Efficiency of urban water detention methods in downstream cities

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Delft University of Technology
Weicheng Chen

TU Delft

Content

Abstract 3
1 Introduction 3
2 Literature Review 5
2.1 Hydrologic background 5
2.1.1 Some definitions 5
2.1.2 Hydrologic processes and flow regime 5
2.2 Identifying the effects of urbanization on flow regime 5
2.2.1 Study methods 5
2.2.2 Impacts of urbanization on flow regime 6
2.3 Methods to limit the flood risks 7
3 Research Method 10
3.1 Tested Segment 11
3.2 Rainfall-Runoff relationship – Sacramento Model 13
3.2.1 Permeable area 14
3.2.2 Permanent impermeable area and temporary impermeable area 14
3.2.3 Actual Evapotranspiration 14
3.2.4 Percolation 15
3.2.5 Routing of surface runoff 15
3.3 The original Sacramento Node for the tested segment 16
3.4 Newly built model for the tested segment 17
4 Storm water detention scenarios 19
4.1 Green Roof 19
4.2 Household storage tank 20
4.3 Pervious Pavement 21
4.4 On-site detention tanks 22
5 Results 27
5.1 Segment Scale 27
5.1.1 Runoff contributed by the detention methods implemented area 27
5.1.2 Hydrograph of the segment 29
5.2 Catchment Scale 31
6 Discussions 32
7 Conclusions 33
7.1 Segment scale 33

7.2 Catchment scale 34
7.3 Answer to the research question 34
8 Recommendations 35
References 36

Abstract

Jakarta is a city which suffers from the problem of urban flooding and inundation. It is urgent to figure out solutions on how to manage the storm water in its urban area. Jakarta locates at the downstream of the catchment; the peak runoff generated in the city can occur earlier than the peak discharge coming from upstream, potentially leading to a 'collision' of both peaks. Since the storm water detention methods have both the runoff reduction and delay functions, whether adopting storm water detention methods in Jakarta is able to limiting the discharge magnitude because of its discharge reduction function or even increase the discharge because of the peak delay function is studied. In this article, there are 4 storm water detention methods, green roofs, household storage tanks, pervious pavements and on-site detention tanks, tested with a precipitation event with 100-year return period in a segment in the Jakarta City. The result of the example in this research shows that the delay functions of these 4 solutions are negligible, and the runoff reduction functions of all the solutions are almost lost in this extreme rainfall event when the discharge from the upstream reaches the peak value. So these 4 storm water detention methods in the example have no influence on the peak discharge in the downstream in this precipitation event. However, because the fast runoff is still contributing to the runoff at the exit of the segment when the peak discharge from the upstream comes, these storm water management methods are still possible to reduce the peak discharge during other precipitation events with smaller total precipitation amount or earlier peak intensity. An off-line detention tanks could have positive effects on reducing the peak discharge in the river to the downstream if they start to collect storm water certain period later so that they are capable to store the runoff when the peak discharge from the upstream comes.

1 Introduction

With the rapidly increased urbanization around the world, land use change of urbanization effects the natural environment in all aspects. Among them, the effect on the water cycle, like water quantity and quality, is widely recognized. [1] And there has been considerable researches concerning the hydrological consequences caused by altering the land use for urbanization. [2] [3] Because of the increasing fraction of impervious areas as well as the construction of the urban drainage systems, the discharge frequency and magnitude can be changed dramatically, and the total runoff can be increased while discharge peak can come earlier. Problems like inundation, channel erosion, ecological alteration might occur. [1] [4] Because of the increasing vulnerability of urban flooding and inundation and the climate change nowadays, measures are introduced to manage storm water in the urban areas these decades, such as green roofs and pervious pavements. These methods not only have the function of reducing the runoff but also to delay the peak. However, in the area downstream of an urban catchment, the earlier coming discharge peak may help to reduce the peak discharge through staggering the peak. [5] So whether adopting these storm water detention methods in a downstream city can limit the discharge magnitude because of its discharge reduction function or even increase the discharge because of the peak delay function is still questioned. In this article, a case study on the efficiency of the urban water detention methods is held in a downstream city, Jakarta.

Jakarta is the capital city of the Republic of Indonesia. It locates at the mouth of the Ciliwung River on Jakarta Bay, which is an inlet of the Java Sea [6].According to the Koppen climate classification system, Jakarta has both a borderline tropical monsoon climate and tropical rainforest climate [6]. The city has distinct wet and dry seasons. The wet season in Jakarta is from October to May (January is the wettest month); and the dry season is from June till December (August is the driest month) [7]. The monthly mean precipitation is shown in Figure 1.

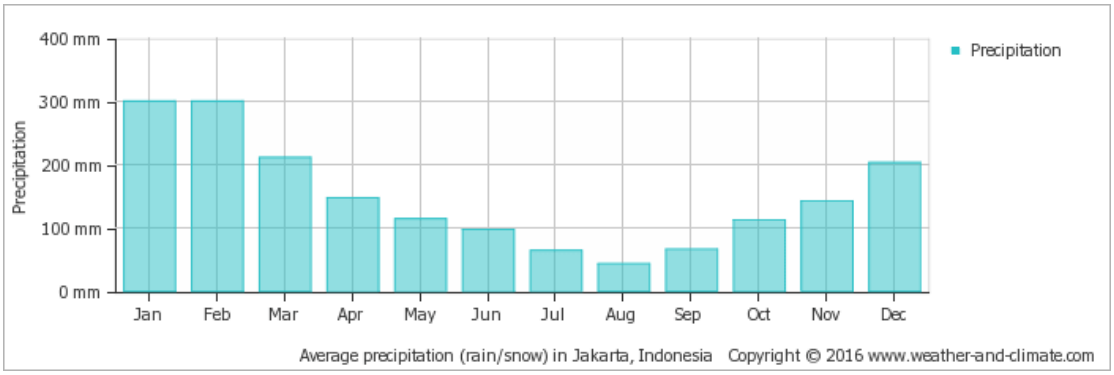


Figure 1 Monthly mean precipitation in Jakarta [7]

In last few decades, Jakarta has experienced rapid urbanization as well as a wide range of urban problems. Urban flooding is one of the major problems. The city lies in a lowland area with 13 rivers, whose tributaries are located in the peripheries of the megacity, strongly related to the flooding situation in Jakarta [8]. The land use change in Jakarta, especially in the peripheries of the megacity, increases the threat of the urban flooding [8]. This city, with 40% area already sitting below the sea level, is still sinking into the ground at an average of three inches every year. The rising sea level due to the global warming even makes the situation worse [9]. In 2007, one of the worst floods in memory inundated about 70% of Jakarta, killed at least 57 people and sent about 450,000 fleeing their houses [8]. In 2013, many parts of Jakarta were inundated following a heavy rainfall event, during which time the flood took away more than 20 people's lives and displaced at least 33,502 people [8]. And in 2016, around 10,538 houses in 8 sub-districts in South and East Jakarta were affected by an urban flooding [8].

Figure 2 The Flood in Jakarta in 2007 [8]

Facing this severe flooding situation, it is an urgent to study on how to limit the discharge in the rivers in Jakarta. In this article, the question, whether applying the storm water detention methods in Jakarta (a city locates at the downstream of the basin) can help to limit the discharge in the river during extreme precipitation event, is studied. To answer this question, the effects of land use change caused by urbanization on the discharge regime in the river are introduced; the hydrological behaviours of the storm water detention methods are studied; scenarios based on these methods are developed and simulated conceptually in the hydrological model.

2 Literature Review

2.1 Hydrologic background

2.1.1 Some definitions

A drainage basin is an area from which all the precipitation converges to a single exit point at a lower elevation and then enters another water body, like rivers, lakes. The discharge at the exit point of a drainage basin depends on hydrologic processes within the basin. The drainage basin contains a whole drainage system, all the elements of the landscape within the area through or over which water travels that influence the hydrologic processes [1]. These elements can be soil, rock, vegetation, or the stream channels. Affected by the human activities, elements like pipes, pavements and other impervious zones are added to the system while the vegetation and pervious area might be diminished, altering the hydrologic processes within the drainage basin.

To describe the alteration that urbanization made to the discharge in the exit of the drainage basin, the concept of flow regime should be introduced. Since the flow in a stream channel, varies among time scales like hours, days, seasons, years, and longer [10]. The flow regime is supposed to be able to fully characterize the timing, quantity as well as variability of the discharge in a certain water body. Proff et al. discussed the concept of "natural flow regime" of river systems, which described the river discharge with 5 components: magnitude of discharge, frequency of occurrence, duration, timing and rate of change [10].

2.1.2 Hydrologic processes and flow regime

There are varies hydrologic processes happening in a drainage basin, such as precipitation, evaporation, transpiration, infiltration and so on. All these processes can affect the runoff originating in the drainage basin. During a precipitation event, a certain amount of water enters in the drainage basin. Through interception, evaporation and transpiration, some water is "lost", while the remaining part (effective precipitation) is going to be drained through the exit of the drainage basin. Therefore, loss processes like these can influence the total amount of the runoff. The discharge at the exit of the basin generally consists of the surface water and groundwater. Both the surface runoff and ground runoff create the peaks of hydrograph together in a drainage basin. Since surface water travels a lot faster than groundwater, characteristics of the elements in the drainage basin like permeability are also able to influence the flow regime dramatically through affecting the ratio of surface runoff and underground runoff. In conclusion, variability in weather (timing, duration and intensity of a precipitation event) and in the properties of elements within a drainage basin (like topography, land use, soil, and vegetation) create a local flow regime together.

2.2 Identifying the effects of urbanization on flow regime

Urbanization can both affect the local climate (urban heat island) and drainage elements in the drainage basin (land use change, etc.). Also, urbanization may add "additional water" to the basin through drinking water network if the source of the drinking water system is not with the drainage basin. In this article, only effects of land use change are discussed.

2.2.1 Study methods

There are three main methods to study the effects of land use change on flow regime, with experimental catchment studies [11], with data analysis [2][3][4], and with model simulation [5][12][13].

Experimental catchment study is a method used in the early stage of hydrologic effects of land use change. The basic idea of this method is to compare the hydrologic indicators of situations which are with and without the land use change. The experiments can be held in two similar basins or a single basin. The limitation of this method is that the experiment areas are commonly relatively small (in most cases, less than 200 ha) [11], it is not suitable to simulate for a large catchment.

Data analysis method is commonly used before the 20th century. To study the effect of land use change with data analysis, the availability of long term records about both the land use pattern and the hydrologic pattern are required, while statistical methods are used here to derive some indicators for comparison. In urbanization studies, usually, the percentage of urbanized area is used to be the indicator of the land use pattern. And the most commonly used indicators are annually and seasonally low flows and high flows, as well as some other flow components (e.g. base flow index) [3]. Also, climate (precipitation) indicators like annual and dry-season precipitation in the same period may be needed to show the extent of the influence of climate change. However, Braud et al. [3] pointed out that these indicators are not sufficient enough to gain a full insight on the effects of urbanization of flow regime. Therefore, filtering techniques were proposed to filter the impact of sewer overflow devices and the infiltration into sewer networks. The weakness of data analysis method is obvious: it is only a statistical model. Discriminating between the impacts of land use changes and that of all other factors on hydrologic cycle is very difficult. Therefore, having a full evidence of the impact of urbanization on certain hydrological processes is nearly impossible.

The model simulation method is becoming more and more popular with the rise of computer technology and modelling techniques. Here, hydrological models are used to simulate the hydrologic processes within drainage basins. In the 1970s, Onstad et al. [14] was one of the first to try to use hydrological modelling to study the impact of land use change on discharge regime. Up to now, there have been several detailed hydrological models that are able to solve rainfall-runoff relationships for urbanized areas, such as SWMM, MIKE-SHE and HSPF. Some hydrological models are suitable to solve the rainfall-runoff simulation for one single precipitation event while others are suitable to simulate the hydrologic processes continuously [1]. Since all the hydrological models are set up according to certain physical bases, it is appropriate to use hydrological models to study the impact of land use change while eliminating influences from other factors. However, the hydrological models are not able to describe all the physical processes happening in the water cycle. And parameters are calibrated from the observation data. Therefore, the uncertainty, coming from the observation data, the imperfect conceptualization, and the parameters, limits the reliability of the results in the hydrological models. Besides, since there may be many feasible parameter sets, the processes of the water cycle can be described in a totally wrong way. This problem of equifinality in the hydrological model also can affect the reliability of the results.

2.2.2 Impacts of urbanization on flow regime

In urban areas, the land use change is mainly expressed as the diminishment of vegetation and the increase of impervious area. Also, the construction of urban drainage system accelerates the drainage in the urban areas. These changes potentially decrease the infiltration rate, the travel time of the water to the exit of the discharge basin, as well as the "water loss" through evaporation and transpiration, which may lead to changes in flow regime.

