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HYDROLOGIC ASPECTS OF ON-SITE RETENTION SYSTEMS FOR URBAN STORM RUNOFF

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By

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ABSTRACT

Current land planning and engineering practice is placing increasing emphasis on the use of "on-site retention" facilities as a solution to several problems related to the development of lands for urban/residential use in south Florida. As a regulatory agency acting under Chapter 373, Florida Statutes, and as an advisory agency to the regional planning agencies which administer Chapter 380, Florida Statutes, the District must necessarily be in a position to intelligently evaluate those surface water management plans and systems in which on-site retention facilities are incorporated. This investigation is undertaken to examine certain of the hydrologic aspects of such systems; specifically the changes in hydrology due to urbanization, the performance of on-site retention facilities in regard to flood protection, the role of retention basin operation in maintaining designed flood protection performance, and the possibility of operating such systems to conserve runoff as well as to provide flood protection.

In this study two models are developed and described. One is a hydrologic accounting simulation model which is used to generate monthly runoff values. The other is a linear decision rule model which is used to determine optimum retention basin size and develop an operating rule or policy for the retention basin selected. The hydrologic accounting model is applied to the Canal 11 basin tributary to Pumping Station 9 in western Broward County and is validated and calibrated against observed period of record discharges at S-9. The model is then used to simulate discharges from the same watershed under a number of assumed conditions of urban/residential development.

The linear decision model is then applied to the problem of retention basin sizing, using the runoff values generated by the hydrologic accounting

simulation model assuming 100% development of the basin (50% impervious cover). An operating rule for the optimum size reservoir is developed. A comparison is made with a retention basin sized in accordance with meeting storage requirements for the design storm having a 5-day duration at a frequency of once-in-100-years.

Constraints imposed in the linear decision model include maintenance of minimum outflows, maximum permissible outflow, maintenance of minimum retention basin stage, and reliability of performance. Retention basin performance in meeting the minimum stage constraint (i.e., long-term average retention of additional water on-site) is evaluated.

The two models used in conjuction represent a methodology which it is suggested can be used to design an on-site retention water management system and develop an operating policy for the retention facilities. Such facilities can be designed and operated to provide both flood protection and long-term on-site water retention.

INTRODUCTION

In recent years in south Florida planners and engineers have increasingly directed their attentions to the design of water control systems for urban/residential developments which will provide a high degree of "on-site water retention." A major impetus for this rather new approach to the design of water control systems in the south Florida area was provided by a number of studies elsewhere in the country which demonstrated and quantified the pollutional load carried by urban storm water runoff into surface waterways. On-site detention of at least the early portion of such runoff was shown to reduce total pollutional load and the impact of early runoff on the receiving stream in such parameters as BOD and suspended solids. Many local jurisdictions and regulatory bodies accordingly adopted standards and criteria for early storm runoff detention.

Further impetus was provided by the fact that in many portions of south Florida, limited outlet capacity is available in surface watercourses to safely accommodate the peak runoff rates generated as a result of urban/residential development. Large portions of western Dade, Broward and Palm Beach Counties are examples of areas in which development pressures are being experienced; but which are, in conventional terms, inadequately served by primary outlet channels. This situation when considered as a planning and engineering problem requires an extension of the approach which satisfies the requirements of detention only of early storm runoff since limitation of outlet capacity usually will mean that more on-site retention capability, and for longer durations, must be provided.

This problem, of course, is one of providing an adequate degree of flood protection for public facilities (such as roads and streets) and private

properties (such as homes and their contents). In south Florida, with the severe constraints imposed by flat topography and high groundwater tables, this flood protection in the past 25 to 30 years has been obtained through a combination of topographical alteration by filling and provision of positive outlet systems having, in some cases, the ability to remove storm runoff at rates up to 4 inches in 24 hours. With a limitation on runoff removal capability, however, the flood protection problem must be solved by providing for greatly increased on-site retention of rainfall excess while still retaining the topographical alteration feature of the earlier conventional solution. The problem solution, therefore, takes on the character of solving, from an engineering standpoint, a problem in reservoir design. In recent years engineers and land planners have approached this problem by providing retention ponds or lakes to accommodate storm runoff resulting from the more frequently occurring rainfall events and open space areas (such as golf courses) on which rainfall excess resulting from more severe events can be temporarily stored.

The solution to the flood protection problem in these cases has taken on an added dimension in the past year as a consequence of the Federal Flood Insurance program. In those communities and local jurisdictions which have accepted the program, the one-in-one-hundred year frequency storm event flood stage becomes the criterion for establishing floor elevations for conventionally designed buildings. The combination of fill at the building site and retention (soil storage plus retention ponds plus "floodable" open space) must be sufficient to keep the one-in-one-hundred year flood stage below the first floor elevation of homesites.

Finally, some additional impetus has been given to the "on-site retention" approach to the design of local water control systems by the need and desire to

recover and conserve fresh water runoff in lieu of discharging seasonal rainfall excess to tidewater. It has been claimed that systems designed with a high degree of on-site retention capability for wet season rainfall excess will also provide the capability to retain on-site water which otherwise, with a more conventional system design, would be discharged off-site to tidewater. Intuitively, this appears to be a reasonable claim of additional benefit to be derived from a water control system whose design, in the first instance, is predicated on the primary consideration of providing a high degree of flood protection under the constraint of limited positive outfall capacity.

The purpose of this investigation is to examine in some detail two of the three considerations which have prompted the current emphasis on the planning and design of "on-site retention" water control systems for urban/residential developments; namely, flood protection and water conservation. The water quality aspects of such systems are not discussed in this study. While only the hydrologic aspects of on-site retention are examined, this analysis may provide some insight, and a starting point, for future investigation of the water quality control performance of a properly designed and operated on-site retention water control system.

Specificially, this study deals with the following questions:

1. How, and to what extent, is the hydrology of an area altered due to urbanization?
2. By what means can an on-site retention system be designed which will provide the required degree of flood protection under a hydrologic regime altered due to urbanization?
3. Is there a rational method for devising an operating rule for an on-site retention system which will preserve the designed flood protection performance?

4. To what extent, if at all, will such a system, designed and operated for flood protection, provide for recovery and conservation of storm runoff?

These questions derive from a basic recognition that some alteration to the hydrology must accompany development, that provision of flood protection for urban developments is a primary consideration, and that maintenance of flood protection and conservation of storm runoff may be mutually exclusive objectives. This study is an attempt to place all of these considerations into the same analytical framework and to quantify, by example, certain of the basic values. By doing so it may be possible to suggest a general methodology for use in solving the planning and design problem indicated earlier in this section, and in evaluating the adequacy of urban/residential development plans.

Finally, this investigation is undertaken in an attempt to develop a rational basis for considering and evaluating present practices and approaches being applied to the solution of current problems in water control system design. There is no intent, stated or implied, to address and reach conclusions on the social-economic, land use, or "quality of life" considerations which are at least equally as important as the technical considerations in the evaluation of land development plans.

PROCEDURE

The procedure used in conducting this investigation followed the sequence given below:

1. Development of a hydrologic accounting simulation model which maintains a running account, based on a monthly time frame, of soil moisture through application of appropriate values for rainfall, evapotranspiration, direct surface runoff, and accretion to groundwater.
2. Development of a linear decision rule model for formulation of an operational policy for a retention basin taking into consideration the constraints of minimum retention basin releases for maintenance of flow, maximum permissible releases, maintenance of minimum retention basin stages, safety factor (freeboard), and reliability.
3. Development of a routing model based on the continuity equation, to evaluate the impact of the present and the proposed regulatory scheme on the hypothetically urbanized watershed basin.
4. Application of the hydrologic accounting simulation model to the 50-square mile area in western Broward County served by Pumping Station 9 in Canal 11, and verification of the simulated basin outflows for the years 1963-1973 by comparison with observed historical discharges at S-9.
5. Using the calibrated hydrologic accounting model, simulation of basin outflows (runoff) for the 1963-1973 condition assuming basin "development" at values from 10% to 100% developed. By definition, 100% development assumes 50% of the area covered by impervious surfaces.

6. Using the values derived from the hydrologic accounting model assuming 100% development with the recurrence of the 1963-1973 condition, determination of the optimum size retention basin and development of an operating rule for the retention basin.
7. Calculation of storage volume required to accommodate the design storm event (100% developed), selection of three hypothetical retention basins whose depth times area equals that required storage volume, determination through application of the linear decision model of any required adjustment in retention pond dimensions, and development of an operating rule for each size-adjusted retention pond.
8. Determination of retention pond performance in the retention on-site of some portion of the storm runoff generated by urban/residential development.
9. Routing of flows through a hypothetical development in the S-9 basin for a historical rainfall period; the flows being generated by use of the hydrologic accounting simulation model and the retention basin being sized and operated by application of the linear decision model.

THEORETICAL DEVELOPMENT

> "As the correct solution of any problem depends primarily on a true understanding of what the problem really is, and wherein lies its difficulty, one may profitably pause upon the threshold of the subject to consider first, in a more general way, its real nature; the causes which impede sound practice; the conditions on which success or failure depends; the direction in which error is most to be feared."
>
> A. M. Wellington (25)

Hydrologic Accounting Model

Better insight into the mechanics of a watershed basin, for planning and management purposes, can be gained by use of a hydrologic accounting model. The hydrologist's problem is to identify the specific inputs and outputs of a particular watershed basin under study. The processes of importance in the hydrologic accounting simulation model are, generally, the following: 1) rainfall, 2) evapotranspiration, 3) soil moisture status, 4) surface runoff, and 5) groundwater flow (see Figure 1).

According to Crawford and Linsley (23) the accounting simulation approach is an indirect approach to the study of the behavior or response of the system. Linsley and Ackerman (23) were the first to introduce this concept to hydrology, but was not extensively used before high speed computers came into use. Digital simulation of hydrologic accounting is a more recent method used to analyze large and complex systems. Essentially, a digital model simulation of the hydrologic accounting maintains a running account of water in the zone of aeration by adding each new rainfall, less direct runoff and accretion to groundwater and subtracting evapotranspiration. The amount of runoff and groundwater accretion is made a function of prevailing soil moisture storage. This is consistent with the infiltration theory in which the infiltration capacity is a function of soil moisture (22).

Though most accounting simulation models are developed for small time increments (hours) for hydrologic analysis, it has been successfully used

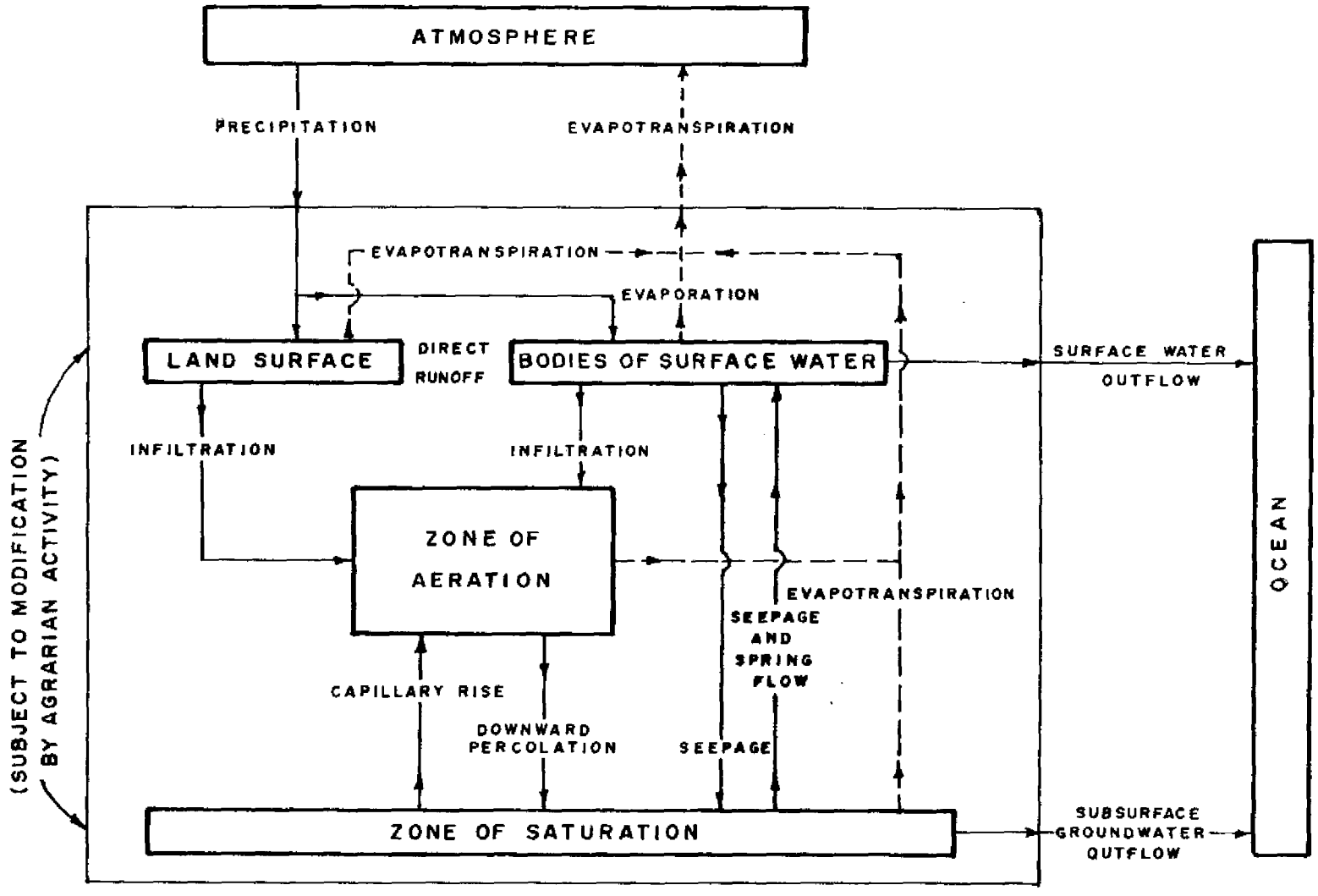

Figure I PRE-URBAN HYDROLOGIC SYSTEM

(SOURCE: AN ATLAS OF LONG ISLAND'S WATER RESOURCES, BY P. COHEN, O.L. FRANKE AND B.L. FOXWORTHY, NEW YORK WATER RESOURCES COMMISSION BULLETIN 62, 1968, PAGE 57

for periods of weeks and months. Palmer (12), and Hufschmidt (8) have used a hydrologic accounting simulation model on a monthly basis for a hydrologic drought analysis and the simulation of a large complex water resource system. Palmer used both the monthly and weekly drought analysis for comparative purposes and states that the weekly and monthly results were in agreement over 90 percent of the time. Based on Palmer's experience and in order to eliminate the processing of massive amounts of input data, the hydrologic accounting simulation model being developed here will be based on a monthly time frame.

