A PRELIMINARY REPORT

ON

URBAN HYDROLOGY AND URBAN WATER RESOURCES:

OAHU, HAWAII

by

Yu-Si Fok

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Project Completion Report

for

FLOOD HYDROLOGY AND URBAN WATER RESOURCES

OF THE ISLAND OF OAHU, HAWAII

PHASE II

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Principal Investigators: Yu-Si Fok, Edmond D. H. Cheng
L. Stephen Lau and Reginald H. F. Young

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Water Resources Research Center
University of Hawaii
2610 Pope Road
Honolulu, Hawaii 96822

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ERRATA SHEET

Please note the following corrections to our Technical Report No. 74 entitled, *A Preliminary Report on Urban Hydrology and Urban Water Resources: Oahu, Hawaii* (by Yu-Si Fok).

Page	Line	Error	Correction
33	17	uniformity of dimensions in Eq. (9).	uniformity of dimensions in Eq. (18).
34	1	in Table 5	in Table 6
34	9	to peak for selected watersheds.	to peak for selected watersheds:
34	10	Locations of the stream gages are shown in Figure 9.	(Locations of the stream gages are shown in Figure 9.)
38	3	constants in Eq. (10)	constants in Eq. (18)

tween time to peak and urbanized area. However, additional data are required to fully establish such a relationship.

Of the mathematical models tested for rainfall-runoff relationship, the nonlinear time variant method gave the best peak discharge simulation. Also Nash's IUH model (1957) gave good simulation.

All watershed simulation models tested indicated that more hydrological data are prerequisite to the development of a reliable urban hydrology simulation model for Oahu. The data collection program has been expanded to include evaporation, soil moisture, wind speed, solar radiation, and water quality in addition to rainfall and streamflow records gathered since initiation of the project.

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INTRODUCTION

With growing urbanization on Oahu, it has become increasingly apparent that the effects of urbanization of flood hydrology and urban water resources is not clearly understood. This project is addressed to meeting this need by reporting the results and activities during Phase II. The background, rationale, and organization of this study has been amply covered in an earlier publication, *A Preliminary Report on Flood Hydrology and Urban Water Resources: Oahu, Hawaii* (Fok 1973). The present report should be considered as a continuation and an integral part of Phase I, since no attempt has been made here to duplicate the previous discussion.

Studies conducted during Phase I of the "Flood Hydrology and Urban Water Resources of the Island of Oahu, Hawaii" project, were divided into four major tasks: The first was an examination of the causes of flooding and flood damages on Oahu. The second task was to evaluate the effect of urbanization upon flood hydrographs from selected watersheds in Oahu. The third task was to establish the rainfall-runoff data collection device in the abutting urban (St. Louis Heights) and natural (Waahila) watersheds. The fourth task was to initiate the studies of watershed simulation models.

The present report is an extension and expansion of the second, third, and fourth tasks. Specifically, the objectives are: (1) to simulate the urban hydrology of Oahu by adapting and evaluating several existing watershed simulation models, (2) to expand the existing rainfall-runoff data collection program in the adjacent research watershed (Waahila Ridge and St. Louis Heights) to include water quality, evaporation, and soil moisture, and (3) to identify problem study areas of urban water resources on Oahu for subsequent studies.

RECENT URBAN HYDROLOGIC MODELING

One of the significant urban hydrological events on Oahu is the floods created by storm runoff. The peak discharge of a flood is an important determining factor in the design of the storm drainage system. An earlier method of determining peak discharge was presented in the *Storm Drainage Standards* prepared by the Engineering Division, Department of Public Works of the City & County of Honolulu (1959). Later on request of the City &

County, Chow (1966) recommended some revisions to the peak discharge design criteria. Subsequently, the revised *Storm Drainage Standards* (1969) was released by the City & County. Chow (1966) also recommended that the study of the urban hydrology and hydrological records on Oahu should be continued to provide the analysis data necessary for better storm drainage criteria.

Wu (1967) compiled the basic hydrological data, including rainfall, runoff, historical floods, watershed characteristics, soil type, and land use from more than 29 small watersheds on Oahu. In the same report Wu also reviewed and proposed methods for flood peak estimation on Oahu. In a subsequent study, Wu (1969) proposed a peak discharge equation using the concept that hydrographs on Oahu are triangular in shape and watershed responses to flood flow are linear. Wang, Wu, and Lau (1970) reported that the instantaneous unit hydrograph theory is applicable to Oahu, Hawaii, based on a statistical analysis of 240 observed flood hydrographs from 29 watersheds. They also concluded that (1) peak discharge varies linearly with the volume of runoff of each watershed; (2) good correlation exists between effective rainfall duration and the area of the watershed; and (3) the two instantaneous unit hydrograph parameters, gamma function argument (N) and the reservoir storage constant (K), are both in high correlation to the drainage area.

EVALUATION OF WATERSHED SIMULATION MODELS IN HONOLULU

The flood hydrograph is considered to possess properties characteristic of the watershed which are reflected in the parameters of the instantaneous unit hydrograph IUH. Two parameter IUH forms are considered in this study. Optimal estimates of the parameters are obtained by an iterative procedure which is the difference of the minimum sum of squares between observed and stimulated runoff. The methodology described here has been applied to determine (1) the best IUH form for flood hydrograph simulation in the natural watersheds; (2) the representative values of the IUH parameters when urbanization information is available; and (3) the effect of urbanization on storm runoff as reflected in the variation of the IUH parameters of the appropriate model with time and progress in urbanization.

Some Instantaneous Unit Hydrograph Form

It is assumed that the watersheds under consideration may be approximated by linear models. Wang, Wu, and Lau (1970) have suggested that "the superposition characteristics of a linear model can be applied for hydrograph analysis of Hawaiian small watersheds". In this study the instantaneous unit hydrograph (IUH) is considered. Thus,

$$Q(t) = \sum_{i-o}^{t-1} H(t-i)R(i), \quad t = 1, 2, \ldots\ldots\ldots \qquad (1)$$

in which Q(t) is outflow as a function of time t, H(t-i) is the kernel function, and R(i) is input rainfall as a function of time i, where $o \leq i \leq \infty$. Equation (1) is known as the convolution summation of the outflow Q(t). The following two-parameter kernel functions, H(t), were considered in this study:

(i) the routed triangle

$$H(t) = (1/K)\ \mathrm{EXP}(-t/K) \qquad (2)$$

(ii) the routed isosceles triangle

$$H(t) = (1/K)\ \mathrm{EXP}(-t/K) \qquad (3)$$

(iii) Nash's successive routing model (1957)

$$H(t) = [(1/K)\Gamma(N)]\ \mathrm{EXP}\ (-t/K)\ (t/L)^{N-1} \qquad (4)$$

(iv) the log-normal frequencey distribution function

$$H(t) = \frac{1}{t\sqrt{\pi h}} \qquad \mathrm{EXP}\ [-(\log t) - g]^2/h \qquad (5)$$

in which K is the linear storage parameter, t is duration of inflow, Γ is the gamma function, N = t/K, which is the number of linear reservoirs in Nash's model; and g and h are the parameters in Eq. (5).

The criterion for the selection of the representative IUH form among the four types of input functions is the minimum sum of squares of the difference between observed and simulated ordinates of the direct runoff hydrograph. Only the hydrographs from rural watersheds within the Honolulu dis-

trict were used to study the representative IUH form for later application. Storm hydrographs were taken from records of streamflow gaging stations maintained by the U.S. Geological Survey, Honolulu. The recording instruments were 15-minute digital records for stations 2385, 2440, and 2470 (since 1965); and strip chart continuous graphs for stations 2290, 2405, and 2470 (up to 1965). Base flow was separated from the total runoff by standard straight lines method.

HYDROGRAPH SIMULATION.

Stage 1. Determination of effective rainfall:

i. Computes the losses due to interception and infiltration,

ii. Substracts the sum of rainfall rate losses during the 30-minute interval for effective rainfall of the given subbasin, and

iii. Sums the effective rainfall from all subbasins for the same 30-minute sampling period to obtain the total effective rainfall, RBASIN(L), of the basin at the L-th sampling interval.

Stage 2. Sequence of effective rainfall derived from Stage 1 used as input into Stage 2:

i. Computes the ordinate, H, of the IUH form being used by an iterative procedure for the values of N and K,

ii. Simulates runoff S(L) by S(L) = H * BASIN (L), and

iii. Computes the cumulative simulated (SS) and the observed (QQ) runoff in cubic feet per second.

DESCRIPTION OF THE MODEL OPERATION. The major abstractive processes during a storm are interception and infiltration. These two processes can be expressed respectively as:

$$j(L) = (SC - J)\{1 - EXP[-PL(L)/SC]\} \qquad (6)$$

and

$$f = f_c + A(S_o - F)^B \qquad (7)$$

Equation (6) was derived from Merriam (1972), in which $J = SC[1-EXP(-P_s/SC)]$ is a water interception function; in which SC = storage capacity in inches; P_s = amount of rainfall per storm in inches; PL = rate of rainfall in inches per hour; EXP = symbol of exponential function; and P designates the amount of rainfall rate reaching the ground, i.e. P = PL - j for any given time

interval; L and j = rate of interception. Equation (7) is Holtan's infiltration equation (1961), in which f = infiltration rate in inches per hour; f_c = infiltration capacity (steady state infiltration rate) in inches per hour; S_o = available porosity at the beginning of a storm, i.e. S_o = TP - SMj, in which TP = total porosity and SMj = soil-moisture within the control volume just before infiltration; F = cumulative infiltration in inches; and A and B are empirical constants.

In view of the diversity of vegetation and soil series in the watersheds, and the lack of field data, no attempt was made to use a fixed value for the coefficients in Equations (6) and (7). Instead, optimal values of these coefficients were obtained through the Hooke and Jeeves (1961) direct search optimization technique which was written as the MAIN routine program. A general flow chart to show the general computational procedures is presented in Figure 1.

SUBROUTINE LOSRES. The subroutine LOSRES is the main routine to evaluate the objective function (the difference of the sum of squares between observed and simulated runoff) which is to be minimized. With the transferred values of the current values of the infiltration equation coefficients, the subroutine simulates runoff in two stages.

Stage 1. Determines effective precipitation:

i. Computes the losses due to interception and infiltration,
ii. Substracts the sum of the losses from the rainfall rate for the 30-minute sampling interval to give the effective precipitation for the given subbasin,
iii. Sums the effective precipitation from the subbasins for corresponding sampling intervals to obtain the total effective precipitation, RBASIN (L), of the basin at the L-th sampling interval.

Stage 2. Used the sequence of effective precipitation derived from Stage 1 as input into Stage 2, which:

i. Computes the ordinate, H, of the IUH form being used;
ii. Simulates runoff S(1) by S(L) = (H)[RBASIN (L)];
iii. Computes the cumulative simulated (SS) and the observed (QQ) runoff in cubic feet per second;
iv. Computes the sum of squares of residuals, SUMSQ, for each pair