In numerous cases, the urbanization increases the flood risks. According to the study of Booth [1], in highly urbanized cases, not only the major flow peaks are amplified, but many new peaks also appear. White et al. [2] studied the impacts of watershed urbanization on the stream hydrology of Los Penaquitos Creek in California with data analysis method, where there is a significant increase in urbanization (from 9% to 37% during 1996 till 2000). The total runoff increases the peak discharge in the exit of the drainage basin as well as the flood magnitudes. In this case, the total runoff was increased by over 200% from 1973 to 2000, when there is no big change in precipitation statistically. Also in this period, the medium and minimum daily discharges are increased significantly, while there is only a slightly increasing trend of maximum daily discharge. This also leads to the change in flood magnitudes (Figure 3). As is shown in the figure, the flood magnitudes increased with the increasing of urbanization in the drainage basin. Also, the lower return interval is, the greater flood magnitude increases. Klöcking and Haberladt [12] modeled the increasing urbanization scenario with ArcEGMO. They found that although the increase of impervious area can decrease the infiltration in a precipitation

event then might reduce the groundwater recharge and stream base flow, the impact of urbanization on base flow differs from catchment to catchment, because of the lower evapotranspiration. In addition, a conclusion was drawn that growing total impervious area fraction cannot cause a proportional increase of basin discharge in general. However, in the research of Yang et al. [5], which simulated an urban area at different locations in White River Basin, it is found that urbanization does not necessarily increase the discharge magnitude at the exit of the drainage basin. The flood magnitude may even be lower than that before the development of urbanization. They also claimed that it is the travel time of water from the urban area to the exit of the basin that has the largest impact on flood peaks. The flood peaks at the exit of the basin are largest when the discharge peaks generated by the urban area and by the whole basin occurs at the same time. And the flood peaks tend to be smaller and comes earlier when the urbanization is developed near the exit of the drainage basin.

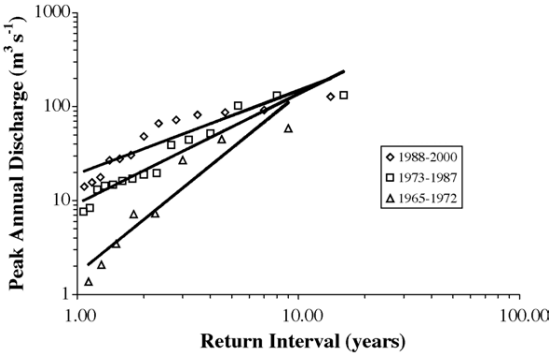


Figure 3 Flood frequencies during three time intervals according to White el al. [2]

2.3 Methods to limit the flood risks

To limit the increase of flood risks, up to now, numerous storm water managing solutions to detain the storm water in the urban area are applied. They are mainly managing the storm water in two ways: increasing the storage capacity for the storm water or increasing the infiltration. The storage can be increased both on the ground surface (like garden ponds) and in the underground (like underground detention tanks), both at a certain site and within a certain region, and both online (within a storm water conveyance system) and offline (diverted with the storm water conveyance system) [18]. Increasing the infiltration is able to prolong the detention time of the storm water through reducing the percentage of surface runoff while producing a larger proportion of groundwater runoff, or temporarily detaining the runoff with permeable materials. For example, installing pervious pavement, decreasing effective impervious area (EIA, the impervious area where the generated runoff is drained via urban stormwater drainage systems [5]).

In the following, four typical storm water managing measures detaining storm water in different ways, like increasing the interception, enhancing the storage capacity (by retaining the water for household use or detaining the water in the urban drainage system), and detaining the storm water through increasing infiltration, are introduced:

- Green roofs

  Green roof (Figure 4) is commonly a multiple-layer storm water detention system [16]. The common construction of layers of green roofs is shown in Figure 5. With the vegetation planted on the roofs, the ‘initial loss” is increased because of larger interception and some precipitation can be detained in the soil layer for a while. As a result, for a certain rainfall event, the discharge volume can be reduced and the peak discharge may also be attenuated.

Figure 4 The green roof (source: http://consciouslivingtv.com/home-garden/what-you-didnt-know-about-green-roofs.html)

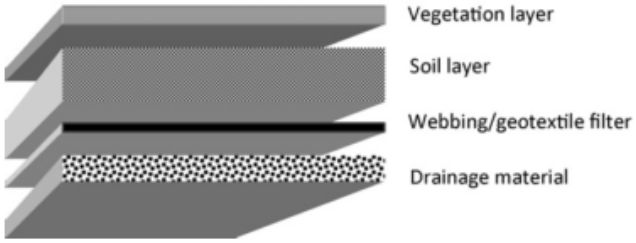


Figure 5 Commonly construction layers of green roofs [16]

- Rainwater harvesting systems

  As shown in Figure 6, it is a very typical rainwater harvesting system. It collects and stores the runoff from the roof. And then the water can be transported to the buildings by a pump for daily use when needed. Besides utilizing rainwater to reduce the potential pressure of water supply, during a rainfall event, the sum of the remained storage capacity of the tanks can also be expressed as a part of "equivalent initial loss" of the precipitation which helps to reduce the total volume of runoff at the exit of drainage basin [17].

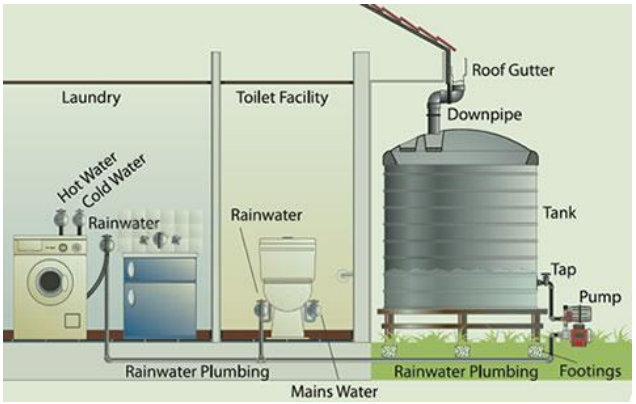
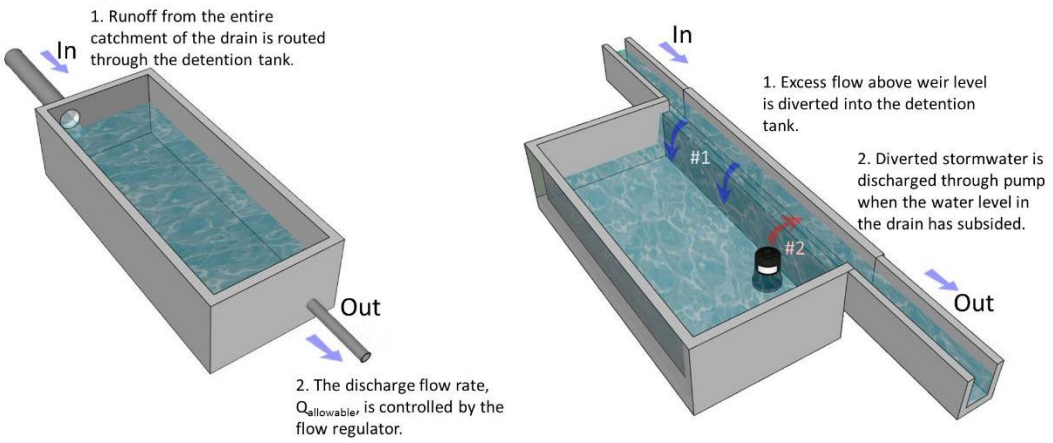


Figure 6 Rainwater harvesting system (source: http://www.rainharvest.co.za/2013/08/preparing-your-site-for-rainwater-tank-installation/)

- Storm water detention tanks

  Storm water detention tanks are functioned to store storm water during a rainfall event, and then release the water at a controlled rate. These tanks can be installed online or offline (Figure 7). Therefore, with these storm water detention tanks, the drainage rate of this site can be slowed down and the peak drainage rate can be attenuated [18].

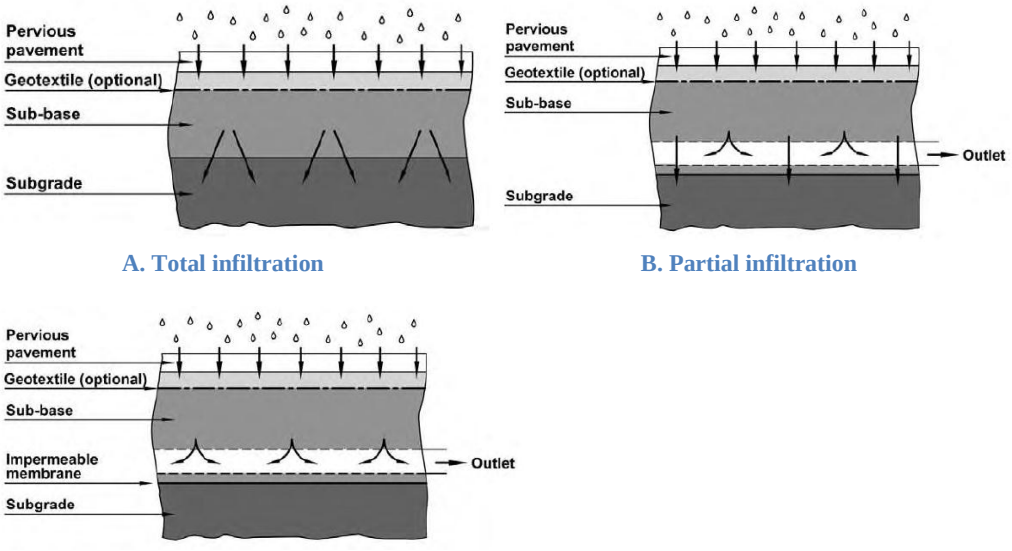


A. Online **B. Offline**

Figure 7 Online and Offline detention tanks [18]

- Permeable pavements

  Permeable pavements allow rainwater to infiltrate through the surface material. The water can be temporarily detained in the underlying layers before being infiltrated to the ground or discharged away [15]. The typical structures of this system are shown in Figure 8. As shown in the figure, for Type A, all the rainwater that infiltrates through the surface is going to infiltrate further into the soil. For Type B, part of the rainwater going through the surface material is going to be drained away, while all the rainwater are going to be drained away for Type C. With the help of permeable layers, the proportion of surface runoff can be reduced, and the detention function of the sub-surface material may be able to attenuate the peak discharge rate.



A. Total infiltration **B. Partial infiltration**

C. No infiltration

Figure 8 Typical structures of permeable pavements [15]

3 Research Method

To investigate whether implementing the storm water detention methods has positive effects or negative ones on the peak discharges of the river, the storm water detention methods introduced above is simulated and studied with an existing model from Jakarta (Figure 9).

This model is a 1D model developed based on SOBEK Rural. It is formed of two modules, RR (Rainfall-Runoff) Module and 1D Flow Model. The sketch of the simplified structure of the model is shown in Figure 10. The whole catchment consists of 494 segments. In each segment, the rainfall-runoff relationship was simulated using RR Module. Here in this existing model, Sacramento Nodes in the RR Module is used. Then the output of the RR Module is transferred to an input of 1D Flow Model with the help of links between these two modules to calculate the discharge as well as the stage in the stream channels at every time step.

In this article, the efficiency of the storm water detention methods is going to be translated into the RR Module and studied in two scales, segment scale and catchment scale. Firstly, a typical urbanized segment in the city is chosen to "adopt" the storm water detention methods one by one, to examine their behaviour and efficiency on the peak flow reduction and delay. Then the hydrograph of the segment is compared with discharge from just upstream of the segments to see whether the implementation of the detention methods in the downstream has positive influences or not. To simplify the simulation, only the Sacramento Node in the tested segment is extracted from the existing large model for the simulation of the rainfall-runoff relationship.

