Effects of tanks on peak flow rate of runoff

Reducing peak flow rates of runoff with tanks

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Foreword

This research project is been conducted at the University of South Australia at the Centre for water management and reuse. I have been here supervised by Dr. Baden Myers and Mr. David Pezzaniti. I would like to thank them for all their help, advice and effort I have received from them during the time I worked on this project. I have learned a lot about the Australian way of water management in urban areas. I have had a real good time during my time in Adelaide. Also I would like to thank Dr. Markus Pahlow who was my supervisor for the University of Twente and has helped me with bringing this project to a good ending.

Summary

The increasing infill in the suburbs of Adelaide results in higher peak flow rates of runoff. This may lead to flow rates of runoff that are beyond the capacity of the drainage system. One way of preventing this from happening might be the use of detention or retention tanks. This report has investigated what the hydrological effects of detention and retention tanks are on the peak flow rates of runoff. Two types of storm simulation techniques are used for this. One type was applying a 19 year continuous series of rainfall data to the models and the other type was applying design events with a duration of 5,10,15 and 20 minutes.

A single allotment and a street with 19 allotments have been modelled in this report. To see what the effect are of infill the models are both made in two variations. One is the pre-developed variant and the other is the redeveloped variant. This last variant is after infill and has a much higher impervious area. The redeveloped allotment has been equipped with detention or retention tanks of varying sizes. Also the street has been equipped with single detention or retention tanks of varying sizes and the allotments on the street have been equipped with the detention or retention tanks of varying sizes.

The flow rates of runoff with an average recurrence interval (ARI) of two years have been calculated for the continuous series and the peak flow rates of runoff generated by the design storm events.

On the allotment scale detention tanks have a much higher impact on the flow rate of runoff with a two year ARI than the retention tanks. This is just for the continuous series, when applying the design events the detention and retention tanks turn out to have almost the same impact on the peak flow rates of runoff. The detention tank reduces the flow rate of runoff for both storm simulation techniques back to the level of the pre-developed allotment. For the design events the retention tanks do this too but during the continuous series none of the retention tanks can lower the two year ARI flow rate of runoff to the level of the pre-developed allotment.

On the street scale the detention tanks per allotment and the single lump sized tank work very well for both storm simulation techniques as well. The two year ARI flow rate of runoff from a pre-developed street is almost the same as from a redeveloped street equipped with a single lump sized detention tank or where the allotments in the street are equipped with detention tanks. Lump sized retention tanks perform much worse and have almost no effect on the two year ARI flow rate of runoff. The street where all allotments are equipped with retention tanks still has a higher two year ARI flow rate of runoff than the pre-developed street. The lump sized retention and detention tanks perform very well during a design event, better than the distributed tanks. This is because the volume of rainfall is lower than the volume of the tanks, except for the 38 kL variant.

The method of calculating the two year ARI flow rate of runoff turned out to be not ideal. The few highest flow rates of runoff occurring during the 19 years' time series had an impact on the two year ARI flow rate of runoff that was higher than it should be. As a result of this the calculated the flow rate of runoff with a two year ARI was higher than it should be. By ignoring the highest few results the two year ARI flow rate of runoff became much more realistic. To calculate an accurate two year ARI flow rate of runoff more research has to be done.

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1 Introduction

In the year 2013 almost 1.3 million people lived in Greater Adelaide (Australian Bureau of Statistics , 2014). It is anticipated that this number will increase to 1.85 million people by 2036 (Government of South Australia, 2010). To meet the demand for new dwellings, new homes will have to be constructed. Currently 50% of the new houses have been constructed in existing built-up areas by planning for higher densities in strategic locations and the remaining 50% has been constructed in new 'greenfield' development on the urban fringe. By the year 2038 infill will represent 70% of new housing (Government of South Australia, 2010). This infill will lead to an increase impermeable surface in existing urban areas. This will have consequences for the current drainage system; it will have to convey more water in a short time frame.

The increased pressure on the drainage system will result in more failures of this system. This may bring high costs with it as a result of flood damage. To minimize the costs of water damage after a storm, the government can take several measures. The drainage system could be upgraded so that it is capable of conveying the increased water flows. This would, however, cost a lot of money and would be very cumbersome for existing residents and for traffic during the upgrade works. An alternative solution is to keep the water flow the same in spite of the increased impermeable surface area. This can be done by retaining and/or detaining runoff water in the catchment. This research report will focus on the latter approach.

1.1 Water sensitive urban design

Water sensitive urban design is a method of urban design that integrates water flows in the urban design. This can help control runoff without requiring drastic measures. One of the ways to do this is to use water tanks to store water for a certain period. These tanks can be placed at every house and so collect rainwater from individual roofs or they can be placed at the end of the street or at the end of a catchment.

1.2 Previous work

This research project is a continuation of the research done in '*Water Sensitive Urban Design Impediments and Potential:Contributions to the Urban Water Blueprint*' (Myers, 2012). In section 6 of this report the effects of retention tanks and detention tanks per allotment for a whole catchment. Only 1 kL, 5 kL and 10 kL have been examined in this report. The results of that research were that detention tanks and retention tanks were not able to reduce the peak flow rate of runoff to the desired level.

In '*WSUD: Basic procedures for 'source control' of stormwater'* (Argue, 2004) the effects of retention and detention tanks have been investigated. But this was also on a larger scale than this research project will do.

In *"Detention/retention storages for peak flow reduction in urban catchments: effects off spatial deployment of storages"* (Pezazaniti, Argue, & Johnston, 2002) the influence of the position of the retention or detentnion tanks on the peak flows out puts of the catchment have been explored.

In *"Potential for Peak Flow Reduction by Rainwater Harvesting Tanks"* (Campisano, Liberto, Modica, & Reitano, 2014)the effects of raintanks of various sizes on the peak flows has been investigated. The conclusion of this research was that the there was a major impact on the peak flow, depending on the size of the tank and the frequecy of the storm events.

1.3 Research objectives

There are two main objectives of this research project. The first one is to investigate the effects of water tanks on the peak flow rate of runoff from a catchment. This will be done for two types of tanks. Detention tanks which hold water for a short period and then release it and retention tanks which hold water indefinitely. This water is subsequently used in and around the house or naturally infiltrated into the ground and will thus not flow into the drainage system. The difference in the effects on the peak flow rate of runoff will be investigated. Also the effects of the location of the tanks, including a small tank near every house or a larger tank at the end of catchment, on the peak flow rate of runoff will be investigated. The peak flow rate of runoff from a catchment where every house has a tank will be compared to the peak flow rate of runoff from a catchment with a single lump sized tank.

The other main objective is to investigate how the tanks will behave during a short design storm event and during a continuous rainfall series of 19 years and what the difference in impact of the detention and retention tanks on the peak flow rates of runoff is.

1.4 Research questions

Overarching research question: *What are the hydrological effects of retention- and detention tanks on runoff on an allotment scale and on a street scale located in Fredericks catchment, Adelaide, and how does this depend on the type of storm event*?

To answer this question the following questions will need to be answered:

1. *What is the peak flow rate of runoff passing through the outlet of a single allotment with a detention tank and with a retention tank? How do these compare to each other?*

2. *What is the peak flow rate of runoff passing through the outlet of a street when using a detention tank or a retention tank fitted to each allotment? How do these compare to each other?*
3. *What is the peak flow rate of runoff passing through the outlet of a street when using a single, lumped size detention tank or retention tank at the end of the street? How do these compare to each other? And how do these compare to the distributed storages?*

4. *How does the use of different storm event simulation techniques impact the peak flow rate of runoff predicted for an allotment and street model?*

2 Method and data

2.1 Software

The software used for this report is Storm Water Management Model (SWMM) version 5.1.007 developed by the United States Environmental Protection Agency. From the website of SWMM

> *SWMM is a dynamic hydrology-hydraulic water quality simulation model. It is used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators.* (United States EPA, 2015)

This description fits very well with the research that has been done for this report.

2.1.1 Accuracy of the software

The accuracy of the software can partly be determined by the output reports generated after every run. This will tell errors made during the calculation of the routing of the water and the errors in the runoff quantity.

Unfortunately there is no data available to verify the outcome of the models. The model of the Fredrick street catchment however has been calibrated and verified and the models used in this research report are based on this model.

2.2 Model setup

In this report two models were developed. Most of the properties of the allotment were derived from the properties of the model of the Frederick Street catchment developed by Myers, (Myers, 2012). A list of all parameters used in the model is provided in Appendix C.

Most of the rainfall that falls on the pervious area will infiltrate in the ground and in that way not influence the peak flow rate of runoff. Only during rain events with a high intensity might runoff from the pervious area occur.

2.2.1 Location

Between 1993 and 2013 the population density of Adelaide has increased. As a consequence a lot more houses have been built in 2013. As a result of this the area that is impervious has increased. This has consequences for the runoff generated by houses and streets. In *Water Sensitive Urban Design Impediments and Potential: Contributions to the Urban Water Blueprint (Phase 1)* (Myers, 2012) a model has been developed to investigate the effects of retention tanks and detention tanks in the Frederick catchment. A map of this area is shown in figure 1. The models used in this research report are based on this model.