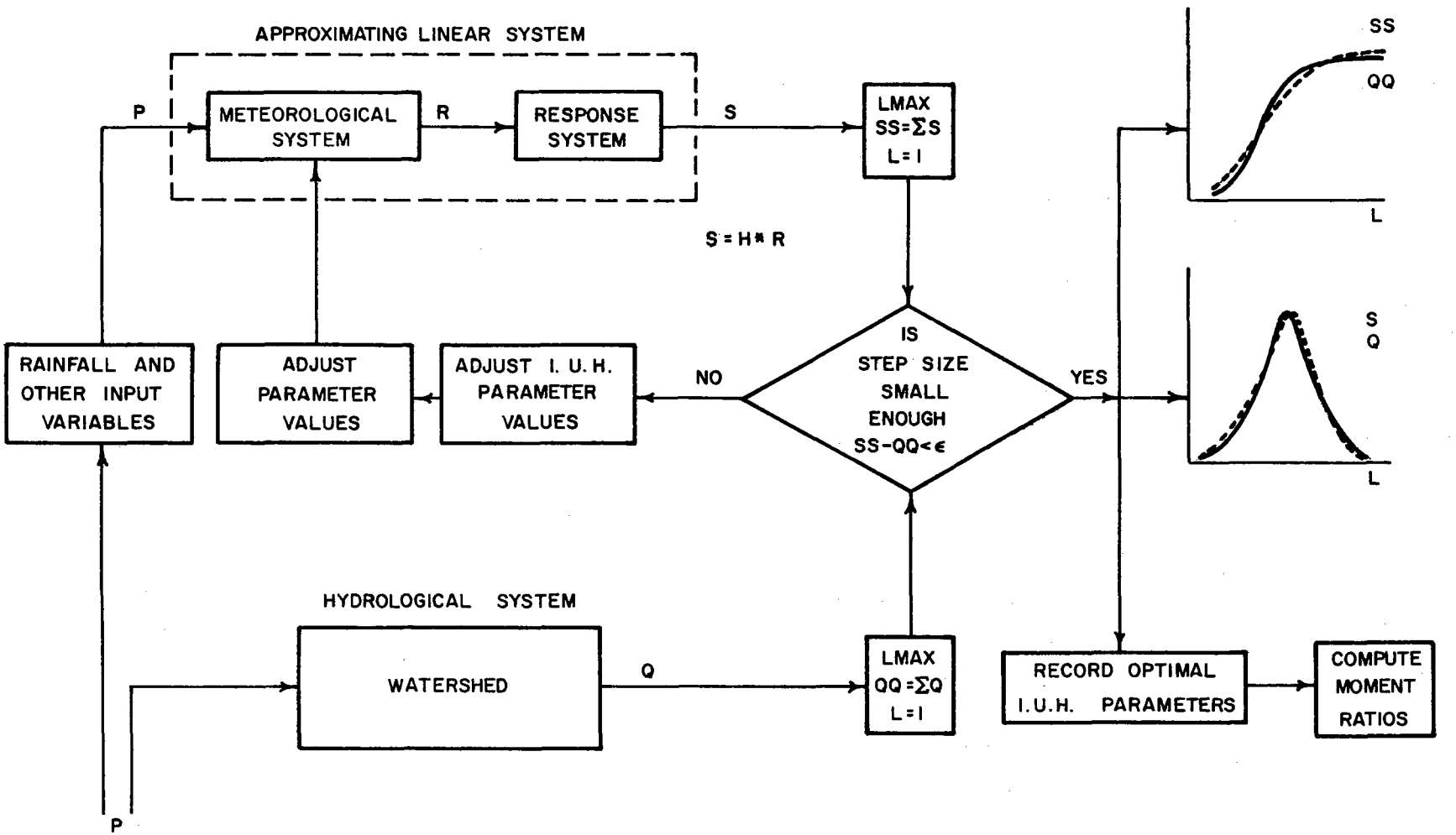

FIGURE 1. GENERAL FLOW CHART

of N and K, compares the current SUMSQ with the current minimum SUMSQ, VMIN. If SUMSQ > VMIN certain variables to be plotted are stored. In either case, the abstractive processes coefficients and N and K are transferred to the main routine, adjusted, and the procedure repeated until the optimal values of these coefficients and parameters are obtained (Figure 2).

MAIN ROUTINE. A brief description is given of the direct search technique used which was developed by Hooke and Jeeves (1961) and has been applied by Huang, Fan, and Kumar (1969) to solve a two-dimensional production scheduling problem and a multi-dimensional (20-variable) production planning problem. A FORTRAN computer program for the method was developed by Huang, Fan, and Kumar (1969). This FORTRAN computer program was modified to adapt to the situation in Stage 1; in other words, the program was used to determine the optimal estimates of the infiltration equation coefficients and the IUH parameters for the overall minimization of the objective function A.

The technique of the MAIN routine consists, essentially, of investigating the local behavior of the objective function, in an n-dimensional space and then moving in a favorable direction for reducing the functional value. For example, given the function, $V(\underline{x})$, to be minimized, where $\underline{x} = (x_1, x_2, \ldots, x_r)$, the argument is varied until the minimum of $V(\underline{x})$ is obtained. The search routine determines the sequence of values of $\underline{x}$. The procedure consists of two types of moves: Exploratory and Pattern. A move is defined as the process of going from a given point to the next point and is a success if the value of $V(\underline{x})$ decreased (for minimization): otherwise it is a failure. The exploratory move, designed to explore the local behavior of the objective function, is made first. The success or failure of the exploratory moves is utilized by combining them into a pattern which indicates a probable direction for a successful move (Hooke and Jeeves 1961).

The exploratory move has been performed as follows:

a. Introduce a starting point $\underline{x}$ with a prescribed step length δ_i in each of the independent variables x_i, $i = 1, 2, \ldots, r$.

b. Compute the objective function, $V(\underline{x})$ where $\underline{x} = (x_1, x_2, \ldots, x_r)$ Set $i = 1$.

c. Compute $V(\underline{x})$ at the trial point

$\underline{x} = (x_1, x_2, \ldots, x_i + \delta_i, x_{i+1}, \ldots, x_r)$.

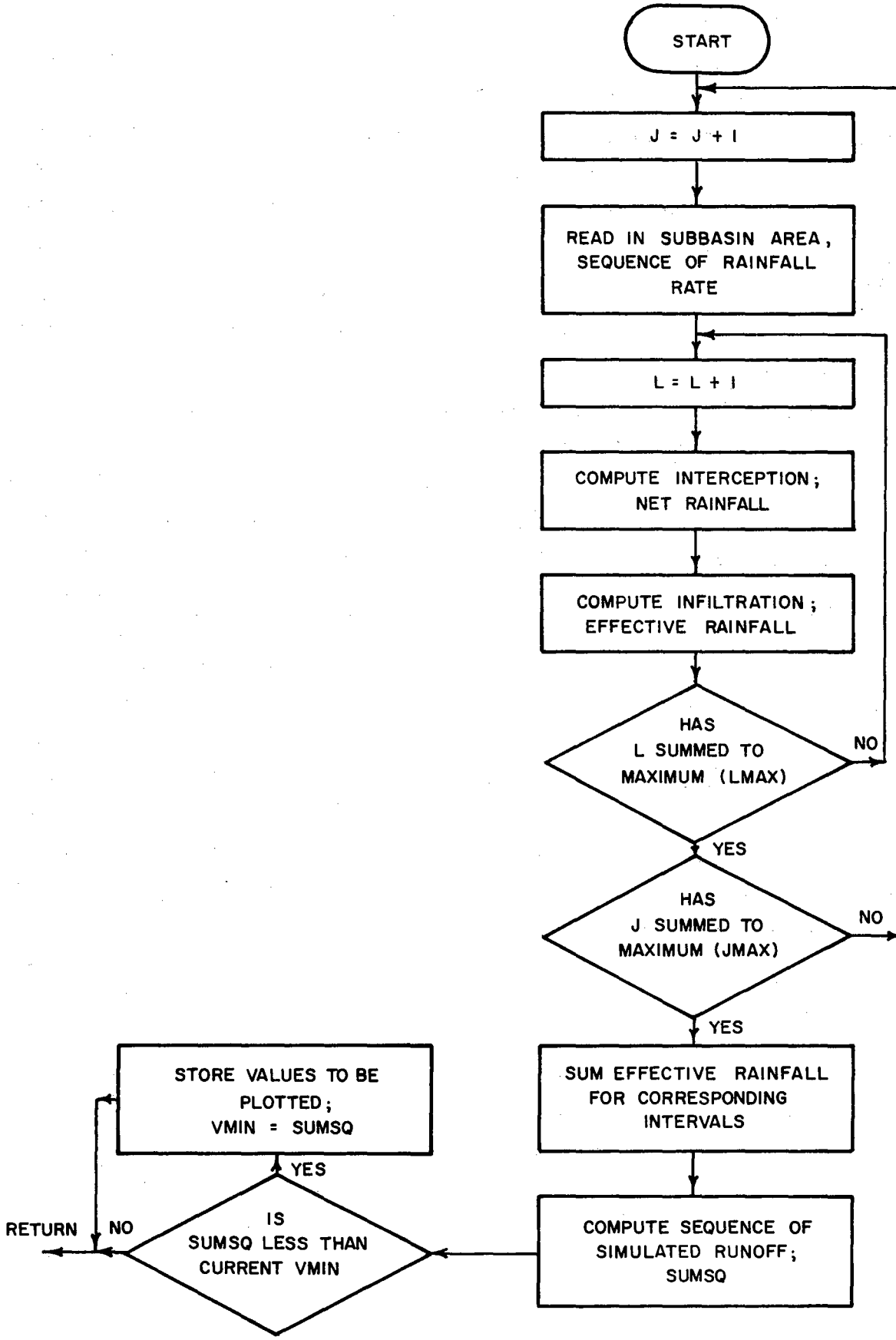

FIGURE 2. DESCRIPTIVE FLOW CHART--SUBROUTINE LOSRES

d. Compare $V_i(\underline{x})$ with $V(\underline{x})$:

i. If $V_i(\underline{x}) < V(\underline{x})$, set $V(\underline{x}) = V_i(\underline{x})$, $\underline{x} = (x_1, x_2, \ldots, x_i, \ldots, x_r) = (x_1, x_2, \ldots, x_i + \delta_i, \ldots, x_r)$, and $i = i + 1$. Consider this trial point as a starting point and repeat from (c).

ii. If $V_i(\underline{x}) \geq V(\underline{x})$, set $\underline{x} = (x_1, x_2, \ldots, x_i - 2\delta_i, \ldots, x_r)$. Compare $V_i(\underline{x})$, and see if $V_i(\underline{x}) < V(\underline{x})$. If this move is a success the new trial point is retained. Set $V(\underline{x}) = V_i(\underline{x})$, $\underline{x} = (x_1, x_2, \ldots, x_i, 111, x_r) = (x_1, x_2, \ldots, x_i - 2\delta_i, \ldots, x_r)$, and $i = i + 1$, and repeat from (c). If again $V_i(\underline{x}) = V(\underline{x})$, then the move is a failure and $\underline{x}_i$ remains unchanged, that is, $\underline{x} = (x_1, x_2, \ldots, x_i, \ldots, x_r)$. Set $i = i + 1$ and repeat (c).

The point $\underline{x}'$ obtained at the end of the exploratory moves, which is reached by repeating step (c) until $i - r$, is defined as the <u>base point</u>. The starting point introduced in step (a) of the exploratory move is a starting base point or point obtained by the pattern move.

The pattern move is designed to utilize the information acquired in the exploratory move, and executes the actual minimization of the function by moving in the direction of the established pattern. The pattern move is a simple step from the current base to the point $\underline{x} = \underline{x}' + (\underline{x}' - \underline{x}'')$ where $\underline{x}''$ is either the starting base point or the preceding base point. Following the pattern move a series of exploratory moves is conducted to further improve the pattern. If the pattern move followed by the exploratory moves does not improve, the pattern move is a failure, and will return to the last base which then becomes a starting base and the process is repeated.

If the exploratory moves from any starting base do not yield a point which is better than this base, the lengths of all the steps are reduced and the moves are repeated. Convergence is assumed when the step lengths, δ_i, have been reduced below predetermined limits.

A descriptive flow diagram for the Hooke and Jeeves pattern search is included as Figure 3. The symbols used in the programming are defined in

