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Article

Flood Frequency Analysis for the Annual Peak Flows Simulated by an Event-Based Rainfall-Runoff Model in an Urban Drainage Basin

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Abstract: The proper assessment of design flood is a major concern for many hydrological applications in small urban watersheds. A number of approaches can be used including statistical approach and the continuous simulation and design storm methods. However, each method has its own limitations and assumptions being applied to the real world. The design storm method has been widely used for a long time because of the simplicity of the method, but three critical assumptions are made such as the equality of the return periods between the rainfall and corresponding flood quantiles and the selections of the rainfall hyetograph and antecedent soil moisture conditions. Continuous simulation cannot be applied to small urban catchments with quick responses of runoff to rainfall. In this paper, a new flood frequency analysis for the simulated annual peak flows (FASAP) is proposed. This method employs the candidate rainfall events selected by considering a time step order of five minutes and a sliding duration without any assumptions about the conventional design storm method in an urban watershed. In addition, the proposed methodology was verified by comparing the results with the conventional method in a real urban watershed.

Keywords: design storm method; continuous simulation method; flood frequency analysis; urban drainage basin

1. Introduction

Evaluating flood frequency and determining the design flood are the final goals for hydrological analysis and the beginning of integrated flood control [1]. A design flood is used in comprehensive flood management to assess the flood defense capacities of facilities and to protect human lives and properties within a watershed [2]. Hydrological research on the determination of the design flood for the evaluation of flood frequency can be classified into two approaches: (1) statistical approach estimates flood quantiles by applying probability models to flood data to determine the design flood; and (2) the design storm method uses the rainfall-runoff model and considers the rainfall quantiles determined by frequency analysis as the input data [3].

Statistical approach using measured annual peak discharge data is considered the standard method for estimating a flood quantile. However, a large error may occur in estimating low-frequency floods when the sample size is not long enough [4,5]. Furthermore, due to changes in land use, achieving stationarity of data for flood frequency analysis has become more difficult [6]. Because changes in land use that occur with urbanization affect the frequency of floods [7,8], studies are being conducted to adjust the past flood data to the present time condition [6]. According to the research results of [9,10], the change in land use clearly affects the hydrological responses of watersheds; however, quantifying these effects are very difficult. Therefore, even if sufficient continuously measured flood data are available for the site of interest, adjusting the flood data measured in the past to the present land use is very difficult and limits the computation of accurate flood quantiles.

Meanwhile, the design storm method is used to estimate flood quantiles by applying rainfall quantiles determined from rainfall frequency analysis to the rainfall-runoff model. This method requires three basic assumptions; the selection of the design rainfall hyetograph (rainfall duration and time distribution), the selection of the antecedent soil moisture conditions before the storm event, and the equality of the return periods between the rainfall quantiles and computed flood quantiles [2,11–13]. According to [14], even though the assumption that the return periods are equal between rainfall quantiles and simulated flood quantiles is not always acceptable [15], some researchers have found that the design storm method can produce acceptable peak discharge for a given return period if this method is used properly [16–18].

As a long-term run-off model or water balance analysis model, the continuous simulation method was suggested as an alternative to overcome the limitations of the design storm method [19]. However, due to the extensive data and computation requirements for the simulation, daily rainfall time-series are normally used [20,21]. Furthermore, because a continuous simulation method using daily rainfall time-series tends to estimate lower peak flow than the actual peak flow [22], this method is not considered a practical method in a small urban watershed where the runoff process is completed within a day [14].

In this study, a new flood frequency analysis to model a design flood is proposed to overcome the limitations of conventional design storm methodology and continuous simulation method. The procedures for the proposed method are as follows:

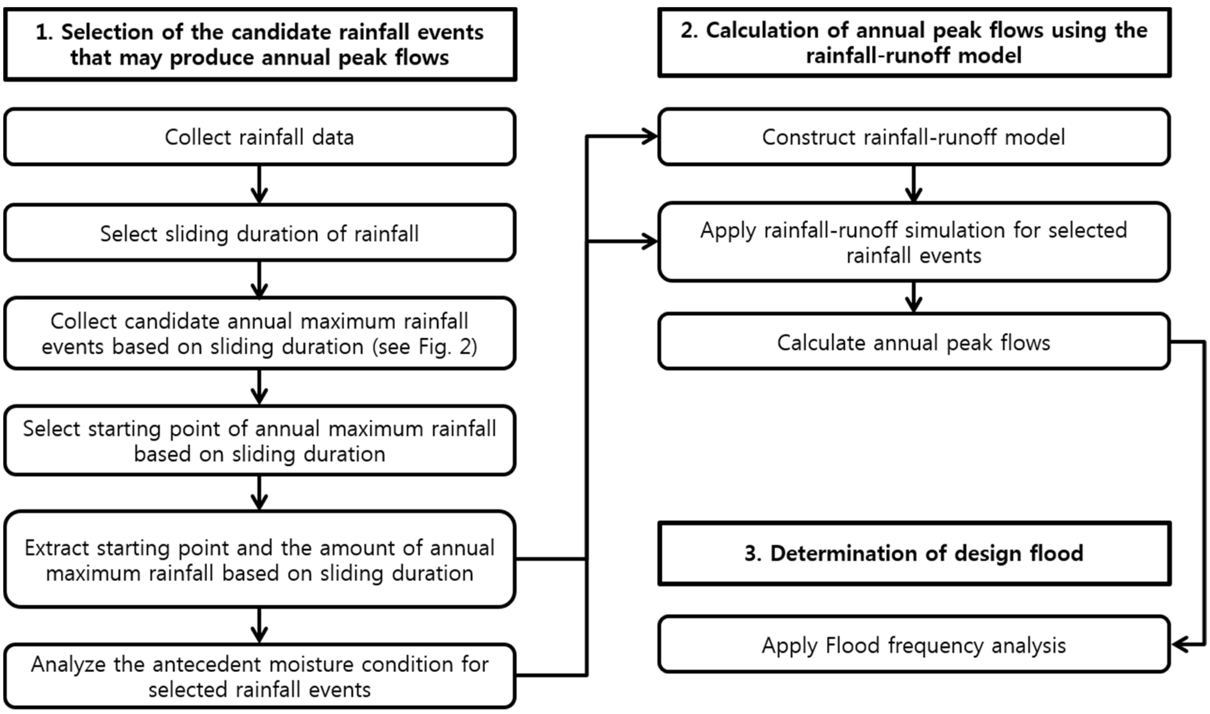
(1) Select the candidate rainfall events which can produce the annual peak flows by analyzing time-series rainfall data and considering time step order and sliding duration;
(2) Simulate the annual peak flows of the selected candidate rainfall events using the event-based model; and
(3) Apply frequency analysis for the simulated annual peak flows (FASAP) from step 2.

The proposed method can overcome data homogeneity due to the lack of flood data and land use changes commonly encountered in flood frequency analyses. Furthermore, this method does not require the three basic assumptions of the design storm method. Lastly, the proposed FASAP seems to possess high efficiency and accuracy for small urban watersheds while following the procedures of the existing continuous simulation methodology.

2. Research Method

In this study, the proposed method is used to determine the design flood by applying frequency analysis to the simulated annual peak flows obtained from the selected candidate rainfall events that may produce the annual peak flows in the design storm method, unlike the continuous simulation method where all the rainfall series are used. The procedure of the proposed method is displayed in Figure 1.

Figure 1. Flow chart for estimating annual peak discharge.



Compared to the continuous simulation method that requires massive amounts of input data and computation time, the proposed method needs less input data and computing time. Additionally, the annual peak flows are directly obtained by using a simple event-based model without requiring the development of new numerical models or any analysis system for the numerical simulation. Particularly in the case of small urban watersheds, runoff is completed within one day. Thus, the time interval of the model must be set to approximately 1 to 15 min in the continuous simulation method [19]; simulating the annual peak flows with this time step using general continuous simulation methods is almost impossible.

In the case of the statistical approach that uses observed flood data, ensuring homogeneity of the observed flood data is impossible due to the variation in the watershed runoff features caused by urbanization. However, the proposed method in this study uses the physical conditions of the watersheds on recent days to develop the numerical rainfall-runoff model, so homogenous flow data can be used for frequency analysis.

