Developing a methodology for appraising rainwater harvesting with integrated source control using a case study from south-west England

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ABSTRACT
A methodology has been developed to design and evaluate rainwater harvesting (RWH) systems incorporating "retention and throttle" passive control. Two drainage design options were developed and assessed using hydraulic models for a case study site in south-west England. Through an iterative design approach, the inclusion of retention and throttle RWH tanks at each house allowed the site's conventionally designed stormwater attenuation tank to be reduced in volume by 44 % from 18 m^3 to 10 m^3 when designed based on the 100 year return period rainfall event. Each RWH tank had a volume of 4 m^3 and a 20 mm diameter outlet orifice located 1.5 m^3 from the top of each tank. The study concluded that the retention and throttle RWH concept has good potential for implementation in the UK. Savings generated by the reduced attenuation requirements could be reallocated towards the cost of RWH installations. Cost estimates for the case study site illustrate that an overall cost saving can be achieved where RWH with throttle and retention tanks were included.

KEYWORDS
Attenuation; rainwater harvesting; source control; SuDS; water efficiency

INTRODUCTION
Rainwater harvesting (RWH) at the residential property scale remains an underexploited technology in England and Wales, as the benefits of RWH in water demand management and source control applications have yet to be fully realized. At present, RWH systems are considered for new developments in order to meet water efficiency drivers. This one-dimensional assessment frequently sees RWH rejected on financial grounds when a whole life cost assessment is undertaken (Roebuck *et al*., 2011). It is considered that uptake of residential systems could be further promoted through technological innovation, such as proposed in the retention and throttle concept (Herrmann & Schmida, 1999), which integrates water efficiency and stormwater management objectives in a single RWH installation. The implementation of RWH in England and Wales is currently driven by water efficiency considerations (BSI, 2009), although water demand management measures such as dual flush toilets, low flow taps and waterless urinals are often used in preference to RWH (Grant, 2006). However, practitioners have suggested that further investigation of the stormwater source control benefits of RWH is warranted (Hurley *et al.*, 2008; Gerolin *et al.*, 2010; Kellagher, 2011). Internationally, DeBusk & Hunt's (2014) comprehensive RWH literature review concludes that further research is required into RWH's benefits as a stormwater management tool. The work conducted in this paper seeks to implement a modelling approach to assess the stormwater management potential of a novel RWH configuration at a case study in England.

Source Control in the England and Wales
Stormwater management in England and Wales is strongly regulated by planning controls (DCLG, 2006) and associated guidance (Kellagher, 2012). New developments incorporate

sustainable drainage systems (SuDS) to manage stormwater runoff and attenuate flows to match those of the undeveloped site. Where RWH systems are installed, they are designed solely to meet the non-potable water demand, and do not usually provide significant source control benefits.

Woods-Ballard *et al.*, (2007) defined the SuDS hierarchy in an effort to minimize stormwater runoff and pollution; 1) Prevention, 2) Source Control, 3) Site Control, 4) Regional Control. Solutions such as; green roofs; infiltration chambers; water butts; and rainwater harvesting can contribute to a source control strategy. Practitioners designing SuDS are encouraged to maximize source control opportunities before considering site wide or regional control strategies such as attenuation tanks. Despite this, there remains a prevalence of end-of-pipe solutions which are frequently deemed to offer the easiest way of complying with the legislation (Bastien, 2009). RWH can reduce stormwater runoff volumes and rates (Leggett, 2001; Debusk & Hunt, 2014; Campisano *et al.*, 2013) although the magnitude of such benefits cannot be generalized as a wide number of site-specific parameters must be evaluated. These include antecedent rainfall, yield, non-potable water demand, and the RWH configuration. Each of these facets must be considered when appraising a RWH system's ability to function as a source control technology.

With the SuDS hierarchy in mind it is desirable to maximize the benefits of a given source control technique (in this case RWH), to minimize additional downstream storage volumes within a site-wide drainage design to achieve a best practice SuDS. To regulate drainage design, hydraulic calculations must be submitted to the Environment Agency (EA) who, in their regulatory role, ensure compliance of new drainage systems (DCLG, 2012). Statutory instruments associated with The Flood and Water Management Act 2010 will see this regulatory duty pass from the EA to Lead Local Flood Authorities (LLFA) during 2014 when the SuDS Approval Bodies are formed. Approaches to integrate RWH within SuDS should therefore be developed to take into account these existing regulatory and design frameworks.