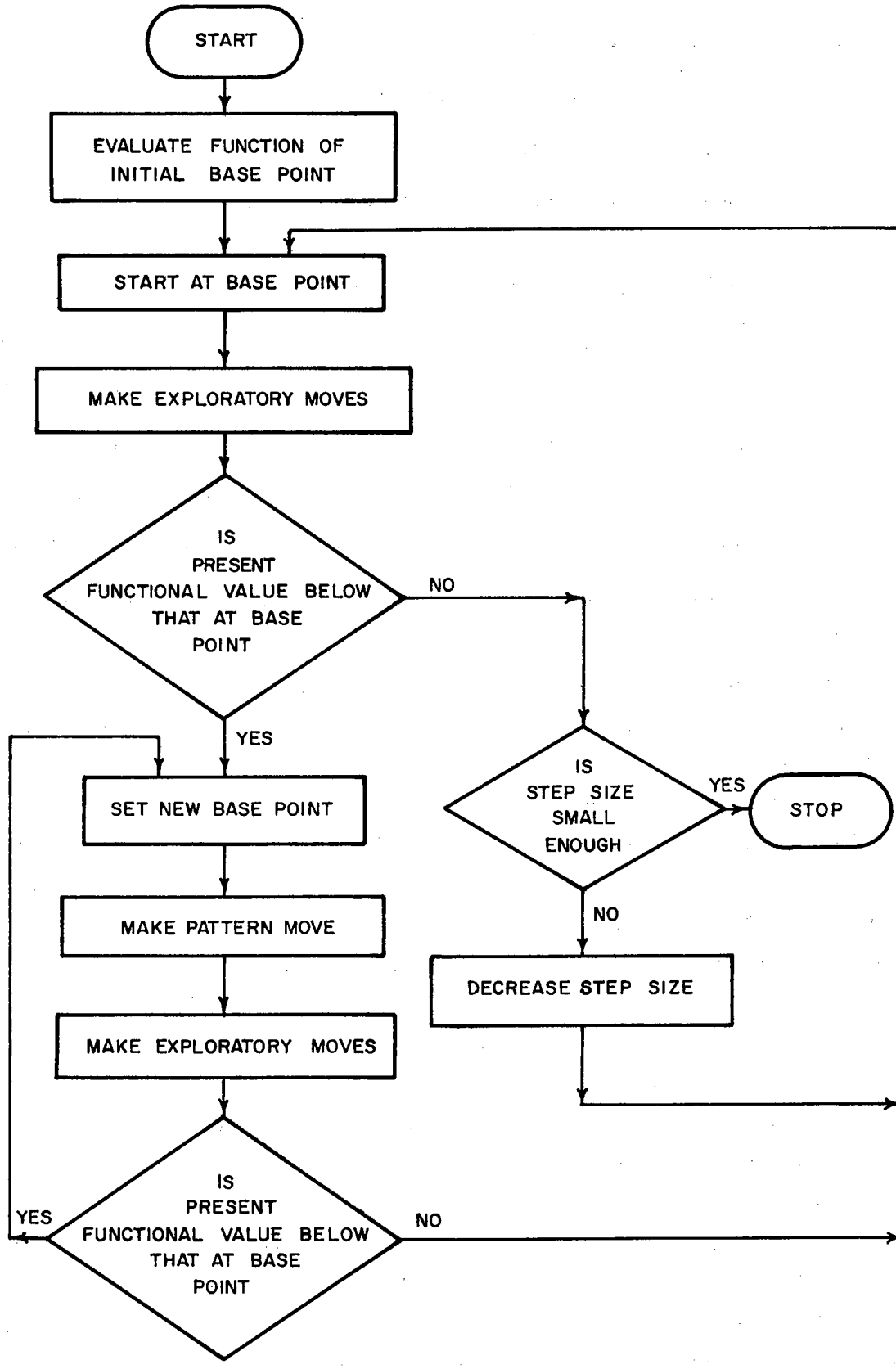

FIGURE 3. DESCRIPTIVE FLOW DIAGRAM FOR HOOKE AND JEEVES PATTERN SEARCH

the FORTRAN program. (The FORTRAN COMPUTER PROGRAM of this study may be obtained from the Water Resources Research Center, University of Hawaii at a cost to cover reproduction and handling. Consult Soronadi Nnaji M.S. thesis "Storm Runoff Response of Watersheds within the Greater Honolulu Watershed, Oahu," University of Hawaii, December 1972.)

DISCUSSION OF RESULTS. The magnitude of VMIN, the difference of the minimum sum of squares between the observed and simulated hydrograph ordinate, is used as a criterion for the selection of the representative IUH form. The smaller the value of VMIN the better the fit for a given storm; however, since only the equations representing the different IUH forms are changed, VMIN could be used to test the relative degree of fit for the same starting values of the parameters to be optimized. The form with the consistently lowest value of VMIN for the storms simulated is considered to best represent the index response to rainfall input of the watersheds in the Greater Honolulu area.

When the recession limb of a hydrograph due to a prior storm merged with the rising limb of that being simulated, the hydrographs were separated by an arbitrary but consistent method. If the separation was not correct to within about 10 cfs per ordinate involved, it was observed that the simulated and observed hydrographs differed significantly (high values of VMIN) such that in the process of simulation the parameters to be optimized had a tendency to take on either large values or values less than zero. In both cases, the optimal estimates of the parameters concerned did not indicate realistic values. In addition, syntax errors occurred such as an <u>underflow</u> error which took place when the computer was ordered to divide a given expression by zero. To circumvent the problem the program was made to terminate computations for any storm when at any given iteration one or more of the parameters to be optimized had a value of zero or less. This action, however, limited the efficiency of the model. Some of the storm simulations thus terminated invariably had high values of VMIN and were not used in the final analysis. For the investigation of the representative IUH form, simulated storms with VMIN greater than 9000 cfs were not included while for the system time invariance 9×10^5 cfs was the limit.

For most of the storms the recession limb of both the simulated and observed hydrograph agreed closely while such agreement for the rising limb range from fair to poor. Nash's conceptual model for a linear water-

shed was considered as the most representative of the four IUH forms, based on the criterion established above and as indicated in Table 1.

TABLE 1. SELECTED OPTIMIZED IUH PARAMETER FORMS.

WATERSHED	STORM	VMIN X10^3	f_c	S_o	N	K
NASH'S CONCEPTUAL MODEL						
UPPER KALIHI	69 01 30	2.61	0.40	0.01	1.50	1.75
	69 05 05	42.13	0.30	0.05	0.05	1.50
WAIHI	69 07 25	81.75	0.30	0.06	0.50	1.50
	70 01 03	2.94	0.30	0.05	1.50	2.25
WAIAKEAKUA	69 07 25	1.97	0.40	0.06	2.00	2.00
	69 11 14	38.20	0.00	0.05	1.00	2.00
PUKELE	69 01 30	0.12	0.31	0.37	1.44	1.00
WAIOMAO	69 01 30	0.45	0.45	0.32	3.00	1.13
	69 11 14	6.01	0.30	0.06	1.50	1.50
THE LOG-NORMAL FREQUENCY DISTRIBUTION MODEL						
UPPER KALIHI	69 01 30	4.82	0.30	0.01	1.00	8.00
	69 05 05	30.62	0.40	0.05	0.50	1.50
WAIHI	69 07 25	32.42	0.20	0.01	1.50	1.00
	70 01 03	3.28	0.30	0.05	1.00	1.25
WAIAKEAKUA	69 07 25	1.62	0.17	0.00	1.50	0.88
	69 11 14	21.24	0.30	0.05	1.00	0.50
PUKELE	69 01 30	0.58	0.30	0.05	0.25	3.00
WAIOMAO	69 01 30	8.65	0.20	0.04	1.00	0.50
	69 11 14	2.19	0.20	0.50	1.00	0.50
THE ROUTED RECTANGE. ROUTED TRIANGLE						
UPPER KALIHI	69 01 30	8.99	0.30	0.02	1.75	4.00
	69 05 05	22.79	0.35	0.05	2.32	3.44
WAIHI	69 07 25	36.26	0.10	0.04	0.80	7.50
	70 01 03	3.74	0.30	0.06	4.00	2.75
WAIAKEAKUA	69 07 25	2.02	0.00	0.08	0.92	6.50
	69 11 14	38.20	0.00	0.05	4.00	2.00
PUKELE	69 01 30	0.08	0.15	1.45	2.18	1.38
WAIOMAO	69 01 30	5.94	0.51	0.08	0.62	4.81
	69 11 14	8.90	0.30	0.06	6.00	1.50

Nash's model was therefore used for the study of system time invariance with Palolo as the test watershed. Results of this study are presented in Table 2. Figures 4 and 5 show how the parameters N and K vary respec-

TABLE 2. SELECTED OPTIMIZED PARAMETERS - PALOLO WATERSHED

STORM	PUA	f_c (INCHES/HR)	S_o (INCHES)	N	K	VMIN $X10^5$CFS
50 12 03	0.21	0.09	1.86	2.25	1.13	9.62
55 01 21	0.25	0.40	0.02	2.88	1.25	0.15
60 05 12	0.27	0.09	0.01	2.50	1.25	4.13
60 12 30	0.27	0.30	0.02	2.00	1.50	4.22
65 10 13		0.04	0.09	1.50	1.50	1.21
66 01 20		0.30	0.02	1.00	1.00	7.30
69 03 16	0.34	0.05	0.06	0.50	0.50	3.37
69 11 14	0.34	0.05	0.05	0.38	1.38	1.35

tively with the percent of urbanized area (PUA) and time in Palolo. For all practical purposes, K may be considered constant with a mean value of 1.20 while N varies inversely with urbanization and time.

The slopes of the channel bottom and valley sides of the watersheds within the Greater Honolulu watershed are steep. The soils for the most part are shallow, being only a few inches deep and comprised in part of areas of exposed rock. The storage of the soil moisture reservoirs, the rate of channel flow, and the overland flow within any given watershed depend on the upland slope and soil depth factors. Urbanization and other activities such as channel improvements generally follow the slopes and contour of the watershed. Given the steep slopes as existing in the watersheds under consideration, the constancy of K, the reservoirs storage parameter may be attributed to the slopes of the channel and valley sides which in general have not been modified by urbanization and channel improvements.

As shown in Figure 4, the systematic decrease in N suggests a decreasing trend in watershed storage with increasing urbanization. Since K is essentially constant as shown in Figure 5 the observed time decrease in watershed storage may be due to change in the time of concentration and