The hydrologic accounting simulation model can be written as:

$$\frac{dSM}{dt} = d\,\frac{(P + MLg)}{dt} - d\,\frac{(Q + PC + ET + MLe)}{dt} \qquad (1)$$

The solution of this partial differential equation can be obtained by solving for the individual components over a preselected time increment, 't'.

$$SM_t = SM_{t-1} + P_t + ML_{gt} - Q_t - PC_t - ET_t - ML_{et} \qquad (2)$$

where,

SM_t, t-1 = Soil moisture at time t, t-1,

P_t = Natural precipitation,

$ML_{et,gt}$ = Minor losses or gain,

PC_t = Deep percolation,

ET_t = Evapotranspiration,

Q_t = Discharge.

Given rainfall and potential evapotranspiration values, the runoff generation process can be simulated by use of the above hydrologic accounting model. The mechanism is based on the soil moisture condition of the previous time period. Based on this, the excess precipitation is routed through the

soil horizons. The runoff component is evaluated for a two layer soil horizon as follows:

$$AE = PE \text{ (when } UZM > 0) \quad (3)$$

$$AE = \frac{LZM}{LZS} \times PE \text{ (when } UZM = 0) \quad (4)$$

$$R_{UZ} = P - AE \text{ (max. of } UZS - UZM) \quad (5)$$

$$R_{LZ} = P - R_{UZ} - PE \text{ (until } UZM = LZS) \quad (6)$$

$$R_{GW} = P - R_{UZ} - R_{LZ} - PE \text{ (const. infiltration rate; when } LZM = LZS) \quad (7)$$

$$SRO = P - R_{UZ} - R_{LZ} - PE - R_{GW} \quad (8)$$

where,

PE = potential evapotranspiration, inches

UZS = upper zone soil storage capacity, inches

LZS = lower zone soil storage capacity, inches

AE = actual evapotranspiration, inches

P = precipitation, inches

R_{UZ} = recharge to upper soil zone, inches

R_{LZ} = recharge to lower soil zone, inches

UZM = upper zone soil moisture, inches

SRO = surface runoff, inches

LZM = lower soil zone moisture, inches

R_{GW} = recharge to groundwater, inches

The following equation is used in order to evaluate base groundwater flow, assuming that the groundwater discharge varies linearly with storage:

$$S = K.Q$$

whwere,

S = storage

K = storage delay time (storage constant).

The discharge from groundwater will be from composite storage where the shallow element (lower soil zone) will be depleted first to a certain extent, after which the discharge will be solely from deep storage. The summation of the two flows (base flow + direct surface runoff) will constitute the total unrouted streamflow from the basin.

Operational Model

Operational models seek rational decision rules or policy functions to simplify decision making in reservoir operations. The final product from operational models is a set of decision rules which are easy to apply and which, when applied, meet the multiple objective criteria of low flow maintenance, flood protection, recreation, and irrigation and associated explicit statements of risk. Readers interested in the history of the development of decision rules are referred to classical works on linear decision rule formulation by Revelle (15) and Nayak (11). The operational model proposed here attempts to apply the simplest form of the linear decision rule to retention basin design and operation. In simplest algebraic form it is written as:

$$X = S - b$$

where,

X = release during a period of retention pond operation,

S = retention pond storage at the end of previous month,

b = a decision parameter chosen to optimize some criterion function.

The release rule as written above is to be interpreted as a user's operational aid in selecting releases to optimize management of their retention basins for dual purposes - water supply and flood control.

Formulations utilizing the linear decision rules lend themselves to classical linear programming problems which optimize a certain objective function subjected to certain constraints. The linear decision rule can be applied to either a) the deterministic framework where the magnitude of each input (inflow) in a sequence is specified in advance, based on historic records or b) the stochastic framework where the magnitudes of the retention basin inputs (inflows) are treated as random variables unknown in advance.

Statement of the Problem

A retention basin is to be built to provide storage of excess runoff due to urbanization for a) recharging groundwater in the basin, b) regulating outflows to the primary channels, and c) providing pools (storage) for flood control during storm events. The regulated outflow will be the minimum releases (q_i) over a specified time interval (i) for downstream beneficial uses. To prevent excessive channel erosion, or other damage that would occur if the releases were too large, the release during period (i) is constrained by the allowable maximum release based on the designed or anticipated pump capacity or gate openings, etc. It is also desirable for esthetic purposes to maintain storage in the retention basin above a minimum level, (S min.). An additional requirement imposed by flood control considerations is that a minimum freeboard (V_i) be available within the reservoir at the end of each month for storing flood water which might occur during the next time period.

The most desired optimum solution to the engineering problem then, is to find an operating policy or set of release values (X_i's) that meet all the above stated requirements while minimizing the size of the retention basin required. The problem is formulated as follows: Given n year sequence of monthly flows from a hypothetical urbanized basin by use of the

hydrologic accounting model as described previously, it is required to determine twelve linear decision rule (release) parameters, one for each month of the year, that minimize the retention pond capacity required to meet the above stated constraints. The following notations are used in the development of the model:

q_i = minimum release to be provided for the beneficial use of downstream users in the ith month of the year,

f_i = maximum allowable release (based on pump capacity, gate openings, etc.) in the ith month of the year,

v_i = flood storage capacity required at the end of the ith month of the year,

b_i = linear decision rule parameter for the ith month of the year, <u>to be determined</u>,

S min = on-site retention basin capacity, <u>to be determined</u>,

r_t = postulated on-site retention basin input in the tth month of operation,

X_t = release during the tth month of operation, <u>to be determined</u> by the linear decision rule,

S_t = storage at the end of the tth month of operation, <u>to be determined</u> by the linear decision rule,

$r_i \cdot 90$ = the flow which is exceeded in period (i) only 10 percent of the time,

$r_i \cdot 10$ = the value which the flow in period i falls below only 10 percent of the time.

The variables q, f, v, and b are indexed by a parameter i = 1, ..., 12 because their values in the ith month are the same from year to year. The values r, x, and s, however, do not follow a regular cyclic pattern

and therefore are indexed by the parameter $t = 1, \ldots, n$, where n = total number of monthly values.

Deterministic formulation has some limitations. First, the deterministic formulation yields no explicit statement of the reliability with which the retention basin will meet the specific performance objectives in the future. Secondly, the retention basin's reliability is fixed fortuitously by the specific postulated input sequence and is not under the direct control of the designer. A chance constrained linear decision rule eliminates these deficiencies.

Readers interested in the deterministic and chance constrained formulation of retention basin design problems are referred to the existing literature on the subject (6, 8, 11, 15). Presented below is the chance constrained linear decision rule formulated in terms of linear programming.

The original rule was written as:

$$X = S - b \tag{15}$$

In order for the decision variable b to take either positive or negative values, the problem is modified as follows:

$$b = h_i - g_i \tag{16}$$

The new modified equation will be:

$$X = S - (h_i - g_i)$$

The problem posed here is to determine a minimum sized retention basin which meets the specified requirements for a) freeboard, b) minimum storage, c) low flow maintenance, and d) high flows. The reliability parameter selected is 90 percent. In other words, 90 percent of the time the specified requirements will be assured.

The problem is formulated as a linear programming problem, as follows:

Minimize C

Subject to:

1) $C + g_i - h_i \geq r_i \cdot .90 + v_i$

$i = 1, 2, \ldots, 12$ (18)

2) $A_m C + g_i - h_i \leq r_i \cdot .10$

$i - 1, 2, \ldots, 12$ (19)

3) $g_i - h_i - g_i - 1 + h_i - 1 \geq q_i - r_i - 1 \cdot .10$

$i = 2, 3, \ldots, 12$ (20)

$g_1 - h_1 - g12 + h_{12} \geq q_1 - r_{12} \cdot .10$ (21)

$A_0 C + g_1 - h_1 \geq q_1$ (22)

4) $g_i - h_i - g_{i-1} + h_{i-1} \leq f_i - r_{i-1} \cdot .90$

$i = 2, 3, \ldots, 12$ (23)

$g_1 - h_1 - g_{12} + h_{12} \leq f_1 - r_{12} \cdot .90$

$A_0 C + g_1 - h_1 \leq f_1$ (24)

The total number of constraints in the above posed linear programming problem is 50. The number of unknowns is 13; they consist of 12 monthly discharge values and a retention pond size of minimum capacity.

It is also appropriate to indicate that not every retention basin design problem formulated as above has a feasible solution. There exists three necessary conditions which have to be satisfied if the possibility exists for a feasible solution. They are:

1) $\sum_{i=1}^{12} q_i \leq \sum_{i=1}^{12} r_i \cdot .10$ (25)

The necessary condition as written above states that no retention basin exists that meets the specified performance objectives under a linear decision rule unless the sum of the desired minimum releases is less than the sum of the monthly inputs occuring at the corresponding desired reliability level (90th percentile flow).

The second necessary condition for the problem to have a feasible solution is:

$$2) \quad \sum_{i=1}^{12} f_i \geq \sum_{i=1}^{12} r_i^{.90} \tag{26}$$

This constraint states that to prevent flooding the maximum release capacity has to be greater than the 10 percentile flow.

The third necessary condition for high and low constraints to hold simultaneously for period i, is as follows:

$$3) \quad f_i - q_i \geq r_i - 1^{.90} - r_i - 1^{.10} \tag{27}$$

Routing Model

In the development of the hydrologic accounting simulation no structural measure or control was superimposed in the model. Strictly, the model was developed to generate monthly runoff values based on pertinent hydrologic parameters of the watershed basin.

The operational model developed in the previous chapter uses the monthly runoff value generated by use of the hydrologic accounting model and other system constraints as described, and sizes a retention pond associated with 12 monthly regulatory stages. The effect of the reservoir capacity and the associated 12 monthly regulatory stages has to be tested by other means to examine the basin response.

By use of the routing model the basin response in terms of water table stages can be computed without actually measuring the stages at several locations in the basin. The water stages generated could then enable the decision maker to determine whether or not a water control structure is required at any point in the basin.

The routing model uses the continuity equation as applied to reservoirs. The response of the operational model will then be compared against the regulatory stage that is presently being used.

The following notations are used in the development of the routing model:

A_t = total area of the basin to be developed,

A_W = area in the retention pond (percentage of the total basin area),

R_W = rainfall in the retention basin,

R_O = runoff generated in the areas contributing to the retention pond,

E = evaporation from the retention pond,

S = seepage from the retention pond,

ΔS = change in storage volume,

Δ Stage = change in water stage in the pond,

BS = initial water storage in the pond,

B Stage = initial water stage in the pond,

ES = end of month storage in the pond,

E Stage = end of month water stage.

$$\Delta S = \frac{R.FALL \times A_W}{12} + \frac{(A_t - A_W) \times R_O}{12} - \frac{E \times A_W}{12} - \frac{S \times A_W}{12} \tag{28}$$

$$ES = BS \pm \Delta S \tag{29}$$

$$\Delta \text{ Stage} = \frac{\Delta S}{A_W} \tag{30}$$

$$E \text{ Stage} = B \text{ Stage} \pm \Delta \text{ Stage} \tag{31}$$

APPLICATIONS

Hydrologic Accounting Model Application

Application of the above described model (equations 2-9) was made for a 50 square mile area located between C-9 and C-11, west of the Flamingo Road Canal, and east of Conservation Area 3 (Figure 2) in western Broward County. The area is flat, low lying, and is characterized by sparse vegetation. The area is covered by mucky material to a depth of approximately 2.0 feet. The permeability of the soil varies from six to twenty inches per hour (16).