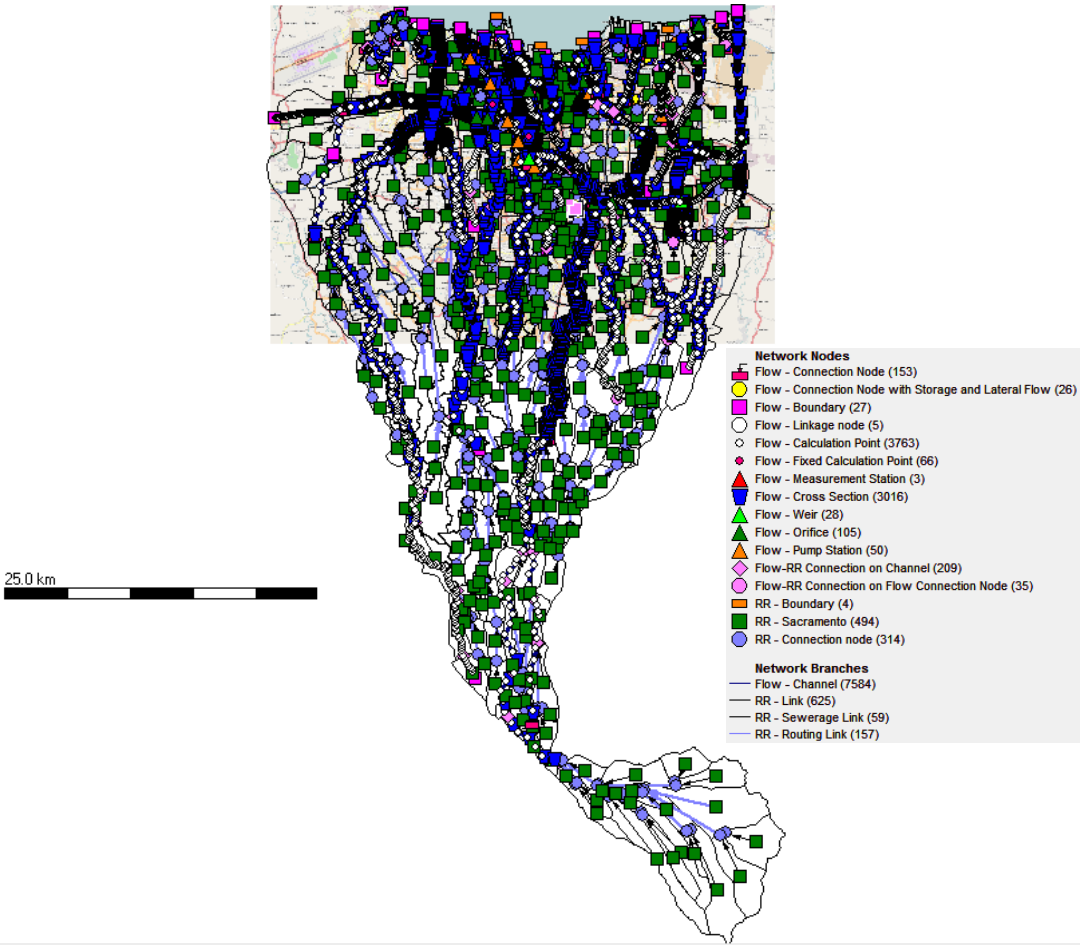


Figure 9 The Jakarta model

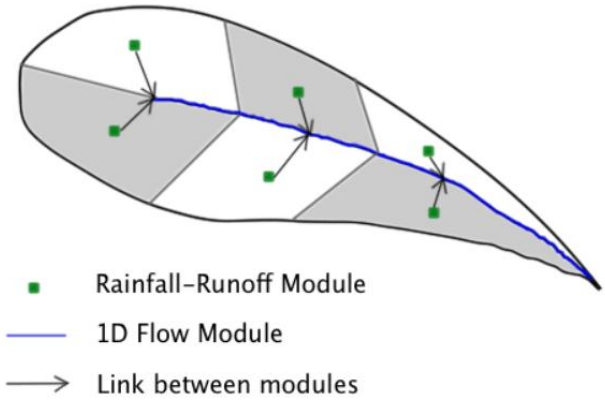


Figure 10 Simplified structure of the Jakarta model

3.1 Tested Segment

To choose a representative segment, the criteria applied is as follows: 1. It should be near the mainstream to ensure that the peak of the runoff from the segment and discharge form upstream is staggered enough. 2. It should be densely residential area so that there would be a large proportion of roof areas of residential houses and potential availability for installing household storage tanks which will be simulated as a scenario. In this article, a segment located in the Jakarta City is chosen (Figure 11). The total area of the segment is around 292 ha, and the satellite photo and the map of this segment are shown in Figure 12. A typical street view in this segment is shown in Figure 13.

Figure 11 Location of the tested segment (Source: Google Earth)

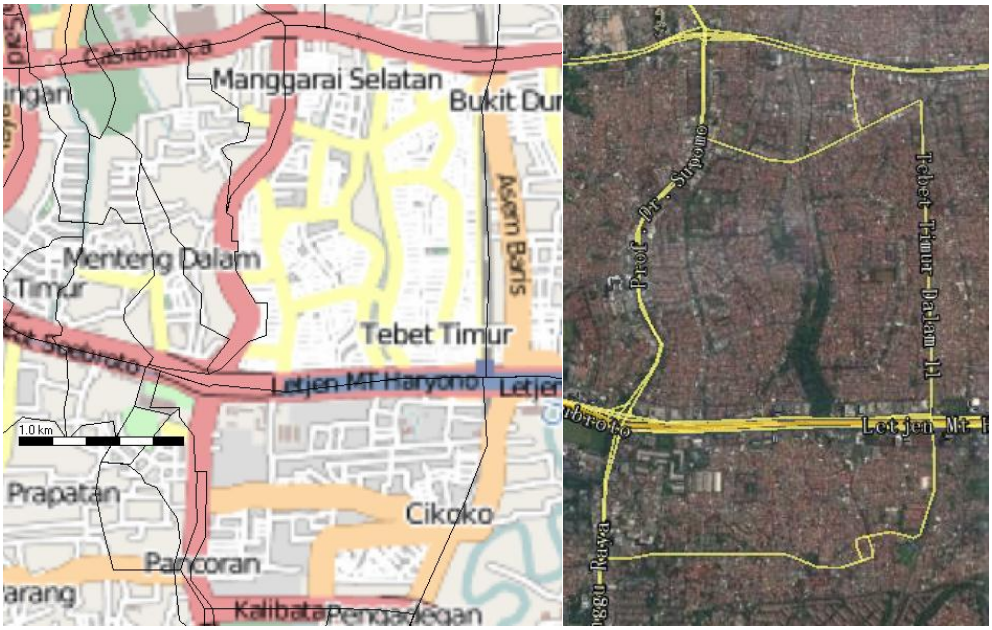


Figure 12 Map and the satellite photo of the segment (Source: Google Earth)

Figure 13 A street view in the tested segment (Address: Tebet Timur, Source: Google Earth)

3.2 Rainfall-Runoff relationship – Sacramento Model

Before translating the storm water detention method into the rainfall-runoff model, the conceptualization and the algorithm of the model used in the RR Module, Sacramento Model, should be introduced here. The Sacramento Model is a conceptual model introduced by Burnash in 1973 [19]. This model is originally used to simulate the hydrological processes (mainly the land-phase) in Sacramento River Basin, USA [20] (Figure 14). The conceptual structure of this model is shown in Figure 15. The ground surface consists of three parts, permeable area, permanent impermeable area and temporary impermeable area. In the permeable area, the soil was divided into the upper zone and the lower zone.

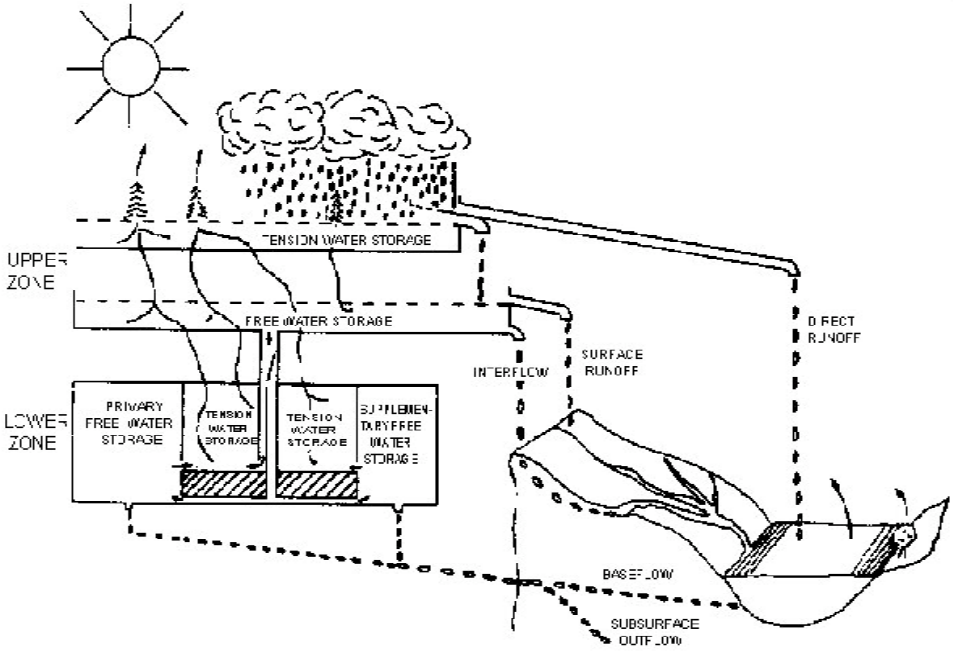


Figure 14 Sacramento Model conceptualization of the rainfall-runoff process in a segment [20]

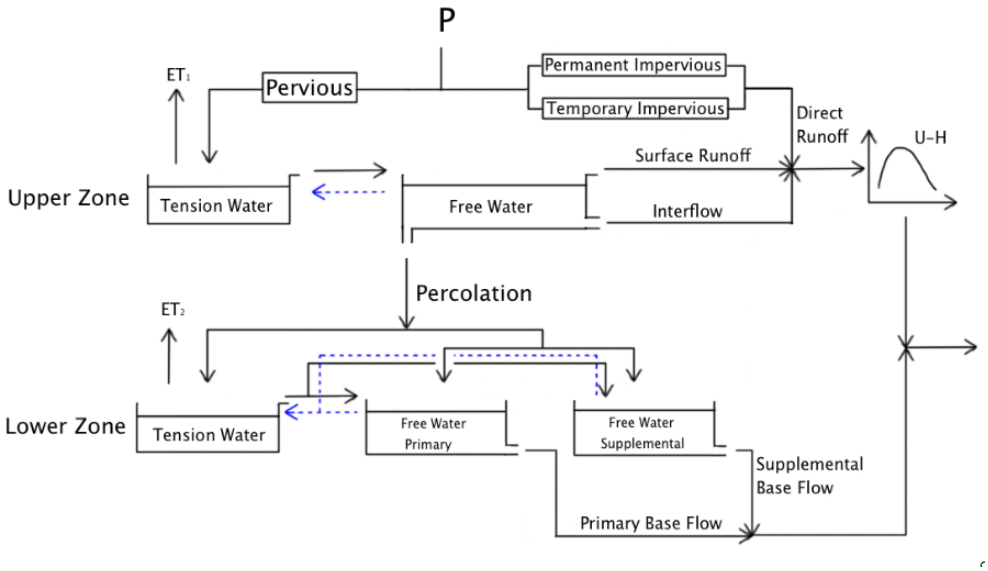


Figure 15 Conceptual structure of Sacramento Model used in SOBEK RURAL

3.2.1 Permeable area

3.2.1.1 Upper zone

In this conceptual structure of the model, the upper zone soil is divided into two reservoirs in the upper zone: tension water reservoir and free water reservoir. The storage capacity of the tension water reservoir is determined as a function of the difference between the field capacity and wilting point of the soil in the upper zone, while the storage capacity of the free water reservoir is determined by the difference between the porosity and field capacity [21]. During a precipitation event, when the storage capacity of the upper zone tension water reservoir (UZTWM) is exceeded, the excess part of rainwater will enter the free water reservoir. The upper zone free water reservoir is where the percolation, interflow and surface runoff are generated. The preferred flow direction of this free water reservoir is vertical (percolation) [20]. When the precipitation rate is higher than the percolation capacity, the interflow will be generated regarding the free water reservoir as a linear reservoir element with a depletion coefficient:

$$Q_{interflow} = UZFWC \times UZK \tag{3.1}$$

where, UZFWC is the current storage in the upper zone free water reservoir, and UZK is the depletion coefficient. Once the upper zone free water reservoir is filled, the excess part will become surface runoff with no delay.

3.2.1.2 Lower zone

As shown in Figure 15, the lower zone consists of one tension water reservoir and two free water reservoirs. Same as the upper zone, the tension water reservoir represents the part of the water that can be held by the soil while the free water reservoirs represent two temporary linear storage reservoirs with two different depletion coefficients. A certain fraction of the percolation (1-PFREE) from the upper zone goes to the tension water reservoir while the other part goes to the two free water reservoirs. When the storage capacity of the lower zone tension water reservoir (LZTWM) is exceeded, the excess part will enter free water reservoirs in the lower zone. The primary and supplemental free water reservoirs are elements that lead to slower base flow and faster base flow, respectively.

3.2.2 Permanent impermeable area and temporary impermeable area

The permanent Impermeable area is used to represent the area in the segment with neither tension water storage nor temporary storage reservoirs. The direct runoff is generated immediately in this area with no "initial loss". And the temporary impermeable area represents the area where only the tension water reservoirs are taken into consideration. The runoff starts to be generated when the upper zone tension water reservoir is full:

$$Q_{temp.imp} = PAV \times \left(\frac{ADIMC - UZTWM}{LZTWM} \right)^2 \tag{3.2}$$

where, $Q_{tem.imp}$ is the runoff generated from the temporary impervious area, PAV stands for the effective precipitation ($PAV = \max(0, P + UZTWC - UZTWM)$), ADIMC is the current tension water storage in the temporary impermeable area[20]. And when all requirements of the tension water reservoirs are met, the temporary impermeable area starts to generate runoff with no delay.

3.2.3 Actual Evapotranspiration

The evaporation rate from the open water equals to the potential rate. The evapotranspiration happens at the two tension water reservoirs. The evapotranspiration rate is determined by the relative water contents of the both tension water reservoirs [20]. During a dry period when the ratio of the content to its storage capacity of free water reservoir exceeds that of the tension water reservoir in the same zone, the water will be transferred from free water reservoir to that tension water reservoir [20]. However, a certain proportion (RSERV) of the storage capacity of the lower zone free water reservoirs is unavailable for transpiration purposes [20].

3.2.4 Percolation

The percolation rate depends on both the percolation demand of the lower zone and the water content in the upper zone free water reservoir relative to its storm water capacity [20]:

$$PERC = PERC_{act.dem} \times \frac{UZTWC}{UZTWM} \tag{3.3}$$

where, PERC stands for the actual percolation from the upper zone to the lower zone, $PERC_{act,dem}$ is the actual percolation demand of the lower zone. And the actual percolation demand depends on water content in the lower zone relative to its capacity [20]:

$$PERC_{\min.dem} = PBASE = LZFPM \times LZPK + LZFSM \times LZSK \tag{3.4}$$

$$PERC_{\max.dem} = PBASE(1 + ZPERC) \tag{3.5}$$

$$G = \left(\frac{(LZTWM - LZTWC) + (LZFPM - LZFPC) + (LZFSM - LZFSC)}{LZTWM + LZFPM + LZFSM} \right)^{REXP} \tag{3.6}$$

$$PERC_{act.dem} = PBASE(1 + ZPERC \times G) \tag{3.7}$$

where, $PERC_{min.dem}$ and $PERC_{max.dem}$ are the minimum and maximum percolation demand of the lower zone, respectively. PBASE is the sum of the primary and supplementary base flow. LZPK and LZSK are the depletion coefficients of the primary and supplementary free water reservoirs. ZPERC is a coefficient stands for the proportional increase in percolation from saturated to dry conditions in the lower zone. Computationally, when calculating the actual percolation, ZPERC should be multiplied by the function G which relates to the relative soil humidity in the lower zone (when G=1, the soil in the lower zone is completely dry; when G=0, the soil in the lower zone is completely saturated) [20]. LZTWM, LZFPM, and LZFSM are the storage capacity of the tension water reservoir, primary free water reservoir, and supplementary free water reservoir in the lower zone, respectively, while LZTWC, LZFPC, and LZFSC are their water content.