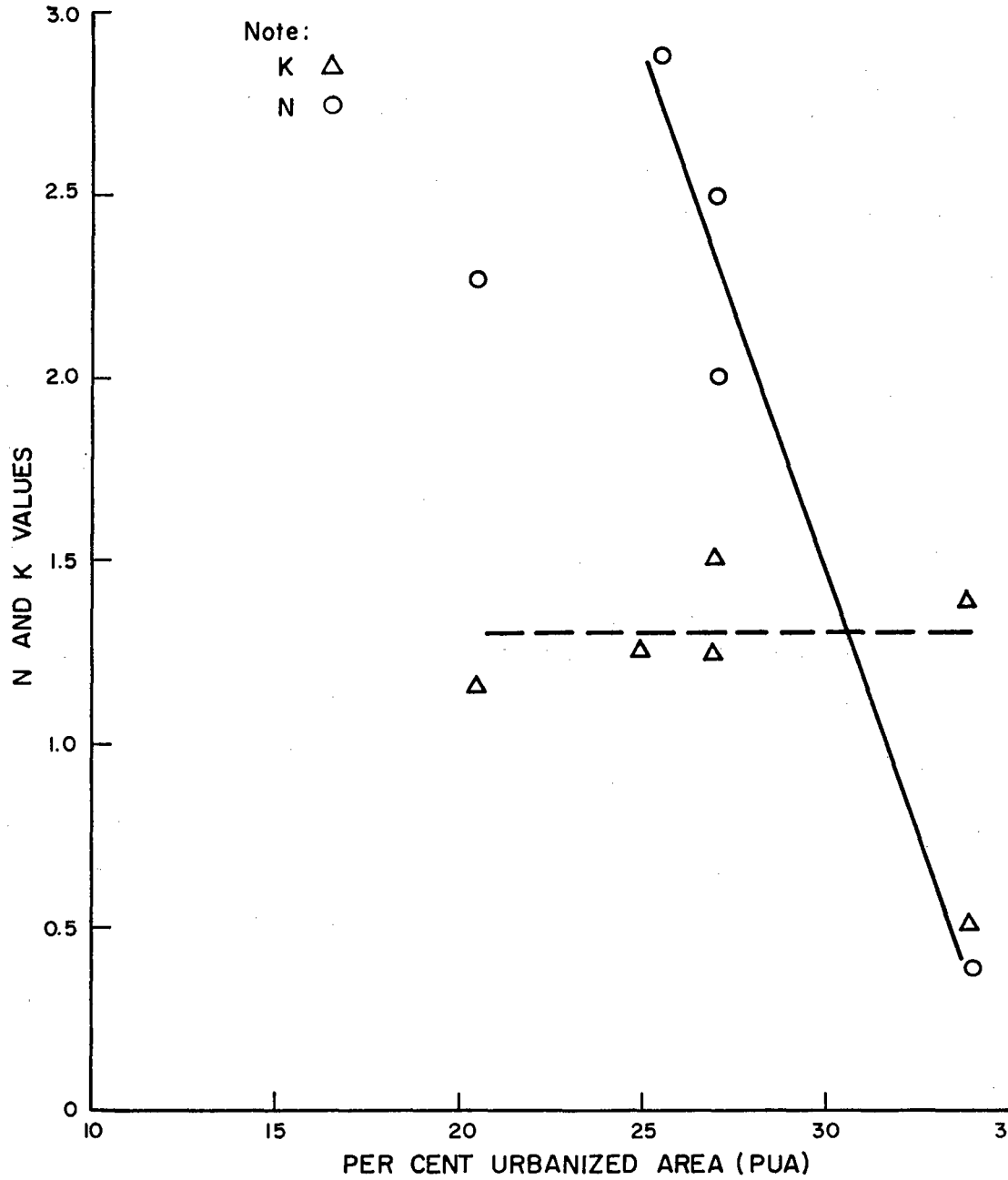

FIGURE 4. NASH'S N AND K VALUES VERSUS PERCENT URBANIZED AREA - PALOLO WATERSHED

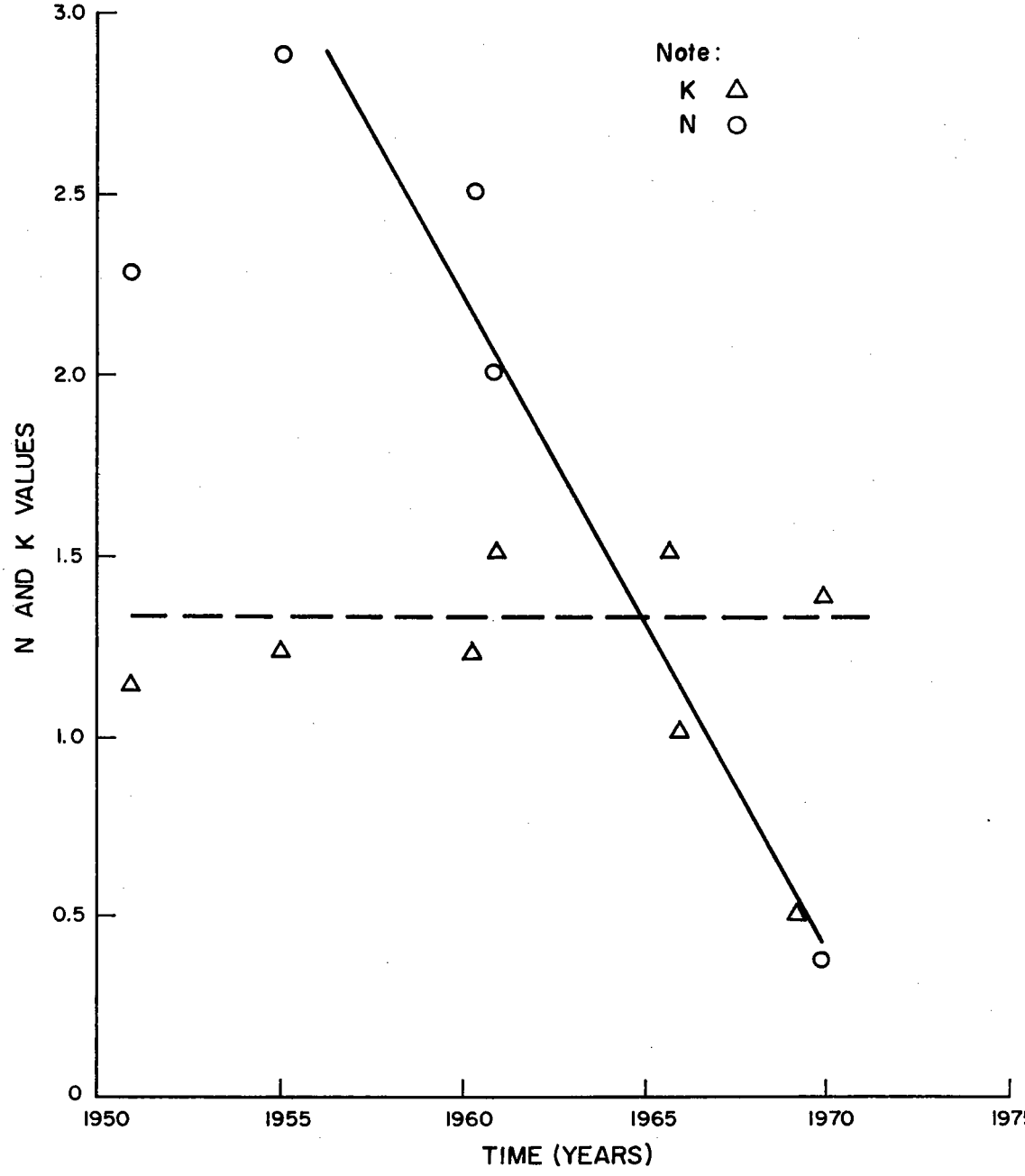

FIGURE 5. NASH'S N AND K VALUES VERSUS PROGRESS OF URBANIZATION IN TIME - PALOLO WATERSHED

the time to peak and consequently of a storage build-up parameter associated with the rising limb of the hydrograph. A physical interpretation of N, the number of reservoirs in Nash's model, suggests that it cannot take on values less than one.

SUMMARY AND CONCLUSIONS. A computer program in FORTRAN was written to simulate the hydrologic processes occurring in a watershed during and after a storm, based on a simplified water balance equation. Depression storage and evapotranspiration were assumed negligible during a storm. The abstractive processes considered were infiltration (Holtan 1969, eq. [7]) and interception (Merriam 1972, eq. [6]) assuming an asymptotic negative buildup of interception storage.

By an iterative procedure, optimal estimates of parameters in Equations (6) and (7) and the two in the equations representing an IUH form were obtained by minimizing the difference of the sum of squares between the observed and simulated runoff. The direct search optimization technique of Hooke and Jeeves (1961) was used.

Selected rural watershed within the Greater Honolulu area were used to investigate the best IUH form applicable to the area of interest. The IUH form yielding the least values of the sum of squares for storms simulated was taken as the most representative. Nash's conceptual model for a linear watershed best satisfied this criterion.

Using data from the Palolo watershed, the effect of urbanization on the hydrologic response of a partially urbanized watershed was investigated. Variations of the instantaneous unit hydrograph parameters, N and K, were used to study the above-mentioned effect. Results of simulation showed that while K was essentially constant, N varied inversely as the percent of the urbanized area increased. Based on the results obtained, it may be concluded that:

1. Of the four two-parameter instantaneous unit hydrograph forms tested, Nash's conceptual model was found to best represent the response of the watersheds within the Greater Honolulu area to rainfall input.
2. The essentially constant value for K obtained is due to the dominant effect of the steep slopes of the channel beds and valley sides, which remain unchanged as urbanization progresses.

3. The decreasing trend of N, the number of reservoirs in Nash's model, is due to a decrease in the overall watershed storage as a result of increasing urbanization and channel improvement.

The Kentucky Watershed Model

As reported by Linsley (1971), hydrology is in a period of transition with the advent of the digital computer, which has made possible the development of complex hydrological simulation models. Furthermore, it has provided the capability to evaluate every conceivable parameter involved in the hydrological process.

The water balance scheme within a watershed has been used to provide the basis for continuous hydrologic simulation using a digital computer. The Stanford Watershed Model (SWM), developed by Linsely and Crawford (1960), was developed under the continuous simulation of the water balance scheme. The SWM was originally written in a digital computer language called BALGOL. Because SWM best simulates the hydrological processes in a watershed, it has been modified and rewritten for wider research into Fortran computer language. After testing and evaluation, the Kentucky Watershed Model (KWM) developed by James (1970) was selected for adaptation in this study.

The major difficulty encountered in the adaptation of the KWM to conditions on Oahu is the need for a large number of watershed parameter values required as input data and acquiring familiarity with over 550 parameter definitions. In this study, emphasis was placed on the testing of the applicability of the KWM on Oahu and the response of the parameters adapted in the Hawaii Watershed Model (HWM).

INPUT DATA PREPARATION. Kalihi, a leeward basin on Oahu, was selected because it is representative of typical Hawaiian terrain and has historical rainfall and streamflow records of rather long duration, as well as a dense network of rain gages.

The water year of 1964, the year with least missing rainfall and streamflow records, was adapted for simulation. Hourly precipitation was derived from rainfall strip charts of four recording gages, (Figure 6) which are quite evenly distributed within the Kalihi watershed. The hourly precipitation and daily streamflow data were punched on IBM cards as in-

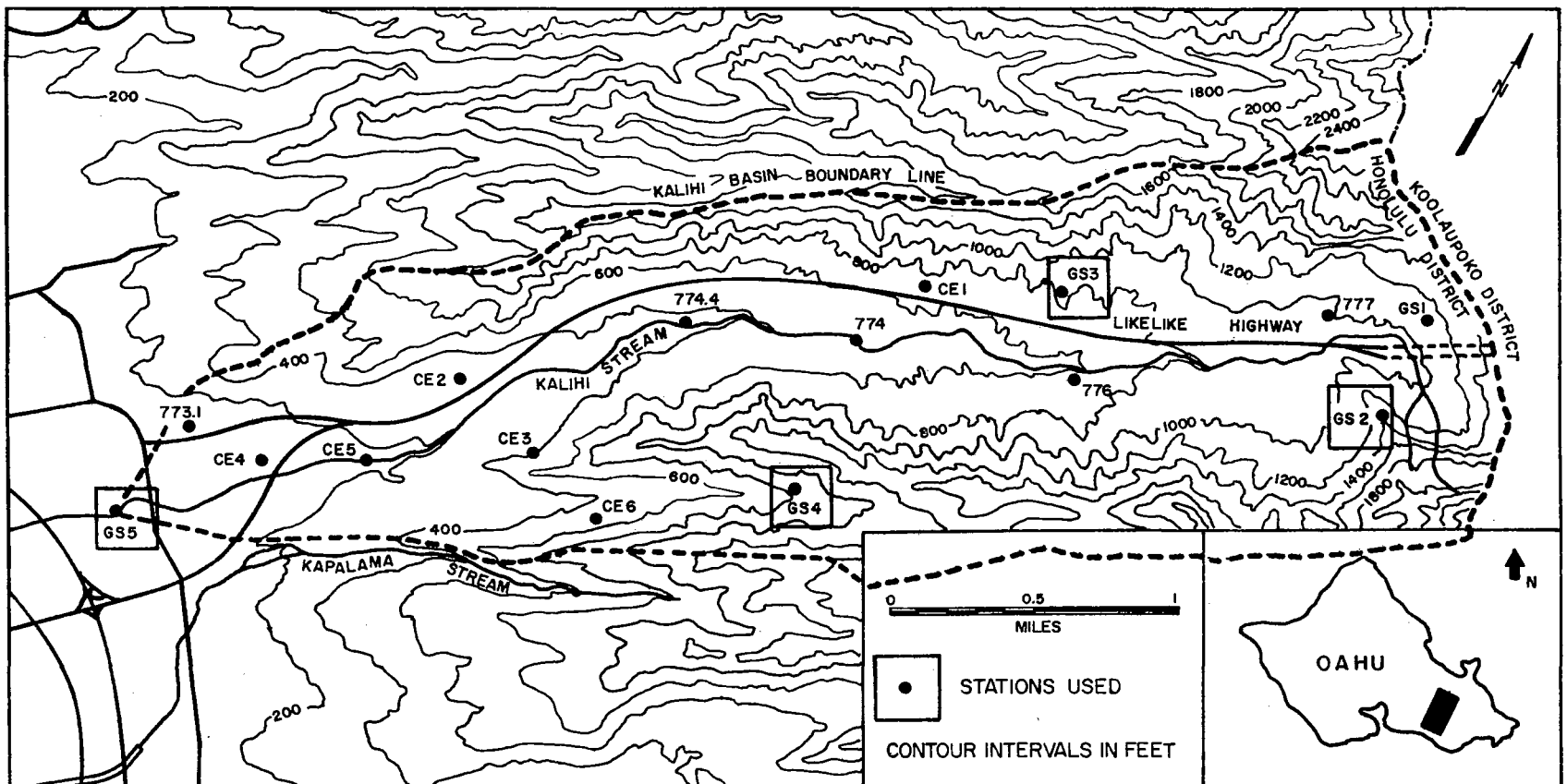

FIGURE 6. RAIN GAGE STATIONS IN THE KALIHI BASIN, OAHU

put information for the computer simulation program in which more than 35,000 data points were processed.

The parameters estimated from observed watershed characteristics were:

1. Area of the watershed (determined from topographic map);
2. Fraction of watershed covered by impervious surface and water surfaces (determined from topographic map);
3. Overland flow surface length and slope (estimated from topographic map);
4. Manning's n for overland flow of natural and impervious surfaces (estimated from Chow [1959]);
5. Vegetative interception (estimated);
6. Subsurface water flow out of the basin (The portion of water entering groundwater storage and leaving the basin through subsurface flow not measured by the stream gage; in this study assumed to equal zero.)
7. Channel capacity: Estimated from *Water Resources Data for Hawaii and other Pacific Areas: (1964)* (U.S. Geological Survey, 1964).

Parameters estimated by OPSET, a computerized parameter optimization procedure developed by Liou (1970), were used in this study. Parameters estimated by OPSET with comparisons of synthesized and recorded streamflow statistics are:

1. Interflow recession constant;
2. Base flow recession constant;
3. Lower zone storage capacity;
4. Basic maximum infiltration rate within the watershed;
5. Seasonal upper zone storage capacity factor;
6. Evapotranspiration loss factor;
7. Basic upper zone storage capacity factor;
8. Seasonal infiltration adjustment constant;
9. Basic interflow volume factor;
10. Number of current time routing increments;
11. Channel storage routing index;
12. Flood plain storage routing index.