Existing approaches to achieve source control using RWH in England Wales

The British Standard for RWH, *BS8515:2009; Rainwater harvesting systems – Code of practice*, (BSI, 2009) focusses on provision of an alternative water resource to meet water demand management drivers. The implementation of RWH as a stormwater source control technique is covered by suggesting designers specify (intentionally) oversized tanks to increase the likelihood that storage is available at the beginning of a storm event. For this scenario to be applicable, the water demand (D) must be greater than runoff yield (Y). Even where D>Y, it cannot be guaranteed that the designed storage will be available as a number of other factors affect the demand. The major limitation of this approach is that it relies upon user behaviour to be consistent with the core assumptions.

Kellagher & Maneiro Franco (2007) used hydraulic models to assess the overall reduction in stormwater runoff volumes and peak flow rates from a development where a large RWH tank was proposed. The study concluded that tanks should be 1.5 to 2.5 times larger than standard RWH tanks to achieve "considerable (*sic)* stormwater benefits". In relation to extreme rainfall events, the study showed a notable reduction in runoff volumes in the 100 year return period rainfall event (23% - 55%). Another study by Memon *et al.* (2009) modelled a development of 200 properties and also concluded that RWH can reduce peak flows in downstream sewers. Gerolin *et al.* (2010) set out a methodology based on demand for water from a RWH system freeing storage capacity for the next storm. The work has since been extended by Kellagher (2011) whereby a number of RWH systems in a residential housing development were monitored. Kellagher (2011) again concluded that RWH systems can positively manage stormwater successfully when the non-potable demand exceeds the yield. Herrmann & Schmida (1999) set out a wide range of RWH typologies and identified a RWH system which achieved stormwater source control through addition of a throttled outlet which provides a retention volume in the upper region of a RWH tank as illustrated in Figure 1.

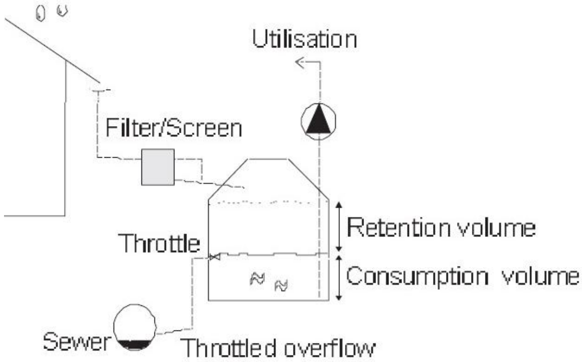

Figure 1 – RWH with integrated source control (after Herrmann & Schmida, 1999).

A comprehensive review of the existing RWH market in the UK confirms that none of the main RWH suppliers offer a product that complies with the retention and throttle specification. As a consequence, to date, no examples of this technology have been identified in the England and Wales, although trials have been identified in the USA. A modelling assessment has been carried out by Huang *et al.,* (2009) in which a RWH system was designed with a 5 m^3 tank capacity and a 50 mm diameter outlet throttle. The study considered a series of scenarios assuming such a system was installed at a development of 242 houses in Kuala Lumpur. This study illustrated that the integration of the retention and throttle approach successfully limited peak rainfall discharges for the 30 minute duration rainfall event by 22% for the 100 year return period rainfall event (Huang *et al.*, 2009). Although not considered in this study, optimization of the retention volume and its discharge rate would potentially allow an improved reduction in peak flow rates to be achieved for the downstream drainage network.

METHODS

Table 1 - Site characteristics, parameters and global design criteria.

