storm event.

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Increases in stormwater runoff from urban developments during frequent storm events intensify the occurrence of erosion, sedimentation and flooding in downstream environments. Urban development also significantly modifies the quality of stormwater. Transportation of contaminants from urban developments to the downstream environment during frequent storm events is facilitated by the hydraulic efficiency of the pipe drainage system.

The use of rainwater tanks to supply domestic hot water, toilet and outdoor uses will significantly reduce peak and volumetric stormwater discharges from urban allotments during frequent storm events and substantially reduce mains water demand (Chapters 3 and 6). Will the introduction of rainwater tanks on domestic allotments in urban developments decrease the requirement for stormwater infrastructure and reduce the impact of urban development on downstream environments?

In this Chapter the current design philosophy for stormwater drainage and a method for the inclusion of rainwater tanks in the hydrological design of urban developments is described. The Allotment Water Balance model described in Chapter 6 is combined with the stormwater management model WUFS (Water Urban Flow Simulator) [Kuczera et al., 2000] to determine the impact of the installation of rainwater tanks on domestic allotments on the provision of stormwater infrastructure and stormwater discharges from a large urban development in the Lower Hunter region of New South Wales, Australia.

7.1 The Current Design Philosophy for Stormwater Drainage Systems

The traditional design process for a stormwater drainage system usually addresses a variety of performance criteria that are defined by design storm events with a given Average Recurrence Interval ARI [IEAust, 1987], including:

- a maintenance and water quality requirement relating to frequently occurring storm events (ARI of up to 2 years),
- a requirement to minimise nuisance or inconvenience from infrequently occurring storm events (ARI: 2 – 10 years),
- a requirement to prevent flood damage to property and risks to human life during rare storm events (100 years ARI), and

- a disaster management requirement for extreme storm events such as a probable maximum flood (ARI >10,000 years).

Local government requires the provision of street drainage systems in new urban developments to collect stormwater runoff from streets and from properties adjoining streets. Stormwater is usually discharged directly from roofs to the street drainage system via small diameter pipes and stormwater from other surfaces on allotments discharge to the street drainage system as sheet flow. The street drainage system consists of networks of gutters, pits and drainage conduits that convey stormwater rapidly to a trunk drainage system or the receiving environment.

The street drainage system is usually required to convey stormwater from streets and adjoining properties without nuisance for storm events with ARIs of 2 – 5 years (minor system) and from storm events up to the 100-year ARI (major system) without flooding of properties or other serious damage [IEAust, 1987 and Argue, 1986].

The minor/major system of drainage design has been widely adopted in Australia. The minor system consists of a gutter and pipe network that is designed to efficiently convey stormwater from minor storm events (ARI: 2 – 5 years) to the receiving environment and prevent nuisance flooding in streets. The major system consists of surface discharge routes including roads and reserves that convey stormwater from major design storm events (such as a 100 year ARI design storm event) to the receiving environment. The majority of stormwater infrastructure is provided to mitigate nuisance and, in recent times, to improve stormwater quality during minor storm events.

Local government also require the minor/major drainage system to protect citizens from hazardous situations in streets and on footpaths, and to protect buildings from flooding [NCC, 1999]. Modern stormwater drainage standards set by local government also typically require that stormwater runoff from urban developments is no greater than pre-development stormwater runoff and that stormwater quality is maintained at pre-development levels. Given that the urbanisation and pipe drainage systems significantly increase stormwater discharges and reduce water quality in urban catchments additional management solutions on allotments or at the end of pipe are required to mitigate these impacts.

7.2 Design Storms, Real Storms and Continuous Simulation of a Time Series of Storm Events

It is current practice in Australia to estimate peak discharges of a given ARI from stormwater catchments using Intensity-Frequency-Duration (IFD) rainfall data and temporal patterns provided in Australian Rainfall and Runoff [IEAust, 1987] that have been derived from storm bursts [Walsh et al., 1991; Srikanthan and Kennedy, 1991; Hill and Mein, 1996 and Rigby and Bannigan, 1996]. The IFD rainfall data and temporal patterns have been derived from rain gauge records of the most intense storm bursts at a particular location for given durations in each year recorded. The annual series of the most intense storm bursts for different durations are used to determine the annual exceedance probability (AEP) of each annual maximum storm burst. A design storm is the combination of a storm burst and a temporal pattern of a given ARI and duration.

It is important to note that a design storm is not a real storm. Design storms were created to enable the derivation of a design peak stormwater discharge with the same probability of occurrence as the actual rainfall event. This process allowed the assessment of risk of the occurrence of a flood event or the risk of failure of a stormwater management measure. However the true probability of a flood event may be obscured because the use of a discrete storm burst to estimate a peak discharge involves many unknowns that will impact on the reliability of the estimate (Figure 7.1). These unknowns include the volume of rainfall in the storm event, antecedent soil moisture conditions in the catchment and continuing losses.

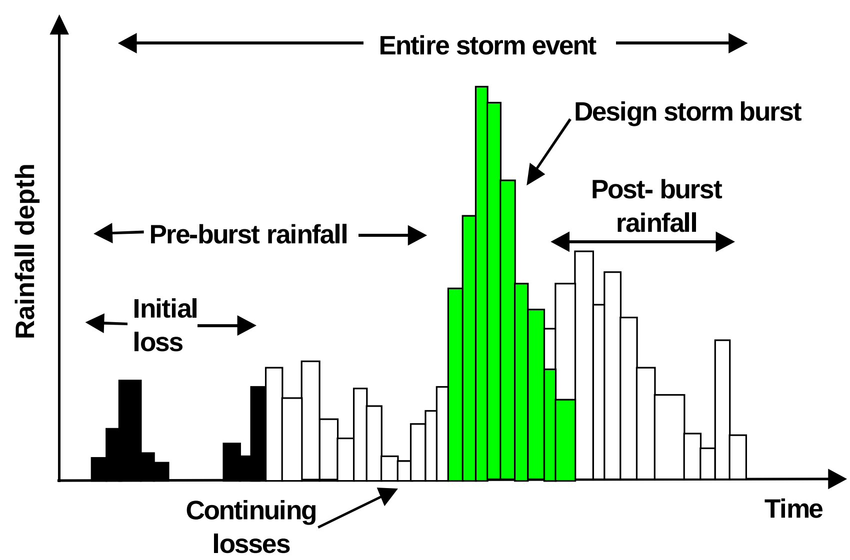

Figure 7.1: Schematic of a storm event showing the design storm burst and losses

It is shown in Figure 7.1 that the design storm approach does not account for an entire storm event. Indeed traditional rainfall/runoff routing using design storms will not account for the pre-burst and post burst rainfall, therefore potentially underestimating or overestimating the volume of a flood event. This may have a significant impact on catchments that contain a considerable amount of natural storage, detention basins or rainwater tanks. The problem is exacerbated by the discrete nature of the design storm approach that requires an estimate of the initial and continuing losses in the catchment to account for a variety of soil moisture conditions and natural storages. Many authors including Waugh [1991], Walsh et al. [1991], and Hill and Mein [1996] found that the estimation of design discharges was very sensitive to the selection of initial conditions particularly at low ARIs.

