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AGRICULTURAL RESERVOIR STUDY

CENTRAL AND SOUTHERN FLORIDA FLOOD CONTROL DISTRICT

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Agricultural Reservoir Study

by

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CONTENTS

TABLES

ILLUSTRATIONS

ABSTRACT

A grove-reservoir system was studied basically by using a water budget analysis. The performance of the reservoir in such a system and its effectiveness in providing on-site supplemental irrigation water and in reducing the peak inflow rate to the primary receiving canal during substantial runoff events were investigated.

The system used in this study consists of a grove area of 874 acres and a reservoir of 160 acres. Analysis based on the gathered information indicates that the reservoir of the type and size under study did not provide a seasonal carried-over storage that alone met irrigation requirements. The groundwater storage supplemented by perimeter seepage was a major source which provided water to satisfy the demand. Increasing the storage capacity of this reservoir to a reasonable extent (50% by depth or 100% by area) will not increase substantially the water availability during the irrigation season. For instance, in the 1973-74 water year, the existing reservoir would be depleted of its storage by the end of January and the two hypothetical enlarged reservoirs by mid-February had the storage not been replenished by periodic pumpage from the grove's drainage system.

Nevertheless, the reservoir served its intended purpose adequately, from a grove management standpoint, by providing on-site temporary storage capacity and furnishing on-site supplemental irrigation water in each of the drainage-irrigation operational cycles in the irrigation season. It also provided additional storage capacity in the rainy season when the releases out of the District's project canal system, necessitated by excessive runoff, took place. This capacity amounts to about 25% of the basin yield in the 1973-74 water year and 35% in a normal rainfall year.

The effectiveness of the reservoir in reducing the peak inflow rate to the primary receiving canal was not substantial due primarily to the operational

measures employed. With a properly developed operation schedule there is no doubt that the reservoir has the potential to substantially reduce the peak inflow rate from its basin to the primary canal system.

GENERAL

The purpose of this study is to investigate the performance of an orange grove-reservoir system. The specific objectives are:

1. Determine the effectiveness of a citrus grove reservoir in providing on-site supplemental irrigation water.
2. Determine the effectiveness of a reservoir in reducing peak inflow rates to the receiving primary canal during substantial runoff events.

This report will address objectives and summarize the hydrologic data collected in the study area from July 1, 1973 to February 16, 1975. The report also presents a methodology that can be used to evaluate hydrologic elements in the water budget analysis of an agricultural area. The water budget presented herein will provide a comprehensive picture of the operations of a grove-reservoir system and also furnish data necessary to meet that which is required for water quality monitoring. The water quality considerations will be presented in another report.

The study area is located on Minute Maid Road adjacent on the north to Florida's Turnpike in northwestern St. Lucie County (see Figure 1) in what formerly was a part of the St. Johns River marsh. A larger reservoir under another ownership is located to the north and abuts the subject reservoir. The District's Canal 25 Extension serves the area for primary flood control purposes, and also to some degree, to supplement irrigation requirements.

The study area consists of a reservoir of 160 acres and an orange grove of 874 acres. Ground elevations in the grove area average about 22 feet above mean sea level with the bottom of the reservoir being somewhat higher at elevation 24 feet mean sea level. The upper soil zone in the study area is composed of sandy clay to a depth of 3 to 4 feet.

The inter-relationship between reservoir and grove and its connection with the hydrologic environment is depicted in Figure 2. In the system, the only conveyance designed to transfer water between the reservoir and grove is a pump station which can either pump for intake purposes or siphon for irrigation withdrawal requirements. In addition, there is a considerable amount of water that comes to the grove due to seepage. The grove's internal drainage-water delivery system also includes a main perimeter canal along with laterals which are spaced approximately 130 feet apart.

In this study the whole system is further divided into two sub-systems, namely the reservoir and the grove. Two separated water budgets were worked out for each of the sub-systems. Both the input and output waters were evaluated and then tabulated separately which serves to facilitate change in storage observations.

