Journal of Hydrology (2008) **362**, 320–336

available at www.sciencedirect.com

ScienceDirect

journal homepage: www.elsevier.com/locate/jhydrol

Identifying peak-imperviousness-recurrence relationships on a growing-impervious watershed, Taiwan

Shao-yang Huang [a], Shin-jen Cheng [b,*], Jet-chau Wen [c], Ju-huang Lee [d]

[a] *Graduate School of Engineering Science and Technology, National Yunlin University of Science and Technology, Douliou, Yunlin 640, Taiwan*
[b] *Department of Environment and Resources Engineering, Diwan University, 87-1, Nan-shih Li, Madou 721, Tainan, Taiwan*
[c] *Department of Safety Health and Environmental Engineering, National Yunlin University of Science and Technology, Douliou, Yunlin 640, Taiwan*
[d] *Water Resources Agency, Ministry of Economic Affairs, Taipei 106, Taiwan*

Received 29 December 2007; received in revised form 21 September 2008; accepted 24 September 2008

KEYWORDS
Block Kriging;
Curve number;
Hydrological parameter;
Peak discharge;
Imperviousness;
Recurrence interval

Summary This study focuses mainly on increases in peak discharges with reductions in recurrence intervals for design hydrographs due to growing imperviousness. Rainfall–runoff simulation is a major basis for analyzing the hydrological effects of urbanization. Available recordings of 50 rainfall–runoff events during 1966–2002 were used as the study sample. Forty events were calibrated to determine the relationships between impervious areas and hydrological parameters in the Nash and SCS models. Block Kriging and non-linear programming methods were used to estimate the mean rainfall and its hourly excess value, respectively. The remaining 10 cases were used to test the established relationships. Calibration and verification results confirm that the methods used in this study effectively illustrate the hydrological and geomorphic conditions in complex urbanization processes. The rainfall–runoff routings, by using the design storm approach and the established relationships, demonstrated the following: (1) peak flows of the design flood hydrographs increased by about 127, 266, 375, 440, 515, 564, 593, and 629 m^3/s for eight return periods, respectively; (2) peak times were individually shortened from 9 to 6 h due to urbanization; (3) the recurrence intervals of 200, 100, 50, and 25 years before urbanization were reduced to about 88, 33, 16, and 8 years also due to urbanization if they would have occurred at the present time within the Wu–Tu watershed. Finally, these analytical results can be obtained easily from the designed diagram that shows the relationships

* Corresponding author. Tel.: +886 6 5718888x878; fax: +886 6 5718014.
E-mail address: sjcheng@dwu.edu.tw (S.-j. Cheng).

0022-1694/$ - see front matter
doi:10.1016/j.jhydrol.2008.09.002

among peak discharges, impervious areas and return periods. This research can be used to prevent disasters, loss of life and property damage.

Introduction

For several decades, populations have gradually occupied the downstream areas of basins, and tribal societies and cities have subsequently developed. The development of urban areas within a watershed is usually accompanied by drastic changes of land use, which generally alter the hydrological functions of that area (Simmons and Reynolds, 1982; Ferguson and Suckling, 1990; Leopold, 1991; Sala and Inbar, 1992; Singh, 1998; Gremillion et al., 2000). As hydrologists acknowledge, urbanization in a basin brings growth of impervious paving that prevents rainwater from accessing the land (Chow et al., 1988). The part of a watershed contributing to surface runoff is proportional to the amount of impervious areas (Brown, 1988; Boyd et al., 1994; Arnold and Gibbons, 1996; Matheussen et al., 2000; Cheng and Wang, 2002; Cheng et al., 2008; Huang et al., 2008).

Reduced flow time and volume increments and peak discharge for surface runoff are familiar problems in urban stormwater management. Surface runoff modeling derived from an instantaneous unit hydrograph (IUH) is a worthy technology used to solve these urbanization problems (Bonta et al., 1997; Kang et al., 1998; Junil et al., 1999; Wong and Li, 1999; White et al., 2002). These models generally have various kernel functions such as a geomorphologic IUH (Franchini and O'Connell, 1996), variable source area modeling from the TOP model (Valeo and Moin, 2000), tank model (Yue and Hashino, 2000; Lee and Singh, 2005), a conceptual linear reservoir (Hannah and Gurnell, 2001; Cheng and Wang, 2002), and morphological IUHs (Rodriguez et al., 2003). System analysis is increasingly used to understand and develop solutions to complex urban problems. It is also advantageous when applied to flood hydrographs characteristics such as runoff volume, flow rates, and urban storm runoff.

Problems encountered in urban systems should be analyzed to account for spatial and temporal variations. To date, lumped runoff modeling remains a useful tool for studying changes to an outlet hydrograph on a watershed. Lumped runoff modeling can be applied to explore changes from the past to the present by ignoring some spatial variations. Both simulation modeling and the design storm approach are frequently used to estimate the magnitude and frequency of flooding in urban areas. A design flood hydrograph is then typically used to evaluate hydrological effects of stormwater runoff on land with increased imperviousness. These combined effects include increased runoff volume, reduced flow time and, especially, an increase in peak discharges with a resulting shift in the flood frequency curve (Hollis, 1975; Moscrip and Montgomery, 1997; Moon et al., 2004). A combination question of urbanization results such as an increment in hydrograph peak, shortening of time to peak, and reduction of design criteria of flood, should be solved for water resources management in Taiwan.

Given the unique topography of the bowl-shaped study watershed, when a larger storm occurs, enormous overland runoff flows rapidly into the downstream is frequently inundated. The imperviousness of the downstream watershed still is developed, resulting in more massive and swifter runoff than that in the past. As a result, preventing flood disasters has become an essential and immediate concern. In flood prevention, changes to runoff hydrographs, such as volume, peak discharge, and time to peak undergoing urbanization must be identified. Hence, this study generates hydrographs of surface runoff based on urban hydrology.

This study has two important objectives. The first is to determine the tendencies of significant parameters that reflect the growth of impervious areas. The second is to draw and design an applicable diagram for relation among peak, imperviousness and recurrence. Fig. 1 shows the research processes. Fig. 1 also shows a simulated runoff hydrograph, the development of which is the main goal of this study. This study applied popular approaches such as block Kriging, non-linear programming, SCS, simple lumped modeling, and the design storm method. The design storm was used to unify rainfall–runoff events with specific durations and recurrence intervals. The process of rainfall translating runoff is given by parameters n and k in the Nash model and CN in the SCS model. These parameters vary with different degrees of imperviousness, and were confirmed by calibration and verification using three criteria. Finally, all changes to increased peak flows and reduced recurrence intervals were integrated into a designed diagram. This diagram is helpful when managing water resources by referring to the relationships among peak discharges, impervious areas and return periods.

Methods

The block Kriging method

Block Kriging, originally developed by Matheron (1971) and frequently applied in various research fields (Lebel and Bastin, 1985; Lebel et al., 1987; Goovaerts, 2000; Syed et al., 2003; Cheng et al., 2007), was used in this study to compute mean rainfall.

The estimator of hourly mean rainfall, Z_K^*, is a linear combination of n available point-rainfall recordings $Z(x_i)$ located at x_i and with weightings λ_i. The Kriging estimator can be expressed as

$$Z_K^* = \sum_{i=1}^{n} \lambda_i Z(x_i) \tag{1}$$

Generally, the optimal weightings, λ_i, are computed from the block Kriging system based on Lagrange's multipliers method (Wackernagel, 1998; Chiles and Delfiner, 1999) and expressed as