Waugh [1991] found that the selection of high runoff events for the derivation of initial loss models has created a bias toward wet antecedent conditions. Initial losses tend to be too low. In a study of five Western Australian catchments Waugh [1991] found that the underestimation of initial losses resulted in up to a 20% over-estimation of the size of flood events. Many large summer storms falling on dry catchments that yield little or no stormwater runoff have not been considered in the determination of initial losses.

In contrast Srikanthan and Kennedy [1991] explained that design storms are derived from bursts within longer storm events. Pre-burst rainfall will wet the catchment before the design storm burst. Thus initial losses determined for storms will be over-estimated for use with design storms. They found that the amount of pre-burst rainfall in a storm event decreased with increasing design storm duration for a given ARI. Hill and Mein [1996] established that the design initial losses should increase with the duration of a design storm burst and the use of inappropriate initial losses could result in significant overestimation of design floods.

The estimation of design discharges will be even more uncertain in a catchment that contains detention basins or rainwater tanks because the storage available for stormwater detention/retention in a detention basin or a rainwater tank is unknown prior to the design storm burst. The continuous simulation of stormwater runoff from a catchment using a long time series of rainfall events will significantly reduce uncertainty about the value of initial conditions (such as soil moisture and the available storage in the catchment) that is common to the design storm approach and will account for the entire volume of rainfall in storm events. This will result in more reliable estimates of the probability of flood events with a given ARI. The Allotment Water Balance model developed in Chapter 6 uses continuous simulation to reduce uncertainty about soil moisture on urban allotments and the water levels in rainwater tanks, infiltration trenches and on site detention tanks. Unfortunately the majority (if not all) of the rainfall/runoff models used to design urban street drainage systems employ the design storm approach. Therefore the design of street drainage systems that include rainwater tanks (or any storage based measure) is subject to not inconsiderable uncertainty.

7.3 A Method to Determine the Retention Storage Available in a Rainwater Tank Using Continuous Simulation

The street drainage system in urban areas is usually designed to cope with 2 to 5 year ARI design storm events and stormwater quality measures are designed to mitigate the impacts of the 0.25 to 2 year ARI design storm events. Design storm events are defined in Australian Rainfall and Runoff [IEAust, 1987]. The impact of rainwater tanks on the provision of water quality and stormwater infrastructure will depend on the retention storage volume available in the tank just prior to the design storm event. Stormwater engineers and local government commonly argue that rainwater tanks will have no impact on the design of the street drainage system because the tanks will not have any retention storage available prior to a design storm event. The results from Chapters 3 and 6 show this assumption to be incorrect although it is important to understand the magnitude of the retention storage available in rainwater tanks prior to a design storm event.

The Allotment Water Balance model presented in Chapter 6 can be used to determine the probable initial airspace storage available (PIAS) in a rainwater tank. This will eliminate uncertainty about what is the retention storage available in a rainwater tank prior to a storm event.

PIAS is determined as follows using continuous simulation: The on-site detention storage volume required to mitigate stormwater runoff from a given ARI storm event is determined for allotments with and without a rainwater tank. The difference between the on-site detention storage volumes at the same ARI storm event will be the retention storage volume available in the rainwater tank prior to the given ARI storm event. The ARI at which a particular detention storage spills ARI_{OSD} can be estimated by

$$ARI_{OSD} = \frac{Years}{Spills} \tag{7.1}$$

where Years is the modelling period in years and Spills is the number of spills from the detention storage in that period.

The volume of the retention storage in a rainwater tank prior to a storm event is a function of the detention storage required on an allotment with no rainwater tank (OSD_{NOTANK}) and the detention storage required on an allotment with a rainwater tank (OSD_{TANK}) for a given ARI equal to n years.

$$PIAS_n = (OSD_{NOTANK} - OSD_{TANK})_n \tag{7.2}$$

The retention storages available in rainwater tanks prior to storm events in the Newcastle area of the Lower Hunter region are determined in Section 7.3.1.

7.3.1 Determination of the Retention Storages Available in Rainwater Tanks in the Newcastle Area

The Allotment Water Balance model and the synthetic pluviograph record for Maryville (Chapter 6) were used to determine the probable retention storage available for stormwater management in rainwater tanks in new developments typical to the Newcastle area. A typical allotment with an area of 600 m^2, a non-roof impervious area of 200 m^2, a house with a roof area of 220 m^2 and four occupants was analysed. Water use patterns from the NW Wallsend (Tables 6.1 and 6.2) zone were used. The household used a rainwater tank that was topped up with mains water at a rate of 18 L/hr when water levels are low in the tank to supply hot water, toilet and outdoor uses. The design of the rainwater tank is shown in Figure 7.2. The OSD tank used in the analysis is described in Section 6.8. For each scenario the discharge pit in the OSD tank is assumed to have a volume of 1 m^3 and a 50 mm diameter orifice on the outlet to the street drainage system.

The design of the rainwater tank includes an air gap of 100 mm above the invert of the stormwater overflow pipe to provide backflow prevention and additional stormwater management (Figure 7.2). Rainwater supply is taken from the tank at a point 100 mm from the base of the tank to ensure that sludge or contaminants are not drawn into the supply from the anaerobic zone. The minimum level that is maintained by the mains water trickle top up is set at 100 mm above the point of supply from the rainwater tank.

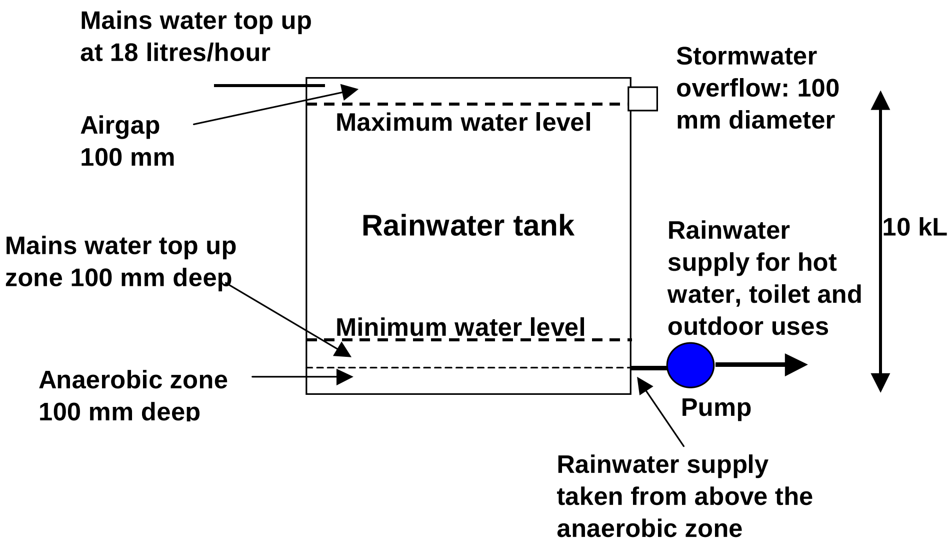

Figure 7.2: Design details of the rainwater tank

The Allotment Water Balance model was run with the 64 year synthetic pluviograph record at Maryville with and without a rainwater tank for different sized OSD storage volumes. From these runs Table 7.1 was developed. The required OSD volume with and without a rainwater tank was interpolated so that the OSD tank spilled for a given ARI. For example, when the OSD tank had a volume of 30.5 m^3 and there was no rainwater tank, the Allotment Water Balance model simulated 64 spills from the OSD tank – hence the OSD tank spills with a 1-year ARI.

