AD-A102 189 HYDROLOGIC ENGINEERING CENTER DAVIS CA F/G 8/8
CONTINUOUS HYDROLOGIC SIMULATION OF THE WEST BRANCH DUPAGE RIVE--ETC(U)
JUL 79 R J CERMAK

UNCLASSIFIED HEC-RN-6 NL

END
DATE
FILMED
8 81
DTIC

LEVEL II

(13)

AD A102189

RESEARCH NOTE NO. 6

CONTINUOUS HYDROLOGIC SIMULATION OF THE WEST BRANCH DUPAGE RIVER ABOVE WEST CHICAGO: AN APPLICATION OF HYDROCOMP'S HSP

Robert J. Cermak

HEC-RN-6

JUL 1979

DTIC FILE COPY

HEC

THE HYDROLOGIC ENGINEERING CENTER
•research
•training
•application

CORPS OF ENGINEERS
U. S. ARMY

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Research Note No. 6	2. GOVT ACCESSION NO. AD-A102 189	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) CONTINUOUS HYDROLOGIC SIMULATION OF THE WEST BRANCH DUPAGE RIVER ABOVE WEST CHICAGO: AN APPLICATION OF HYDROCOMP'S HSP		5. TYPE OF REPORT & PERIOD COVERED
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Robert J. Cermak		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U.S. Army Corps of Engineers The Hydrologic Engineering Center 609 Second Street, Davis, CA 95616		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS		12. REPORT DATE July 1979
		13. NUMBER OF PAGES 71
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Distribution of this publication is unlimited

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Urban hydrology, Illinois, Model studies, Watersheds(Basins), River basins, Mathematical models, Precipitation(Atmospheric), Evaporation, Infiltration, Land use, Calibrations, Discharge frequency, Discharge(Water), Urban runoff, Simulation and analysis, DuPage River Basin(IL), Data requirements.

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The purpose of this report was to document the calibration and application of an HSP model for the DuPage River basin, and to evaluate HSP as a tool in urban hydrologic analysis. Topics were discussed in the following order: model structure, parameters, and data requirements; a description of the study basin, parameter calibration, and land use; calibration results, long record simulation, estimation of discharge frequency, and conclusions. An HSP continuous hydrologic (CONTINUED)

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20 (CONTINUED)

simulation model of the West Branch DuPage River above West Chicago was calibrated and verified, long-record precipitation and other meteorologic data were assembled and provided as input to the model, and resulting annual peak discharges used to estimate discharge frequency at the gage site for constant present conditions. Continuous hydrologic simulation provides a viable alternative as a method of analysis for urban hydrologic systems. It requires large amounts of data and significant labor to calibrate an HSP model to a specific basin. For studies that warrant such a detailed definition of the hydrologic process, the model provides a rational representation of basin hydrology that could be applied to the investigation of complex water resource problems.

RESEARCH NOTE NO. 6

CONTINUOUS HYDROLOGIC SIMULATION OF THE WEST BRANCH DUPAGE RIVER ABOVE WEST CHICAGO: AN APPLICATION OF HYDROCOMP'S HSP

by

Robert J. Cermak

July 1979

The Hydrologic Engineering Center
U.S. Army Corps of Engineers
609 Second Street
Davis, California
95616

FOREWORD

This Research Note describes the experience of the Hydrologic Engineering Center in applying Hydrocomp's HSP, a continuous hydrologic simulation computer program, to model the West Branch DuPage River above West Chicago, Illinois.

The major source of funding for this study was Work Unit No. 31007, Effects of Urbanization on Flood Discharges, of the Analytical Techniques - Water Resources Planning Studies, Corps of Engineers R & D Program. Additional funds were provided by the Chicago District Corps of Engineers.

CONTENTS

CONTENTS (Continued)

LIST OF TABLES

CONTENTS (Continued)

LIST OF FIGURES

Introduction

Land use change and urban development are known to effect a watershed's hydrologic response to storm rainfall. Formerly agricultural and open fields become partially covered with an impervious surface of rooftops, driveways, parking lots, and roads, connected directly to the channel by storm sewers. This transformation of land cover tends to (1) increase the total volume of direct runoff by decreasing the opportunity for infiltration, and (2) reduce the time it takes runoff to reach the channel system by traveling in efficient hydraulic conduits rather than on top of or through the soil; thus, increasing the peak flow in the stream. As one result, discharge frequency curves, derived from a series of annual peaks that occurred under varying urban conditions, will be inadequate for representing the present probability of flooding in the basin.

The Chicago District and North Central Division were concerned that just such a situation exists in the DuPage River Basin. The District has contracted with an A/E firm to update and expand a HEC-1 model of the basin (Ref. 3). The A/E was required to calibrate loss rates against available discharge frequency curves. Significant urbanization has taken place in parts of the basin over the past 15 years. For example, between 1964 and 1975 rural land use decreased in one subbasin from 84 to 51 percent. Annual peaks for this period represent a nonstationary time-series and would bias both discharge frequency curves and HEC-1 model parameters calibrated to the curves.

The Hydrologic Engineering Center (HEC) was asked in July, 1977 by the District to make a separate study of the basin using Hydrocomp's HSP, a continuous hydrologic simulation computer program. HSP can explicitly account for spatial and temporal variation in the proportion of impervious area, a surrogate measure of urban development, and can generate a long record of annual peaks from observed precipitation. Continuous simulation models are calibrated only to observed rainfall-runoff events and not probabilistic estimates of annual peak discharge. The intended products from HEC's effort were: (1) discharge frequency curves computed from simulated peaks and representing constant present conditions; and (2) general recommendations, based on the modeling experience, concerning the value and requirements of continuous hydrologic simulation in urbanizing basins.

The purpose of this report is to document the calibration and application of an HSP model for the DuPage River Basin, and to evaluate HSP as a tool in urban hydrologic analysis. Topics are to be discussed in the following order: model structure, parameters, and data requirements; a description of the study basin, parameter calibration, and land use; calibration results, long-record simulation, estimation of discharge frequency, and conclusions. We begin with a brief look at how the model works. (A complete technical description of HSP can be found in Reference 1.)

Model Structure

HSP attempts to simulate continuously the complete hydrologic cycle of a watershed. Given precipitation and potential evapotranspiration, the LANDS module accounts for the following: (1) storage, as interception, upper zone (surface or near-surface), lower zone (sub-surface), and groundwater; and (2) flow, impervious runoff, interflow, overland flow, infiltration, percolation, base flow and actual evapotranspiration. Inflows to the channel system are routed downstream in the CHANNELS module using kinematic wave routing. Figure 1 is a schematic of HSP's major components.

The model has the additional capability to simulate the complex snowfall-snowmelt process. Accumulation of precipitation in a snowpack and its eventual release as snowmelt is modeled using the equations published in the Corps of Engineers "Snow Hydrology". Melt is the combined result of direct solar radiation, convection, condensation, rain on snow, and groundmelt. Precipitation and temperature decide when snow is falling and, together with snow density, determine depth and equivalent water content of the pack.

Model Parameters

Sixteen LANDS parameters, listed and defined in Table 1, control the capacity of storages and the functional relationship between flow rates and storages. For some, numerical values can be assigned directly from maps or other data; e.g., overland flow length and slope, groundwater and interflow recession constants, percent impervious area, and segment-to-gage rainfall ratio. But others, including the parameters most important for accurate calibration of runoff volume and peak flow timing and magnitude, are not physically based and require trial-and-error adjustment. Hydrocomp suggests "typical" value ranges for upper and lower zone storage nominal, infiltration, and interflow. However, several runs of the model with different parameter sets and/or previous HSP experience in the same geographical region are still necessary for accurate calibration.

Twelve additional parameters are involved in the snowmelt routine (Table 1). Mean elevation, elevation difference, and forest cover can be measured from maps but the other nine variables are difficult to set without previous experience in modeling snowmelt for similar meteorological conditions.

The channel system is divided into reach lengths having approximate constant cross-sectional geometry and hydraulic roughness. Time-of-travel at bankfull capacity and the simulation's computational time step are also considered when setting up the CHANNELS model. For trapezoidal channels, each reach is defined by 9 parameters (Table 2): channel length and slope, contributing drainage area, top and bottom width, depth, flood plain slope, and Manning's n for channel and flood plain.

Data Requirements

Hourly precipitation and potential evapotranspiration, derived from daily pan evaporation data, are the basic input data required by LANDS. Continuous daily streamflow and hourly hydrographs for selected storms are used for comparison with simulated flows. Meteorologic data prerequisites for snowmelt include daily: maximum and minimum air temperature, wind movement, dewpoint temperature, cloud cover, and incident solar radiation. Point source discharges, such as from municipal or industrial sewage treatment plants, can contribute a surprising large share to the total volume of annual runoff. To achieve an accurate account of water balance, records for the large point sources should be included in the data base.

The time-series data mentioned above provide both initial input to the model and a standard to evaluate the simulated output. Supplemental information on more static characteristics of the watershed are needed for estimating parameter values. Land use/land cover help determine: imperviousness, forest cover, and hydraulic roughness for overland flow. Slope, length, soil permeability, and channel cross-section geometry define several parameters in LANDS and CHANNELS. When basin characteristics change with time, as land use does in urbanizing basins, multiple measurements may be necessary.

Description of Study Basin

The DuPage River is located 20 miles west of Chicago (Figure 2) and has a drainage area of 324 square miles (measured above the USGS stream gaging station near Shorewood, Illinois). The East and West Branches of the DuPage River join below the town of Naperville and together account for nearly two-thirds of the basin drainage. The largest concentrations of urban land are along the East Branch and in the Cook County portion of the West Branch. Below their confluence, the mainstem of the DuPage River collects runoff from predominantly agricultural and rural lands. Continued urban development has been projected for both East and West Branches.

It was originally intended for HEC to study the entire watershed and to determine discharge frequency curves at all of the basin's six stream gaging stations. However, after learning how to use HSP and applying the model to one DuPage River subbasin it was decided that time, data, and funding limitations would prevent development of a complete basin model. Rather, one subbasin was selected for full analysis: West Branch DuPage River above the USGS stream gage near West Chicago. The selected subbasin (abbreviated in this report as "WBWC") reflects, in miniature, the development situation facing the entire basin. Recent residential construction in the headwater villages of Hanover Park and Schaumburg has changed the subbasin's hydrologic behavior and could significantly effect channel flows through lower rural areas. Restricting the application of HSP to WBWC should not, therefore, interfere with the study's primary objective: to evaluate a continuous hydrologic simulation model under changing urban conditions.

Calibration of Model Parameters

This study was fortunate in being able to acquire a data base for the whole basin from the Northeastern Illinois Planning Commission (NIPC). A product of a previous NIPC water quality study of the DuPage River, the data base contained 10 years of meteorologic, streamflow, and point source data. Table 3 lists the data series used in the calibration of the WBWC model. Land use classification at three points in time, maps, reports, and channel cross-sections were obtained from NIPC and the Chicago District and were very useful in setting up the model structure and estimating model parameters. NIPC's water quality study utilized HSP's LANDS, CHANNELS, AND QUALITY modules. Initial values of many variables in the present study were derived from NIPC's Hydrologic Calibration report (Reference 2), a documentation of their own application of LANDS and CHANNELS to the DuPage River.

The ten-year data base was divided into two intervals, 1965-69 and 1970-74, and analyzed separately. Model parameters were calibrated using the earlier period data and a four-step iterative procedure:

1. Assign new parameters values.

2. Run simulation on 5-year calibration data.

3. Compare simulated with observed streamflow.

4. Determine which LANDS parameters should be changed.

   Return to step 1.

When predicted and observed runoff were sufficiently close the model was run unchanged on the remaining 5-year verification data. If this test showed major systematic bias the calibration process was restarted. Otherwise, the model was accepted as a valid representation of basin hydrology.

Initially, four land surface runoff (LSRO) segments were modeled: agricultural, grassland, forest, and impervious. There was a problem, however, in calibrating with such a detailed land use breakdown. At the gaging stations only total runoff from all surfaces is known and not the individual component runoff from agricultural land, grassland, etc. Differences between parameter values assigned to the several LSRO segments cannot, therefore, be justified by observed data. When parameters were set to what appeared to be reasonable values, it was noticed that the most important parameters remained the same for all segments.

Because of the above reasons, a simpler two segment model was adopted. A distinction was made between runoff from impervious surfaces and runoff from everthing else, referred to as URBAN and

RURAL, respectively. Since only impervious runoff is permitted in URBAN, values assigned to the LANDS parameters for this segment are arbitrary*. In contrast, RURAL allows for all possible runoff except from impervious surfaces. Its final LANDS parameter values, including snowmelt, are given in Table 4.

The structure of CHANNELS was held constant for all computer runs. The WBWC subbasin was split into three channel reaches of nearly equal contributing drainage area. Runoff from URBAN and RURAL segments was added in proportion to the observed land use of each reach. Cross-section dimensions and the other CHANNELS parameters are listed in Table 4.

Land Use Imperviousness

Inventories of land use/land cover reported by NIPC for 1964, 1970, and 1975 provided a means for quantifying change in the amount of impervious surface between 1965 and 1974. First, dissimilar classifications were collapsed into three categories: HI, LO, and RURAL, as shown in Table 5. Then land use was translated into effective imperviousness by the formula IMP=(0.1)LO + (0.2)HI. An attempt was made to relate imperviousness and land use by actual measurement of roof-tops, driveways, streets, etc., from aerial photographs. This approach, however, produced unrealistic high imperviousness values. The above

*Except for A=1.0 (i.e., 100% imperviousness) and the snowmelt parameters.

simple linear equation provided reasonable estimates of present imperviousness and would, at a minimum, be able to show an increase in imperviousness with increasing urban development.

Applying the imperviousness relationship to 1970 and 1975 land use (Table 6.1), percent impervious surface (Table 6.2) was calculated for the total subbasin and the contributing drainage area of each channel reach. The proportional combination of land surface runoff from URBAN and RURAL segments is identical to the impervious/pervious breakdown of Table 6.2.

Calibration/Verification Results

The HSP model was evaluted by how well it reproduced runoff volumes (annual, monthly, and daily) and runoff peak discharges. Even in a study, such as the present, where peak flows are the main object of interest, it is necessary to be able to generate volumes accurately over long periods of time. This is because of the significant effect antecedent storage volumes above and below the soil surface have on the volume and timing of storm runoff. Annual and monthly volumes are shown in Tables 7 and 8 for the calibration and verification periods, respectively. It is apparent from the tables that March to April were generally low in simulated volume and July to September generally high. Spring runoff, either rain plus melting snow or rain on frozen ground, was very difficult to simulate due to the complex nature and data uncertainties of the snowmelt process. Runoff from summer thunderstorms is likewise a problem because of the hit-or-miss relationship between small local storm cells and a single point observation of rainfall.

Figures 3 and 4 are accumulative, or mass balance, comparisons of these same simulated and recorded runoff volumes. They show that after an early positive error corresponding to the summer storms of 1965, the mass balance remained roughly paralled to the "balance" line until June 1970. Then a 21 month period of consecutive high volumes began, followed by 2-1/2 years of lows. Relative to the calibration period the model tested poorly for volume in the verification years. Partial explanation of this change in performance can be found in Figure 5.1 where annual runoff is plotted against annual precipitation. Notice the last 3 years of the 1970-74 period had significantly higher runoff-to-precipitation ratios (indicated by the slope of line segments from the origin) than in the 1965-69 period. Using published data for 1975-77, the trend for an increased runoff-rainfall ratio is, in general, continued to be observed. It is reasonable to assume the increase in runoff per unit rainfall is caused in part by increased urbanization. Figure 5.2, a plot of both accumulative annual precipitation and accumulative annual runoff against time, supports this hypothesis by contrasting stationary basin rainfall with a visably changed incremental runoff volume. The break in slope of the runoff curve corresponds directly with the occurrence of large-scale residential construction in the upper region of the subbasin.

As mentioned before in the land use discussion, a change was made in the proportion of impervious surface from 2.6% in 1965-69 to 5.5% in 1970-74. It may be that a larger increase was necessary to generate higher runoff volume for the latter period. To be sure a detailed examination of several individual storm volumes should be made. Data limitations prevented such an analysis from being included in the present study.

Mean daily flows can be considered as both average flow rate (cfs) and daily runoff volume (cfsd). Continuous hydrographs for water years 1965-74, Figures 6.1 to 6.10, present a graphical comparison of simulated and observed daily flows. Plots such as these were routinely generated during calibration. They provided an opportunity to see not only peaks and total volumes but also the relative contribution of component flows; e.g., surface runoff, interflow, and groundwater. The average of the simple correlation coefficients (r), calculated separately for each water year on Figures 6.1 - 6.10, was 0.82.

A primary study objective was the generation of long record synthetic peak discharge. Consequently, a great deal of attention was given to the accurate reproduction of large peak flows. Tables 9 and 10 list observed and simulated peak discharge for events corresponding to the USGS published partial-duration series*. Mean error for the

*Not included in the tables are events for which either (1) the simulated peak was not printed on the computer output, or (2) the observed discharge was below the specified base of the partial-duration series.

calibration period was much smaller than for the verification period (+2.0% vs -22.6%), with both periods having about equal "spread" in the distribution of error. Simulated peaks for water years 1971-74 were consistently low. An explanation identical to that provided for the occurrance of low flow volumes applies to the present case; i.e., a combined result of unusually high runoff (for a given precipitation) and inadequate increase in impervious surface.