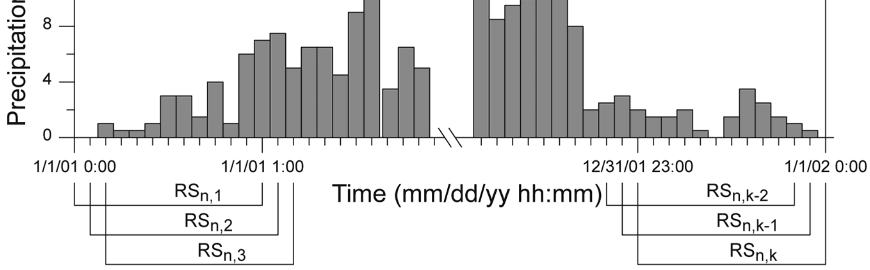
2.1. Sliding Maxima

According to [23], peak flow is strongly correlated to the maximum 15-min depth of precipitation. A basic assumption of this study is that runoff is positively correlated with rainfall. If the physical conditions of the watershed are fixed, then the characteristics of runoff are determined by rainfall attributes such as the rainfall amount, rainfall duration, and temporal distribution of rainfall. In this study, the candidate rainfall events that may produce annual maximum peak flows are selected by considering these three characteristics of rainfall. For this purpose, sliding maxima [24] were applied.

A range of durations for sliding maxima (sliding duration) analysis is defined properly by considering the runoff characteristics of the interested watersheds. In small urban watersheds, the upper limits of the range may be determined from the runoff duration time of the watersheds. The scales of sliding durations are determined empirically by considering the rainfall durations and the temporal distributions of heavy rainstorms. Large scales of sliding durations may be unable to address extremely short and intense rainfall, and these small scales require excessive computing resources.

Sliding maxima analysis is the first step to identifying individual rainfall events that may produce annual maximum peak flows for flood frequency analysis, so the procedure is just concerned with finding a point of time when an annual maximum peak flow occurs. Figure 2 shows the processes for selecting the annual maximum rainfall based on the sliding duration. By moving forward a nominated sliding duration in an annual rainfall time series, the rainfall amount ($RS_{n,k}$) for each sliding duration (n) is calculated. Where k is the total number of sliding duration for one year. The maximum $RS_{n,k}$ is the sliding maxima of the year, and the candidate rainfall events are selected around the corresponding time.

Figure 2. Selection of annual maximum rainfall based on the sliding duration.



The annual sliding maxima are determined according to the number of sliding durations, but the corresponding times will be concentrated on extreme rainfall events during the year. Therefore, the number of candidate rainfall events can be expected to be approximately 1–3 for each year.

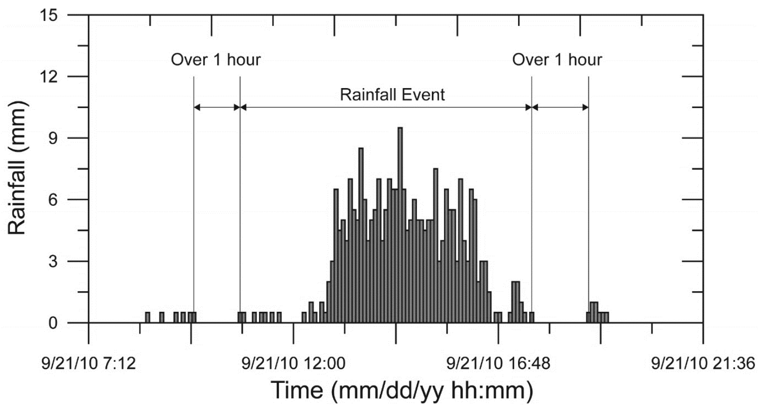
2.2. Identifying Independent Rainfall Events

Rainfall is routinely reported as falling in "events" or "storms" whose beginning and end are defined by rainless intervals of a nominated duration (minimum inter-event time, MIT) [25]. Research on the methods used to separate independent rainfall events from precipitation records began to represent the stochastic characteristics of precipitation [26,27]. Dunkerley [25] reviewed the research about MIT used in the recognition of rainfall events for various purposes such as climate change and soil erosion. However, using MIT for identifying rainfall events is affected by complex conditions such as rainfall depth, rainfall intensity, and watershed characteristics. In this study, a new method to define MIT is proposed for a small urban watershed.

Unlike a continuous simulation method, the proposed method, *i.e.*, FASAP, uses individual rainfall events for input data, so the rainfall events are required to be physically independent of runoff. If the MIT is not long enough, runoff produced by an antecedent event affects the peak flow produced by the on-going event. In this study, runoff time after the end of rainfall events is analyzed to separate the rainfall events, which is independent of runoff.

The method to identify independent rainfall events follows this procedure: (1) temporarily separate all the candidate events based on a rainless interval of 1 h (Figure 3); (2) simulate the annual peak flows from those events; (3) calculate the time required from the end of the rainfall event to the end of the runoff event for all the candidate events (see Section 3.5 for detailed descriptions); and (4) determine the MIT based on the separation rule from precipitation records of the physically independent rainfall events.

Figure 3. Separation condition for independent rainfall events.



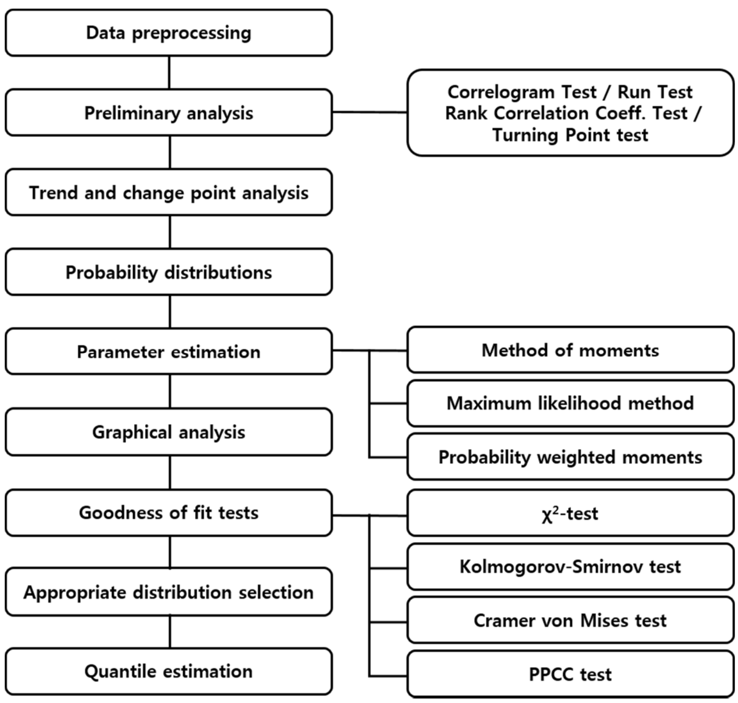
2.3. Event-Based Simulation

In this study, the event-based model is applied to evaluate annual peak flows using the candidate rainfall events. In general, the event-based approach is used to evaluate design floods that correspond to design storms estimated by rainfall frequency analysis. Assumptions are made about the antecedent soil moisture conditions, the base flow, the simplified hyetograph shape, and the concept of critical rainfall duration, which are not necessary in water balance models or long-term runoff models in a continuous simulation method. However, the proposed method in this study can adopt the event-based approach without these assumptions. This method uses single and individual rainfall events selected from the precipitation time series recorded with a fine time resolution, so a simplified hyetograph shape and the concept of critical rainfall duration are not necessary to simulate the peak flows. In a small urban watershed the base flow is very small compared to direct runoff when extreme storms are coming, and can be ignored. The antecedent moisture condition is an important factor that determines the initial conditions of event-based models, and also has a large impact on the final simulation results. In the conventional design storm method, the antecedent moisture condition should be assumed because of insufficient information. However, the proposed method in this study uses the real rainfall records prior to the candidate rainfall events (see Section 3.6 for detailed descriptions), so the antecedent moisture condition is not an assumption but a deterministic condition.

2.4. Frequency Analysis

Frequency analysis has been applied to estimate the quantiles for the annual maximum rainfall and flood data. For this purpose, the frequency analysis software FARD 2006 is used in this study (National Disaster Management Institute, Seoul, Korea). This software can be used to estimate the quantiles for the normal, lognormal, gamma, log-Pearson type III, generalized extreme value (GEV), Gumbel (GUM), log-Gumbel, Weibull, Wakeby, generalized logistic, generalized Pareto, and kappa probability distributions. The general procedure of frequency analysis is summarized in Figure 4. In addition, the chi-square-test, Kolomogorov-Smirnov test, Cramer von Mises test, and probability plot correlation coefficient (PPCC) test are used for the goodness of fit test in this software.

Figure 4. The procedure of frequency analysis in the software FARD [28]. Reproduced with permission from National Disaster Management Institute, FARD User Manual; published by National Disaster Management Institute, 2006. PPCC: Probability plot correlation coefficient.