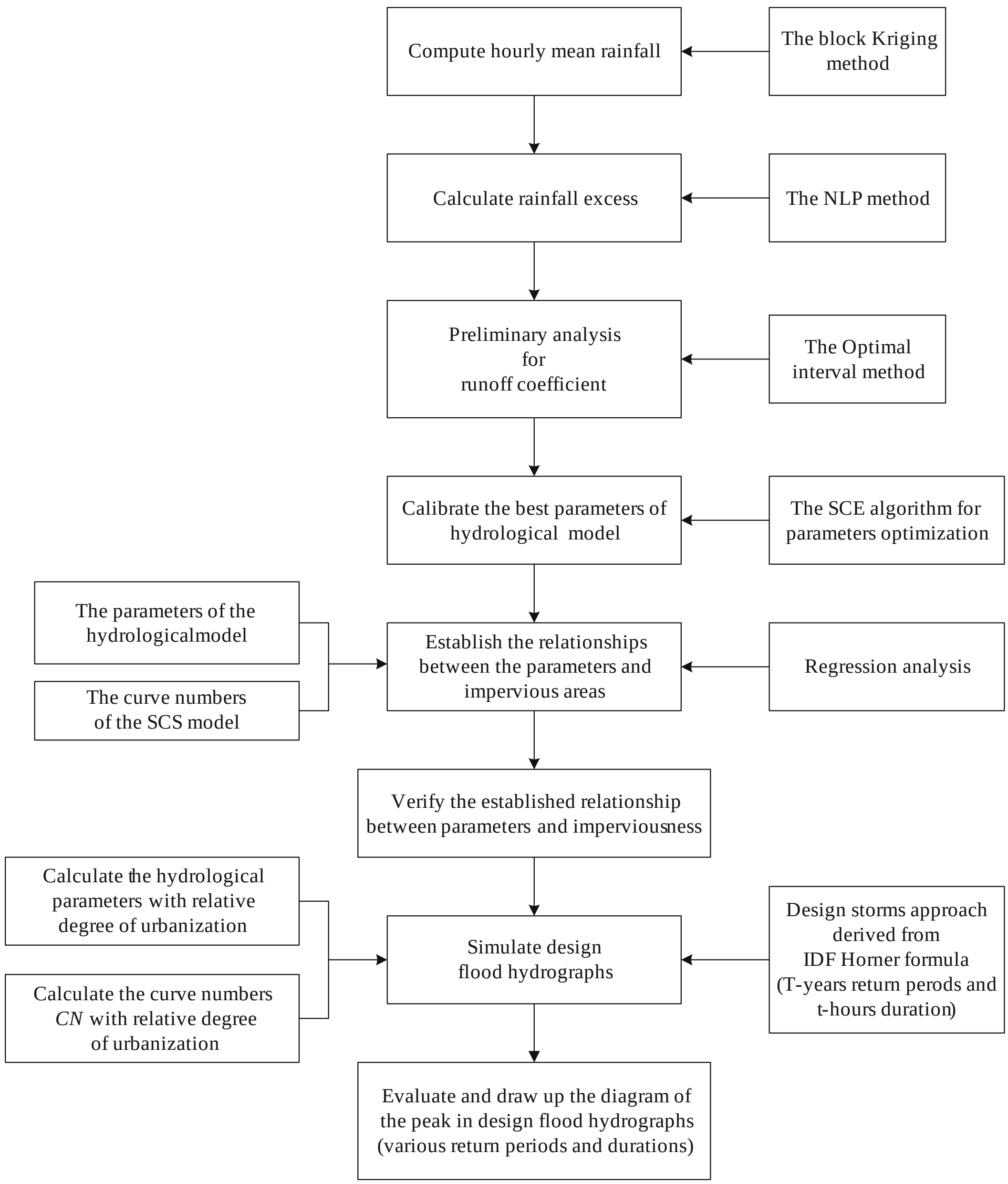

Figure 1 The flowchart of the research procedure of this study.

$$\begin{cases} \sum_{j=1}^{n} \lambda_j \gamma(x_i, x_j) + \mu = \bar{\gamma}(V, x_i), & i = 1, 2, \ldots, n \\ \sum_{i=1}^{n} \lambda_i = 1 \end{cases} \tag{2}$$

$$\sigma_K^2 = \sum_{i=1}^{n} \lambda_i \bar{\gamma}(V, x_i) + \mu \tag{3}$$

where $\gamma(x_i, x_j)$ is the semivariogram of raingauges x_i and x_j (mm^2), $\bar{\gamma}(V, x_i)$ is the average semivariogram of estimated area V and raingauge x_i (mm^2), λ_i is the weighting of raingauge x_i, σ_K^2 is the Kriging estimated variance (mm^2), and μ denotes Lagrange's multipliers (mm^2).

For practical applications, dimensionless rainfall data was used to establish the basic semivariogram of a project basin (Cheng et al., 2007). The basic experimental semivariogram is

$$\gamma(t, h_{ij}) = s^2(t)\gamma_d^*(h_{ij}, a) \tag{4}$$

in which

$$\gamma_d^*(h_{ij}, a) = \frac{1}{2T} \sum_{t=1}^{T} \left\{ \left[\frac{p(t, x_i) - p(t, x_j)}{s(t)} \right]^2 \right\} \tag{5}$$

where $\gamma_d^*(h_{ij}, a)$ is the scaled climatological mean semivariogram (mm^2), which is time-invariant; h_{ij} is the distance between arbitrary raingauges x_i and x_j (m); a is the range of the scaled climatological mean semivariogram (m); $p(t, x_i)$ is rainfall measured by raingauge x_i for time period t (mm); T is total duration of all rainfall events (h); and $s(t)$

is the standard deviation of rainfall at all raingauges for time period t (mm).

Non-linear programming

Cheng and Wang (2002) demonstrated that the time-variant distribution of excess rainfall results from non-linear programming (NLP). Notably, NLP method is helpful for acquiring appropriate hydrological parameters to illustrate urbanization characteristics of specific watersheds. NLP method is unlike the Φ-index method (Chow et al., 1988), which only obtains a time-invariant value of rainfall loss. Thus, NLP was used to calculate the excess amounts of hourly mean rainfall.

Inputs and outputs are total hourly rainfall I_t and direct runoff Q_t of an event, respectively. Rainfall losses H_t, unit hydrograph U_t, and estimated errors Z_t and V_t are all decision variables. The objective function is composed of the minimized summation of estimated errors Z_t and V_t where Z_t represents the deviation of the t-th time period for the simulated direct runoff below the observed direct runoff, and V_t is the simulation above the observation at the t-th period. The constraint equations include the discrete convolution integral, volume of effective rainfall equal to direct runoff, volume of the unit hydrograph is one and other non-negative constraints. For the detailed expressions of NLP, refer to by Cheng and Wang (2002).

The SCS model

The value range of a curve number, CN, is 0–100 (Chow et al., 1988). A high curve number implies that potential watershed retention of stormwater is reduced and, consequently, potential runoff volume is increased. This study applied the SCS model with the following equations (Tsihrintzis and Hamid, 1997) to estimate the cumulative runoff depth based on cumulative rainfall depth:

$$\begin{cases} \sum q_t = \frac{\left(\sum r_t - KS\right)^2}{\sum r_t + (1-K)S}, & \sum r_t > KS \\ \sum q_t = 0, & \sum r_t \leqslant KS \end{cases} \tag{6}$$

and

$$S = \frac{1000}{\text{CN}} - 10 \tag{7}$$

where Σq_t is cumulative runoff depth at time period t, Σr_t is cumulative rainfall depth at time period t, S is potential maximum retention or ultimate storage capacity of soil, and CN is the curve number. The term $I_a = KS$ represents the initial abstraction of rainfall by infiltration, surface storage, and interception. The value for K is typically taken as 0.2. All variable units are depths (inches), except for CN, which is dimensionless. The instantaneous runoff depth q_t (inches) and discharge Q_t (cfs) for a time period t are given by

$$q_t = \sum q_t - \sum q_{t-1} \tag{8}$$

$$Q_t = \alpha \frac{q_t}{\Delta t} A \tag{9}$$

where A is the watershed drainage area (acre), Δt is the time interval over a total duration (h), and α is a unit conversion factor.

The lumped hydrological model

The conceptual models derived from an instantaneous unit hydrograph (IUH) generally possess special definitions for parameters. These parameters with physical significances are conveniently used to represent the hydrological status of urbanized watersheds at different periods. The general form of the IUH U_n from the n-th linear cascaded reservoir with different storage constants k_n and time period t can be expressed as (Hsieh and Wang, 1999)

$$U_n(t) = \int_0^t U_{n-1}(\tau)\frac{1}{k_n}e^{-\frac{t-\tau}{k_n}}d\tau$$

$$\begin{cases} \frac{1}{k_1}e^{-\frac{t}{k_1}}, & n = 1 \\ \sum_{i=1}^{n} \frac{k_i^{n-2}}{\prod_{j=1,j\neq i}^{n}(k_i-k_j)} e^{-\frac{t}{k_i}}, & n \geqslant 2 \end{cases} \tag{10}$$

A common case of the above expression assumes that the storage coefficients for all of the linear cascaded reservoirs have the same value. The instantaneous unit hydrograph of n linear cascaded reservoirs for one SI unit of effective rainfall (Nash, 1957) is

$$U(t) = \frac{1}{k\Gamma(n)}e^{-\frac{t}{k}}\left(\frac{t}{k}\right)^{n-1} \tag{11}$$

The design storm approach

The design storm is mainly based on the concept of the probability distribution and is generally produced from the intensity-duration-frequency (IDF) curves or from other statistical means from rainfall records. The alternating block method, instantaneous intensity method, and triangular hyetograph method (Chow et al., 1988) are usually used to yield a reasonable design storm. The design storm is frequently coupled with the rational formula or a unit hydrograph method to simulate the design flood hydrograph for water resources planning. However, these approaches neglect the storage carryover effect that may exist in a drainage system by ignoring the time interval between storms (Chow et al., 1988).

Model evaluation

To evaluate the model's suitability for the basin of interest, three criteria were chosen to analyze the degree of goodness-of-fit. These criteria are as follows:

1. Coefficient of efficiency, CE, is defined as

$$\text{CE} = 1 - \frac{\sum_{t=1}^{T}[Q_{\text{est}}(t) - Q_{\text{obs}}(t)]^2}{\sum_{t=1}^{T}[Q_{\text{obs}}(t) - \bar{Q}_{\text{obs}}(t)]^2} \tag{12}$$

where $Q_{\text{est}}(t)$ denotes the discharge of the simulated hydrograph for time period t (m^3/s), $Q_{\text{obs}}(t)$ is the discharge of the observed hydrograph for time period t (m^3/s), and $\bar{Q}_{\text{obs}}(t)$ is the average discharge of the observed hydrograph for time period t (m^3/s). The better the fit, the closer CE is to one.

2. The error of peak discharge, $EQ_p(\%)$, is defined as

$$EQ_p(\%) = \frac{Q_{p,est} - Q_{p,obs}}{Q_{p,obs}} \times 100\% \tag{13}$$

where $Q_{p,est}$ is the peak discharge of the simulated hydrograph (m^3/s), and $Q_{p,obs}$ is the peak discharge of the observed hydrograph (m^3/s).