Long-Record Simulation

When the incremental improvement in model fit became small for successive changes in parameter values, the calibration process was stopped. The final set of parameter values and the percent imperviousness used in the verification runs (i.e., "present conditions") together make the HSP model a mechanism for translating observed precipitation into streamflow at the subbasin outlet. Having a calibrated model, the next task was to assemble a long continuous record of the necessary meteorologic variables.

Both the length and accuracy of a simulation are frequently limited by the data available, or rather, unavailable. There were very few hourly recording stations in existence 25 years ago. And even if collected, precipitation data did not begin to be systematically coded in computer-compatible format by the National Weather Service until 1948. Depending upon local spatial variation in precipitation, the quality of a simulation may be adversely affected by the distance to and location of the rain gage(s). These same factors apply to the other required meteorologic data.

The longest hourly precipitation record in the vicinity of the basin is station 1582, Chicago WB City, located in downtown Chicago near the lakefront. Data has been collected here since July 1899, but available on magnetic tape only from September 1948. Therefore, the period of long-record simulation was determined by default to be water years 1949-64*. Pan evaporation data for this same period was not readily available. Instead potential evapotranspiration at O'Hare Airport for 1965-74, as used in the calibration/verification phase, was duplicated twice to create an artificial potential evapotranspiration record for 1949-64. A similar procedure was followed to construct radiation and wind speed data files for the long-record years. Minimum and maximum daily temperatures were measured at the Chicago WB City station and were used in the snowmelt calculations. Table 11 summarizes the long record-simulation input data.

Table 12 contains the simulated monthly and annual flow volumes (all generated for constant "present conditions") for the following time intervals: (1) long-record period, 1949-64;

---

*Because the Chicago WB City station was discontinued in November 1964, a double mass plot of accumulative precipitation for Chicago WB City vs. Roselle could not be made. Data was available for two years, 1967-68, for the replacement station Chicago WB City #2. However, due to the short concurrent period, a double mass plot against the Roselle station was not conclusive. Without additional information, we were forced to assume that the long-record Chicago WB City data is representative of the true basin precipitation for the period 1949-64.

(2) calibration period, 1965-69, rerun under present land use conditions; and (3) verification period, 1970-74, identical to values previously reported in Table 8. Table 13 lists both simulated annual peak discharge for these same periods and the corresponding published values of observed peak flow.

Discharge Frequency

Discharge frequency curves were computed from the simulated and observed annual peak series of Table 13, and plotted as Figure 7.1. The curves represent the probability distribution of annual peak discharge for two distinct populations: the real (observed) world and the model (simulated) world. Although the two curves appear to be different (which is what we would expect them to be, as discussed later), it will be useful to measure how significant their difference really is.

Discharge frequency curves are defined by assuming a distribution (in our case, log-Pearson Type III) and estimating the necessary parameters (mean, standard deviation, and skew). A generalized skew value of 0.0 was adopted for the present study. Hypothesis tests were performed on the remaining parameters to determine, in probabilistic terms, if the differences in sample statistics were significant*. Using a t-test to examine the hypothesis H: $\mu_1 = \mu_2$ (that is,

*Details of the tests are given in Appendix A.

equivalence of means), it was found that at a significance level $\alpha = 0.40$ the hypothesis could not be rejected. This implies that even with the test structured so that there was a 40% chance of rejecting a true hypothesis (Type I error) the equivalence of means was not rejected. An F-test of the hypothesis H: $\sigma_1^2 = \sigma_2^2$ (equal variance) showed that at a significance level $\alpha = 0.50$ the null hypothesis once again could not be rejected. From a statistical viewpoint, therefore, the difference between the two sample means and the difference between the two sample variances are not significant.

Despite the statistical comparison, there are several reasons why we should expect the curves to be different.

(1) The observed frequency curve is based on a series of annual peaks that occurred under varying degrees of urbanization whereas the simulation series, and the frequency curve associated with it, was deliberately intended to represent constant "present" urban conditions.

(2) The simulation record consisted of twenty-six annual peaks for the period 1949-74. This is a longer and different set of years than the observed record's eighteen peaks from the period 1961-78.

(3) The model is a simplification of the real world's hydrologic and hydraulic processes. It uses discrete point measurements

of the basic input, precipitation, and attempts to generalize this information both in time and space. It must, therefore, generate peak flows that do not agree exactly with the actual storm events.

(4) As described previously in the section on long-record simulation, both a different recording station (e.g., precipitation and temperature) and an artificial data record (e.g., potential evapotranspiration and solar radiation) were used to generate streamflow for the period 1949-64. The magnitude of the error introduced into the simulation by the non-representativeness of these data is unknown, but such error surely exists.

An attempt was made to examine separately some of the possible explanations of why the curves of Figure 7.1 should be different. In Figure 7.2 is a plot of the simulated and observed frequency curves for the same ten years, 1965-74. Recall that this was the calibration/verification period for the simulation model, and that during this period the model's percent imperviousness was changed from 2.6% in 1965-69 to 5.5% in 1970-74. This transition was made to try to imitate the actual change in urbanization that was occurring in the watershed. Hypothetically, the two curves of Figure 7.2 should represent the same changing urban conditions and differ only in the degree to which the model was unable to imitate the real world.

As before, hypothesis tests were made on the equivalence of means and the equivalence of variances. It was found that the hypothesis H: $\mu_1 = \mu_2$ could not be rejected at the $\alpha = 0.50$ significance level, and the hypothesis $\sigma_1^2 = \sigma_2^2$ could not be rejected at the $\alpha = 0.20$ significance level. The conclusion that should be made is the following: the small sample size ($n = 10$) does <u>not</u> permit the inference (from the observed differences in sample means and variances) that the corresponding population parameters are significantly different.

However, visual examination of Figure 7.2 shows that at the tails of the distribution (low and high exceedance probabilities) the curves do depart. For example, at the 1% event the simulated frequency curve is 230 cfs higher than the observed frequency curve (1310 vs. 1080). To put this magnitude in perspective, 230 cfs is more than twice the difference between the 1% and 2% flood events of the observed curve (1080 and 982, respectively). The plotted annual peaks for this time period reveals a general trend: the three largest simulated peaks exceed or are nearly equal to the observed peaks, whereas the remaining seven simulated peaks are all lower than the comparable observed peaks. Rather than random variation about the observed data there appears to be a systematic bias in the calibrated model of oversimulating large events while undersimulating small ones.

Figure 7.3 illustrates the effect of the longer simulation record. Discharge frequency curves were computed from the simulated peaks (at constant present urban conditions) for two overlapping periods: 1949-74 (26 years) and 1961-74 (14 years). Both hypotheses, H: $\mu_1 = \mu_2$ and H: $\sigma_1^2 = \sigma_2^2$, were not rejected at the $\alpha = 0.50$ significance level. This result should have been expected as the two curves were derived from non-disjoint samples of the same population. Notice in Figure 7.3 the difference between the simulated frequency curves based on different lengths of record is small relative to the difference between simulated and observed curves for the same period, 1961-74.

The simulated frequency curve of Figure 7.1 was intended to show (by means of the simulated peaks) what would have been the actual frequency curve if urbanization were held constant at its present (1974) level and annual peak data were available for the longer 1949-74 period. That hypothetical present-day (1974) frequency curve is contrasted in Figure 7.1 with the curve derived from historically observed peak streamflows. From the above analysis it seems unlikely that changing urban conditions and/or a longer record can explain all the differences between the two curves. Differences due to these causes have very likely been overshadowed by the model, as calibrated, generating annual peak events that contain the systematic bias previously discussed.

Conclusions

The application of HSP to urban watersheds for the purpose of determining discharge frequency can be evaluated by the following criteria:

(1) How accurate was the model's simulation? Accuracy can be judged at four levels: monthly and annual volumes, daily flows, peak discharge, and discharge frequency.

(a) The average absolute monthly volume error (Tables 7 and 8) was 32.1% and 28.1% for the calibration and verification periods, respectively. A seasonal pattern of underestimating volumes in the spring and overestimating volumes in the late summer tended to cancel each other, as evidenced by the smaller absolute yearly volume errors of 11.1% and 16.1% for calibration and verification, respectively.

(b) Comparing daily flows will show how the model performed in distributing runoff volume into daily increments. The correlation of mean daily flows within each of the ten calibration/verification water years, Figures 6.1 to 6.10, ranged from 0.75 to 0.92, and averaged 0.82.

(c) Peak discharges for 27 events in the calibration period and 35 events in the verification period were available for comparison. The 5-year calibration group (Table 9) had an average absolute error of 26.0%. High and low errors tended to compensate for each other, producing a simple average error of 2.0%. Peaks from the 5-year verification period (Table 10) had an average absolute error of 36.0% and a simple average error of -22.6%, demonstrating a consistent low bias for nearly all storm events in this period. Possible reasons for this behavior of the model were previously discussed.

(d) A series of statistical tests on the discharge frequency curve parameters (Figure 7.1) concluded that differences between sample statistics were not significant. Examination of the potential reasons as to why the curves should be (and visually appear to be) different demonstrated that error in the calibrated model was the primary cause of this difference.

(2) Can the model account for changing urbanization? Theoretically yes - the model can be structured (e.g., separate pervious and impervious land surface runoff segments) to conveniently account for changing urban conditions. The problem arises, as is the case in all urban hydrologic models, in going from known land use/land cover to

the amount of effective impervious surface. Other changes associated with urban areas, e.g., channel improvements or flood control structures, can be included in the model.

(3) Is the model easy to use?

To operate - yes, to calibrate - no. Model structure, parameter values, and input data to be used and output data to be saved are all specified by simple program statements. The storage, retrieval and manipulation of large time-series data structures, and the selection of graphical and statistical operations are defined with a set of program commands written in natural language with a logical well-defined syntax. Without previous experience, however, calibration to a specific study basin can be a difficult and time-consuming task. There are sixteen parameters in LANDS to manipulate and with snowmelt almost twice that many. Several LANDS parameters initially assigned on the basis of maps or other data were found later to require changing to obtain an acceptable fit. The most important variables (UZSN, LZSN, INTERFLOW, INFILTRATION) are not physically derived. Without having experience with the model in a similar hydrologic area it is very hard to know what values to start with for these parameters, or how much to change them on successive computer runs.

(4) What are the data requirements?

Extensive, both in quantity of data and hours of labor necessary for preprocessing. Hourly precipitation and daily evaporation data are required for input and daily streamflow to evaluate simulation output. Hourly precipitation prior to 1948 is seldom available in computer format and the enormous job of locating, coding, cleaning, and keypunching could be restrictive in some studies. The snowmelt routine requires at least semi-daily temperature; if not available, daily radiation, wind speed, and dewpoint can be estimated. Temperature is usually measured at many stations but the other meteorologic variables, including evaporation, can be difficult to obtain.

(5) Can runoff from several different land surfaces be modeled?

In theory - yes. The model is structured so that runoff caused by any possible combination of hydrologic and meteorologic factors can be generated separately; e.g., from different rain gages for forest, pasture, residential, and agricultural land cover. The problem in using this capability is not knowing what the individual runoff from each segment should be, only their aggregate contribution at the streamgage. Parameter modifications for multi-land use watersheds must therefore be based on rational thought rather than empirical evidence.

(6) What additional benefits are there in continuous simulation? One benefit is a greater understanding of and appreciation for the complexity of a watershed's hydrologic cycle. Relative contributions of surface runoff, interflow, and groundwater are noticed as are evaporation, infiltration, and percolation. The seasonal pattern of rainfall-runoff and their variations in volume from year-to-year are made explicit. Perhaps having improved knowledge of a basin's hydrologic behavior will permit a greater number and/or variety of water resource management alternatives to be considered.

(7) How long does it take and what does it cost to develop an HSP model?
Starting with a nearly complete hydrologic/meteorologic data base, it took one person 6 months to assemble and analyze additional data, and to learn how to use the model. Another 6 months were spent in calibration and long-record simulation. Time gained by experience (e.g., shortening of calibration time) would probably be canceled by what would have been required to construct the original data base. A realistic estimate for studying a basin similar to the DuPage River would be 9 to 12 person-months. The cost for a single LANDS simulation of the 5-year calibration period was about \$30. Total computer cost should probably be less than \$2,000. Although these figures apply to the modeling of a

28.5 mi^2 basin, it is expected that cost would increase less than proportionately when modeling a larger watershed.

(8) Are continuous simulation models well suited for studies involving only the estimation of discharge frequency in urbanizing basins?

They do the job, but considering the amount of work (and data) required, it is doubtful that they are the best tool for such limited studies. More comprehensive study objectives (for example, water supply, flood control, or water quality, which explicitly consider runoff volume and water balance) would take advantage of the continuous simulation's detailed analysis. Discharge frequency under changing urban conditions is a problem that could be handled by simpler, quicker, less costly approaches requiring much less data; e.g., design storms or several historical events used as input to a single-event model, or a continuous model with a less complex soil-moisture accounting algorithm.

In summary, an HSP continuous hydrologic simulation model of the West Branch DuPage River above West Chicago was calibrated and verified, long-record precipitation and other meteorologic data were assembled and provided as input to the model, and resulting annual peak discharges used to estimate discharge frequency at the gage site for constant present conditions. Considering the major strengths and weaknesses identified above, the following conclusion can be made.

Continuous hydrologic simulation provides a viable alternative as a method of analysis for urban hydrologic systems. It requires large amounts of data and significant labor to calibrate an HSP model to a specific basin. For studies which warrant such a detailed definition of the hydrologic process, the model provides a rational representation of basin hydrology that could be applied to the investigation of complex water resource problems.

Acknowledgments

Many individuals contributed to the present study, and five deserve special mention: Dennis Dreher (Northeast Illinois Planning Commission), Vefa Yucel (Hydrocomp), James Mazanec (Chicago District, COE), Art Pabst (HEC), and Arlen D. Feldman (HEC). Their participation was extremely helpful and greatly appreciated.

References

1. Hydrocomp, Hydrologic Simulation Operations Manual, Jan. 1976.

2. NIPC, DuPage River Hydrologic Calibration, Dec, 1977.

3. Corps of Engineers, Chicago District, Preliminary Feasibility Report, CSELM, DuPage River Basin, Sept 1978 (Draft).

TABLE 1*
DUPAGE RIVER BASIN
MODEL PARAMETER DEFINITIONS
LANDS

LAND

K1	Ratio of average segment rainfall to average gage
A	Impervious area (fraction)
EPXM	Interception storage (maximum value)
UZSN	Nominal upper zone soil moisture storage
LZSN	Nominal lower zone soil moisture storage
K3	Actual evaporation rate parameter
K24L	Seepage to 'deep' groundwater
K24EL	Evaporation from perched groundwater
INFILTRATION	Infiltration
INTERFLOW	Interflow
L	Length of overland flow
SS	Overland flow slope (ft/ft)
NN	Manning's N for overland flow
IRC	Daily interflow recession rate
KV	Groundwater recession, variable rate
KK24	Groundwater recession, constant rate

SNOW

RADCON	Radiation melt parameter
CONDS-CONV	Convection melt parameter
SCF	Snow correction factor to gage record
ELDIF	Elevation difference (gage to segment)
IDNS	Initial density of new snow
F	Forest cover
DGM	Daily ground melt (inches)
WC	Water content of snowpack maximum
MPACK	Snowpack at complete areal coverage
EVAPSNOW	Snow evaporation parameter
MELEV	Mean watershed segment elevation (ft)
TSNOW	Upper limit of temperature at which precipitation is snow

* Adopted from reference 2.

TABLE 2*
DUPAGE RIVER
MODEL PARAMETER DEFINITIONS
CHANNEL

REACH	Reach Number
LIKE	Reach number *that has an identical cross section*
TYPE	The type of channel:
	RECT: Trapezoidal channel cross section
	CIRC: Circular Conduit
	IMAG: Feeder reach without routing
	DAM : Reservoir
TO#	Reach number to which the reach is tributary
SEG#	Land surface segment that contains the reach
LEN	Length of the reach in miles
AREA	Local area tributary to the reach in sq. miles
UPSTR	Upstream channel bottom elevation in the reach
DNSTR	Downstream channel bottom elevation in the reach
W1	Incised channel bottom width in feet for trapezoidal channels, or the diameter in inches for circular channels
W2	Incised channel top width in feet for trapezoidal channels or Manning's N for circular channels
DEPTH	Incised channel depth in feet
S-FP	Transverse slope of the flood plain in feet per foot
N-CH	Manning's N for the incised channel
N-FP	Manning's N for the flood plain

* Adopted from reference 2.