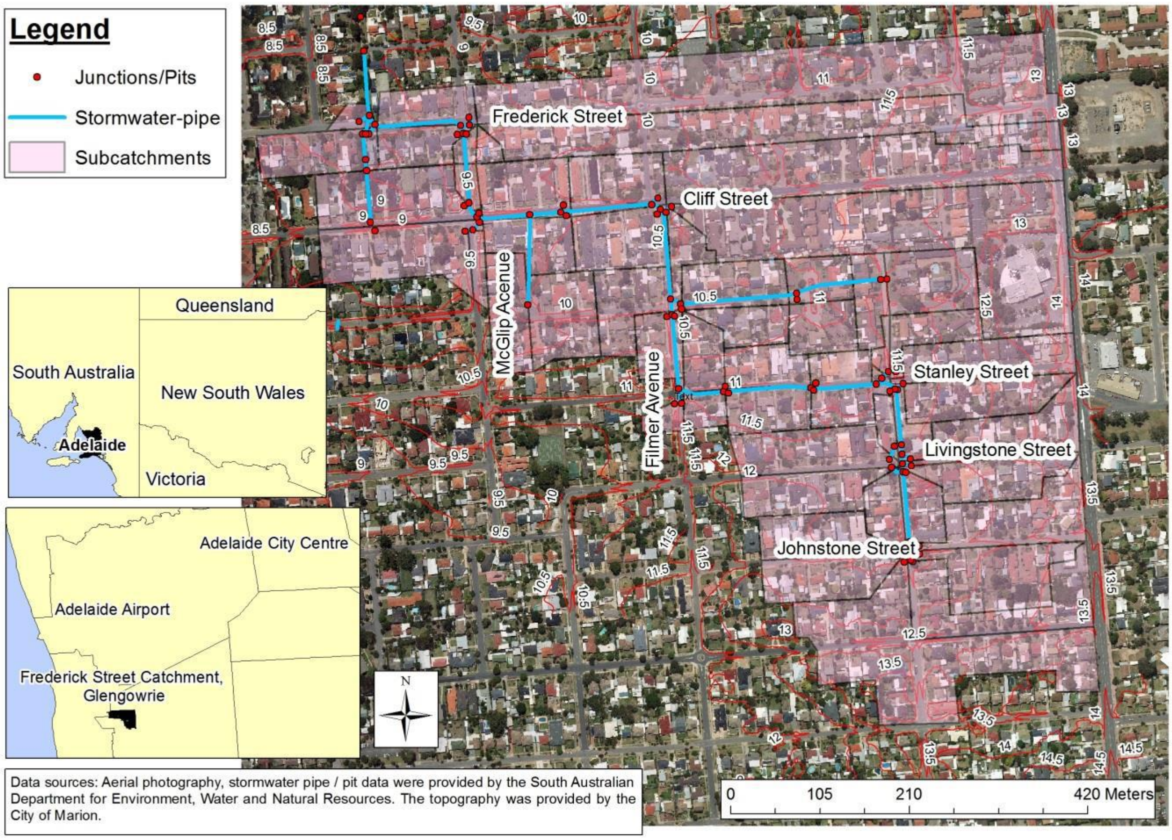

Figure 1 Frederick Street catchment

2.2.1.1 Allotment

In this research report a standard infill scenario representing a pre-developed and a redeveloped allotment is used in the modelling. This model of an allotment was developed by Argue in WSUD: "Basic procedures for 'source control' of stormwater" (Argue, 2004). In this scenario the impervious area has increased after the allotment is redeveloped, after redeveloping there are two houses on one allotment. In Figure 2 the layout of the two scenarios is shown.

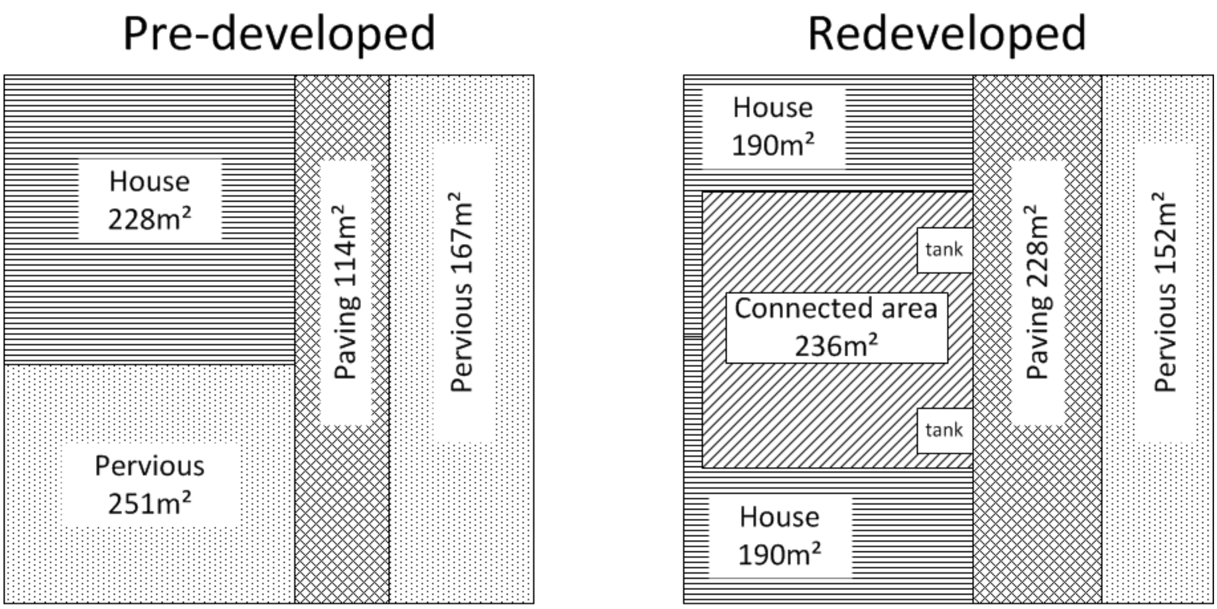

Figure 2 Layout allotments

In the allotment scenario it is assumed that all the runoff from the impervious area will flow directly into the drainage system, except for flows which are connected to the detention or retention tanks. The first part of the drainage system is the gutter of the street. All runoff generated by an allotment will flow onto the street. In this allotment scenario only the new impervious area will be connected to the tanks. This is 236m^2 for this allotment.

2.2.1.2 Street

The model of the street is based on two sub catchments of the Frederick Street catchment model, representing a whole street. In these two sub catchments there are 19 allotments and a road. Figure 3 shows the layout of the street, both the distributed tanks and the single lump sized tanks are displayed in this figure. The allotments in this street are developed in the same way as the single allotments. The redeveloped street exists thus 38 houses. There are two main scenarios investigated in the street scale model – on site management of runoff, or street scale management of runoff. In the former case, each of the allotments will have either a detention tank or a retention tank constructed at each home. This makes two tanks per allotment (just like the scaled up version of the allotment model). The other scenario considered in this project is one with a single detention or retention tank for the whole street. Whereas the tanks on the allotment are only connected to the new impervious area, the lump sized tanks for the street will be connected to all the impervious area. This was considered a reasonable assumption because all runoff collected from allotments and the road is conveyed by the kerb and gutter to the point of collection. It is not realistic to separate the new and existing development runoff.

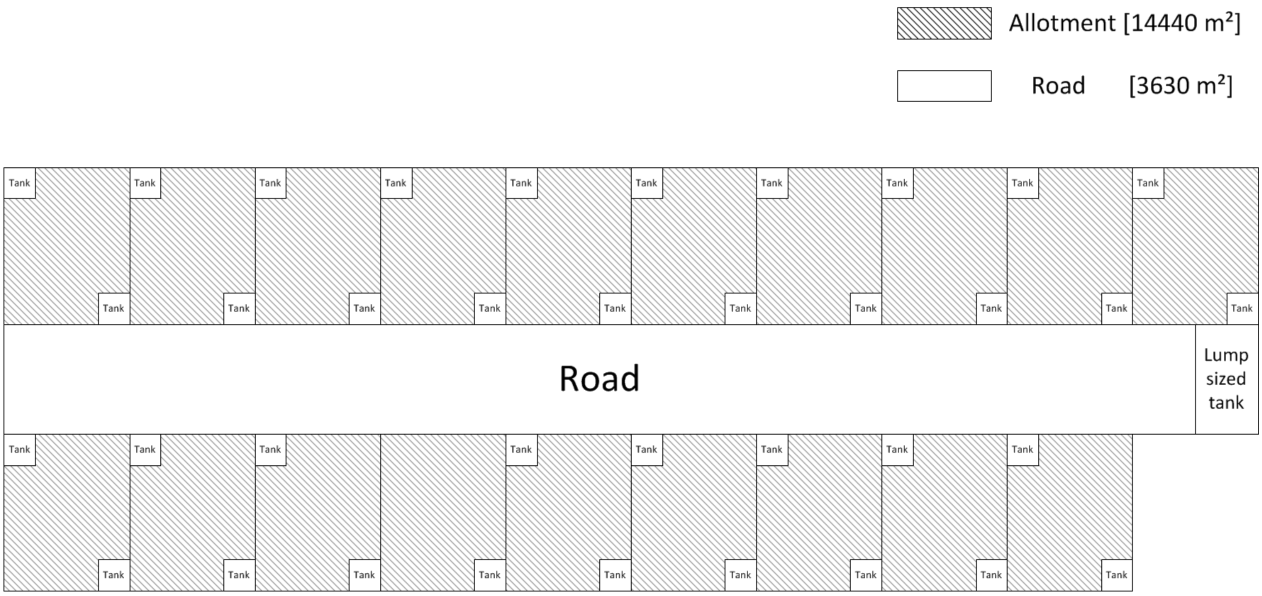

Figure 3 Layout street

2.2.2 Storm events

In this research project two types of storm events have been considered: a continuous series and a design event. These two types of events have been chosen to provide a good insight in the long term as well as the short term effects of the retention and detention tanks.

2.2.2.1 Design events

This project has focused on the runoff as result of an event that happens once in every two years. The drainage system that is currently used in Frederick Street is capable of processing flows that are generated by a storm event with a two year average recurrence interval (ARI). This means that every two year a storm will occur that causes the drainage system of the Frederick Street catchment to flood. Because of this the design events for this research project will be storm evens with a two year ARI. The design events were derived from the continuous series. This was done instead of using the standard design events in order to produce design events that may be directly compared to the results from the continuous series. In Appendix B it is shown how the design events were derived from the continuous series. Four design events of various durations have been used in this research project. The hydrographs for storm events of the five, ten, fifteen and twenty minute storm events are shown in Figure 4. The storm event with duration of 20 minutes has produced the highest peak intensity with 53.3mm per hour.