3. The error of the time for peak to arrive, ET_p, is defined as

$$ET_p = T_{p,est} - T_{p,obs} \tag{14}$$

where $T_{p,est}$ denotes the time for the peak arrival (h) of simulated hydrographs, and $T_{p,obs}$ is the time (h) required for the arrival of peaks in observed hydrographs.

Watershed description

Geographical features

The Tanshui River is the third longest river in Taiwan (Fig. 2). The Tanshui River Basin covers the Taipei Pan, which passes through the Da-Han Stream, Hsin-Tien Stream and Kee-Lung River. The Tanshui River system is 159 km long and has a drainage area of 2726 km^2. Taipei City and Taipei County stand on the Taipei Pan with a total population of over five million.

The Wu–Tu watershed is located on upstream of the Kee-Lung River. This watershed was chosen to study the changes of peak discharges in flood hydrographs undergoing urbanization. The watershed surrounds Taipei City, which is in northern Taiwan (Fig. 3). The Wu–Tu watershed covers about 204 km^2. Mean annual precipitation and runoff depth in the watershed are 2865 mm and 2177 mm, respectively. This watershed consists of large pervious area (high mountains) and a smaller impervious area (watershed downstream), i.e., the greater part of runoff contributing area is from pervious area. Due to the rugged topography of the watershed, runoff pathlines are short and steep, and rainfall is not uniform in terms of both time and space. Large floods occur rapidly in the middle-to-downstream reaches of the watershed, causing serious damage during summer.

Hydrological data

Fourteen raingauges are located along the Tanshui River, in which there are three raingauges (Jui-Fang, Wu–Tu, and Huo-Shao-Liao), and one discharge site (Wu–Tu) within the Wu–Tu watershed. Available records for 50 rainfall–runoff events during 1966–2002 were used as the study sample. Table 1 gives the main characteristics of these events. The annual data for population density and imperviousness percentage serve as the degree of urbanization in the research area.

In this study, the impervious area was considered as a major cause of urbanization, and its definition is one in which all rainfall falling on the surface is presumed to produce surface runoff. The annual imperviousness percentage was computed from the above definition and applied to streets, roads, railroad lines, highways, roofs, buildings, parking lots, ponds and lakes, and waterways for each year. Table 2 lists the population density and percentage of impervious areas in the Wu–Tu watershed during 1966–2002. Notably, an obvious correlation exists between population and imperviousness. This percentage is convenient for approximating the extent of urbanization and demonstrating the significance of hydrological consequences of ongoing urbanization of the Wu–Tu watershed.

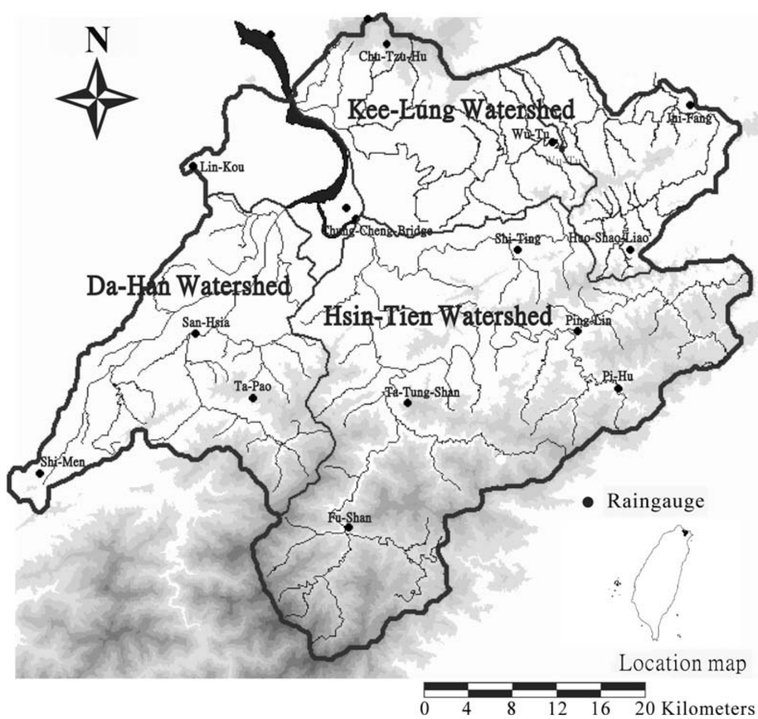

Figure 2 The distribution map of raingauges in Tamshui River Basin.

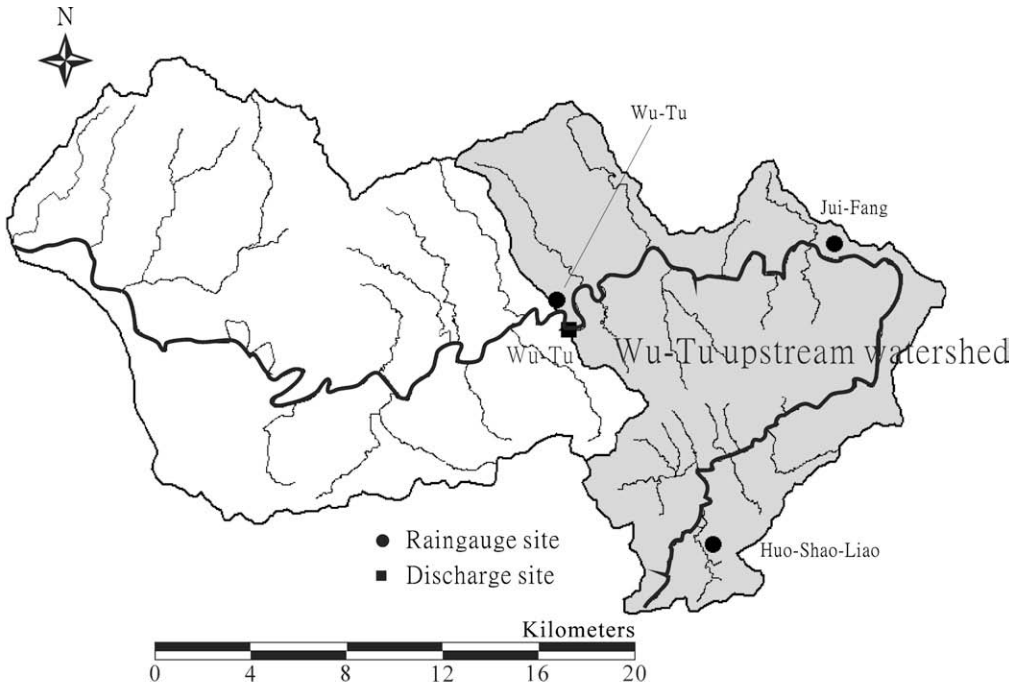

Figure 3 Location map and observation sites of the Wu—Tu watershed.

Hydrological parameters and growing imperviousness

Identification of hydrological parameters that respond to the growth of impervious areas in an urbanized region is a major goal of this study. Thus, storm modeling is used as the main tool. The SCS method and a conceptual hydrological model were also utilized. The relationships between impervious areas and parameters, including hydrological and SCS models, were established to understand the different statuses of urban development.

Hourly mean rainfall

The set of time sequences for discontinuous point-rainfall depths $p(t,x)$ can be viewed as a realization of two-dimensional random fields. Considering n raingauges in a river basin, for each time period t, a realization $\pi(t)$ of the random n vector can be expressed as

$$\pi(t) = \{p(t,x_1), p(t,x_2), \ldots, p(t,x_n)\} \tag{15}$$

The semivariogram of hourly rainfall is a function of time period t, isotropy, and a time average form with nonzero and T time intervals. The calculated results of climatological mean semivariogram for hourly recordings from 14 raingauges for the 50 selected rainfall events within the Tanshui River Basin and the power form applied for fitting are as follows:

$$\gamma_d^*(h_{ij}, a) = \omega_0 h^a = 0.137h^{0.209}, \quad r^2 = 0.834 \tag{16}$$

where ω_0 is the sill of the scaled climatological mean semivariogram (mm^2). Variance $s^2(t)$ of a realization $\pi(t)$ for each time period t can be easily calculated from hourly rainfall measurements. Hourly semivariograms of rainfall events can then be directly calculated using Eqs. (4) and (16).

Change of runoff coefficient

In a preliminary urbanization analysis, direct runoff for the 50 chosen events was calculated by assuming that the baseflow is constant. The hourly mean rainfall computed from the three raingauges was completed to acquire its excess values using NLP method. Hence, changes in degree of urbanization can be observed using the runoff coefficient. Hydrologists acknowledge that a rainfall—runoff event is independent. This truth can be found based on the seemingly disorderly and unsystematic values of the runoff coefficient calculated from the 50 cases. Therefore, this study used the optimal interval method to smooth the behavior of the runoff coefficient with different degree of imperviousness.

The primary procedure of the optimal interval method is to determine a suitable interval based on variations in parameters and impervious areas from R^2 results of regression analysis. Restated, these hydrological parameters and imperviousness in the same determined intervals were viewed as having identical average values. By this method, a smooth/clear behavior of different degree of urbanization can be observed easily over the years. This analytical result was plotted (Fig. 4) and proves that urbanization exists in the Wu—Tu watershed.