TABLE 3

Calibration Data

Data	Interval	Location
Precipitation	Hourly	Roselle
Evaporation	Daily	Chicago O'Hare
Temperature	Semi-Daily	Chicago O'Hare
Wind	Daily	Chicago O'Hare
Cloud Cover	Daily	Chicago O'Hare
Dewpoint	Daily	Chicago O'Hare
Radiation	Daily	Chicago O'Hare
Streamflow	Daily	West Branch DuPage River near West Chicago
Point Source	Daily	Hanover #1 STP
Point Source	Daily	Hanover Park STP
Point Source	Daily	Old Bartlett STP

TABLE 4

Adopted Parameter Values

LANDS		SNOWMELT		CHANNELS		
K1	.95	RAD-CON	1	RCH	5	10	15
A	0	CONDS-CONV	1	LIKE	0	0	0
EPXM	.1	SCF	1.6	TYPE	RECT	RECT	RECT
UZSN	1.5	ELDIF	.1	TRIB-TO	10	15	20
LZSN	10	IDNS	.1	SEGMT	1	2	3
K3	.4	F	.045	LENGTH	5.9	3.5	4.4
K24L	.4	DGM	.01	TRIB-AREA	10.2	9.0	9.3
K24EL	0	WC	.05	EL-UP	817	765	743
INFL	.03	MPACK	.5	EL-DOWN	765	743	719
INTR	22	EVAP-SNOW	.1	W1	14	18	21
L	350	MELEV	800	W2	22	43	37
SS	.02	TSNØW	33	H	4	5	4
NN	.25			S-FP	.01	.01	.01
IRC	.6			N-CH	.04	.04	.045
KV	1.5			N-FP	.045	.08	.085
KK24	.99						

TABLE 5

Land Use/Land Cover Classifications

1964 NIPC	1970 NIPC	1975 LANDSAT	PRESENT STUDY
Commercial Industiral Mining	Manufacturing Trade Services-Private Warehouse Shopping Centers Hotel/Motel Parking Mining	High-Intensity Developed Land	HI
Residential Streets Transpo/Com/ Utilities Public Bldgs	Res-Single Fam Res-Milti Fam Mobile Homes Streets Transpo/Com/Util Airports Railroads Services-Inst Public Bldgs Military	Low-Intensity Developed Land	LO
Public Open Space Agric/Vacnt Water	Public Open Space Entertainment Assembly Cemetary Vacant-Under Devlp Vacant/Agric/Forst Water	Grass Trees Soybeans Corn Other Crops Open Space Water	RURAL

Table 6.1

Land Use Distribution (Percent)

Reach #	DA (mi^2)	1970 HI	1970 LO	1970 RURAL	1975 HI	1975 LO	1975 RURAL
5	10.2	1.8	39.1	59.1	9.0	59.9	31.1
10	9.0	1.1	15.6	83.3	7.0	49.6	43.4
15	9.3	1.4	10.7	87.9	1.2	18.4	80.4
WBWC	28.5	1.5	22.7	75.8	5.9	43.5	50.6

Table 6.2

Impervious Surfac Distribution (Percent)

Reach #	DA (mi^2)	1970 Impervious[1]	1970 Pervious[2]	1975 Impervious[1]	1975 Pervious[2]
5	10.2	4.3	95.7	7.8	92.2
10	9.0	1.8	98.2	6.4	93.6
15	9.3	1.4	98.6	2.1	97.9
WBWC	28.5	2.6	97.4	5.5	94.5

1. Impervious = (.1) LO + (.2)HI
2. Pervious = 100 - (Impervious)

NOTE: Typical Entry
Recorded Volume (Inches)
Simulated Volume (Inches)
Difference (Inches)
Difference (%)

TABLE 7

RUNOFF VOLUME COMPARISON

1965-69 CALIBRATION PERIOD

WATER YEAR	ANNUAL	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
65	11.64	.16	.28	.42	1.33	1.80	1.93	3.20	.80	.36	.26	.32	.77
	14.59	.21	.22	.37	1.70	1.66	1.46	2.83	.86	.87	1.02	1.42	1.98
	2.95	.05	-.06	-.05	.37	-.14	-.47	-.37	.06	.51	.76	1.10	1.21
	25.3	31.3	-21.4	-11.9	27.8	-7.8	-24.4	-11.6	7.5	141.7	292.3	343.8	157.1
66	11.33	.50	.34	1.15	.70	1.86	1.35	1.60	2.80	.65	.22	.11	.05
	11.82	.64	.50	1.11	.61	1.01	1.11	2.06	2.96	.92	.41	.28	.21
	.49	.14	.16	-.04	-.09	-.85	-.24	.46	.16	.27	.19	.17	.16
	4.3	28.0	47.1	-3.5	-12.9	-45.7	-17.8	28.8	5.7	41.5	86.4	154.5	320.0
67	13.50	.10	.32	.61	.27	.34	2.34	3.05	1.07	3.75	.78	.56	.30
	12.64	.33	.57	.74	.45	.73	.81	2.29	1.21	3.34	.88	.83	.46
	-.86	.23	.25	.13	.18	.39	-1.53	-.76	.14	-.41	.10	.27	.16
	-6.4	230.0	78.1	21.3	66.7	114.7	-65.4	-24.9	13.1	-10.9	12.8	48.2	53.3
68	8.81	.41	1.14	.98	.59	.91	.50	.85	.61	.52	.41	1.38	.51
	10.91	.60	1.01	1.21	.73	.94	.56	.73	.48	.57	.63	2.86	.59
	2.10	.19	-.13	.23	.14	.03	.06	-.12	-.13	.05	.22	1.48	.08
	23.8	46.3	-11.4	23.5	23.7	3.3	12.0	-14.1	-21.3	9.6	53.7	107.2	15.7
69	12.90	.31	.61	1.28	1.99	.66	.84	2.20	.93	1.78	1.40	.48	.43
	12.82	.38	.61	.81	1.67	.86	.73	1.63	.84	2.48	1.36	.45	1.00
	-.08	.07	.0	-.47	-.32	.20	-.11	-.57	-.09	.70	-.04	-.03	.57
	-0.6	22.6	.0	-36.7	-16.1	30.3	-13.1	-25.9	-9.7	39.3	-2.9	-6.3	132.6
65-69	58.18	1.48	2.69	4.44	4.88	5.57	6.96	10.90	6.21	7.06	3.07	2.85	2.06
	62.78	2.16	2.91	4.24	5.16	5.20	4.67	9.54	6.35	8.18	4.30	5.84	4.24
	4.60	.68	.22	-.20	.28	-.37	-2.29	-1.36	.14	1.12	1.23	2.99	2.18
Avg Abs YVE = 11.1%	7.9	45.9	8.2	-4.5	5.7	-6.6	-32.9	-12.5	2.3	15.9	40.1	104.9	105.8

Percent Average Absolute Monthly Volume Error (MVE)
M = number of months
MQ = monthly observed volume (inches)
MS = monthly simulated volume (inches)

$$\text{Avg Abs MVE} = \left[\frac{\sum_{i=1}^{M} \left| \frac{MS_i - MQ_i}{M} \right|}{\sum_{i=1}^{M} \frac{MQ_i}{M}} \right] (100) = 32.1\%$$

Percent Average Absolute Yearly Volume Error (YVE)
Calculated similar to MVE

NOTE: Typical Entry
Recorded Volume (Inches)
Simulated Volume (Inches)
Difference (Inches)
Difference (%)

TABLE 8

RUNOFF VOLUME COMPARISON

1970-74 VERIFICATION PERIOD

WATER YEAR	ANNUAL	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
70	17.61 17.83 .22 1.2	1.29 1.52 .23 17.8	.97 .65 -.32 -33.0	.48 .39 -.09 -18.8	.43 .59 .16 37.2	.65 .53 -.12 -18.5	1.39 .86 -.53 -38.1	2.39 1.50 -.89 -37.2	3.13 3.21 .08 2.6	3.44 3.79 .35 10.2	.80 1.04 .24 30.0	.49 .64 .15 30.6	2.16 3.12 .96 44.4
71	8.79 11.74 2.95 33.6	.80 .95 .15 18.8	1.02 1.12 .10 9.8	1.34 1.47 .13 9.7	.32 .95 .63 196.9	1.43 1.64 .21 14.7	1.51 1.47 -.04 -2.6	.69 .86 .17 24.6	.56 .76 .20 35.7	.42 .60 .18 42.9	.26 .68 .42 161.5	.25 .62 .37 148.0	.18 .61 .43 238.9
72	23.21 18.01 -5.20 -22.4	.25 .51 .26 104.0	.28 .53 .25 89.3	1.36 1.21 -.15 -11.0	.59 .78 .19 32.2	.45 .56 .11 24.4	2.05 1.03 -1.02 -49.8	3.08 1.91 -1.17 -38.0	1.56 1.07 -.49 -31.4	3.20 1.57 -1.63 -50.9	2.26 1.34 -.92 -40.7	3.61 3.93 .32 8.9	4.52 3.58 -.94 -20.8
73	21.19 19.22 -1.97 -9.3	2.52 1.90 -.62 -24.6	2.27 1.46 -.81 -35.7	1.65 1.66 .01 0.6	2.11 2.26 .15 7.1	1.12 .98 -.14 -12.5	2.52 1.72 -.80 -31.7	3.10 2.35 -.75 -24.2	2.35 1.97 -.38 -16.2	1.34 1.15 -.19 -14.2	1.02 1.75 .73 71.6	.43 .60 .17 39.5	.77 1.41 .64 83.1
74	22.89 18.13 -4.76 -20.8	1.30 1.85 .55 42.3	.64 .75 .11 17.2	2.09 1.38 -.71 -34.0	2.96 2.72 -.24 -8.1	2.27 1.71 -.56 -24.7	2.63 1.98 -.65 -24.7	2.14 1.65 -.49 -22.9	4.13 2.22 -1.91 -46.2	2.76 1.87 -.89 -32.2	.83 .88 .05 6.0	.65 .76 .11 16.9	.48 .37 -.11 -22.9
70-74 Avg Abs YVE = 16.1%	93.69 84.93 -8.76 -9.3	6.16 6.73 .57 9.3	5.18 4.51 -.67 -12.9	6.92 6.11 -.81 -11.7	6.41 7.30 .89 13.9	5.92 5.42 -.50 -8.4	10.10 7.06 -3.04 -30.1	11.40 8.27 -3.13 -27.5	11.73 9.23 -2.50 -21.3	11.16 8.98 -2.18 -19.5	5.17 5.69 .52 10.1	5.43 6.55 1.12 20.6	8.11 9.09 .98 12.1

Percent Average Absolute Monthly Volume Error (MVE)
M = number of months
MQ = monthly observed volume (inches)
MS = monthly simulated volume (inches)

$$\text{Avg Abs MVE} = \left[\frac{\sum_{i=1}^{M} \left| \frac{MS_i - MQ_i}{M} \right|}{\sum_{i=1}^{M} \frac{MQ_i}{M}} \right] (100) = 28.1\%$$

Percent Average Absolute Yearly Volume Error (YVE)
Calculated similar to MVE

TABLE 9

PEAK DISCHARGE COMPARISON
CALIBRATION PERIOD 1965-69

WATER YEAR	DATE	Peak Discharge (cfs) RECORDED	SIMULATED	DIFFERENCE (cfs)	DIFFERENCE (%)
65	1/2	151	174	23	15.2
	1/23	116	164	48	41.4
	2/7	250	207	-43	-17.2
	4/1	180	147	-33	-18.3
	4/6	196	215	19	9.7
	4/15	113	104	-9	-8.0
	4/25	105	119	14	13.3
66	12/25	133	151	18	13.5
	2/10	371	152	-219	-59.0
	4/21	106	114	8	7.5
	4/24	124	140	16	12.9
	4/28	144	315	171	118.8
	5/12	537	474	-63	-11.7
67	4/1	461	255	-206	-44.7
	4/30	120	163	43	35.8
	6/10	805	912	107	13.3
	6/17	201	114	-87	-43.3
	6/28	133	112	-21	-15.8
68	8/17	340	651	311	91.5
69	11/29	111	115	4	3.6
	1/23	223	203	-20	-9.0
	1/30	142	138	-4	-2.8
	4/5	183	130	-53	-29.0
	4/18	192	111	-81	-42.2
	6/9	399	358	-41	-10.3
	7/1	100	102	2	2.0
	7/20	177	154	-23	-13.0
				Average Error	2.0
				Average Absolute Error	26.0
				Standard Error	38.2

TABLE 10

PEAK DISCHARGE COMPARISON
VERIFICATION PERIOD 1970-74

WATER YEAR	DATE	Peak Discharge (cfs) RECORDED	SIMULATED	DIFFERENCE (cfs)	DIFFERENCE (%)
70	5/1	237	186	-51	-21.5
	5/14	205	286	81	39.5
	6/2	521	438	-83	-15.9
	6/13	317	241	-76	-24.0
	6/27	180	305	125	69.4
	9/7	186	400	214	115.1
	9/25	207	199	-80	-3.9
71	2/20	333	120	-213	-64.0
72	12/15	344	259	-85	-24.7
	3/13	209	118	-91	-43.5
	4/17	404	229	-175	-43.3
	4/22	324	135	-189	-58.3
	6/15	562	231	-331	-58.9
	6/20	555	292	-263	-47.4
	7/18	380	130	-250	-65.8
	8/26	715	687	-28	-3.9
	9/14	438	288	-150	-34.2
	9/18	448	377	-71	-15.8
	9/29	378	290	-88	-23.3
73	10/12	195	153	-42	-21.5
	10/23	259	138	-121	-46.7
	12/30	535	444	-91	-17.0
	4-22	383	300	-83	-21.7
	5/28	154	103	-51	-33.1
	6/17	158	177	19	12.0
74	10/13	234	157	-77	-32.9
	12/5	234	125	-109	-46.6
	1/21	244	227	-17	-7.0
	1/27	522	314	-208	-39.8
	3/5	220	187	-33	-15.0
	4/14	167	111	-56	-33.5
	5/17	472	182	-290	-61.4
	5/22	376	151	-225	-59.8
	6/10	243	222	-21	-8.6
	6/23	168	113	-55	-32.7
				Average Error	-22.6
				Average Absolute Error	36.0
				Standard Error	36.9

TABLE 11

Input Data for Long-Record Simulation

Water Years 1949-64

Variable	Interval	Data Source
Precipitation	Hourly	Chicago WB City; missing data filled in from Chicago University.
Potential Evapotranspiration	Daily	Double duplicate of Chicago O'Hare, 1965-74.
Temperature	Daily	Chicago WB City; missing data filled in from Chicago University.
Wind Speed	Daily	Double duplicate of Chicago O'Hare, 1965-74.
Radiation	Daily	Double duplicate of Chicago O'Hare, 1965-74.

TABLE 12

Simulated Monthly and Annual Flow Volumes
(Inches of runoff for 28.5 sq. mi.)
Present Land Use Conditions

WATER YEAR	ANNUAL	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1949-64													
49	10.44	.17	.24	.38	1.15	1.60	1.39	1.42	.52	1.62	1.46	.24	.25
50	16.98	.35	.23	1.38	2.30	1.22	3.34	2.97	.64	2.51	1.34	.47	.23
51	11.42	.23	.20	.27	1.27	1.49	1.22	1.18	1.73	.55	1.36	.83	1.10
52	13.15	.84	1.23	.51	3.11	.82	2.06	2.21	.76	.64	.38	.36	.21
53	6.66	.15	.29	.50	.46	.58	1.40	.77	.82	.65	.43	.38	.22
54	11.20	.16	.13	.29	.25	.46	2.16	2.88	.75	1.09	1.26	1.47	.30
55	12.99	4.74	.42	.61	.77	1.04	1.29	.91	.52	.85	.73	.74	.34
56	10.72	2.24	.61	.59	.33	.86	1.27	1.66	1.64	.32	.34	.56	.27
57	13.35	.11	.15	.15	.24	.51	.69	1.97	1.14	1.71	3.85	2.36	.46
58	8.90	.70	1.30	.67	.85	.74	.47	.55	.38	1.09	1.55	.28	.32
59	11.32	.29	.22	.17	.13	1.05	2.45	2.10	1.57	.81	1.72	.56	.26
60	12.63	.50	.87	.72	2.49	1.56	1.61	1.24	1.04	1.28	.72	.41	.19
61	11.20	.24	.20	.16	.16	.20	1.15	1.25	.50	.34	.38	.40	6.22
62	11.39	1.12	.77	.44	.75	1.37	3.19	.89	1.01	.37	.92	.38	.18
63	6.56	.19	.15	.09	.07	.14	.65	1.08	1.48	.61	1.39	.44	.25
64	5.54	.15	.12	.13	.15	.13	.65	1.80	.81	.40	.61	.25	.35
1965-69													
65	15.44	.20	.27	.44	1.80	1.72	1.51	2.89	.90	.94	1.15	1.55	2.08
66	12.31	.65	.55	1.17	.61	1.04	1.15	2.17	3.02	.99	.47	.28	.22
67	13.54	.43	.69	.79	.48	.78	.86	2.38	1.24	3.50	.96	.91	.53
68	11.72	.69	1.04	1.27	.75	.96	.57	.81	.55	.68	.72	2.99	.68
69	13.63	.42	.71	.86	1.75	.84	.76	1.70	.93	2.65	1.44	.47	1.10
1970-74													
70	17.83	1.52	.65	.39	.59	.53	.86	1.50	3.21	3.79	1.04	.64	3.12
71	11.74	.95	1.12	1.47	.95	1.64	1.47	.86	.76	.60	.68	.62	.61
72	18.01	.51	.53	1.21	.78	.56	1.03	1.91	1.07	1.57	1.34	3.93	3.58
73	19.22	1.90	1.46	1.66	2.26	.98	1.72	2.35	1.97	1.15	1.75	.60	1.41
74	18.13	1.85	.75	1.38	2.72	1.71	1.98	1.65	2.22	1.87	.88	.76	.37

