

Implications to stormwater management as a result of lot scale rainwater tank systems: a case study in Western Sydney, Australia

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ABSTRACT

Rainwater tanks are increasingly adopted in Australia to reduce potable water demand and are perceived to reduce the volume of stormwater discharge from developments. This paper investigates the water balance of rainwater tanks, in particular the possible impacts these tanks could have in controlling the stormwater discharge volume. The study collected water quantity data from two sites in the Hawkesbury City Council area, New South Wales, Australia and utilised the collected data in a simple water balance model to assess the effectiveness of rainwater tanks in reducing the stormwater discharge volume. The results indicate that a significant reduction in discharge volume from a lot scale development can be achieved if the rainwater tank is connected to multiple end-uses, but is minimal when using irrigation alone. In addition, the commonly used volumetric runoff coefficient of 0.9 was found to over-estimate the runoff from the roof areas and to thereby underestimate the available volume within the rainwater tanks for retention or detention. Also, sole reliance on the water in the rainwater tanks can make the users aware of their water use pattern and water availability, resulting in significant reductions in water use as the supply dwindles, through self-imposed water restrictions.

Key words | rainwater tanks, runoff coefficients, stormwater, water balance, Western Sydney

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INTRODUCTION

Best Management Practice (BMP) techniques in urban stormwater management such as Water Sensitive Urban Design (WSUD), Low Impact Development (LID) or Sustainable Urban Development (SUDS) have recently been adopted by many countries. The BMP design promotes the use of rainwater tanks for non-potable water supply. Many studies have investigated the benefits of having rainwater tanks in urban development projects (e.g. Höllander *et al.* 1993; Herrmann & Schmida 1999; Lye, 2002; Spinks *et al.* 2003; Evans *et al.* 2006; van Olmen 2009; DeBusk *et al.* 2010; Ward *et al.* 2010). Most of these indicated that this water is suitable for non-potable uses; however, only a few studies have focused on the overall impact of rainwater tanks on urban stormwater management. This is especially important, as stormwater infrastructure is aging and is often not upgraded until problems occur or when there is a danger to the public (O'Loughlin *et al.* 1995). This aging infrastructure is also incapable of handling increased

demands from infill or knockdown-and-replace developments, which often increases the urban impervious area significantly. Australia has a separated stormwater system and therefore the stormwater system is well suited to WSUD and source control. Since the 1980s, on-site detention (OSD) has been used in Australia to constrain post development design flows; however, with the introduction of WSUD in the 1990s (Argue 2004) and mandatory water restrictions by many water authorities, rainwater tanks have become popular in urban development projects. The impact of rainwater tanks on the reduction of stormwater discharge rate and volume is important, because this can potentially reduce the size (orifice diameter and volume) and cost of an OSD.

Several research studies in Australia have focused on the replacement of OSD with rainwater tanks (e.g. Argue 1997; Coombes *et al.* 2001). Coombes *et al.* (2001) focused on the Upper Parramatta River Catchment (UPRCT) and

compared the outflow of OSD, rainwater tanks, and a combination of these two systems. The tank was assumed to be connected to the laundry, toilets and outdoor areas and the OSD system was designed according to the UPRCT handbook (UPRCT 2005). The results showed that the air-space in a rainwater tank can be used as a credit for OSD. This credit varies for each development type and ranges from 32 to 65% (Coombes *et al.* 2001).

In addition, Argue & Scott (2000) indicated that with a large-scale hypothetical model, the OSD and rainwater systems produce a similar hydrograph. They agreed that the peak discharge on a lot scale is larger for a rainwater tank than for OSD, but the cumulative effect of volume reduction obliterates the effect of high peak discharges delivered by individual sites in medium to large catchments. Rainwater tanks can also provide a significant monetary saving to both citizens and government authorities, as it reduces potable water use and defers the need for upgrades of the water supply network (Barry & Coombes 2006; Lucas *et al.* 2009). The design of these systems is also highly dependent on the spatial rainfall pattern, water demand and top-up rate and therefore requires specialist knowledge (Barry & Coombes 2006).