Calibrations of hydrological parameters and curve numbers

In the optimal calibration process, model parameters reflecting change to land use were obtained through the shuffled complex evolution (SCE) optimal algorithm (Duan and Gupta, 1993; Sorooshian et al., 1993). Table 3 presents comparisons of simulated and observed runoff hydrographs using three criteria CE, EQ_p and ET_p. Figs. 5 and 6 plot two of the calibrated rainfall—runoff events.

Table 1 Main characteristics of selected rainfall–runoff events

Event name (occurred time)	Rainfall duration (h)	Maximum rainfall (mm)	Rainfall amount (mm)	Maximum discharge (m^3/s)
Storm (1966.06.20)	6	62.2	225.7	641.0
ALICE (1966.09.02)	18	24.3	117.4	106.0
CORA (1966.09.06)	16	22.3	155.6	827.0
ELSIE (1966.09.13)	95	29.9	636.4	892.0
GILDA (1967.11.16)	59	32.0	339.5	739.0
NADINE (1968.07.26)	24	19.0	116.1	269.0
BETTY (1969.08.07)	11	30.0	110.0	355.0
Storm (1969.08.09)	6	29.0	50.0	96.0
BETTY (1972.08.16)	40	15.2	177.2	708.0
JEAN (1974.07.19)	17	17.4	79.0	116.0
BESS (1974.10.11)	63	27.4	470.7	726.0
NINA (1975.08.04)	7	23.4	63.3	187.0
BILLIE (1976.08.09)	15	14.2	80.9	260.0
Storm (1976.08.11)	5	57.4	94.7	48.8
Storm (1976.09.16)	47	22.5	128.2	161.0
VERA (1977.07.31)	22	18.8	163.7	758.0
Storm (1977.11.15)	56	15.5	260.6	587.0
ANDY (1982.07.29)	35	17.5	165.4	364.0
CECIL (1982.08.09)	34	25.8	235.7	682.0
Storm (1984.06.02)	18	48.7	212.7	1420.0
FREDA (1984.08.06)	30	44.9	242.1	509.0
GERALD (1984.08.14)	119	25.2	497.2	600.0
BILL (1984.11.18)	52	16.0	204.9	401.0
ALEX (1987.07.27)	18	47.9	155.8	527.0
LYNN (1987.10.23)	113	68.1	1605.9	1980.0
Storm (1988.09.29)	91	26.3	614.3	734.0
SARAH (1989.09.10)	51	29.8	262.5	494.0
OFELIA (1990.06.22)	30	22.7	165.7	535.0
ABE (1990.08.30)	21	16.6	181.7	789.0
Storm (1990.09.01)	11	42.8	111.8	327.0
Storm (1990.09.02)	22	32.8	184.5	857.0
Storm (1991.09.22)	20	21.0	166.5	109.0
NAT (1991.09.29)	24	17.1	134.7	340.0
RUTH (1991.10.28)	72	26.1	448.6	583.0
Storm (1994.06.18)	12	82.0	150.6	532.0
DOUG (1994.08.07)	32	17.4	162.0	344.0
GLADYS (1994.09.01)	18	40.4	184.1	439.0
HERB (1996.07.31)	27	32.1	265.6	1090.0
ZANE (1996.09.27)	66	31.5	377.8	678.0
CASS (1997.08.29)	26	32.8	250.4	960.0
Storm (1998.10.04)	83	23.0	362.5	428.0
ZEB (1998.10.15)	44	50.7	644.6	1050.0
Storm (1999.12.13)	154	12.9	401.8	238.0
BILIS (2000.08.22)	54	52.2	227.0	350.0
BEBINCA (2000.11.08)	62	20.2	332.5	632.0
Storm (2000.12.13)	40	11.0	170.4	364.0
Storm (2000.12.19)	41	13.4	129.1	250.0
NARI (2001.09.16)	72	74.5	978.6	2040.0
LEKIMA (2001.09.25)	108	18.2	458.9	441.0
RAMMASUN (2002.07.04)	17	19.9	115.5	149.0

Table 1 describes characteristics of 50 chosen events. They include rainfall duration, maximum rainfall, rainfall amount, and peak discharge.

For the CE for model calibration, all calibrated events exceed 0.85 (Table 3). For EQ_p, 39 cases are smaller than, and one (BETTY, 1972.08.16) is slightly equal to, 20%. The ET_p values are all less than 2 h. Model calibration using the three criteria demonstrates that the calibrated parameters are adequate for illustrating the status of the study watershed during urbanization.

The increase in the number of impervious areas-parking lots, streets, and roofs-reduce the amount of infiltration. That is, storage capacity is decreased in urbanized water-

Table 2 The growths of population and imperviousness on the Wu—Tu watershed

Year	Pop. D[a] (people/km^2)	Im[b] (%)	Year	Pop. D[a] (people/km^2)	Im[b] (%)
1966	385.01	4.78	1985	437.76	7.27
1967	390.54	5.02	1986	436.68	7.33
1968	398.78	5.10	1987	434.72	7.41
1969	407.20	5.18	1988	439.27	7.59
1970	413.84	5.26	1989	452.59	7.76
1971	417.49	5.34	1990	459.68	9.59
1972	419.68	5.42	1991	470.22	10.90
1973	418.58	5.50	1992	484.38	10.95
1974	420.29	5.57	1993	506.20	11.03
1975	419.86	5.65	1994	517.06	10.65
1976	422.20	5.73	1995	537.83	10.67
1977	424.16	5.83	1996	560.12	10.27
1978	419.79	5.95	1997	540.55	10.44
1979	418.75	6.12	1998	549.31	10.52
1980	426.46	6.54	1999	558.35	10.60
1981	433.01	6.80	2000	639.33	10.92
1982	438.36	6.99	2001	649.84	10.92
1983	436.00	7.12	2002	654.80	12.46
1984	437.74	7.19			

Table 2 illustrates increasing tendencies of population and imperviousness of the Wu-Tu watershed from 1966 to 2002.
[a] Pop. D. represents population density.
[b] Im. is denotes as impervious area.

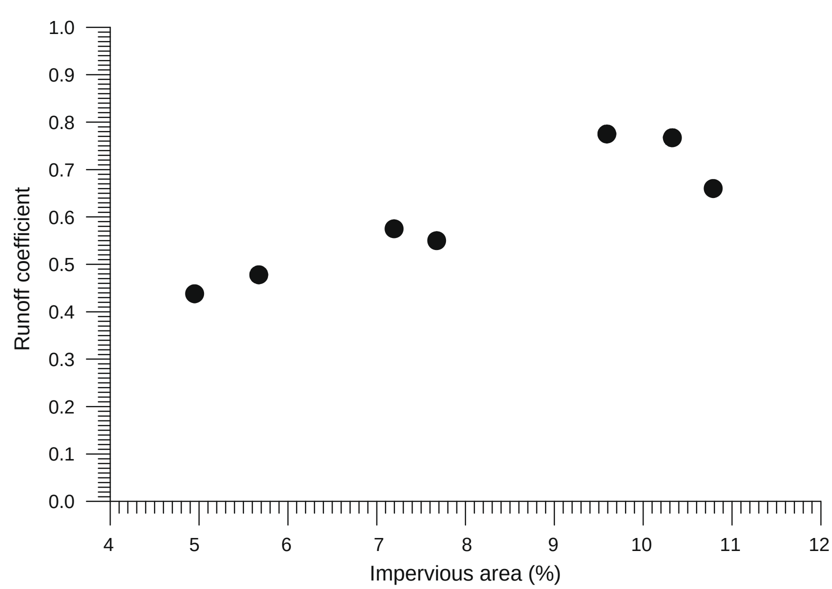

Figure 4 Change tendency of runoff coefficient for different degree of urbanization degree.

sheds. This important issue is also ascertained by hydrologists. This study utilized the SCS method and mainly using a continuous simulation to understand change of storage capacity.

Instantaneous runoff depth q_t (or hourly outlet discharge, Q_t) was adopted from the hourly streamflow observations of over 40 calibrated rainfall—runoff events. Hourly mean rainfall is the same as that computed using the block Kriging method. Finally, the composite curve number CN, which reflects the hydrological and geomorphic statuses, can be obtained using Eqs. (6)—(9) and the non-linear least square method.