3.2.5 Routing of surface runoff

To calculate the discharge distribution of surface runoff, interflow, and from at every time step at the exit of the segment, they are transformed according to a unit hydrograph. A unit hydrograph is a runoff hydrograph resulting from one unit of constant intensity uniform effective rainfall occurring over the entire segment. [22] Usually, in rural area, it is influenced by variables such as the shape and size of the segment, the slope of the segment, the distribution of storage of some elements like channels as well as their storage capacity, etc. [23]. Then the discharge at the exit of the segment is calculated after adding up with the base flow.

The parameters of Sacramento Model can generally be divided into four groups. The summation the parameters are shown in Table 1.

Table 1 Parameters of Sacramento Node used in SOBEK Rural [20]

Land use		
Parameter	**Physical meaning**	**Unit**
Surface area	The surface area of the segment	ha/m2
PCTIM	Fraction of permanent impervious area contiguous with stream channels	-
ADIMP	Fraction of additional impervious area when all tension water requirements are met	-
SARVA	Fraction of areas covered by streams and channels	-
Storage capacity of soil		

Parameter	Physical meaning	Unit
UZTWM	Storage capacity of upper zone tension water reservoir	mm
UZFWM	Storage capacity of upper zone free water reservoir	mm
LZTWM	Storage capacity of lower zone tension water reservoir	mm
LZFPM	Storage capacity of lower zone primary free water reservoir	mm
LZFSM	Storage capacity of lower zone supplementary free water reservoir	mm
Depletion coefficient		
Parameter	**Physical meaning**	**Unit**
UZK	Depletion coefficient of upper zone free water reservoir	/day
LZPK	Depletion coefficient of lower zone primary free water reservoir	/day
LZSK	Depletion coefficient of lower zone supplementary free water reservoir	/day
Percolation		
Parameter	**Physical meaning**	**Unit**
ZPERC	Proportional increase in percolation from saturated to dry conditions in lower zone	-
REXP	Exponent in percolation equation	-
PFREE	Fraction of percolated water, which drains directly to lower zone free water storages	-
Other		
Parameter	**Physical meaning**	**unit**
RESERV	Fraction of lower zone free water storage capacity which is unavailable for transpiration	-
SIDE	Fraction of base flow not observed in the streams and channels	-
SSOUT	Subsurface outflow	mm/Δt
Unit hydrograph	runoff hydrograph resulting from one unit of constant intensity uniform effective rainfall occurring over the entire segment	/Δt

3.3 The original Sacramento Node for the tested segment

Since every scenario in the tested segment is going to be simulated one by one, to simplify the model, only the Sacramento Node of this segment is extracted from the existing large model for the simulation of the rainfall-runoff relationship. The values of the parameters in the land use group of the Sacramento Nodes of the tested segments are listed in Table 2.

Table 2 The values of the parameters in the land use group of the Sacramento Node

Parameter	Value
Area(ha)	292.06
PCTIM	0.02
ADIMP	0.61
SARVA	0.01

As shown in the table above, the fraction of the permanent impervious area (PCTIM) is too low here to represent an urbanized area, where there is a large proportion of impervious surfaces like roofs and pavements. Also, it is found that this value also equals to those of the Sacramento Nodes in the very upstream where there is rarely any urbanization. The only parameter from the land use group differs obviously from segment to segment is ADIMP, the fraction of the temporary impervious area. In the whole model, generally speaking, the value of ADIMP of the segment changes from 0.05 (very upstream) to 0.61 (downstream, urbanized area) while the value of PCTIM stays at 0.02. It is mentioned above that the temporary impervious area stands for areas where only tension water storages are taken into consideration. And the temporary impervious area shares the same tension water storage capacities with the pervious area. It can be reasonable to model the tension water storage for the impervious areas like roofs and pavements as their "initial loss". However, the storage capacities for these elements are much less than that of the pervious area, so it is more suitable to model the roofs and impervious pavement as permanent impervious area. In conclusion, the values of parameters in the land use group of the original Sacramento Nodes are not suitable to simulate the rainfall-runoff relationship for an urbanized area.

In addition, the Sacramento Model is not a good option for simulate the rainfall-runoff relationships for an urban area. Urbanization in the segment made the hydrological processes much more complicated and some hydrologic processes cannot be included in the Sacramento Model. For example, the construction of the drinking water supply system may influence the water balance in the segment. Also, the interaction between the water on the ground surface in the urban area and in the urban drainage system is not able to be included in this model. Since the generated runoff in the urban area starts as overland flow on the surface before entering the urban drainage system, if the intake capacity of the drainage system is not big enough, only a limited part of the water can enter the drainage system, while the other part of water will be remained on the surface or enters drainage system from another inlet. This process can influence the routing of the runoff at the exit of the segment. Furthermore, water may escape from the urban drainage system when the storage capacity of the drainage system is excessed. This phenomenon can also influence the routing of the runoff.

3.4 Newly built model for the tested segment

In this article, the Sacramento Node for the tested segment is replaced by a combination of two Sacramento Nodes (Figure 16), one representing the effective impervious area (roofs and pavements) from which the storm water is drained through the urban drainage system and another representing other areas. Because of the limitation of the Sacramento Node and lacking of the information about the urban water system, the rainfall-runoff relationship is still simulated in a simplified way based on a series of assumptions. Firstly, the effective impervious areas of roof and pavements are assumed to be 35% and 16% of the segment, respectively. These parts of areas are extracted from the temporary impervious area from the original Sacramento Node while the percentage of the pervious area stays the same. The values of the parameters in the land use group of the new model are listed in Table 3. Also, though it is not wise to use the same unit-hydrograph in the Sacramento Node representing the effective impervious area with that for the unpaved areas, because the runoff is drained faster in a sewerage system. Since the structure of the urban drainage system is unknown, and there is no data available for calibration for the rainfall-runoff relationship in a single segment. So the unit-hydrograph of the Sacramento Node from the original model is kept in both the two Sacramento Nodes of the new model. In addition, the influences of the drinking water supply system as well as the interaction between the water on the surface and in the urban drainage system are still neglected.

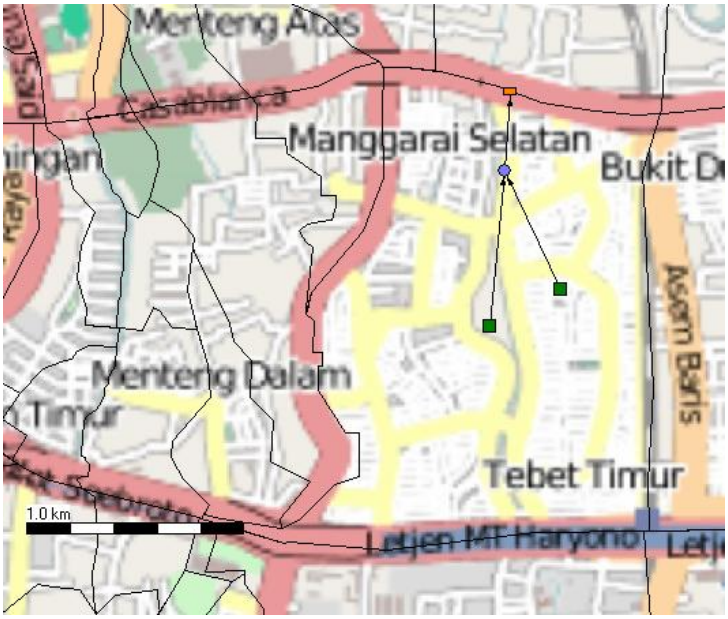


Figure 16 The new model for the tested segment

Table 3 Values of the parameters in the new model

Land use		
Parameter	Effective impervious area	other
Area(ha)	148.95	143.11
PCTIM	1.00	0.04
ADIMP	0.00	0.20
SARVA	0.00	0.20
Storage capacity of soil		
Parameter	0.00	other
UZTWM(mm)	0.00	50
UZFWM(mm)	0.00	150
LZTWM(mm)	0.00	150
LZFPM(mm)	0.00	200
LZFSM(mm)	0.00	150
Depletion coefficient		
Parameter	**Physical meaning**	**Unit**
UZK(1/d)	0.00	0.5
LZPK(1/d)	0.00	0.055
LZSK(1/d)	0.00	0.003
Percolation		
Parameter	**Physical meaning**	**Unit**
ZPERC	0.00	75
REXP	0.00	1.5
PFREE	0.00	0.4

4 Storm water detention scenarios

In this article, there are totally five scenarios built for the tested segments of the Jakarta Model:

- **Scenario 1: Null** (No storm water detention method is implicated in the segment)
- **Scenario 2: Green roof**
- **Scenario 3: Household storage tank**
- **Scenario 4: Pervious Pavement**
- **Scenario 5: On-site detention tanks**

To study the efficiency of these methods, it is assumed that the storm water detention methods are implemented in approximately 30% of the segment area (for the early implementation stage), which means the hydrological processes as well as the rainfall-runoff relationship of around 30% of the roof or/and impervious pavement areas are going to be changed. To simplify the simulation, it is also assumed that these areas are evenly distributed in the segments. Since all the scenarios are simulated with the help of SOBEK Rural, another Sacramento Node is created to represent the area where these methods are applied (Figure 17). And all the area of this new Sacramento Node is extracted from the Sacramento Node representing roofs and impervious pavements. The details of the scenario and the simulation method for each storm water detention method using the Sacramento Node are explained in the following.

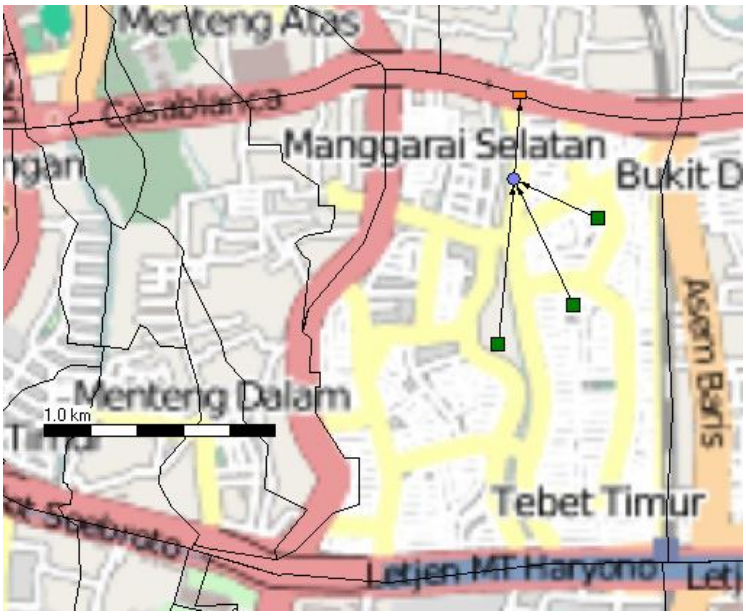


Figure 17 Simulation of the scenarios

4.1 Green Roof

In this scenario, there is 30 ha of roof area in the segment that is installed with green roof. It is introduced above that the green roof contains a vegetated soil layer. Apart from the runoff reduction function because of the increase of interception and the water storage capacity, the green roof may also have the runoff delay function. However, a research of Van Spengen shows that when the water storage capacity of the green roof is filled, the runoff generation of the green roof is similar to the normal roof, and the delay function of the green roof is negligible (Figure 18) [24]. As a result, when creating the Sacramento Nodes for the green roof, only the storage capacity of the soil layer as well as the evapotranspiration from the roof are going to be considered. Additionally, the thickness of the soil layer of green roofs is so small that there is no need to be divided into two zones. So the green roof area was simulated as a pervious area which has only a single tension water reservoir. The value of the parameter set for green roofs is shown in Table 4. The fractions of permanent and temporary impervious area as well as the channels (PCTIM, ADIMP & SARVA) are all 0, because all the green roof areas are set to be a pervious area. The storage capacity of the upper zone tension water reservoir (UZTWM) is set to 20 mm, an empirical data of storm water detention capacity (storage capacity) of

green roof with 80 mm substrate according to Stovin [25]. And the storage capacities of the upper zone free water reservoir and lower zone tension water reservoir (UZFWM &LZTWM) are set to be extremely small numbers since they cannot be 0 in Sacramento Nodes. The depletion coefficient of the upper zone free water reservoir is 1 so that there is no delay in this temporary storage reservoir (Equation 3.1). The depletion coefficients of the lower zone free water zones (LZPK & LZSK) are set to 0 to avoid base flow and percolation (Equation 3.3-3.7).