For the Western Sydney region, often hypothetical data is used to model the stormwater system due to limited availability of local experimental data. This paper focuses on a water balance study using data from two continuously monitored rainwater tanks located in the Western Sydney region. The aim of the water balance study is to compute the overflow discharge from the rainwater tanks and to provide some indication on the retention volume available in rainwater tanks on a lot scale.

METHODOLOGY

Site selection

The study was conducted within Hawkesbury City Council (HCC) located 65 km west of Sydney, Australia (see Figure 1). The state of New South Wales (NSW), in which HCC is located, is expected to have a population growth of 48% by 2051 (Australian Bureau of Statistics 2007), therefore new developments are expected to be built in this fringe area. These developments would increase the strain on the existing stormwater system. Currently, HCC requires an OSD system to mitigate the impact of new or infill developments, which are designed to control up to the 1 in 100-year average recurrence interval (ARI) flow to pre-development

levels (HCC 2000). HCC has a temperate climate: the mean maximum temperature is 23.9 °C, the mean minimum temperature is 10.5 °C, and the average annual rainfall is 796 mm (Bureau of Meteorology (BOM) 2011). Summer (October to February) rainfall is higher than the winter (March to September) rainfall; and results in local flooding of the stormwater system during frequent low to medium intensity summer rainfall (i.e. <5 years ARI).

The first Site (Site 1) is a 4,500 L Zinalume[®] rainwater tank in a semi-rural area. The tank is situated on the western wall of a cottage with a 144 m² Zinalume[®] roof. The tank water is used as potable water for a three person household and used for the shower, toilet and kitchen purposes, including drinking water. The roof runoff is captured by a small gutter with a single downpipe and discharges to the top of the tank through a screen. The overflow (50 mm diameter) discharges freely into a collection container, which is used for drinking water for the poultry and for irrigation. The second site (Site 2) is a 13,000 L polyester rainwater tank on the verge of the urban area. The tank is situated on the north wall of a dwelling with a 170 m² tiled roof and is used for irrigation purpose only. The overflow (90 mm diameter) drains into the garden area.

Data collection

The sampling of these tanks was conducted from 1 October 2008 until 6 October 2009 and both sites were equipped with continuous data collection system: flowmeters (ABB MagMaster), tipping bucket rain gauges (RIMCO 8020) and a datalogger (Logosense2) installed to log the flow and rainfall (5 s intervals, downloaded weekly using a data management software program (HYDRAS 3)). The water levels in the tanks and overflow volume were measured weekly, due to resource constraints. The water level was measured using a floating water level gauge, whilst the first 2,500 mL of overflow was collected and the volume recorded (0 mL, <2,500 mL or >2,500 mL). Rainfall and flow data were analysed using a freeware program (HEC-DSSVue) and an in-house developed Matlab program.

Water balance

A weekly (time t) water balance was prepared in Excel, using the standard water balance equation (see Equation (1)), followed by a daily (time t) water balance.