The relationships between calibrated parameters and imperviousness

The relationships between calibrated parameters and impervious areas in an urbanized watershed can be analyzed from the perspective of using urban hydrology. Based on this

Table 3 The calibrated results of the selected rainfall–runoff events

Event name (occurred time)	Evaluated criteria		
	CE	EQ_p	ET_p
Storm (1966.06.20)	0.951	−1.153	0
ALICE (1966.09.02)	0.912	−2.137	0
CORA (1966.09.06)	0.993	−0.943	0
ELSIE (1966.09.13)	0.969	−12.345	1
GILDA (1967.11.16)	0.980	−4.918	0
NADINE (1968.07.26)	0.988	−2.729	1
BETTY (1969.08.07)	0.929	−11.388	0
Storm (1969.08.09)	0.883	−8.204	1
BETTY (1972.08.16)	0.948	−20.343	1
JEAN (1974.07.19)	0.970	−12.075	1
BESS (1974.10.11)	0.913	−1.606	0
NINA (1975.08.04)	0.914	−12.793	0
BILLIE (1976.08.09)	0.929	−4.938	1
Storm (1976.08.11)	0.868	−16.490	1
Storm (1976.09.16)	0.951	−13.317	0
VERA (1977.07.31)	0.940	−14.608	0
Storm (1977.11.15)	0.982	−3.304	1
ANDY (1982.07.29)	0.971	−1.690	0
CECIL (1982.08.09)	0.987	−0.534	0
Storm (1984.06.02)	0.948	−9.182	0
FREDA (1984.08.06)	0.904	−13.781	1
GERALD (1984.08.14)	0.949	−8.842	0
BILL (1984.11.18)	0.997	−0.942	1
ALEX (1987.07.27)	0.868	−4.811	−1
LYNN (1987.10.23)	0.858	−18.392	−1
Storm (1988.09.29)	0.990	−5.180	2
SARAH (1989.09.10)	0.975	−3.022	2
OFELIA (1990.06.22)	0.971	−8.420	0
ABE (1990.08.30)	0.953	−12.878	1
Storm (1990.09.01)	0.948	−12.778	1
Storm (1990.09.02)	0.900	−8.274	1
Storm (1991.09.22)	0.946	−10.194	−2
NAT (1991.09.29)	0.997	−2.788	0
RUTH (1991.10.28)	0.990	−4.733	0
Storm (1994.06.18)	0.914	−9.681	1
DOUG (1994.08.07)	0.970	−4.953	1
GLADYS (1994.09.01)	0.927	−10.260	0
HERB (1996.07.31)	0.964	−6.855	0
ZANE (1996.09.27)	0.974	−12.873	0
CASS (1997.08.29)	0.964	−6.098	0

Table 3 shows calibration results of 40 events under evaluation of three criteria *CE*, EQ_p and ET_p.

perspective, outlet hydrographs of the watershed were simulated to determine the possible effects of urbanization on rainfall–runoff relationships. In the simulated rainfall–runoff processes, some hydrological uncertainties exist, such as the influences of weather, antecedent moisture conditions, and other uncontrollable factors. These factors often cause calibrated parameters to appear irregularly with no obvious tendencies.

To avoid this possibility, this study used the optimal interval method to enable changes of the calibrated parameters and impervious areas clear. Table 4 lists the analytical results for calibrated parameters and imperviousness obtained using the optimal interval method. The power equation was used to fit the data representing the CN, and CN is assumed a function of imperviousness (Fig. 7), which is expressed as

$$\mathrm{CN} = 21.80\mathrm{Im}^{0.32}, \quad R^2 = 0.72 \tag{17}$$

where CN is the composite value of the curve number, and Im is the percentage of impervious areas in the Wu–Tu watershed.

This study found that the dispersion of parameter *n* in the hydrological model is larger than parameter *k* (Table 4). This comparison reveals that an impervious portion of the watershed is more responsive to the number of linear reservoirs than the storage coefficient in the conceptual

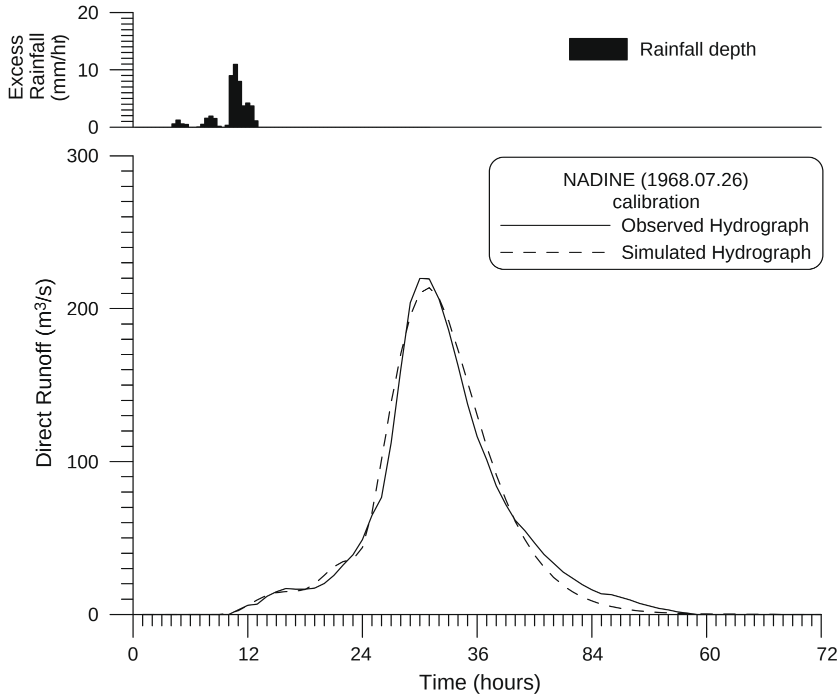

Figure 5 Calibration of observed and simulated hydrographs for NADINE typhoon.

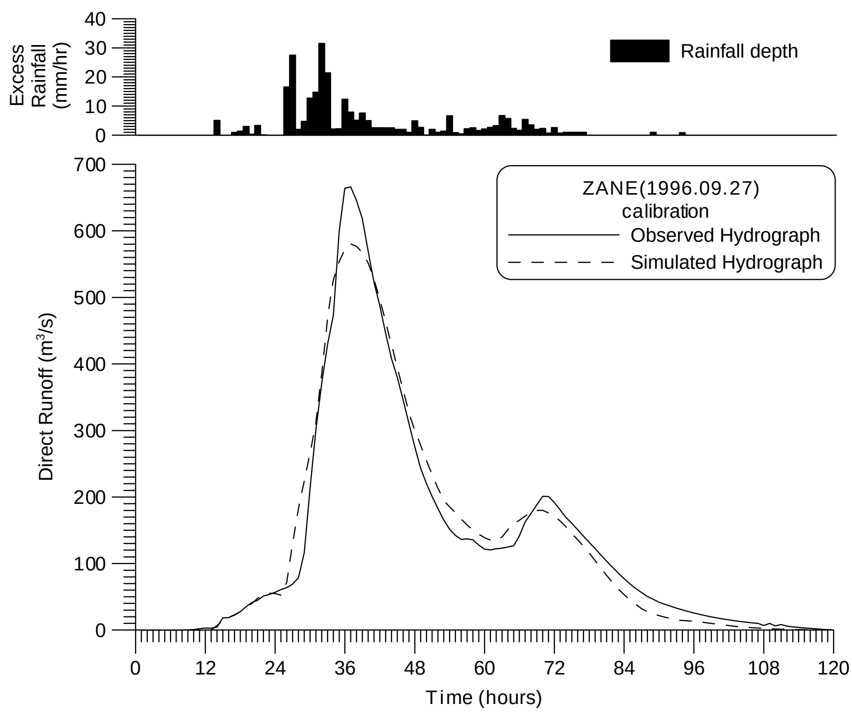

Figure 6 Calibration of observed and simulated hydrographs for ZANE typhoon.

hydrological model. This analytical result increases the range of variation of the decreased parameter n for selected samples and reduces the variation for parameter k. Hence, storage parameter k is considered a constant; its value is the average of each interval (Table 4). The mean value of parameter k is 2.201. For further application, the variation of parameter n should be provided with a continuous function. The power form of the regression equation was applied again to fit the data on the parameters n (third column in Table 4). The fitting equation (Fig. 8) is expressed as

$$n = 15.75\mathrm{Im}^{-0.57}, \quad R^2 = 0.78 \tag{18}$$

where n is the parameter of the hydrological model that represents the number of cascaded linear reservoirs.

Table 4 Changes of calibrated parameters undergoing urbanization

Impervious area (%)	SCS model	Nash model	
	CN	*n*	*k*
4.780	36.616	7.450	1.855
5.180	34.041	6.160	1.824
5.705	38.792	5.600	1.991
6.990	43.030	4.600	1.990
7.263	38.610	4.983	2.360
7.675	46.707	4.450	2.755
9.590	45.104	4.150	2.260
10.327	44.383	4.333	2.030
10.775	45.846	4.583	2.743
Mean	41.459	5.145	2.201

Table 4 displays changes of parameters of Nash and SCS models because of urbanization.

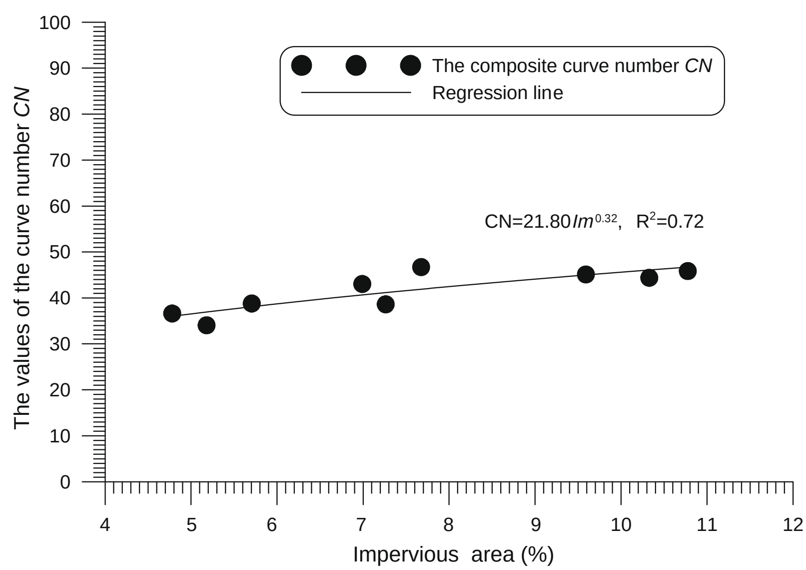

Figure 7 Change of the composite curve number with increased impervious areas.