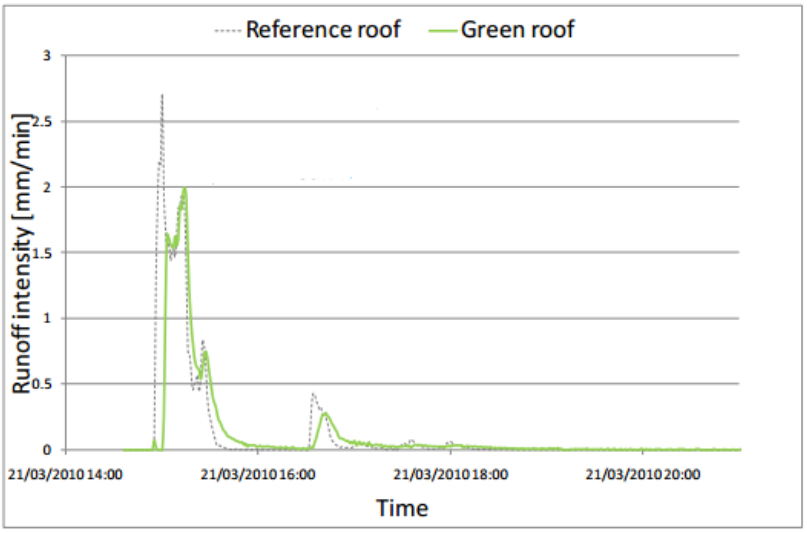


Figure 18 Behaviour of green roof [24]

Table 4 Parameter set for green roofs

Parameter	Value
PCTIM	0
ADIMP	0
SARVA	0
UZTWM	20
UZTWC	5
UZFWM	10^{-10}
UZFWC	0
UZK	1
LZTWM	10^{-10}
LZTWC	0
LZPK	0
LZSK	0

4.2 Household storage tank

This storm water retention method is also applied in the areas with 30 ha of roof area in total. According to the satellite picture from Google Earth, the average area of the roof in this segment is approximately 100 m^2. As a result, the household storage tank is installed in 3,000 houses. As for the size of the tanks, various sizes of household storage tanks can be chosen based on elements like local climate, roof areas, public health requirements, etc. [26] [27]. Because it is mentioned above that Jakarta is a humid area, a relatively large size, 8m^3, is chosen to be the average storage capacity of the tanks.

According to Matthew, the household storage tank plays a role in increasing the "equivalent initial loss" because of the increased available system storage capacity [17]. In the conceptual simulation of the household storage tank, the roof area of the houses with household storage tank is modelled to be a pervious area with only an upper zone free water reservoir with no interflow (UZK = 0) but no lower zone. The storage capacity of the reservoir is determined by the roof area as well as the volume of the

tank. As assumed above, the average roof area of a residential house is 100 m^2, and the average size of the household storage tanks is 8 m^3. Then the storage capacity of the reservoir can be calculated:

$$d = \frac{8m^3}{100m^2} * 1000mm/m = 80mm \tag{4.1}$$

In the meantime, it is assumed that there is in average 0.5 m^3 of water rest in the tank at the beginning of the precipitation event. Then the water content of the reservoir at the beginning is 5 mm. The value of the parameter set for household storage tank is shown in Table 5. The storage capacity of the upper zone tension water reservoir is assumed to be a very small number because the evaporation loss from the roof is neglected here like that from other effective impervious area. Furthermore, because the household storage reservoir is also a part of the rainwater harvesting system, the water in the storage tank can also be used and goes to the drainage system. And the yield for residential consumption from the storage tank should be calculated with the estimation of the residential consumption over time, the water storage in the tank as well as the runoff from roofs. According to Latham [28], the relationship between these elements can be written as $Y_t = \min(D_t, V_{t-1} + \theta Q_t)$, where Y_t, D_t, Q_t are the yield from the storage tank, the residential water demand and the total inflow at time interval t, respectively. And V_{t-1} is the water storage in the tank at time interval t-1. However, information like population data for each segment is unknown so that the model cannot be built based on these details. Therefore, efficiency (90%) is applied to represent the used water which goes to the drainage system directly, as well as a part of the runoff, which is not successfully collected by the system from the roofs. So the PCTIM is set to be 0.1 here. The depletion coefficient of the upper zone free water reservoir is 0 so that there will be no interflow. When the reservoir is filled, the runoff will be generated with no delay.

Table 5 Parameter set for household storage tank

Parameter	Value
PCTIM	0.1
ADIMP	0
SARVA	0
UZTWM	1
UZTWC	0
UZFWM	80
UZFWC	5
UZK	0
LZTWM	10^{-10}
LZTWC	0
LZPK	0
LZSK	0

4.3 Pervious Pavement

14 ha of impervious pavements are changed into pervious ones in this scenario. There are many factors involved in the hydrological processes in pervious pavement areas, such as the material (permeability, clogging situation), storage capacity, drainage method, etc. [29]. In this article, the runoff generation processes in pervious pavement area are simulated focusing on the changing infiltration capacity. According to Green-Ampt Infiltration Model [30], the infiltration capacity can be calculated with the equation:

$$f = K_0 \left[1 + \frac{(h_e + H)(\theta_s - \theta_i)}{F} \right] \tag{4.2}$$

where, f is the infiltration capacity at time t, K_0 is hydraulic conductivity, h_e is effective suction head, H is surface water depth, θ_s is volumetric soil moisture content, θ_i represents initial volumetric soil

moisture content, and F is the net change in total soil moisture above the moving wetting front. This equation indicates that during a rainfall event, the infiltration capacity will decrease with the increase of the total net infiltration and approaches a final infiltration capacity. And if the rainfall intensity exceeds the infiltration capacity of the pervious material, then the excessed part of the rainfall becomes Hortonian overland flow.

These processes are modelled with Sacramento Nodes in SOBEK Rural regarding the pervious pavements as temporary impervious areas (ADIMP = 1). As explain above, the temporary impervious area is conceptually simulated as an area with only tension water reservoirs. Based on Equation 3.2, the part $\left(\frac{ADIMC - UZTWM}{LZTWM}\right)^2$ can be seen as a "runoff coefficient", a dimensionless coefficient relating the amount of generated runoff to the amount of received rainfall [31] (See Figure 19). This runoff coefficient increases during a rainfall event till the tension water reservoirs are filled, which is similar to the Hortonian overland flow generation process. Because of the lack of information about the behaviour of the pervious pavement, the storage capacity as well as the division of the upper zone and lower zone kept unknown. Since the storage capacities of both the upper zone and lower zone tension water reservoirs in the original model (50 mm and 100 mm, respectively) are acceptable to represent pervious pavement, these numbers are also used in the Sacramento Node of the pervious pavements.

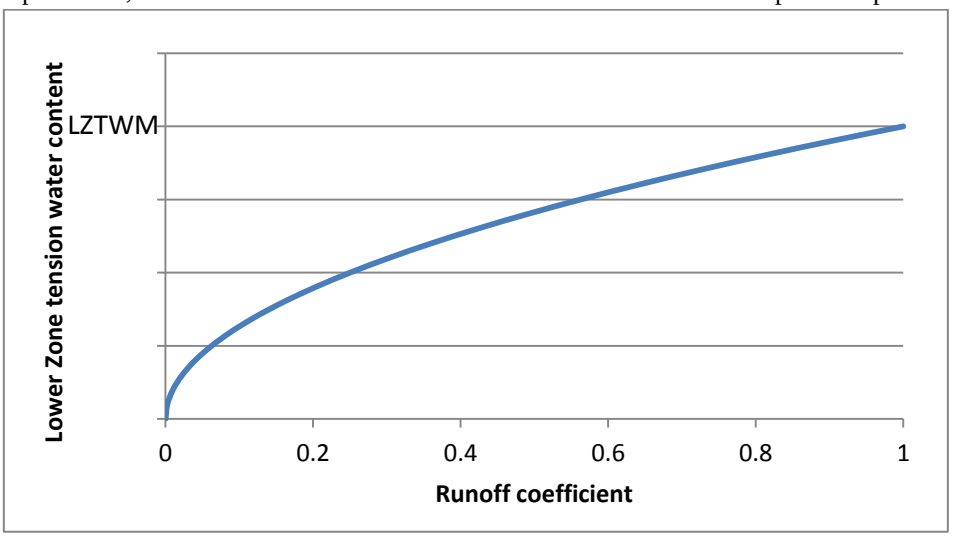


Figure 19 Changing "Runoff Coefficient" in temporary impervious area

4.4 On-site detention tanks

The detention tanks collect and store the storm water runoff from roofs, pavements or other impervious surfaces during a precipitation event. The collected water will be released at a controlled rate into the downstream drainage system so that the peak discharge can be reduced. An example is shown in Figure 20. In this article, 88 detention tanks are assumed to be evenly distributed in the segment, collecting storm water from 44 ha of effective impervious area (one detention tank every 0.5 ha of impervious surfaces on average). The storage capacity of the detention tank is usually designed based on the long term records of precipitation data, designed discharge rate as well as the designed return period. For example, drawing storage-discharge capacity-frequency curve (SDF curve) (Figure 21) could be helpful [32]. Alternatively, the storage of the on-site detention tank can be designed using intensity-duration-frequency curve (IDF curve) according to the technical guides published by Public Utilities Board of Singapore National Water Agency [18].

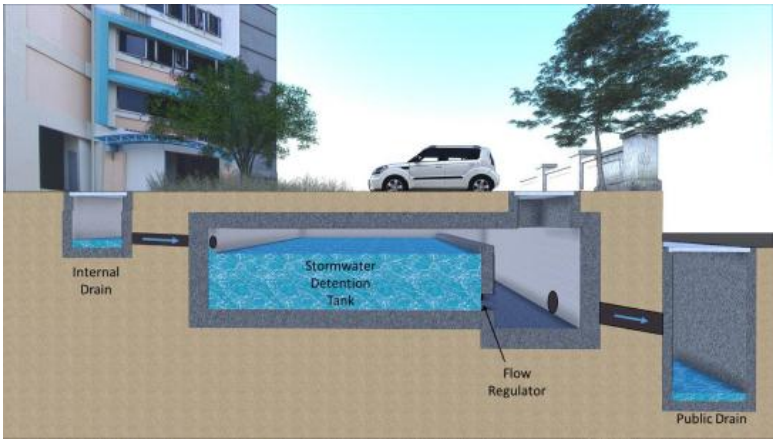


Figure 20 An example of the on-site storm water detention tank [18]

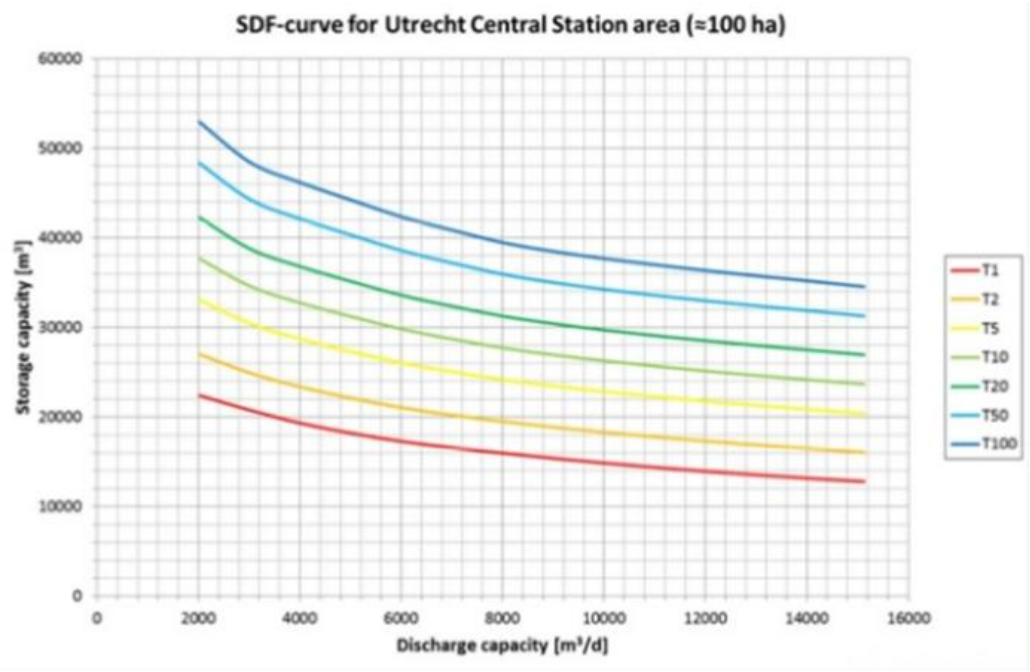


Figure 21 An example of storage-discharge capacity-frequency curve [32]

4.4.1 Design of the storage capacity of the on-site detention tanks

The storage capacity was designed following the technical guide of the National Water Agency of Singapore [18]. According to the CEN-standard for sewer and drainage networks developed by the European Commission, the design flooding frequency for residential area is once per 20 years [33]. Taken the increasing frequency of extreme weathers caused by global warming into consideration, the storage capacity of the on-site detention tanks was designed based on the IDF curve of Halim Perdannakusuma Airport in Jakarta with 25-year return period according to the reports of Intensity Frequency Duration and Flood Frequencies Determination Meeting held by UNESCO [35] (Figure X). This curve can expressed using Talbot's Formula as:

$$i_{25} = \frac{12380}{t + 36.4} \tag{4.3}$$

where i_{25} (mm/h) is the rainfall intensity for 25 year return period in t minute duration. Then the post-development peak runoff from a development site with no runoff controls with 25 year return period can be calculated using the formula:

$$Q_{post} = C_{post} iA \tag{4.4}$$

where Q_{post} is the peak runoff at the point of design before the installation of the storage tank at the site of design, C_{post} is the runoff coefficient before the installation of the storage tank (usually assumed to be 1 for impervious area), A is the catchment area of the site (0.5 ha on average), and i is the average

rainfall intensity within the time of concentration (approximately 5 min for 0.5-2 ha catchment area). And in this article the target of the detention tank is designed to limit the peak runoff at the point of design to be 25% of the post-development one at most (C_{target}=0.25) .