$$\Delta S = I - O \quad (1)$$

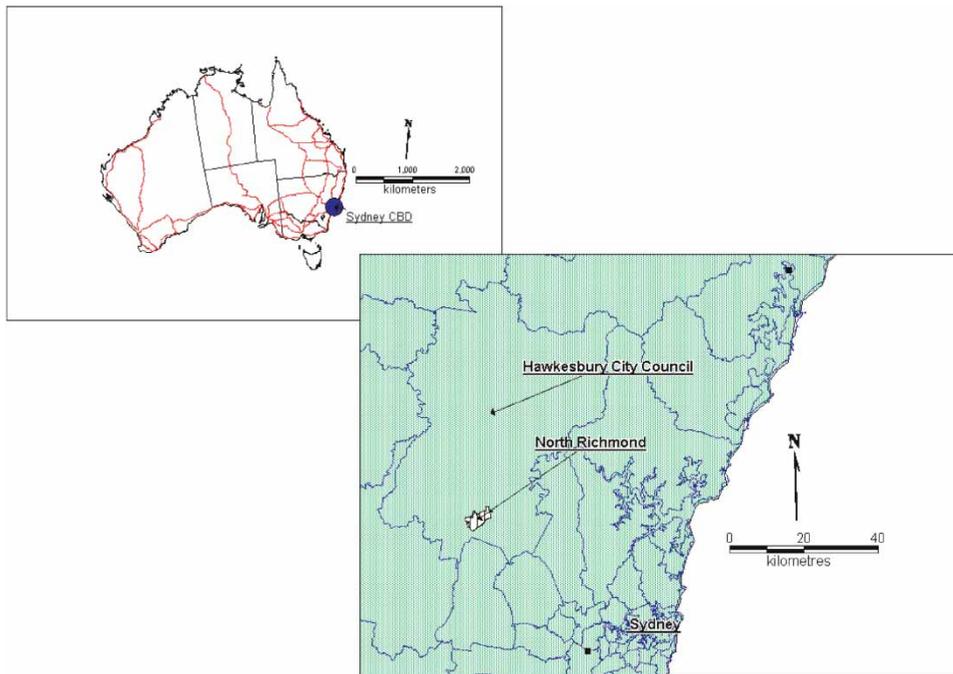


Figure 1 | Location of study area and testing location.

where ΔS is the change in storage during time t (m^3), I is the inflow from the roof during time t (m^3) and O is the outflow from the tank through the overflow (m^3) and usage (m^3) during time t . The water level was converted to weekly storage volume using Equation (2).

$$S_t = H_t \times \pi r^2 \quad (2)$$

where S_t is the storage volume at time t (m^3) H_t is the water depth at time t (m) and r is the radius of the tank (m).

The inflow was derived from the flowmeter data. An error occurred in the recorded flow for Site 2 (ants made their nest in the flowmeter), therefore no flow data was recorded for Site 2. A volumetric runoff coefficient (C) was estimated for Site 2 using rainfall events without a tank overflow and throughout period of non-rainwater use (e.g. the users were on holidays), giving a runoff coefficient of 0.75. For Site 2, Equation (3) was used to estimate the inflow.

$$I_t = i_t / 1000 \times A_{\text{roof}} \times C \quad (3)$$

where I_t is the calculated inflow at time t (m^3), i_t is the recorded rainfall (mm) at time t , A_{roof} is the area (m^2) of the roof discharging into the tank and C the computed runoff coefficient.

The outflow was only partially known and therefore Equation (4) was used.

$$O_t = O_{\text{use}} \quad O_{\text{over}} \leq 0 \text{ m}^3$$

$$O_t = O_{\text{use}} + O_{\text{over}} \quad 0 < O_{\text{over}} \leq 2.5 \times 10^{-3} \text{ m}^3$$

$$O_t = O_{\mu \text{use}} + O_{\text{over}} \quad O_{\text{over}} > 2.5 \times 10^{-3} \text{ m}^3 \quad (4)$$

where O_t is the outflow at time t (m^3), O_{use} is the usage volume at time t (m^3), $O_{\mu \text{use}}$ is the mean usage at time t (m^3) and O_{over} is the overflow volume (m^3) at time t . O_{use} was estimated in between storm events and where O_{over} was known (e.g. $< 2.5 \times 10^{-3} \text{ m}^3$). Where O_{over} was unknown $O_{\mu \text{use}}$ was used to estimate O_t . After estimation of the weekly usage rates (O_{use} and $O_{\mu \text{use}}$), a daily water (time t) balance model was established, which assumed that the daily usage rate was constant over the week.

RESULTS

The measured total rainfall values for Site 1 and Site 2 were 918.6 mm and 609 mm, respectively during the study period. The nearby BOM (2011) gauge recorded 766 mm for the

same period. The flow rates and time-to-peak depend on the roof area and gutter configuration and was less than 5 min for both roof configurations.

The volumetric runoff coefficients (C) for Site 1 were analysed on a storm event basis using Equation (5) (Pilgrim 1987, reprinted 1998).

$$C = \frac{Q}{iA} \quad (5)$$

where Q was event runoff volume (m^3), i is the total depth of rainfall (m) and A the area of the roof (m^2). The volumetric runoff coefficients varied with maximum burst duration (Table 1). The continuous rainfall data was analysed for maximum burst and durations using the autocorrelation methods described in Adams & Papa (2000). The ARI was estimated by comparing the event rainfall with the local design rainfall data obtained from ARR Volume 2 (1987). No storm specific volumetric runoff coefficient was obtained for Site 2, as no reliable inflow records were obtained.