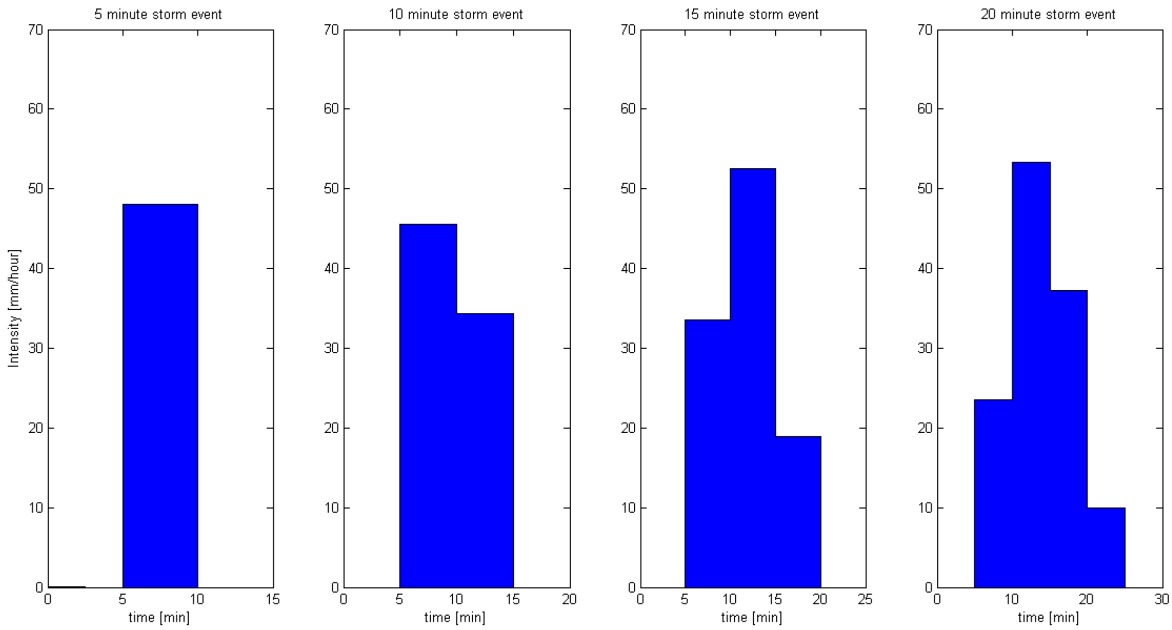

Figure 4 Hydrographs design events

2.2.2.2 Continuous series

The continuous series consists of the rainfall data from Parafield Airport (Adelaide). This is rainfall data from a pluviograph monitored by the Australian Bureau of Meteorology (Gauge number 023013). The data used was formatted at six-minute intervals. The data period was 1973 to 1992. This period was selected because it represents a long record of high quality data in Adelaide. When the model is run with this data it is possible to see the effect that the tanks have on the peak flow rates of runoff over a longer period. It enables a partial series analysis of the flow rates of runoff; this will be explained in chapter 2.3.

2.2.3 Retention and detention tanks

One means of relieving pressure on the drainage system is to temporarily detain, or permanently retain runoff in a catchment. This research project has investigated the effects of retention and detention tanks. The tanks were placed either per allotment, per street or per catchment. The impervious area that is connected to the tanks varies per scale, as has been explained in chapter 2.2.1. All the tanks are assumed to be two meters high and the area of the base will vary depending on the size of the tanks. An example of a tank on an allotment is shown in figure 5. A detention tank and retention tank will look the same. Just the way the tanks empty will be slightly different.

Figure 5 Tank on allotment

2.2.3.1 Retention tank

The retention tank will collect and retain water from the runoff system. Physically, it acts in the same way as a rainwater tank. All the water that is collected in a retention tank will gradually be used. It will get reused in various manners, from irrigation to flushing the toilet. A sketch of the retention tank is shown in figure 6. Based on current data on rainwater tank demand in Adelaide, South Australia, this results in an outflow of 100 L per day per house (Myers, 2012). On the allotment retention tanks will be placed with a volume ranging between 1 kL and 10 kL. When a tank is full it will overflow and the water will flow to the drainage system. The end of street or end of catchment ' lumped' retention tanks will be as big as all the tanks on all the allotments combined and will drain at the same rate as all the tanks combined (100 L/day per home).

2.2.3.2 Detention tank

The detention tanks will store water for a certain period of time. However, in contrast to the retention tanks, the detention tanks will gradually release the water back into the drainage system.

The volume of the detention tanks on the allotments will range from 1kL to 10kL. The lump sized detention tanks will have the same size as all the tanks on the allotments combined. On street scale the volume of the lump sized tanks varies from 38 kL to 380 kL.

All detention tanks have an orifice outlet located at the bottom through which the water flows back into the system. The time it takes to empty the tanks depends on the orifice size and the tank size. This can have an impact on the performance of the tank and the outflow hydrograph of the catchment. To determine the optimal orifice size a range of orifice sizes for the detention tanks will be assessed. These different scenarios will all be run on the basis of the data from the continuous series. To determine which orifice size is the best the runoff with a two year ARI will be compared for the respective orifice sizes. This two year ARI peak flow rate will be determined via the method described in chapter 2.3.

All the runoff generated by the impervious area will flow through the detention tank. As long as the tank is not full the only runoff flow from the catchment is from the water that flows through the orifice in detention tank. The flow rate through the orifice is depending on the water depth in the tank, a higher water depth results in a higher flow. When a detention tanks is full the runoff from the catchment will be the runoff flow generated by the impervious area connected and the outflow of the detention tank.

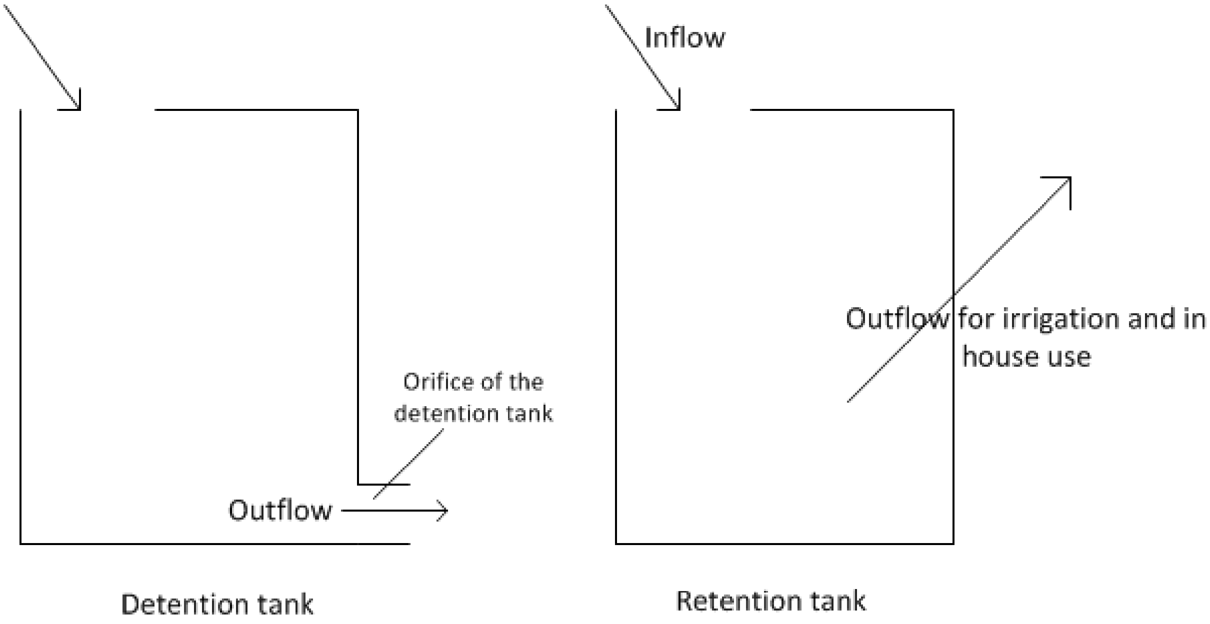

Figure 6 Detention and retention tank

2.3 Comparing results

The results of the detention tank and the retention tanks when the continuous series and the design storm are applied will be compared in two steps. First the difference in effect of the tanks when the two storm types are applied will be investigated. Second the difference in effect of the tank type and size will be investigated.

The impact of the different storm event simulation techniques on the peak flow rate of runoff will be compared by comparing how much the peak flow or the two year ARI is reduced as a result of the retention or detention tanks.

The tank type and size will be compared to each other per type storm simulation technique. To compare the scenarios for the continuous series the peak flow rate of runoff with a two year ARI will be calculated. The peak flow rates with a two year ARI will be compared. To calculate the two year

ARI 5 steps were performed according to the method described in *Australian Rainfall and Runoff* (Pilgrim, 1999):

1. All the flows of runoff that are lower than the lowest peak flow rate of runoff of the annual peak flow rate of runoff will be ignored.
2. The flows of runoff will be sorted from high to low and ranked. The highest peak flow rate of runoff will get rank one, the second highest peak flow rate of runoff event will get rank two, etc.
3. The ARI for the flows of runoff is calculated by the Cunnane formula: $ARI = \frac{year + 0.2}{rank - 0.4}$
4. The runoff will be plotted against the ARI and a line of best fitting will be drawn for the flows of runoff. For this report a polynomial of the third order is used to approach the line of best fit. To get the best line of fit for calculating the two year ARI it was necessary to ignore some events. The amount of events has to be calculated and this will be done in Chapter 3.1
5. Using the polynomial the two year ARI can be calculated.

For the design storm events the peak flow rates of runoff will be compared under the different scenarios.