Table 7.1: Determination of initial airspace storages available in a 10 kL rainwater tank

ARI of spills from the OSD tank (years)	Required OSD storage		PIAS volume (m^3)
	Tank (m^3)	No tank (m^3)	
0.25	11.75	15.25	3.5
1	26.5	30.5	4.0
2	37.5	41.0	3.5
5	46.0	49.0	3.0
10	57.0	54.0	3.0

The PIAS available in a 10 kL rainwater tank used to supply hot water, toilet and outdoor uses for various ARI storm events is shown in Table 7.1. The initial airspace volume ranges from 4 m^3 for an ARI of 1 year to 3.5 m^3 for an ARI of 2 years. The PIAS volumes of 3.5

m^3 for the 0.25 year ARI is less than the PIAS volume of 4 m^3 for the 1 year ARI is of interest. The 0.25 year ARI storm events may coincide with periods of frequent rainfall that fill the tank more often resulting in a reduced PIAS volume in the tank.

The design storms prescribed in Australian Rainfall and Runoff [IEAust, 1987] for use by the stormwater industry are storm bursts within storm events rather than an entire storm event for a given ARI. The results obtained for the PIAS volume in a rainwater tank for given ARIs using the Allotment Water Balance model will need to be calibrated to design storms of the same ARI for use in rainfall/runoff models that use the design storm approach. This will be attempted in Section 7.4.

Two different design scenarios were analysed in this section. The normal scenario (Figure 7.2) has a minimum air gap of 100 mm whereas the airspace for detention scenario (Figure 7.3) has a larger air gap and a smaller stormwater overflow pipe diameter to allow additional stormwater management.

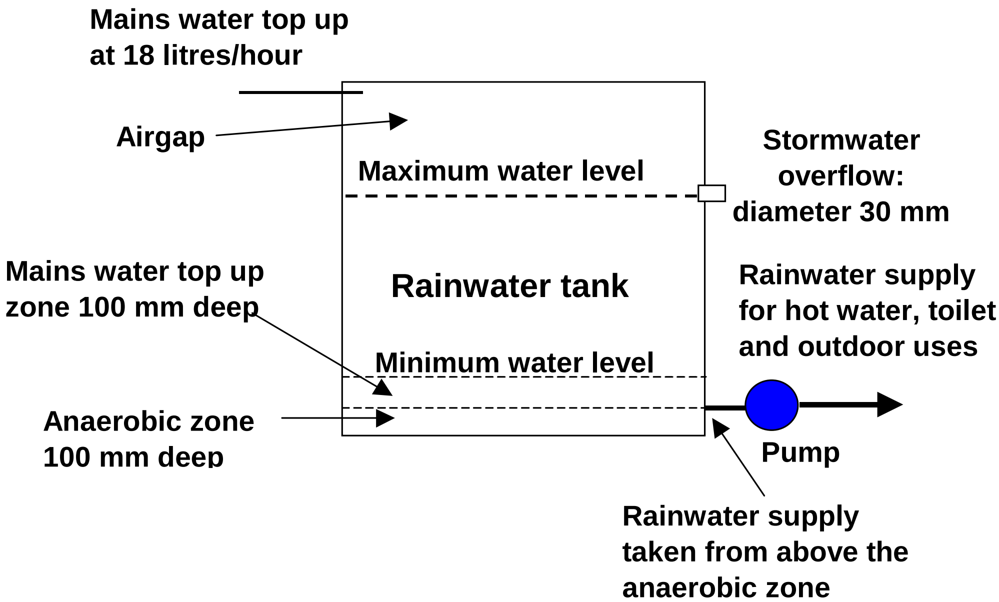

Figure 7.3: Design details for a rainwater tank with additional provision for stormwater management

Four different rainwater tank designs were analysed to determine the PIAS for different ARIs. This will enable the inclusion of the rainwater tank designs in stormwater catchment modelling to determine the impact of rainwater tanks on the provision of stormwater infrastructure. The different rainwater tank designs include a 10 kL rainwater tank (Table 7.1), a 10 kL rainwater tank with airspace for detention of 3.75 m^3, a 15 kL rainwater tank and a 15 kL rainwater tank with airspace for detention of 5.63 m^3. The results are shown in Table 7.2.

Table 7.2: Initial airspace storages in different rainwater tank designs

ARI (years)	PIAS prior to a storm event (m^3)			
	10 kL tank	**10 kL tank with 3.75 m^3 detention storage**	**15 kL tank**	**15 kL tank with 5.63 m^3 detention storage**
0.25	3.5	5.25	4.65	6.25
1	4	5.8	4.8	7
2	3.5	5	4.3	7
5	3	3.75	3.2	5.63
10	3	3.75	3.2	5.63

In Table 7.2 it is shown that the provision of airspace for stormwater detention and an overflow pipe with a small diameter (30 mm) results in larger PIAS volumes being available prior to a storm event. The PIAS volume is smaller than the airspace for detention plus the initial airspace storage in the similar tank without airspace for detention. For example the 10 kL tank without airspace for detention has a PIAS of 4 m^3 and the 10 kL tank with a 3.75 m^3 airspace for detention has a PIAS of 5.8 m^3 for the 1-year ARI storm event. The PIAS of 5.8 m^3 for the tank with airspace for detention is smaller than the volume of 7.7 m^3 that consists of the airspace for detention plus the PIAS from the tank without detention (4 m^3). This highlights an important phenomenon. Stormwater only discharges from rainwater tanks without airspace for detention after the tank fills. Thus the volume of stormwater retained in the tank without airspace for detention during the storm event is the PIAS volume. In the case of the tanks with airspace for detention the PIAS volume represents the airspace volume in the tank prior to the storm event less the volume of stormwater that discharges via the overflow pipe during a storm event prior to the tank filling to its maximum possible volume.

Some caution is required in the use of the PIAS for a rainwater tank with airspace for detention in a rainfall/runoff model. If the rainfall/runoff model includes rainwater tanks as a simple storage the PIAS volume will be an adequate initial condition because it will account for the additional discharge from the tank during the storm event. However if the rainfall/runoff model includes rainwater tanks as storage with the capacity to route stormwater through the overflow pipe during a storm event the use of the PIAS volume will underestimate the true initial storage volume in the tank.

The reduction in mains water use was 43% (113 kL/year) for the 10 kL rainwater tank, 38% (100 kL/year) for the 10 kL rainwater tank with 3.75 m^3 detention storage, 47% (124 kL/year) for the 15 kL rainwater tank and 42% (111 kL/year) for the 15 kL rainwater tank with 5.63 m^3 detention storage. The provision of rainwater tanks provides very significant mains water savings although the use of detention storage in the rainwater tank will result in small reductions in the magnitude of mains water savings. The results from Table 7.2 are used in Section 7.4 to calibrate the initial retention storages in rainwater tanks for given ARIs for use in a rainfall/runoff model that uses the design storm approach. This will allow the determination of the impact of rainwater tanks on the provision of stormwater infrastructure in Section 7.5.