3. Application

3.1. Watershed Description

The Hyoja drainage basin, which is the target region of this study, is located in the Jungnang treatment area at Seoul, Korea. It is composed of six sub-drainage basin and covers an area of 528.90 ha. The climate of Seoul features a humid subtropical summer and continental dry winter with mean annual precipitation of about 1450 mm. About 60% of precipitation falls in the summer monsoon period between June and August. The upstream area of this basin has a steep incline in a mountainous region with about 55% of impervious area ratio. Towards the central and downstream areas, the incline softens and becomes an urbanized flatland as a traditional urban drainage basin with almost 100% of impervious area ratio. The Hyoja drainage basin and sub-drainage basins are shown in Figure 5 and the areas of the Hyoja drainage basin and sub-drainage basins are shown in Table 1.

Figure 5. The drainage network of the Hyoja drainage basin.

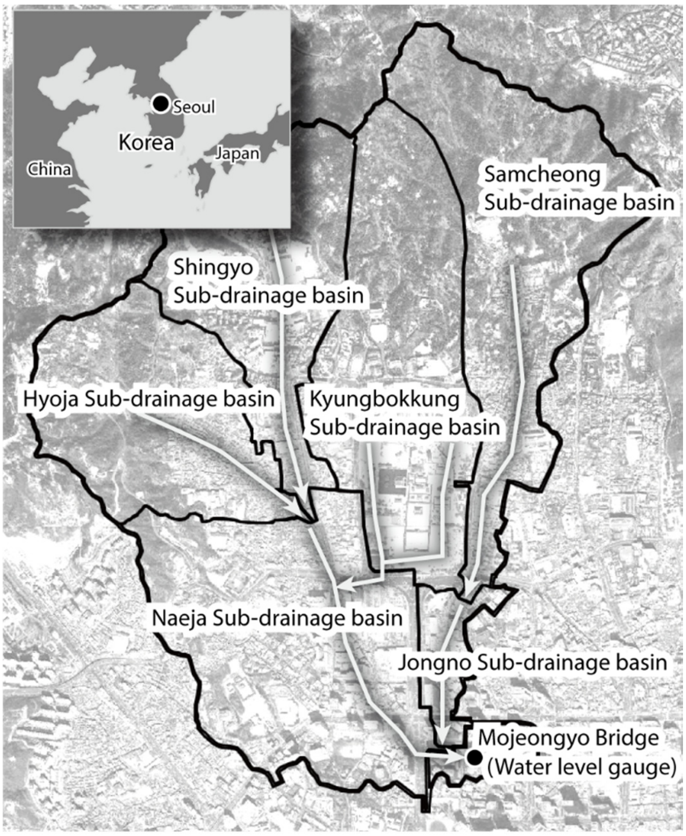


Table 1. The areas of the sub-drainage basins in the Hyoja drainage basin.

Drainage Basin	Sub-Drainage Basin		Area (ha)	Sub-Drainage Basin		Area (ha)
Hyoja drainage basin	Baegundong	Naeja	121.84	Joonghakcheon	Kyungbokkung	110.96
		Singyo	85.75		Samcheong	114.42
		Hyoja	81.14		Jongno	14.79
		Subtotal	288.73		Subtotal	240.17
	Total					528.90

3.2. Composition of the Numerical Model

This study used XP-SWMM 2011 (XP Solutions Inc., Queensland, Australia), a sewage and storm sewer piping network analysis program based on the SWMM (Storm Water Management Model) engine of the US EPA (Environmental Protection Agency, Washington, DC, USA). XP-SWMM 2011 is a model used normally for analyzing temporal changes in runoff and water quality by simulating the surface runoff, groundwater flow and flow inside the sewerage system that occurs due to single or continuous storm events in urban watersheds with artificial drain systems [29].

The composed model uses the SCS (Soil conservation service) CN (Curve number) method for infiltration, non-linear runoff routing (U.S. EPA Runoff Method) for simulating runoff, and fully dynamic flow equation (St. Venant) for hydraulic routing of open channel and closed-conduit. Table 2 shows base flow has ignorable portion of stream flow in the study area so that groundwater was not considered in this study. In addition, the rainfall intervals can be set as desired and the computing time interval can be arbitrarily adjusted for rainfall events. The six sub-drainage basins with a total area of 528.9 ha were separated into 172 unit catchments and the model was composed of 172 nodes and 170 links. A GIS (Geographic Information System) tool and a land use map (environmental geo-information system, Ministry of Environment, Sejong, Korea [30]) were used to classify the impervious area of each sub-basin and a detailed soil map (Korean soil information system, Rural Development Administration, Jeonju, Korea [31]) was used to calculate the CN of each sub-basin.

Table 2. Observed flow and channel data of the watershed case study.

Date	Stage (m)	Area Discharge Section (m^2)	Average Velocity (m/s)	Discharge (m^3/s)
17 June 2010	0.12	4.27	0.23	0.98
27 August 2010	0.37	8.24	1.02	8.40
29 August 2010	0.65	12.68	2.01	25.49
21 September 2010	1.30	23.00	3.53	81.19

3.3. Model Calibration and Validation

To calibrate and validate the model, peak flows were measured four times in 2010 (Table 2). In the target watershed, river restoration construction, improvements to the performance of the storm drainage system, and parks have been completed since the early 2000s, so maintaining the homogeneity between the flood data observed in the past and the currently observed data is difficult. Therefore, only the water level and flow velocity data observed in 2010, which is the year of interest for this research, were used to set up the rating curve and to calibrate the model (Figure 6). The calibration was conducted using the storm event of 21 September 2010 (Figure 7). To validate the calibrated model the simulated results using XP-SWMM 2011 were compared to flow data observed on 27 August, and 29 August (Figure 7). The model simulated flows with time-interval 0.5 s and the results were stored in every 1 min. The following simulations were carried out with those time conditions. The relative errors on peak flow were 7% for the 27 August event and 10% for the 29 August event, indicative of a good representation of the runoff dynamics and water balance, and only the simulated flow was slightly underestimated at the recession limb of the hydrograph.

Figure 6. Stage-discharge rating curve of the watershed case study.

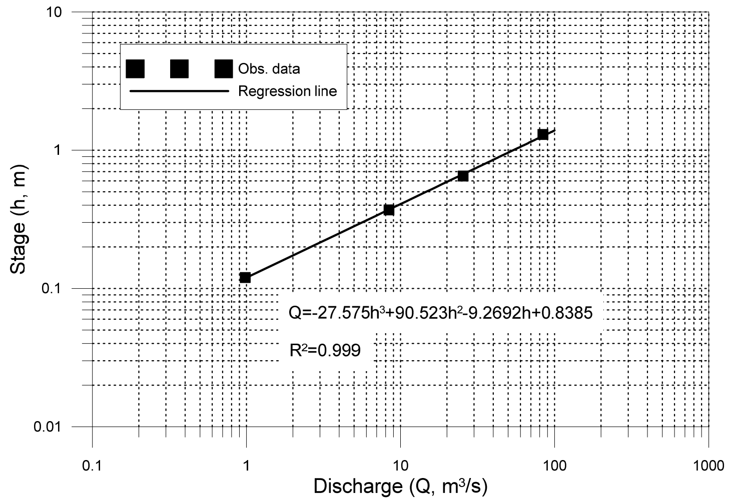
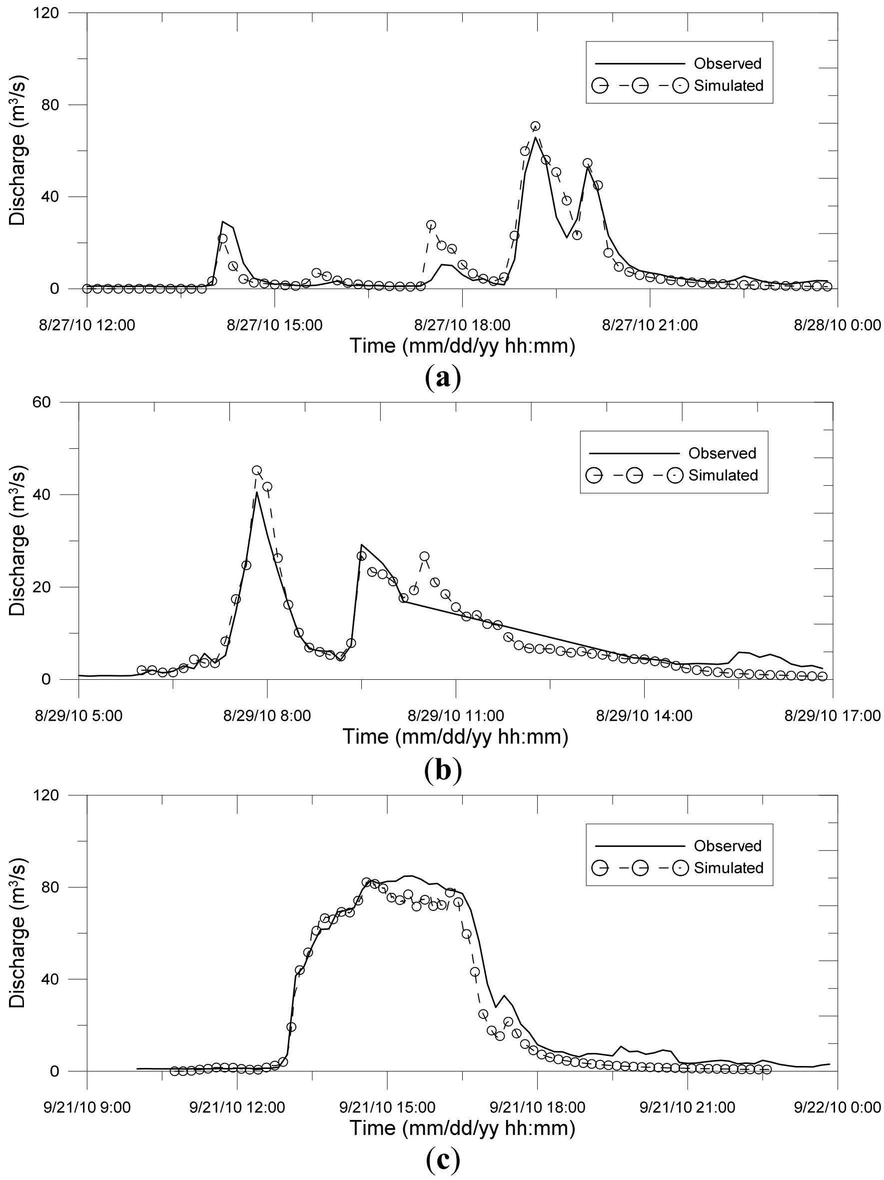