DATA COLLECTION

An inventory of the instruments which were used to collect the basic hydrologic data is presented in Table I. The original proposal called for the measurements of precipitation, pan evaporation, air temperature, pan water temperature, air movement, relative humidity, solar radiation, groundwater table fluctuation, surface water elevations in the reservoir and grove drainage system, and amounts of water pumped out of and siphoned into the grove area. The solar radiation and air movement were measured intermittently due to the faulty functions associated with the pyroheliograph and the anemometer. The remainder of the measurements were considered fairly good.

The measurement of solar radiation and air movement is essential in estimating reservoir evaporation by using climate parameters. Since the pan evaporation was measured, it can be used in turn to compute the solar radiation and the reservoir evaporation with the equation available.[1] The measurements at Vero Beach by the U.S. Weather Bureau were used to fill the missing records for air movement.

The hydrograph of daily mean water surface elevation in the reservoir together with the rainfall bargraph, and the hydrograph of daily mean water surface elevation in C-25 Extension are presented respectively in Figures 3 and 4. Figure 5 shows the hydrographs of daily mean water surface elevation measured from perimeter ditches and groundwater fluctuations measured from two observation wells in the grove.

The wells used to observe groundwater fluctuations penetrated a relatively impervious upper soil zone into a stratum which was of high permeability. This stratum behaved similar to a conduit which integrated the groundwater observation wells, the laterals and the perimeter canal into an interconnected

INSTALLATIONS FOR HYDROLOGIC DATA COLLECTION

Installation	Measurement
Surface Water Recorder 1	Water Surface Elevations at North Pump - Upstream
Surface Water Recorder 2	Water Surface Elevations at North Pump - Downstream
Surface Water Recorder 3	Reservoir Water Surface Elevation
Surface Water Recorder 4	Water Surface Elevation at South Pump - Upstream
Surface Water Recorder 5	Water Surface Elevation at South Pump - Downstream
Groundwater Recorder 1, 2	Groundwater Table in the Grove
Rain Gage	Rainfall
Evaporation Pan	USWB Class-A Pan Evaporation
Tempscribe	Pan Water Temperature
Hydrothermograph	Air Temperature and Relative Humidity
Tachometer 1	Operation at North Pump
Tachometer 2	Operation at South Pump
Anemometer	Air Movement
Pyroheliograph	Solar Radiation
Soil Moisture Indicators	Soil Moisture in the Grove

TABLE I

system. Figure 5 shows that when the surface water elevation was above the lateral bottoms (elevation 18 ± feet) that the hydrograph of water table fluctuations in the observation wells responded fairly close to the surface water fluctuations. As the surface water receded below the bottom of the laterals, the lag time of groundwater fluctuation to that of the surface water in the perimeter canal was increased.

RESERVOIR WATER BUDGET

The water balance equation for the reservoir system as shown in Figure 2 can be written as:

$$(P + Q_p) - (Q_s + Q_c + SP + E_p) = \Delta S_R \qquad (1)$$

in which P = precipitation, Q_p = pumped inflow, Q_s = siphoned outflow, Q_c = overflow through a culvert, SP = seepage outflow, E_p = evaporation from reservoir water surface, and ΔS_R = change in reservoir storage. The units used in Equation 1 and all the subsequent water budget equations are in inches.

The amount of water pumped from the drainage ditch into the reservoir through a pipe of 3-foot I.D. can be evaluated from a pump head-capacity curve (see Figure 6) with the measured differential head, engine RPM, and period of pumpage. The head-capacity curve was calibrated by employing the water balance equation in the following form:

$$Q_p = Q_s + Q_c + SP + E_p + \Delta S_R - P \qquad (2)$$

During the period of pumpage $Q_s = 0$. Q_c, E_p, ΔS_R and P were measured. The seepage, SP, which is in a smaller order of magnitude than the amount of water being pumped, can be estimated from pump-free and siphon-free periods.