Effective Rainfall

Effective rainfall for the basin was estimated by use of eleven rain gage stations scattered around the basin (Figure 2). Use was made of an in-house model to determine the Theissen coefficients. The Theissen polygon method gives the weight of each station with respect to the S-9 basin. Out of eleven FCD stations, only five stations were found to have influence on the S-9 basin and the weight of each of these stations are presented in Table 1 below:

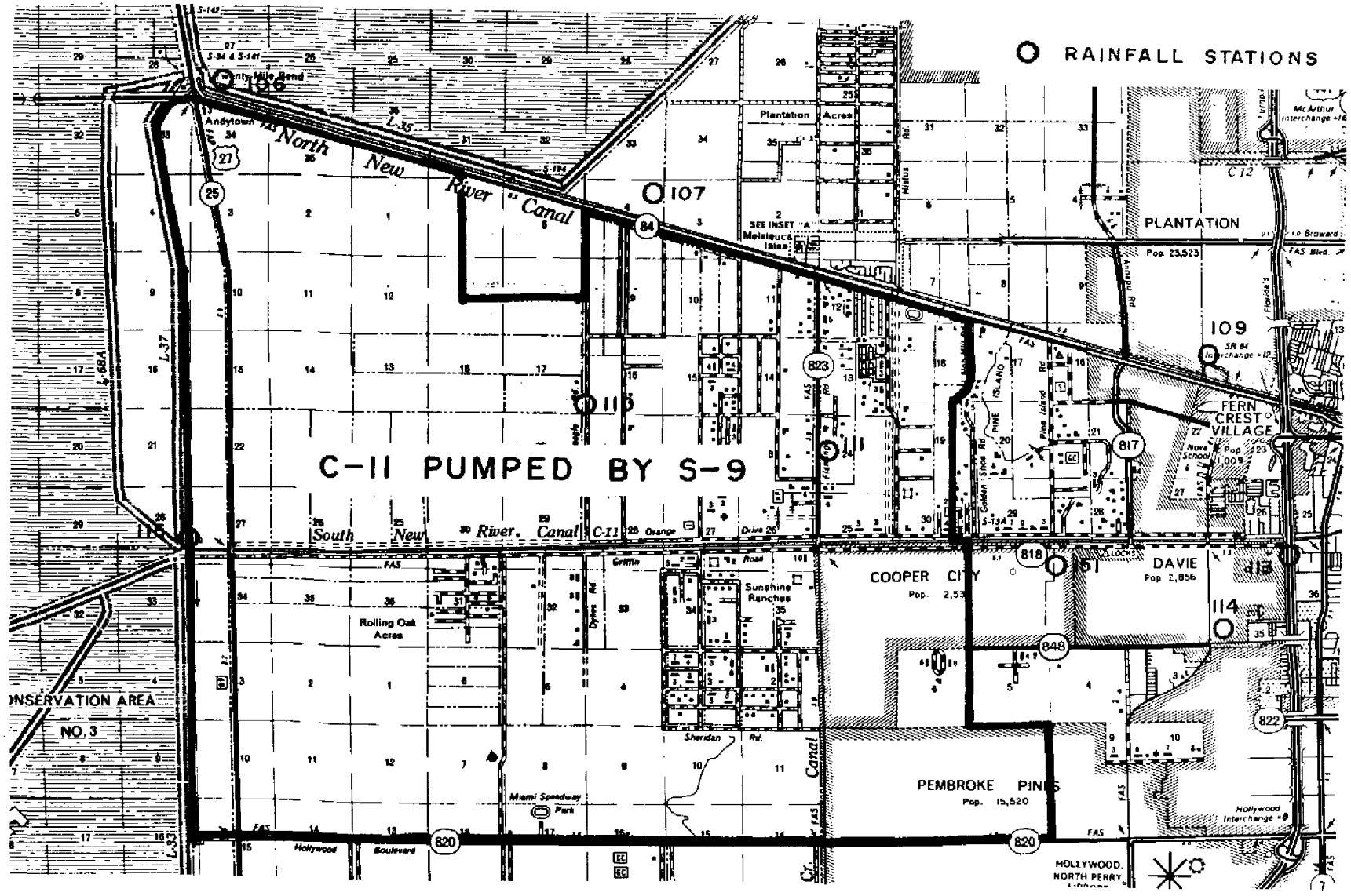

Figure 2 PROJECT AREA

TABLE 1 - THEISSEN COEFFICIENTS - S-9 BASIN

FCD STATION NUMBER	RESPECTIVE WEIGHT
115	.267
106	.093
107	.053
110	.392
151	.195
TOTAL	1.000

The effective rainfall presented in Table 2 was estimated by using the above weights to each of the rainfall values from the above listed stations.

TABLE 2 MONTHLY EFFECTIVE RAINFALL FOR S-9 BASIN - (INCHES)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
1963	1.20	3.72	.05	.86	4.06	4.01	2.86	4.63	11.25	5.26	2.09	4.01	44.00
1964	2.00	2.48	.51	5.11	7.21	6.41	5.99	6.90	4.24	11.94	3.08	1.64	57.51
1965	.33	3.13	.66	.45	.24	8.72	8.97	3.62	7.12	7.32	1.87	.46	42.91
1966	3.41	3.21	1.80	2.13	4.30	18.20	7.47	6.18	5.48	7.38	1.37	.40	61.33
1967	2.48	1.71	1.11	.15	1.22	15.21	3.56	5.78	8.44	10.01	1.88	2.21	53.76
1968	1.21	3.44	1.15	1.05	17.27	15.55	5.12	4.39	4.89	6.97	.82	.02	61.88
1969	3.46	1.82	3.16	3.60	4.83	7.72	7.99	4.73	5.79	17.81	2.44	.70	63.75
1970	3.59	2.29	8.66	.17	7.84	6.64	3.48	4.91	6.21	4.70	.15	.21	48.85
1971	.71	.80	.38	.17	3.91	8.74	6.72	5.71	7.26	8.43	2.20	2.10	47.13
1972	1.60	2.11	5.26	2.82	8.31	11.10	10.25	7.10	5.22	3.74	4.11	2.53	64.15
1973	1.89	1.35	3.51	.59	3.47	11.79	10.83	9.44	9.49	3.97	3.16	1.82	61.31
MEAN	1.99	2.37	2.39	1.55	5.70	10.34	6.66	5.76	6.85	7.96	2.11	1.46	55.14

Evapotranspiration

Monthly evapotranspiration values as estimated by Stuart and Mills (20) for the Plantation area, were used in the hydrologic accounting model. These data are presented in Table 3 below.

TABLE 3 - WEIGHTED MONTHLY EVAPOTRANSPIRATION, INCHES

MONTH	ET, INCHES
January	2.02
February	2.51
March	3.25
April	4.21
May	5.21
June	4.25
July	4.81
August	4.79
September	3.85
October	3.42
November	2.50
December	1.92
TOTAL	42.74

Estimation of Seepage from Conservation Area 3A and 3B

Seepage from the levees bordering the Conservation Areas into the project area was estimated by Leach, Klein, and Hampton (22) to be 8-9.6 cubic feet per second (cfs) per mile of levee. The length of the levee within the project area is approximately 10 miles; therefore, the total seepage flow according to their estimate is 80-96 cfs.

Seepage was also estimated to use of recession curve analysis for the month of January (18-25), 1971, and the month of April (3-10), 1972. The average seepage flow from the recession curves is estimated to be 105 cfs.

Moisture Holding Capacity of the Soil

The soil moisture status for the hydrologic accounting simulation was determined from rain that occurred during the previous month, and the total moisture holding capacity of the soil. The Soil Conservation Service report (16) on the soil classification of Broward County estimates the soil horizon to be 14 inches deep with a moisture holding capacity of .20 - .30 inches per inch for the horizon. As a consequence of discussions with colleagues concerning the moisture holding capacity of the soil within the project area, it was decided to use the lower storage capacity of .20 inches per inch in the model. In other words,

2.80 inches (14 X .20 inches per inch) of moisture will be held by the soil when it is at field capacity. After the soil is brought to field capacity any excess amount of water will either be discharged as runoff from the basin, act as recharge to the water table aquifer, or both.

Assumptions

Several assumptions were made in the model as follows:

1. For the start of hydrologic accounting simulation it was estimated that 2.40 inches of rainfall was required to bring the soil moisture to field capacity.
2. Runoff would not occur until the soil moisture is brought to field capacity.
3. Storage coefficient of the soil is .2. Thus one inch of rain will bring 5 inches of soil to field capacity (13).
4. Evapotranspiration takes place at a potential rate up to a depth of 6 inches and is then reduced in a linear fashion up to a depth of 7.0 feet; ET then ceases completely.

Results

The hydrologic accounting model represented by equations 2-9 in the text and satisfied by the assumptions as stated

above was run on a monthly time frame for the years 1963-1973. In Table 4 below are presented the measured and the computed discharge for the years simulated in the undeveloped condition. These data are also presented in Figure 3.

TABLE 4 - COMPARISON OF MEASURED VS. COMPUTED DISCHARGE IN THE S-9 BASIN (INCHES)

YEAR	MEASURED	COMPUTED
1963	9.74	16.98
1964	20.88	21.85
1965	20.61	22.24
1966	34.66	28.51
1967	26.47	28.24
1968	33.45	33.17
1969	30.65	30.50
1970	16.31	18.67
1971	25.42	20.47
1972	30.01	25.37
1973	28.59	24.97
MEAN	25.16	24.63
STD. DEV.	7.62	5.11

The computed discharge matches fairly well with the measured discharge.

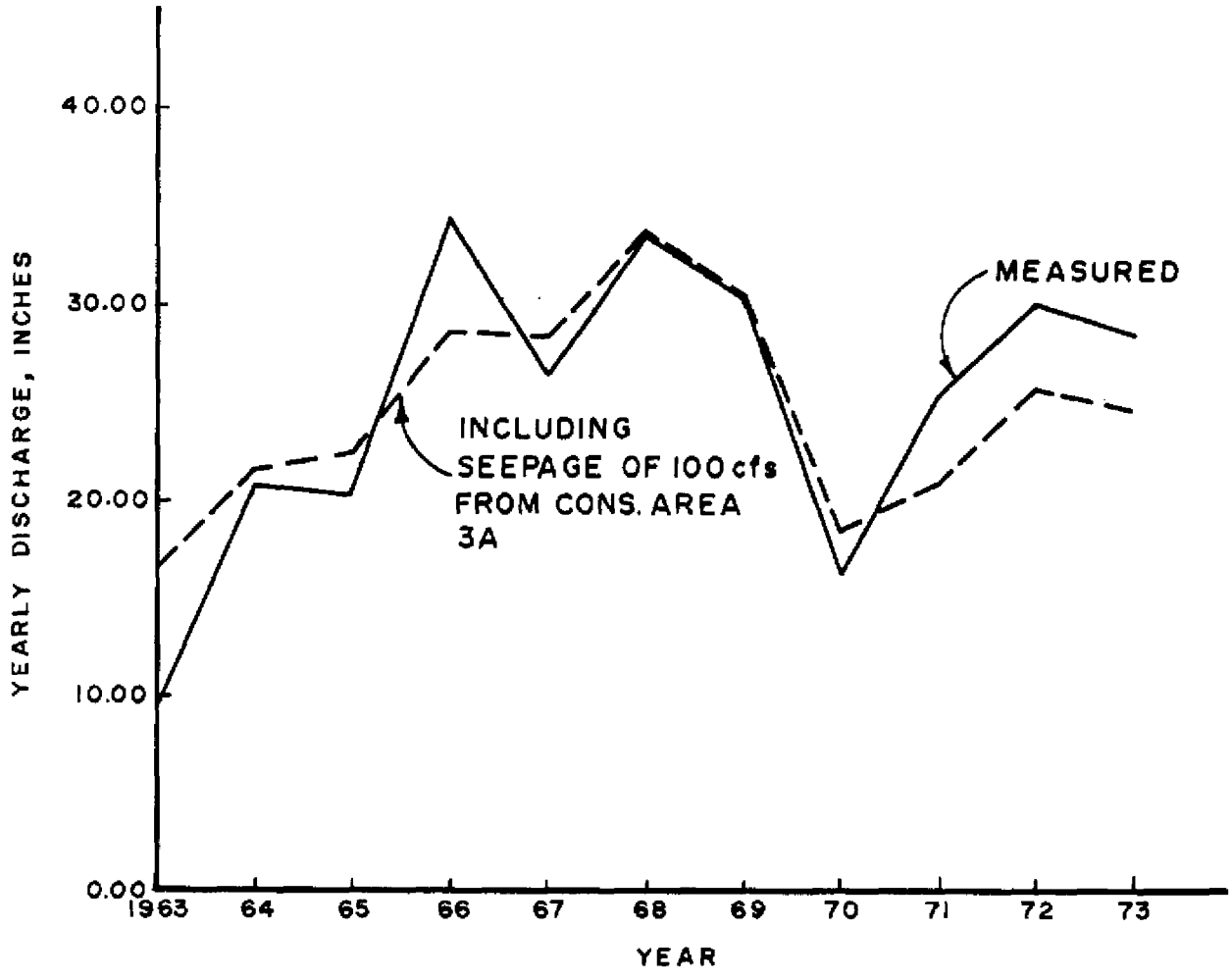

Figure 3 COMPUTED vs MEASURED DISCHARGE THRU STRUCTURE S-9

Hydrologic Accounting Model as Applied to Land Use Changes

Having calibrated the hydrologic accounting model by comparing the generated discharges with the measured discharges for the undeveloped condition, the model was then used to generate runoff for several hypothetical urbanized configurations (see Figure 4). Harley's (23) statement on the difference between urban and rural hydrology was in terms of the impervious cover placed on the natural land. When an impervious cover is placed on the natural land the evapotranspiration which takes place from the subsurface is reduced and concurrently, runoff is increased. Due to lack of pertinent data, and also due to the fact that ET is the major factor in increasing or decreasing the amount of runoff from an area, it was assumed that if a certain percentage of the drainage basin is urbanized, there will be a concomitant decrease in subsurface ET by the same percentage for that portion of the total drainage basin.

A recent study by the U. S. Geological Survey (22) indicates that out of 42 inches of average annual evapotranspiration, 20 inches takes place from subsurface soils and 22 inches from the surface. For the sake of simplicity, it is assumed here that the ratio of surface to subsurface ET is on a one to one basis.