Parameter	Input Data
Location:	Exeter, UK
Total site area:	1230m^2
Existing site runoff rate (1 in 100 year event):	7.2 l/s
Maximum future site discharge rate (during 1 in 100 year rainfall event):	7.2 l/s
Allowance for climate change:	Rainfall intensity increased by 30%
Proposed roof areas:	334m^2
Proposed parking and roadway areas:	340m^2
Total impermeable area:	674m^2
Design rainfall event:	1 in 100 year, critical duration event
Design criteria:	No above ground flood during Design Event
Runoff coefficient (all impermeable surfaces):	0.84
Runoff coefficient (all permeable surfaces):	0
Range of rainfall events tested:	15 minutes (155mm/hour) to 168 hours (1.26mm/hour)
Rainfall model:	Flood Estimation Handbook

Design Process

Two drainage design options have been developed for a small residential development in south-west England to evaluate the viability of using retention and throttle RWH tanks. Each design option assumes that all houses include a RWH system for non-potable reuse in the property's WC's. The non-potable reuse volume of these tanks is assumed to be full at the start of all drainage simulations. The global variables selected for the study were fixed and applied for each option based on figures set out in Table 1.

RWH design for non-potable reuse:

RWH tank volumes were calculated using the 'intermediate method' set out in BS8515:2009 (BSI, 2009) for each of the seven houses. This defines the tank volume required for each RWH system as the lesser of two volumes (YR or DN) calculated using Equations 1 and 2 illustrated below;

$$YR = A \times e \times h \times \mathrm{f} \times 0.05 \qquad (1)$$

YR is the annual rainwater yield (l);
A is the collecting area (m^2);
e is the yield coefficient (%);
h is the annual depth of rainfall (mm);
f is the hydraulic filter efficiency.

$$DN = Pd \times n \times 365 \times 0.05 \qquad (2)$$

DN is the annual non-potable water demand (l);
Pd is the daily requirement per person (l);
and n is the number of persons.

Daily non-potable water demand was estimated assuming 5 flushes/person/day and average flush of 4.5 l (MTP, 2010; Waterwise, 2014). Occupancy was taken from the site's design drawings as either 4 or 5 people per house. No allowance for the use of irrigation or laundry water was made. For this stage, the roof areas ranging from 42-50m^2 were used to establish the "non-potable reuse volume" (V_{NP}) required for the RWH system at each house.

OPTION 1 – Site-wide attenuation tank: An outline drainage design was developed to route all stormwater from roofs and paved surfaces into a single online attenuation tank. The design was modelled as a geocellular storage tank located beneath the parking area with a proprietary vortex flow regulator controlling discharges. An iterative approach was implemented to reduce the volume of the required storage tank from an initial estimate of 30 m^3 until a minimum size tank was identified that met the design criteria.

OPTION 2 – Decentralized retention and throttle RWH: An outline drainage design was developed for Option 2 to test the concept of utilizing individual RWH tanks at each house which have been oversized to include an additional storage volume that can drain down via an orifice following each storm event. The aim is to provide a RWH tank that can passively attenuate stormwater runoff from all roof areas and thus provide 100% of the SuDS attenuation required.

With the V_{NP} established, the source control volume (V_{SC}) was identified and added to the V_{NP} to obtain the total RWH tank volume (V_{RWH}). Limitations in the simulation model meant that a single calculation for a roof of 50 m^2 was used to represent a typical house. Firstly, a range of head-discharge relationships were calculated for orifice outlets from 0 mm - 50 mm, at 5 mm graduations. These relationships were calculated using the standard orifice equation with a fixed coefficient of discharge of 0.6 (Butler & Davies, 2011). With head-discharge curves defined, a series of tank sizes was tested in a proprietary hydrological/hydraulic sewer design package, Micro Drainage (Windes, 2013) starting with no outlet orifice. This identified a maximum storage volume of 3.7 m^3 was required to attenuate the critical rainfall event. All modelling for Option 2 was therefore carried out for a house with a roof area of 50 m^2 discharging roof-runoff to a tank with a footprint of 4 m^2 and a depth of 1 m.

Maximum water volumes were recorded for each rainfall event and the full range of orifice sizes were tested. The method allows for a maximum discharge rate to be plotted against the maximum tank storage volume. The critical rainfall event was identified as the event that generates the highest storage volume for a given orifice diameter. With the roof areas attenuated using the retention and throttle RWH, the remaining impermeable areas were addressed. Roadways and parking areas have been modelled as draining into a geocellular storage tank. A similar iterative approach to that used in Option 1 has been used to minimize the tank size in light of the reduced input flows.