3 Results

3.1 Partial analysis

The method for the calculation of the two year ARI peak flow rate of runoff has a major influence on the results of this research. The two year ARI peak flow rate of runoff is estimated based on a trend line through the peak flow rates of runoff occurring during the 19 year that are higher than the lowest annual maximum for the whole period. In figure 7 the trend line has been plotted for a 76 kL and 228 kL tank at the end of the street. The vertical line indicates where the two year ARI is. Figure 8 shows the same but now the nine highest events have been ignored. In the first case the peak flow rate of runoff with a two year ARI is over estimated, the trend line deviates from the actual events around the two year ARI. This is because the few events that have an ARI that is higher than two year have a much higher peak flow rate of runoff and raise the trend line. When the last nine events are ignored the trend line will be lower. Because the two year ARI events are around the point where the trend line switches from a linear line to an exponential line the effects of the last few events are important. However the effects of ignoring events on the trend line will not be the same for all scenarios. This has to be further investigated so a more precise method can be used for the calculation of the two year ARI flow rate of runoff.

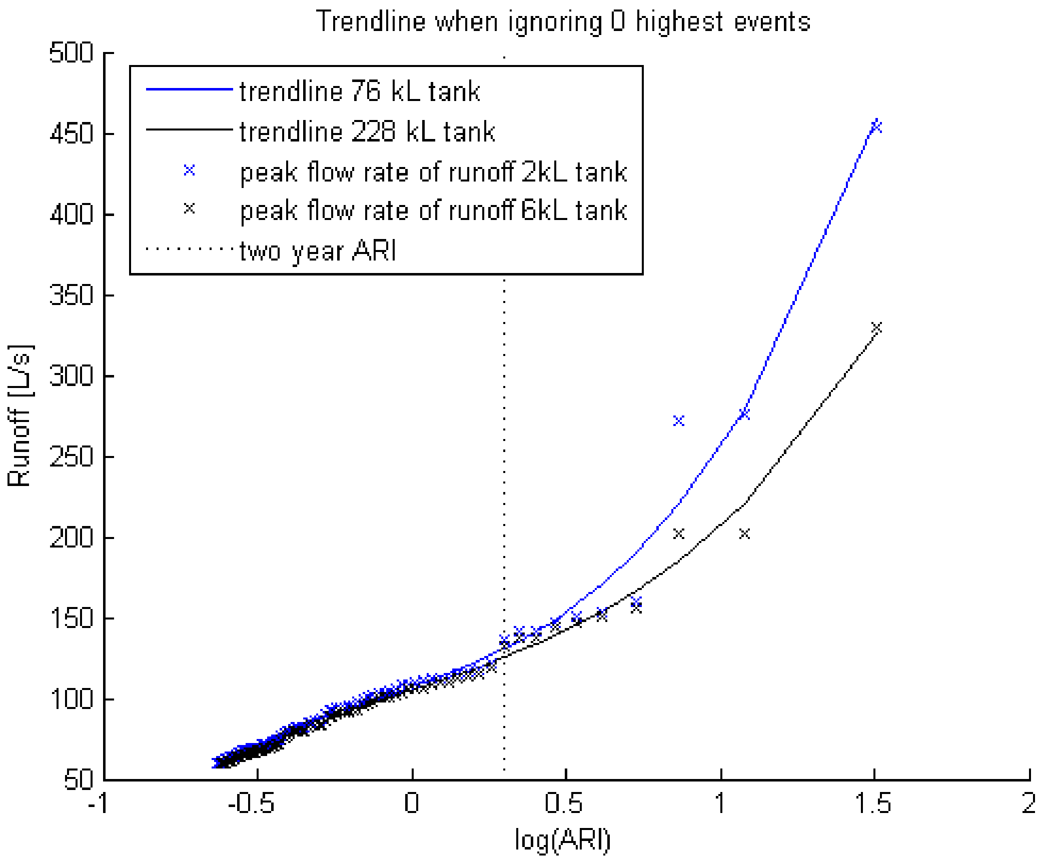

Figure 7 Trend line ignoring no events

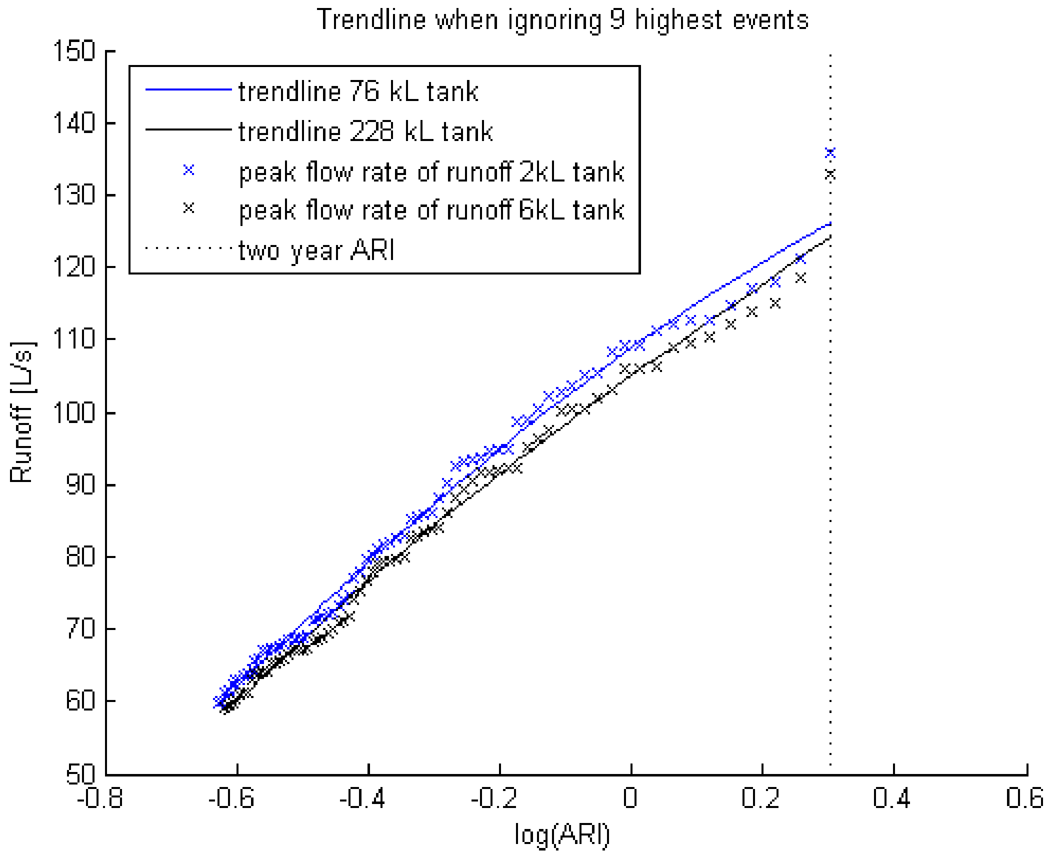

Figure 8 Trend line when ignoring 9 highest events

But when the two year ARI peak flow rate of runoff from the street is calculated the results are unrealistic. As can be seen in figure 9 the two year ARI peak flow rate of runoff from the street with the lump sized retention tank is higher than from the street without any tanks. This is impossible. To get a more realistic peak flow rate of runoff less events are ignored. Without the top three events the result are the most realistic.

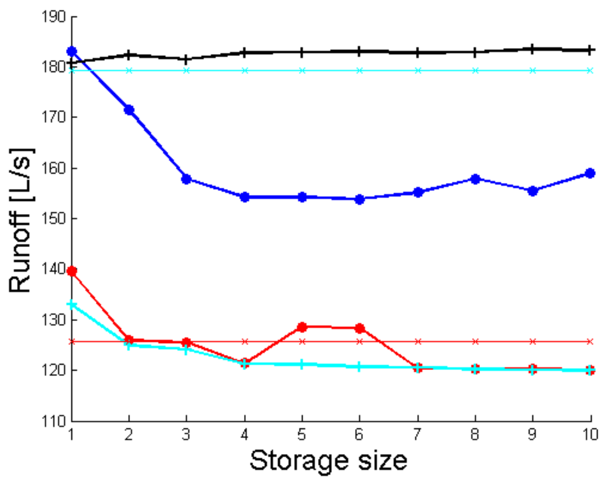

Figure 9 2 year ARI peak flow rate or runoff from the street

3.2 Allotment scale

The runoff from the allotment has been modelled for a range of different scenarios. The two types of storm events have been applied to the model of the pre-developed allotment, the redeveloped allotment and the redeveloped allotment with the detention or retention tanks of the various sizes on it.

3.2.1 Orifice sizes for detention tanks

The two year ARI peak flow rate of runoff has been calculated for an allotment with two detention tanks with various orifice sizes. The optimum orifice size for each tank is shown in table 1.

Tank size [m³]	1	2	3	4	5	6	7	8	9	10
Orifice size [mm]	160	120	120	60	60	60	40	40	40	40

Table 3.2-1 Orifice size detention tank allotment

In Appendix C the figure is shown with the two year ARI peak flow rate of runoff of all the tanks and orifice sizes.

3.2.2 Continuous series

In Figure 10 the peak flow runoff with a two year ARI is shown computed by the method explained in 2.3. The 2 year ARI peak flow rate of runoff from the pre-developed allotment is 4.28 L/s. After redeveloping the allotment the runoff has increased to 7.37 L/s. The extra 3.09 L/s has to be compensated by the tanks on the allotment.