Then the sizing of the detention tank is followed with the Modified Rational Method which is mainly used for preliminary sizing of detention facilities in urban areas. It only serves the storage design for areas which are less than 8 ha. The Modified Rational Method considers a family of trapezoidal runoff graphs instead of triangular ones (Figure 22). As shown in the figure, the first triangular hydrograph represents the discharge in a rainfall event whose duration is equal to the concentration time (t_c). Other hydrographs are all trapezoidal and show the discharge in rainfall events with same return period but different duration (t_d), the peak runoff in can be calculated with the Rational Formula, $Q = CiA$, and the rainfall intensity formula, $i_{25} = \dfrac{12380}{t_d + 36.4}$ (mm/h), where t_d is the duration of the rainfall event.

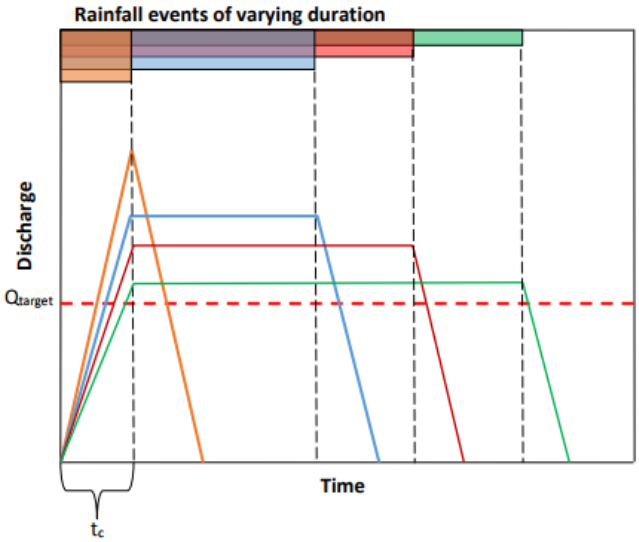


Figure 22 MRM runoff hydrographs for storms of various precipitation durations but a same return period [18]

The storage volume required for a certain storm duration is represented by the shaded area between the inflow and outflow hydrographs as shown in Figure 23 and expressed by the following equations:

$$V = Q_{\text{in}}(t_x + t_c) - \frac{1}{2}Q_{target}(t_x + 2t_c) \qquad (4.5)$$

$$Q_{\text{in}} = C_{post} \cdot i(t_d) \cdot A \qquad Q_{target} = C_{target} \cdot i(t_c) \cdot A$$

Where t_c stands for the time of concentration, and t_x is the difference of duration of the storm event and concentration time. Then Iterative calculation steps can be taken to obtain various storage volume of the detention tank normally up to 4 hours [18], the maximum one is designed to be its designed volume.

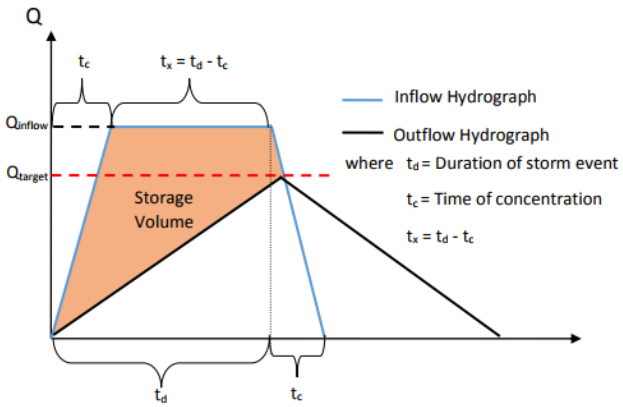


Figure 23 An Example for the calculation of needed storage volume [18]

Alternatively, a direct mathematical solution for this Modified Rational Method can be derived to the equation as follow:

$$V_t = C_{post}\frac{a}{t_d+b}At_d - \frac{1}{2}C_{target}\frac{a}{t_c+b}A(t_d+t_c)$$
$$\therefore \frac{dV_t}{dt_d} = \frac{C_{post}abA}{(t_d+b)^2} - \frac{1}{2}C_{target}\frac{a}{t_c+b}A \quad (4.6)$$

Where a and b are constant (12380 and 36.4, respectively, in this article). The value of Vt reaches its maximum when the derivative $\frac{dV_t}{dt_d}$ equals to 0:

$$\frac{dV_t}{dt_d} = \frac{C_{post}abA}{(t_d+b)^2} - \frac{1}{2}C_{target}\frac{a}{t_c+b}A = 0$$
$$t_{d\max} = -b + \sqrt{\frac{2C_{post}b(t_c+b)}{C_{target}}} \quad (4.7)$$

Where the t_{dmax} is the precipitation duration when the needed storage volume of reaches its maximum. In addition, the inflow hydrograph corresponding to t_{dmax} may lie below the target peak discharge, Q_{target}. Thus, the threshold t_{dlimit} that corresponds to the inflow hydrograph with a peak discharge equals to Q_{target} must be determined:

$$C_{post}\frac{a}{t_{d\,limit}+b}A = C_{target}\frac{a}{t_c+b}A$$
$$\therefore t_{d\,limit} = \frac{C_{post}t_c + (C_{post}-C_{target})b}{C_{target}} \quad (4.8)$$

When t_{dmax} is larger than t_{dlimit}, the maximum storage value occurs when $t_d=t_{dlimit}$; otherwise, the maximum storage value occurs when $t_d=t_{dmax}$. In this research, the calculated value of t_{dmax} is 73.4 min while the value of t_{dlimit} is 129.2 min. Thus, the designed storage capacity of the detention tank is 445.5 m^3. The water detained in the tanks will be discharged through orifices, thus the sizes of the tanks and orifices should be designed. Since the spare space in the highly residential area is limited while designed volume of the detention tank is large, placing these tanks underneath the pavement would be a good solution. Table X shows the size of both the detention tank and orifice.

Table 6 Sizes of detention tank and orifice

Detention tank	L (m)	W (m)	H (m)
	27.7	4	4
Orifice	$A = Q/(C\sqrt{2gh})$ (m^2, C=0.61)		
	1.15		

In this article, the on-site detention tanks are designed to be online. Because the residence time of storm water in the sewer system is very short (in minutes) [34], and also the online detention tanks are a part of the drainage system, the hydrograph of this scenario is not going to be calculated by modifying the values of parameters of the Sacramento Node, but calculated based on the numbers and size of the tanks, orifice, as well as the hydrograph of the same area before the application of the detention tanks. The outflow trough the orifice is calculated as follows:

- Before the detention tank is filled, the discharge follows the orifice discharge equation:

$$Q = CA\sqrt{2gh} \quad (4.9)$$

- Once the detention tank is filled, the outflow rate equals to the inflow rate.

In conclusion, the 5 scenarios can be described as follows:

- **Scenario 1:** null. No storm water retention method is implicated in the segment.
- **Scenario 2:** the green roof is implemented in 30 ha of roof areas in the segment.

- **Scenario 3:** there are 3000 houses with 30 ha of roof area in total using household storage tanks with 8 m3 storage capacity on average.
- **Scenario 4:** 14 ha of impervious pavements are replaced by pervious ones.
- **Scenario 5:** There is 44 ha of impervious area contributing to the inflow of 88 on-site detention tanks with 445.5m3 of storage capacity with discharge capacity up to 0.1 m^3/s.

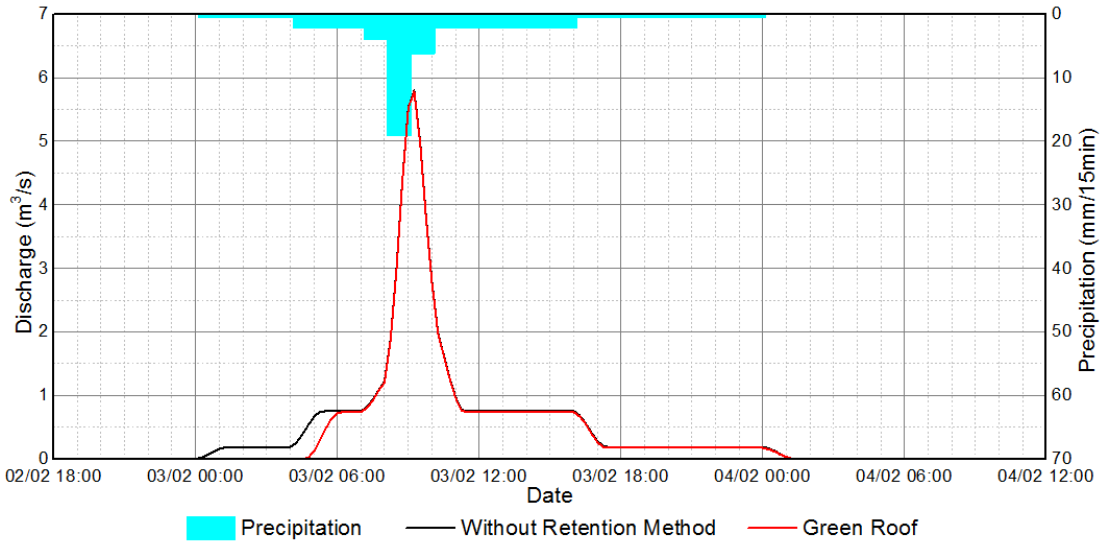
5 Results

A designed rainfall event with 100-year return period using a 15-minutes time step provided by Deltares was run for the simulation of all the scenarios in two segments, and the runoff was routed at the exit of the segment. The analyses were performed based on both the segment scale and catchment scale.

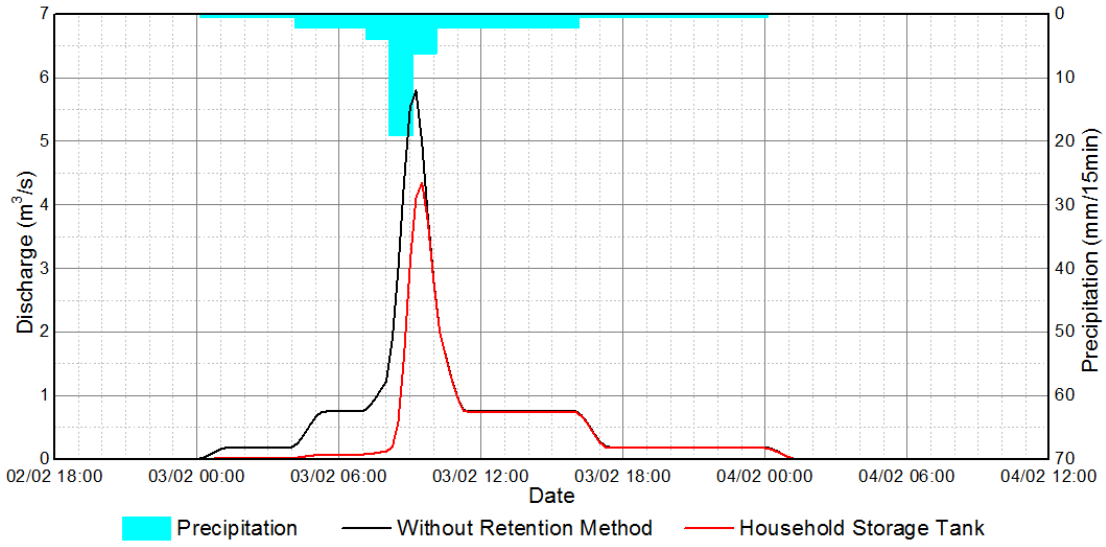
5.1 Segment Scale

5.1.1 Runoff contributed by the detention methods implemented area

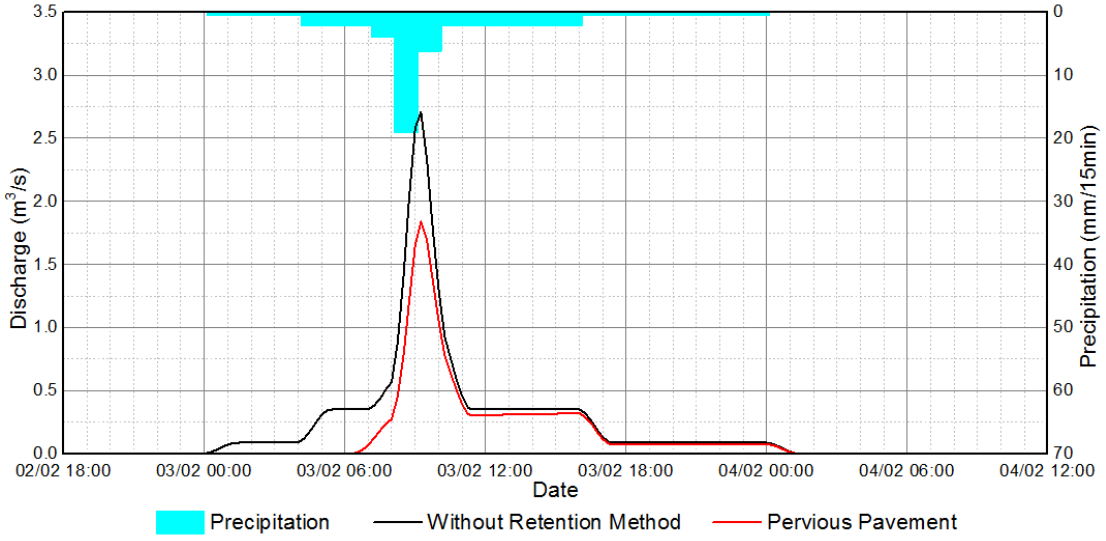
According to the scenarios and the simulation strategy explained above, the comparison of the runoff contributed by the area where the storm water detention methods are shown in Figure 23. According to these figures, the peak is not delayed in the Green roof, On-site detention tank, and the pervious pavement scenarios. The peak inflow generated by the household storage tanks scenario and detention tank scenario is delayed for 15 minutes. And all the runoff reduction is efficient at the beginning of the runoff generation, but the function is all lost later. The runoff reduction function of the green roof is lost before the peak precipitation intensity. The green roofs start to acting like a normal roof after that. Thus, there is seldom any peak flow reduction in the green roof scenario and the on-site detention tank scenario. However, the peak reduction efficiency of the household storage tank, the pervious pavements and detention tank is considerable. It is because that the household storage tanks and detention tanks have a relatively large storage capacity. Thus, the tanks are not filled at the peak of the precipitation rate. Thus the peak runoff is sufficiently reduced. As for the pervious pavement, the infiltration capacity is still relatively large when the precipitation rate reaches its peak. In addition, in the on-site storage tank scenario, at the tail of the hydrograph, the discharge is slightly larger than that in the null scenario to discharge the storm water detained during the precipitation event.