RESULTS

Option 1: RWH tanks for non-potable use were designed to comply with BS8515:2009's intermediate approach at each property. Six of the properties require V_{NP} of 2.05 m^3 with the seventh's lower occupancy indicating that 1.65 m^3 will suffice. The drainage design approach modelled in Option 1 has identified that a geocellular storage tank of 18 m^3 is the minimum tank size required to accommodate stormwater runoff from the entire site to comply with the global design criteria.

Option 2: Roof-runoff was routed into individual RWH tanks at each house. Flows from the upper part of each tank are controlled using an orifice. Approximately half of the impermeable areas proposed on site comprise roofs, and therefore half of the site's 7.2 l/s discharge rate has been allocated to the RWH outlets, equating to 0.5 l/s/RWH tank. Figure 2 illustrates the range of source control volumes required to accommodate the critical rainfall events for a range of orifice sizes. At peak discharge rates of 0.5 l/s a 1.5 m^3 V_{SC} is required with a 20 mm orifice outlet. The remaining 3.5 l/s of discharge remains allocated to the parking and roadways so that a separate storage tank can control runoff from these areas. A total of 10 m^3 of storage is the minimum volume identified for this tank.

The overall RWH tank size implemented in Option 2 is given in Equation 3;

$$V_{RWH} = V_{NP} + V_{SC} \quad (3)$$

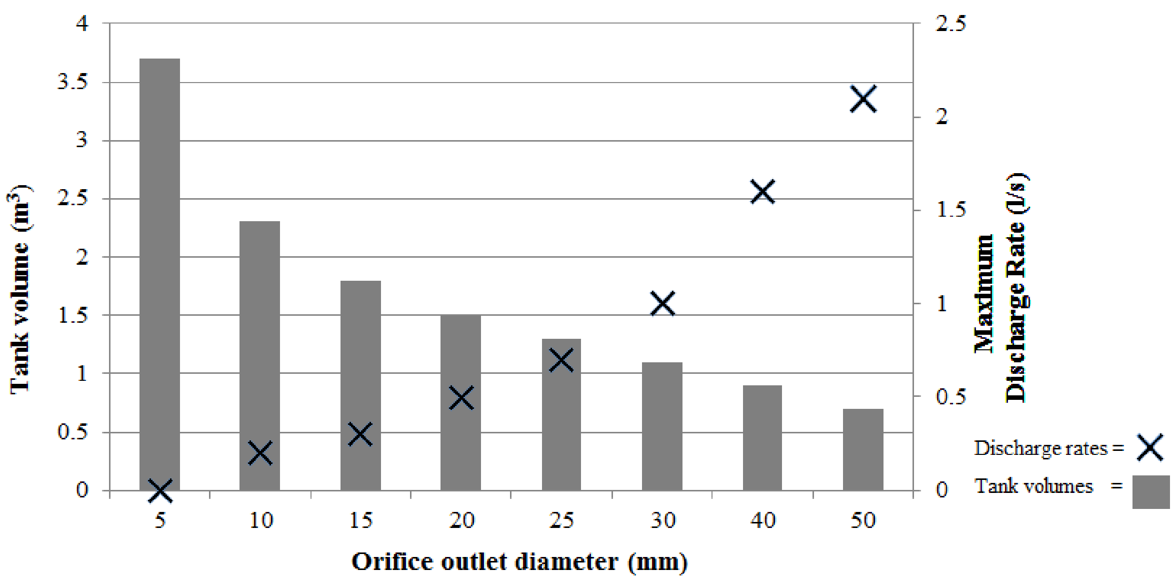

Figure 2 – V_{SC} and maximum discharge rates for a range of orifices tested.

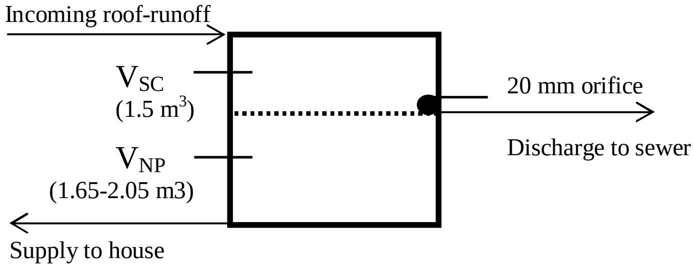

Figure 3 – Proposed retention and throttle RWH tanks for the case study site.