Site observations also included behavioural changes in the use of rainwater tanks. Site 1 relies on a small tank for all indoor water needs and no significant rainfall was recorded in this site from 24 June 2009 to 28 August 2009. The users were aware of the reduction in tank water level and adjusted their water use accordingly. This resulted in Site 1 not running out of water for their basic needs. Site 2, on the other hand, utilised their 13,000 L tank only for irrigation, but due to extensive use in dry periods, required mains water to top-up the tank.

Water balance

The data was used to prepare a water budget for both sites. The weighted average runoff coefficient of 0.22 was used for Site 1 as determined by the storm analysis and a runoff coefficient of 0.75 was used for Site 2. Figure 2 shows an example of the measured, computed weekly and daily water levels for Site 1 and similar computations and goodness of fit was achieved for Site 2.

The maximum water volume of 4,500 L is never reached at Site 1, as the overflow point is located at the 4,100 L. The water-use patterns for Site 1 showed a significant reduction of water use, which is evident in the fluctuations in rate of change when the tank was close to empty; the consumption of water was reduced from 30 to 10 L day⁻¹. The water use for Site 2 was for irrigation only and hence no water was used during rainfall. The water use was computed to be approximately 150 L day⁻¹. The users ran out of water in January 2009 and filled the tank with mains water.

The volume available in the tank on a daily basis was analysed (see Figure 3(a) and 3(b)). The draw-down on Tank 1 caused a greater volume to be available for storage more frequently than for Tank 2. Tank 2 was used for irrigation only and the frequency of the volume available for storm storage in the tank was significantly less, although the total volume was significantly higher as Tank 2 is a larger tank. It could be assumed in rainwater tank design that a typical irrigation tank has no detention volume available, but a reuse tank has definitely some detention volume, no matter what size the tank is.

Table 1 | Volumetric runoff coefficient and number of storm events for both sites

ARI (1 in x months)	Average total depth of rainfall (mm)	5 min average maximum burst intensity (mm hr ⁻¹)	Site		Occurrence of storm event (no.)	
			1	2	1	2
<0.20	0.36	2.4	0.07	–	65	67
0.30	3.3	4.8	0.27	–	26	18
0.35	4.1	7.2	0.65	–	6	8
0.40	7.3	9.6	0.66	–	4	7
0.50	6.6	12.0	0.71	–	2	5
0.62	7.1	14.4	0.56	–	1	2
0.75	8.5	16.8	0.92	–	3	1
1	12.6	19.2	0.90	–	1	2
12–24	35.2	110.4	–	–	–	1
Weighted average			0.22	–		
Total					108	111