The time required for runoff to travel downstream is handled by logging flows through the use of a time-area histogram. The time-area histogram separates the basin into zones by isochrones of travel time to the

watershed outlet. In constructing the time-area histogram, the time of concentration for the Kalihi watershed was computed from the Kirpich formula:

$$Tc = 0.0078 \, [L/\sqrt{S}\,]^{0.77} \qquad (8)$$

in which S = H/L, L = max. length of travel in feet, H = difference in elevation, and S = slope.

Necessary data and information were obtained from Wu (1967) and *Storm Drainage Standards* published by the Department of Public Works, City & County of Honolulu (1969).

To initiate the study, a complete set of the computer source deck which consists of two main programs and twenty subroutines, a total of more than three thousand statements, has been prepared. (The FORTRAN COMPUTER PROGRAM of this study may be obtained from the Water Resources Research Center, University of Hawaii at a cost to cover reproduction and handling.)

RESULTS. Results of six selected test runs were presented in Table 3. In test run No. 1, the output synthetic peak flow and peak time of the two storm hydrographs from the computer program at stream gage #2293 are far from the recorded values. Further, there is a 34.2% deviation from synthetic to recorded annual flows. In test run No. 2, the overland flow surface slope (OFSS) was reevaluated. The only improvement shown by the output was the close match of synthetic and recorded peak time of the October 4, 1964 storm hydrograph. Unfortunately, the percentage of deviation from the synthetic to the recorded annual stream flows increased from 34.2% in test run no. 1 to 64.1%. In test run No. 3, as a result of adjusting the recording gage precipitation multiplier (RGPMB) from 1.2 to 1.0, the percentage of deviation of the synthetic annual flow to the recorded data sharply decreased to 6.3% as the best simulated output among the six test runs. However, the simulated storm hydrographs are still meaningless. In test run No. 4, based on rain gages CS2 and CS4, the time-area histogram was changed from four to two isochronic zones. As shown in Table 3, the synthetic peak time for both storm hydrographs showed a close match with recorded data. In test run No. 5, based on rain gages, CS2 and CS5, percentages of the two isochronic zones were recalculated, as it

TABLE 3. INPUT AND OUTPUT OF SIX SELECTED TEST RUNS FOR WATER YEAR 1964.

	1	2	3	4	5	6
AREA	5.360	5.360	5.360	5.360	5.360	5.360
BFNLR	0.990	0.990	0.990	0.990	0.990	0.990
BFNX	0.025	0.025	0.025	0.025	0.025	0.025
BFRC	0.938	0.938	0.938	0.938	0.938	0.938
BIVF	0.000	0.000	0.000	0.000	0.000	0.000
BMIF	1.200	1.200	1.200	2.129	2.129	2.129
BUIC	1.500	1.500	1.500	2.674	2.674	2.674
CHCAP	1200.000	1200.000	1200.000	1200.000	1200.000	1200.000
CSRX	0.900	0.900	0.900	0.900	0.900	0.900
CTRI	0.269, 0.285 0.286, 0.160	0.269, 0.285 0.286, 0.160	0.269, 0.285 0.286, 0.160	0.554, 0.446	0.756, 0.244	0.756, 0.244
EPAET	73.850	73.850	73.850	73.850	73.850	73.850
ETLF	0.250	0.250	0.250	0.354	0.354	0.354
EXQPV	0.300	0.300	0.300	0.300	0.300	0.300
FIMP	0.250	0.250	0.250	0.250	0.250	0.250
FSRX	0.900	0.900	0.900	0.900	0.900	0.900
FWTR	0.002	0.002	0.002	0.002	0.002	0.002
GWETF	0.000	0.000	0.000	0.000	0.000	0.000
GWS	0.503	0.503	0.503	0.503	0.503	0.503
IFRC	0.100	0.100	0.100	0.100	0.100	0.100
IFS	0.000	0.000	0.000	0.000	0.000	0.000
LZC	12.000	1.200	1.200	17.430	17.430	17.430
LZS	6.000	6.000	6.000	25.898	25.898	25.898
MNRD	130.000	130.000	130.000	130.000	130.000	130.000
NCTRI	4.000	4.000	4.000	2.000	2.000	2.000
OFMN	0.030	0.030	0.030	0.030	0.030	0.030
OFMNIS	0.020	0.020	0.020	0.020	0.020	0.020
OFSL	700.000	700.000	700.000	700.000	700.000	700.000
OFSS	0.250	0.450	0.450	0.450	0.450	0.450
RGPMB	1.200	1.200	1.000	1.000	1.000	1.000
RMPF	100.000	100.000	100.000	100.000	100.000	100.000
SIAC	0.900	0.900	0.900	1.678	1.678	1.678
SUBWF	0.000	0.000	0.000	0.000	0.000	0.000
SUZC	1.300	1.300	1.300	1.936	1.936	1.936
UZS	0.000	0.000	0.000	0.000	0.000	0.000
VINTMR	0.150	0.150	0.150	0.150	0.150	0.150

TABLE 3. INPUT AND OUTPUT OF SIX SELECTED TEST RUNS FOR WATER YEAR 1964 (CONT'D).

	SIMULATED OUTPUT								
	CASE 1	CASE 2	CASE 3 *	CASE 4	CASE 5	CASE 6			RECORDED
PEAK FLOW (OCT. 4)	2493	3127	2331	3017	2093	2093	2194	2176	3720.00 cfs
PEAK TIME (CLOCK TIME)	2:45	1:45	2:45	1:45	1:45	1:45	1:45	1:45	1:36 PM
PEAK FLOW (MAR. 22)	263.30	356.00	166.40	212.20	225.30	225.30	225.30	225.30	975.00 cfs
PEAK TIME (MAR. 22)	6:45	7:45	8:45	5:45	4:45	4:45	4:45	4:45	4:22 PM
TOL. SYNTHETIC ANNUAL FLOWS (SFD)	5043	6164	3992	3495	3511	3511	3482	3429	3756.80 STD
SOIL MOISTURE	6.0-14.8	1.8-6.0	0.5-6.0	22.2-26.2	22.2-26.2	22.2-26.2	20.4-24.2	19.3-23.0	N.A.
% OF ERROR OF SYNTHETIC FLOWS	34.20	64.10	6.30	7.00	6.50		8.70		

REMARKS * TWO ZONES WERE USED

TABLE 4. GLOSSARY OF VARIABLES USED IN TABLE 3.

AREA	AREA OF WATERSHED (SQ MI)
BFNLR	BASIC FLOW NONLINEAR RECESSION ADJUSTMENT FACTOR
BFNX	CURRENT VALUE OF BASE FLOW NONLINEAR RECESSION INDEX
BFRC	BASIC FLOW RECESSION CONSTANT
BIVF	BASIC INTERFLOW VOLUME FACTOR
BMIR	BASIC MAXIMUM INFILTRATION RATE WITHIN WATERSHED (IN/HR)
BUZC	BASIC UPPERZONE STORAGE CAPACITY FACTOR
CHCAP	CHANNEL CAPACITY - INDEXED TO BASIN OUTLET (cfs)
CSRX	CHANNEL STORAGE ROUTING INDEX
CTRI	CURRENT TIME ROUTING INCREMENTS
EPAET	ESTIMATED MAXIMUM ANNUAL EVAPOTRANSPIRATION
ETLF	EVAPOTRANSPIRATION LOSS FACTOR
EXQPU	EXPONENT OF FLOW PROPORTIONAL TO VELOCITY
FIMP	FRACTION OF WATERSHED BEING IMPERVIOUS
FSRX	FLOOD PLAIN STORAGE ROUTING INDEX
FWTR	FRACTION OF WATERSHED BEING WATER
GWETF	GROUNDWATER EVAPOTRANSPIRATION FACTOR
GWS	CURRENT GROUNDWATER STORAGE (IN)
IFRC	INTERFLOW RECESSION CONSTANT
IFS	INTERFLOW STORAGE
LZC	LOWER ZONE STORAGE CAPACITY (IN)
LZS	CURRENT LOWER ZONE STORAGE (IN)
MNRD	MEAN ANNUAL NUMBER OF RAINY DAYS
NCTRI	NUMBER OF CURRENT ROUTING INCREMENTS
OFMN	OVERLAND FLOW MANNING'S N

TABLE 4. - CONTINUED

OFMNIS	OVERLAND FLOW MANNING'S N, IMPERVIOUS SURFACES
OFSL	OVERLAND FLOW SURFACE LENGTH (FT)
OFSS	OVERLAND FLOW SURFACE SLOPE
RGPMB	RECORDING GAGE PRECIPITATION MULTIPLIER - BASIC
RMPF	REQUESTED MINIMUM DAILY PEAK FLOW TO BE PRINTED
SIAC	SEASONAL INFILTRATION ADJUSTMENT CONSTANT
SUBWF	SUBSURFACE WATER FLOW OUT OF THE BASIN
SUZC	SEASONAL UPPER ZONE STORAGE CAPACITY FACTOR
UZS	CURRENT UPPER ZONE STORAGE (IN)
VINTMR	VEGETATIVE INTERCEPTION - MAXIMUM RATE

turned out to be the best output among the test runs in the simulation computer program. In test run No. 6, the input data were recycled three times as three years' input data. The outcome was more or less the same as in test run No. 5 but the percentage of deviation of the synthetic to the recorded annual streamflow increased from 6.5% to 8.7%.

For all six test runs, the results of simulated daily streamflow or flood runoff at the outlet stream gage (#2293) of Kalihi watershed are encouraging. The discrepancy between recorded and simulated peak flows should not be a surprise because, in this study, the KWM was used without major revision to find its applicability to the Hawaii watershed modeling study. As a matter of fact, the basic strategy of watershed modeling is to simulate a sequence of streamflows from input climatological data through a defined computational procedure based on "equations" containing parameters. In the Kentucky Watershed Model, these "equations", which represent the physical processes governing the hydrological phenomena, are empirically developed and their applicability may not necessarily apply to conditions on Oahu.

CONCLUSION. In conclusion it is felt that the basic logic of SWM is applicable to the Hawaii Watershed Model Study. In light of this study,the major research effort is the task of combining mathematical expressions representing the major physical processes governing the Hawaii hydrological phenomena into a debugged computer program. This task will take two or three more years and the committment of sufficient research manpower as has been experienced in the development of the KWM by James (1970).

The Road Research Laboratory Model

The Road Research Laboratory is an agency of the Ministry of Transport of the British government. Just after World War II, personnel of this laboratory recognized the deficiencies in using the rational and unit hydrograph methods for urban storm drainage systems design. Their findings were obtained after a study using 286 storm events collected from 12 urban basins. The new British RRL Hydrograph Method was devised. The development and application of the RRL Model have been described by Watkins (1962), and later by Terstriep and Stall (1969), and Stall and Terstriep (1972). The Road Research Laboratory (1963) provided an input-output computer program guide for design engineers and thus greatly simplified the designer's