7.4 Calibration of Initial Airspace Storage Volumes in Rainwater Tanks to Design Storms

The majority of rainfall/runoff models use the design storm approach to simulate the performance of stormwater infrastructure in catchments. As shown in Figure 7.1 the use of design storms involves the estimation of a number of unknown conditions including the antecedent soil moisture condition and the storage available in the catchment prior to the storm event. Design storms were established to produce stormwater peak discharges with an ARI that is the same as the ARI of the storm event.

The PIAS was developed using continuous simulation of real storm events over a long period in Section 7.3 but the value of the PIAS to be used with design storms is uncertain. This Section calibrates the PIAS obtained to design storms so that the PIAS values can be used in a rainfall/runoff model that employs the design storm approach.

The objective of the calibration is to adjust the PIAs to ensure that the rainfall/runoff model using a design storm of a given ARI produces a peak stormwater discharge from the allotment equal to the peak discharge derived by continuous simulation (see Table 7.3) for the same ARI. The WUFS (Water Urban Flow Simulator) [Kuczera et al., 2000] rainfall/runoff model was used in the calibration. Peak discharges from the allotment for a given ARI determined using design storms in the rainfall/runoff model were calibrated to the peak discharges from allotment without a tank by altering the antecedent moisture condition (AMC) and the pervious surface depression storage (PDS) in the rainfall/runoff model.

Table 7.3: Peak discharges determined from continuous simulation of the performance of the allotment

ARI (years)	Peak discharge from the allotment (m^3/s)				
	No tank	10 kL tank	10 kL tank with 3.75 m^3 detention	15 kL tank	15 kL tank with 5.63 m^3 detention
0.25	0.001	0.000	0.000	0.000	0.000
1	0.009	0.006	0.005	0.005	0.005
2	0.012	0.010	0.010	0.010	0.008
5	0.016	0.014	0.012	0.014	0.011
10	0.025	0.022	0.021	0.022	0.019

The values for antecedent conditions such as soil moisture (AMC) and pervious depression storages (PDS) are unknown in the design storm approach but in the continuous simulation approach values for the antecedent conditions are implicitly included in the peak discharges for a given ARI. Thus calibration of peak discharges derived from design storms to peak discharges derived from continuous simulation should reduce the uncertainty about initial conditions in the design storm approach.

Following the calibration of the peak discharges from the rainfall/runoff model to the peak discharges from continuous simulation using the AMC and the PDS, a rainwater tank was included in the rainfall/runoff model. A clay soil type was assumed for the analysis. Note that the WUFS rainfall/runoff model allows routing of discharges from a rainwater tank during a storm event. The initial storage in the rainwater tank was altered until the peak discharge from the rainfall/runoff model equalled the peak discharge from the continuous simulation for an allotment with the same rainwater tank for a given ARI. The initial airspace volumes for each rainwater tank design using the rainfall/runoff model are shown in Table 7.4.

Table 7.4: Initial airspace storages for different rainwater tank designs to be used with design storms

ARI (years)	AMC	PDS (mm)	Initial airspace volume (m^3)			
			10 kL tank	10 kL tank with 3.75 m^3 detention	15 kL tank	15 kL tank with 5.63 m^3 detention
0.25	1	20	3.35	3.85	3.4	5.7
1	1	20	4.0	6.9	4.1	8.44
2	1	20	5.0	3.75	4.7	5.63
5	1	20	6.2	3.75	6.1	5.63
10	3	5	6.2	3.75	6.1	5.63

The values for antecedent conditions (AMC and PDS) used to calibrate the peak discharges from the allotment without rainwater tanks are shown in Table 7.4 to vary with ARIs. The results indicate that the 0.25, 1, 2, and 5 year ARI storm events occurred when the catchment was relatively dry whilst the 10 year ARI storm event occurred when the catchment was wet. For example an AMC of 1 with a PDS of 20 mm indicates that the soil is dry and there is 20 mm of depression storage available on the pervious surfaces whilst an AMC of 3 and a PDS of 5 mm indicates that the soil is relatively wet and only 5 mm of depression storage remains on pervious surfaces. Although the values for AMC and PDS indicate a dry catchment for low ARIs, these values may also indicate that design storm approach has not properly accounted for pre-burst rainfall (see Figure 7.1) or the design storm has a substantially different temporal pattern to real storms with low ARIs or the design storms are overestimating the rainfall volumes at low ARIs.

In Table 7.4 it is shown that the values for the initial airspace volumes in the 10 kL and 15 kL rainwater tanks without airspace for detention estimated using design storms was consistently larger than the values for initial airspace volumes estimated using continuous simulation for all ARIs except 0.25 years. This result seems to indicate that the design storms have larger volumes than the real storm events of the same ARI from the continuous simulation. A larger volume of rainfall in a storm event falling on a roof in an allotment with a rainwater tank will result in a larger volume of roof runoff entering the tank. A larger initial airspace volume will be required in the tank to reduce peak discharges by a given amount from the allotment.

In Table 7.4 it is also shown that the values for the initial airspace volumes in the 10 kL and 15 kL rainwater tanks with airspace for detention estimated using design storms was consistently larger than the values for initial airspace volumes estimated using continuous simulation for the 1-year ARI storm event. This result was expected. The WUFS rainfall/runoff model allowed routing of stormwater from a rainwater tank during a storm event. Thus the actual initial storage volume available in the tank prior to the design storm was determined whereas the PIAS calculated using continuous simulation was the actual initial storage volume less the volume of stormwater discharge from the tank during the storm event. However the values for the initial airspace volumes in the 10 kL and 15 kL rainwater tanks with airspace for detention estimated using design storms were similar or smaller than the values for initial airspace volumes estimated using continuous simulation for the 0.25, 2, 5 and 10 year ARI storm events.

The WUFS rainfall/runoff model that uses the design storm approach is used to determine the impact of the use of rainwater tanks on the provision of stormwater infrastructure in Section 7.5. The values for initial airspace storages determined by calibration to design storms are used in the analysis to maintain consistency with the traditional design storm approach.

In this section it was shown that antecedent conditions can vary with the ARI of a storm event and the volume and temporal pattern of a design storm may not be similar to the volume and temporal pattern of a real storm with the same ARI. Given that the majority of urban catchments contain a number of natural and man-made storages including detention basins, constructed wet lands, on-site detention tanks and, to some extent, the pipe drainage system this result casts some doubt on the reliability of current practice that uses design storms to determine the provision of stormwater infrastructure. The current practice of assigning the same antecedent conditions to design storm events with different ARIs in the simulation of the performance of urban stormwater drainage also seems to have uncertain reliability given the variance of antecedent conditions found in this Section. It is believed that the use of continuous simulation in preference to the design storm approach will produce more reliable analysis of the performance of urban stormwater catchments.

7.5 Analysis of an Urban Development that Includes Rainwater Tanks

In this Section the performance of the traditional drainage system with and without rainwater tanks is compared to determine the impact of the use of household rainwater tanks on the provision of stormwater infrastructure and the receiving environment. The WUFS [Kuczera et al., 2000] rainfall/runoff model is used to compare the different stormwater management scenarios. The retention storage volumes for the different rainwater tank scenarios (Table 7.4) determined using the Allotment Water Balance modelling and calibrated to design storms are used in the WUFS model to establish the impact of the rainwater tanks.