Figure 7. Observed and simulated flow on (**a**) 27 August 2010; (**b**) 29 August 2010; (**c**) 21 September 2010.



3.4. Selecting Candidate Rainfall Events by Sliding Maxima Analysis

Rainfall data which have a minimum threshold of 0.5 mm in five-minute intervals for 50 years (1961–2010) were collected for analysis. Additionally, three sliding durations such as 1-h, 2-h, and 3-h were determined by taking into consideration the characteristics of the target watershed in which the time of concentration was less than 2 h. Finally, 66 candidate rainfall events that may produce the annual peak flows were selected as shown in Table 3. All the sliding maxima (1-h, 2-h, 3-h) pointed to a single event in each of the 35 years, so 35 candidate events were selected. For 14 years two sliding maxima (1-h, 2-h or 2-h, 3-h) pointed to a single event and for 1 year all the sliding maxima pointed to three individual events.

Table 3. Number of events for each sliding duration.

No. of Years	Sliding Durations	No. of Events
35	1-h, 2-h, 3-h	35
6	1-h, 2-h	6
	3-h	6
8	1-h	8
	2-h, 3-h	8
1	1-h	1
	2-h	1
	3-h	1
Total		66

3.5. Determination of Independent Candidate Rainfall

Figure 8 shows the estimated peak flows, the flows at the end point of the rainfall event, and the flows after 1, 2, and 3 h from the end point of the rainfall event for the rainfall events selected for simulation. As shown in Figure 8, the maximum, minimum, and standard deviation of the peak flows were 99.37 m^3/s, 13.19 m^3/s, and 18.59 m^3/s, respectively. However, as time passed after the rainfall, deviations in the flows gradually decreased, and after 3 h from the end point of the rainfall event, the median of flows decreased below 1 m^3/s. This result shows that in the studied watershed, runoff ends approximately 3 h after the end of the rainfall event and that a rainless interval of 3 h is appropriate for the MIT to separate the candidate rainfall events to be simulated. The rainfall events temporarily separated by a MIT of 1 h were investigated whether the antecedent rainfall is 3 h from the beginning of the subsequent event. If antecedent rainfall occurs, then the candidate rainfall events are confirmed as a combination of the present and the corresponding antecedent rainfall; otherwise the candidate rainfall events are confirmed as themselves. This method could not be applied to large basin, because in which, the MIT will be long and determined independent rainfall event will have a very long duration as rainfall record for continuous simulation. The duration of the confirmed 66 events is shown in Table 4.

Figure 8. Box plot of the peak for the selected rainfall events.

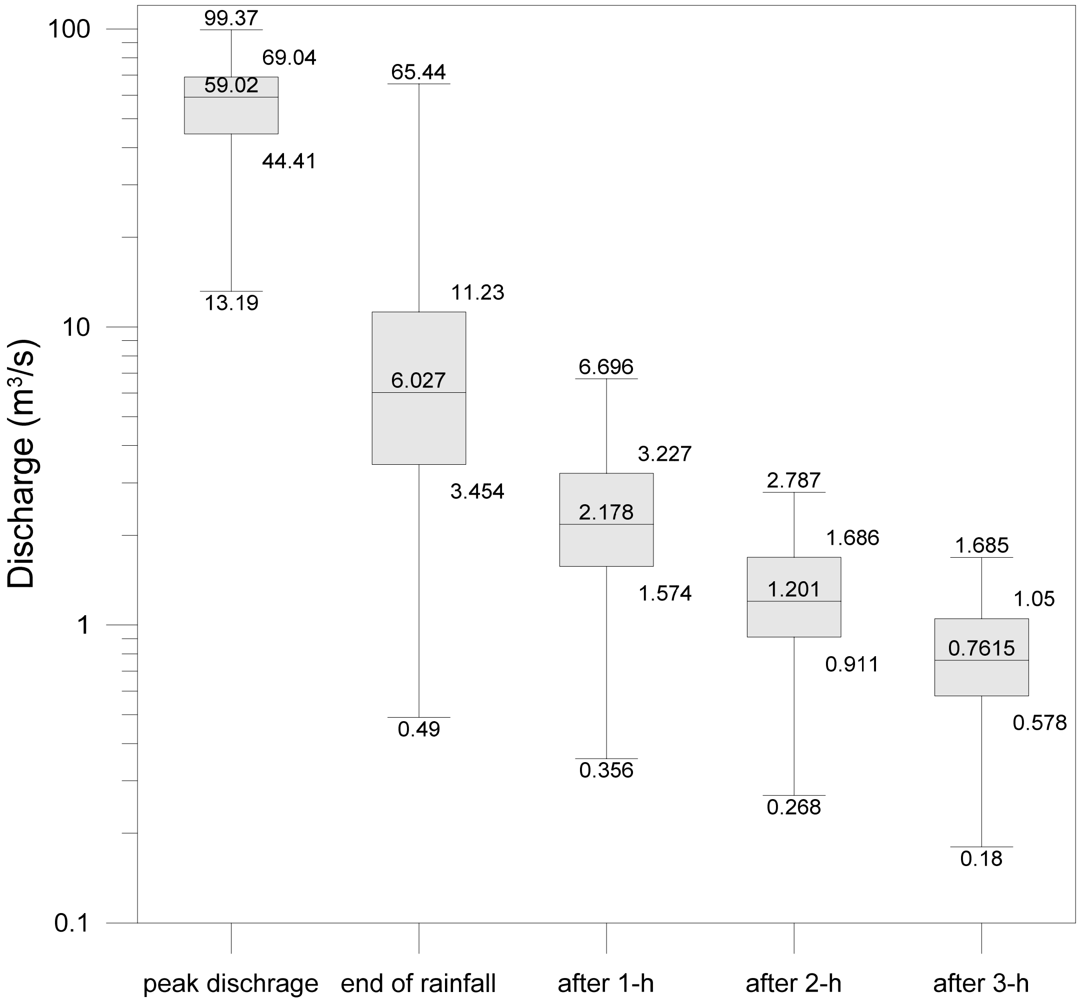


Table 4. Number of the candidate events for different rainfall durations.