TABLE 13

Simulated and Recorded Annual Peak Discharge

West Branch DuPage River near West Chicago

	Simulated+		Recorded	
WY	Date	Peak Discharge (cfs)	Date	Peak Discharge (cfs)
49	6-15	438		
50	6-3	522		
51	5-10	364		
52	1-14	460		
53	3-14	196		
54	3-25	421		
55	10-10	947		
56	10-6	403		
57	7-12	1432		
58	7-2	340		
59	4-28	450		
60	1-12	473		
61	9-14	756	9-26	450
62	7-2	378	3-19	361
63	7-19	351	4-30	217
64	7-7	214	7-19	201
65	7-14	312	2-7	250
66	5-12	508	5-12	537
67	6-10	1025	6-10	805
68	8-17	739	8-17	340
69	6-9	389	6-9	399
70	6-2	438	6-2	521
71	12-11	208	2-20	333
72	8-26	687	8-26	715
73	12-30	444	12-30	535
74	1-27	314	1-27	522
75			4-19	537
76			3-5	557
77			3-29	158
78			9-18	468*

+ Constant "present" land use conditions *preliminary

Figure 1

FLOWCHART FOR HYDROCOMP'S HYDROLOGIC SIMULATION PROGRAM (HSP)

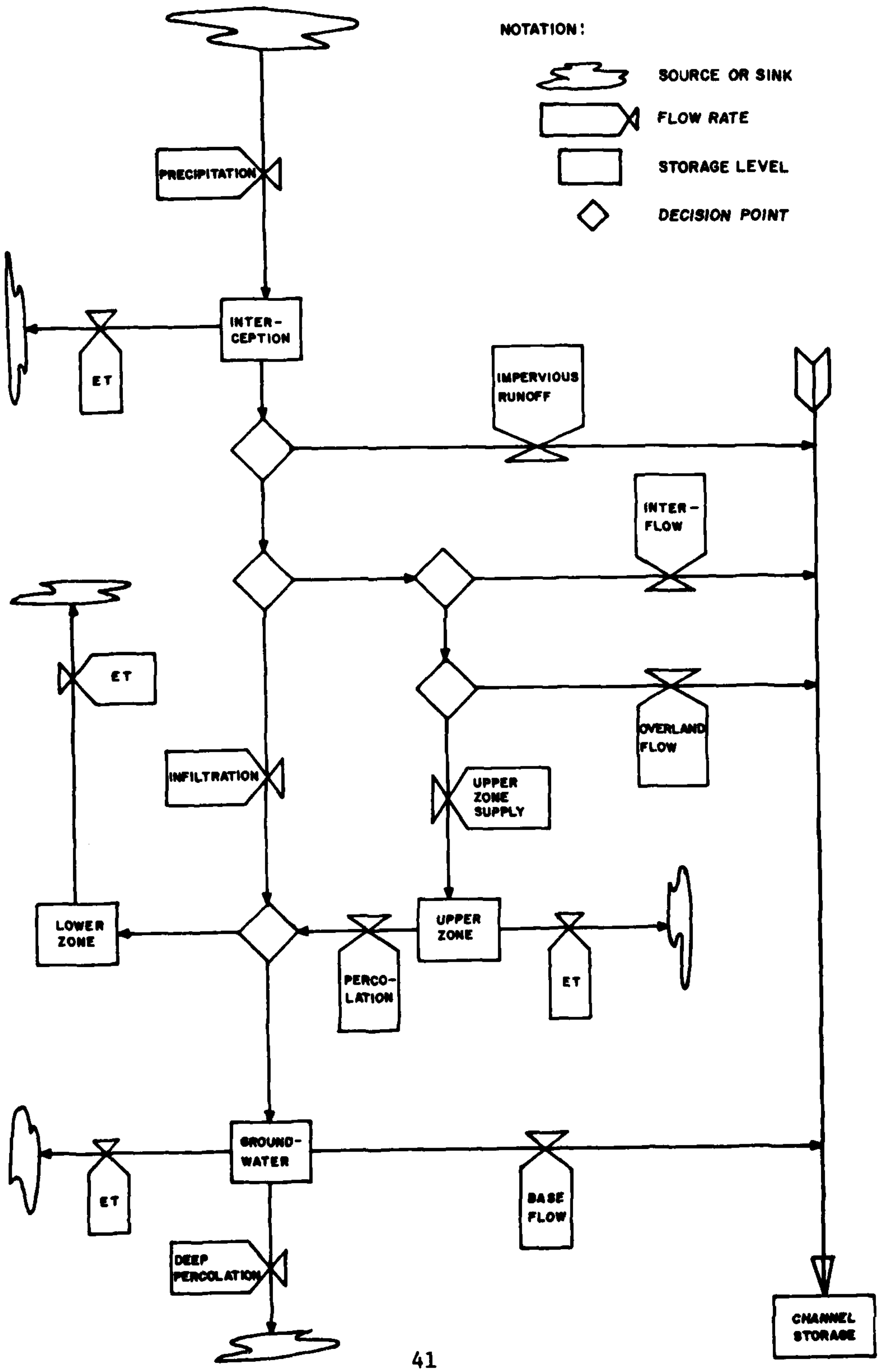


Figure 2

LOCATION MAP

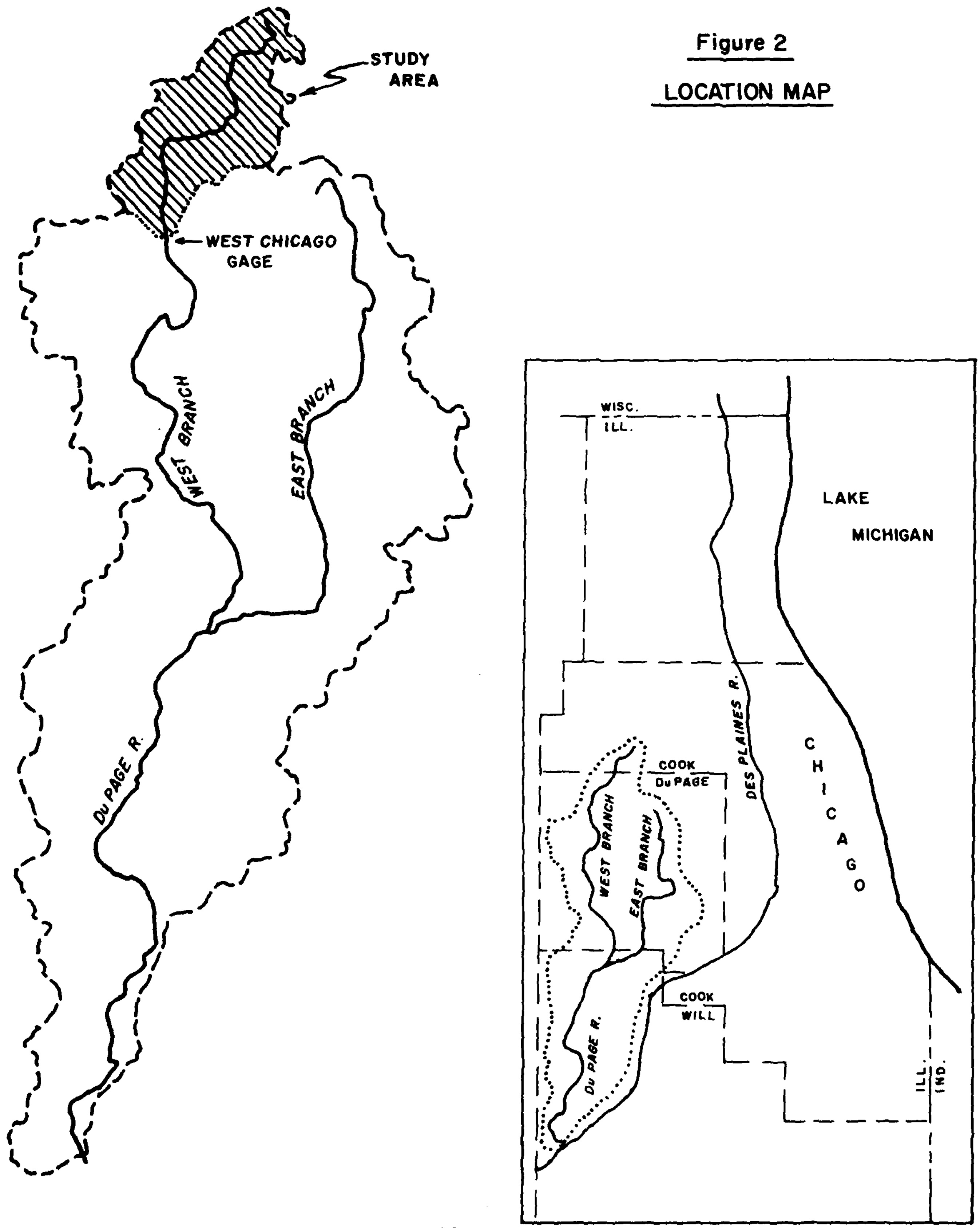
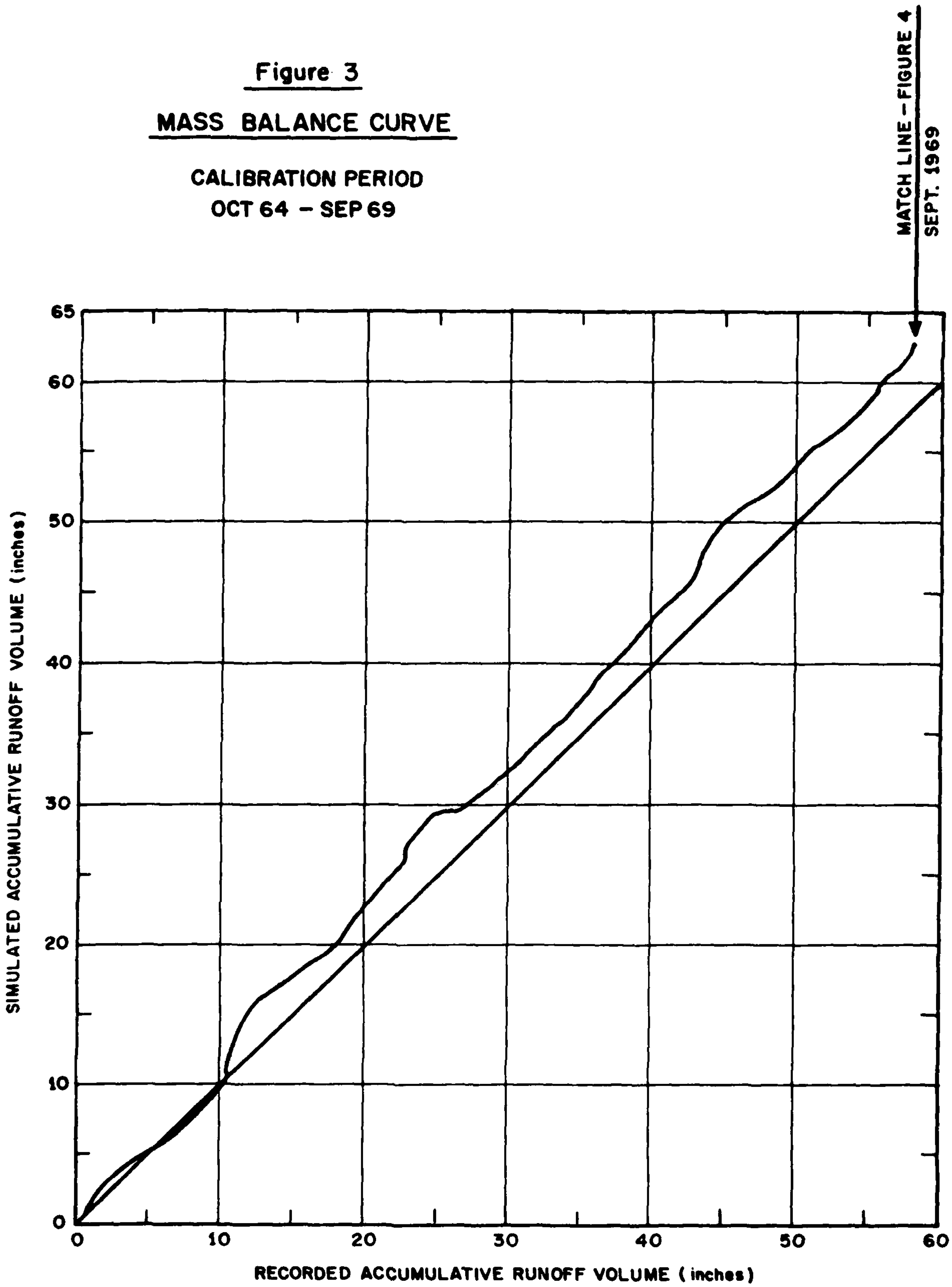
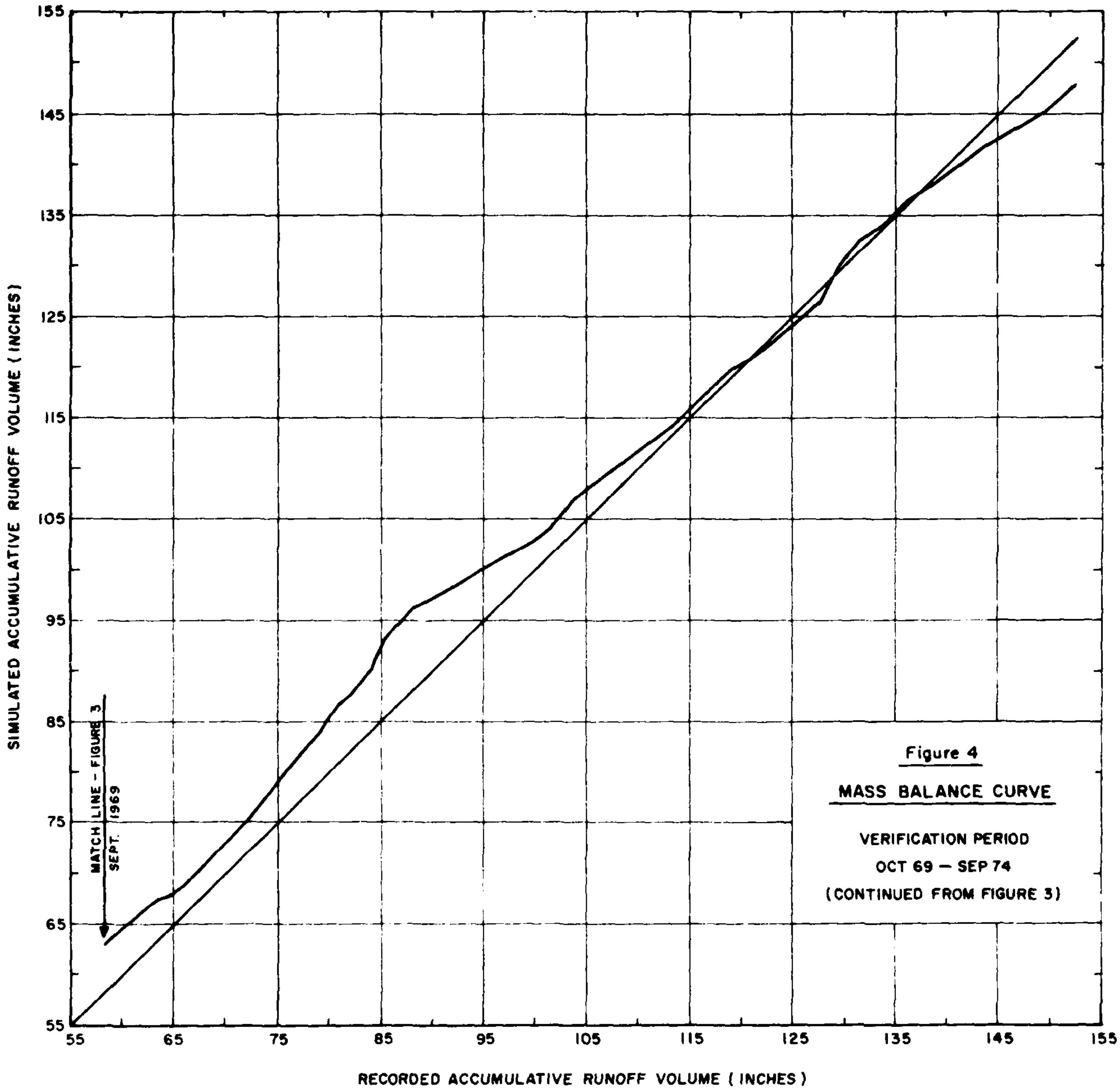