Verification of the established functional relationships

To test and verify the usability of parameters *n* and CN in urban areas, 10 cases for 1998–2002 were chosen. The baseflow was again assumed a constant. The excess rainfall of each event was estimated using the SCS method, in which the value of the parameter CN was computed using Eq. (17) with the relative annual change in imperviousness. The hydrological model applied fixed parameter *k* ($k = 2.201$). Eq. (18) and the annual impervious percentage were used to obtain the corresponding parameter *n*. The effective rainfall resulting from the SCS model was convoluted to direct runoff. Table 5 and Figs. 9 and 10 present verification results. The values for the coefficient of efficiency of model verification exceed 0.75, except for one event (LEKIMA, 2001.09.25), and the error of peak discharge is less than 25%, except for one event (RAMMASUN, 2002.07.04). The error in the arrival time of the peak for all examined events is 3 h or less when the RAMMASUN typhoon is excluded.

Although the relationships among parameters and imperviousness still have incomplete correlations, calibration and verification still indicate that these regression equations are the major trends in the Wu–Tu watershed with ongoing urbanization. Consequently, these analytical results demonstrate that imperviousness is the primary influence of urbanization and can be further applied to other applications.

Peak-imperviousness-recurrence relationships

This study compared peak characteristics in design flood hydrographs for various impervious areas. After identifying the usable relationships among hydrological parameters and impervious areas, a diagram of was drawn. As proposed by Kibler et al. (1981), increased peak discharge causes the flood frequency curve to shift and reduce. However, the work by Kibler et al. lacked quantification. This study quan-

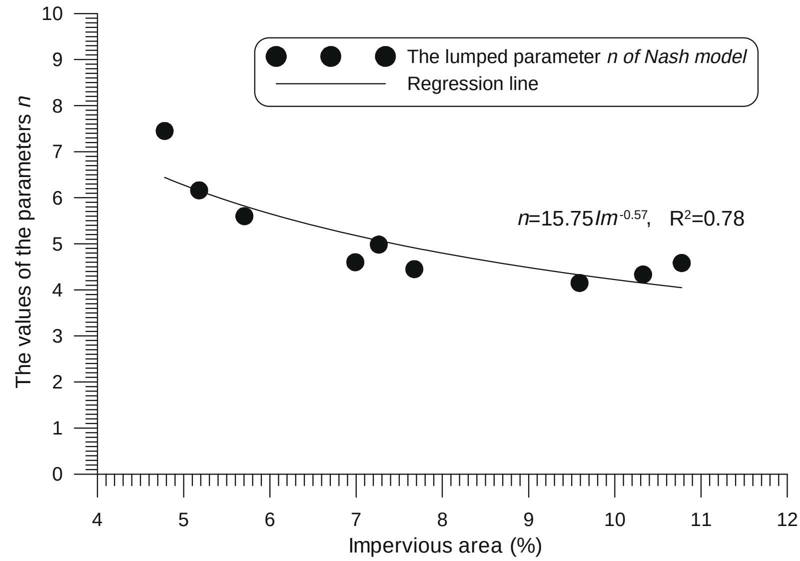

Figure 8 Decreased trend of parameter n with increased impervious areas.

Table 5 The verification of the selected rainfall—runoff events

Event name (occurred time)	Evaluated criteria		
	CE	EQ_p	ET_p
Storm (1998.10.04)	0.943	4.137	0
ZEB (1998.10.15)	0.941	5.356	1
Storm (1999.12.13)	0.915	16.492	−2
BILIS (2000.08.22)	0.787	2.901	2
BEBINCA (2000.11.08)	0.867	−7.691	2
Storm (2000.12.13)	0.955	−7.843	1
Storm (2000.12.19)	0.795	14.602	3
NARI (2001.09.16)	0.858	−10.970	2
LEKIMA (2001.09.25)	0.710	20.601	2
RAMMASUN (2002.07.04)	0.819	29.594	4

Table 5 shows verification results of 10 cases under evaluation of three criteria CE, EQ_p and ET_p.

tified peak discharges with specific rainfall magnitudes and impervious areas and presented these peak discharges in a useful diagram. This excellent result is useful when managing water resources.

Varied shapes and changed peaks of flood hydrographs

The characteristics of flood hydrographs always have been a concern in water resources management tasks such as flood-control and drainage engineering. Storms are independent and typically generate different flood disasters; these variations are complex and difficult to understand clearly. This study employs probability distribution to standardize storms. Simulation routings are considered a refined method of stormwater modeling. The study adopted the alternating block method (Chow et al., 1988) to structure the design storm at a single raingauge from the intensity-duration-frequency (IDF) curve. The duration of these design hyetographs is 48 h, and the design return periods are 1.1, 2, 5, 10, 25, 50, 100, and 200 years, respectively.

The mean design storms of the studied watershed are linear combinations computed by the block Kriging method using the design hyetographs of three raingauges. The input for the rainfall—runoff model is the effective mean design storm, which was obtained using the SCS method with its corresponding curve number and impervious surfaces, as shown in Eq. (17). Furthermore, the effective mean design storms were convoluted to surface runoff using the lumped hydrological model. Parameter n in the lumped hydrological model is a function of imperviousness, as shown in Eq. (18), and parameter k is fixed value at 2.201. These routing results for the design storms can be used to understand the shapes and peaks of outlet flood hydrographs during watershed research of various surface coverings and storm intensities.

The different shapes of hydrographs for a return period of 200 years, which represents the hydrological statuses of the watershed from that of the past (1966) to the present (2002), are shown in Fig. 11. Table 6 lists the changed peak discharges in the various design floods. The shape of the flood hydrograph becomes steeper and higher (Fig. 11).

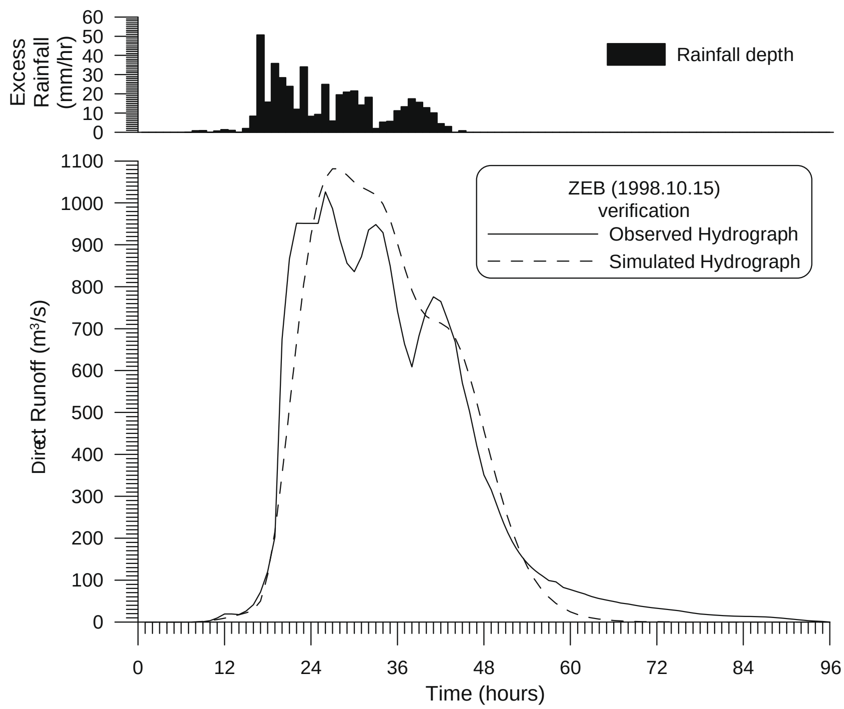

Figure 9 Verification of observed and simulated hydrographs for ZEB typhoon.

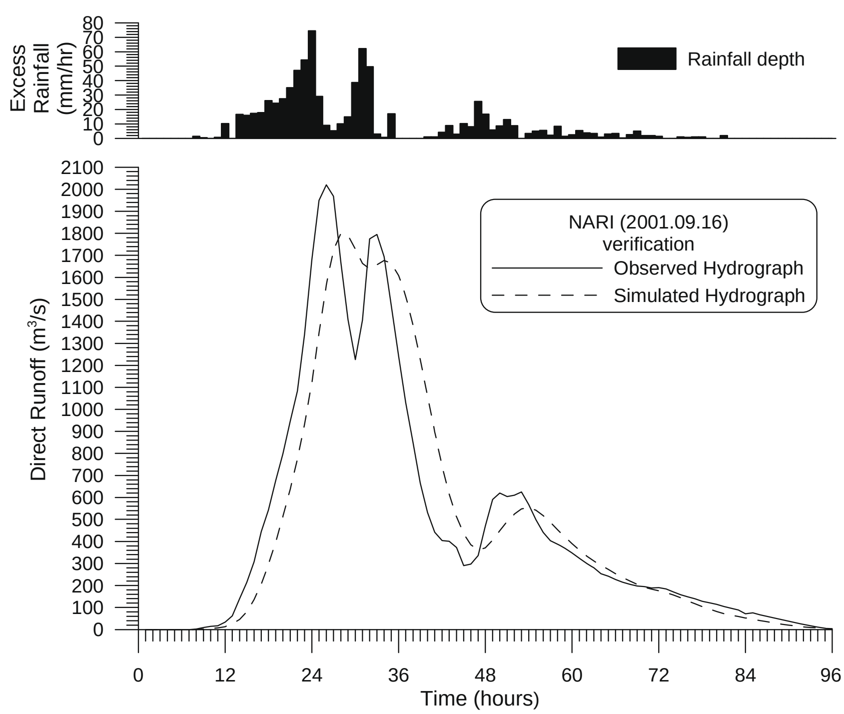

Figure 10 Verification of observed and simulated hydrographs for NARI typhoon.