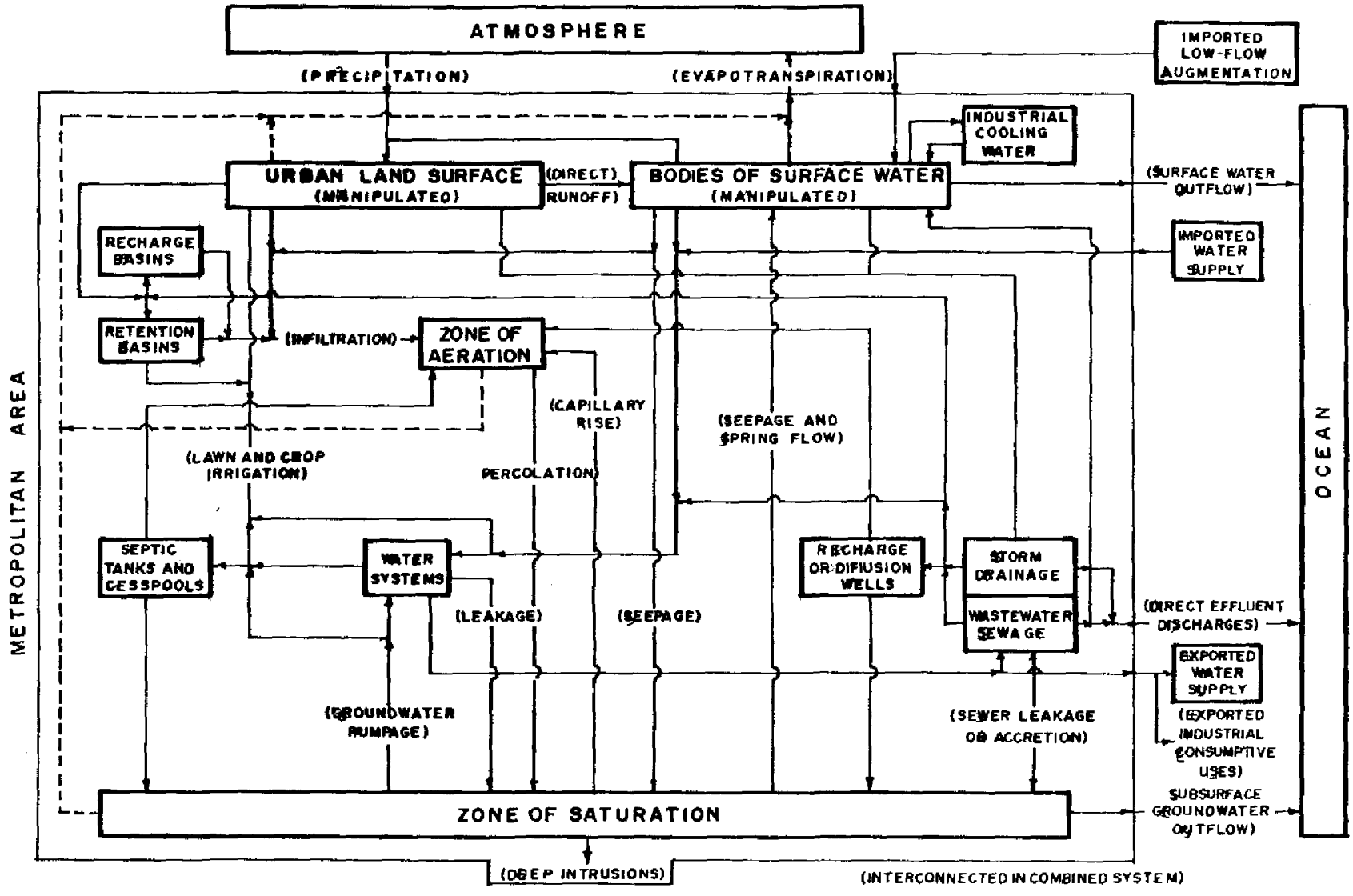

Figure 4 URBAN HYDROLOGIC SYSTEM

(ADAPTED FROM: "SUMMARY OF THE HYDROLOGICAL SITUATION IN LONG ISLAND, N.Y. AS A GUIDE TO WATER MANAGEMENT ALTERNATIVES," BY O.L. FRANKE AND N.E. McCLYMONDS, U.S. GEOLOGICAL SURVEY PROFESSIONAL PAPER 627-F, 1972

Based on past experience, if 50 percent of the total area is covered with impervious surface, it is assumed in this study that the whole basin is fully developed.

With assumptions as stated above, the hydrologic accounting model which was calibrated for the natural condition was run for the hypothetical urbanized condition. Table 5 below, presents the increase in yearly runoff due to this hypothetical urbanization. They are also presented in Figure 5. It is appropriate to mention here that as more land in the basin is developed (and raised) to higher elevations, seepage from the conservation areas is reduced consistent with changes in hydraulic gradients.

TABLE 5 - INCREASE IN RUNOFF FROM THE S-9 BASIN FOR VARIOUS PERCENTAGES OF HYPOTHETICAL LAND USE CHANGES (IN INCHES)

YEAR	0	10	20	30	40	50	60	80	100
1963	16.98	17.97	18.96	20.32	19.79	20.84	22.83	25.89	27.44
1964	21.85	21.17	23.21	23.90	26.04	26.81	28.95	33.23	37.50
1965	22.24	22.39	23.14	24.85	25.11	24.02	24.40	26.74	29.79
1966	28.51	30.45	31.74	32.36	34.50	36.64	38.65	41.32	41.38
1967	28.24	28.57	29.02	29.56	30.97	31.67	31.41	33.03	36.46
1968	33.17	33.87	34.65	36.28	38.11	39.94	41.77	44.85	47.32
1969	30.50	29.79	32.02	33.60	34.50	34.04	36.30	38.11	42.63
1970	18.67	20.65	21.22	21.79	23.72	24.45	25.10	29.13	33.15
1971	20.47	20/03	21.75	22.12	22.48	23.71	24.71	28.48	31.46
1972	25.37	27.63	29.76	31.90	32.68	33.461	35.59	38.51	42.78
1973	24.97	26.83	26.05	27.62	27.48	29.51	31.47	35.75	38.66
AVG	25.16	25.40	26.50	27.66	28.67	29.55	31.02	34.09	37.14

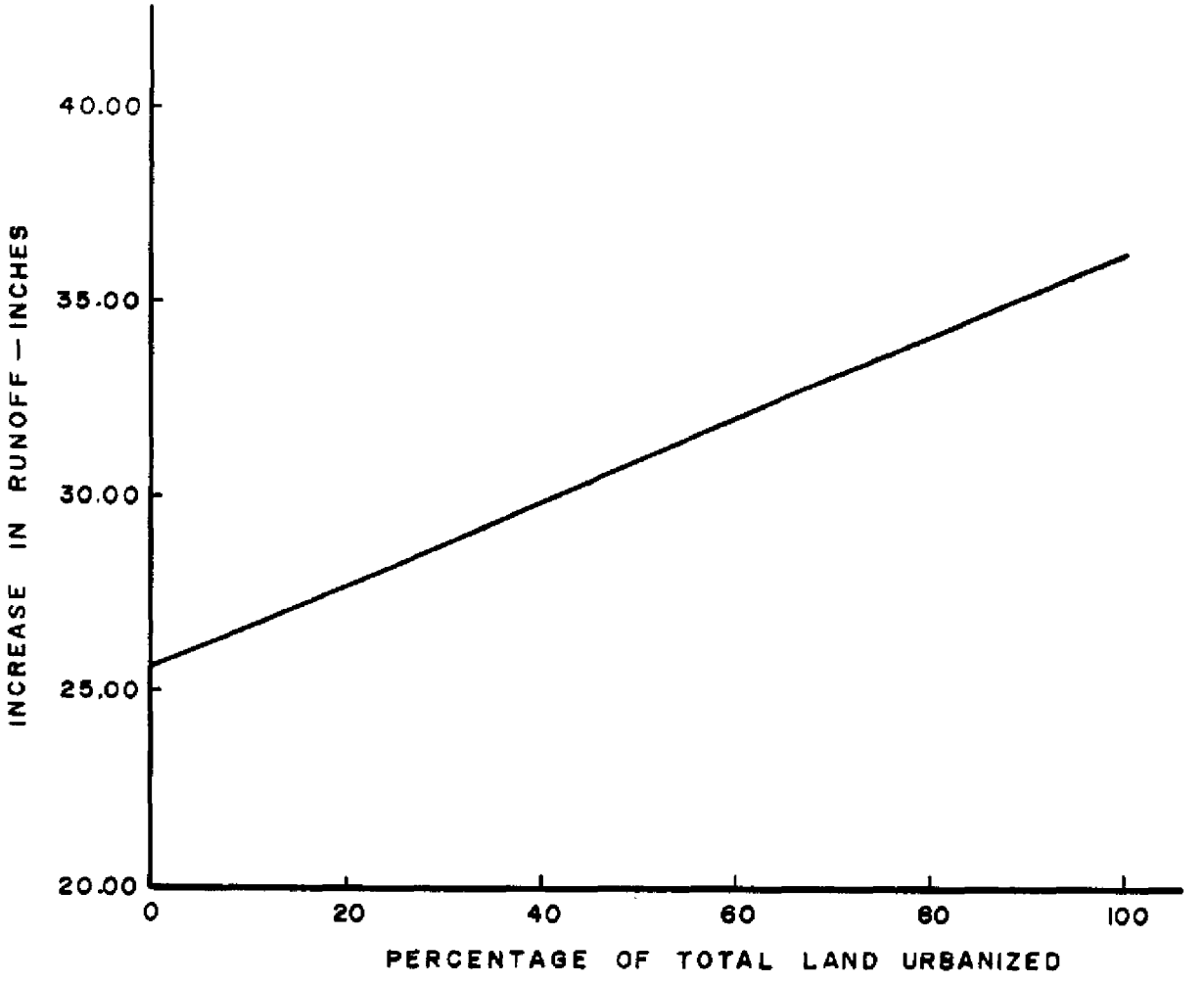

Figure 5 INCREASE IN RUNOFF FOR VARIOUS PERCENTAGES OF LAND USE CHANGES

Results

As seen from Table 5, and Figure 5, the increase in runoff from 0 to 100 percent urbanization is about 78 percent on average. This percentage increase in runoff generation does not seem to be out of proportion. As stated earlier in the text, out of total subsurface ET of 21 inches, when the whole basin is urbanized (50 percent in impervious cover), the subsurface reduction in ET is considered to be approximately 10.5 inches. If this amount is superimposed on the runoff generated from the unurbanized basin (see Table 4), the runoff amounts to 35.66 inches in lieu of 37.14 inches generated by use of the hydrologic accounting model (Table 5).

It is appropriate at this point to compare the results obtained from our hypothetical urbanization analysis with other published sources. John B. Stall (19) states that for small basins in Jackson, Mississippi, the magnitude of the mean annual flood for a totally urbanized basin without any provision for on-site retention was about 4½ times that of a smaller rural basin, and that the 50 year flood for an urban basin was about 3 times that of a rural basin.

By use of streamflow frequency analysis, Anderson (4) found the ratio of average to peak flood for different

degrees of imperviousness. For a flood of 1 in 20 year frequency the ratio of the average to peak flow was 1.8 - 3.0 or 66 percent higher. As the return period increased the ratio of the average to peak flood decreased.

Application of the Operational Model

This portion of the overall study of the S-9 basin is concerned with only one phase of the total task—that of decision making in relation to operation of the basin. By use of the operational model a set of rules for storing and releasing water from the basin will be generated.

Certain basic considerations referred to as determinants, govern the decision making. In the context of this study, they are a) system parameters, b) basic hydrologic data, and c) the internal condition of the system.

The basic hydrologic data include primary information on inflows to the basin. In the table below, are presented the mean monthly inflows generated due to hypothetical urbanization of the basin (100 percent case; 50 percent in impervious cover) and the associated standard deviations.

TABLE 6 - THE FIRST AND SECOND MOMENTS OF THE MONTHLY INFLOWS GENERATED DUE TO 100 PERCENT URBANIZATION OF THE S-9 BASIN (INCHES)

MONTH	MEAN	STANDARD DEVIATION
January	1.18	.59
February	1.12	.62
March	2.04	1.84
April	1.24	.48
May	3.41	3.56
June	7.17	4.87
July	4.24	2.76
August	3.37	1.62
September	4.56	1.98
October	5.98	4.30
November	1.24	.76
December	1.40	.60
TOTAL	36.95	

A comparison of Tables 5 and 6 show that the mean yearly values obtained by adding the mean values of each month is slightly lower than the mean value obtained by averaging the total yearly values. The discrepancy, which is due to rounding errors, is small. A normal probability distribution was fitted to the monthly runoff values generated due to 100 percent hypothetical urbanization (50 percent in impervious cover) and the 10th and 90th percentile of the probability distribution of flows were computed. They are presented in Table 7.

TABLE 7 - THE 10th AND 90th PERCENTILE OF THE PROBABILITY DISTRIBUTION OF MONTHLY RUNOFF VALUES GENERATED DUE TO 100 PERCENT URBANIZATION (INCHES)

MONTH	FLOW EXCEEDED 10 PERCENT OF TIME $r_i \cdot 90$ - INCHES	FLOW EXCEEDED 90 PERCENT OF TIME $r_i \cdot 90$ - INCHES
January	1.94	.42
February	1.41	.33
March	4.40	.32
April	1.85	.63
May	7.97	1.15
June	13.40	.94
July	7.77	.71
August	5.44	1.30
September	7.09	2.03
October	11.48	.48
November	2.21	.27
December	2.17	.63
TOTAL	67.13	9.21

Presented in Table 8 below are the monthly minimum releases incorporated in the operational model development that are committed to downstream beneficial uses.

TABLE 8 - ASSUMED MINIMUM RELEASES FOR EACH MONTH FROM THE S-9 BASIN (INCHES)

MONTH	MINIMUM RELEASES, q_i, INCHES
January	1.0
February	1.0
March	2.0
April	2.0
May	1.0
June	0.0
July	0.0
August	0.0
September	0.0
October	0.0
November	1.0
December	1.0
TOTAL	9.00

TABLE 7 - THE 10th AND 90th PERCENTILE OF THE PROBABILITY DISTRIBUTION OF MONTHLY RUNOFF VALUES GENERATED DUE TO 100 PERCENT URBANIZATION (INCHES)

MONTH	FLOW EXCEEDED 10 PERCENT OF TIME $r_i \cdot 90$ - INCHES	FLOW EXCEEDED 90 PERCENT OF TIME $r_i \cdot 90$ - INCHES
January	1.94	.42
February	1.41	.33
March	4.40	.32
April	1.85	.63
May	7.97	1.15
June	13.40	.94
July	7.77	.71
August	5.44	1.30
September	7.09	2.03
October	11.48	.48
November	2.21	.27
December	2.17	.63
TOTAL	67.13	9.21

Presented in Table 8 below are the monthly minimum releases incorporated in the operational model development that are committed to downstream beneficial uses.

TABLE 8 - ASSUMED MINIMUM RELEASES FOR EACH MONTH FROM THE S-9 BASIN (INCHES)

MONTH	MINIMUM RELEASES, q_i, INCHES
January	1.0
February	1.0
March	2.0
April	2.0
May	1.0
June	0.0
July	0.0
August	0.0
September	0.0
October	0.0
November	1.0
December	1.0
TOTAL	9.00

The internal condition of the system at any point in time is created jointly by past inflows and by the operating procedure. Basically they focus on the surface and subsurface water content in the basin.

In the past, the S-9 basin was operated by maintaining a water stage of 3.5 feet MSL. In addition, the system design of the basin is such that the discharge during storm periods is limited to 3/4 inches per day. However, on a cumulative basis, based on 30.5 days in a month, the allowable discharge could be as much as 22.88 inches per month.