The Bernoulli Equation for a steady flow through a closed conduit due to differential head can be written in the following form to determine the flow rate due to siphoning:

$$\Delta H = C \frac{V^2}{2g} \qquad (3)$$

In the equation, ΔH = differential head in feet, V = average velocity in the pipe in feet per second, g = gravitational acceleration in feet/sec^2, and C = lumped-sum loss coefficient due to entrance, exit and pump blades. The coefficient C was estimated as being equal to 18.8 by using a set of data from flow measurements and checked with a water balance equation similar to Equation 2.

The final equation used for computing the flow rate in cfs, is:

$$Q_s = 0.325 \ A \sqrt{g \Delta H} \tag{4}$$

in which A = cross sectional area of the pipe in square feet. The pumpage into or siphoning out of the reservoir can also be determined accurately with Equation 2. This method was used almost exclusively in the reservoir water budget analysis.

A culvert with a stop-log riser was installed in the reservoir to regulate the reservoir elevations. The stop-log riser consists of two sets of 2-inch by 6-inch by 3-foot boards which form a weir 5.5 feet in length. The overflow rate was evaluted by using a sharp crested weir formula. The weir coefficient was determined from the calculation of rate stage decrease right after the lowering of the weir crest. The equation is:

$$Q_c = C \ L \ \Delta h^{3/2} \tag{5}$$

in which Q_c = overflow rate in cfs, L = length of weir in feet, Δh = head above crest of weir in feet, and C is the weir coefficient which was determined to be equal to 3.02. The crest of the weir was usually lowered by eleven inches before the rainy season and restored to a normal elevation of 27.4 feet above mean sea level shortly before the irrigation season.

No measurements were conducted in this project to provide data for the direct determination of reservoir seepage rates. The total seepage rate in inches per day, SP, was computed by using the following water balance equation after every entry except SP had been evaluated.

$$SP = P + Q_p - Q_s - Q_c - E_p - \Delta S_R \tag{6}$$

The total monthly seepage loss computed by Equation 6 is shown in Table II and has values ranging from -1.19 to 15.44 inches. The minus value is due to the offsetting seepage inflow from the adjacent reservoir.

WATER LOSSES FROM RESERVOIR DUE TO SEEPAGE

	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1973												
Total, inches							6.56	.82	-1.19	6.66	6.75	11.84
cfs/mile							.95	.12	-.17	.97	.98	1.72
Mean Stage, feet							26.02	26.46	26.41	26.09	25.83	26.62
1974												
Total, inches	10.44	8.37	7.73	7.09	5.61	11.94	8.97	7.39	7.32	10.35	10.31	10.17
cfs/mile	1.51	1.21	1.12	1.03	.81	1.73	1.30	1.07	1.06	1.50	1.49	1.47
Mean Stage, feet	26.65	26.18	25.55	25.22	24.48	26.33	26.77	26.92	26.68	26.63	26.50	26.96
1975												
Total, inches	15.44											
cfs/mile	2.24											
Mean Stage, feet	26.72											

Note: Seepage through a perimeter of 1.5 miles being assumed.

TABLE II

The maximum seepage of 15.44 inches is equivalent to an out-seepage rate of 2.24 cfs per mile through the entire reservoir periphery, excluding that side which abuts the adjacent reservoir. This peak rate occurred in January, 1975 when the reservoir had a relatively high monthly mean stage of 26.72 feet msl. The seepage rate during the study period varies from (-) 0.17 cfs per mile to a high of 2.24 cfs per mile. This range is considered acceptable in terms of order of magnitude relative to several other measurements made in Central and Southern Florida.[2,3]

The daily evaporation from the reservoir water surface was estimated by using an evaporation formula developed from the U.S. Weather Bureau lake evaporation nomograph.[1,4] This formula relates the lake evaporation to four key factors in climate (air temperature, wind movement, dew temperature, and USWB Class A pan evaporation) and is expressed:

$$E_p = 0.7\,\frac{(Q_n\,\Delta + E_a\gamma)}{\Delta + \gamma} \tag{7}$$

where Q_n is net radiation exchange, Δ is slope of saturation vapor pressure curve as a function of air temperature, γ = 0.0105 inch Hg/^{o}F, and E_a is the pan evaporation assuming a pan water temperature equal to the air temperature. $Q_n\Delta$ can be computed from the pan evaporation by using the following formula:

$$Q_n\Delta = \frac{EP\ (\Delta + \gamma_p) - E_a\ \gamma_p}{\Delta} \tag{8}$$

where EP is the pan evaporation rate and γ_p = 0.025 in Hg/^{o}F.