The peak of flood hydrograph shifts forward due to the increase in impervious cover from that of the past to the present. The comparison in Table 6 reveals that peak flows increase to about 127, 266, 375, 440, 515, 564, 593, and 629 m^3/s for the eight return periods compared to that of natural status (1966) and urban status (2002). The peak times of the eight return periods are shortened from 9 to 6 h.

These comparison results demonstrate that the hydrological status of the study watershed was altered by urbanization. Simultaneously, the peaks in flood hydrographs from design storms at present have larger peaks than those of

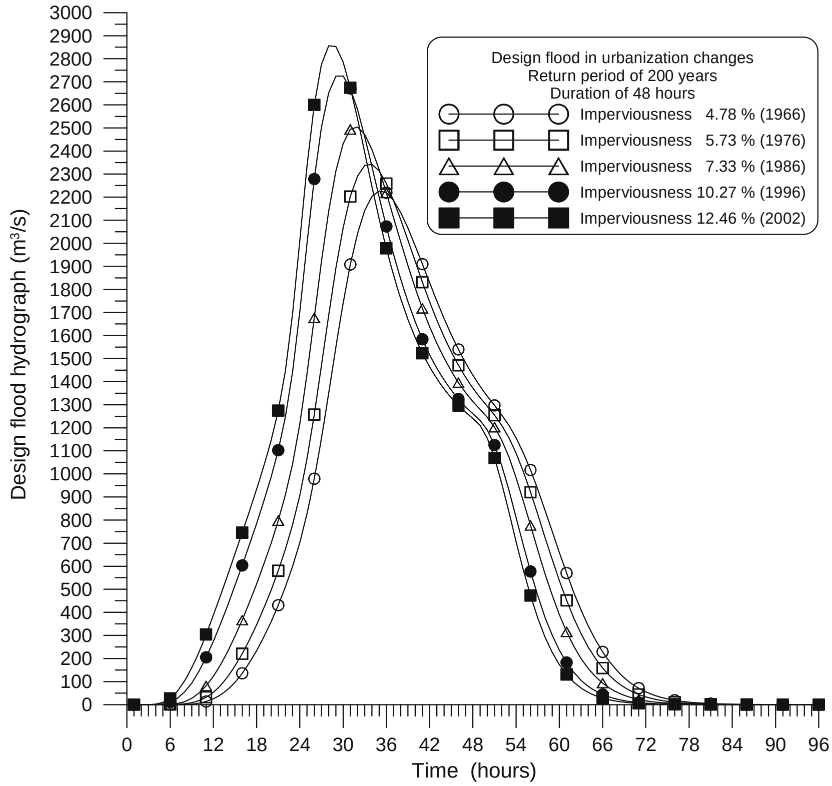

Figure 11 Shapes of design flood hydrographs for return period 200 years over the years.

Table 6 Changes of peak discharge and time to peak in the flood hydrographs for various design criteria from the natural status to the urban status

Return period (years)	Time to peak (h)			Peak flow (m^3/s)		
	Natural	Urban	Change (%)	Natural	Urban	Change (%)
1.1	41	30	−26.83	82.45	209.29	153.84
2	37	29	−21.62	282.73	549.18	94.24
5	36	29	−19.44	539.77	915.14	69.54
10	36	29	−19.44	761.83	1201.56	57.72
25	36	29	−19.44	1097.39	1612.19	46.91
50	36	29	−19.44	1381.24	1944.86	40.81
100	35	29	−17.14	1723.80	2317.23	34.43
200	34	28	−17.65	2226.93	2856.18	28.26

Table 6 explains changes of characteristics of runoff hydrograph for ongoing urbanization of Wu-Tu watershed.

past. Moreover, a large increment occurs with a storm of higher intensity (longer return period) and contrary to the changed results of time to peak.

Reduction of design criteria due to increasing peak discharge

Two problems concerning the comparison of peak discharges in flood hydrographs in the research watershed undergoing urbanization are need attention. The first problem is that as economic demand continually increases, the amount of surface runoff from impervious areas typically increases. This greatly increases the amounts of peak discharges. The second problem is that the effectiveness of the design criterion (return period) for water resource planning and management, such as flood-control or drainage engineering, will be reduced due to the effect of increased peak flows.

By understanding and solving these two problems, the growth of impervious paving is assumed as continuous. The flood hydrographs were derived from the design storms with eight return periods and a duration of 48 h. Fig. 12 presents the peak discharges in flood hydrographs resulting from the effects of different rainfall intensities on various impervious degrees of surface imperviousness.

This diagram (Fig. 12) presents the relationships among peak discharges, impervious areas, and return periods. These relationships can be used to obtain quickly the peak

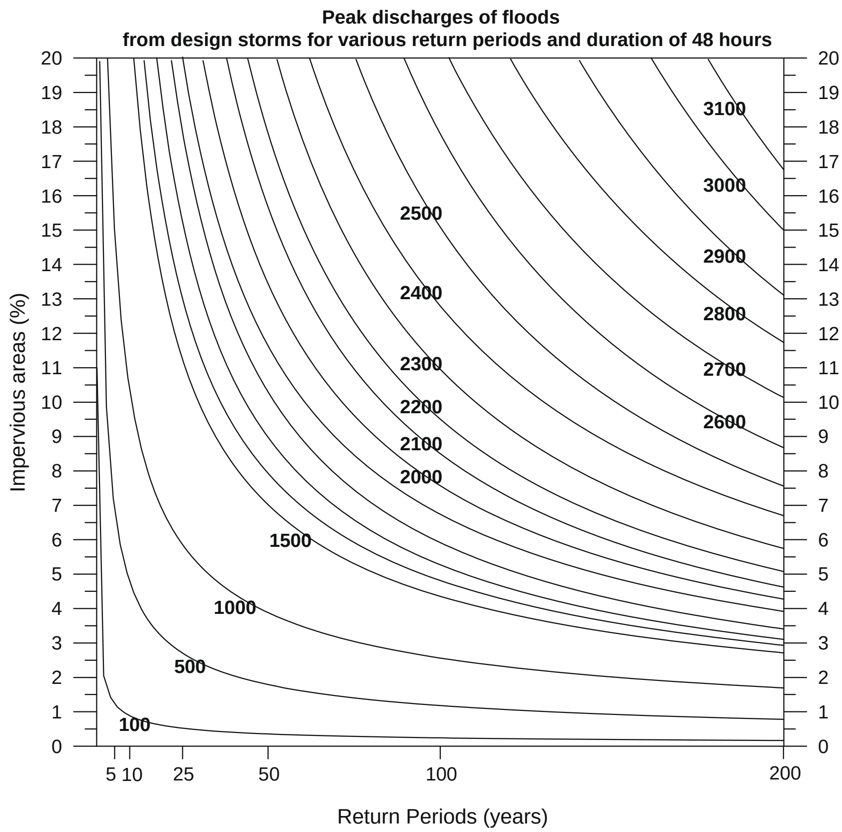

Figure 12 The diagram of peak discharge-imperviousness-return period on the Wu—Tu watershed.

discharge for a specific degree of imperviousness and return period. For example, in 1966, the flood peak for a return period of 200 years is 2227 m^3/s (Table 6), which is close to the curve for 2200 m^3/s (Fig. 12). While the imperviousness percentage has increased to 12.46% (2002), the peak discharge is 2856 m^3/s (Table 6). This value is approximately 2900 m^3/s (Fig. 12) because the imperviousness percentage increased from 4.78% to 12.46% (1966—2002). Table 6 lists the values of 2856 or 2900 m^3/s for peak flow; these values can also be directly derived using simple linear interpolation (Fig. 12).

Furthermore, due to ongoing urbanization, the original design criterion for a return period of 200 years with a peak discharge of 2227 m^3/s in the natural status becomes a return period of about 88 years. Similarly, the design criteria for 100, 50, and 25 years of peak flows in the natural status (1724, 1381, and 1097 m^3/s (Table 6)) are reduced to approximately 33, 16, and 8 years, respectively. These large reductions in high frequencies of flood peaks deserve consideration by hydrologists and engineers.

This study successfully evaluates the relationships among peak flow, impervious area, and recurrence interval using several simple models and methods. The procedure used fits with perspectives of the hydrological cycle, and can also be applied to other urbanized watersheds. By following this procedure, a positive causal relationship can be obtained for this and other watersheds. Peak discharge and its corresponding criterion for the recurrence interval can be directly found in the established diagram. This diagram is of practical importance in real hydrological analyses, especially for water resource planning and management. Other similar diagrams for different durations of design storms are also developed using this procedure.