Derivation of the operating rules based on system design, internal condition of the basin and the hydrologic condition as outlined above is forumlated as follows:

Given monthly inflows (Table 7) for a 100 percent hypothetical urbanized situation (50 percent in impervious cover) and the committed monthly releases (Table 8), it is required to compute the minimal retention pond capacity and the monthly operational stages subject to the following system constraints:

a) Maintenance of low flows (90 percentile flow) for downstream beneficial uses.

b) Prevention of flood damage up to and including the 10th percentile flows.

c) Regulatory allowable discharge of 3/4 inch per day from the basin.

d) A minimum of ten percent of the total storage capacity of the retention pond is to be maintained at all times. The lower limit volume capacity is committed for water table recharge purposes. This percentage of course can be increased if desired, to hold more water for groundwater recharge purposes. However, the capacity of the retention pond will also be increased due to the fact that storage must be available on top of this minimal storage to accommodate the water generated by storms of varying frequency.

e) Ninety percent reliability level on all the above stated constraints.

The three necessary criteria (equations 26, 27, and 28) as outlined in the text are checked to determine the feasibility of a solution by use of the Linear Programming model.

The first criterion states that the minimum regulatory outflow has to be equal to or smaller than the base flow from the basin (equation 26). The minimum 90th percentile

flow is 9.21 inches (Table 7) and the minimum base flow to be maintained is 9.00 inches (Table 8). Therefore, the first necessary condition is met.

The second necessary criterion is that of maximum yearly discharge capacity. This quantity must be greater than the 10th percentile flow (equation 27). The total yearly inflow which exceeds 10 percent of time is 67.13 inches (Table 7). The yearly discharge capacity is (12 X 22.28) 267.36 inches. Thus, the second necessary condition is also satisfied.

The third necessary criterion states that the difference between the 10th and the 90th percentile flow should be lower than the difference between the maximum and the minimum releases for each time period. A check on the high flow and low flow reveals that the third necessary criterion is also met (equation 28).

The fractional storage to be maintained with 90 percent reliability was assumed to be .10. In other words, 90 percent of the time the reservoir will be within 10 percent of the total storage capacity.

The linear decision model is used on a monthly time frame to develop the operational policy. Monthly runoff values presented in Table 7 were multiplied by the total S-9

basin area to convert them from inches to acre-feet. The monthly regulatory releases (Table 8) also were converted to acre-feet.

Results

Equations 18-24 which incorporate the above stated constraints on inflows, allowable discharge, regulatory releases, and maintenance of the 10 percent of storage capacity was programmed in the computer. It was determined by use of the linear decision model (equations 18-24) that the minimum volume required to satisfy the above stated constraints was 47,433 acre-feet. If this volume of water is impounded in 20, 25, and 30 percent of the total basin (6,400 8,000 and 9,600 acres), the depth of water in the retention pond would be 7.41 feet, 5.93 feet and 4.67 feet respectively. These water depths were obtained from the linear decision rule model (equations 18-24) and should be added to the average groundwater stage of 3.5 feet MSL in the basin. In other words, the highest groundwater stages in the S-9 basin will be 10.91 feet, 9.43 feet and 8.17 feet MSL respectively. The impact and consequences of these options is explored in greater detail in a subsequent section of this study.

In addition to the retention pond volume, the linear decision model also generates 12 monthly operational storage values. They are converted to appropriate groundwater stages for 20, 25, and 30 percent of the total basin area in retention ponds, and are presented in Table 9 and restated as Figures 6, 7, and 8.

In the retention pond, the stages derived by use of the operational model is similar to the rule curve type of regulation being used in the operation of Lake Okeechobee and the conservation areas. Thus, stages in the retention ponds will be lowered to the 10 percent storage level during the start of the rainy period and will be raised gradually reaching optimal levels as the rainy season ends. There is minimal storage (10 percent of the total capacity) maintained throughout the entire cycle. This is the quantity that can be added to the natural recharge as calculated along classical lines. This average additional recharge is estimated to be 3.6 inches for the above three cases over the basin (1.19 X 12 X 8,000/32,000) which is equivalent to 9520 acre-feet. Evaporation from the retention basin and outflow from the pond via seepage has not been taken into account. These parameters will be considered in a subsequent section of this study which addresses the routing model.

TABLE 9 - REGULATORY STAGES ABOVE EXISTING GROUNDWATER TABLE ELEVATIONS (FEET)

	PERCENTAGE OF TOTAL AREA IN RETENTION POND		
MONTH	20%	25%	30%
January	2.11	1.69	1.41
February	1.87	1.50	1.25
March	1.18	.94	.79
April	.48	.28	.32
May	.26	.21	.18
June	.56	.45	.37
July	.95	.76	.63
August	1.24	1.00	.83
September	1.78	1.43	1.19
October	2.63	2.10	1.75
November	2.57	2.06	1.71
December	2.27	1.81	1.51
AVERAGE	1.50	1.19	1.00

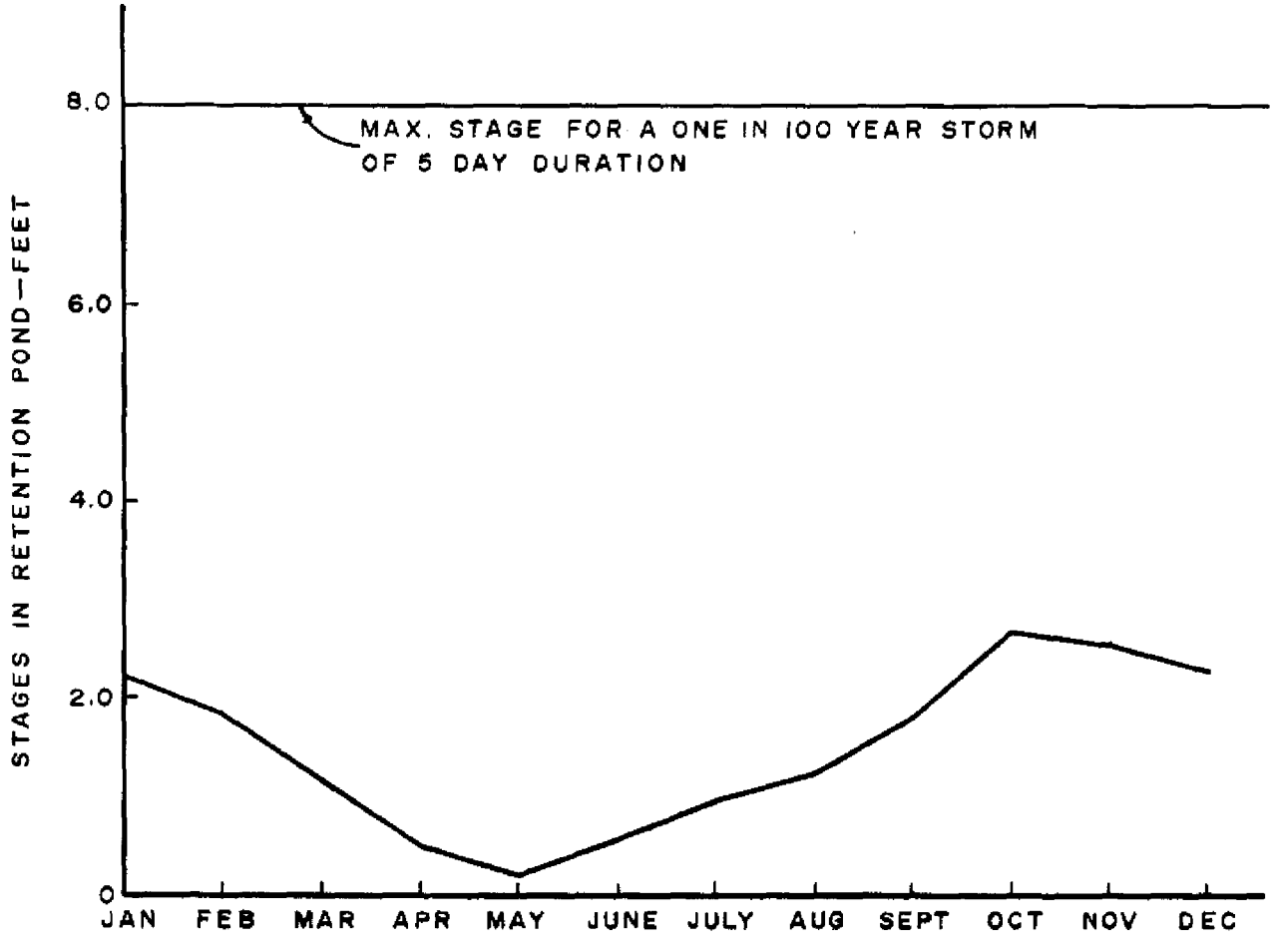

Figure 6 MONTHLY OPERATIONAL STAGES FOR 20 PERCENT OF THE AREA IN RETENTION POND

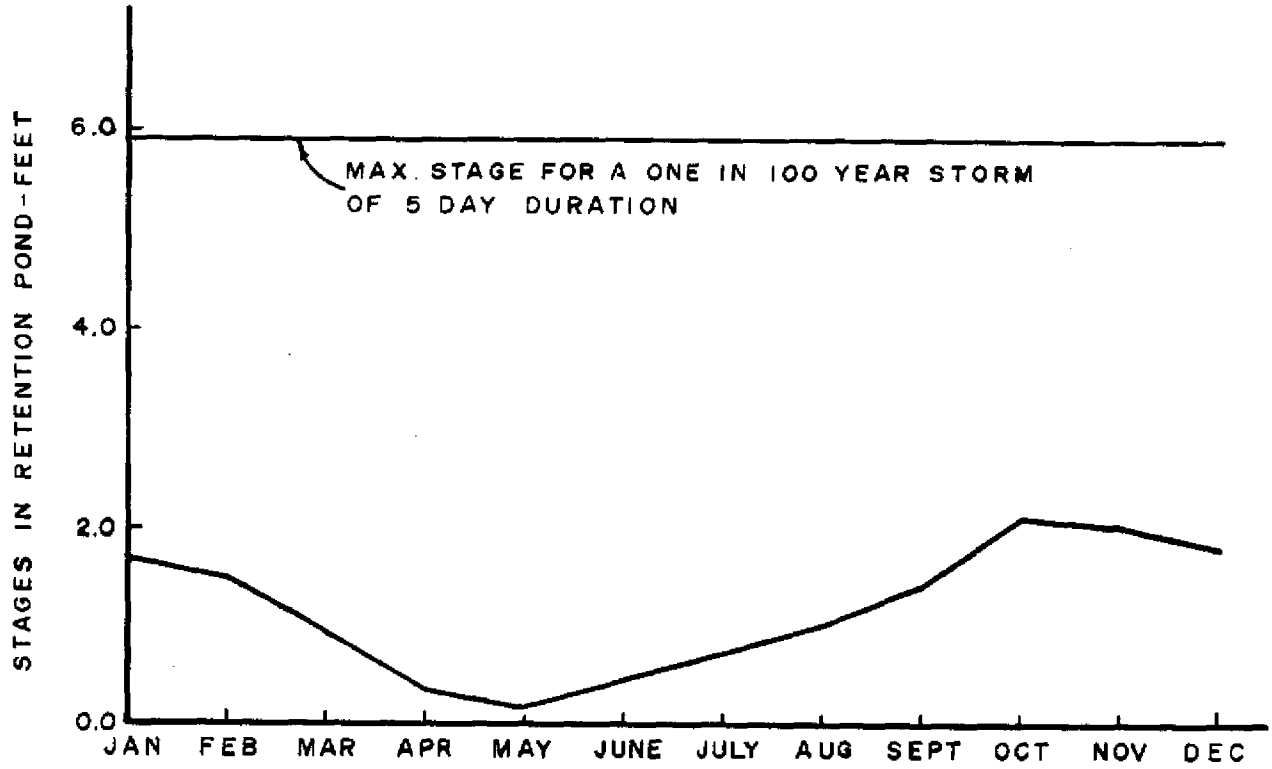

Figure 7 MONTHLY OPERATIONAL STAGES FOR 25 PERCENT OF THE AREA IN RETENTION POND

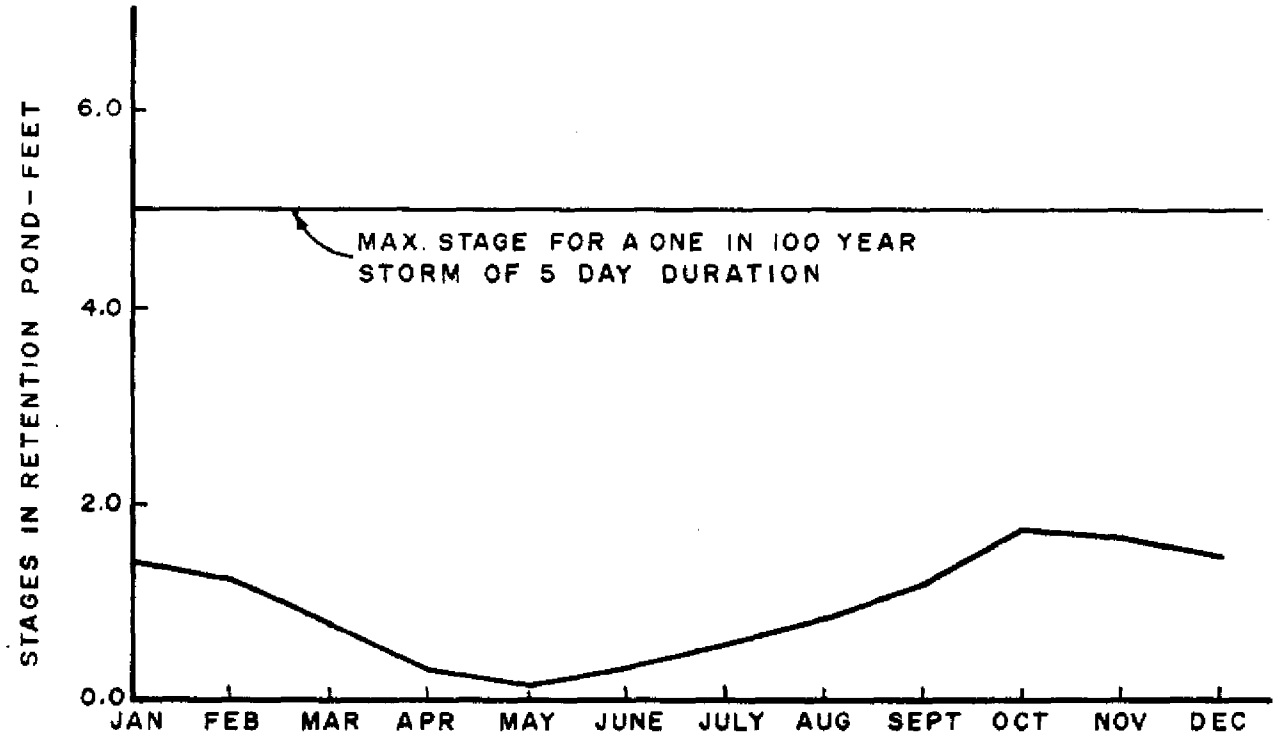

Figure 8 MONTHLY OPERATIONAL STAGES FOR 30 PERCENT OF THE AREA IN RETENTION POND

Conventional Method of Sizing the Retention Pond

The stages that were generated by use of the operational model by devoting 20, 25, and 30 percent of an urbanized area in retention ponds were previously described. At the present time Broward County requires developers to store on-site the 5 day 1 in 100 year flood event (16.8 inches or runoff based on 20.8 inches of rainfall). It is therefore pertinent to compare this requirement with the several options developed in the operational model.