Figure 3

MASS BALANCE CURVE

CALIBRATION PERIOD
OCT 64 – SEP 69




155
145
135
125
115
105
95
85
75
65
55
55
65
75
85
95
105
115
125
135
145
155
SIMULATED ACCUMULATIVE RUNOFF VOLUME (INCHES)
RECORDED ACCUMULATIVE RUNOFF VOLUME (INCHES)
MATCH LINE - FIGURE 3
SEPT. 1969
Figure 4
MASS BALANCE CURVE
VERIFICATION PERIOD
OCT 69 — SEP 74
(CONTINUED FROM FIGURE 3)

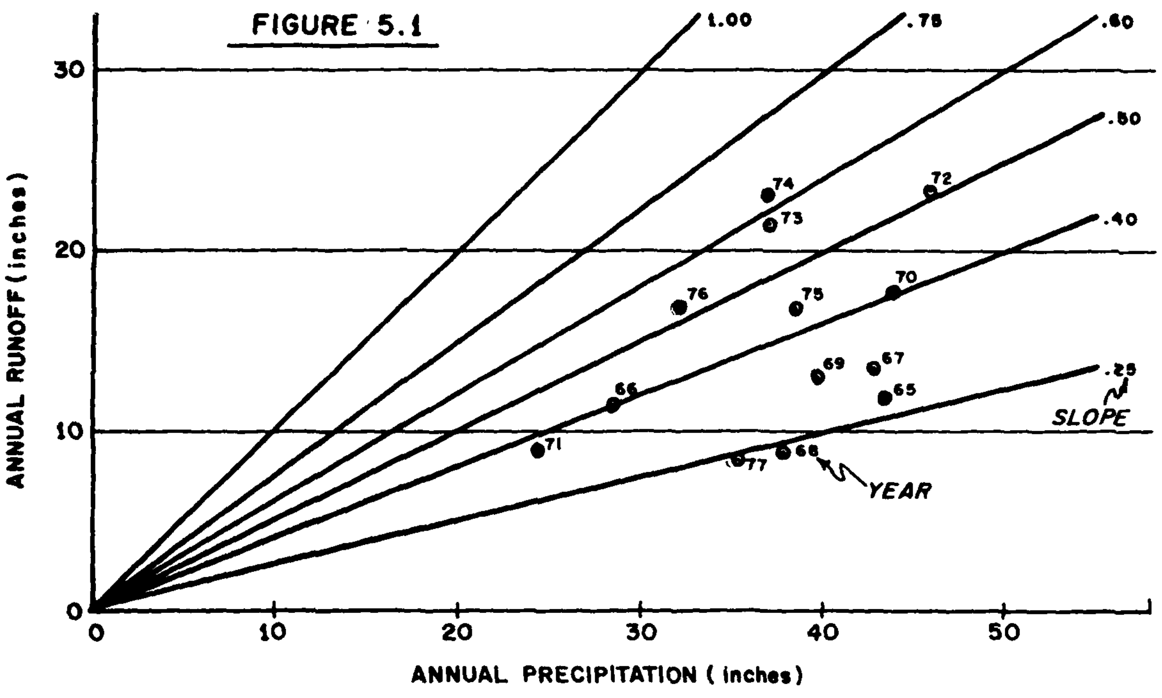


FIGURE 5.1
1.00
.75
.60
.50
.40
.25
SLOPE
YEAR
ANNUAL RUNOFF (inches)
ANNUAL PRECIPITATION (inches)
0
10
20
30
40
50

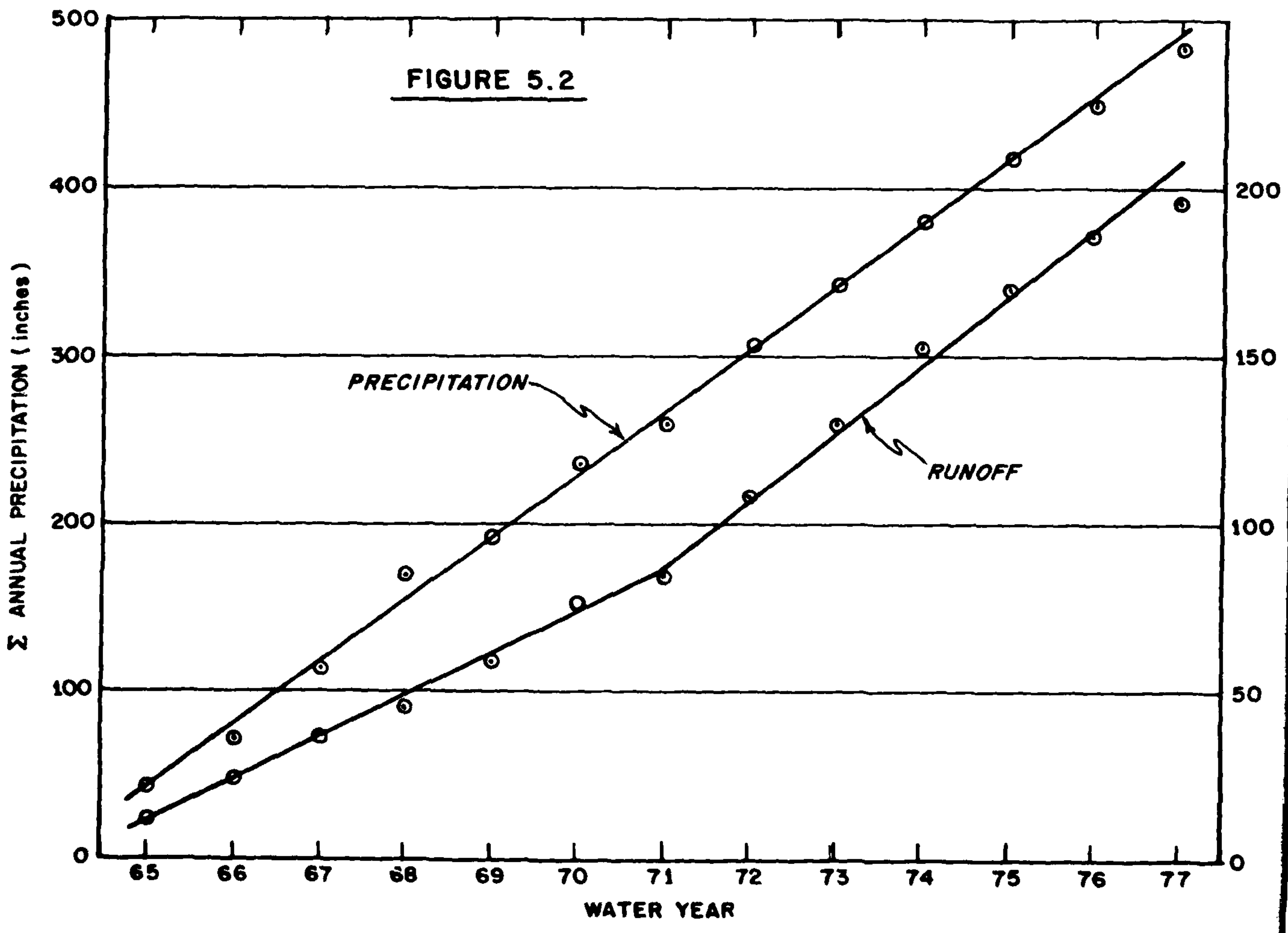


FIGURE 5.2
PRECIPITATION
RUNOFF
Σ ANNUAL PRECIPITATION (inches)
WATER YEAR
0
100
200
300
400
500
50
150
65
66
67
68
69
70
71
72
73
74
75
76
77

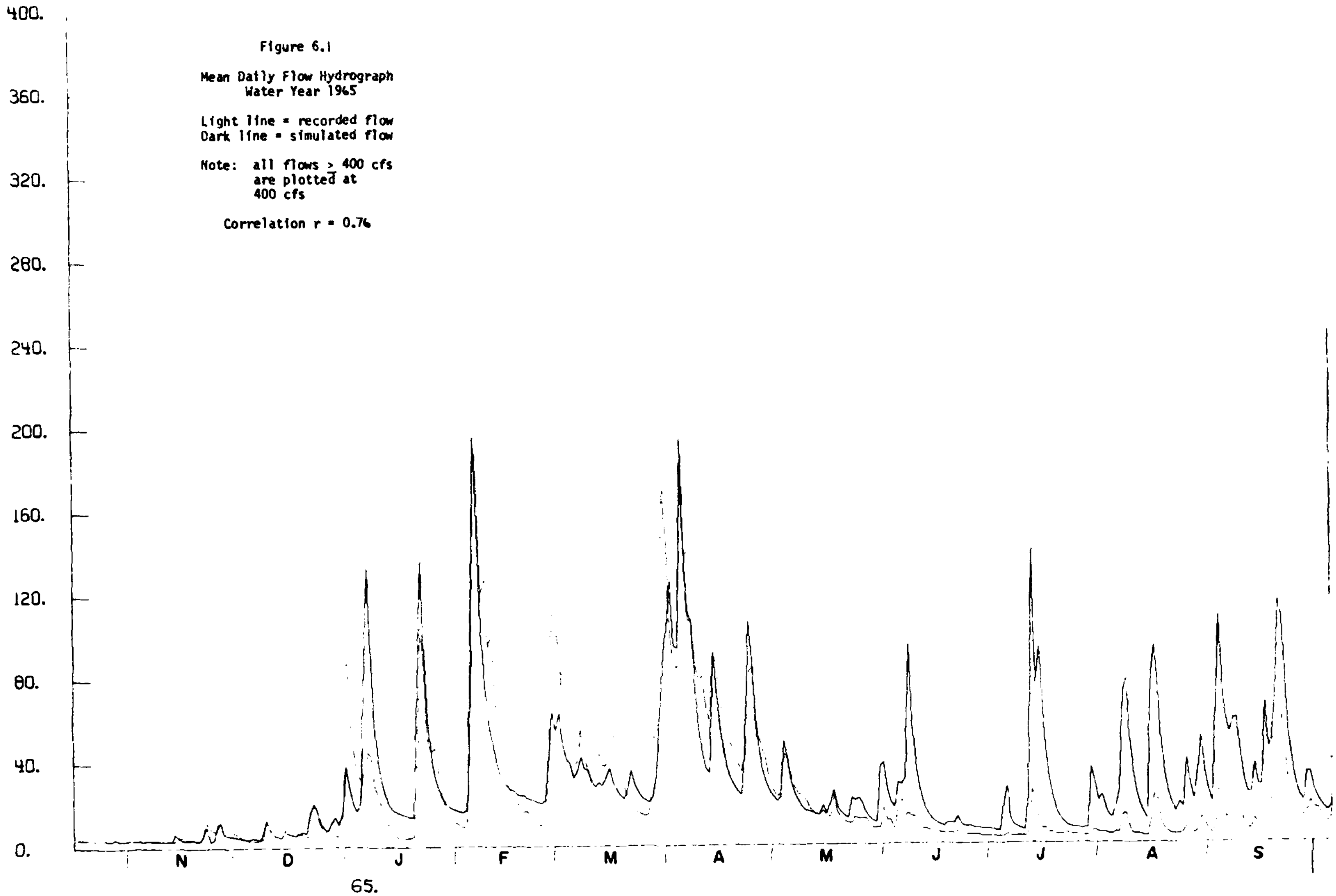


Figure 6.1
Mean Daily Flow Hydrograph
Water Year 1965
Light line = recorded flow
Dark line = simulated flow
Note: all flows > 400 cfs
are plotted at
400 cfs
Correlation r = 0.74
400.
360.
320.
280.
240.
200.
160.
120.
80.
40.
0.
N
D
J
F
M
A
M
J
J
A
S
65.