Duration Class (h)	No. of Events	Duration Class (h)	No. of Events
Under 2	2	6–12	25
2–3	7	12–24	10
3–6	18	Over 24	4
	Total		66

3.6. Determining Antecedent Moisture Conditions

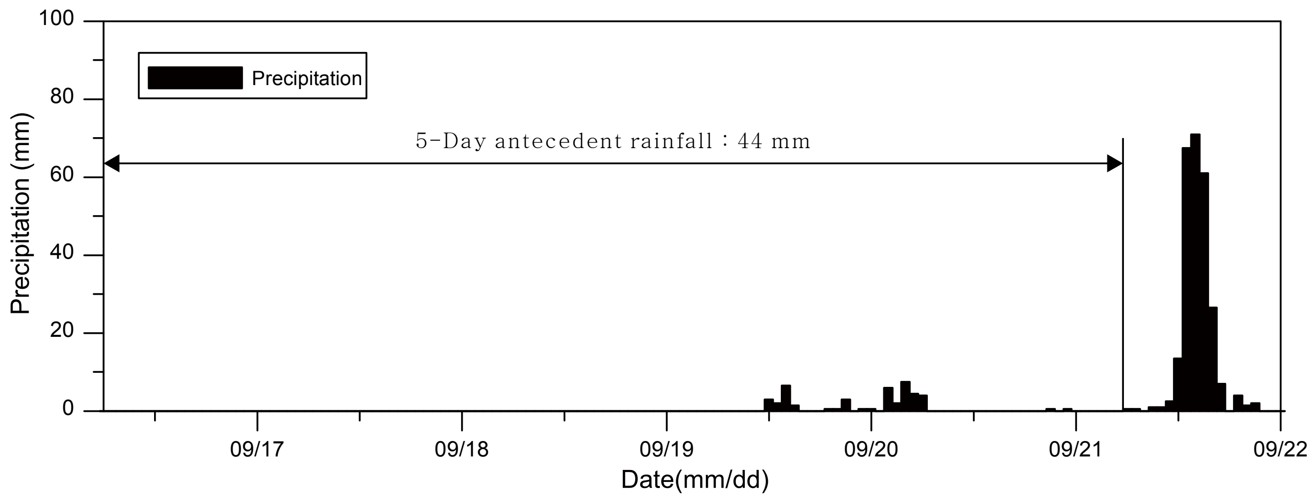
The major factors that affect runoff are primarily rainfall intensity and rainfall duration. However, the effect of soil features on the peak flows is also an important factor. In this study, the NRCS (National Resources Conservation Service) method was used to determine the antecedent moisture conditions in the watershed [32]. In the design storm method, the antecedent moisture condition should be assumed by the user even though physical characteristics such as soil characteristics and vegetative cover of the watershed are determined in advance. However, the proposed method simulates peak flows with the actual annual maximum rainfall events by considering the antecedent rainfall events and NCRS method, so assuming the antecedent moisture conditions is unnecessary.

AMC (Antecedent moisture condition) was determined from the previous 5-day rainfall totals which are grouped as shown in Table 5. Figure 9 shows the example how AMCs were selected. The 5-day antecedent rainfall of candidate event was 44 mm and September is growing season in Korea, so that AMCII was selected for the simulation according to Table 5. This procedure was carried out for all 66 candidate events. Out of the 66 events selected for simulation, 2 were AMC-I, 5 were AMC-II, and the remaining 59 events were AMC-III conditions.

Table 5. Rainfall groups for antecedent soil moisture conditions during growing and dormant season. AMC: Antecedent moisture condition.

AMC Group	5-Day Antecedent Rainfall, P5 (mm)	
	Growing Season	Dormant Season
I	$P5 < 35.56$	$P5 < 12.70$
II	$35.56 \leq P5 < 53.34$	$12.70 \leq P5 < 27.94$
III	$P5 \geq 53.34$	$P5 \geq 27.94$

Figure 9. Rainfall hyetograph of candidate event and previous 5-day in 2010.



3.7. Determining Annual Peak Flows

The annual peak flows were computed for the selected 66 candidate rainfall events based on the watershed conditions in 2010 (Table 6). As shown in Table 5, if one candidate rainfall event occurs per year, then the corresponding simulated peak flow becomes the annual peak flow. However, if multiple candidate rainfall events occur per year, then the maximum value of the peak flows becomes the annual peak flow (bold and italic numbers in Table 6).

Table 5 shows a strong tendency existed for selecting multiple rainfall events per year after 2003 and annual peak flows happened for relatively short sliding durations (1-h or 2-h). This result seems to be due to the effects of localized extreme rainfall with short durations and strong intensity that occurred frequently in the urban watersheds of Korea after 2000.

Table 6. The simulated peak flows and maximum annual peak flows.

Year	Sliding Duration	Peak Flow (m^3/s)	Year	Sliding Duration	Peak Flow (m^3/s)	Year	Sliding Duration	Peak Flow (m^3/s)
1961	1-h, 2-h, 3-h	***37.7***	1980	1-h	33.3	1998	1-h	***72.7***
1962	1-h, 2-h	***37.4***		2-h, 3-h	***36.8***		2-h, 3-h	71.9
	3-h	18.3	1981	1-h, 2-h	23.8	1999	1-h, 2-h, 3-h	***62.7***
1963	1-h, 2-h, 3-h	***67.1***		3-h	***41.3***	2000	1-h, 2-h, 3-h	***65.3***
1964	1-h, 2-h, 3-h	***96.7***	1982	1-h, 2-h, 3-h	***70.6***	2001	1-h, 2-h, 3-h	***99.4***
1965	1-h, 2-h, 3-h	***63.4***	1983	1-h, 2-h, 3-h	***75.4***	2002	1-h, 2-h, 3-h	***74.4***
1966	1-h, 2-h, 3-h	***83.0***	1984	1-h, 2-h, 3-h	***73.5***	2003	1-h	***78.4***
1967	1-h, 2-h, 3-h	***71.0***	1985	1-h, 2-h, 3-h	***75.2***		2-h, 3-h	76.5
1968	1-h, 2-h, 3-h	***68.2***	1986	1-h	51.0	2004	1-h	***62.5***
1969	1-h, 2-h, 3-h	***66.8***		2-h, 3-h	***55.8***		2-h	31.3
1970	1-h, 2-h, 3-h	***64.5***	1987	1-h, 2-h, 3-h	***82.1***		3-h	38.2
1971	1-h, 2-h, 3-h	***81.0***	1988	1-h, 2-h, 3-h	***47.8***	2005	1-h, 2-h	***69.0***
1972	1-h, 2-h, 3-h	***72.1***	1989	1-h, 2-h, 3-h	***46.5***		3-h	65.3
1973	1-h, 2-h, 3-h	***21.1***	1990	1-h	***53.7***	2006	1-h	61.0
1974	1-h	***53.1***		2-h, 3-h	46.5		2-h, 3-h	***67.6***
	2-h, 3-h	46.3	1991	1-h, 2-h, 3-h	***65.3***	2007	1-h, 2-h	***48.5***
1975	1-h, 2-h, 3-h	***63.1***	1992	1-h, 2-h, 3-h	***74.7***		3-h	24.3
1976	1-h, 2-h	***62.0***	1993	1-h, 2-h, 3-h	***72.5***	2008	1-h, 2-h	***54.6***
	3-h	46.3	1994	1-h, 2-h, 3-h	***37.8***		3-h	53.1
1977	1-h, 2-h, 3-h	***60.6***	1995	1-h, 2-h, 3-h	***52.6***	2009	1-h	***64.5***
1978	1-h, 2-h, 3-h	***41.3***	1996	1-h, 2-h, 3-h	***67.2***		2-h, 3-h	62.1
1979	1-h, 2-h, 3-h	***52.1***	1997	1-h, 2-h, 3-h	***49.0***	2010	1-h, 2-h, 3-h	***83.3***

3.8. Flood Frequency Analysis

Frequency analysis was applied to the simulated annual peak flows (Table 6) based on the method of probability weighted moments. The GEV, GUM, 2-parameter Weibull (WBU2), 3-parameter Weibull (WBU3), and generalized logistic (GLO) were selected for appropriate probability models. Next, the goodness of fit tests such as χ^2-test, Kolmogorov-Smirnov test, Cramer von Mises test, and PPCC test were applied to the five probability distributions at significance level of 5% as shown in Table 7. The maximum annual peak flows of the five probability models are displayed for the return periods of 2, 3, 5, 10, 20, 30, 50, 70, 80, and 100 years in Figure 10. The Gumbel distribution rejected for the PPCC and χ^2-tests shows completely different quantiles compared with the quantiles of the four other probability distributions. As shown in Table 7, the GEV, WBU2, WBU3, and GLO models all passed the goodness of fit tests. However, in this study, the GLO distribution was selected as an appropriate model, which is known to be suitable for flood frequency analysis [33], and the GEV distribution, which is widely used for flood frequency analysis, was also selected for comparison purposes.

Figure 10. Flood quantiles for the applied probability models.