This volume (16.8 inches) of runoff from 32,000 acres (50 square mile area project area) would amount to 44,800 acre-feet. If this volume of water is impounded in 20, 25, and 30 percent of the total basin area, the depth of water above the existing average static water level (3.5 feet MSL) would then be 10.50 feet, and 8.17 feet MSL respectively, in lieu of 10.91 feet, 9.43 feet, and 8.17 feet MSL generated by the operational model. It is interesting to note that for 30 percent of the total area option in retention pond, the same maximum stage is generated by both the methods. The higher groundwater stages obtained by use of the operational model stems from the fact that 10 percent of the storage is maintained all the times for recharge purposes during dry periods.

Both the linear decision model and the conventional method of approach determines the capacity of the retention pond. However, the conventional method does not produce an operational policy with system constraints as was imposed in the development of the operational model. In current practice a retention pond is sized for flood control purposes primarily, and groundwater recharge as an incidental benefit. Groundwater recharge would then occur as an incidental consequence of water being temporarily stored on site.

Routing Model Application

Some form of operational scheme is necessary for any water resources planning and design; however, it is not sufficient by itself. The performance of the watershed basin in totality (including operational scheme) has to be examined by other means via routing techniques.

This portion of the study is intended to examine the S-9 basin response via routing. The proposed and the present regulatory schemes will be evaluated by use of the routing model and will be compared. It is appropriate to state here that in the proposed regulatory scheme the groundwater stages are slightly higher than the present regulatory stages during dry periods for water conservation purposes.

A retention reservoir which is 25 percent of the total basin area (32,000 acres) is assumed. Inflows and outflows from the basin are routed through this reservoir. The pertinent hydrologic parameters that were incorporated in the routing model are as follows: a) rainfall that falls directly on the retention reservoir, b) evaporation that takes place from the retention reservoir (evaporation was estimated to be 125 percent of the evapotranspiration that was used in the hydrologic accounting model), c) groundwater elevation of 2.2 feet MSL for the start of routing, and

d) seepage that is estimated to take place from the retention pond.

Seepage Estimation From the Retention Pond

The following parameters were assumed to be known for the static groundwater condition:

Transmissivity (T) - 2 X 10^6 gallons/day/foot (24)

Hydraulic gradient (I) - Stage in the retention reservoir (maximum of 5.0 feet).

- The stages in the primary canals (assumed to be 3.5 feet MSL) over some distance (assumed to be 3.0 miles).

The total volume of the retention reservoir is 47,433 acre-feet (retention capacity determined from the linear decision rule). The retention reservoir is assumed to be circular and the perimeter of the circle from which seepage takes place is calculated as follows:

$$\frac{47,433}{\text{maximum proposed regulatory stage of 5.0 feet}} = \pi r^2 =$$

area of circle

radius = r = 11,471 feet. Perimeter (L) of the circle = $2\ \pi r^2$ = 72,043 feet.

Seepage estimation was made by use of the generalized form of the Darcy equation:

$Q = T.I.L.$

Where,

T, I, and L are as defined above

Using the above approximated values, one obtains:

$Q = 1.92$ inches/month

From the above calculation it can be stated that under non steady state conditions the seepage from the retention reservoir to the adjoining land would be around .16 X 8000 = 1280 acre-feet/month.

Routing

Basin routing was approached in the following fashion. The effect of local rainfall on the retention reservoir, the evaporation from the reservoir, the inflow from the remaining 75 percent of the total drainage basin to the retention reservoir and the seepage loss from the reservoir, were incorporated in the model (equations 28-31) to determine the monthly rate of change of groundwater stage in the reservoir. If a positive change occurred, it was added to the previous month's end of month groundwater stage, and if a negative change occurred, it was subtracted from the previous month's groundwater stage. Eleven years (1963-1973 inclusive) of simulated routing was performed using: 1) the proposed 12 monthly stages developed by use of the operational model (Table 9, Column 2), and 2) the present regulatory stage of

3.5 feet MSL. The results of these routings are presented in Tables 10 and 11.

Observations

A. Present Regulation: Under the present regulation scheme the groundwater stage fluctuates from a minimum of 2.2 feet MSL to a maximum of 7.79 feet MSL (43rd value, Table 10, and Figure 9). Under the present discharge criteria of 3/4 inches per day it would take 17 days to bring the 7.79 groundwater stage to the 3.5 feet MSL regulatory stage.

B. Proposed Regulation: Under the proposed variable monthly regulatory stages (developed by use of the operational model), the groundwater stages also fluctuate between 2.2 feet MSL and 8.64 feet MSL, (43rd value, Table 11, and Figure 9) slightly higher by .85 feet under this scheme. The pumping days required to bring the 8.64 feet stage to the proposed regulatory stage is slightly higher than under the present regulatory scheme (19 days). The yearly average discharge from both regulation schemes are presented in Table 12.

TABLE 10 - ROUTING USING THE EXISTING MONTHLY REGULATORY SCHEME

Month	Rainfall Ret.Pond (Inches)	Evapor. Ret.Pond (Inches)	Inflo to the Ret.Pond (Inches)	Stage of the Ret.Pond (Feet)	Begin-ning Mo. Stage (Feet)	Regula-tory Stage (Feet)	End of Month Stage (Feet)	Days to Bring Stages to Reg.Stages	Outflow From the S-9 Basin (Inches)
1	0	0	0	0	2.20	0	2.20	0	0
2	1.20	2.53	1.36	.07	2.27	3.50	2.27	0	0
3	3.72	3.14	1.36	.23	2.50	3.50	2.50	0	0
4	.05	4.06	1.36	-0.15	2.34	3.50	2.34	0	0
5	.86	5.26	1.36	-0.19	2.16	3.50	2.16	0	0
6	4.06	6.51	1.36	-0.02	2.13	3.50	2.13	0	0
7	4.01	5.31	1.36	.07	2.20	3.50	2.20	0	0
8	2.86	6.01	.28	-0.35	1.85	3.50	1.85	0	0
9	4.63	5.99	2.23	.28	2.14	3.50	2.14	0	0
10	11.25	4.81	9.32	2.71	4.84	3.50	3.50	5.37	4.03
11	5.26	4.28	3.55	.81	4.31	3.50	3.50	3.24	2.43
12	2.09	3.13	.84	-0.04	3.46	3.50	3.46	0	0
13	4.01	2.40	3.05	.74	4.20	3.50	3.5	2.80	2.10
14	2.00	2.53	.99	.04	3.54	3.50	3.5	.18	.13
15	2.48	3.14	1.22	.09	3.59	3.50	3.5	.36	.27
16	.51	4.06	1.36	-0.12	3.38	3.50	3.38	0	0
17	5.11	5.26	1.89	.30	3.68	3.50	3.50	.74	.55
18	7.21	6.51	4.60	1.05	4.55	3.50	3.50	4.19	3.14
19	6.41	5.31	4.28	1.00	4.50	3.50	3.50	4.01	3.00
20	5.99	6.01	3.59	.74	4.24	3.50	3.50	2.94	2.21
21	6.90	5.99	4.51	1.04	4.54	3.50	3.50	4.17	3.13
22	4.24	4.81	2.31	.37	3.87	3.50	3.50	1.48	1.11
23	11.94	4.28	10.23	3.04	6.54	3.50	3.50	12.14	9.11
24	3.08	3.13	1.83	.29	3.79	3.50	3.50	1.18	.88
25	1.64	2.40	.68	-0.05	3.45	3.50	3.45	0	0
26	.33	2.53	1.36	-0.00	3.44	3.50	3.44	0	0
27	3.13	3.14	1.20	.14	3.58	3.50	3.50	.33	.25
28	.66	4.06	1.36	-0.10	3.40	3.50	3.40	0	0
29	.45	5.26	1.36	-0.22	3.18	3.50	3.18	0	0
30	.24	6.51	1.36	-0.34	2.83	3.50	2.83	0	0
31	8.72	5.31	2.56	.76	3.60	3.50	3.50	.39	.29
32	8.97	6.01	6.56	1.73	5.23	3.50	3.50	6.91	5.18
33	3.62	5.99	1.22	-0.05	3.45	3.50	3.45	0	0
34	7.12	4.81	5.19	1.33	4.78	3.50	3.50	5.11	3.83
35	7.34	4.28	5.63	1.50	5.00	3.50	3.50	6.01	4.51
36	1.87	3.13	.62	-0.11	3.39	3.50	3.39	0	0
37	.46	2.40	1.36	.02	3.41	3.50	3.41	0	0
38	3.41	2.53	1.90	.39	3.80	3.50	3.50	1.19	.89
39	3.21	3.14	1.95	.33	3.83	3.50	3.50	1.33	1.00
40	1.80	4.06	.17	-0.31	3.19	3.50	3.19	0	0
41	2.13	5.26	.03	-0.41	2.78	3.50	2.78	0	0
42	4.30	6.51	1.70	.08	2.86	3.50	2.86	0	0
43	18.20	5.31	16.08	4.93	7.79	3.50	3.50	17.18	12.89
44	7.47	6.01	5.07	1.23	4.73	3.50	3.50	4.92	3.69
45	6.18	5.99	3.78	.80	4.30	3.50	3.50	3.20	2.40
46	5.48	4.81	3.55	.78	4.28	3.50	3.50	3.13	2.35
47	7.38	4.28	5.67	1.52	5.02	3.50	3.50	6.06	4.55
48	1.37	3.13	.12	-0.28	3.22	3.50	3.22	0	0
49	.40	2.40	1.36	.01	3.24	3.50	3.24	0	0
50	2.48	2.53	.91	.06	3.30	3.50	3.30	0	0
51	1.71	3.14	.46	-0.16	3.14	3.50	3.14	0	0
52	1.11	4.06	1.36	-0.07	3.07	3.50	3.07	0	0
53	.15	5.26	1.36	-0.25	2.82	3.50	2.82	0	0
54	1.22	6.51	1.36	-0.26	2.56	3.50	2.56	0	0