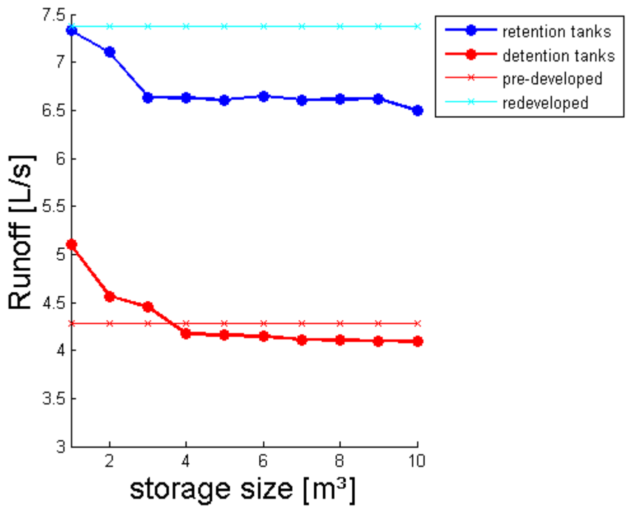

Figure 10 two year ARI flow rate of runoff

The detention tanks have a far bigger impact on the runoff peak flow rates than the retention tanks. The one kilolitre retention tank has the least effect on the peak flow rates of runoff. The peak flow rate of runoff with a two year ARI is 7.33 L/s. The difference between the peak flow rate of runoff with a two year ARI from an allotment with two three kilolitre retention tanks and from an allotment with two ten kilolitre retention tanks is relatively insignificant. The peak flow runoff from the first is 6.64 L/s and from the other 6.49 L/s. None of the retention tanks included in the model are able to lower the runoff to the runoff form the pre-developed allotment. The highest value of reduction by a retention tank is 0.88 L/s. This is accomplished by the 10 kilolitres retention tank.

A one kilolitre detention tank reduces the peak flow rate of runoff by 2.27 L/s to 5.10 L/s. This is not enough to reduce peak flow rates to the pre-developed allotment case. An allotment with two four kilolitre detention tanks has a runoff of 4.17 L/s which is 0.1 L/s less than the peak flow rate of runoff from the pre-developed allotment. Larger tanks do not reduce the runoff much more than this. Two ten kilolitre tanks bring the runoff back to 4.09 L/s.

3.2.3 Design event

The four design events of various durations were applied to the model of the allotment. The resulting peak flow rates of runoff from the allotment scenarios are shown in Figure 11.

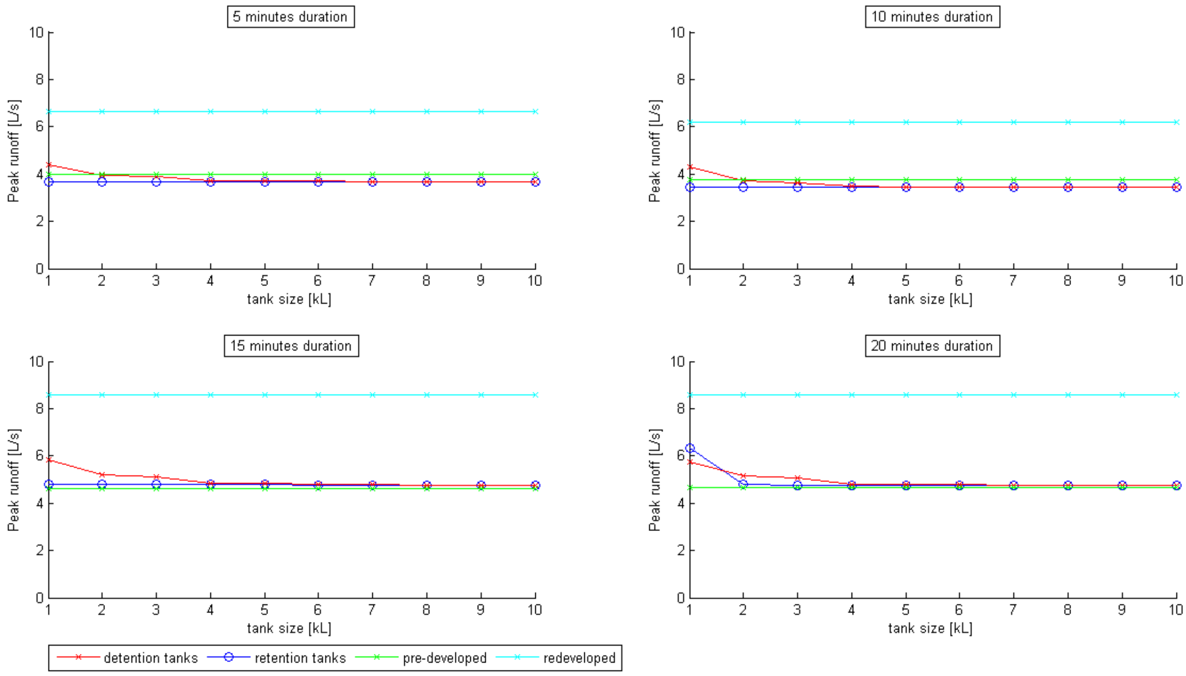

Figure 11 Peak runoff during design events from allotment

The effects of all the tanks on the runoff are almost exactly the same. For a design event with a length shorter than 20 minutes all the tanks are able to reduce the runoff coming from the redeveloped allotment to the rate of runoff from the pre-developed allotment. In the storm event with duration of 20 minutes the 1kL retention tank fills completely. The tanks that are bigger than one kilolitre do not fill up completely. They do however fill up and release a certain amount of water at the same time. This will add a small flow to the total flow rate of runoff. To get the best performance from the detention tanks larger than one kilolitre during the 20 minute design events the orifice size would be zero. This is because the tanks have enough volume to store all the runoff generated by this design event. The volume of the runoff from the area connected to the tank that is generated by the event is 2.8 kL. This is less than the volume of the two individual two kilolitre tanks on each redeveloped allotment. When the duration of the design events increases also the larger tanks will fill up and an orifice will be required to reduce the peak runoff again.

3.2.4 Comparison allotment scale

3.2.4.1 Storm Simulation Type

The results from the different storm event approaches are significantly different. Where the retention and detention tanks have a similar runoff during one of the design events, the two year ARI flow rate of runoff predicted from the continuous series is not similar at all. This is due the fact that for the design events the tanks are empty at the start of the rainfall event. During the continuous series the tanks are not empty before every major storm events occur. The retention tanks are more likely to be full because they empty at a rate that is much slower than the detention tanks.

3.2.4.2 Tank type and size

Even the worst (1 kL) detention tank is much more effective than the best (10 kL) retention tank. The low effectiveness of the retention tanks, when the continuous rainfall data is applied to the model, is caused by the smaller rain events before a major storm event. The runoff generated by the smaller events fill the tank, leaving less storage space for the runoff generated by the major event. In Figure 12 the water depth is depicted for a 2 kL detention tank, using the optimum orifice size for this tank, and a 2 kL retention tank during an event on 1983-03-02. The highest runoff occurs at 17.15 but by that time both types of tank are already full and cannot influence the runoff.

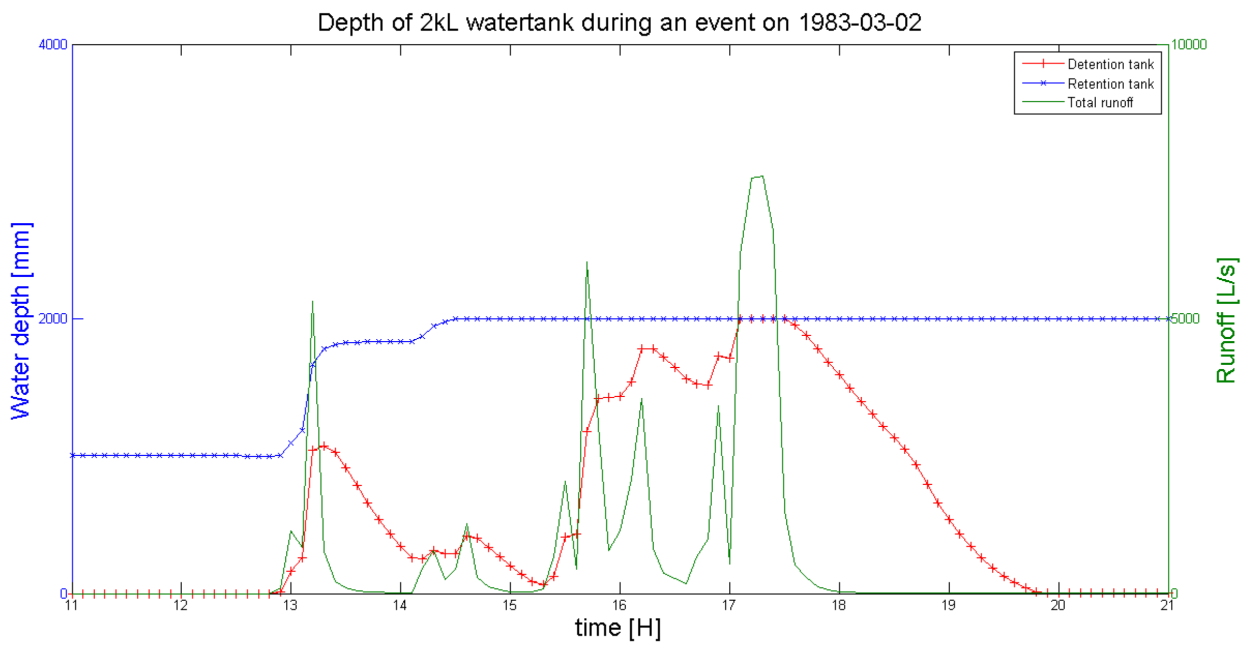

Figure 12 Water depth of 2 kL water tank during an event on 198-03-02

The detention tanks empty faster than the retention tanks. This makes the influence of smaller events before major events smaller in the case of the detention tanks compared to the retention tanks. Because all the retention tanks empty at the same rate the runoff with a two year ARI per size varies little. The volume of the tank has a much lower influence on the runoff compared to the detention tanks.