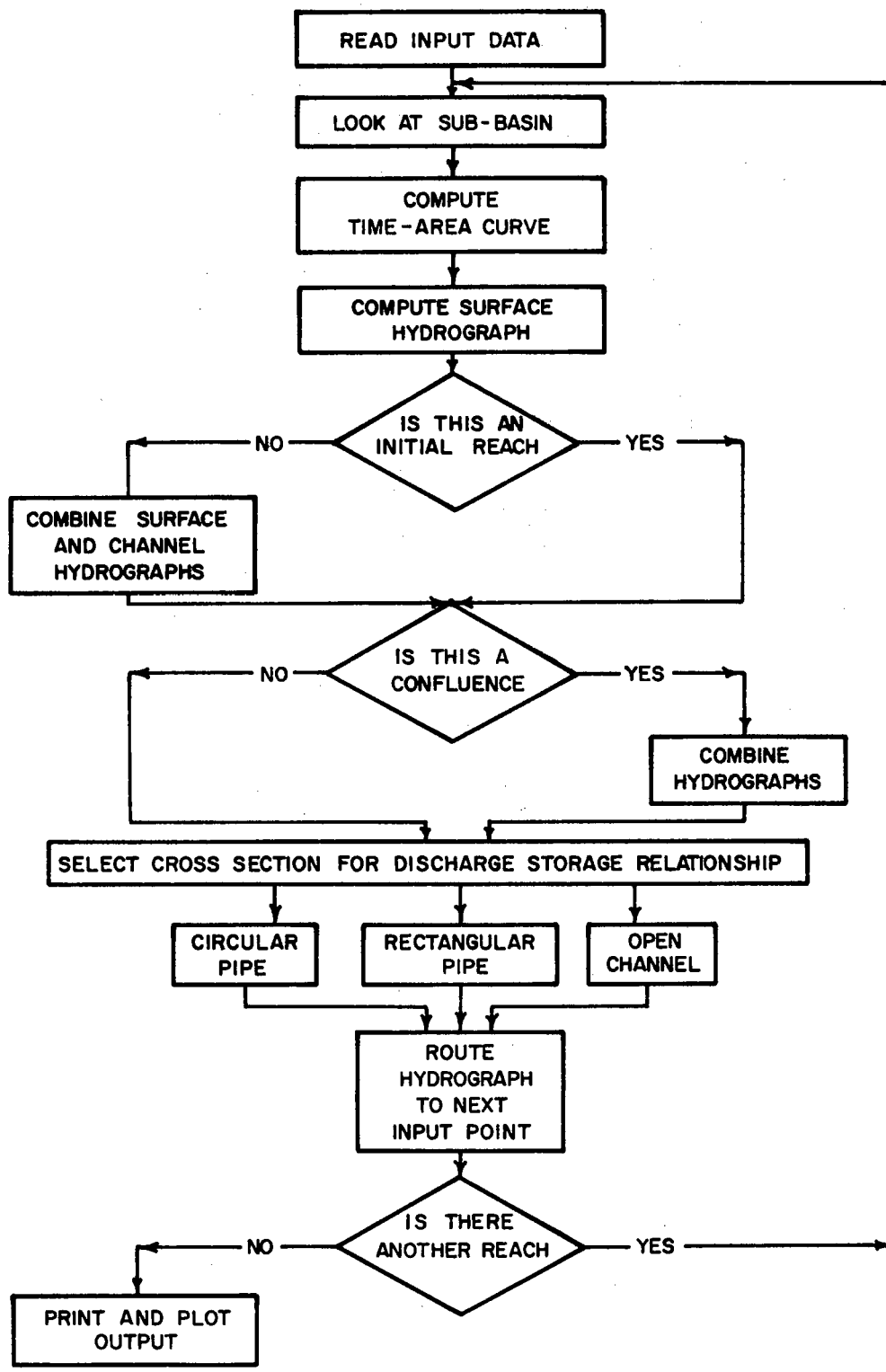

FIGURE 7. FLOW CHART FOR THE RRL METHOD COMPUTER PROGRAM.

work. It was reported that in England in 1972 about 80 percent of the storm drainage system design or modification was carried out by the RRL method.

PROCEDURE. The RRL method involves five principal steps as reported by Terstriep and Stall (1969):

1. The physical aspects of the basin are described in detail. The storm drainage system is mapped and specified hydraulically in terms of length, diameter, slope, and roughness.
2. For the directly connected impervious areas, the following hydraulic computations are made:
   a. Plane surface flow velocities,
   b. Flow velocities in gutters and at the inlets,
   c. Flow velocities in the storm drain systems: laterals, main interceptors, and open channels,
   d. Develop the isochrones (equal travel time to the subbasin or the basin outlets) on the basin map using the flow velocities,
   e. Develop a time-area diagram using areas between the isochrones.
3. For the input, the basin rainfall pattern is areally presented on Thiessen polygons or an isohyetal map, and is presented in time by one or more mass curves of rainfall amounts. After abstraction of the losses, the incremental rainfall and time-area data are used to construct the inflow hydrograph.
4. Develop a storage-discharge relationship to account for the effects of storage within the basin on runoff.
   a. For gaged basin: use a number of observed hydrograph recession curves to derive a general recession curve to provide the indicated storage.
   b. For un-gaged basin: compute a discharge-storage relation in proportional depth by assuming the flow depth throughout the basin is proportional to the depth at the outlet.
5. Develop the runoff hydrograph using a simplified one-step storage-routing procedure. A flow chart for the computer program of the RRL Method reported by Stall and Terstriep (1972) is presented in Figure 7.

APPLICATION. The RRL Method is thought to be useful as a storm hydrograph in urban areas. An attempt has been made to apply the RRL Method to sim-

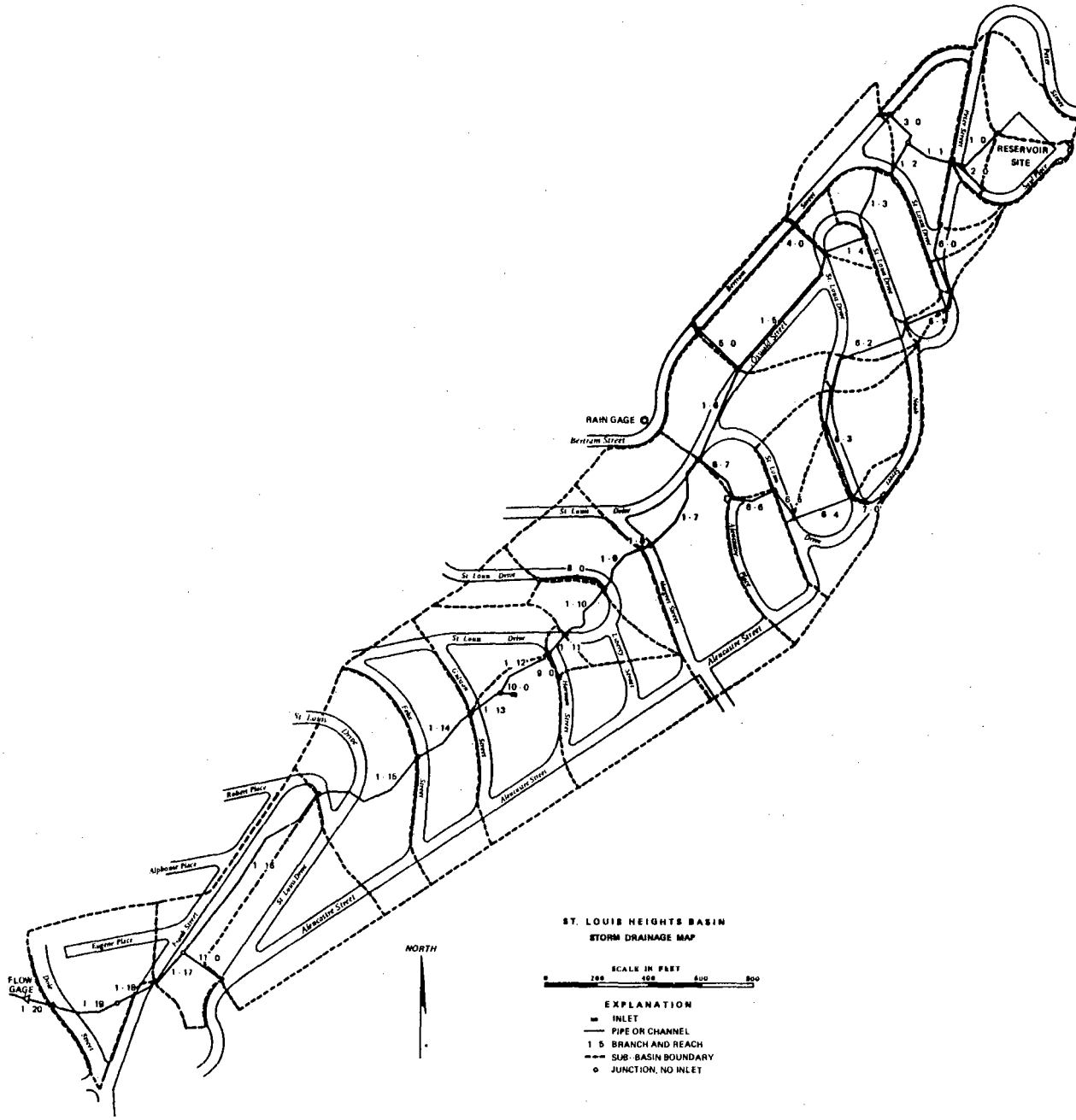

FIGURE 8. ST. LOUIS HEIGHTS BASIN STORM DRAINAGE.

ulate the storm hydrographs obtained from the research urban watershed of St. Louis Heights in Honolulu during 1972.

Following the procedures of the RRL Method, a map showing the storm drainage system in St. Louis Heights was obtained from the Department of Public Works, City & County of Honolulu. After a field investigation, all the directly connected impervious areas were documented along with the length, slope, roughness, shape,and dimensions of the storm drains. A systematic numerical designation of all the branches of the drainage system, locations of the rain gage and the streamflow gage are presented in Figure 8.

RESULTS AND CONCLUSIONS. The computed and recorded hydrographs don't quite agree with each other. The reasons for these differences may be many. However, the major reason may be the instrumentation. There is evidence that some of the rainfall and runoff records do not correspond. Furthermore, the recorded stage versus streamflow discharge has not been calibrated. For the time being, only Manning's formula has been used for the stage-discharge calibration. As more rainfall runoff data is obtained, the management and operation of the hydrological data network can be improved.

In conclusion, the RRL method does have an adaptation potential for use on Oahu, and it is recommended that this method be used on subsequent collected rainfall-runoff data from the St. Louis Heights watershed. The current study, with only three storms, did not provide sufficient information to test the applicability of the RRL method in Honolulu.

COMPARISON OF THE LINEAR AND NONLINEAR MODELS

Characteristics of rainfall response from watersheds on Oahu, Hawaii have been studied by researchers. Their studies applied the unit hydrograph method or instantaneous unit hydrograph method. One of the important assumptions of the two methods is that the hydrological system involved is assumed to be linear and time invariant for the simplicity of mathematical modeling of the rainfall-runoff relation. However, it is known that the physical properties of a watershed are not homogeneous and is subject to change with seasonal fluctuations and human activities. Therefore, it is very important to make a comparison of the watershed response mathematical

models with the same set of recorded rainfall and runoff data.

In this study, only six sets of rainfall and runoff data obtained from the Kalihi watershed were used for the comparison of the four types of watershed response mathematical models:

Linear time invariant Model (Nash 1957)

Linear Time Variant Model (Chiu and Bittler 1969)

Nonlinear Time Invariant Model (Prasad 1967)

Nonlinear Time Variant Model (Chiu and Huang 1970).

Although the derivation and application of individual mathematical models are presented in the references cited, the models are presented here for general review.

NASH'S LINEAR TIME INVARIANT MODEL (IUH) (1957)

$$Q_N = \left[\frac{A}{K\Gamma(N)} \quad e^{-t/k} \quad (t/k)^{N-1}\right] I \tag{9}$$

in which A = drainage area

I = rainfall depth

K = storage coefficient

t = time

e = base of natural logarithm

N = number of conceptual reservoirs

$\Gamma(N)$ = Gamma function

CHIU AND BITTLER'S LINEAR TIME VARIANT MODEL (1969)

$$\frac{K(t)dQ(t)}{dt} + Q(t)\,\frac{[1 + dK(t)]}{dt} = I(t) \tag{10}$$

or

$$Q(t) = \left[\frac{1}{K(t)D + \frac{dK(t)}{dt} + 1}\right] I(t) \tag{11}$$

in which K(t) = storage coefficient as a function of time, t

Q(t) = discharge as function of time, t

I(t) = input rainfall as function of time, t

dt = differential time

D = differential operator

PRASAD'S NONLINEAR TIME INVARIANT MODEL (1966)

$$K_2 \frac{d^2Q(t)}{dt^2} + K_1N_1Q(t)^{N_1-1} \frac{dQ(t)}{dt} + Q(t) = I(t) \qquad (12)$$

or

$$Q(t) = \left[\frac{1}{K_2D^2 + K,N,Q(t)^{N_1-1} D + 1}\right] I(t) \qquad (13)$$

in which K_2 = storage coefficient, may be a complicated function of wedge-storage and storage-discharge relationships

K_1 = parameter of direct runoff hydrograph

N_1 = parameter of direct runoff hydrograph

D = differential operator

$Q(t)$ = runoff as function of time, t

$I(t)$ = Input rainfall as function of time, t

CHIU AND HUANG'S NONLINEAR TIME VARIANT MODEL (1970).

$$K_2(t)N_2 \left[\frac{dQ(t)}{dt}\right]^{N_2-1} \frac{d^2Q(t)}{dt^2} + \left[KNQ(t)^{N_1-1} + \frac{dK_2(t)}{dt}\left(\frac{dQ(t)}{dt}\right)^{N_2-1}\right]$$

$$\frac{dQ(t)}{dt} + Q(t) = I(t) \qquad (14)$$

or

$$Q(t) = \frac{1}{f_2(t)D^2 + f_3(t)D + 1} I(t) \qquad (15)$$

in which

$$f_2 = K_2(t)N_2 \frac{dQ(t)}{dt}^{N_2-1} \qquad (16)$$

$$f_3 = KNQ(t)^{N_1-1} + \frac{dK_2(t)}{dt} \cdot \frac{dQ(t)}{dt}^{N_2-1} \qquad (17)$$

K_2 = a variable coefficient which depends on time elapse

RESULTS AND DISCUSSION. The correlation coefficients of the observed and simulated peak discharges, Qp are listed in the following:

Method	Correlation Coefficient	Standard Deviation
Chiu and Huang's Method (Nonlinear, Time Variant)	0.995	4.42

Chiu and Bittler's Method (Linear, Time Variant)	0.9922	13.35
IUH (Nash) Method (Linear, Time Invariant)	0.9157	45.25
Prasad's Method (Nonlinear, Time Invariant)	0.7544	56.08

It is clear that the nonlinear time variant method and the linear time variant method are the better methods for peak discharge simulation of the Kalihi watershed with six sets of data.