55	15.21	5.31	9.78	3.11	5.67	3.50	3.50	8.69	6.52
56	3.56	6.01	1.16	-0.07	3.43	3.50	3.43	0	0
57	5.78	5.99	3.39	.67	4.10	3.50	3.50	2.38	1.79
58	8.44	4.81	6.51	1.77	5.27	3.50	3.50	7.08	5.31
59	10.01	4.28	8.30	2.39	5.89	3.50	3.50	9.57	7.18
60	1.88	3.13	.63	-0.11	3.39	3.50	3.39	0	0
61	2.21	2.40	1.25	.14	3.53	3.50	3.50	.12	.09
62	1.21	2.53	.20	-0.22	3.28	3.50	3.28	0	0
63	3.44	3.14	2.18	.41	3.69	3.50	3.50	.76	.57
64	1.15	4.06	1.36	-0.06	3.44	3.50	3.44	0	0
65	1.05	5.26	1.36	-0.17	3.27	3.50	3.27	0	0
66	17.27	6.51	13.14	4.02	7.29	3.50	3.50	15.15	11.36
67	15.55	5.31	13.42	4.05	7.55	3.50	3.50	16.19	12.14
68	5.12	6.01	2.72	.45	3.95	3.50	3.50	1.78	1.34
69	4.39	5.99	1.99	.20	3.70	3.50	3.50	.82	.61
70	4.89	4.81	2.96	.59	4.09	3.50	3.50	2.35	1.76
71	6.97	4.28	5.26	1.38	4.88	3.50	3.50	5.52	4.14
72	.82	3.13	1.36	-0.01	3.49	3.50	3.49	0	0
73	.02	2.40	1.36	-0.02	3.47	3.50	3.47	0	0
74	3.59	2.53	1.08	.20	3.67	3.50	3.50	.67	.50
75	2.29	3.14	.57	-0.09	3.41	3.50	3.41	0	0
76	8.66	4.06	1.54	.61	4.02	3.50	3.50	2.08	1.56
77	.17	5.26	1.49	-0.21	3.29	3.50	3.29	0	0
78	7.84	6.51	2.22	.51	3.79	3.50	3.50	1.18	.88
79	6.64	5.31	5.29	1.27	4.77	3.50	3.50	5.09	3.82
80	3.48	6.01	5.59	1.03	4.53	3.50	3.50	4.11	3.08
81	4.91	5.99	2.33	.33	3.83	3.50	3.50	1.33	1.00
82	6.21	4.81	3.86	.92	4.42	3.50	3.50	3.69	2.76
83	4.70	4.28	16.10	3.90	7.40	3.50	3.50	15.60	11.70
84	.15	3.13	1.19	-0.11	3.39	3.50	3.39	0	0
85	.21	2.40	1.36	-0.00	3.39	3.50	3.39	0	0
86	3.46	2.53	2.32	.50	3.89	3.50	3.50	1.54	1.16
87	1.82	3.14	1.03	-0.01	3.49	3.50	3.49	0	0
88	3.16	4.06	7.03	1.52	5.01	3.50	3.50	6.04	4.53
89	3.60	5.26	1.36	.04	3.54	3.50	3.50	.17	.12
90	4.83	6.51	3.30	.52	4.02	3.50	3.50	2.10	1.57
91	7.42	5.31	4.51	1.14	4.64	3.50	3.50	4.57	3.43
92	7.99	6.01	1.08	.27	3.77	3.50	3.50	1.10	.82
93	4.73	5.99	2.52	.37	3.87	3.50	3.50	1.46	1.10
94	5.79	4.81	4.28	.99	4.49	3.50	3.50	3.97	2.97
95	17.81	4.28	2.99	1.72	5.22	3.50	3.50	6.86	5.15
96	2.44	3.13	1.36	.12	3.62	3.50	3.50	.49	.37
97	.70	2.40	1.36	.04	3.54	3.50	3.50	.15	.11
98	.71	2.53	1.36	.03	3.53	3.50	3.50	.11	.09
99	.80	3.14	1.36	-0.01	3.49	3.50	3.49	0	0
100	.38	4.06	1.36	-0.13	3.36	3.50	3.36	0	0
101	.17	5.26	1.36	-0.24	3.11	3.50	3.11	0	0
102	3.91	6.51	1.36	-0.04	3.08	3.50	3.08	0	0
103	8.74	5.31	2.89	.85	3.93	3.50	3.50	1.70	1.28
104	6.72	6.01	4.32	.98	4.48	3.50	3.50	3.92	2.94
105	5.71	5.99	3.31	.64	4.14	3.50	3.50	2.58	1.93
106	7.26	4.81	5.33	1.38	4.88	3.50	3.50	5.51	4.13
107	8.43	4.28	6.72	1.87	5.37	3.50	3.50	7.46	5.60
108	2.20	3.13	.95	.00	3.50	3.50	3.50	.00	.00
109	2.10	2.40	1.14	.10	3.60	3.50	3.50	.40	.30
110	1.60	2.53	.59	-0.09	3.41	3.50	3.41	0	0
111	2.11	3.14	.85	-0.03	3.38	3.50	3.38	0	0
112	5.26	4.06	3.64	.85	4.23	3.50	3.50	2.91	2.18
113	2.82	5.26	.71	-0.19	3.31	3.50	3.31	0	0
114	8.31	6.51	5.71	1.42	4.73	3.50	3.50	4.92	3.69
115	11.10	5.31	8.98	2.57	6.07	3.50	3.50	10.27	7.70
116	10.25	6.01	7.84	2.15	5.65	3.50	3.50	8.61	6.46

117	7.10	5.99	4.71	1.11	4.61	3.50	3.50	4.44	3.33
118	5.22	4.81	3.29	.70	4.20	3.50	3.50	2.79	2.09
119	3.74	4.28	2.03	.30	3.80	3.50	3.50	1.21	.91
120	4.11	3.13	2.86	.64	4.14	3.50	3.50	2.55	1.91
121	2.53	2.40	1.57	.24	3.74	3.50	3.50	.97	.73
122	1.89	2.53	.88	.01	3.51	3.50	3.50	.03	.02
123	1.35	3.14	.09	-0.29	3.21	3.50	3.21	0	0
124	3.51	4.06	1.89	.27	3.48	3.50	3.48	0	0
125	.59	5.26	1.36	-0.21	3.27	3.50	3.27	0	0
126	3.47	6.51	1.36	-0.07	3.20	3.50	3.20	0	0
127	11.79	5.31	9.02	2.63	5.83	3.50	3.50	9.33	7.00
128	10.83	6.01	8.43	2.35	5.85	3.50	3.50	9.40	7.05
129	9.44	5.99	7.04	1.89	5.39	3.50	3.50	7.55	5.66
130	5.49	4.81	3.56	.79	4.29	3.50	3.50	3.15	2.36
131	3.97	4.28	2.26	.38	3.88	3.50	3.50	1.52	1.14
132	3.16	3.13	1.91	.32	3.82	3.50	3.50	1.28	.96
133	1.82	2.40	.86	.01	3.51	3.50	3.50	.03	.02

TABLE 11 - ROUTING USING THE PROPOSED MONTHLY REGULATORY SCHEME

Month	Rainfall Ret.Pond (Inches)	Evapora. Ret.Pond (Inches)	Inflow to the Ret.Pond (Inches)	Stage of the Ret.Pond (Feet)	Beginning Mo. Stage (Feet)	Regulatory Stage (Feet)	End of Month Stage (Feet)	Days to Bring Stages to Reg.Stages	Outflow From the S-9 Basin (Inches)
1	0	0	0	0	2.20	0	2.20	0	0
2	1.20	2.53	1.36	.07	2.27	5.19	2.27	0	0
3	3.72	3.14	1.36	.23	2.50	5.00	2.50	0	0
4	.05	4.06	1.36	-0.15	2.34	4.44	2.34	0	0
5	.86	5.26	1.36	-0.19	2.16	3.88	2.16	0	0
6	4.06	6.51	1.36	-0.02	2.13	3.71	2.13	0	0
7	4.01	5.31	1.36	.07	2.20	3.95	2.20	0	0
8	2.86	6.01	.28	-0.35	1.85	4.26	1.85	0	0
9	4.63	5.99	2.23	.28	2.14	4.50	2.14	0	0
10	11.25	4.81	9.32	2.71	4.84	4.93	4.84	0	0
11	5.26	4.28	3.55	.81	5.65	5.60	5.60	.21	.15
12	2.09	3.13	.84	-0.04	5.56	5.56	5.56	.02	.01
13	4.01	2.40	3.05	.74	6.30	5.31	5.31	3.95	2.96
14	2.00	2.53	.99	.04	5.35	5.19	5.19	.65	.49
15	2.48	3.14	1.22	.09	5.28	5.00	5.00	1.12	.84
16	.51	4.06	1.36	-0.12	4.88	4.44	4.44	1.78	1.33
17	5.11	5.26	1.89	.30	4.74	3.88	3.88	3.44	2.58
18	7.21	6.51	4.60	1.05	4.93	3.71	3.71	4.87	3.65
19	6.41	5.31	4.28	1.00	4.71	3.95	3.95	3.05	2.28
20	5.99	6.01	3.59	.74	4.69	4.26	4.26	1.70	1.28
21	6.90	5.99	4.51	1.04	5.30	4.50	4.50	3.21	2.41
22	4.24	4.81	2.31	.37	4.87	4.93	4.87	0	0
23	11.94	4.28	10.23	3.04	7.91	5.60	5.60	9.22	6.92
24	3.08	3.13	1.83	.29	5.89	5.56	5.56	1.33	1.00
25	1.64	2.40	.68	-0.05	5.51	5.31	5.31	.79	.59
26	.33	2.53	1.36	-0.00	5.31	5.19	5.19	.47	.35
27	3.13	3.14	1.20	.14	5.33	5.00	5.00	1.32	.99
28	.66	4.06	1.36	-0.10	4.90	4.44	4.44	1.83	1.37
29	.45	5.26	1.36	-0.22	4.22	3.88	3.88	1.36	1.02
30	.24	6.51	1.36	-0.34	3.54	3.71	3.54	0	0
31	8.72	5.31	2.56	.76	4.30	3.95	3.95	1.40	1.05
32	8.97	6.01	6.56	1.73	5.68	4.26	4.26	5.67	4.25
33	3.62	5.99	1.22	-0.05	4.21	4.50	4.21	0	0
34	7.12	4.81	5.19	1.33	5.54	4.93	4.93	2.43	1.82
35	7.34	4.28	5.63	1.50	6.43	5.60	5.60	3.33	2.50
36	1.87	3.13	.62	-0.11	5.49	5.56	5.49	0	0
37	.46	2.40	1.36	.02	5.51	5.31	5.31	.79	.60
38	3.41	2.53	1.90	.39	5.70	5.19	5.19	2.04	1.53
39	3.21	3.14	1.95	.33	5.52	5.00	5.00	2.09	1.57
40	1.80	4.06	.17	-0.31	4.69	4.44	4.44	1.02	.76
41	2.13	5.26	.03	-0.41	4.03	3.88	3.88	.59	.44
42	4.30	6.51	1.70	.08	3.96	3.71	3.71	1.00	.75
43	18.20	5.31	16.08	4.93	8.64	3.95	3.95	18.78	14.08
44	7.47	6.01	5.07	1.23	5.18	4.26	4.26	3.68	2.76
45	6.18	5.99	3.78	.80	5.06	4.50	4.50	2.24	1.68
46	5.48	4.81	3.55	.78	5.28	4.93	4.93	1.41	1.06
47	7.38	4.28	5.67	1.52	6.45	5.60	5.60	3.38	2.54
48	1.37	3.13	.12	-0.28	5.32	5.56	5.32	0	0
49	.40	2.40	1.36	.01	5.34	5.31	5.31	.11	.08
50	2.48	2.53	.91	.06	5.37	5.19	5.19	.73	.55
51	1.71	3.14	.46	-0.16	5.03	5.00	5.00	.10	.08
52	1.11	4.06	1.36	-0.07	4.93	4.44	4.44	1.98	1.48
53	.15	5.26	1.36	-0.25	4.19	3.88	3.88	1.26	.94
54	1.22	6.51	1.36	-0.26	3.62	3.71	3.62	0	0

55	15.21	5.31	9.78	3.11	6.73	3.95	3.95	11.11	8.34
56	3.56	6.01	1.16	-0.07	3.88	4.26	3.88	0	0
57	5.78	5.99	3.39	.67	4.55	4.50	4.50	.18	.14
58	8.44	4.81	6.51	1.77	6.27	4.93	4.93	5.36	4.02
59	10.01	4.28	8.30	2.39	7.32	5.60	5.60	6.89	5.17
60	1.88	3.13	.63	-0.11	5.49	5.56	5.49	0	0
61	2.21	2.40	1.25	.14	5.63	5.31	5.31	1.28	.96
62	1.21	2.53	.20	-0.22	5.09	5.19	5.09	0	0
63	3.44	3.14	2.18	.41	5.50	5.00	5.00	2.00	1.50
64	1.15	4.06	1.36	-0.06	4.94	4.44	4.44	1.99	1.49
65	1.05	5.26	1.36	-0.17	4.27	3.88	3.88	1.56	1.17
66	17.27	6.51	13.14	4.02	7.90	3.71	3.71	16.77	12.57
67	15.55	5.31	13.42	4.05	7.76	3.95	3.95	15.23	11.42
68	5.12	6.01	2.72	.45	4.40	4.26	4.26	.54	.41
69	4.39	5.99	1.99	.20	4.46	4.50	4.46	0	0
70	4.89	4.81	2.96	.59	5.05	4.93	4.93	.48	.36
71	6.97	4.28	5.26	1.38	6.31	5.60	5.60	2.84	2.13
72	.82	3.13	1.36	-0.01	5.59	5.56	5.56	.11	.08
73	.02	2.40	1.36	-0.02	5.54	5.31	5.31	.93	.70
74	3.59	2.53	1.08	.20	5.51	5.19	5.19	1.27	.96
75	2.29	3.14	.57	-0.09	5.10	5.00	5.00	.41	.31
76	8.66	4.06	1.54	.61	5.61	4.44	4.44	4.67	3.50
77	.17	5.26	1.49	-0.21	4.23	3.88	3.88	1.39	1.04
78	7.84	6.51	2.22	.51	4.39	3.71	3.71	2.70	2.03
79	6.64	5.31	5.29	1.27	4.98	3.95	3.95	4.13	3.10
80	3.48	6.01	5.59	1.03	4.98	4.26	4.26	2.87	2.15
81	4.91	5.99	2.33	.33	4.59	4.50	4.50	.37	.28
82	6.21	4.81	3.86	.92	5.42	4.93	4.93	1.97	1.47
83	4.70	4.28	16.10	3.90	8.83	5.60	5.60	12.92	9.69
84	.15	3.13	1.19	-0.11	5.49	5.56	5.49	0	0
85	.21	2.40	1.36	-0.00	5.49	5.31	5.31	.71	.53
86	3.46	2.53	2.32	.50	5.81	5.19	5.19	2.47	1.85
87	1.82	3.14	1.03	-0.01	5.18	5.00	5.00	.71	.53
88	3.16	4.06	7.03	1.52	6.52	4.44	4.44	8.33	6.25
89	3.60	5.26	1.36	.04	4.48	3.88	3.88	2.41	1.80
90	4.83	6.51	3.30	.52	4.40	3.71	3.71	2.78	2.08
91	7.42	5.31	4.51	1.14	4.85	3.95	3.95	3.61	2.71
92	7.99	6.01	1.08	.27	4.22	4.26	4.22	0	0
93	4.73	5.99	2.52	.37	4.59	4.50	4.50	.36	.27
94	5.79	4.81	4.28	.99	5.49	4.93	4.93	2.25	1.68
95	17.81	4.28	2.99	1.72	6.65	5.60	5.60	4.18	3.14
96	2.44	3.13	1.36	.12	5.72	5.56	5.56	.65	.49
97	.70	2.40	1.36	.04	5.60	5.31	5.31	1.15	.87
98	.71	2.53	1.36	.03	5.34	5.19	5.19	.59	.45
99	.80	3.14	1.36	-0.01	5.18	5.00	5.00	.70	.53
100	.38	4.06	1.36	-0.13	4.87	4.44	4.44	1.73	1.30
101	.17	5.26	1.36	-0.24	4.20	3.88	3.88	1.26	.95
102	3.91	6.51	1.36	-0.04	3.84	3.71	3.71	.53	.40
103	8.74	5.31	2.89	.85	4.56	3.95	3.95	2.43	1.82
104	6.72	6.01	4.32	.98	4.93	4.26	4.26	2.68	2.01
105	5.71	5.99	3.31	.64	4.90	4.50	4.50	1.62	1.21
106	7.26	4.81	5.33	1.38	5.88	4.93	4.93	3.79	2.84
107	8.43	4.28	6.72	1.87	6.80	5.60	5.60	4.78	3.59
108	2.20	3.13	.95	.00	5.60	5.56	5.56	.16	.12
109	2.10	2.40	1.14	.10	5.66	5.31	5.31	1.40	1.05
110	1.60	2.53	.59	-0.09	5.22	5.19	5.19	.12	.09
111	2.11	3.14	.85	-0.03	5.16	5.00	5.00	.63	.47
112	5.26	4.06	3.64	.85	5.85	4.44	4.44	5.64	4.23
113	2.82	5.26	.71	-0.19	4.25	3.88	3.88	1.50	1.12
114	8.31	6.51	5.71	1.42	5.30	3.71	3.71	6.35	4.76
115	11.10	5.31	8.98	2.57	6.28	3.95	3.95	9.31	6.98
116	10.25	6.01	7.84	2.15	6.10	4.26	4.26	7.37	5.53