7.5.1 The Subdivision

An existing subdivision (the Fletcher development) in the NW Wallsend zone with 384 allotments and a total area of 28.79 ha is used in the analysis (Figure 7.4).

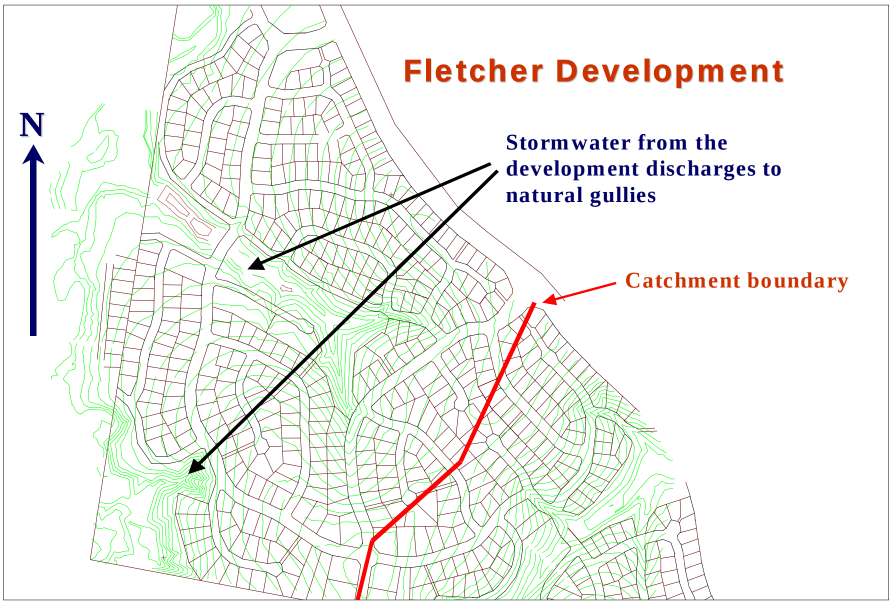

Figure 7.4: The Fletcher development used in the analysis of a stormwater catchment

The Fletcher development includes an average allotment size of 600 m^2, 7 metre wide road pavements in 16 metre wide road reserves and the site has ground slopes up to 12%. Stormwater from all roads and allotments on the northern side of the catchment boundary drains to two natural gullies (Figure 7.4). Stormwater discharges to the natural gullies via five outlet points. The pipe drainage system is designed to cope with two-year ARI design storm events. A sediment trap and a gross pollutant trap are provided at each discharge point to the natural gullies.

A pipe drainage system was designed for each scenario that will convey stormwater from two-year ARI design storm events without surcharging from drainage pits and in the 100-year ARI design storm convey stormwater with a depth of less than 150 mm at the kerb, with a velocity-depth product that is less than 0.4 m^2/s in streets and with flow depths that do not exceed the height of the crown of the road pavement. No attempt was made to limit stormwater discharges to predevelopment rates for this development. Two soil types were analysed: a clay soil and a sand soil that was moderately wet prior to the design storm events. The values for initial airspace volumes in rainwater tanks were assumed to be independent of soil type because the rainwater tanks only collect rainwater from roofs.

Therefore the values for the initial airspace volume in rainwater tanks from Table 7.4 were used in both the clay and sandy soil types scenarios.

Stormwater discharges from the allotments were directed to the street drainage system. In situations where the slope of the land does not allow discharge of stormwater from allotments to a street stormwater is discharged to an inter-allotment drainage system. The inter-allotment drainage system consists of table drains and small diameter pipes. A minimum pipe diameter of 300 mm is used for drainage in roads and 150 mm for drainage in inter-allotment drains.

The results for the retention storages available in rainwater tanks prior to storm events (Table 7.4) were used as initial conditions in WUFS for scenarios that included rainwater tanks as part of the stormwater drainage system. In the rainwater tank scenarios a rainwater tank with a design in accordance with Figure 7.2 or 7.3 were placed on every allotment in the WUFS model and roofwater was routed through the tank.

7.5.2 The WUFS Rainfall/Runoff Model

The WUFS rainfall/runoff program is used to analyse the performance of the Fletcher development with and without rainwater tanks. WUFS simulates stormwater discharges from urban catchments using traditional conveyance systems and water sensitive urban design elements such as rainwater tanks, grass swales and infiltration trenches.

WUFS uses the time-area method to simulate stormwater runoff from impervious and pervious areas that discharge to the drainage network. The drainage network is described by a series of nodes that are interconnected by links. The nodes represent subcatchments, detention/retention basins, junctions, drainage pits and outfalls, and the links represent underground drainage pipes and surface discharges via overland flow. Subcatchments contain one or more identical subareas that usually include impervious and pervious areas associated with an allotment. Each subcatchment discharges to an outlet (sag or on-grade drainage pit) allowing the conveyance of stormwater via pipes or overland flow. The program allows the simulation of multiple design storms from Australian Rainfall and Runoff [IEAust, 1987].

7.5.3 Results from the Different Stormwater Management Scenarios

The performance of the different stormwater management scenarios was evaluated using WUFS for a catchment using a clay soil and a sandy soil. Figure 7.5 presents the flood frequency curve for peak discharges at the subdivision outlet for a clay soil. A traditional scenario without rainwater tanks was compared to scenarios with the different rainwater tank designs shown in Table 7.4. The results are shown in Appendix H.