For the case study assessed, the final size of each RWH tank is illustrated in Figure 3 and a summary of results is set out in Table 2.

Table 2 - Summary of Results

Description		**Option 1** Large attenuation tank with throttled outlet discharging all drainage	**Option 2** Small attenuation tank with retention and throttle RWH tanks discharging roof drainage from each house
Discharge rates (l/s)	*Roof Areas*	-	3.5
	Paved Areas	-	3.5
	Total	<7.2	<7.2
Attenuation Tank Size		18 m^3	10 m^3
Minimum RWH tank sizes		6 x 2.05 m^3 & 1 x 1.65 m^3	6 x 3.55 m^3 & 1 x 3.15 m^3
Commercially available RWH tank sizes		7 x 2.7 m^3 (Total V_{RWH} = 18.9 m^3)	7 x 3.8 m^3 & 20 mm orifice (Total V_{RWH} = 26.6 m^3)

DISCUSSION

The study set out to identify and test a design method to integrate passive source control into RWH tanks without relying on water demand to drain the tanks. The designs achieved for the case study site show that the concept is feasible, and the method can be implemented using currently available software. Results from Option 2 demonstrate that a site's attenuation tank can be reduced in volume as a result of including retention and throttle RWH systems. Through demonstrating a reduction in the overall volume of the attenuation tank, capital savings are generated. For Option 1, the smallest appropriate RWH tank identified from a local supplier was 2.7 m^3, each costing £3,440 installed. In Option 2, a 3.8 m^3 tank is commercially available costing £3,600 each. The integration of retention and throttle RWH would therefore cost an additional £1,120 for the case study site. This cost is offset by the reduced attenuation tank size which is 8 m^3 smaller than the design implemented in Option 1. It is conservatively estimated that the reduced size geocellular storage tank would generate a saving in excess of £2,500 for the drainage contractor. It is therefore very likely that the implementation of the retention and throttle RWH design at the case study site would generate cash savings for the property developer over the traditional design option.

It is noted that water quality issues may arise though risks of contamination from the sewer overflowing into the orifice. However, a one way valve is frequently deployed to guard against this risk in existing RWH overflow systems. Secondly, the small orifice outlet is at risk of blockage. Installing appropriate filters for water entering the tank; a mechanical clearance mechanism or a blockage alarm could all help overcome the risks associated with blockage. By implementing retention and throttle RWH, runoff from roof areas can be fully attenuated without other SuDS being deployed as the runoff entering the tank discharges at the specified rate up to the 1 in 100 year return period event. Furthermore, with a reduced attenuation tank proposed for the remainder of the development, it may be feasible to address the remaining runoff using swales, infiltration trenches or above ground features integrated into the development layout, thus allowing the attenuation tank to be completely removed from the development costs. The proposed design methodology will potentially allow RWH to become more economically viable at new developments. Further work in this field is needed to provide empirical evidence that the modelled strategy set out in this study is technically feasible and to test its wider applicability at a pilot site.

CONCLUSIONS

The methodology developed and tested in this paper has demonstrated that passive stormwater source control could potentially be incorporated into household RWH systems with a relatively small adjustment to the design configuration. Conclusions can be drawn as follows; (1) The retention and throttle concept can be used for source control by installing oversized RWH tanks which incorporate an additional mid-level outlet throttle; (2) Optimal sizing of the RWH volumes and outlet orifices is needed on a site specific basis; (3) Retention and throttle RWH at residential properties within England and Wales may be achieved at a relatively low cost; (4) Technical barriers to implementation, such as site-specific constraints (e.g. ensuring the throttle outlet can gravitate into existing drainage infrastructure) will prevent this methodology providing an integrated solution that will suit all development plots; (5) Further development of this concept could include the development of a design tool to allow designers to select appropriate RWH attenuation volumes and orifice sizes for specific locations; (6) Passive RWH can contribute to source control within a SuDS system when either of the following factors are applicable; the Yield/Demand ratio is <0.95 or passive source control is integrated into the tank design using the throttle and retention concept.

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