Δ and E_a in the above equations can be determined by using the following equations:

$$\Delta = 0.0328\ (0.0041\ T_a + 0.676)^7 - 0.000019$$

and

$$E_a = (e_s - e_a)^{0.88}\ \ (0.37 + 0.0041W) \tag{9}$$

in which T_a = air temperature, W = wind movement in miles per day and

$$e_s - e_a = (0.0041\ T_a + 0.676)^8 - (0.0041\ T_a + 0.676)^8 - 0.000019\ (T_a - T_d) \qquad (10)$$

where T_d = dew temperature.

If the solar radiation, R, (in langleys per day) is measured, the following equations can be used to determine the lake evaporation.

$$E_p = \{e^{(T_a - 212)\ (0.1024 - 0.01066\ \ln R)} - 0.0001 + 0.0105\ (e_s - e_a)^{0.88}$$
$$(0.37 + 0.0041\ W)\} \times \{0.015 + (T_a + 398.36)^{-2}\ (6.8554 \times 10^{10})$$
$$\times\ e^{-7482.6/(T_a + 398.36)}\}^{-1} \qquad (11)$$

where e is the Napierian base.

Equation 7 and associated Equations 8, 9 and 10 were used to estimate daily evaporation in this study. The pan coefficients were then evaluated for individual months and are listed in Table III. The average pan coefficient for the water year 1973-1974 is 0.76. This value can be compared with 0.70 (an average value that has been developed for practical purposes from limited available data[5]) and the 0.80 value developed by the Corps of Engineers for Lake Okeechobee.

The change in reservoir storage, ΔS_R was readily determined from the stage variation. It is to be noted that the reservoir is assumed to have a uniform surface area regardless of the change of water surface elevation. Generally, the range of water surface elevation changes was relatively small. However, a special consideration has been given to the storage estimation when the water surface elevation was below 24.5 feet msl, since below this stage the reservoir water surface area is greatly reduced. Under this condition the pumped inflow and siphoned outflow were evaluated, together with estimated seepage and evaporation, in order to obtain storage change.

The summary of monthly water budgets for the period of study from

PAN COEFFICIENTS

RESERVOIR

YEAR	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1973							0.75	0.74	0.73	0.75	0.78	0.77
1974	0.75	0.78	0.80	0.81	0.76	0.77	0.73	0.74	0.73	0.73	0.70	0.72
1975	0.70											

GROVE

YEAR	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1973							0.76	0.73	0.73	0.67	0.66	0.61
1974	0.64	0.60	0.65	0.66	0.70	0.61	0.76	0.76	0.72	0.65	0.64	0.65
1975	0.62											

TABLE III

July 1973 to January 1975 is presented numerically in Table IV and graphically in Figure 7.