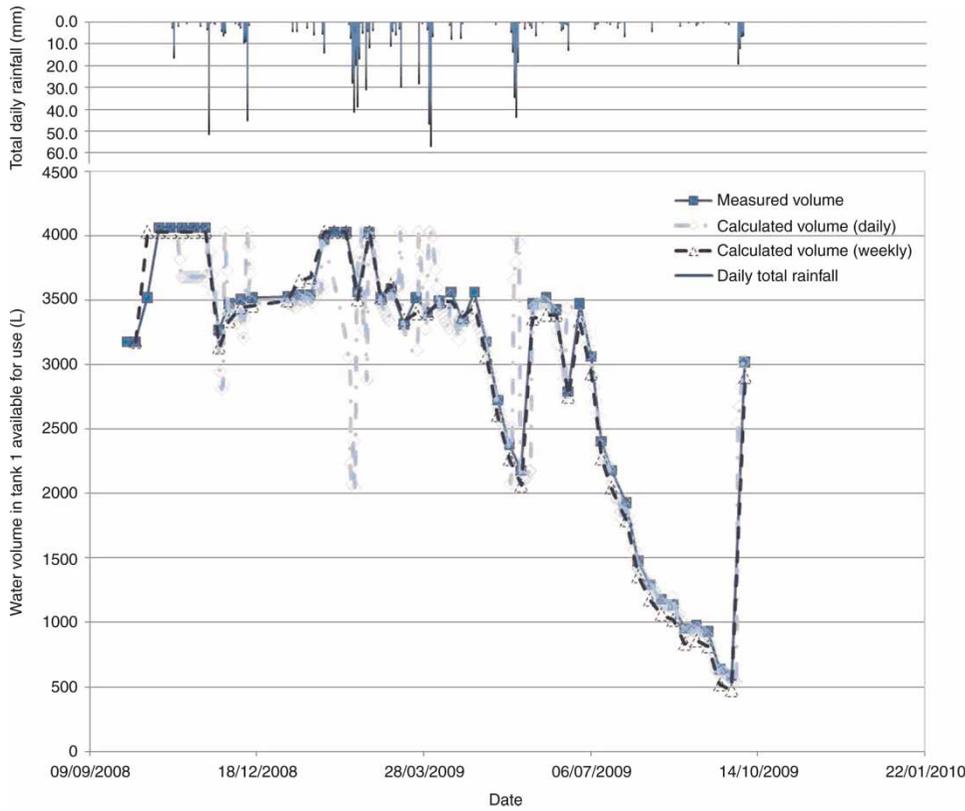


Figure 2 | Example of measured, daily and weekly computed volumes in the rainwater tank on Site 1.

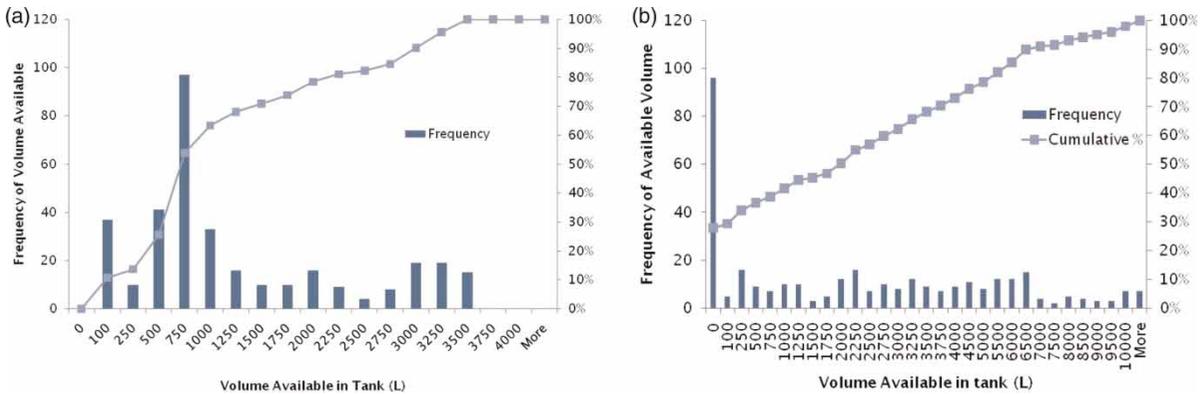


Figure 3 | Available volume in: (a) Tank 1; and (b) Tank 2.

The water volumes computed in the water balance study were used to determine the daily reduction of volume running from the roof. The daily reduction was computed using Equation (6).

$$I_t > 0, R_{ed} = \frac{O_t - I_t}{I_t} \times 100\% \tag{6}$$

where R_{ed} is the percentage reduction in runoff, O_t is the

volume of overflow at time t (m^3) and I_t is the inflow into the tank at time t (m^3). The reduction in volume discharging to the stormwater system is highly dependent on tank usage, as is evident at Site 1. This site has lower flows due to higher demands and shows, on average, a 97.5% reduction in the volume of outflow in comparison to the direct discharge from the roof, with a less than 90% reduction for 5.6% of the rainfall events. The reduction to runoff volume discharge to the stormwater system

from Site 2 is, on average, 91%; however, 25% of the daily discharge through the overflow shows no reduction at Site 2 throughout the year compared to 1.4% at Site 1. This indicates that those tanks used for multiple end-uses, such as laundering, irrigation and toilet flushing in urban areas, potentially have a greater and more frequent reduction in overflow discharges than those tanks used for irrigation only. This situation would potentially affect the stormwater management for small frequent events only (<5 year ARI) and not affect the larger flood events (up to 100 year ARI). Small frequent events are often the constraining design storms for stormwater systems and are often the governing design parameter for the minimum orifice size in OSD systems.