The correlation coefficients of the observed and simulated time to peak Tp, are listed in the following:

Method	Correlation Coefficient	Standard Deviation
Chiu and Bittler's Method (Linear, Time Variant)	1.000	0
IUH (Nash) Method (Linear, Time Invariant)	0.740	0.16
Prasad's Method (Nonlinear, Time Invariant)	0.707	0.18

In the above comparisons, Chiu and Huang's Method has not been included because the observed time to peak is used as a predetermined condition. Again, it is very clear that the linear time variant method is the better method for time to peak simulation for the Kalihi Watershed with the six sets of rainfall runoff data.

CONCLUSIONS. From the comparisons listed, it may be concluded that for the simulation of peak discharges on Kalihi Watershed the nonlinear time variant model proposed by Chiu and Huang (1970) is the best based upon the data used. The inadequacy of available data is a limiting factor for extensive examination of the four mathematical models on other watersheds. It is worth noting that the nature of the Chiu and Huang model is applicable only to single peak hydrographs, since it is not designed for the simulation of multiple peak hydrographs. According to Chiu and Huang (1970), their model offers better simulation on large watersheds and storms of short duration and high intensity. The linear time invariant model (IUH Model) offers good simulation of the peak discharge and a moderate simula-

tion of time to peak. It may be explained that this model adapts very well to the small watershed such as the Kalihi watershed.

PEAK DISCHARGE AND URBANIZATION

Two important parameters of the storm flood hydrograph are peak discharge and time to peak; these two flood parameters are necessary for a storm drainage system. One way to examine the effect of urbanization on peak discharge, Qp, of a given storm is by the multiple regression method of correlating recorded peak discharge with the urbanized area. It is logical to assume that peak discharge reflects all the hydrological factors involved in a storm; by the same token, the effect of urbanization on time to peak can also be studied.

THE MODELS. An empirical equation expressed as Eq. (18) served as the model to correlate peak discharge with other major hydrological factors.

$$Qp = e^{(a+b\frac{Au}{A})} R^c T_p^d A^f \qquad (18)$$

in which e = the base of the natural logarithm; Au = urbanized area; A = drainage area; R = rainfall depth; T_p = time to peak; and a, b, c, d, and f are empirical constants. Dimensionally, the values of c and f should equal to one while the value of T_p should equal to negative one to maintain the uniformity of dimensions in Eq. (9).

An empirical equation was used to evaluate the time to peak of a storm hydrograph in this study which is expressed as

$$T_p = e^m L^n S^p (Au/A)^q \qquad (19)$$

in which T_p = time to peak, L = length of mainstream and S = average slope of the mainstream, and m, n, p, and q are the empirical constants.

EVALUATION OF THE CONSTANTS IN THE EMPIRICAL EQUATIONS FOR PEAK DISCHARGE AND TIME TO PEAK. For the selected urbanized watersheds on Oahu, data for the peak discharge and time to peak were obtained from the streamflow records compiled by the U.S. Geological Survey; rainfall data were obtained from the National Weather Services; values of urbanized areas were taken from the Land Study Bureau, University of Hawaii. These data are com-

piled in Table 5 and were used as input data for the multiple regression study. Locations of USGS stream gages are presented in Figure 9.

The evaluation of the peak discharge, Qp, as a function of the urbanized area, A_u, drainage area A, rainfall depth, R, and time to peak, T_p, were performed by computer using the standard program of the least square method to determine the empirical constants a, b, c, d, and f and correlation coefficient R.

Data of the mainstream length and average slope were obtained from Wu (1969) for the study of the time to peak for selected watersheds. Locations of the stream gages are shown in Figure 9.

TABLE 5. WATERSHED CHARACTERISTICS AND AVERAGE TIME PARAMETER OF SMALL HAWAIIAN WATERSHEDS.

WATERSHED NO.	WATERSHED AREA SQ MI	WATERSHED AREA ACRES	LENGTH OF MAIN STREAM FT	AVERAGE SLOPE OF MAIN STREAM %	HEIGHT OF WATERSHED FT	ESTIMATED TIME OF CONCEN-TRATION t_c HR	TIME PARAMETERS TIME TO PEAK t_p HR	TIME PARAMETERS RECESSION CONSTANT K_1 HR
2000	1.38	883	16,840	3.63	1510	0.59	1.11	0.76
2080	4.04	2586	49,100	0.73	1600	1.90	1.92	1.61
2116	2.13	1363	15,720	12.40	3121	0.41	0.38	0.72
2118	3.27	2093	12,000	13.30	3566	0.29	1.34	1.44
2128	4.29	2746	47,640	2.64	2096	1.70	0.95	0.74
2130	45.70	29,248					2.58	2.40
2160	26.40	16,896					0.90	1.61
2230	6.07	3885	50,940	1.58	2568	2.29	1.30	1.03
2245	2.59	1658	38,840	3.75	2792	1.21	0.25	0.75
2270	8.78	5519					1.25	2.80
2280	2.73	1747	22,840	3.50	2480	0.68	1.27	1.01
2290	2.61	1670	14,660	4.63	2276	0.42	0.71	0.66
2390	1.06	678	7283	12.60	1920	0.20	0.76	0.72
2400	1.14	730	9100	14.90	2814	0.23	0.66	0.83
2440	1.18	755	11,700	8.65	2141	0.34	0.75	0.70
2460	1.04	666	12,240	9.40	2165	0.35	0.83	0.73
2470	3.63	2323	22,480	4.92	2435	0.68	0.58	1.27
2540	2.04	1306	14,360	4.43	2540	0.40	0.94	0.54
2739	4.39	2803	25,480	1.15	2782	0.75	0.80	1.32
2750	0.97	621	5594	11.80	2128	0.14	0.70	0.48
2830	0.28	179	1438	23.00	2222	0.03	0.58	0.43
2838	0.31	198	2250	19.70	2069	0.05	0.55	0.59
2840	0.93	595	4875	11.40	2377	0.19	0.80	0.42
2910	0.99	634	3750	6.92	2500	0.09	0.79	0.45
2960	3.74	2394					0.75	0.94
2965	3.74	2394	21,700	2.73	2630	0.63	0.66	1.34
3030	2.78	1779					0.74	0.75
3300	9.79	6266	69,040	2.60	2260	2.53	0.90	2.43
3450	2.98	1907	68,300	1.81	1740	2.75	0.57	1.64

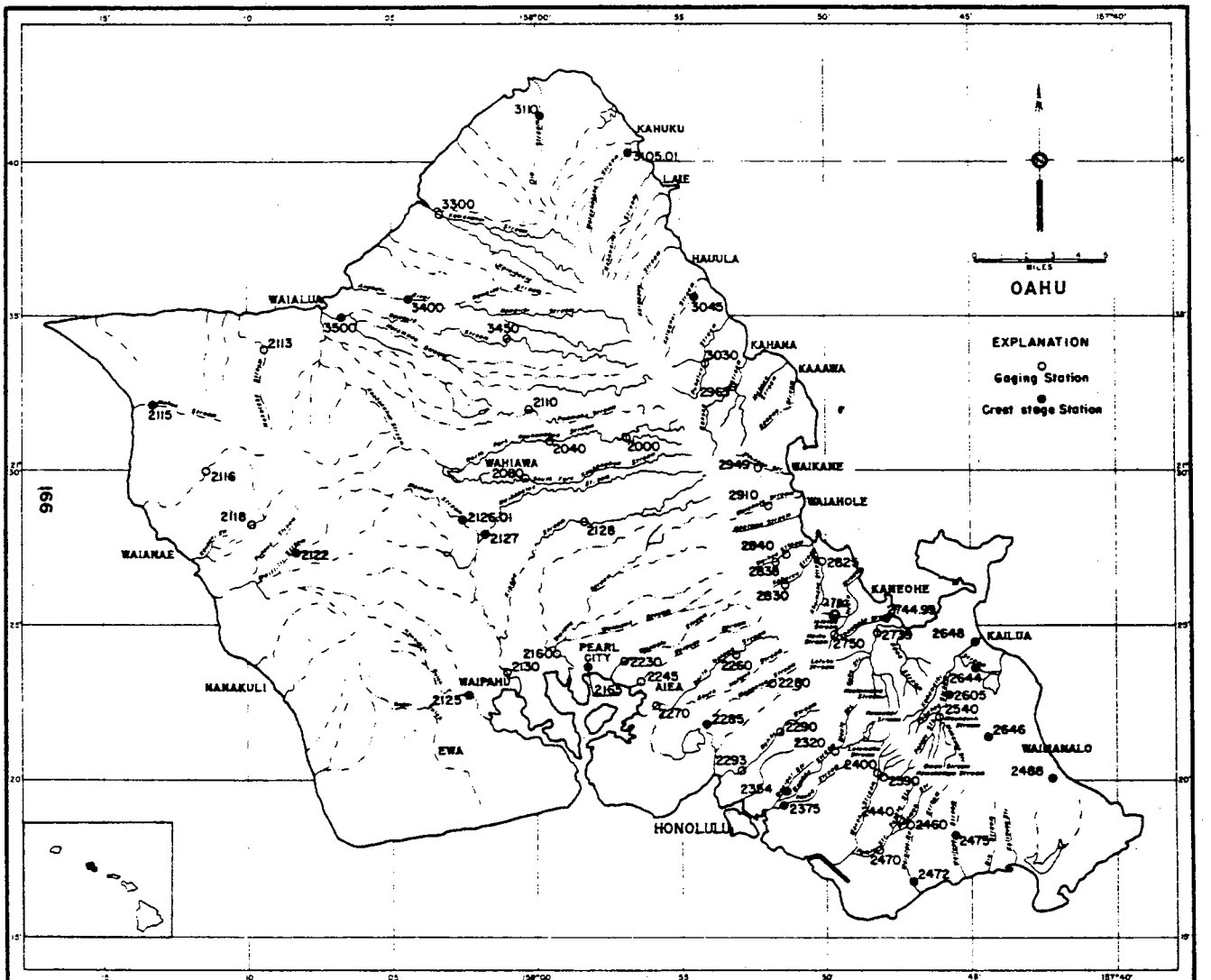

FIGURE 9. GAGING STATIONS ON OAHU, FISCAL YEAR 1967

The resulting empirical equations for selected urbanized watersheds on Oahu are listed according to their U.S. Geological Survey gaging station number:

Results of Peak Discharge Studies

Station No.	Equation	$\overline{R}$, Correlation Factor
2440	$Q_p = e^{(0.21 - 1.92\frac{Au}{A})} R^{0.98} T_p^{-0.37} A^{0.96}$	0.98
2460	$Q_p = e^{(-3.44 - 9.4\frac{Au}{A})} R^{0.8} T_p^{-0.25} A^{1.56}$	0.99
2470	$Q_p = e^{(4.75 + 3.53\frac{Au}{A})} R^{1.15} T_p^{-0.19} A^{0.25}$	0.90
2440, 2460, 2470	$Q_p = e^{(-3.92 - 2.88\frac{Au}{A})} R^{0.91} T_p^{-0.31} A^{1.57}$	0.96
2130, 2230, 2245, 2270	$Q_p = e^{(5.59 - 47.51\frac{Au}{A})} R^{1.63} T_p^{-1.54} A^{1.01}$	0.91
2440, 2460, 2470	$T_p = e^{7.526(\frac{Au}{A})^{0.07}} S^{1.18} L^{-0.05}$	0.56
2245, 2270, 2440, 2460, 2470	$T_p = e^{24.75(\frac{Au}{A})^{-0.03}} S^{-1.8} L^{-2.69}$	0.58

DISCUSSIONS. The selection of the empirical equation in the form of Equation (18) is good, considering the high correlation factors obtained from this study.

In general, the proposed empirical equation, Equation (18), can be considered as another form of the Rational Formula. Combining the terms of the rainfall depth and the time to peak may be considered the equivalent term for the rainfall intensity in the Rational Formula. The runoff coefficient, C, of the Rational Formula is bound by zero and one, $0 < C \leq 1$; however, the counterpart in Equation (18), $e(a+b\frac{Au}{A})$, is not necessarily bound by zero and one. The disagreements may be due to the fact that the

TABLE 6. IMPORTANT FLOOD HYDROGRAPH CHARACTERISTICS OF THE PALOLO WATERSHED.