RESERVOIR MONTHLY WATER BUDGET IN INCHES

	Precip-itation	Pumped from Grove	Total INFLOW	Siphoned to Grove	Losses: Through Culvert	Losses: Seepage	Losses: ET	Total OUTFLOW	Storage Change	Reservoir Stage
1973										
July	7.83	0	7.83	0	1.78	6.56	3.45	11.79	-3.96	26.02
Aug.	8.33	32.18	40.51	0	26.68	.82	3.29	30.79	9.72	26.46
Sept.	4.13	15.91	20.04	0	21.10	-1.19	3.25	23.16	-3.12	26.41
Oct.	6.07	1.67	7.74	0	4.59	6.66	3.33	14.58	-6.84	26.09
Nov.	1.04	10.12	11.16	6.39	0	6.75	3.42	16.56	-5.40	25.83
Dec.	1.29	38.80	40.09	5.09	0	11.84	2.04	18.97	21.12	26.62
Total	28.69	98.68	127.37	11.48	54.15	31.45	18.78	115.85	11.52	
1974										
Jan.	0.86	4.83	5.69	5.08	0	10.44	2.29	17.81	-12.12	26.95
Feb.	1.13	12.79	13.92	3.04	0	8.37	3.23	14.64	-0.72	26.18
Mar.	0.48	10.25	10.73	9.77	0	7.73	5.23	22.73	-12.00	25.55
Apr.	1.42	21.91	23.33	18.45	0	7.09	5.35	30.89	-7.56	25.22
May	3.11	16.24	19.35	7.70	0	5.61	5.56	18.87	0.48	24.48
June	18.64	37.40	56.04	3.03	2.47	11.94	3.92	21.36	34.68	26.33
July	10.86	14.45	25.31	0	16.50	8.97	2.73	28.19	-2.88	26.77
Aug.	10.61	20.02	30.63	0	25.88	7.39	3.24	36.51	-5.88	26.92
Sept.	6.14	13.20	19.34	0	10.01	7.32	3.21	20.54	-1.20	26.68
Oct.	2.56	22.33	24.89	3.67	7.06	10.35	3.33	24.41	0.48	26.63
Nov.	2.38	19.63	22.01	4.37	0.85	10.31	2.16	17.69	4.32	26.50
Dec.	1.32	16.04	17.36	3.65	0.32	10.17	1.66	15.80	1.56	26.96
Yearly Total	59.51	209.09	268.60	58.76	63.09	105.69	41.91	269.44	-.84	26.26
1975										
Jan.	0.14	17.81	17.95	4.25	0.89	15.44	2.17	22.75	-4.80	26.32
Sum	88.34	325.58	413.92	74.49	118.13	152.58	62.86	408.04	5.88	

TABLE IV

WATER BUDGET FOR GROVE AREA

The water balance equation for the grove system can be written as:

$$(P + Q_s + Q_s' + S_e) - (Q_p + Q_p' + ET) = \Delta S_o + SM \quad (12)$$

in which P = precipitation, Q_s and Q_s' are respectively the siphoned inflows from the reservoir and from C-25 Extension, S_e = seepage inflow, Q_p = the pumped outflow to reservoir, Q_p' = the pumped outflow to C-25 Extension, ET = evapotranspiration, ΔS_o = storage increase in the groundwater aquifer and the grove's surface water system (drainage system), and SM = increase in soil moisture content. It was estimated that the drainage system has a total water surface area of twelve acres.

The siphoned inflow from C-25 Extension (Q_s') through the south pump was calculated by using the same formula that has been developed for evaluating Q_s, primarily due to the nearly identical set-up in the two pump stations. To estimate pumped outflow to C-25 (Q_p'), head-capacity curves derived from a measured flow rate in the drainage ditch at the inlet sections along with the head-capacity curves developed for the reservoir (or north) pump were integrated. The resulting head-capacity curves and a measured point are shown in Figure 8.

Several methods have been investigated in order to make a reasonably good estimation of evapotranspiration rates in the grove area. The method finally adopted was the multiple correlation method developed by Christiansen[6], primarily due to its good estimation results in some experimental areas, and also due to the type of data available in the subject study area.

The method estimates evaporation by using the following regression equation:

$$ET = C\ E_p\ C_T\ C_W\ C_H\ C_S \quad (13)$$

in which E_p = Class A pan evaporation.

$$C_T = 0.670 + 0.476\ (T/T_o) - 0.146\ (T/T_o)^2$$

where T is mean air temperature and T_o = 68 degrees Farenheit,

$$C_W = 1.189 - 0.240\ (W/W_o) + 0.051\ (W/W_o)^2$$

where W is the mean air velocity 2 meters above ground level in miles per day and W_o = 100 miles per day,

$$C_H = 0.499 + 0.620\ (H_m/H_{m_o}) - 0.119\ (H_m/H_{m_o})^2$$

where H_m is mean relative humidity and H_{m_o} = 60 percent, and

$$C_S = 0.904 + 0.008\ (S/S_o) + 0.088\ (S/S_o)^2$$

where S is the percent of possible sunshine and S_o = 80 percent. The constant C was revised due to the geographic location and land use applicable to the study area.