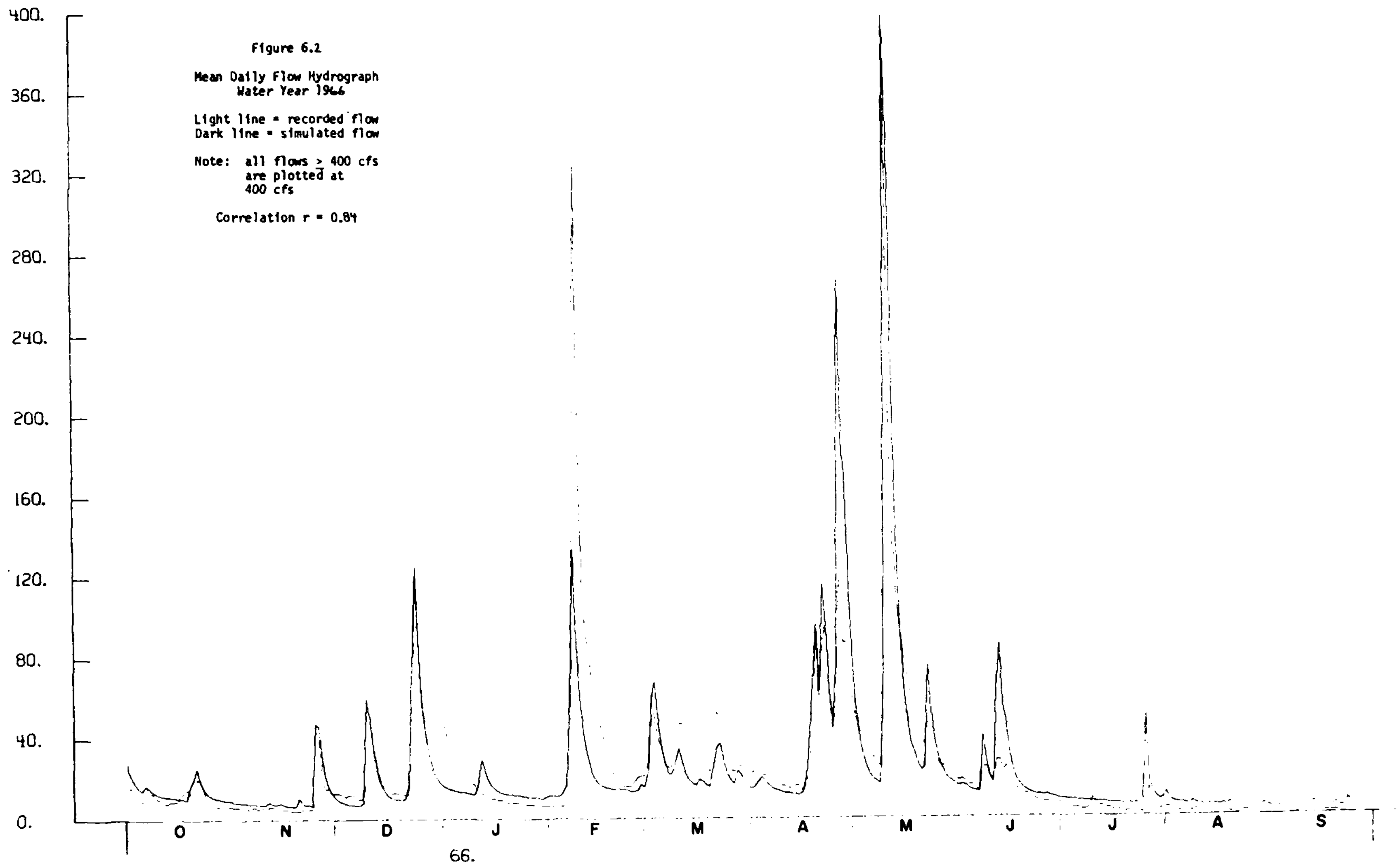


Figure 6.2

Mean Daily Flow Hydrograph
Water Year 1966

Light line = recorded flow
Dark line = simulated flow

Note: all flows > 400 cfs
are plotted at
400 cfs

Correlation r = 0.84

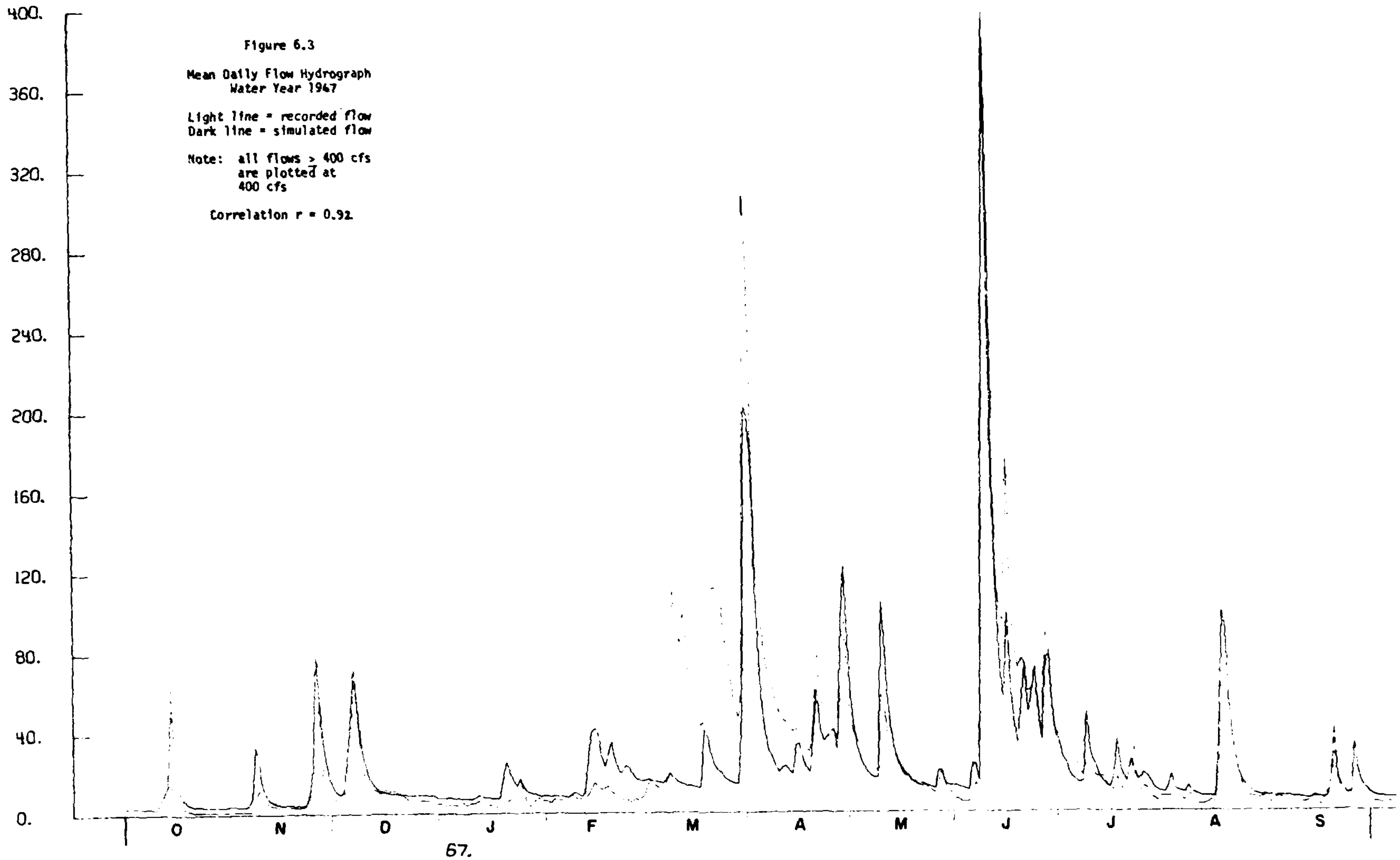


Figure 6.3

Mean Daily Flow Hydrograph
Water Year 1967

Light line = recorded flow
Dark line = simulated flow

Note: all flows > 400 cfs
are plotted at
400 cfs

Correlation r = 0.92

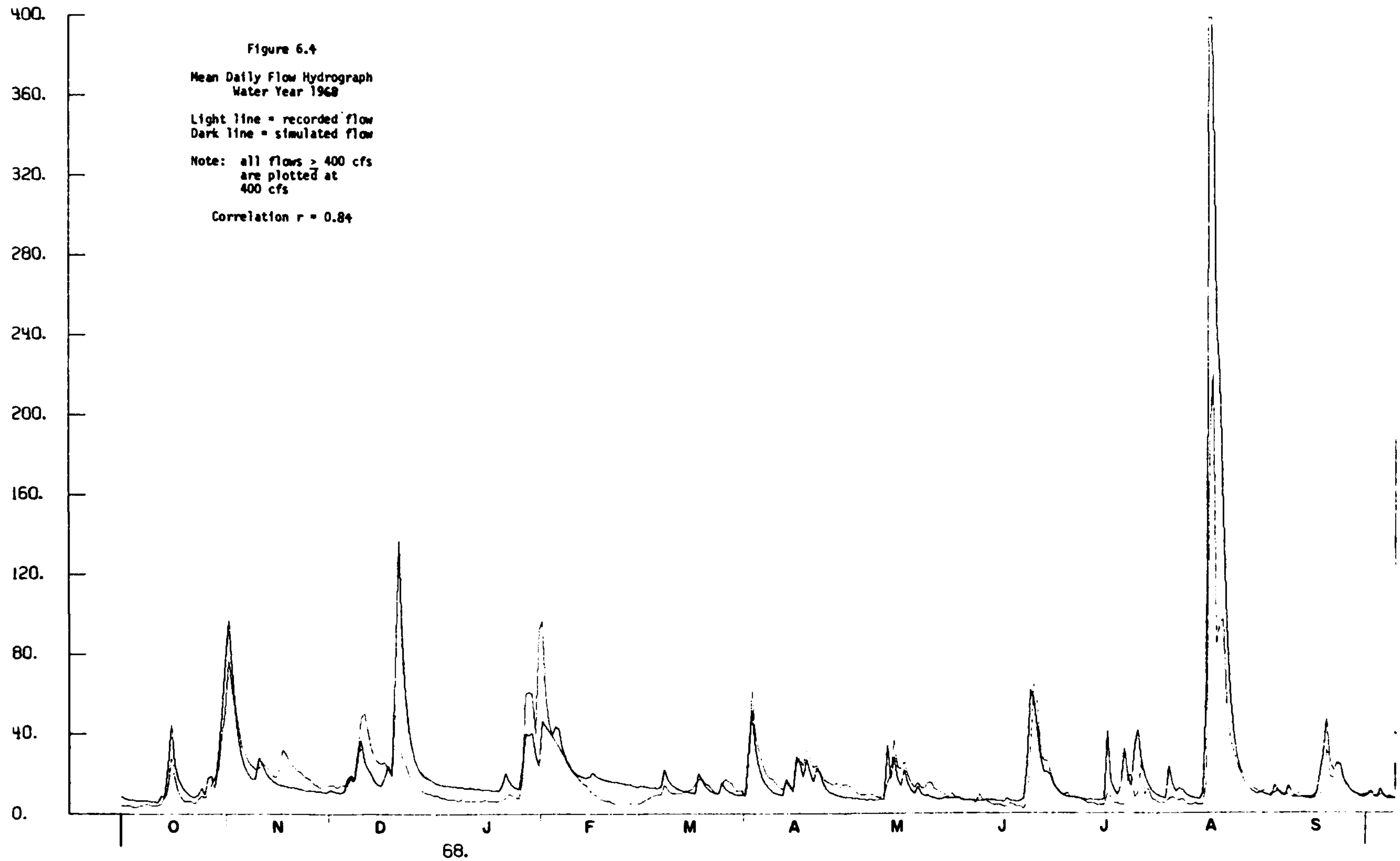


Figure 6.4

Mean Daily Flow Hydrograph
Water Year 1968

Light line = recorded flow
Dark line = simulated flow

Note: all flows > 400 cfs are plotted at 400 cfs

Correlation r = 0.84

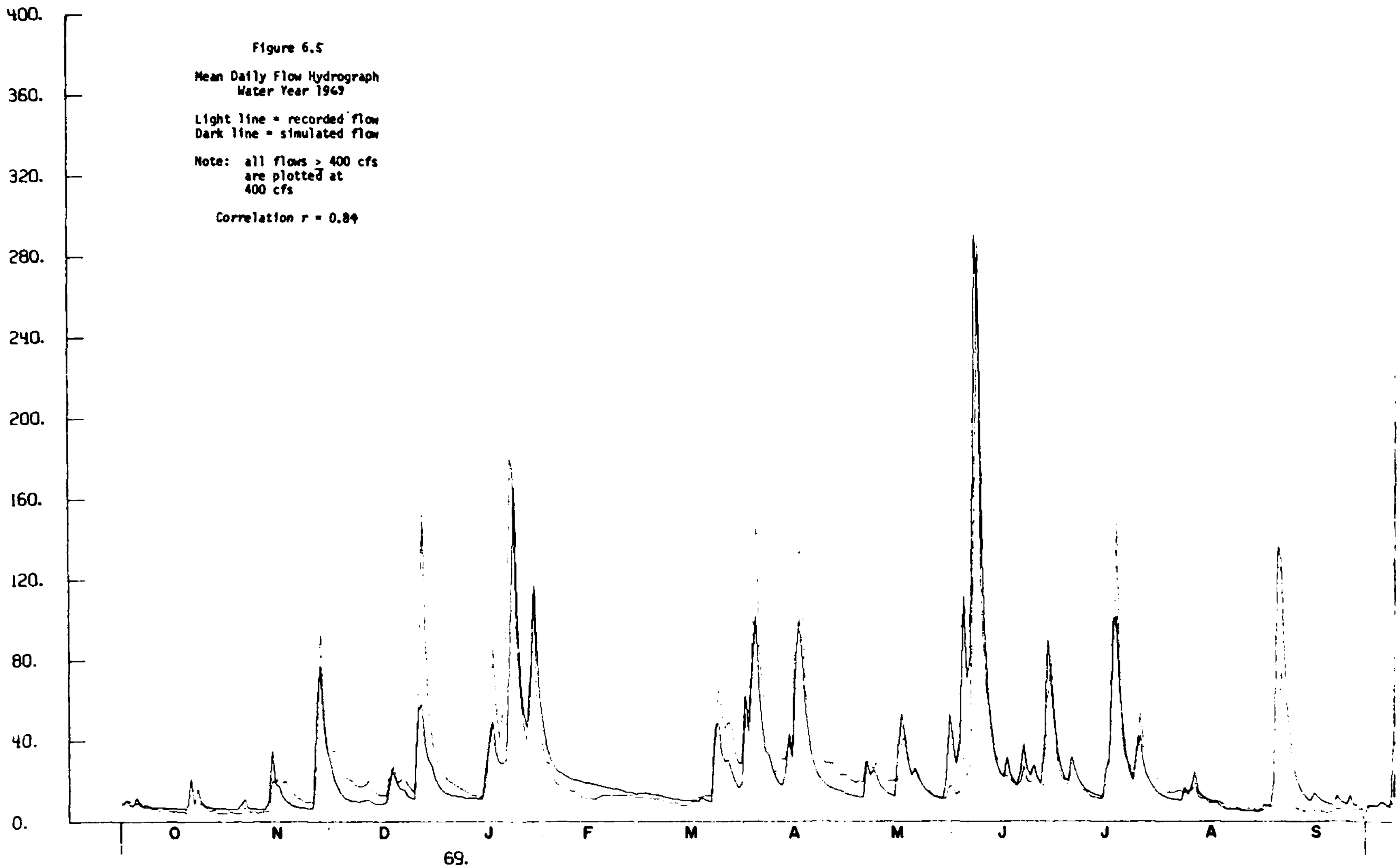


Figure 6.5

Mean Daily Flow Hydrograph
Water Year 1969

Light line = recorded flow
Dark line = simulated flow

Note: all flows > 400 cfs are plotted at 400 cfs

Correlation r = 0.84

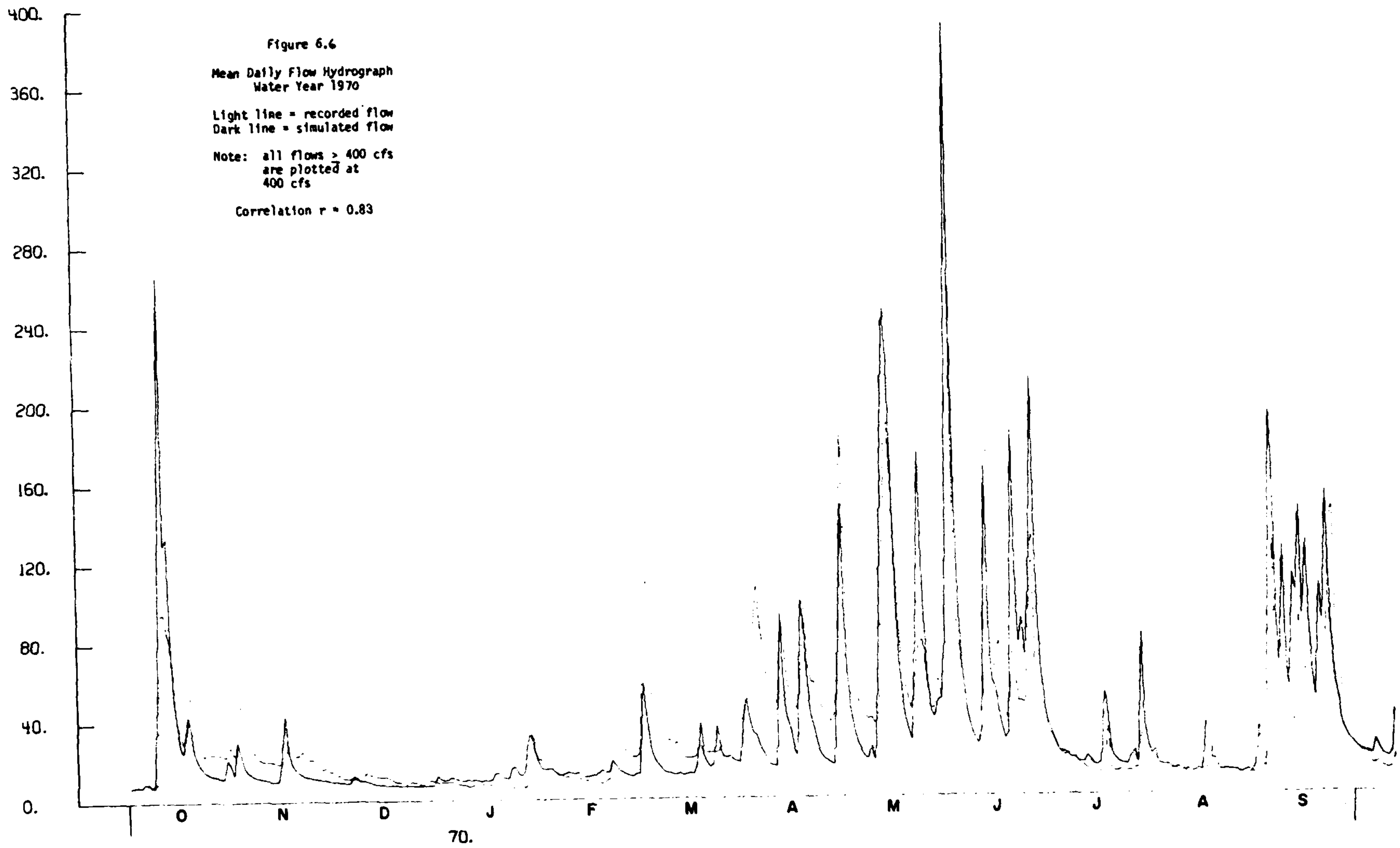


Figure 6.6

Mean Daily Flow Hydrograph
Water Year 1970

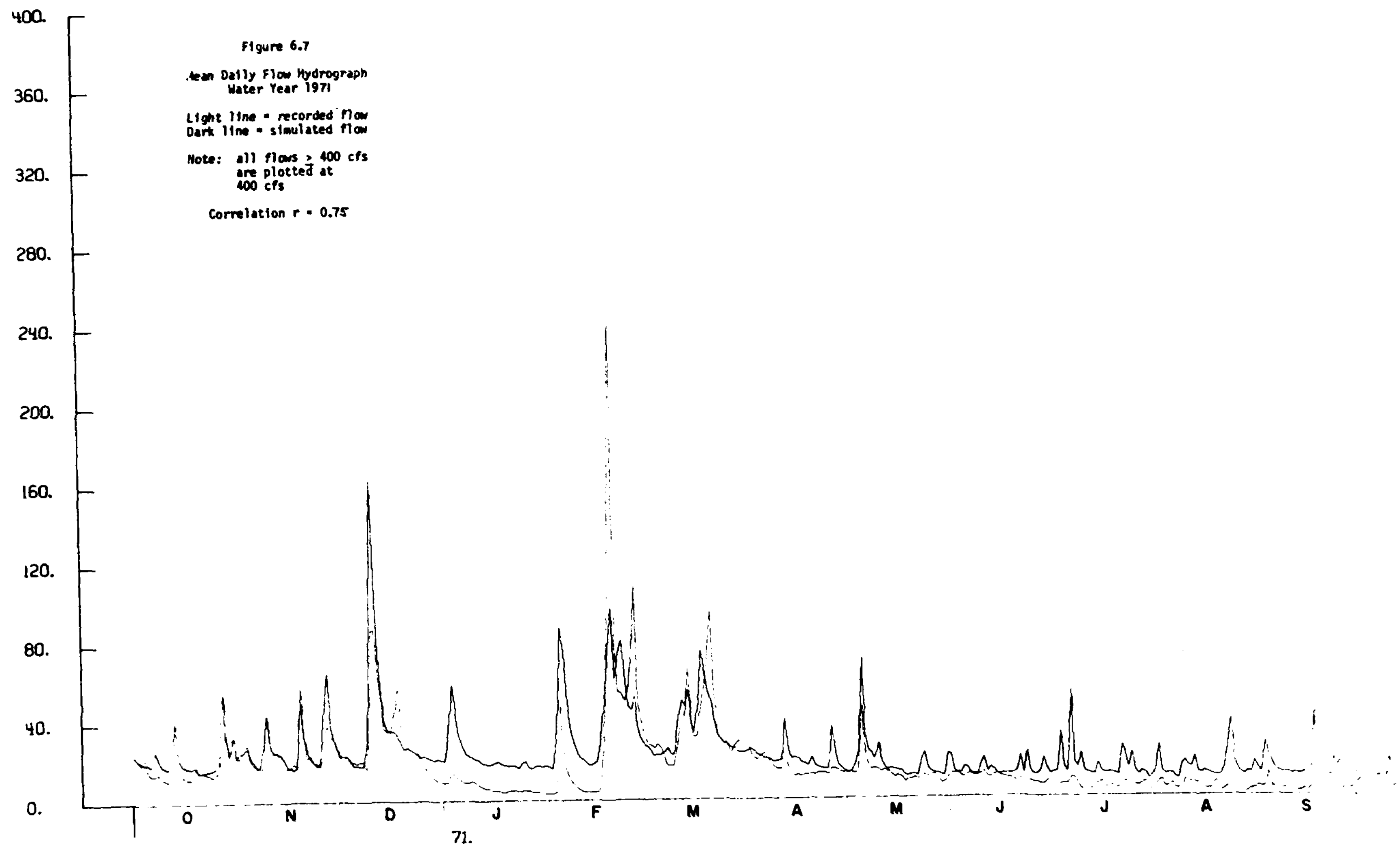


Figure 6.7
Mean Daily Flow Hydrograph
Water Year 1971
Light line = recorded flow
Dark line = simulated flow
Note: all flows ≥ 400 cfs
are plotted at
400 cfs
Correlation r = 0.75
400.
360.
320.
280.
240.
200.
160.
120.
80.
40.
0.
O
N
D
J
F
M
A
M
J
J
A
S
71.