117	7.10	5.99	4.71	1.11	5.37	4.50	4.50	3.48	2.61
118	5.22	4.81	3.29	.70	5.20	4.93	4.93	1.07	.80
119	3.74	4.28	2.03	.30	5.23	5.60	5.23	0	0
120	4.11	3.13	2.86	.64	5.87	5.56	5.56	1.24	.93
121	2.53	2.40	1.57	.24	5.80	5.31	5.31	1.97	1.48
122	1.89	2.53	.88	.01	5.32	5.19	5.19	.51	.38
123	1.35	3.14	.09	-0.29	4.90	5.00	4.90	0	0
124	3.51	4.06	1.89	.27	5.17	4.44	4.44	2.92	2.19
125	.59	5.26	1.36	-0.21	4.23	3.88	3.88	1.40	1.05
126	3.47	6.51	1.36	-0.07	3.81	3.71	3.71	.39	.29
127	11.79	5.31	9.02	2.63	6.34	3.95	3.95	9.58	7.18
128	10.83	6.01	8.43	2.35	6.30	4.26	4.26	8.16	6.12
129	9.44	5.99	7.04	1.89	6.15	4.50	4.50	6.59	4.94
130	5.49	4.81	3.56	.79	5.29	4.93	4.93	1.43	1.07
131	3.97	4.28	2.26	.38	5.31	5.60	5.31	0	0
132	3.16	3.13	1.91	.32	5.63	5.56	5.56	.28	.21
133	1.82	2.40	.86	.01	5.57	5.31	5.31	1.03	.77

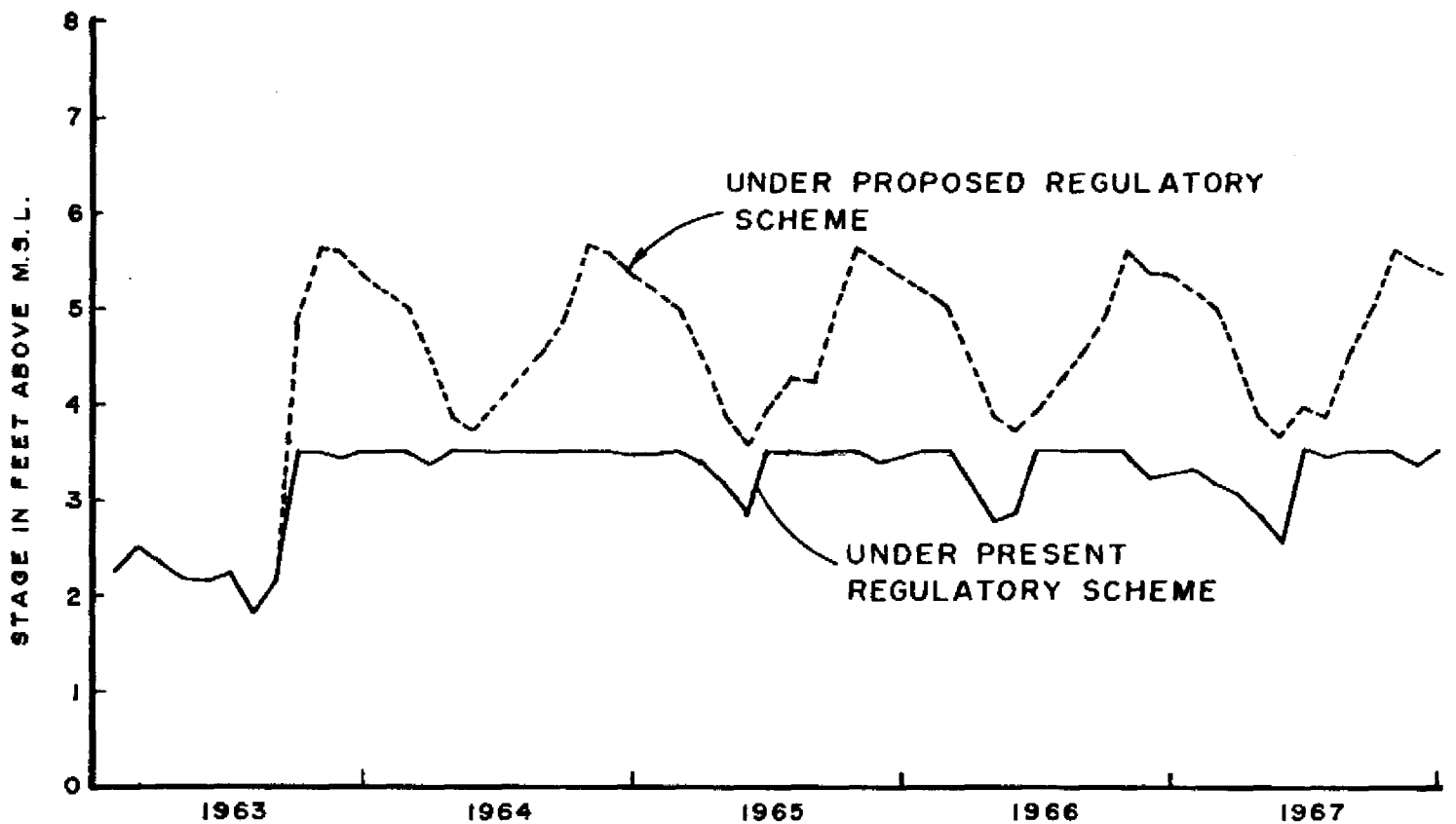

Figure 9 HYDROGRAPH OF MONTHLY WATER STAGES

SHEET 1 OF 2

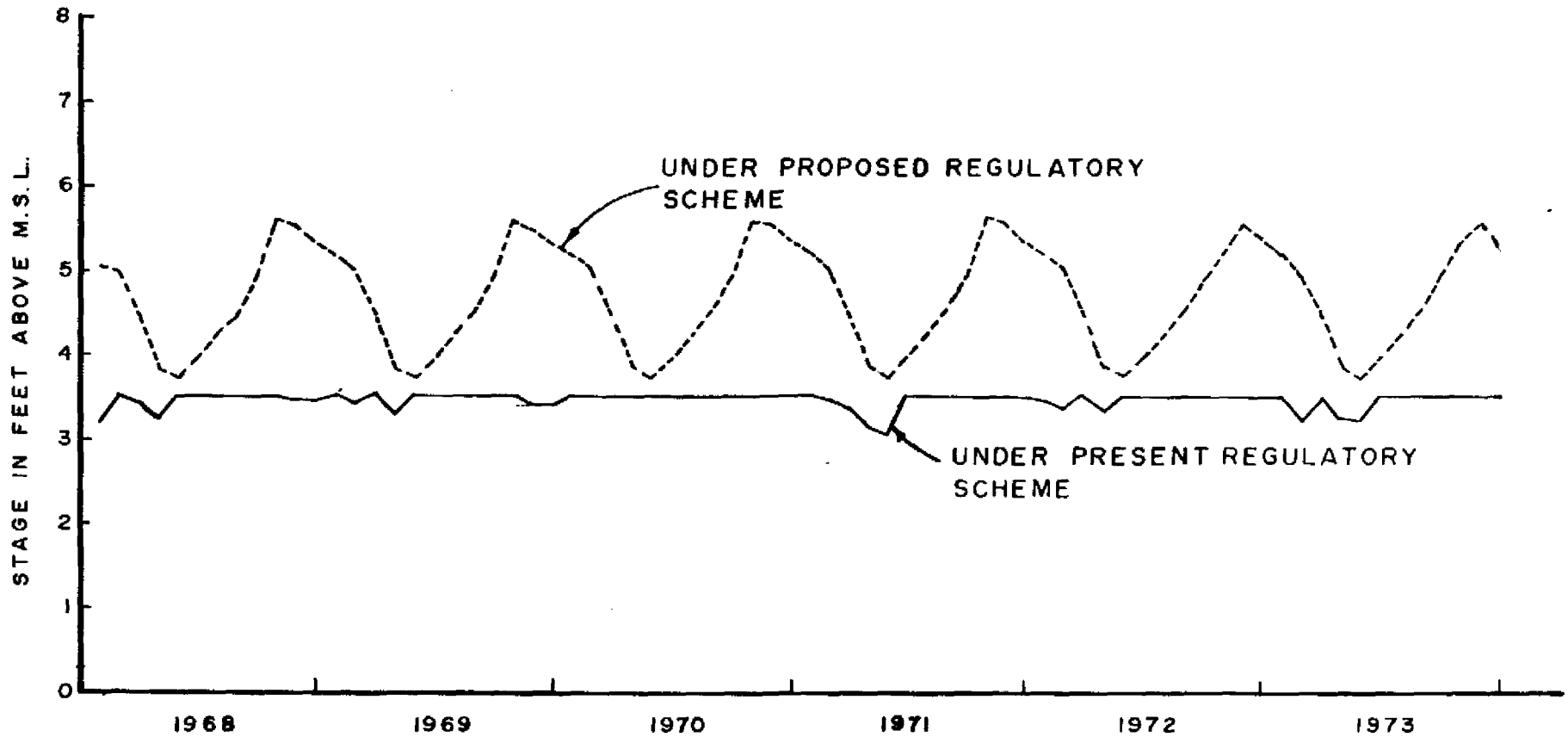

Figure 9 HYDROGRAPH OF MONTHLY WATER STAGES

SHEET 2 OF 2

TABLE 12 - ROUTED DISCHARGE FROM S-9 BASIN UNDER PRESENT AND PROPOSED REGULATORY SCHEMES (INCHES)

YEAR	PRESENT REGULATORY SCHEME (INCHES)	PROPOSED VARIABLE MONTHLY REGULATORY SCHEME (INCHES)
1963	8.56	3.12
1964	23.53	23.38
1965	14.06	13.95
1966	27.77	27.25
1967	20.89	21.68
1968	31.92	31.83
1969	25.30	25.06
1970	21.33	21.67
1971	16.27	16.27
1972	29.00	29.00
1973	24.21	24.20
AVERAGE	22.08	21.57

On the average, the discharge from the S-9 basin under the two regulatory schemes is almost the same. However, to raise the present regulatory stage to the proposed new stage, some discharge water was retained from the 1st year's discharge values. Once the proposed regulatory stage is reached the discharges under both the schemes are similar.

C. Under no external stress condition (municipal pumpage) imposed on the basin the proposed regulatory stage has the flexibility to store 1.19 feet of water in the 8,000 acre-foot retention basin. This amount, when prorated to the whole S-9 drainage basin, gives an additional recharge capability of 3.57 inches over the basin (1.19 X 12 X 8,000/32,000). Thus, an average stage of 4.69 MSL is maintained under the proposed schedule in lieu of the 3.5 MSL regulation. This "made" water comes from raising the groundwater stages slightly higher during the non-rainy period (Table 9). Groundwater stage hydrographs using both the present and the proposed schemes are presented in Figure 9. However, it should be stressed that under the proposed scheme

the regulatory stages are brought down to almost the present regulatory stages of 3.5 feet MSL during the rainy season. These two routing studies demonstrate that, in fact, the groundwater stages can be raised safely to proposed regulatory stages without causing any adverse impact in the S-9 basin if all the prestated criteria are met.

CONCLUSIONS

From this study it is concluded that several methods are available whereby an on-site retention basin can be sized which, with an acceptable degree of reliability, will perform in such fashion as to provide flood protection. However, to ensure the continued availability of storage capacity to maintain the designed degree of flood protection a retention basin operating rule must be devised. The simplest operating rule is a procedure which requires the retention basin to be drawn down to its base elevation as quickly as possible after every storm event regardless of the time of year. Such a procedure, on a long-term basis, will provide no increase in on-site water availability for beneficial use.

Therefore, it is a further conclusion of this study, that unless a rationally-based operating rule for the retention basin is developed and implemented no basis exists for claiming that long-term on-site water retention will result. The study demonstrates, however, that a methodology is available by which a retention area can be optimally sized and an operating rule developed which will result in satisfactory performance in terms of both flood protection and long-term, average on-site water retention. The methodology suggested herein is a linear decision rule model.

Another major conclusion of this study is that urbanization of a watershed alters the hydrology of the watershed. The principal expression of that alteration is an increase in watershed runoff which, in turn, is a consequence of the suppression of evapotranspiration losses due to the replacement of the natural pervious surface with impervious surfaces. The hydrologic accounting simulation model described in this study offers a method whereby runoff values for an urban watershed can be generated for use in conjunction with the linear decision model.

As an additional conclusion it is noted that for flood protection purposes the ability to draw down retention area stages to provide flood storage capacity is an important consideration. The speed with which this drawdown can be accomplished is, of course, a function of basin outlet capability; the greater the outlet capacity the more quickly can stages be lowered to prescribed operating levels after a storm event. The matter of outlet capability becomes a possibly critical consideration as "back-to-back" storm events occur on areas having limited outlet capability.

Finally, it is concluded that the generalized models developed and described in this study, with necessary modifications to suit local conditions and constraints, can be used in the evaluation of water management systems having on-site retention facilities. This methodology also has the potential for application to design and operation, as well as evaluation.

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