For a short design event it is most effective to store all the runoff generated by the area connected to the tank. Only the one kilolitre tank will fill up during the design events. During a design event a retention tank has no outflow and thus performs better than the detention tanks which do not store all the runoff. When looking to the performance of the tanks during the 19 year continuous simulations the detention tanks are much more effective than the retention tanks. An allotment with two detention tanks (total allotment storage of 4 kilolitres) has a runoff that is lower than the runoff from the pre-developed allotment. The retention tanks are much less effective and can reduce the peak flow rate of runoff gap by a maximum of 34.5%

3.3 Street scale

The same scenarios that were applied to the model of the allotment scenarios (SECTION 2.2.1.1) were applied to the street scale, where 19 allotments with one home were assumed to undergo infill and result in 38 homes as described in Section 2.2.1.2. Two additional scenarios were also explored, a scenario where all the runoff flows into a single, end of street detention tank and one where it flows into a single, end of street retention tank.

3.3.1 Orifice sizes for detention tanks

For the scenarios where every house has a detention tank, the orifice sizes will be the orifice sizes as are found in 3.1.1 and shown in table 1. For the scenarios where the street is equipped with a single detention tank the peak flow rate of runoff with a two year ARI has been calculated for a range or orifices sizes. The optimum orifice sizes are shown in Table 2.

Tank size [m^3]	38	76	114	152	190	228	266	304	342	380
Orifice size [mm]	550	450	450	250	250	200	200	150	150	150

Table 3.3-1 Orifice size end of street detention tank

3.3.2 Continuous series

In figure 13 are the two year ARI flow rate of runoff plotted for the various tanks. The two year ARI runoff from the pre-developed street is 126.0 L/s. For a redeveloped street this will increase to 183.8 L/s. This is an increase of 57.8 L/s.

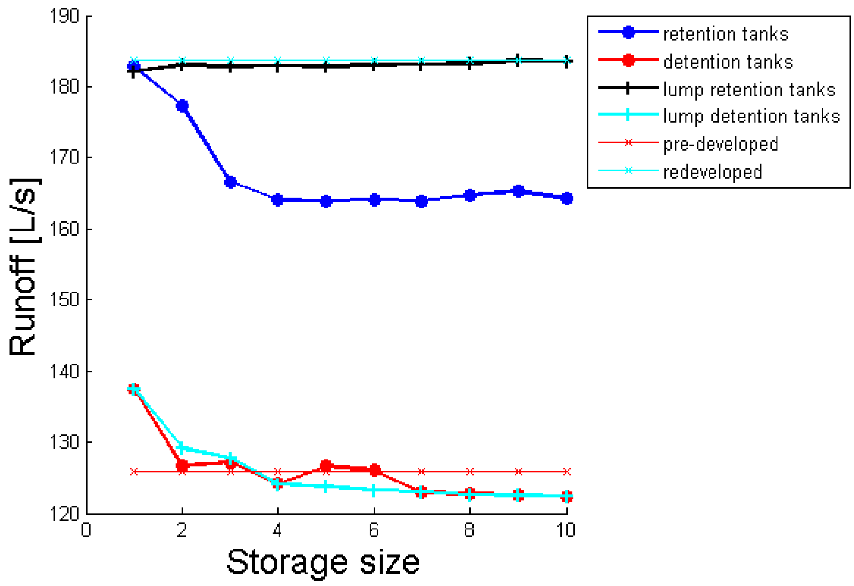

Figure 13 two year ARI runoff derived

The detention tanks are the most effective way to reduce the runoff on the street scale. Despite the fact that all the impervious area was connected to the lump sized tanks the peak flow rate of runoff was similar to the peak flow rate of runoff from the street with distributed detention tanks. On the street scale, just as on the allotment scale, two four kilolitre detention tanks per allotment were enough to reduce the runoff back to the same level as the pre-developed street. A street where every allotment had two four kilolitre detention tanks or where there was one end of street equivalent 152 kL tank generates a runoff of 124 L/s. When all houses are equipped with a ten kilolitre detention tank or the street with the equivalent 380 kL tank the runoff decreases to 122.4 L/s.

Also on the street scale it seems that the scenario with the distributed detention tanks has a lower two year ARI flow rate of runoff than the pre-developed street. Just like with the detention tanks on the allotment scale this is not possible and is probably caused by the same errors as mentioned for the allotment scale in section 3.1.2.

The lump sized retention tanks have almost no influence on the two year ARI runoff. This might be because the increased connected area makes the tanks fill up faster. Because they fill up faster because they are connected to all impervious area in the catchment, they may be full when major storm events occur. The distributed retention tanks are better than the lumped size retention tanks. When every allotment is equipped with two ten kilolitre tanks, the difference in runoff between the redeveloped and pre-developed street can be reduced by 19.5 L/s. The minimum flow rate of runoff that can be achieved by using retention tanks is 164.2 L/s.

3.3.3 Design event

The highest peak flow from the pre-developed street occurs during the 15 minute design event and is 188.2 L/s. During the same event the peak runoff from the redeveloped street is 277.9 L/s. Figure 14 shows what peak runoffs occur when the street is equipped with a single tank or when all the allotments have two tanks.

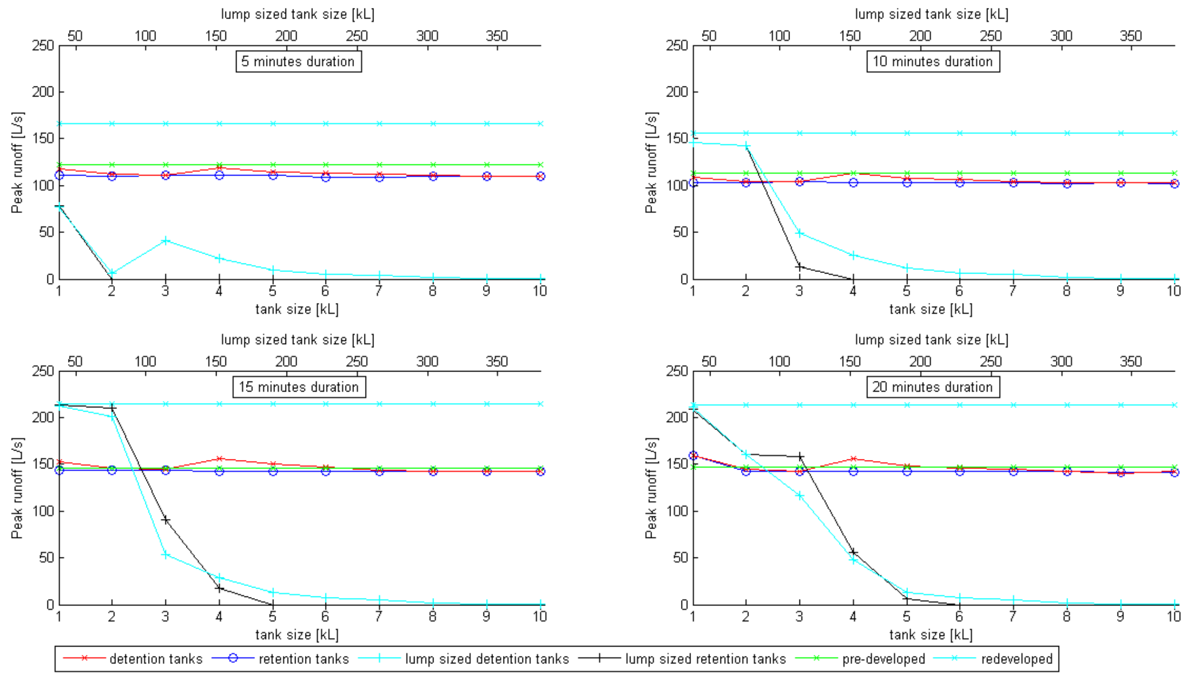

Figure 14 Peak runoff during design events from street

With neither detention nor retention tanks on the allotments is it possible to get a peak flow rate of runoff that is lower than the runoff from the pre-developed street. With a single lump sized tank this may be feasible, due to the connection of all the impervious area - when the tank is large enough the runoff will be reduced to zero. This is caused by the difference in the area that is connected to the tanks. A street where every allotment has two individual two kilolitre tanks is enough to reduce the runoff to the pre-developed level. Detention and retention tanks have almost the same performance because the volume of the tank is the most important factor here. When a tank overflows it will have a major impact on the runoff from the street. The lump sized tanks are connected to all the impervious area and thus collect more water than all the distributed tanks combined. A lump sized tank of 76 kilolitres is therefore, in most cases, not enough to restore the runoff to the pre-developed street runoff. A 152 kilolitre tank will be enough for the design events of all four durations. Tanks that are bigger than 228 kilolitres will store all the runoff generated by the design event. During the 20 minute design event a total of 10.33 mm rain falls, the runoff volume generated from the impervious are by this event is 156.8 m^3.

3.3.4 Comparison street scale

3.3.4.1 Storm Simulation Type

The performance of the tanks is very different when the two storm types are applied. The biggest difference is with the lump sized retention tank. During the continuous series the impact of the lump sized retention tanks is almost zero while during the design events it can store all the runoff volume, provided that the tank is large enough. The lump sized detention tanks are also capable of storing all the runoff during the design events. The distributed detention tanks have similar performance for the two types of storm events. In both cases the tanks are able to reduce the peak flow rates of runoff to the level of the pre-developed street. The lumped size detention tanks perform much better when a design storm event is applied. When the tank is bigger than 190 kL it is able to store all the runoff. With the continuous series applied the runoff can only be reduced to a little less than the two year ARI flow rate of runoff coming from the pre-developed street.

The street with a detention tank bigger than 114 kilolitres or detention tanks larger than 3 kL per house have a lower two year ARI runoff than the pre-developed street. During the design event the street with distributed detention tanks never has a lower runoff than the pre-developed street.

3.3.4.2 Tank type and size

Retention tanks at the end of the street that collect all the runoff from the impervious area are very ineffective in reducing the two year ARI runoff. Retention tanks on every allotment are more effective than the lump sized variant. Detention tanks are however the most effective. Streets with detention tanks or where every allotment has detention tanks have a lower two year ARI runoff than the pre-developed street. This is only when the tanks on the allotment are bigger than four kilolitres or when the lump sized tank is bigger than 152 kilolitres. During a design event the tanks on every allotment are effective to achieve a runoff that is the same as the runoff from the pre-developed street. Only the one kilolitre tanks will overflow during these events. Because the lump sized tanks are connected to a larger area they will collect more water and thus overflow earlier than the distributed equivalents. Lump sized tanks can bring the peak flow rate of runoff down to zero L/s.