STREAM GAGING STATION NO.	DRAINAGE AREA/ACRE	URBAN AREA (ACRES)	DATE	TIME TO PEAK (HOURS)	DURATION INDEX (HOURS)	PEAK DISCHARGE (CFS)	RAINFALL INDEX (INCHES)
2440	755	10.00	5-16-27	1.00	1.96	1160	2.03
		33.15	11-17-48	1.00	1.72	950	1.43
		33.15	12-30-60	0.50	0.91	1100	1.12
		75.5	11-14-65	1.65	2.28	428	0.97
		75.5	1-27-68	0.80	1.57	214	0.46
		75.5	4-16-68	0.75	1.52	114	0.16
2460	666	1.2	10-15-38	0.91	1.77	931	1.35
		4.74	11-17-49	0.75	1.77	781	1.06
		7.11	3-06-63	0.67	1.20	406	0.37
		66.00	1-27-68	0.75	1.01	115	0.39
		66.00	3-16-68	0.40	0.86	507	1.18
		66.00	11-14-69	1.35	1.97	92	0.23
		66.00	1-30-69	1.40	1.74	139	0.37
		66.00	1-03-70	2.50	2.75	243	0.95
2470	2323	464.6	12-03-50	0.92	2.12	2090	1.23
		464.6	3-11-51	0.50	1.89	1180	0.77
		627.0	12-29-60	0.33	0.67	1460	0.45
		675.0	2-04-65	0.25	0.66	1927	0.66
		675.0	10-13-65	0.27	0.69	1061	0.52
		675.0	11-12-65	0.62	1.19	1095	0.53
		675.0	11-14-65	1.00	1.83	3520	1.33
		768.0	1-27-68	0.41	1.66	1150	0.35
		791.0	3-16-69	0.73	1.48	674	0.28
		791.0	11-14-69	0.50	1.09	784	0.41

resulting empirical constants c and f are not equal to one, and d is not equal to negative one; or it may be concluded that sampling data were insufficient to provide the best statistical results. Evaluation of sampling data demands time and labor, which is not possible in this particular investigation of the study.

Because of the low correlation factor obtained from the evaluation of Eq. (19), this form of empirical equation is not recommended in evaluating the time to peak as input in Eq (18). A new time to peak empirical equation is yet to be found.

CONCLUSIONS. Eq. (18) may be considered a good empirical equation to demonstrate peak discharge as a function of the ratio of the urban area to drainage area, rainfall depth, time to peak, and the drainage area for the selected watershed on Oahu because all the correlation factors are above 0.9. Sufficient follow-up sampling data is desirable for each urban watershed on Oahu. The time to peak empirical equation registered a low corre-

lation factor, and therefore the adaptation of Eq. (19) is not recommended for any design. However, more data are needed to reevaluate the empirical constants in Eq. (10) in this particular investigation.

EXPANDED DATA COLLECTION PROGRAM

The 1969 ASCE Report, *Basic Information Needs in Urban Hydrology*, recommends the collection on a continuous basis of rainfall, runoff, and water quality data. In addition, studies of water balance of an urban watershed, soil moisture, infiltration, and evapotranspiration data are also recommended. In the Phase II study, the data collection effort in the St. Louis Heights research watershed was expanded from the Phase I rainfall-runoff data collection program into a rainfall, runoff, soil moisture, evaporation, and water quality data collection program for the period July 1972 to September 1973.

RAINFALL. Rainfall data were collected with natural siphon, auto-recording rainfall recorders at two locations in the St. Louis Heights watershed, namely the upper and middle gages, since January 19, 1972 as reported by Fok (1973). The recording of rainfall was changed from a weekly into a daily chart since September 1972 in order to provide better interpretation of 15-minute interval rainfall rates. The recording charts have been changed on a 4-day schedule. Locations of the rain gages and other hydrologic instruments are presented in Figure 10. Rainfall data have been stored in IBM cards based on 15-minute interval.

RUNOFF. In the water year 1971-72, the streamflow stage auto-recorders had been set to record the streamflow stages on a weekly chart in the two research watersheds, Waahila (natural), and St. Louis Heights (urban). However, in order to increase sensitivity on the time scale, the clocks of the recorders have been changed to cover the weekly chart in every 4 days since September 1972.

EVAPORATION. A U.S. Standard Evaporation Type A pan has been installed since August 1972 in the fenced-in water tank area in the upper gage area of the St. Louis Heights watershed. An evaporation meter has also been installed near the evaporation pan since January 1973. The evaporation pan was installed in this area because the lawn has never been irrigated

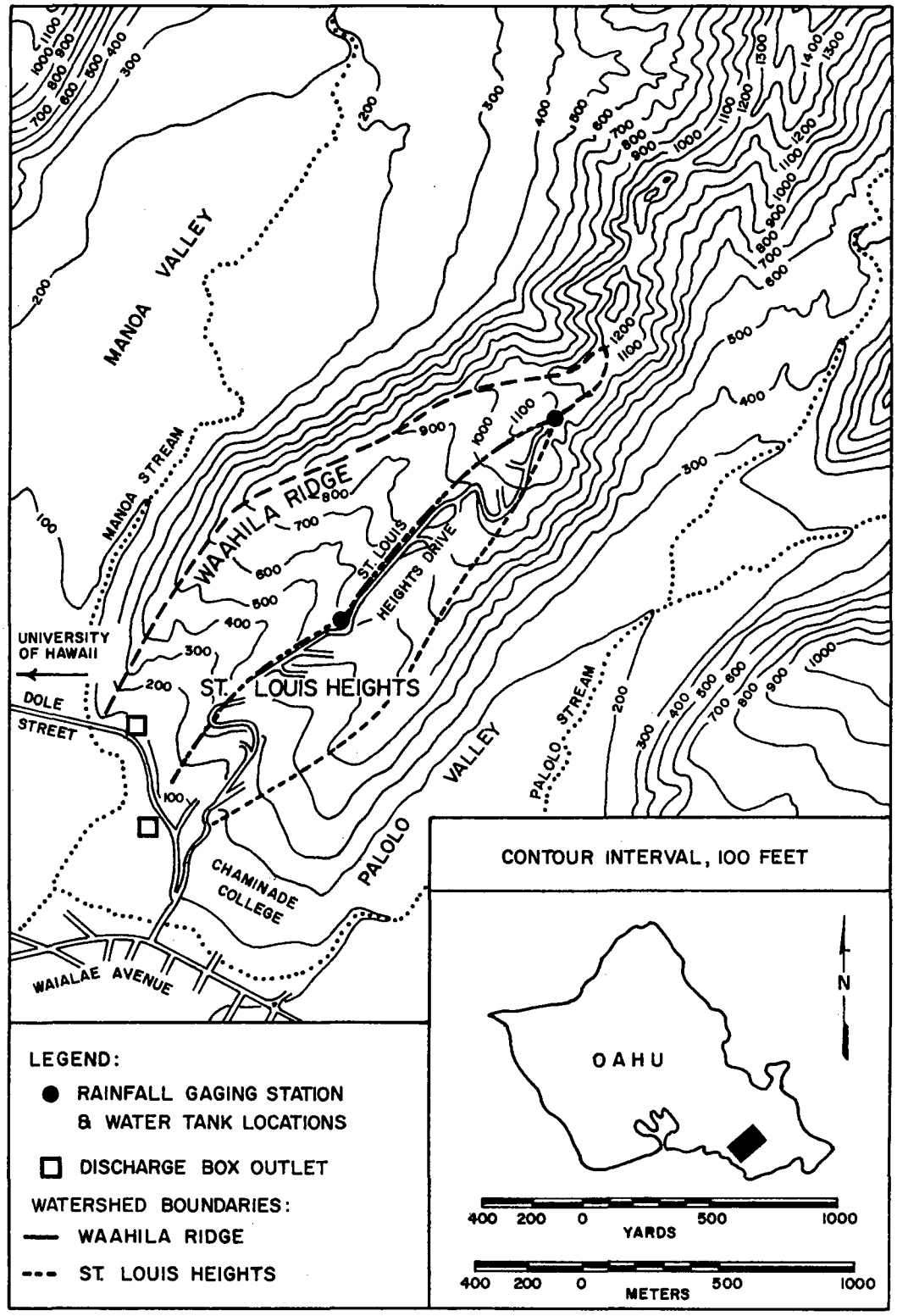

FIGURE 10. INSTRUMENTATION SITES IN THE ST. LOUIS HEIGHTS AND WAAHILA RIDGE WATERSHEDS.

by sprinklers; therefore, the patterns of natural evaporation have not been altered artificially.

SOIL MOISTURE. In order to monitor the variation of soil moisture in the soil profile on a long-term base, two kinds of soil moisture sensing blocks made of fiberglass and gypsum have been installed in the upper rain gage area. In the soil profile, pairs of fiberglass and gypsum blocks were installed at 1st, 2nd, 3rd, 4th, and 5th foot depth from the soil surface. Soil samples were also collected for use in the soil moisture content versus electrical resistance reading calibration study. The apparent specific gravity of the soil samples were also determined as:

Soil Depth (feet)	1	2	3	4	5
Apparent Specific Gravity	1.434	1.414	1.395	1.255	1.255

WATER QUALITY OF STORMWATER. In order to learn more about the storm water quality in relation to time and the storm hydrograph, water quality tests were made on total solids, total organic carbon, total nitrogen, total phosphorus, chloride, color units, and specific conductivity. The measurement procedures used were the standard methods of the American Public Health Association (1971).

Water quality samplers were planned to be installed nearby the streamflow stage recorders in order to determine the storm water quality from the natural watershed (Waahila Ridge) and from the urban watershed (St. Louis Heights). Since samples for various intervals were needed, the Sigmamotor Automatic sampler, Model WM-2-24, was selected. This particular sampler contained 24 individual bottles which were usually filled at 10-minute cycles during which the sampler would pump the fluid for 6 minutes with a 4-minute interval before taking the next sample. This sampler has also a 15-foot pumping head capacity which is ideal for this study.

RESULTS. Due to difficulties in establishing a platform for the sampler by the stream gage of the St. Louis Heights watershed, no water quality sample was taken from the St. Louis Heights watershed.

The study encountered a dry water year during 1972-73;only a very few samples were taken from the Waahila watershed and it was therefore very difficult to draw useful conclusions from them. Based on the limited data available, the following water quality trends were observed:

(1) Total solids ranged from 150 mg/l to 440 mg/l in the storm water; increased or decreased in quantity in relation to the fluctuations of the storm hydrograph;

(2) Total organic carbon ranged from 2 ppm to 20 ppm; at the beginning increased and fluctuated over time, then decreased slowly as the storm runoff decreased very rapidly;

(3) Total nitrogen ranged from 0.01 ppm to 0.04 ppm and increased and decreased as the storm hydrograph rose and fell;

(4) Total phosphorus ranged from 0.1 ppm to 0.8 ppm, increased rapidly but decreased slowly with the storm hydrograph;

(5) Chloride ranged from 5 ppm to 90 ppm, increased in leveling off and slowly decreased with the storm hydrograph;

(6) Color units of the storm runoff increased quickly and remained constant, leveled off, whereas in the storm hydrograph, the storm runoff decreased rapidly;

(7) Specific conductivity ranged from 80 to 1600 micromohs/sq cm fluctuated from low to high, then from high to low and rose again in relation to the storm hydrograph.

CONCLUSIONS. It should be stressed here that more observed data are needed to substantiate fully the above observations on water quality of the Waahila Ridge watershed.

CONCLUSIONS

In this study a search for a better model or models for the simulation of urban hydrology suitable for use on Oahu is planned. On the other hand, it is also planned to continue the hydrological data collection program of the adjacent Waahila Ridge and St. Louis Heights research watersheds.

The following points have been drawn from conclusions made at the end of individual task investigations throughout this report:

1. The Nash successive routing method is the better model for the instantaneous unit hydrograph study on Oahu. The parameter, N, of Nash's Model was observed to decrease when urbanization increased.
2. The adaptation of the Kentucky Watershed Model (KWM) to conditions on Oahu would require two or three more years. Preliminary results using the KWM to simulate daily flow is good.

3. The use of the Road Research Laboratory model to simulate urban storm runoff has not been completed due to input data errors.
4. The empirical multiple regression study correlating peak discharge with an urbanized area has been found very promising.
5. The nonlinear time variant watershed rainfall runoff simulation model is found to be the best of the three models tested. The results of the linear time invariant (IUH) model would give good results in peak discharge simulation.
6. More adequate and reliable hydrological data are needed on Oahu for the development of a better hydrologic simulation model.

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