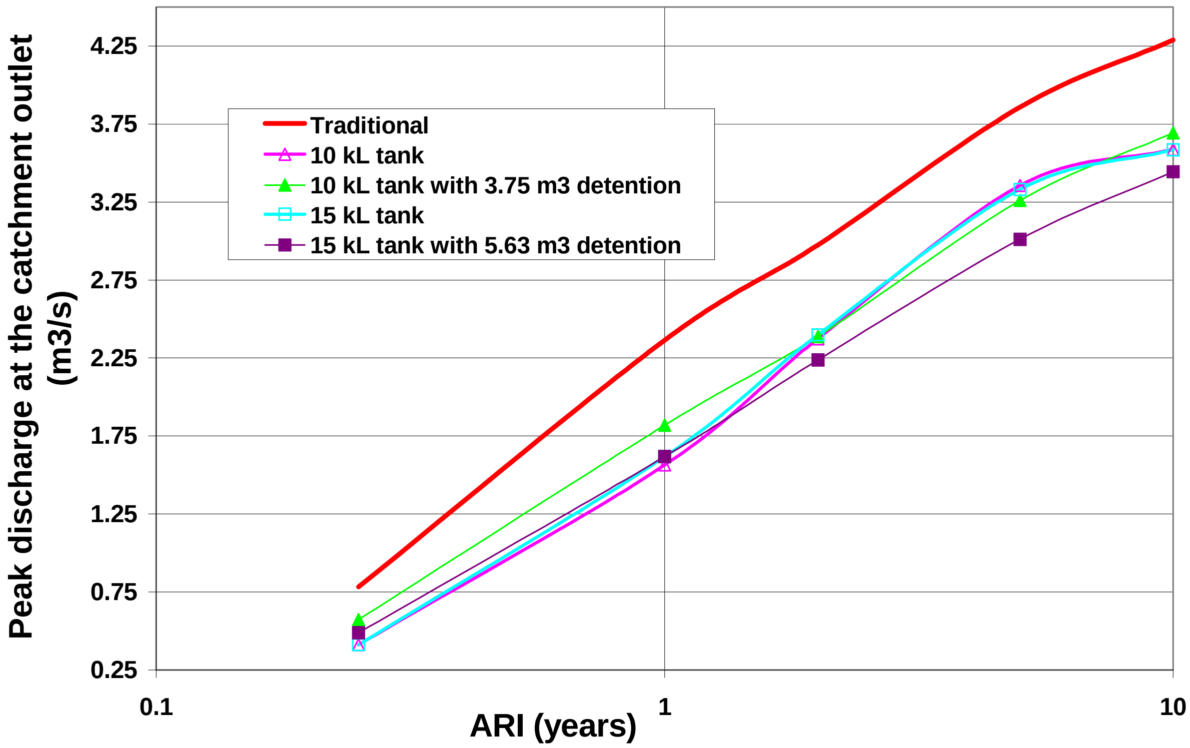

Figure 7.5: The performance of the different stormwater management scenarios in the Fletcher development with a clay soil.

It is shown in Figure 7.5 that the use of rainwater tanks to supply domestic hot water, toilet and outdoor uses in a subdivision will significantly reduce peak stormwater discharges. The scenarios with normal rainwater tanks (overflow at the top of the tank) (10 kL and 15 kL tank) show greater reductions in peak discharges than the scenarios with rainwater tanks with airspace for detention for the 0.25 and 1 year ARI. The rainwater tanks (10 kL and 15 kL tank) retain the majority of roofwater whilst the rainwater tanks with airspace for detention discharge some of the roofwater during the 0.25 and 1 year ARI events. After the 1 year ARI event the rainwater tanks with airspace for detention are shown to reduce peak discharges by a greater amount than the rainwater tanks without airspace for detention because the rainwater tanks without airspace for detention are overflowing whilst the rainwater tanks with airspace for detention are overflowing at a reduced flowrate due to the smaller outlet diameter. The large reduction in peak discharges for the scenarios with rainwater tank without airspace detention at the 10 year ARI could be caused by the differences between real storm and design storm events in the estimation of the initial airspace storages. The results are shown in Tables 7.5 and 7.6.

Table 7.5: Peak discharges from the different stormwater management scenarios at the catchment outlet for a clay soil

ARI (years)	Peak discharges at the catchment outlet (m^3/s)				
	Traditional	**10 kL tanks**	**10 kL tank with 3.75 m^3 detention**	**15 kL tanks**	**15 kL tank with 5.63 m^3 detention**
0.25	0.782	0.412	0.573	0.412	0.488
1	2.365	1.564	1.819	1.618	1.618
2	2.975	2.374	2.388	2.398	2.237
5	3.859	3.356	3.26	3.332	3.011
10	4.289	3.588	3.693	3.585	3.445

Table 7.6: Reductions in peak discharges from the different stormwater management scenarios at the catchment outlet for a clay soil

ARI (years)	Reduction in peak discharges at the catchment outlet (%)				
	Traditional	**10 kL tanks**	**10 kL tank with 3.75 m^3 detention**	**15 kL tanks**	**15 kL tank with 5.63 m^3 detention**
0.25	0	47	27	47	38
1	0	34	23	32	32
2	0	20	20	19	25
5	0	13	16	14	22
10	0	16	14	16	20

The use of rainwater tanks on all allotments in the Fletcher subdivision with a clay soil type will substantially reduce the impact of the development on the environment as indicated by the substantial reduction in peak discharges for the 0.25 and 1 year ARIs (Tables 7.5 and 7.6). This will reduce the requirement for water quality infrastructure. The use of rainwater tanks will also substantially reduce the requirement for street drainage infrastructure as indicated by the reductions in peak discharges for the 2 year ARI storm event.

The scenarios that included rainwater tanks provided significant reductions in stormwater discharges. However it is apparent that the inclusion of airspaces for the detention in the tanks can further reduce the impact of the use of rainwater tanks on peak stormwater discharges. The scenarios with airspace for detention increased the reduction in stormwater discharges that can be achieved by using rainwater tanks for the 5-year and 10-year ARI storm events. It is important to note that the volume of airspaces for detention in the rainwater tanks was arbitrarily assigned. Therefore even greater reductions in stormwater discharges may be possible with optimisation of rainwater tank dimensions. Figure 7.6 presents the flood frequency curve for peak discharges at the subdivision outlet for a sandy soil.

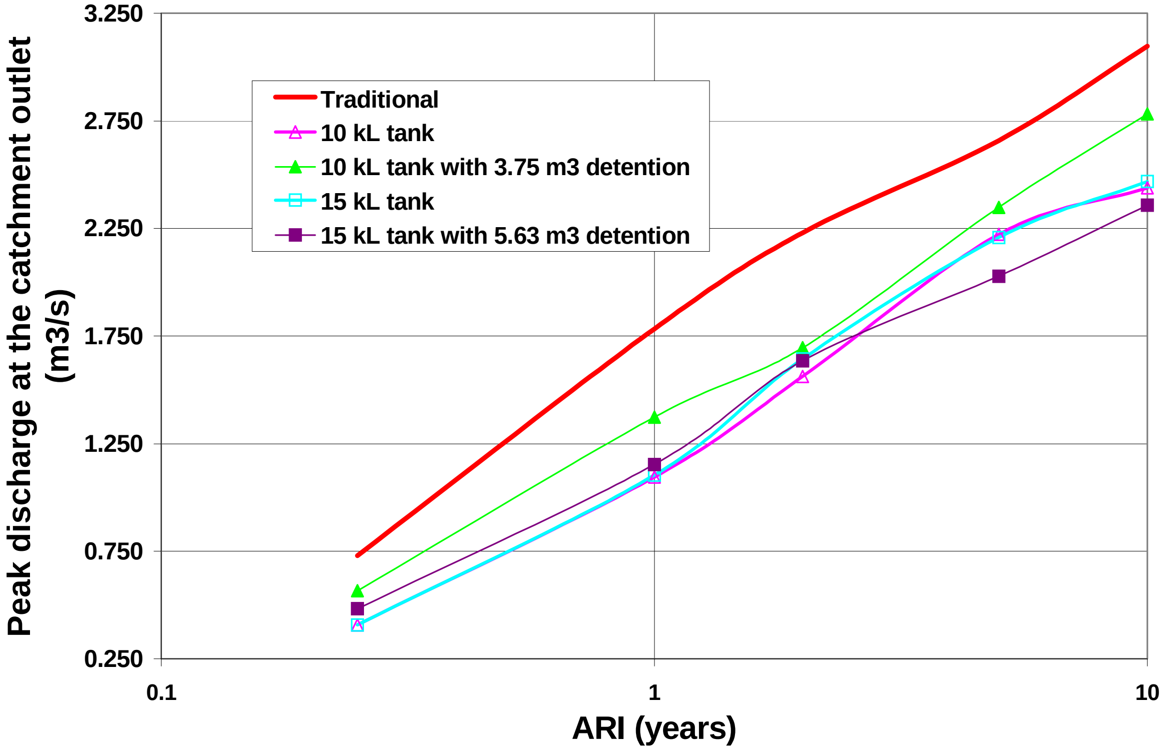

Figure 7.6: The performance of the different stormwater management scenarios in the Fletcher development with a sandy soil.