When applying the design event to the models the tanks seem very effective. This is not the case when the continuous series is applied. The retention tanks have no notable influence on the two year ARI runoff from the street.

4 Conclusion

The peak flow rate of runoff with a two year ARI from a pre-developed allotment is 4.28 L/s, after the allotment is redeveloped this is increased to 7.37 L/s. When the allotment is equipped with two four kilolitre detention tanks the two year ARI peak flow rate of runoff from the allotment is the same as from the pre-developed allotment. Increasing the tank size after this doesn't have much influence on the two year ARI peak flow rate of runoff. A retention tank is less effective than a detention tank. The lowest two year ARI peak flow rate of runoff from an allotment with retention tanks is from one equipped with two ten kilolitre tanks and is 6.49 L/s.

The performance of the retention tanks and detention tanks is almost exactly the same when the design events are applied. During every design event an allotment equipped with two individual two kilolitre tanks are able to reduce the runoff coming from the redeveloped allotment to the rate of runoff from the pre-developed allotment. Only during the 20 minute storm the allotments with one kilolitre tanks have a higher peak flow rate of runoff than the pre-developed allotment but much lower than the redeveloped allotment without tanks.

The peak flow rate of runoff with a two year ARI from a pre-developed street is 126.0 L/s, after the allotment is redeveloped this is increased to 183.8 L/s. Equipping the street with distributed detention tanks or with one lump sized detention tank gives almost the same two year ARI peak flow rates of runoff. Also in this case it is enough to have distributed two four kilolitre detention tanks or a 152 kL detention tank to reduce the peak flow rate of runoff with a two year ARI to the level of the pre-developed street. The two year ARI peak flow rate of runoff is than 124 L/s. A street with distributed retention tanks has always a higher two year ARI peak flow rate of runoff than the pre-developed street. The lowest peak flowrate of runoff achieved with distributed retention tanks is with the 10 kL tanks and is 164.2 L/s.

The redeveloped street with the disturbed tanks has a peak flow rate of runoff that is the same as for the pre-developed street for both tank types and for all sizes. The street equipped with lump sized tanks, however are able to generate almost no runoff at all.

The type of storm event has a major influence on the hydrological effects. When the design events are used both type tanks performed almost similar. All type tanks had a major impact on the peak flow rates of runoff. When the continuous simulation was used the lump sized retention tanks on the street scale had almost no effect on the two year ARI peak flow rate of runoff. A lump sized retention tank does not have any influence on the two year ARI peak flow rate of runoff from the street. The retention tanks per allotment had some impact on it but not enough to lower the two year ARI peak flow rate of runoff to the same level as from the pre-developed allotment or street. Detention tanks are able to do this. On an allotment or a street where every allotment has two four kilolitre detention tanks the two year ARI peak flow rate of runoff is lower than from the pre-developed street or allotment.

5 Discussion

This research report has raised a major issue that needs to be further investigated. This is the method to calculate the two year ARI flow rate of runoff. The method of calculating the two year ARI peak flow rate of runoff has a major influence on the results. In this research project an approach is used that gave the most reasonable results. This meant that some events had to be ignored. To get the best comparison between the different scenarios, other ARI value should be explored, including the two year ARI peak flow rate of runoff has to be calculated as accurately as possible. To do this more research is needed to find the best method for doing this.

When this method is established the effects of detention tanks and retention tanks on a larger scale can be investigated. Also several interesting variations on the currently used scenarios are worth investigating. The retention tanks are assumed to empty with a rate of 100 L/day in this research project. This is based on the minimum water demand out of the tanks. In the summer, when the heavy summer storm event with high intensity rainfall tend to occur in the Adelaide region the water demand out of the tanks will be higher. The need for water for irrigation will be higher for instance. This might make the retention tanks more effective during the summer and thus overall and the impact of this should be explored.

The design events as used in this research report are not optimal for testing the effects of water tanks on the flow rate of runoff. This is because the tanks are empty at the start of the events. When the tanks are assumed to be filled up by a certain percentage (or volume) they may give a more realistic effect on the peak flow rate of runoff. Further research can be done to what the most realistic percentage is for the tanks to be full.

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Appendixes

A. The model

Most of the calibration of the model has already been done for previous research. All the parameters used in the models in this report are explained in the tables 3 till 8.

General		units	Source
Subcatchment			
- **% Slope**	0.5	%	(Myers, 2012)
- **N-Imperv**	0.0013		(Myers, 2012)
- **N-Perv**	0.03		(Myers, 2012)
- **Dstore-Imperv**	0.05	Mm	(Myers, 2012)
- **Dstore-Perv**	5	Mm	(Myers, 2012)
- **%Zero-Imperv**	0	%	(Myers, 2012)
Infiltration Model			
- **Max. Infil. Rate**	100	Mm/hr	(Myers, 2012)
- **Min. Infil. Rate**	8	Mm/hr	(Myers, 2012)
- **Decay Constant**	3	1/hr	(Myers, 2012)
- **Drying Time**	5	Day	(Myers, 2012)
- **Max. Volume**	0	Mm	(Myers, 2012)
Nodes			
- **Node Invert**	0.1	m	Assumption
- **Node Max. Depth**	2	m	Assumption
- **Node ponded Area**	0	m^2	Assumption
Conduit			
- **Conduit length**	10	m	Assumption
- **Conduit Geometry**	Circular	-	Assumption
- **Conduit roughness**	0.01	-	(Myers, 2012)
Link Offsets	Elevation		Assumption
Routing Method	Kinematic Wave		(Myers, 2012)
Force Main Equation	Hazen-Williams		(Myers, 2012)

Table A-1 General properties

Allotment			Source
	Predeveloped	Redeveloped	
Area [m^2]	760	760	(Myers, 2012)
Width	42	42	(Myers, 2012)
%Imperv	45	80	(Argue, 2004)

Table A-2 Allotment properties

Street			Source
	Pre-developed	Redeveloped	
Area [m^2]	18070	18070	(Myers, 2012)
Width	993.8	993.8	(Myers, 2012)
%Imperv	56	84	(Argue, 2004) + (Myers, 2012)

Table A-3 Street scale properties

The retention tanks are designed to empty with a rate of 100L per day. In the model this is realised by letting 100L per day seep into the soil. The seepage rate per tank size is shown in Table 7.

Retention tank		Units	Source
Berm height	1000	mm	Assumption
Vegetation Volume Fraction	0.0	-	(Myers, 2012)
Surface Roughness	0		(Myers, 2012)
Surface Slope	0	%	(Myers, 2012)
Thickness	1100	mm	Assumption
Void Ratio	0.91	-	(Myers, 2012)
Clogging Factor	0	-	(Myers, 2012)
Flow Coefficient	0	mm/hr	(Myers, 2012)
Flow Exponent	0.51		(Myers, 2012)
Offset Height	0	mm	(Myers, 2012)

Table A-4 Properties of Retention tank

Retention tank Size	1kL	2kL	3kL	4kL	5kL	6kL	7kL	8kL	9kL	10kL
Seepage Rate [mm/hr]	8.33	4.17	2.78	2.08	1.67	1.39	1.19	1.04	0.93	0.83

Table A-5 Seepage rate for Retention tanks

Detention tanks		Units	Source
Berm height	1000	mm	Assumption
Vegetation Volume Fraction	0.0	-	(Myers, 2012)
Surface Roughness	0		(Myers, 2012)
Surface Slope	0	%	(Myers, 2012)
Thickness	1100	mm	Assumption
Void Ratio	0.91	-	(Myers, 2012)
Seepage Rate	0	-	(Myers, 2012)
Clogging Factor	0	mm/hr	(Myers, 2012)
Flow Exponent	0.51		(Myers, 2012)
Offset Height	0	mm	(Myers, 2012)
Flow coefficient	Depended on orifice size	mm/hr	

Table A-6 Properties detention tank

B. Derivation of design events

In this report are the design events derived from the continuous series from section 2.2.2.2. This is done because the model is calibrated for rainfall intensities occurring in that area. With design storm derived from the continuous series the results will be more accurate.

To be able to derive the intensities for the design events with a duration of five, ten, fifteen and twenty minutes with an ARI of two years the rainfall data from the continuous series has to be modified. The data is formatted in six minutes intervals, to get the intensities for longer events the data is modified to intervals of 12,18,24 and 30 minutes. This is done by taking the moving average for these periods from the data. To this data is the partial analysis applied, all the events are taken account. The intensities of an event with a two year ARI for the various durations are shown in table 9. In table 10 the intensities for design events of various durations used by the Bureau of Meteorology shown (Bureau of Meteorology).