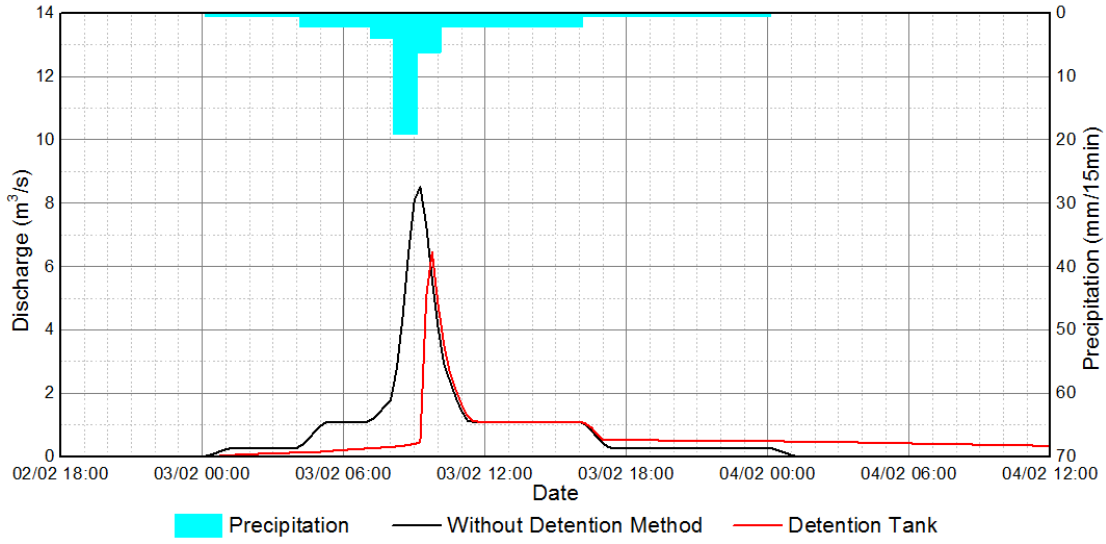
(a)



(b)



(c)

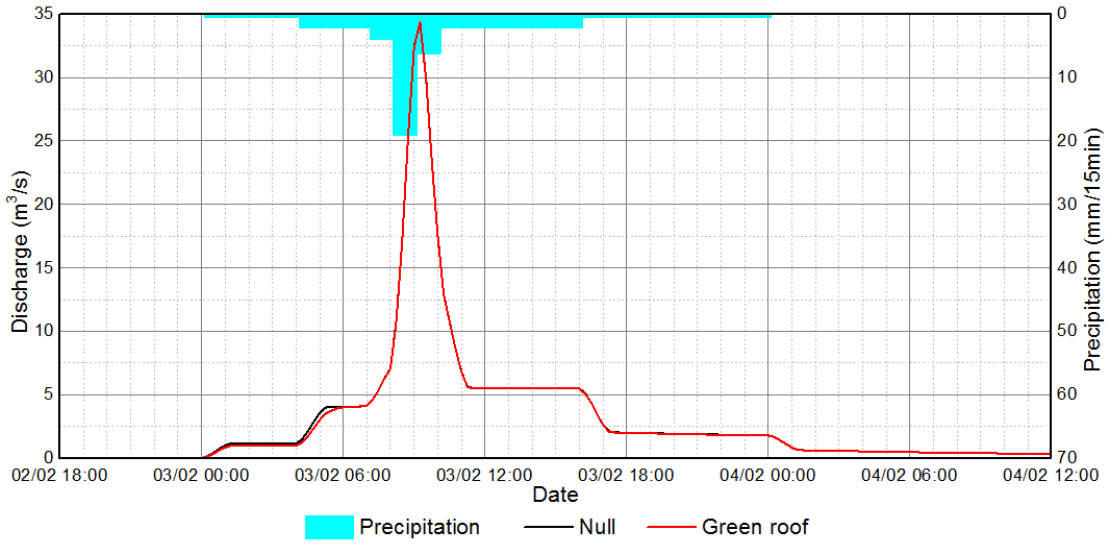


(d)

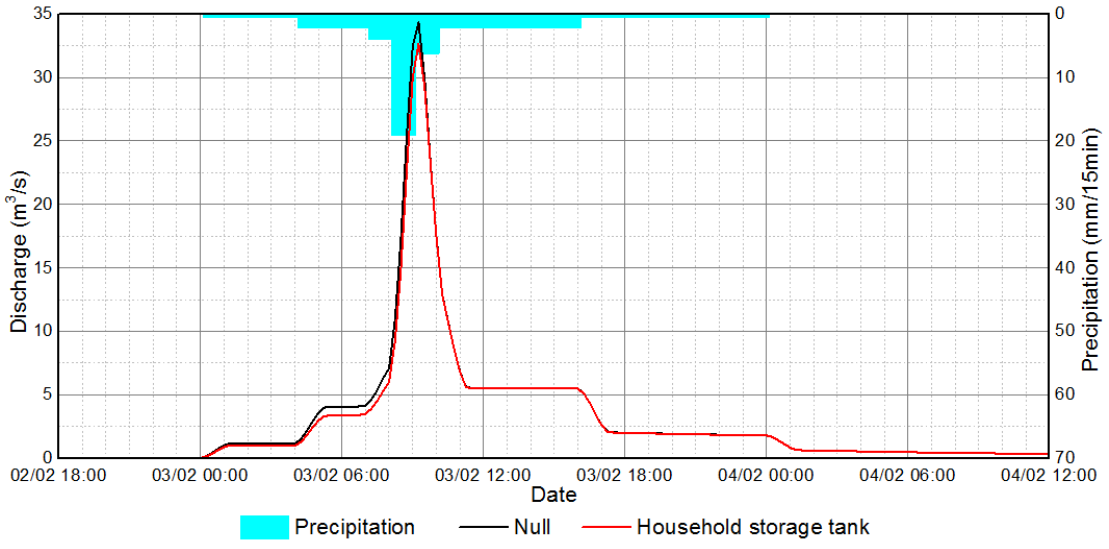
Figure 24 Comparison of the runoff contributed by the area where the storm water detention tank is used

5.1.2 Hydrograph of the segment

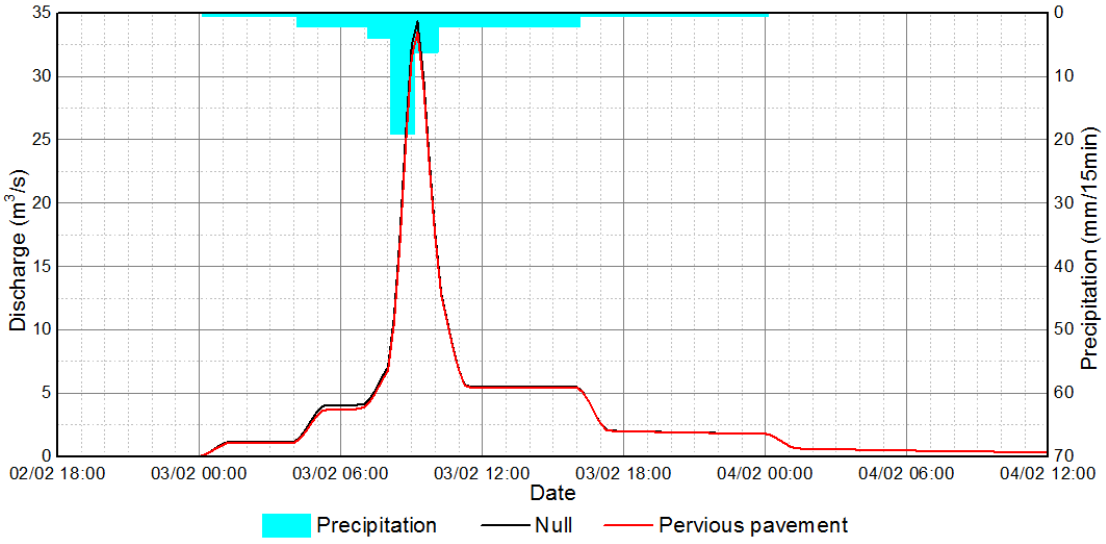
In Figure 24, the hydrograph of the segment with the detention methods are compared with the original hydrograph. It can be seen from the figure that there is seldom any peak delay for all the scenarios except the on-site detention tank scenario (15 min). And the summary of the peak reduction efficiency is shown in Table 6. As shown in the table, the peak flow reduction efficiency in the green roof scenario is very tiny (0.04%). The peak flow reduction efficiency of the pervious pavement is relatively small (2.53%). The household storage tanks have higher reduction efficiency (4.92%), while the reduction and on-site detention tanks is pretty impressive (19.4%). The difference of the peak flow reduction efficiencies among green roof scenario, household tank scenario and on-site detention tank scenario is highly related to their difference in the storage capacity increase. In the green roof scenario, 6000 m^3 of extra storage capacity is added to the study area. The houshlod tanks brings 24000 m^3 more storage capacity to the study area in Scenario 2, while in the detention tank scenario, the extra storage capacity added in the catchment reaches 39204 m^3. Generally speaking, the peak runoff generated form this segment are all reduced by varies degrees, but their efficiency are relatively small except that in the on-site detention tank scenario. Besides, the peak flow reduction and delay, there is a only a slight runoff raise at the end of the hydrograph in the detention tank scenario, while the runoff from the segment in the other storm water detention scenarios is all the same with that in the null scenario.



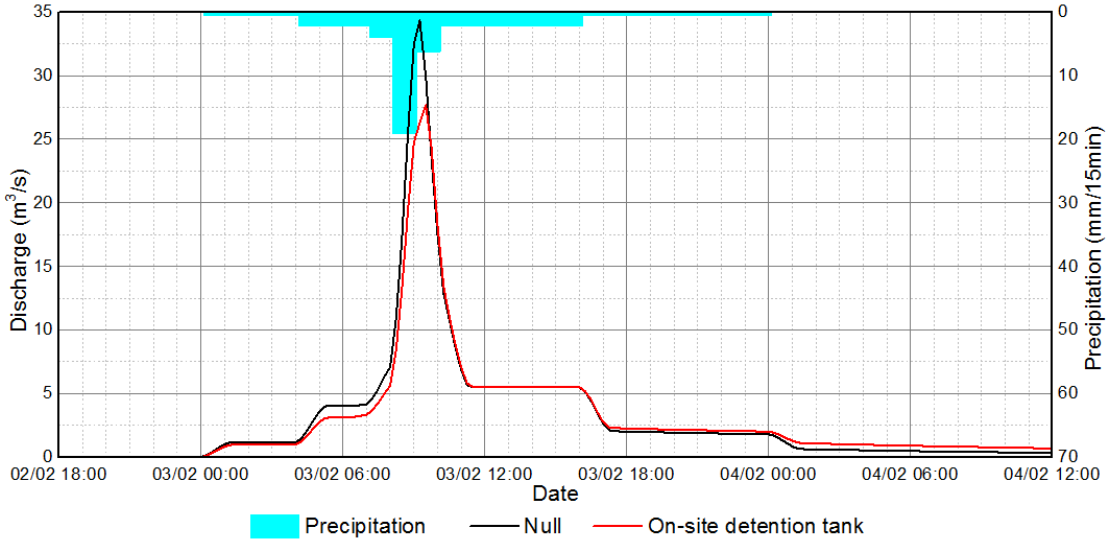
(a)



(b)



(c)



(d)

Figure 25 Comparison of hydrographs with and without storm water detention methods

Table 7 Reduction efficiency of peak flow

Scenarios	Null	Green Roof	Household Storage Tank	Pervious Pavement	On-site Detention Tank
Peak flow (m^3/s)	34.34	34.33	32.65	33.47	27.68
Reduction efficiency (%)		0.04	4.92	2.53	19.40

5.2 Catchment Scale

In this phase, the hydrographs of the scenarios as well as the discharge coming from upstream are plotted and compared (Figure 26). It is mentioned above that the peak runoff from the segment is not delayed and has only a reduction in all scenarios except on-site detention tank scenario. In these three scenarios, the runoff reaches the peak at 9:15 at the first day of the event, 7.25 hours before the peak discharge coming from upstream of the segment. At the peak of the discharge form upstream, there is seldom any change happens in the runoff generated in the segment. Thus, in these three scenarios, the storm water detention methods have no influence on the discharge to the downstream of the segment. In the on-site detention tank scenario, the peak runoff generated in the segment is reduced by a considerable amount. There is no difference between the runoff generated in the segment in null scenario and detention tank scenario from 9:30 to 17:00 on the first day of the event; and the discharge from upstream reaches its peak value at 16:30 on the first day, indicating that the discharge to the downstream of the segment is not changed after installing the on-site detention tank in the segment.