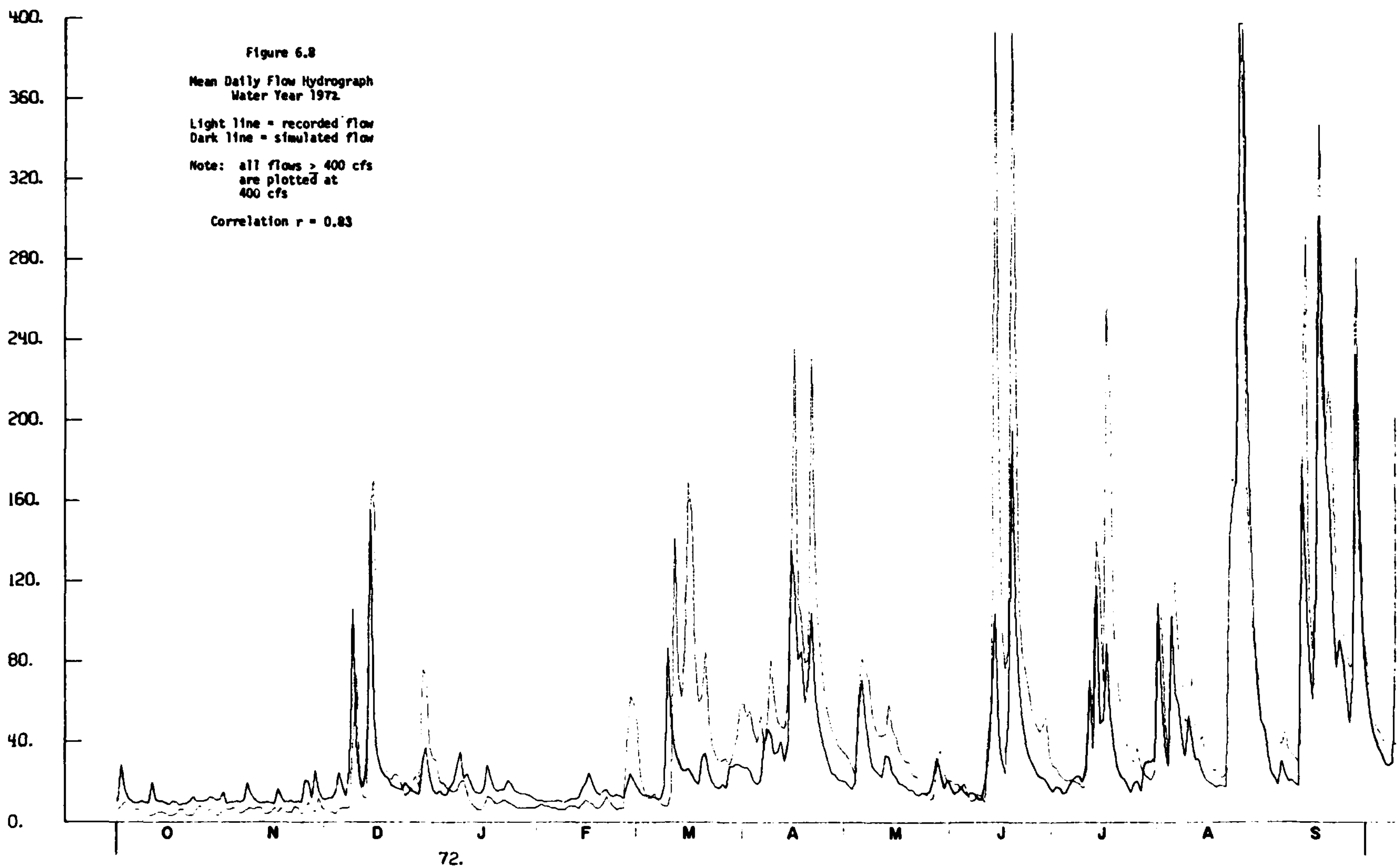


Figure 6.8
Mean Daily Flow Hydrograph
Water Year 1972
Light line = recorded flow
Dark line = simulated flow
Note: all flows > 400 cfs
are plotted at
400 cfs
Correlation r = 0.83
400.
360.
320.
280.
240.
200.
160.
120.
80.
40.
0.
O
N
D
72.
J
F
M
A
M
J
J
A
S