Duration [min]	6	12	18	24	30
Intensity [mm/hour]	46.4	37.4	28.1	25.0	18.9

Table B-1 Intensities of events

Duration [min]	5	6	10	20	30
Intensity [mm/hour]	54.6	50.8	40.8	28.8	22.9

Table B-2 Official intensities of events

These are both plotted in figure 15. The official line is a little steeper but it is close to the derived intensities. Therefor it is reasonable to use the derived intensities. From this graph the intensities for the design evens with duration of 5, 10, 15 and 20 minutes can be read off. These intensities are shown in table 11

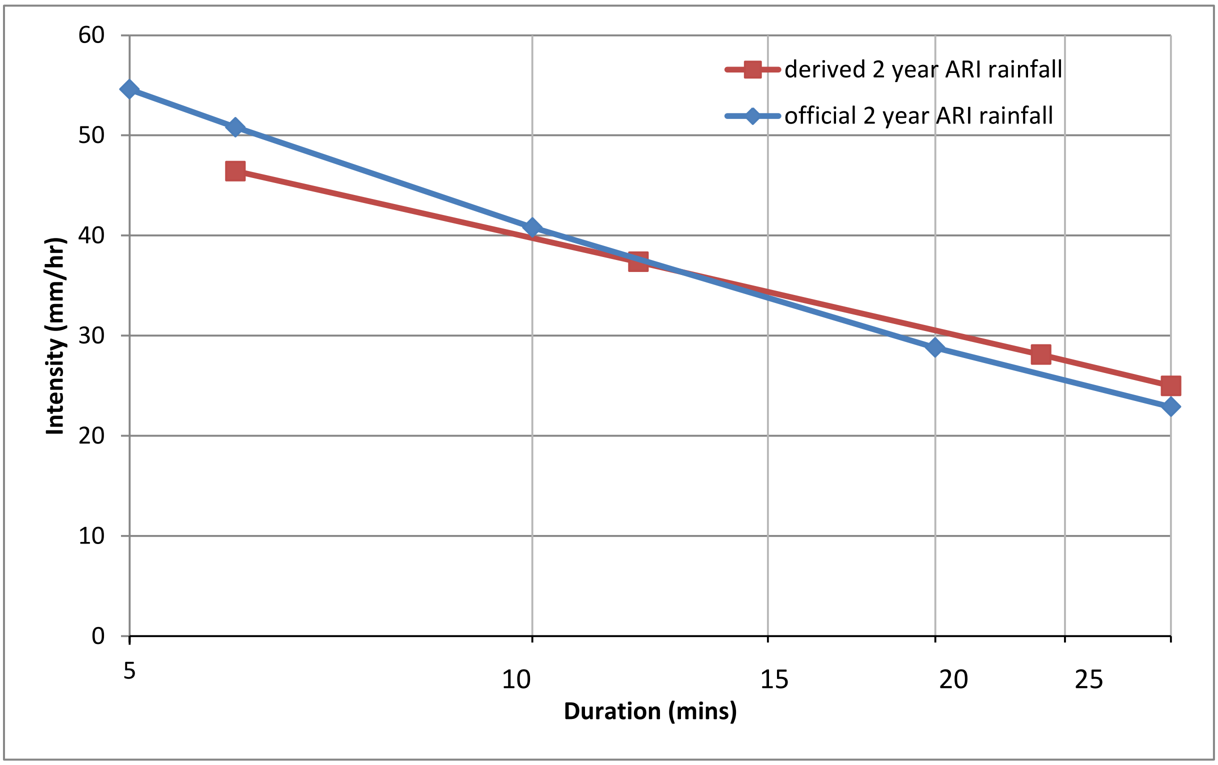

Figure 15 IDF curves

Duration [min]	5	10	15	20
Intensity [mm/hour]	48	40	35	31

Table B-3 Derived intensities for design event

The design events have to be formatted in periods of five minutes. To do this the intensities are divided according to the guidelines of the Bureau of Meteorology. This is shown in table 12 and table 13.

Duration	**5 min**		
Increment	**5 min**		
Intensity (mm/hr)	**48 mm/hr**		
Total rain	**4 mm**		
mins	**Proportion [%]**	**mm**	**mm/hr**
0	0	0	0
5	100	4	48
10	0	0	0

Duration	**10 min**		
Increment	**5 min**		
Intensity (mm/hr)	**40 mm/hr**		
Total rain	**6.7 mm**		
mins	**Proportion [%]**	**mm**	**mm/hr**
0	0	0	0
5	57	3.8	45.6
10	43	2.9	34.4
15	0	0	0

Table B-4 5 and 10 minute design events

Duration	15 min			Duration	20 min		
Increment	5 min			Increment	5 min		
Intensity (mm/hr)	35 mm/hr			Intensity (mm/hr)	31 m/hr		
Total rain	8.8 mm			Total rain	10.3 mm		
mins	Proportion [%]	mm	mm/hr	mins	Proportion [%]	mm	mm/hr
0	0	0	0	0	0	0	0
5	32	2.8	33.6	5	19	2.0	23.6
10	50	4.4	52.5	10	43	4.4	53.3
15	18	1.6	18.9	15	30	3.1	37.2
20	0	0	0	20	8	0.8	9.9
				25	0	0	0

Table B-5 15 and 20 min design events

C. Detention tanks

The effectiveness of the detention tanks is depending on the size of the orifice in the tank. An orifice that is too small would prevent the tank from emptying fast enough. This would result in a tank that is too full at the start of a major event. When the orifice is too big the outflow from tank would increase the peak flows and thus only have a negative influence. There for it is important to use the optimum orifice size. The best orifice size is determined by comparing the two year ARI flow rate of runoff from the allotment and street scale.

The two year runoff on the allotment scale is determined for tanks with orifice sizes varying from 0mm up to 300mm with increments of 20mm. In Figure 16 is shown what the two year ARI flow rate of runoff is for the various tank sizes with the various orifice sizes.

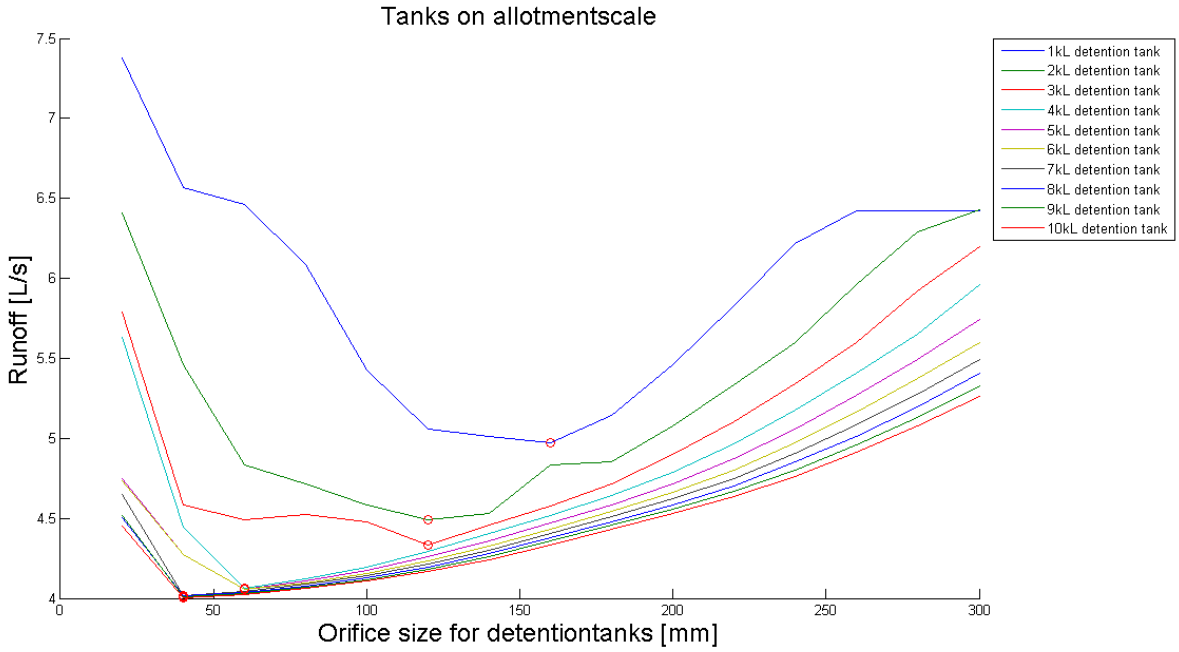

Figure 16 Runoff per orifice size per tank on allotment

For the lump sized tanks on the street scale the tested orifice sizes vary from 0mm to 1200mm with 50mm increments. The results are shown in Figure 17. In Table C-1 Optimal orifice sizes is a summary of the optimal orifice sizes.

Table C-1 Optimal orifice sizes

Tank size [kL]	1	2	3	4	5	6	7	8	9	10
Orifice size [mm]	160	120	120	60	60	60	40	40	40	40
Tank size [kL]	38	76	114	152	190	228	266	304	342	380
Orifice size [mm]	550	450	450	250	250	200	200	150	150	150

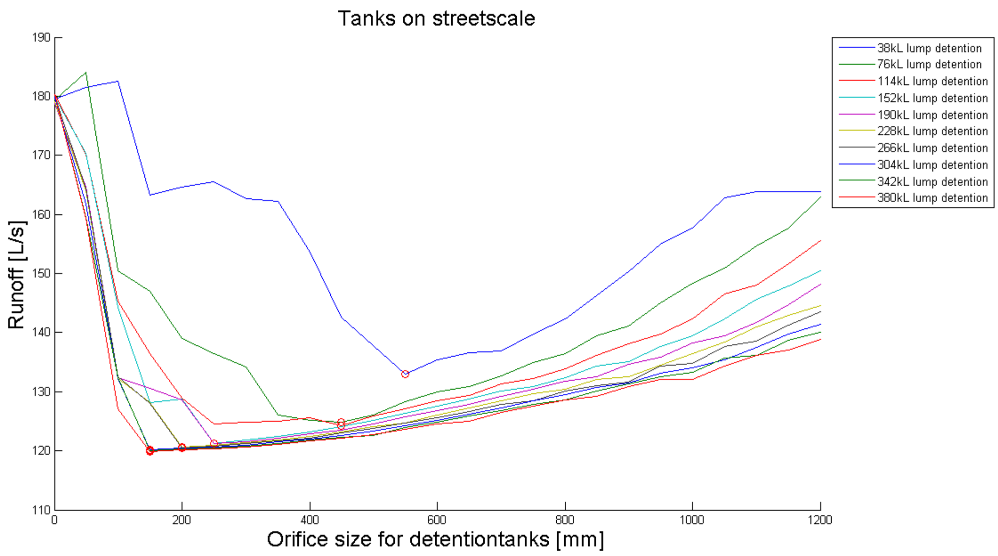

Figure 17 Runoff per orifice size per tank on street