In Figure 7.6 it is shown that on sandy soils the rainwater tanks without airspace for detention reduce peak discharges by a greater amount than the 10 kL rainwater tank with airspace for detention for the 0.25 to 10 year ARIs. This result differs from the results from the clay soil because there is reduced stormwater runoff from pervious surfaces due to the greater infiltration capacity of the sandy soil. Roof runoff is a greater portion of total stormwater runoff. Therefore the rainwater tanks without airspace for detention are retaining a greater proportion of total stormwater runoff whilst the discharges and overflows from the 10 kL rainwater tank with airspace for detention form a greater portion of the total stormwater runoff due to the reduced volume available for capture of roofwater in the tank. The results are shown in Tables 7.7 and 7.8.

The results in Tables 7.7 and 7.8 when compared with those in Tables 7.5 and 7.6 show that the rainwater tanks without airspace for detention and the 15 kL rainwater tank with airspace for detention generally reduce stormwater peak discharges by a marginally greater amount for a catchment with a sandy soil than for a catchment with a clay soil. The 10 kL rainwater tank with airspace for detention reduced stormwater peak discharges by a smaller amount in the sandy catchment than in the clay catchment. The results (for example the 10 kL rainwater tank with airspace for detention) show that the design of rainwater tanks for optimum reductions in stormwater peak discharges in catchments will require modification to suit catchment conditions.

Table 7.7: Peak discharges from the different stormwater management scenarios at the catchment outlet for a sandy soil

ARI (years)	Peak discharges at the catchment outlet (m^3/s)				
	Traditional	**10 kL tanks**	**10 kL tank with 3.75 m^3 detention**	**15 kL tanks**	**15 kL tank with 5.63 m^3 detention**
0.25	0.73	0.408	0.567	0.408	0.483
1	1.783	1.095	1.373	1.104	1.154
2	2.230	1.563	1.696	1.644	1.636
5	2.658	2.223	2.348	2.208	2.029
10	3.097	2.539	2.782	2.568	2.358

Table 7.8: Reductions in peak discharges from the different stormwater management scenarios at the catchment outlet for a sandy soil

ARI (years)	Reduction in peak discharges at the catchment outlet (%)				
	Traditional	**10 kL tanks**	**10 kL tank with 3.75 m^3 detention**	**15 kL tanks**	**15 kL tank with 5.63 m^3 detention**
0.25	0	44	22	44	34
1	0	39	23	38	35
2	0	30	24	26	27
5	0	16	12	17	24
10	0	18	10	17	24

The use of rainwater tanks in the different scenarios also produced substantial mains water savings for the entire development (Table 7.9). The use of rainwater tanks in the Fletcher development will reduce the growth in mains water demand by 38.4 megalitres (ML) to 47.6 ML per year. The mains water savings may significantly reduce the impact of the subdivision on water treatment plants and water storage dams.

Table 7.9: Mains water savings resulting from the use of rainwater tanks in the Fletcher development

Scenario	Mains water savings (kL/year)	
	Individual household	Entire subdivision
10 kL tanks	113	43,392
10 kL tanks with 3.75 m^3 detention	100	38,400
15 kL tanks	124	47,616
15 kL tanks with 5.63 m^3 detention	111	42,624

7.5.4 Comparative Costs of the Stormwater Drainage Systems

The reductions in peak stormwater discharges from the subdivision for the various rainwater tank scenarios decreased the requirement for street drainage and water quality infrastructure. It is believed that reduced stormwater peak discharges will reduce the size of water quality devices required to capture contaminants. The unit costs to install stormwater drainage infrastructure used in the analysis were provided by consultants in the stormwater management industry [G. O'Loughlin, Robinson GRC, personal communication, 2001; and B. Stanilands, Stanwill Consulting, personal communication, 2000]. The unit costs are shown in Table 7.10.

Table 7.10: Unit costs to install stormwater pipes

Pipe internal diameter (mm)	Installation Cost ($/m)
150	33
225	45
300	65
375	85
450	90
525	120
600	160
675	180
750	210

The cost to install an on-grade stormwater pit was $1,400 and the cost to install a sag pit was $1,040. The cost to install water quality measures including sediment and gross pollutant traps was assumed to be $60,000 per outlet if the one-year ARI discharge from the catchment was greater than 2 m^3/s, $40,000 per outlet if the discharge is between 2 m^3/s and 1.5 m^3/s, and $30,000 per outlet if the discharge is less than 1.5 m^3/s. The cost to install the 10 kL and 15 kL tanks were determined from the Maryville construction costs (Table 3.4) and are shown in Table 7.11. The construction costs for the rainwater tank systems has increased from when the rainwater tank was installed at Maryville due to the introduction of the goods and services tax (GST) and increases in prices of components due a rising inflation rate.

Table 7.11: Costs to install the rainwater tanks

Item	**Cost to install ($)**	
	10 kL tank	**15 kL tank**
Tank	840	1020
Pump + pressure controller	200 + 146	200 + 146
Plumber and fittings	500	500
Float system	100	100
Concrete base	200	200
GST	199	214
Total	**2,185**	**2,380**

The comparative costs for the stormwater infrastructure from each scenario for the catchment with a clay soil are shown in Table 7.12. Only the costs of infrastructure items (such as pipes, pits and water quality devices) that have varied in each scenario are considered.

Table 7.12: Stormwater infrastructure costs (excluding tanks) for the catchment with clay soil

Scenario	**Costs to install ($)**			**Total ($)**	**Cost saving ($)**	**Saving ($/lot)**
	Pipes	**Pits**	**Water quality devices**			
Traditional	502,515	362,100	300,000	1,164,615	0	0
10 kL tanks	419,375	355,200	200,000	974,575	190,040	495
10 kL tanks with detention	421,065	355,200	200,000	976,265	188,350	490
15 kL tanks	423,625	355,200	200,000	978,825	185,790	484
15 kL tanks with detention	413,255	355,200	200,000	968,455	196,160	511

It is shown in Table 7.12 that the introduction of rainwater tanks on allotments can produce significant stormwater infrastructure cost savings ranging from $484 to $511 per allotment in the catchment with clay soils. However these cost savings will not fully cover the cost of the installation of the rainwater tanks. An additional $1,674 - $1,701 per allotment is required for the scenarios with 10 kL rainwater tanks and $1,896 - $1,869 per allotment for the scenarios with 15 kL tanks.

The use of rainwater tanks in the subdivision has also substantially reduced mains water demand. This will have the potential to defer the construction of new dams and water treatment plants thereby reducing the headworks charges for each allotment. The reduced headworks charges and the stormwater infrastructure savings may pay for the installation of the rainwater tanks in new subdivisions. The impact of rainwater tanks on water supply headworks infrastructure will be evaluated in Chapter 9. The bulk purchase of rainwater tanks and pumps may also significantly reduce the cost of installing rainwater tanks in the subdivision.

The use of rainwater tanks in the subdivision will also have significant long-term benefits to the community that are derived from a reduced requirement for infrastructure and decreased impacts on the downstream environment. The local government will have decreased depreciation and maintenance costs that result from the provision of a reduced amount of stormwater infrastructure.