Conclusions

This study successfully synthesized several simple hydrological methods to analyze the hydrological effects of an urbanizing watershed. It is concluded that the block Kriging and NLP methods are useful for obtaining suitable parameters that reflect the hydrological and geomorphic conditions. Establishment of the functional relationships between parameters and imperviousness using the optimal interval method can assist in determining hydrological effects of urbanization.

In the urbanization process, the parameter n in the Nash model clearly decreases as imperviousness increase, and parameter k can be assumed constant. The change of the parameter CN in the SCS model is the same as the tendency of an impervious area. These analytical results demonstrate that peak flows increased to about 127, 266, 375, 440, 515, 564, 593, and 629 m^3/s for eight return periods, respectively, from natural status (1966) to urban status (2002). The peak times of the eight return periods were shortened from 9 to 6 h due to urbanization. Consequently, a large increment exists in a design storm with a long return period and is contrary to the peak time.

Due to urbanization, increased peak discharge causes the return periods to reduce for peak flows when they occurred in the present, and peak flows remained the same as those in the past. This study found that recurrence intervals of 200, 100, 50, and 25 years prior to urbanization are reduced to about 88, 33, 16, and 8 years for peak discharges when they occurred in the present and remained the same amounts as those in the past in the Wu—Tu watershed. Two actualities can be synthesized (1) The time to peak decreases as the return period increases for the pre-urbanized state, and is both shorter and relatively constant over a range of return periods for the urbanized state (2) Peak flows increase as return period increase for both pre-urbanized and urbanized states. The relative difference in peak flows is greatest for short return intervals.

All changes to increased peak flows and reduced recurrence intervals can be obtained directly from the designed diagram that shows the relationships among peak discharges, impervious areas, and return periods. This diagram design will be of considerable assistance in water resource planning and management. The proposed procedure can also be applied to other urbanized watersheds due to its reasonable hydrological perspectives.

References

Arnold, C.L.J., Gibbons, C.J., 1996. Impervious surface coverage-the emergence of a key environmental indicator. Journal of the American Planning Association 62, 243—256.

Bonta, J.V., Amerman, C.R., Harlukowicz, T.J., Dick, W.A., 1997. Impact of coal surface mining on three Ohio Watersheds-surface-water hydrology. Journal of the American Water Resources Association 33 (4), 907—917.

Boyd, M.J., Bufill, M.C., Knee, R.M., 1994. Predicting pervious and impervious storm runoff from urban basins. Hydrological Science Journal 39, 321—332.

Brown, R.G., 1988. Effects of precipitation and land use on storm runoff. Water Resources Bulletin 24 (2), 421—426.

Cheng, S.J., Wang, R.Y., 2002. An approach for evaluating the hydrological effects of urbanization and its application. Hydrological Processes 16 (7), 1403—1418.

Cheng, S.J., Hsieh, H.H., Wang, Y.M., 2007. Geostatistical interpolation of space-time rainfall on Tamshui River Basin, Taiwan. Hydrological Processes 21 (23), 3136—3145.

Cheng, S.J., Hsieh, H.H., Lee, C.F., Wang, Y.M., 2008. The storage potential of different surface coverings for various scale storms on Wu-Tu watershed, Taiwan. Natural Hazards 44, 129—146.

Chiles, J.P., Delfiner, P., 1999. Geostatistics: Modeling Spatial Uncertainty. Wiley, New York.

Chow, V.T., Maidment, D.R., Mays, L.W., 1988. Applied Hydrology. McGraw-Hill Book Company., New York.

Duan, Q., Gupta, V.K., 1993. Sorooshian S Shuffled complex evolution approach for effective and efficient global minimization. Journal of Optimization Theory Application 76 (3), 501—521.

Ferguson, B.K., Suckling, P.W., 1990. Changing rainfall—runoff relationships in the urbanizing Peachtree Creek Watershed, Atlanta, Georgia. Water Resources Bulletin 26 (2), 313—322.

Franchini, M., O'Connell, P.E., 1996. An analysis of the dynamic component of the geomorphologic instantaneous unit hydrograph. Journal of Hydrology 175, 407—428.

Goovaerts, P., 2000. Geostatistical approaches for incorporating elevation into the spatial interpolation of rainfall. Journal of Hydrology 228, 113—129.

Gremillion, P., Gonyeau, A., Wanielista, M., 2000. Application of alternative hydrograph separation models to detect changes in flow paths in a watershed undergoing urban development. Hydrological Processes 14, 1485—1501.

Hannah, D.M., Gurnell, A.M., 2001. A conceptual, linear reservoir runoff model to investigate melt season changes in cirque glacier hydrology. Journal of Hydrology 246, 123—141.

Hollis, G.E., 1975. The effect of urbanization on floods of different recurrence interval. Water Resources Research 11 (3), 431—435.

Hsieh, L.S., Wang, R.Y., 1999. A semi-distributed parallel-type linear reservoir rainfall—runoff model and its application in Taiwan. Hydrological Processes 13, 1247—1268.

Huang, H.J., Cheng, S.J., Wen, J.C., Lee, J.H., 2008. Effect of growing watershed imperviousness on hydrograph parameters and peak discharge. Hydrological Processes 22, 2075—2085.

Junil, P., Kang, I.S., Singh, V.P., 1999. Comparison of simple runoff models used in Korea for small watersheds. Hydrological Processes 13, 1527—1540.

Kang, I.S., Park, J.I., Singh, V.P., 1998. Effect of urbanization on runoff characteristics of the On-Cheon Stream Watershed in Pusan, Korea. Hydrological Processes 12, 351—363.

Kibler, D.F., Froelich, D.C., Aron, G., 1981. Analyzing urbanization impacts on Pennsylvaniz flood peaks. Water Resources Bulletin 17 (2), 270—274.

Lebel, T., Bastin, G., 1985. Variogram identification by the mean squared interpolation error method with application to hydrologic field. Journal of Hydrology 77, 31—56.

Lebel, T., Bastin, G., Obled, C., Creutin, J.D., 1987. On the accuracy of areal rainfall estimation: A case study. Water Resources Research 23 (11), 2123—2134.

Lee, Y.H., Singh, V.P., 2005. Tank model for sediment yield. Water Resources Management 19, 349—362.

Leopold, L.B., 1991. Lag times for small drainage basins. Catena 18, 157—171.

Matheron, G., 1971. The theory of regionalized variables and its Application. Fountainbleau, France: Cahiers du Centre de Morphologic Mathematique, Ecole des Mines.

Matheussen, B., Kirschbaum, R.L., Goodman, I.A., O'Donnell, G.M., 2000. Effects of land cover changes on streamflow in the Interior Columbia River Basin (USA and Canada). Hydrological Processes 14, 867—885.

Moon, J., Kim, J.H., Yoo, C., 2004. Storm-coverage effect on dynamic flood-frequency analysis: empirical data analysis. Hydrological Processes 18, 159—178.

Moscrip, A.L., Montgomery, D.R., 1997. Urbanization, flood frequency, and salmon abundancee in Puget Lowland Streams. Journal of the American Water Resources Association 33 (6), 1289—1297.

Nash, J.E., 1957The form of the Instantaneous Unit Hydrograph, vol. 45. IAHS Publications, pp. 112—121.

Rodriguez, F., Andrieu, H., Creutin, J.D., 2003. Surface runoff in urban catchments: morphological identification of unit hydrographs from urban databanks. Journal of Hydrology 283, 146—168.

Sala, M., Inbar, M., 1992. Some hydrologic effects of urbanization in Catalan Rivers. Catena 19, 363—378.

Simmons, D.L., Reynolds, R.J., 1982. Effects of urbanization on base flow of selected south-shore streams, Long Island, New York. Water Resources Bulletin 18, 797—805.

Singh, R.B., 1998. Land use/cover changes, extreme events and ecohydrological response in the Himalayan Region. Hydrological Processes 12, 2043—2055.

Sorooshian, S., Duan, Q., Gupta, V.K., 1993. Calibration of rainfall—runoff models: application of global optimization to the Sacramento Soil Moisture Accounting Model. Water Resources Research 29, 1185—1194.

Syed, K.H., Goodrich, D.C., Myers, D.E., Sorooshian, S., 2003. Spatial characteristics of thunderstorm rainfall fields and their relation to runoff. Journal of Hydrology 271, 1—21.

Tsihrintzis, V.A., Hamid, R., 1997. Urban stormwater quantity/quality modeling using the SCS method and empirical equations. Journal of the American Water Resources Association 33 (1), 163—176.

Valeo, C., Moin, S.M.A., 2000. Variable source area modelling in urbanizing watersheds. Journal of Hydrology 228, 68—81.

Wackernagel, H., 1998. Multivariate Geostatistics. Springer-Verlag, Berlin.

White, L.W., Vieux, B., Armand, D., 2002. Surface flow model: inverse problems and predictions. Advances in Water Resources 25, 317—324.

Wong, T.S.W., Li, Y., 1999. Theoretical assessment of changes in design flood peak of an overland plane for two opposing urbanization sequences. Hydrological Processes 13, 1629—1647.

Yue, S., Hashino, M., 2000. Unit hydrographs to model quick and slow runoff components of streamflow. Journal of Hydrology 227, 195—206.