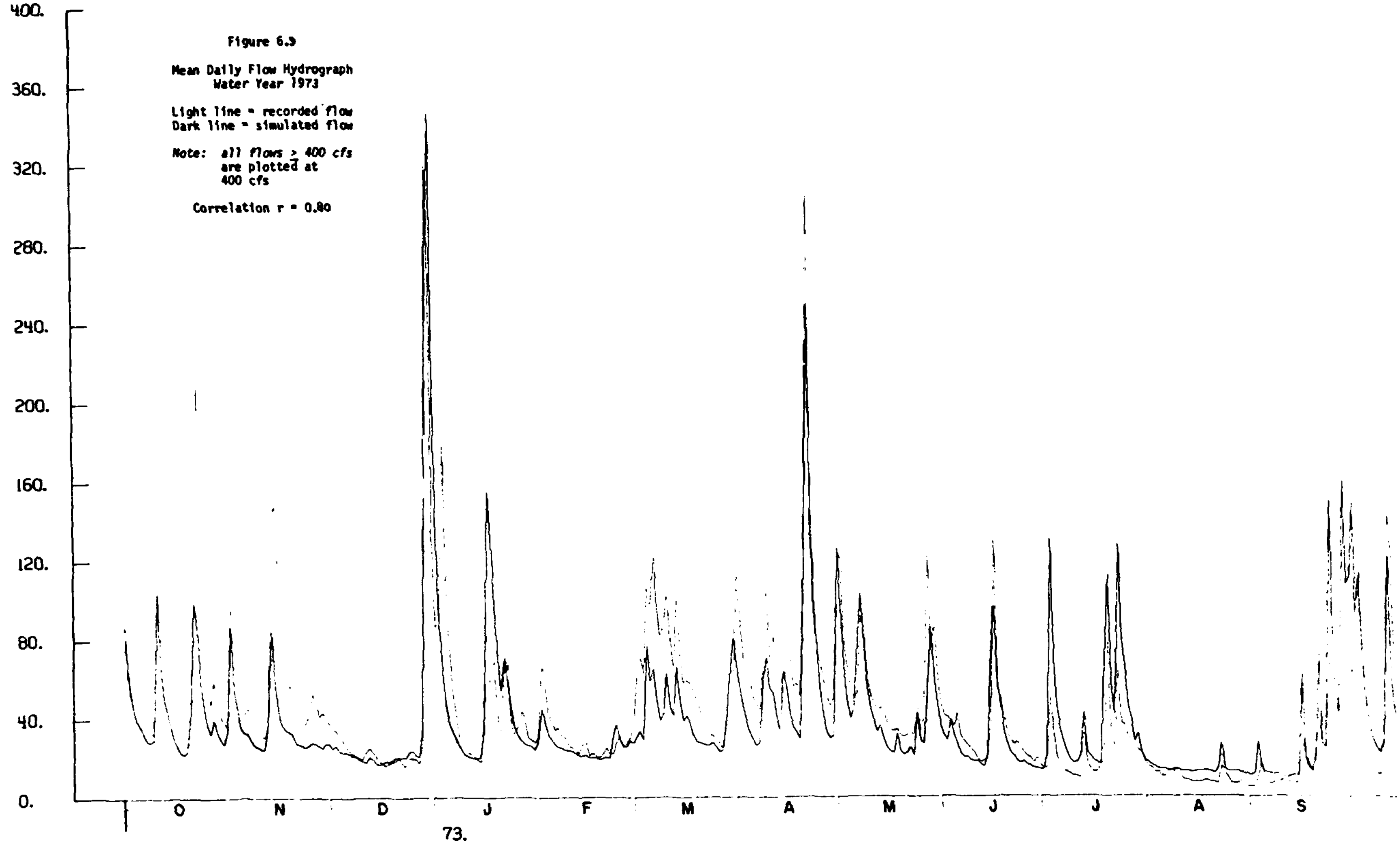


Figure 6.3

Mean Daily Flow Hydrograph
Water Year 1973

Light line = recorded flow
Dark line = simulated flow

Note: all flows ≥ 400 cfs
are plotted at
400 cfs

Correlation r = 0.80

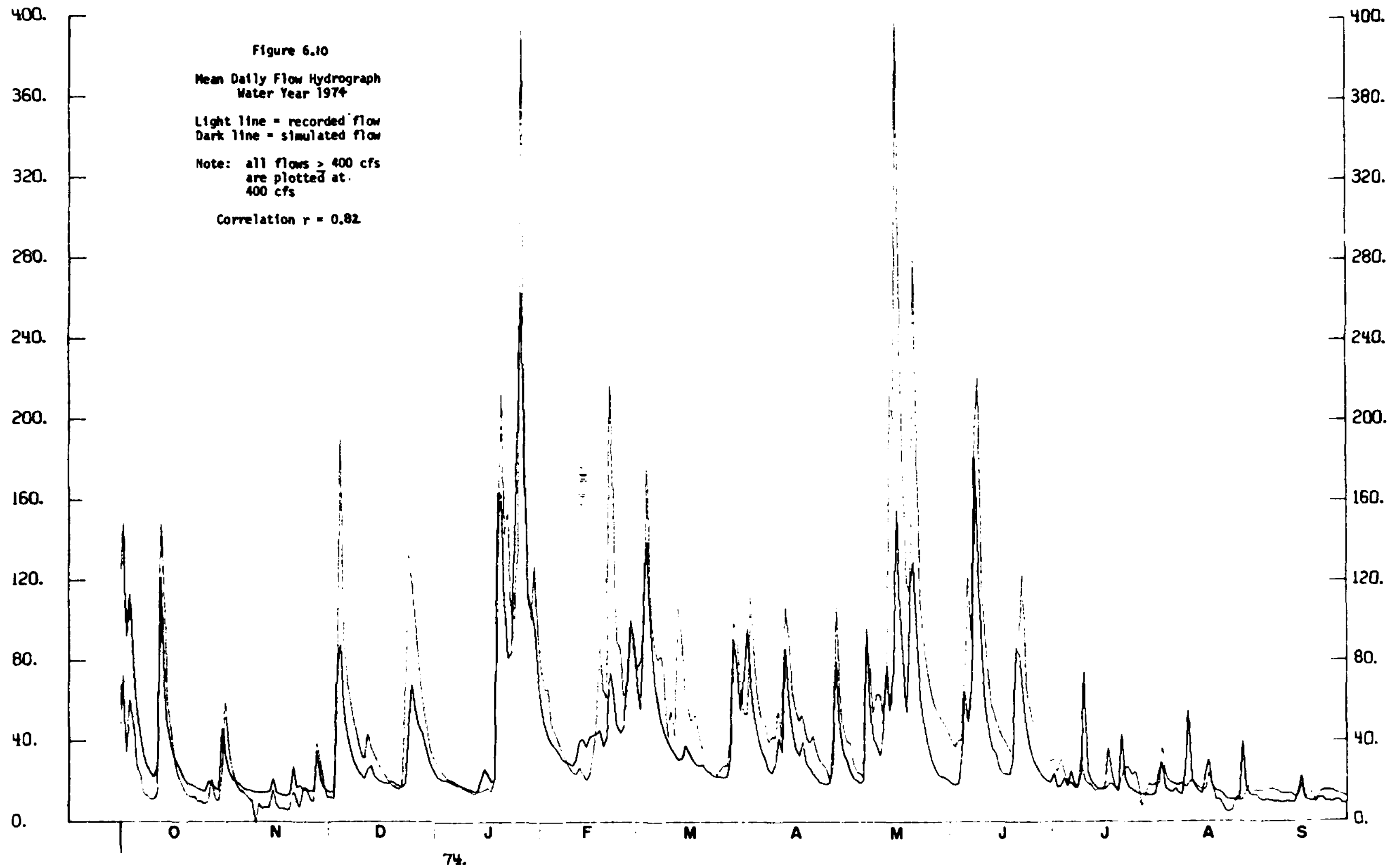


Figure 6.10

Mean Daily Flow Hydrograph
Water Year 1974

Light line = recorded flow
Dark line = simulated flow

Note: all flows > 400 cfs are plotted at 400 cfs

Correlation r = 0.82

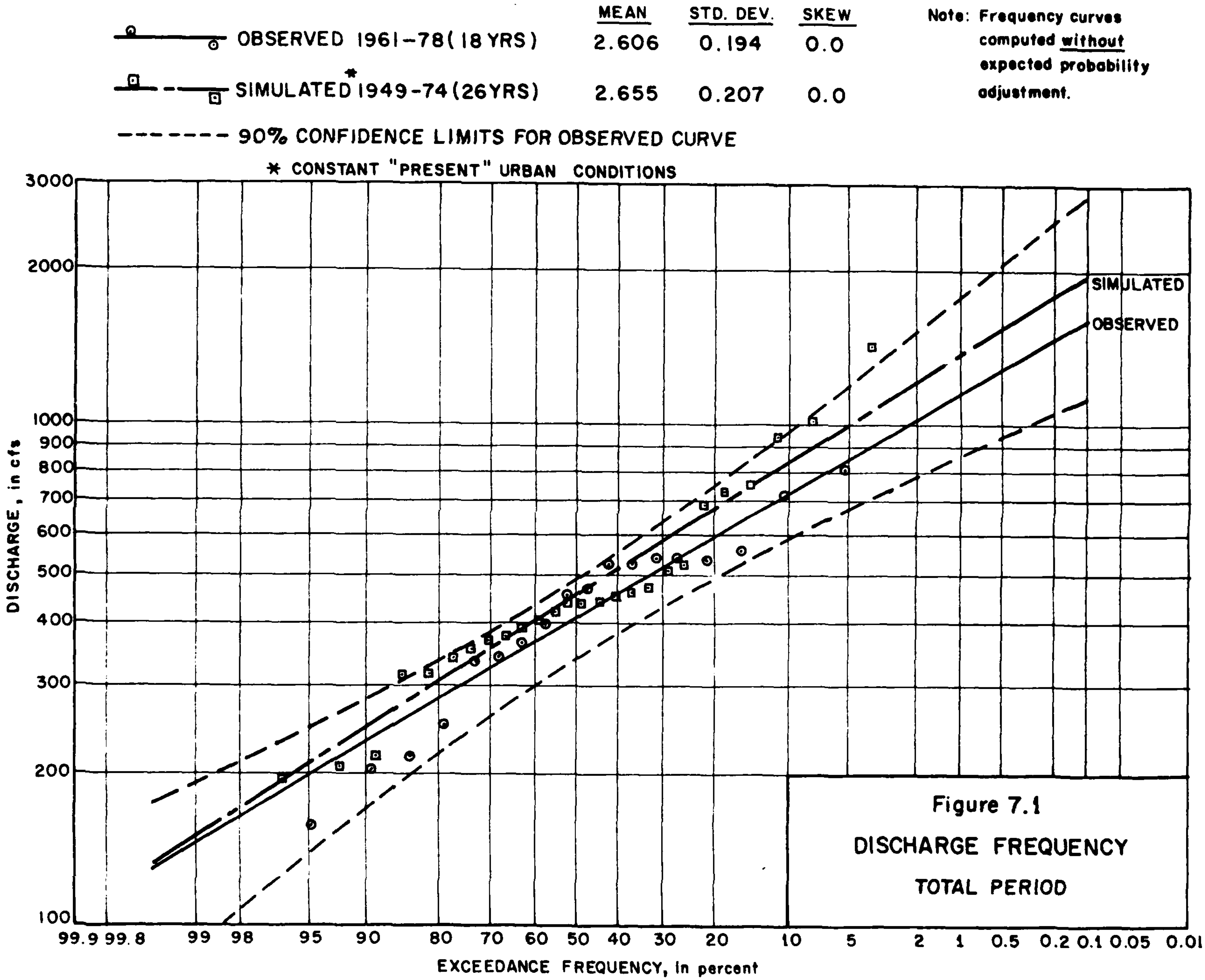


Figure 7.1
DISCHARGE FREQUENCY
TOTAL PERIOD

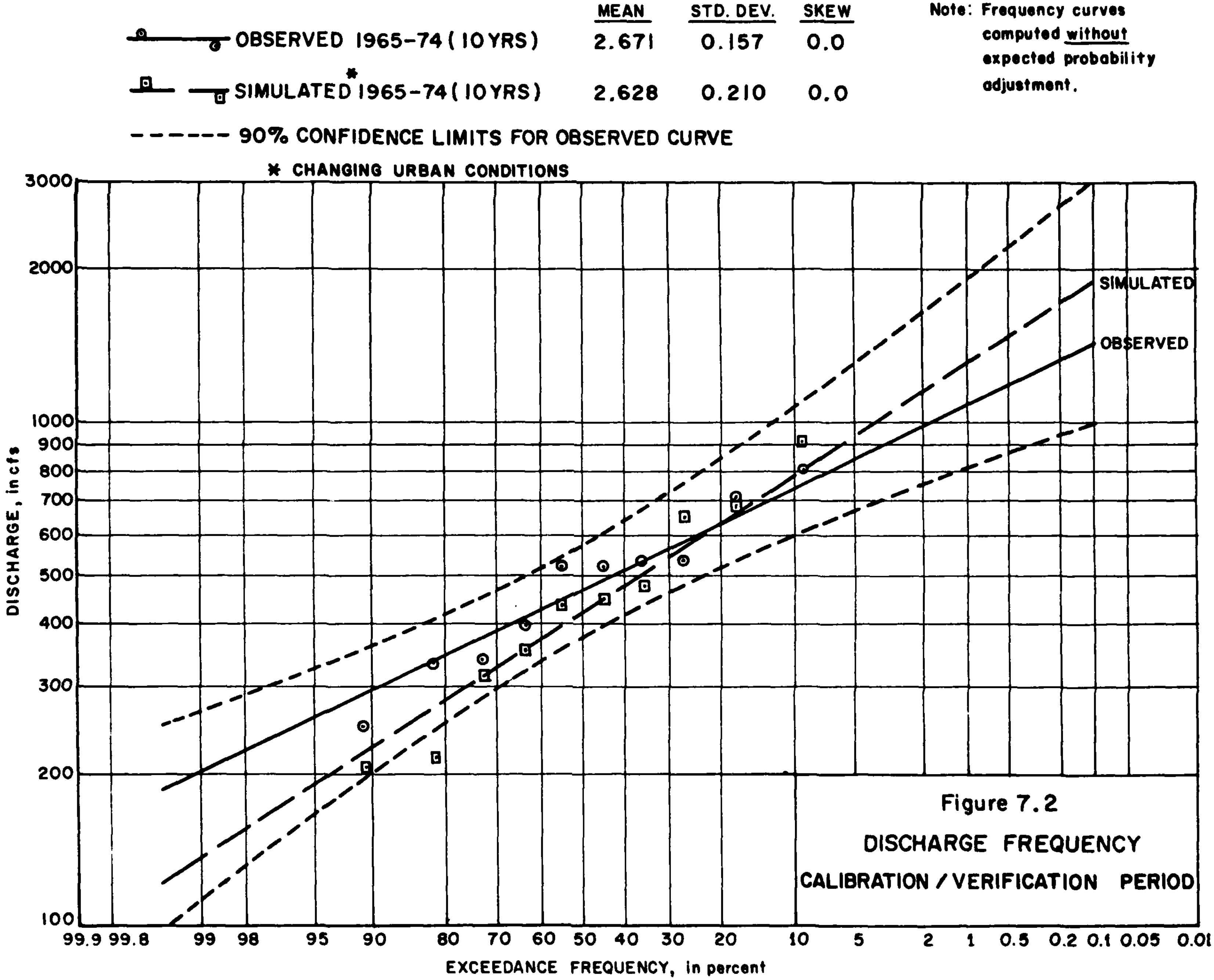


Figure 7.2
DISCHARGE FREQUENCY
CALIBRATION / VERIFICATION PERIOD

	MEAN	STD. DEV.	SKEW
—— SIMULATED* 1949-74 (26 YRS)	2.655	0.207	0.0
- —— - SIMULATED* 1961-74 (14 YRS)	2.639	0.205	0.0
- - - - - - OBSERVED 1961-74 (14 YRS)	2.611	0.182	0.0

Note: Frequency curves computed without expected probability adjustment.

* CONSTANT "PRESENT" URBAN CONDITIONS

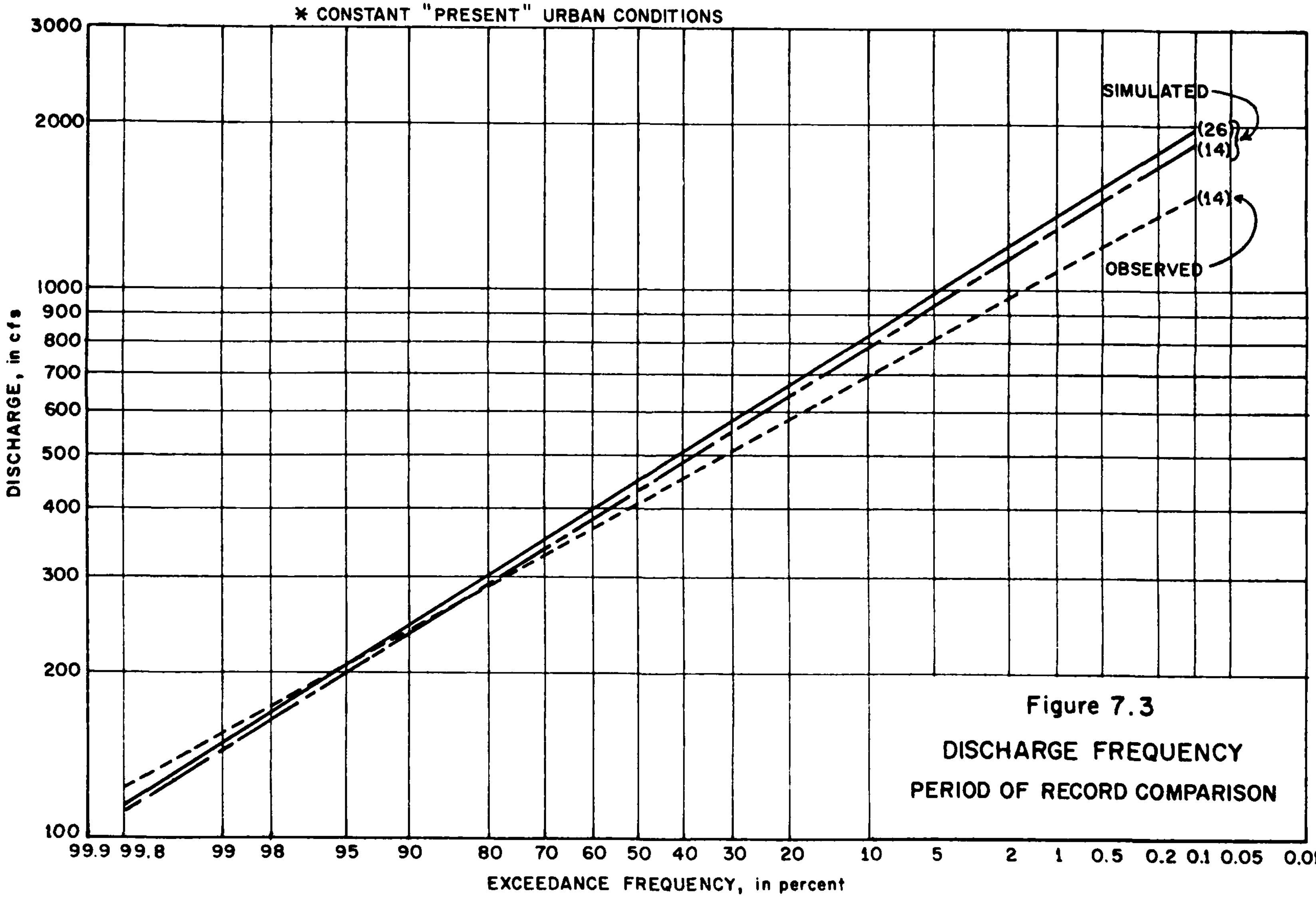


Figure 7.3

DISCHARGE FREQUENCY

PERIOD OF RECORD COMPARISON

Appendix A

Hypothesis Testing

Refer to Figure 7.1

$H: \mu_1 = \mu_2$ (assume $\sigma_1^2 = \sigma_2^2$)

Observed	Simulated
$\bar{X}_1 = 2.6056$	$\bar{X}_2 = 2.6553$
$S_1 = 0.1935$	$S_2 = 0.2067$
$n_1 = 18$	$n_2 = 26$

$$t = \frac{(\bar{X}_1 - \bar{X}_2)}{S_{\bar{x}_1 - \bar{x}_2}} \qquad \text{where} \qquad S^2_{\bar{x}_1 - \bar{x}_2} = \frac{S^2}{n_1} + \frac{S^2}{n_2}$$

and $$S^2 = [(n_1 - 1)\, S_1^2 + (n_2 - 1)\, S_2^2]/(n_1 + n_2 - 2)$$

$$S^2 = [(18-1)(0.1935)^2 + (26-1)(0.2067)^2]/(18+26-2) = 0.0406$$

$$S_{\bar{x}_1 - \bar{x}_2} = \left[\frac{(0.0406)}{18} + \frac{(0.0406)}{26}\right]^{1/2} = 0.0618$$

$$t = \frac{(2.6056 - 2.6553)}{(0.0618)} = -0.8046$$

for $\alpha = 0.05$, $t_{(1-\alpha/2)(n_1+n_2-2)} = t_{(.975)(42)} = 2.018$

$-2.018 < -0.8046 < 2.018$ therefore, do not reject H

Refer to Figure 7.1 (Cont.)

for $\alpha = 0.40$, $t_{(.80)(42)} = 0.851$ Still cannot reject H

would require $\alpha = 0.50$, $t_{(.75)(42)} = 0.681$ to reject H

H: $\sigma_1^2 = \sigma_2^2$

$$F = \frac{s_1^2}{s_2^2} = \frac{(0.1935)^2}{(0.2067)^2} = \frac{0.0374}{0.0427} = 0.8759$$

for $\alpha = 0.05$, $F_{(\alpha/2)(n_1-1,\ n_2-1)} = F_{(.025)(17,25)} = 0.370$

$F_{(1-\alpha/2)(n_1-1,n_2-1)} = F_{(.975)(17,25)} = 2.44$

Note: nearest df in table are (15,24)

$0.370 < 0.8759 < 2.44$ therefore, do not reject H

for $\alpha = 0.50$, $F_{(.25)(17,25)} = 0.712$

$F_{(.75)(17,25)} = 1.35$

$0.712 < 0.8759 < 1.35$ still cannot reject H

Refer to Figure 7.2

$H: \mu_1 = \mu_2$ (assume $\alpha_1^2 \neq \alpha_2^2$)

Observed

$\bar{X}_1 = 2.6706$

$S_1 = 0.1566$

$n_1 = 10$

Simulated

$\bar{X}_2 = 2.6276$

$S_2 = 0.2101$

$n_2 = 10$

$$t' = \frac{\bar{X}_1 - \bar{X}_2}{\left(\frac{S_1^2}{n_1} + \frac{S_2^2}{n_2}\right)^{1/2}}$$

reject if $t' \leq -(w_1t_1 + w_2t_2)/(w_1 + w_2)$

or $t' \geq (w_1t_1 + w_2t_2)/(w_1 + w_2)$

where $w_1 = \frac{S_1^2}{n_1}$ $t_1 = t_{(1-\alpha/2)(n_1-1)}$

$w_2 = \frac{S_2^2}{n_2}$ $t_2 = t_{(1-\alpha/2)(n_2-1)}$

Sample	$\bar{X}$	S^2	n	$w = \frac{S^2}{n}$	for α = 0.05 t
1	2.6706	0.0245	10	0.0025	2.262
2	2.6276	0.0441	10	0.0044	2.262

Refer to Figure 7.2 (cont)

$$t' = \frac{(2.6706 - 2.6276)}{\left[\dfrac{(0.1566)^2 + (0.2101)^2}{10}\right]^{1/2}} = 0.5189$$

$$\frac{(w_1 t_1 + w_2 t_2)}{(w_1 + w_2)} = \frac{(0.0025)(2.262) + (0.0044)(2.262)}{(0.0025 + 0.0044)} = 2.262$$

$-2.262 < 0.5189 < 2.262$ therefore, do not reject H

for $\alpha = 0.50$, $t_{(1-\alpha/2)(9)} = 0.703$

$-0.703 < 0.5189 < 0.703$ still cannot reject H

Test same hypothesis $(H: \mu_1 = \mu_2)$, now assuming $\sigma_1^2 = \sigma_2^2$

$$t = \frac{(\bar{x}_1 - \bar{x}_2)}{S_{\bar{x}_1 - \bar{x}_2}} \quad \text{where } S^2_{\bar{x}_1 - \bar{x}_2} = \frac{S^2}{n_1} + \frac{S^2}{n_2}$$

and $S^2 = [(n_1 - 1)\, S_1^2 + (n_2 - 1)\, S_2^2] \,/\, (n_1 + n_2 - 2)$

but since $n_1 = n_2$, the above simplify to

$$S^2_{\bar{x}_1 - \bar{x}_2} = \frac{(S_1^2 + S_2^2)}{n}$$

$$S^2_{\bar{x}_1 - \bar{x}_2} = \frac{[(0.1566)^2 + (0.2101)^2]}{10} = 0.0069$$

Refer to Figure 7.2 (cont.)

$$t = \frac{(2.6706 - 2.6276)}{(0.0069)^{1/2}} = 0.5189$$

for $\alpha = 0.05$, $t_{(1-\alpha/2)(n_1+n_2-2)} = 2.101$

$-2.101 < 0.5189 < 2.101$ therefore, do not reject H

even with $\alpha = 0.50$, still cannot reject H

Refer to Figure 7.2

$H: \sigma_1^2 = \sigma_2^2$

$$F = \frac{S_1^2}{S_2^2} = \frac{(0.1566)^2}{(0.2101)^2} = \frac{0.0245}{0.0441} = 0.5556$$

for $\alpha = 0.05$, $F_{(\alpha/2)(n_1-1,\ n_2-1)} = F_{(.025)(9,9)} = 0.248$

$F_{(1-\alpha/2)(n_1-1,\ n_2-1)} = F_{(.975)(9,9)} = 4.03$

$0.248 < 0.5556 < 4.03$ therefore, do not reject H

for $\alpha = 0.20$, $0.410 < 0.5556 < 2.44$ still cannot reject H

Refer to Figure 7.3

H: $\mu_1 = \mu_2$ (assume $\sigma_1^2 = \sigma_2^2$)

Simulated (26)	Simulated (14)
$\bar{X}_1 = 2.6553$	$\bar{X}_2 = 2.6387$
$S_1 = 0.2067$	$S_2 = 0.2050$
$n_1 = 26$	$n_2 = 14$

$$t = \frac{(\bar{X}_1 - \bar{X}_2)}{S_{\bar{X}_1 - \bar{X}_2}}$$ where $S_{\bar{X}_1 - \bar{X}_2}$ defined as before

$$S^2 = [(25)(0.2067)^2 + (13)(0.2050)^2] / (38) = 0.0425$$

$$S_{\bar{X}_1 - \bar{X}_2} = \left(\frac{0.0425}{26} + \frac{0.0425}{14}\right)^{1/2} = 0.0683$$

$$t = \frac{(2.6553 - 2.6387)}{0.0683} = 0.2429$$

for $\alpha = 0.05$, $t_{(1-\alpha/2)(n_1+n_2-2)} = t_{(.975)(38)} = 2.025$

$-2.025 < 0.2429 < 2.025$ therefore, do not reject H

for $\alpha = 0.50$, $t_{(.75)(38)} = 0.681$

$-0.681 < 0.2429 < 0.681$ still cannot reject H

Refer to Figure 7.3

$H: \sigma_1^2 = \sigma_2^2$

$$F = \frac{S_1^2}{S_2^2} = \frac{(0.2067)^2}{(0.2050)^2} = 1.0167$$

for $\alpha = 0.05$,

$$F_{(\alpha/2)(n_1-1,\ n_2-1)} = F_{(.025)(25,13)} = 0.331$$

$$F_{(1-\alpha/2)(n_1-1,\ n_2-1)} = F_{(.975)(25,13)} = 2.54$$

Note: nearest df in tables are (24, 12)

$0.331 < 1.0167 < 2.54$ therefore, do not reject H

for $\alpha = 0.50$

$$F_{(.25)(25,13)} = 0.684$$

$$F_{(.75)(25,13)} = 1.36$$

$0.684 < 1.0167 < 1.36$ still cannot reject H

DAT
ILM