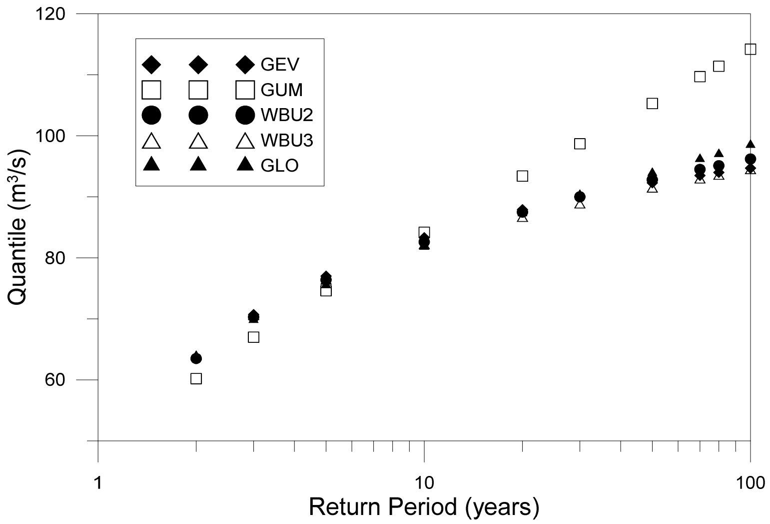


Table 7. The results of the goodness of fit tests for the applied probability models. GEV: Generalized extreme value; GUM: Gumbel; WBU2: 2-parameter Weibull; WBU3: 3-parameter Weibull; GLO: generalized logistic; PPCC: probability plot correlation coefficient.

Goodness of Fit Test	Probability Distribution				
	GEV	GUM	WBU2	WBU3	GLO
χ^2	O	X	O	O	O
Kolmogorov-Smirnov	O	O	O	O	O
Cramer von Mises	O	O	O	O	O
PPCC	O	X	O	O	O

4. Sensitivity Analysis

Sensitivity analysis was conducted for methodological appropriateness of the results of this study. To reveal that accurate simulation results cannot be obtained in small-scale urban watersheds using the continuous simulation method, the simulated peak flows were compared for both the short time step order (5 min) of this study and the long time step order (1 h) of the continuous simulation method. In addition, the peak flows from the sliding durations of 1, 2, and 3 h applied in this study were compared with peak flows from a sufficiently short period of sliding duration (10 min) to show whether the applied sliding durations in this study were appropriate for analyzing the storm events in the watershed.

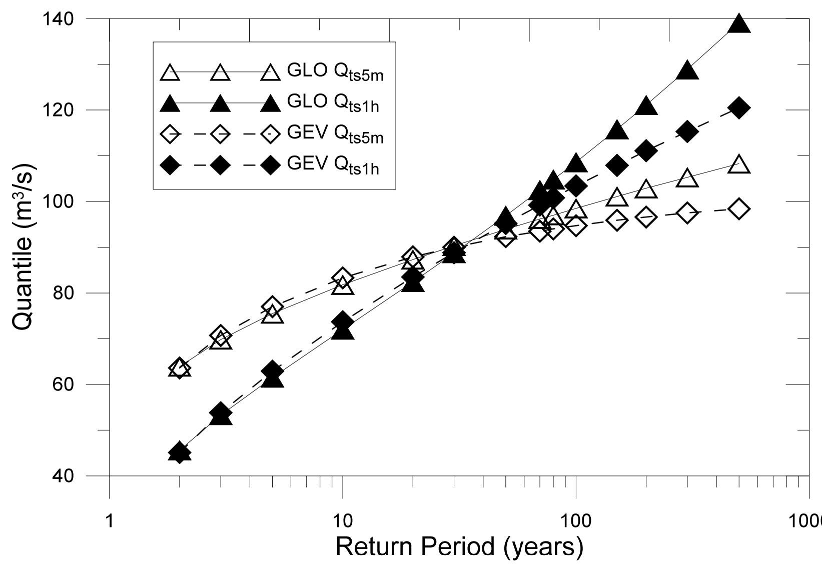
4.1. Time Step Order

The continuous simulation method is generally applied to large-scale watersheds with time steps in the order of 1 h–1 day. For agricultural catchments with slow responses of runoff from rainfall, time steps on the order of 1 h to 1 day may be appropriate for the simulation, while sub-hourly time steps are required for small urban watersheds that have a relatively quick hydrologic response. The estimated flood quantiles for time step orders of 5 min (Q_{ts5m}) and 1 h (Q_{ts1h}) for the GLO and GEV distributions are displayed in Table 8 and Figure 11. The flood quantiles for a time step order of 5 min are larger than the quantiles obtained for a time step of 1 h when the return period (T) is less than or equal to 30 years, while vice versa when the return period is more than 30 years. The differences between Q_{ts5m} and Q_{ts1h} are, respectively, −28.64% to 28.44% for the GLO model and −29.09% and 22.46% for the GEV model. Q_{ts5m} of the GLO model is a little bit smaller than the value in the GEV model for $T \leq 30$ years, while vice versa for $T > 30$ years because the GLO model has a heavier upper tail than the GEV model. According to Figure 11, a time step order of 1 h, which is usually applied in the continuous simulation method, is not appropriate to simulate proper annual peak flows for a small urban watershed in which the hydrologic response is fast.

Table 8. The estimated flood quantiles and differences for the GLO and GEV distributions based on time step orders of 5 min and 1 h.

Return Period	GLO			GEV		
(Year)	Q_{ts5m} (m^3/s)	Q_{ts1h} (m^3/s)	Difference	Q_{ts5m} (m^3/s)	Q_{ts1h} (m^3/s)	Difference
2	63.9	45.6	−28.64%	63.6	45.1	−29.09%
3	69.8	53.3	−23.64%	70.7	53.8	−23.90%
5	75.5	61.5	−18.54%	77.0	62.9	−18.31%
10	81.8	72.0	−11.98%	83.3	73.7	−11.52%
20	87.3	82.4	−5.61%	87.9	83.5	−5.01%
30	90.3	88.7	−1.77%	90.0	88.8	−1.33%
50	93.9	96.9	3.19%	92.3	95.2	3.14%
70	96.2	102.5	6.55%	93.5	99.2	6.10%
80	97.0	104.8	8.04%	94.0	100.8	7.23%
100	98.5	108.6	10.25%	94.7	103.4	9.19%
150	101.1	115.8	14.54%	95.9	107.9	12.51%
200	102.9	121.1	17.69%	96.6	111.1	15.01%
300	105.3	128.9	22.41%	97.5	115.3	18.26%
500	108.3	139.1	28.44%	98.4	120.5	22.46%

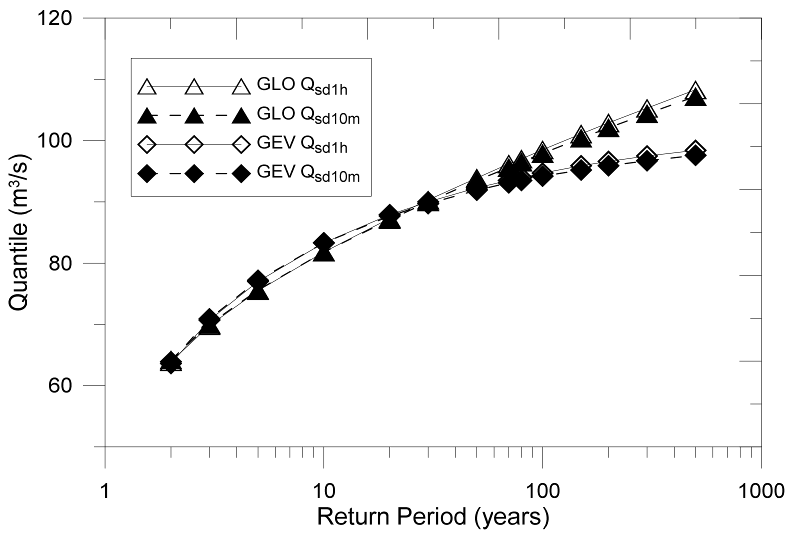
Figure 11. The estimated flood quantiles for the GLO and GEV models based on time step orders of 5 min and 1 h.