DISCUSSION

This analysis indicated that the volumetric runoff coefficients were 0.07–0.90 (weighted average = 0.22), which differs from other typical estimates, 0.8 and 0.95 for roof catchments (Gottschalk & Weingartner 1998; Herrmann & Schmida 1999; Brodie 2008; Ghisi *et al.* 2009; Khastagir & Jayasuriya 2010). Volumetric runoff coefficient for different roof materials have also been determined by others, e.g. a flat roof has a volumetric runoff coefficient of 0.6, a steep roof with tiles could be as high as 0.75 (van Olmen 2009), which is very similar to the volumetric runoff coefficients from this study. Site 2 has a steeper roof with tiles than the flat corrugated roof on Site 1. Other research has shown that the runoff coefficient is dependent on initial loss and number of rain days (Chiew & McMahon 1999). The research, presented in this paper, shows that the volumetric runoff coefficient is related to the ARI of the storm event and the maximum burst intensity (see Table 1), as has been highlighted by other researchers (e.g. Hotchkiss & Provaznik, 1995; Dhakal *et al.* 2010). The use of a >0.80 runoff coefficient for rainwater tank design can overestimate the volume discharging into the tank and in-turn underestimate the amount of storage available for detention or retention purposes. The assumption of an appropriate runoff coefficient is critical in the design of rainwater tanks, as their function relies heavily on the high frequency rainfall events. By overestimating the runoff coefficient of the roof and the volume entering the tank during rainfall events, the actual constructed rainwater tanks can have a significantly lower volume of water available for reuse.

The water usage volume was computed to be, on average, 30 L day⁻¹ for Site 1 and 150 L day⁻¹ for Site 2. Using the

demand equation from Duncan & Mitchell (2008), Site 1 was expected to use between 123 and 242 L day⁻¹. The water-use pattern for Site 1 was significantly reduced when the tank was almost empty. The users are very aware of their limited water supply and have put in place management techniques to minimise water wastage. The techniques were shorter showers, minimal toilet flushing and minimising dishwashing. This awareness is an important contributor to the water savings habits and methods implemented. Randolph & Troy (2008) indicated that only 20% of mains water users knew their quarterly water use, which is significantly different to those with a sole reliance on rainwater tanks. Site 2 was also aware of the amount of water that was needed to irrigate their garden and managed their water supply. In December 2008 and January 2009, minimal rainfall was recorded in the study area. The users on Site 2 filled their water tank with mains water with a conventional garden hose, but only after careful consideration of the predicted potential rainfall, which is readily available on the Bureau of Meteorology website.

CONCLUSION

This study investigated two rainwater tanks for one year in the HCC area in Western Sydney, Australia. The rainwater tanks were monitored for flow, rainfall and use. It was found that the volumetric runoff coefficient can be significantly lower than the commonly used 0.8 or 0.9. The volume available in the tank was dependent on the end-uses implemented on the site and, as expected, the tank used for multiple end-uses has a higher frequency of volume available for retention in the tank, thereby reducing the volume of overflow into the existing system. It was further noted that water use behaviour was altered as a result of the declining supply of water through the use of a rainwater tank. The users at Site 1 reduced their consumption to 10 L day⁻¹ when the rainwater tank was nearly empty, whilst Site 2 filled their empty tank with mains water. The sole reliance on the water in the rainwater tanks can thus make the users aware of their water use and real-time water availability, which can result in significant reductions in water use as the supply dwindles through progressive self-imposed water restrictions, which does not necessarily happen if the rainwater tank is connected to mains water.

The result of the difference in water use and topping up the rainwater tank is evident in the volume reduction presented in this paper. Although Site 2 has a greater tank

volume, the volume available for retention is more frequent at Site 1, but potentially smaller due to the size of the tank. It is evident that multiple end-uses should be implemented to ensure a continuing draw-down on the rainwater tank, thereby creating a larger volume available for retention and reducing the volume of discharge to the receiving systems. Multiple end-uses are supported by the NSW Government and implemented through the BASIX initiative for new developments. Further research is being conducted to determine the reduction in peak runoff as a result of these rainwater tanks on the lot scale for the more frequent storm events (<5 year ARI).

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