The useful life of stormwater infrastructure is defined as 70 years [Department of Local Government, 1995]. The annual depreciation costs for the traditional scenario are shown in Table 7.13. The annual maintenance costs of stormwater infrastructure are considered similar to the annual replacement costs.

The maintenance costs of water quality infrastructure (gross pollutant and sediment traps) are about 5% of the capital cost of the device per year [M. Powell, CDS Technology, personal communication, 2001]. The maintenance and replacement costs for the water quality infrastructure are shown in Table 7.13. Water quality infrastructure is assumed to depreciate at the same rate as stormwater infrastructure.

It is shown in Table 7.13 that the introduction of rainwater tanks on allotments can reduce annual replacement and maintenance costs by $23.90 to $23.13 per allotment in the catchment with clay soils.

Table 7.13: Stormwater infrastructure replacement and maintenance costs (excluding tanks) for the catchment with clay soil

Scenario	Annual replacement and maintenance costs ($)		Total annual cost ($)	Annual cost saving ($)	Annual saving ($/lot)
	Stormwater infrastructure	Water quality devices			
Traditional	24,704	19,286	43,990	0	0
10 kL tanks	22,130	12,857	34,987	9,003	23.45
10 kL tanks with detention	22,188	12,857	35,045	8,945	`23.29
15 kL tanks	22,252	12,857	35,109	8,881	23.13
15 kL tanks with detention	21,956	12,857	34,813	9,177	23.90

The comparative costs for the stormwater infrastructure from each scenario for the catchment with a sandy soil are shown in Table 7.14.

Table 7.14: Stormwater infrastructure costs (excluding tanks) for the catchment with sandy soil

Scenario	Costs to install ($)			Total ($)	Cost saving ($)	Saving ($/lot)
	Pipes	Pits	Water quality devices			
Traditional	409,015	313,960	200,000	922,975	0	0
10 kL tanks	375,245	313,960	150,000	839,205	83,770	218
10 kL tanks with detention	375,295	313,960	150,000	839,255	83,720	218
15 kL tanks	378,275	313,960	150,000	842,235	80,740	210
15 kL tanks with detention	373,995	313,960	150,000	837,955	85,020	221

It is shown in Table 7.14 that the introduction of rainwater tanks on allotments can produce stormwater infrastructure cost savings ranging from $210 to $221 per allotment in the catchment with sandy soils. Rainwater tanks installed in the catchment with sandy soils produces savings in the requirement for stormwater infrastructure that are approximately half the savings experienced in the catchment with clay soils because the requirement for stormwater infrastructure is significantly reduced in a sandy catchment. Similar to the results in the clay catchment these cost savings will not fully cover the cost of the installation of the rainwater tanks. An additional $1,964 - $1,975 per allotment is required for the scenarios with 10 kL rainwater tanks and $2,170 - $2,159 per allotment for the scenarios with 15 kL tanks. However there are significant opportunities to combine rainwater tanks with infiltration measures on urban allotments in sandy catchments that could eliminate the requirement for kerb and guttering on streets within the catchment. This could produce a further $768,000 saving (cost of kerb and gutter estimated to be $125 per metre and the cost of infiltration devices estimated to be $500 per allotment). The replacement and maintenance cost savings for the sandy catchment are presented in Table 7.15.

It is shown in Table 7.15 that the introduction of rainwater tanks on allotments can reduce annual replacement and maintenance costs by $10.65 to $10.98 per allotment in the catchment with sandy soils. The replacement and maintenance cost savings are approximately half the savings experienced in the catchment with clay soils because significantly less stormwater infrastructure is required in the sandy catchment.

Table 7.15: Stormwater infrastructure replacement and maintenance costs (excluding tanks) for the catchment with sandy soil

Scenario	**Annual replacement and maintenance costs ($)**		**Total annual cost ($)**	**Annual cost saving ($)**	**Annual saving ($/lot)**
	Stormwater infrastructure	**Water quality devices**			
Traditional	20,656	12,857	33,513	0	0
10 kL tanks	19,692	9,643	29,335	4,178	10.88
10 kL tanks with detention	19,694	9,643	29,337	4,176	10.87
15 kL tanks	19,779	9,643	29,422	4,091	10.65
15 kL tanks with detention	19,656	9,643	29,299	4,214	10.98

The significant decreases in peak stormwater discharges that result from the installation of rainwater tanks will also reduce erosion, sedimentation and flooding in the downstream environment. These savings have not been assessed. However, there is a less obvious and very significant saving which is the subject of Chapters 8 and 9. The whole of urban water cycle economics resulting from the widespread installation of rainwater tanks will be evaluated in Chapter 10.

7.6 Summary

In this Chapter using Allotment Water Balance model developed in Chapter 6 a method to determine the probable initial airspace storage (PIAS) available in a rainwater tank prior to a storm event with a given ARI was described. The PIAS was developed using continuous simulation of real storm events over a long period in Section 7.3 but the value of the PIAS to be used with design storms was uncertain. The PIAS obtained to design storms was calibrated so that the PIAS values can be used in a rainfall/runoff model that employs the design storm approach. The objective of the calibration is to adjust the PIAs to ensure that the rainfall/runoff model using a design storm of a given ARI produces a peak stormwater discharge from the allotment equal to the peak discharge derived by continuous simulation for the same ARI. The WUFS (Water Urban Flow Simulator) [Kuczera et al., 2000] rainfall/runoff model was used in the calibration.

Considerable differences between the volumes and temporal patterns of real storms and design storms of the same ARI were apparent. These differences appear to exacerbated by the variation in antecedent conditions for storm events with different ARIs. The reliability of the design storm approach for the analysis of urban subdivisions is therefore subject to question. This analysis only represents an initial foray. There is very considerable scope for further work.

The performance of a subdivision that includes rainwater tanks on domestic allotments for hot water, toilet and outdoor uses was evaluated using knowledge of the retention storages available in each tank prior to design storm events in the WUFS stormwater management model for a catchment with a clay soil type and for a catchment with a sandy soil type.

Stormwater peak discharges, mains water demand and the requirement for stormwater infrastructure in the subdivision were substantially reduced. This resulted in cost savings that ranged from $484 to $511 per allotment for clay soils and $210 to $221 per allotment for sandy soils in the provision of stormwater infrastructure. The cost savings in the provision of stormwater infrastructure resulting from the installation of rainwater tanks in catchments with sandy soils were half the savings experienced in catchments with clay soils because there is a reduced requirement for stormwater infrastructure in sandy catchments. These savings will vary with climate, soil types and economic conditions in different regions. Nevertheless the savings in the provision of stormwater infrastructure will partially pay for the installation of rainwater tanks. The introduction of rainwater tanks will reduce mains water consumption. An immediate and obvious saving is the reduced payment for mains water usage. However, there is a less obvious and more significant saving which arises when there is a community wide uptake of rainwater tanks. This will be the subject of Chapters 8 and 9.

The installation of rainwater tanks in subdivisions will also provide significant lifecycle benefits to the community that are derived from reductions in depreciation costs for infrastructure, maintenance costs and impacts on the environment. These benefits arising from the use of rainwater tanks will be analysed as part of a whole of water cycle economic evaluation in Chapter 10.