4.2. Sliding Duration

For a small urban watershed, extracting rainfall events that produce the peak flows may be difficult if the sliding durations and time intervals are not selected properly because of the quick hydrologic response. In this study, sliding durations of 1-h, 2-h, and 3-h were selected. Therefore, the results from setting the sliding duration to 10 min was compared to 24 time intervals of 10 min (e.g., from 10 to 240 min with 10 min interval) to review whether these parameters were appropriately selected. Upon setting the sliding duration and time interval at 10 min for comparison purposes, 12 rainfall events in 11 years were added as candidate rainfall events that may produce the annual peak flows. After simulating the added 12 rainfall events, only one event showed a bigger annual peak flow when the sliding duration and time interval were 1 h (11 rainfall events showed smaller annual peak flows in the 1 h case). As a result, the annual peak flows from the 1 h setting were the same as the previous results for 49 rainfall events out of 50 years (1961–2010), while only one annual peak flow from the 10 min setting was bigger than the peak flow from the 1 h setting. Figure 12 shows the estimated flood quantiles of the sliding durations and time intervals of 10 min (Q_{sd10m}) and 1 h (Q_{sd1h}) for the GLO and GEV distributions. As shown in Figure 12, the differences between the 10 min and 1 h settings are less than 1% for both the applied probability models. Thus, setting the sliding duration and time interval at 1 h judged as appropriate in terms of differences and computing time.

Figure 12. The estimated flood quantiles of sliding durations and time intervals of 10 min and 1 h for the GLO and GEV distributions.



5. Comparative Analysis

Comparative analysis was conducted for methodological the feasibility of the results of this study. The conventional design storm method and continuous simulation were carried out with same numerical model and rainfall records used in FASAP. The objective of the comparison between conventional design storm method and FASAP was to verify that the results of FASAP corresponded to former research and the comparison between continuous simulation and FASAP was to verify that FASAP precisely found the annual maximum peak flows events. The feasibility of the results of this study was judged by comparing these results from the design storm method and continuous simulation.

5.1. Design Storm Method

In this section, the FASAP was compared with the design storm method. The method of probability weighted moments was applied to rainfall frequency analysis in the conventional design storm method. In the results of rainfall frequency analysis for all the rainfall durations considered, the GEV distribution was selected as an appropriate probability model. In addition, the GLO distribution selected as an appropriate model in the proposed FASAP method was also added for comparison purposes. Huff's quartile method (third quartile in this case) was used as the rainfall time distribution according to the regulations for design flood calculations (Ministry of Land, Transport and Maritime Affairs (MoLIT), Seoul, Korea [34]).

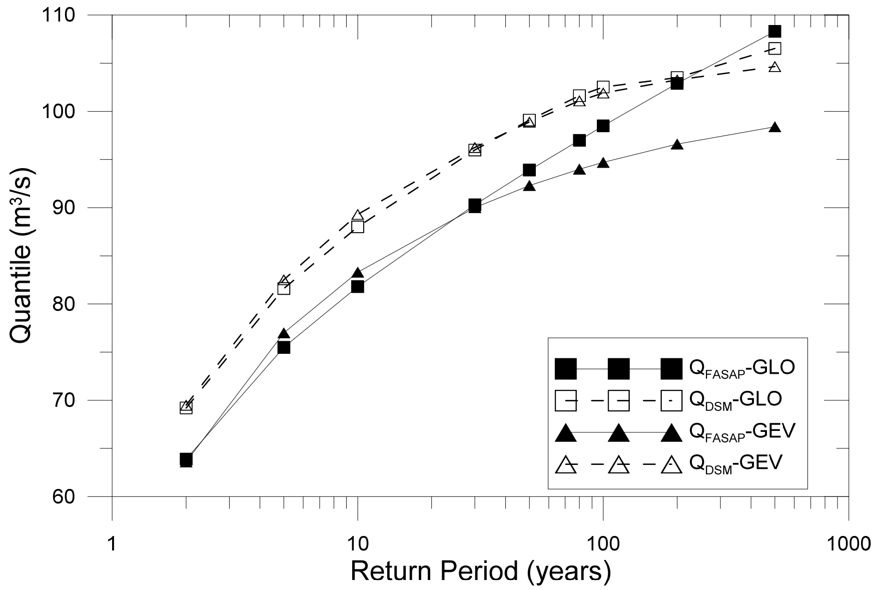
Table 9 shows the estimated flood quantiles based on the conventional design storm and FASAP methods and the differences between the two methods. The flood quantiles from the FASAP method were smaller than those quantiles from the conventional design method by 6.36% to 9.26% for the GEV model and by −1.63% to 8.30% for GLO model. As shown in Figure 13, the estimated flood quantiles for the GEV model show a similar pattern of increasing trend with an average difference between the two methods of 7.35%, while the differences in the flood quantiles for the GLO model are initially smaller and then show the reverse phenomenon as the return period increases over the return period of 200 years.

Table 9. The estimated flood quantiles and the differences for the applied methods. FASAP: flood frequency analysis for the simulated annual peak flows; DSM: Design storm method.

Return Period (Year)	GEV (m^3/s)			GLO (m^3/s)		
	DSM	FASAP	Difference	DSM	FASAP	Difference
2	69.5	63.6	9.26%	69.2	63.9	8.30%
5	82.5	77.0	7.17%	81.6	75.5	8.08%
10	89.3	83.3	7.21%	88.0	81.8	7.59%
30	96.2	90.0	6.94%	96.0	90.3	6.29%
50	98.9	92.3	7.14%	99.1	93.9	5.53%
80	101.1	94.0	7.55%	101.6	97.0	4.78%
100	101.9	94.7	7.62%	102.5	98.5	4.09%
200	103.3	96.6	6.91%	103.5	102.9	0.59%
500	104.7	98.4	6.36%	106.5	108.3	−1.63%

For urban watersheds, runoff is made through the storm management system with limited conveyance. If the rainfall quantile is over the design storm (e.g., approximately 30 years in Korea), then the increasing rate of peak flows in the storm management system is decreased because of limited conveyance capability. The estimated flood quantiles from the proposed FASAP method were approximately 10% smaller than the quantiles from the conventional design storm method, which is consistent with the research results of [2] that compare the flood quantiles using flood frequency analysis, the design storm method, and the continuous simulation method. According to research comparing the design storm method and flood frequency analysis in gauged watersheds [35–37], the conventional design storm method tends to overestimate the design flood. This indicates that the method in this study is appropriate to estimate design floods.

Figure 13. The estimated flood quantiles of design storm method and FASAP.

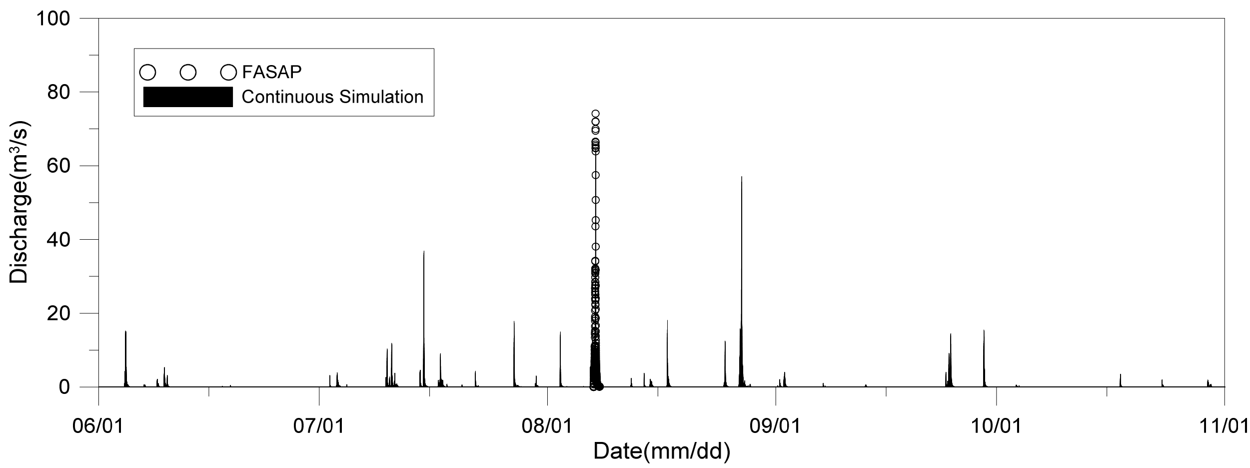


5.2. Continuous Simulation

To verify that FASAP precisely found the annual maximum peak flows events, continuous stream flows data are required. Continuous simulation were carried out to make continuous stream flows data. It was composed based on the numerical model of FASAP and used rainfall records of 5 months from June to October which is the rainy season of Korea. XP-SWMM 2011 cannot handle rainfall data recorded in 5 min time step for 5 months which were 44,076 records, so that continuous simulation were carried out with 1 h time step order as the conventional continuous simulation did. The computational time of continuous simulation was about 6 h for 1 year and 300 h for all 50 years. Comparing with computational time of FASAP (about 2 h for 66 cases), continuous simulation required computational time about 150 times as long as FASAP to determine the annual maximum peak flows.