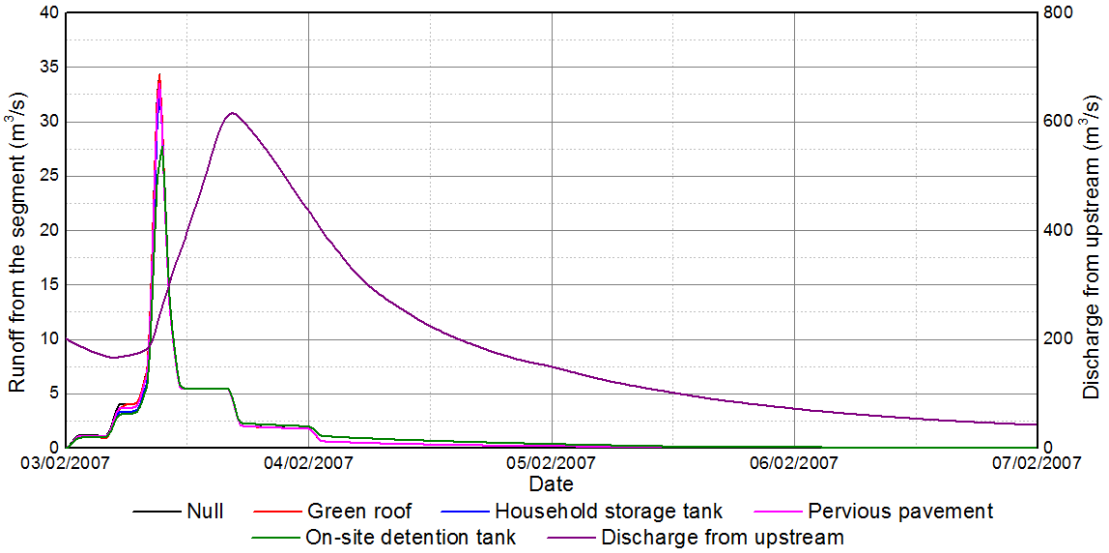


Figure 26 Runoff from the segment and upstream

6 Discussions

In this article, approaches to interpret the urban water detention methods in the hydrological model (Sacramento Model) is discussed to study their influence on the discharge regime when they are applied in the downstream cities. In the interpretation stage, uncertainty and reliability of the modelling strategy should be discussed.

Firstly, the behaviour and efficiency of the storm water detention methods are tested via a rainfall-runoff model built for a segment in Jakarta. However, the newly built model itself has some uncertainties because there are many assumptions made for this model. These assumptions can influence the accuracy of the hydrological model, for example, using the same unit hydrograph in all the Sacramento Nodes in the segment. As mentioned above, the construction of the urban drainage system can reduce the concentration time of the runoff from the urbanized area (the time it takes for runoff to travel from the most hydraulically distant point to the outlet [34]). As a result, the unit-hydrograph of the Sacramento Nodes representing the effective impervious areas and the area where the storm water management solutions where "adopted" should be different from the one used in the Sacramento Node representing the unpaved area. Besides, facts like lacking of available data for calibration for this single small segment and excluding some other hydrological processes which can happen in the urban area also raise the uncertainty of the hydrological model.

Problem also occurs in the simulation of the storm water detention methods using the Sacramento Node. Firstly, in the simulation of the household storage tank the water consumption of each time step is simulated as a proportion of the precipitation of that time step. However, if the population data, water consumption data as well as the diurnal water consumption pattern are all known, it is more reasonable to simulate the water consumption from the tanks based on the consumption and the water storage in the tanks. Secondly, in the simulation of pervious pavements, the tension water storage capacity is also questionable. In addition, in the simulation of on-site detention tanks, the storage and discharge capacity are chosen based on a design for the central area of Utrecht, which has totally different climate properties from that in Jakarta. Designing a proper storage and discharge capacity basing on the local condition may be able to help to buffer the flooding event in the area.

In addition, using only one design precipitation event to test the behaviour of urban water detention methods is not representative enough. Firstly, the antecedent conditions are unknown. The efficiencies of these water detention methods are highly related to the antecedent condition such as the storage in the household storage tanks, and in the on –site detention tanks as well as the soil moisture before the precipitation. Secondly, the design storm is taken from a point on the IDF curve. Pre-rain and post-rain are neglected in this study though they can have a substantial influence. In addition, the behaviour of urban water detention methods can be influenced by varies factors such as the pattern, amount, and duration of the precipitation event. Their behaviour and peak reduction efficiency can be changed during a different rainfall event. Thus, scenario test for only one rainfall event may be not representative enough.

7 Conclusions

In this research, to investigate whether implementing the storm water detention methods has positive or negative effects, an existing 1D model developed in SOBEK Rural for Jakarta is used. Four storm water detention methods, green roof, household storage tanks, pervious pavements, and on-site detention tanks are introduced and simulated in a dense residential segment in this model. And their runoff reduction and peak delay efficiency are tested using a 100-year return period precipitation event. Though there are some limitations in this study, based on the results gotten from the simulation, the conclusions are drawn in the following.

7.1 Segment scale

- Green roof

Compared with the null scenario, the runoff generated by the green roof area is reduced at the early stage of the precipitation event before the storage capacity of the green roof is reached. However, because the storage capacity of the green roof is limited, the detention function is almost lost later. As a result, the runoff reduction efficiency of the green roof at the peak is negligible. And this tiny runoff reduction efficiency at the peak comes from the additional evapotranspiration from the green roof. Therefore, the efficiency of the green roof in the extreme precipitation event is negligible.

- Household storage tank

The highest peak runoff from the whole segment is reduced the most efficiently in the household storage tank scenario. It is because the storage capacity is big enough so that the tanks are not totally full at the time of the precipitation intensity peak. As mentioned above, the household storage tanks increases the "equivalent initial loss" because of the increase of the available system storage capacity. Therefore, once the storage tanks are filled, the runoff reduction function of the household storage tanks is lost. After that, the generated runoff from the segment is the same as that in the null scenario. In conclusion, adding large storage capacity in the urban area is an efficient way to reduce the runoff from the segment even in an extreme precipitation event.

- Pervious pavement

It can be seen from the result that the runoff reduction simulated in the pervious pavement area at the peak is a quite considerable one among these four storm water detention scenarios. Though the runoff reduction efficiency in the whole segment of the pervious pavement scenario and the household storage tank scenario are the highest two, the runoff reduction efficiency in pervious pavement scenario is around half of that in the household storage tank scenario. It is because that the area of the pavements is limited when compared with the roof areas.

- On-site detention tank

Compared to the available storage capacity in the household storage tank scenario before the rainfall event (24000 m^3 in total, 545.5 m^3/ha on average), the available storage capacity in this scenario is lager (39204 m^3 in total, 891 m^3/ha on average). Combined with 8.8 m^3/s discharge capacity, in this scenario, the peak runoff reduction efficiency is much larger than that in the household storage tank scenario since detention tanks are not full at that time. After all the tanks are filled (15 min later), the on-site detention tanks have no more influence on the discharge reduction in the precipitation event until when the generated runoff is smaller than its discharge capacity again. At the tail of the hydrograph, when the runoff generated in the detention methods implemented area is smaller than the discharge capacity of the detention tanks, the discharge in this scenario becomes slightly larger than that in the null scenario until the detention tank is emptied.

All in all, it can be concluded as follows:

- The water detention function of the green roof area in an extreme precipitation event is negligible.

- Adding storage capacity to the urban area is an efficient storm water detention method, and it also can be able to reduce the peak runoff of from the segment if the storage capacity is big enough even in an extreme precipitation event.
- The runoff reduction efficiency of pervious pavement area is considerable. However, the area of the pavement itself is limited, which limits the runoff reduction efficiency of the whole segment. But it also shows that increasing the infiltration in the segment like implementing infiltration trenches and soakaways are also helpful.

7.2 Catchment scale

Among all the storm water detention methods, unlike the runoff reduction efficiency at the peak flow, the efficiency of peak runoff delay is negligible. In the example shown here, the peak runoff from the segment happens about 7.25 hours earlier than the time of the peak discharge coming from the upstream of the segment. At that time, solutions like household storage tanks and on-site detention tanks still have considerable runoff reduction efficiency. However, when the peak discharge that comes from the upstream occurs, the runoff reduction efficiency in all scenarios are negligible. As a result, the implementation of methods hardly has any influence on peak flow reduction in the river just downstream of the segment in this event; it only decreases the discharge before the peak. Besides, in the on-site detention tank scenario, the discharge to the downstream also increases slightly after the peak. This could slightly increase the peak in the river discharge further downstream. A real-time control facility is recommended to stop the discharge from on-site detention tanks by the time the peak discharge in the river occurs to avoid this negative effect.

7.3 Answer to the research question

In the introduction chapter, a question is raised that whether adopting the storm water detention methods in Jakarta, a city locates at the downstream of a basin, can still reduce the discharge magnitude because of its discharge reduction function or even increase the river discharge because of the peak delay function. To answer this question, the behaviour of 4 storm water management solutions are simulated with a designed precipitation event as the input. And analysis based on the segment scale as well as the catchment scale is performed. From the analysis above, the peak runoff generated in the segment is not delayed in the green roof, household detention tank, pervious pavement scenarios but in the detention tank scenario. The peak runoff in the segment occurs when the detention tanks are filled (only 15 min later); after that, the detention tank lost its function until the discharge capacity exceeds the generated runoff again. Thus, it is concluded that the delay function of the storm water detention methods simulated in this article is almost negligible. Also, in this example, the peak runoff reduction efficiencies in the segment of all the storm water detention solutions at the time are small except the detention tank scenario because of its large storage capacity. However, when the peak flow from upstream comes, the runoff reduction is negligible in all the scenarios. So there is almost no effects of these 4 storm water detention method on reducing the peak discharge in the river just downstream of the segment in this precipitation event. In addition, the runoff generated in the segment is raised in the on-line detention tank scenario 30 min later than the time when the peak discharge in the river comes in this case, however, it indicates that if adopt this scenario in lower stream, the risk of increasing the pick discharge to the just downstream increases. It is mentioned that, the peak reduction can be caused by a sufficient storage capacity for storm water detain. This result also indicate that off-line detention tank could have a positive effects on reducing the peak discharge in the river to the downstream if the off-line detention tanks start to detain storm water certain periods later and haven't reached their storage capacity when the peak discharge from the upstream comes. Besides, the simulation with just one precipitation event is not representative enough. The storm water detention methods can have different behaviour in different precipitation event. If reduction efficiency of the runoff from the segment still exists (because of smaller total precipitation or earlier peak precipitation intensity), the peak discharge just downstream of the segment can still be reduced.

8 Recommendations

In this article, the simulation of the hydrological process in a small segment in the Jakarta Basin is based on an existing lumped conceptual hydrological model. This fact challenges the simulation of the storm water detention methods. Firstly, the Sacramento model may not be sufficient to describe the hydrological processes comprehensively in an urban area. And it is not sufficient to simulate the hydrological processes in the segment where the storm water detention methods are applied through just adjusting the parameter of the single Sacramento Model, because that the runoff generation mechanism of the storm water detention methods is different from those in both the pervious and impervious areas. For example, when simulating rainfall-runoff relationship in a segment contains green roofs, it is not sufficient to just simply adding storage capacity to the upper zone in the Sacramento Node in the original model, because though there is a permeable soil layer in the green roof area that contains certain storage capacity, it cannot be simulated as a permeable area with hydrological processes like percolation. To solve these problems, a new model is introduced, dividing the segment into three areas, effective impervious surfaces, storm water detention methods implementation areas, and other areas. However, problems like lacking of data for calibration still influences the reliability of the model. Therefore, to simulate of the storm water detention methods, a distributed model including the detailed information (such as surface elevation and urban drainage system) about the urban area (built based on SOBEK Urban, for example) is preferred.

It is mentioned above that the input precipitation data used in this article is a rainfall event with 100-year return period. The effects of the storm water detention methods in other precipitation events with different total rainfall and temporal distribution on the peak discharge reduction is still unknown and to be tested. A long term recorded precipitation data is needed for further study on how the storm water detention methods alters the discharge regime of the segment as well as the whole catchment.

Furthermore, there are only 4 storm water management solutions tested here, and not even in a combination of the four. However, the efficiency of other storm water management solutions and combinations of these solutions is also to be studied, especially real time control solution. For example, real time control structures which linked to off-line detention tanks. Because it can be seen from the results that the runoff reduction function of the online on-site detention tank at the early stage is quite efficiency, if the detention tanks are offline and shifted into operation during the time when peak discharge from upstream comes, the runoff reduction efficiency would be different.

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