Figure 14 shows the example of flow hydrograph simulated by continuous simulation and FASAP overlapped on same time line. Continuous simulation determined the annual peak flow after simulate 3673 h but FASAP needed less than 12 h simulation.

Figure 14. Flow hydrograph from FASAP and continuous simulation in 1992.



The annual peak flows of continuous simulation and FASAP are listed in Table 10. The annual maximum peak flows simulated by continuous simulation were smaller than FASAP for all years. This is consist with the comments of former researches [14,22] in which they referred that using long time step tends to smooth simulated peak flows. The estimated time when annual maximum peak flows (peak time) were occurred are listed in Table 10. The shadowed cells with italic numbers indicate cases not matched with each other. For all 50 years, annual peak flows were occurred in same events in 41 years and the 9 cases were not matched. 82% of the peak times for FASAP were matched with continuous simulation. Considering the errors from long time step of continuous simulation, FASAP can be regarded as good method to determine annual maximum peak flows.

Table 10. The simulated annual peak flows and the peak times of continuous simulation and FASAP.

Year	Continuous Simulation		FASAP		Year	Continuous Simulation		FASAP	
	Peak Flow	Peak Time	Peak Flow	Peak Time		Peak Flow	Peak Time	Peak Flow	Peak Time
1961	26.8	07–14 05:04	37.7	07–14 04:45	1986	54.5	07–24 05:00	55.8	07–24 04:32
1962	***19.6***	***08–08 09:59***	***37.4***	***08–05 10:10***	1987	72.1	07–27 04:00	82.1	07–27 04:13
1963	***55.0***	***07–17 10:00***	***67.1***	***08–14 04:34***	1988	24.0	07–09 08:00	47.8	07–09 08:02
1964	92.8	09–13 03:00	96.7	09–13 02:51	1989	38.0	08–11 20:00	46.5	08–11 19:47
1965	61.4	07–20 05:54	63.4	07–20 05:54	1990	36.9	09–11 08:00	53.7	09–11 08:17
1966	79.7	07–15 16:00	83.0	07–15 16:15	1991	58.5	07–25 17:49	65.3	07–25 17:57
1967	57.4	08–25 18:00	71.0	08–25 18:19	1992	64.9	08–07 12:54	74.7	08–07 12:31
1968	68.1	07–04 01:00	68.2	07–04 00:15	1993	64.0	07–11 14:00	72.5	07–11 13:27
1969	57.7	09–19 12:05	66.8	09–19 12:35	1994	32.5	07–05 09:00	37.8	07–05 09:06
1970	57.1	06–25 12:50	64.5	06–25 12:06	***1995***	***52.5***	***08–19 17:05***	***52.6***	***07–10 04:22***
1971	66.7	07–17 06:00	81.0	07–17 04:40	1996	56.3	07–26 15:00	67.2	07–26 15:05
1972	62.2	08–19 11:00	72.1	08–19 10:18	1997	36.1	07–01 08:00	49.0	07–01 07:35
1973	***14.6***	***08–17 17:00***	***21.1***	***07–30 00:58***	1998	69.9	08–08 03:59	72.7	08–08 03:55
1974	***35.9***	***08–03 05:00***	***53.1***	***07–29 14:17***	1999	55.2	08–02 09:25	62.7	08–02 08:46
1975	40.6	07–25 09:00	63.1	07–25 08:17	2000	58.2	08–28 03:00	65.3	08–28 02:51
1976	54.5	08–13 18:59	62.0	08–13 19:01	2001	90.2	07–15 03:00	99.4	07–15 03:09
1977	33.3	07–04 04:05	60.6	07–04 04:47	2002	65.9	08–07 05:00	74.4	08–07 04:35
1978	29.9	06–25 16:00	41.3	06–25 21:19	2003	69.3	08–24 20:00	78.4	08–24 20:01
1979	31.0	08–02 12:00	52.1	08–02 09:24	2004	50.3	07–06 19:05	62.5	07–06 18:36
1980	22.4	07–14 05:04	36.8	07–14 05:52	***2005***	***61.2***	***06–26 22:00***	***69.0***	***07–28 03:46***
1981	***31.8***	***07–12 08:00***	***41.3***	***07–01 21:26***	***2006***	***59.5***	***07–12 10:00***	***67.6***	***07–16 00:56***
1982	54.4	07–27 02:00	70.6	07–27 02:06	2007	24.9	08–04 12:00	48.5	08–04 11:54
1983	69.6	09–02 06:00	75.4	09–02 05:16	***2008***	***35.2***	***06–02 20:00***	***54.6***	***07–24 05:04***
1984	64.9	09–01 06:54	73.5	09–01 06:17	2009	57.5	07–09 14:00	62.1	07–09 14:17
1985	64.9	08–16 17:00	75.2	08–16 16:41	2010	81.6	09–21 15:05	83.3	09–21 14:42

6. Conclusions

The flood quantile estimation methods currently being used have critical limitations and assumptions. Statistical approach requires representative flood records and has the limitation that the measured flood data must be adjusted due to changes in the characteristics of the watershed. The design storm method has some limitations such as the selection of the design rainfall hyetograph (rainfall duration and time distribution), the selection of antecedent soil moisture conditions, and the equality of the return period between rainfall quantiles and computed flood quantiles. While the continuous simulation method is assessed to be an effective tool to replace the above two methods, this method cannot be applied in small-scale watersheds with a quick hydrologic response such as in small urban watersheds because of numerous input data and long computational time. In this study, the FASAP method that minimizes the limitations and assumptions of the design storm and the continuous simulation methods was proposed to estimate the flood quantiles of a small urban watershed. The proposed method was applied to estimate the flood quantiles of the Hyoja drainage basin in Seoul using the urban runoff model XP-SWMM 2011. The conclusions of this research are as follows.

(1) To extract and separate the rainfall events the MIT was initially set to 1 h. After applying the studied urban watershed, the remaining runoff was maintained at less than 1 m^3/s after 3 h from the end of rainfall. Thus, setting the MIT to 3 h was determined as the rule for candidate rainfall event separation.

(2) The estimated flood quantiles based on the time step orders of 5 min and 1 h showed that the differences between these time steps was between −29.09% and +28.44%. Estimating appropriate flood quantiles with the time step order of 1 h was not possible in a small urban watershed with a quick hydrologic response, which is the time step usually used in the continuous simulation method.

(3) To review the appropriate sliding duration, sliding durations of 1-h, 2-h, and 3-h were tested, and the results from comparing the flood quantiles estimated by setting the sliding duration and time interval at 10 min showed less than 1% differences between the GEV and GLO distributions. Setting the sliding duration and time interval at 1 h was appropriate for a small urban watershed.

(4) The estimated flood quantiles from the proposed FASAP method were approximately 10% smaller than those quantiles from the conventional design storm method for the GEV distribution. This result is consistent with existing research results that compare the flood quantiles using statistical approach, the design storm method, and the continuous simulation method.

(5) The peak times of FASAP were in good agreement with those obtained by continuous simulation. This is a results of comparison between continuous simulation with 1 h time step and FASAP with 5 min time step. If they had a same level of 5 min time step order the accordance rate would be expected to be higher than this results. However, the results indicate that FASAP could find the time when annual maximum peak flows were occurred successfully and could be regarded as an appropriate method to estimate design flood.

Considering the quantity and quality of flood data amassed up until now, the statistical characteristics of rainfall data and flood data, and the reliability of the rainfall-runoff model, a perfect method to evaluate design floods is still lacking. Some assumptions and limitations are necessary in representing the real world with a numerical model. If the number of such assumptions can be reduced, then the uncertainties of models can also be reduced for estimating flood quantiles. The proposed FASAP method eliminates the assumption of equality of the return periods between rainfall and runoff, which is generally considered an invalid assumption. Additionally, this method does not need any assumptions for the antecedent moisture condition and the temporal distribution and duration of rainfall. The proposed

FASAP method could not be a general methodology to estimate design flood, because it is not effective method for large basin where continuous simulation make good results. However FASAP enables the concept of the continuous simulation, which is the most advanced method to determine annual maximum peak flows and estimate design floods, to be applied to urban watersheds where continuous simulation cannot be applied. Therefore, it would be useful to estimate rational design flood in ungauged urban watershed.

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Author Contributions

Jeonghwan Ahn established the methodology of the research and wrote the first edition of the manuscript; Woncheol Cho designed the numerical simulation; Taereem Kim collected the dataset and calibrated the numerical model; Hongjoon Shin contributed to the statistical analysis; and Jun-Haeng Heo analyzed the results and wrote paper.

Conflicts of Interest

The authors declare no conflict of interest.

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