A seasonal variation coefficient, C_g, was added to Equation 13 to reflect the variation of potential consumptive use of the citrus over the year. C_s was dropped because the required data was not available. The seasonal variation coefficients were developed from the growth stage coefficients listed for use in the Blaney-Criddle Formula[8] and are listed below.

JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
0.92	0.97	0.99	1.02	1.04	1.04	1.04	1.04	1.02	0.99	0.98	0.94

The resulting equation is:

$$ET = C\ E_p\ C_T\ C_W\ C_H\ C_g \tag{14}$$

in which C = 0.55, a constant which was derived from the annual ET of a citrus grove. The annual ET was estimated on the basis of Koo's study[7] and the Blaney-Criddle Formula. The pan coefficients resulting from this analysis are listed in Table III along with the pan coefficients for the reservoir.

The changes in groundwater storage and surface water in the drainage system were evaluated separately and then combined. The soil of the study area is mainly composed of relatively impervious materials. It ranges from sandy clay for the top few feet, to sandy clay mixed with shell below four feet ground level. Several undisturbed soil samples were collected for determining the soil porosity and storage coefficient.

The laboratory test shows that these soil samples have a porosity of about 33% and 35% and a relatively small value of specific yield which is difficult to detect with a reasonable accuracy in the laboratory. Accordingly, it was felt that it would be more realistic to determine the amount of water released from groundwater storage due to a unit depth of drawdown by virtue of the water balance equation.

During a precipitation-free period, it was assumed that the moisture content in the upper soil zone had a negligible influence on that in the lower soil zone where the groundwater table fluctuates. The seepage flow during the same period was evaluated by accounting for the storage change, and amount of pumping and siphoning, which can be expressed as follows:

$$S_e = \Delta S_0 + Q_p - Q_s$$

The equation can be reduced to the following simplified form if a pumpage-free and siphon-free period is selected:

$$S_e = \Delta S_W + \Delta S_G \qquad (15)$$

in which ΔS_W = change in storage in the drainage system and ΔS_G = change in groundwater storage.

The storage coefficient for determining ΔS_G was derived by using Equation 15 and applying an iterative technique. The procedure for such computation is outlined as follows:

1. Assign a best estimated storage coefficient and use it to compute ΔS_G.
2. Compute seepage inflow for the selected rainfall-free, pumpage-free and siphon-free periods by using Equation 15.

3. Use seepage flow obtained from Step 2 to compute the regression coefficients in a regression equation for daily seepage flow (will be discussed later).

4. Compute seepage flow for entire study period with regression equation and determine daily soil moisture contents with Equation 12.

5. Integrate the moisture contents over a year which begins and ends with almost the same soil moisture contents.

6. Assign a new storage coefficient and repeat Steps 2 to 5 if the integrated soil moisture is not within a practical limit.

Following the above procedure it was estimated that 0.04 foot of free water was released with a foot drawdown in the groundwater table.

The monthly water budget with entries evaluated on the basis of the methodology discussed is tabulated in Table V. The mass curves for monthly rainfall, seepage, and evapotranspiration are plotted in Figure 9.

GROVE MONTHLY WATER BUDGET IN INCHES

	Precip-itation	Siphon-ed from Resv.	Siphon-ed from C-25	Seepage In	INFLOW Total	Soil Moisture Change	Pumped to res-ervoir	Pumped to C-25	ET	OUTFLOW Total	Change in Storage	Ground Water Table	Surface Water Elev.
1973													
July	7.83	0	0	4.51	12.34	1.40	0	7.37	3.48	10.85	0.09	17.79	17.57
Aug.	8.33	0	0	4.20	12.53	2.08	5.88	1.51	3.20	10.59	-0.14	17.94	17.67
Sept.	4.13	0	0	3.73	7.86	-0.93	2.90	2.33	3.24	8.47	0.32	17.74	17.60
Oct.	6.07	0	0	2.69	8.76	0.08	0.31	5.35	2.97	8.63	0.05	18.09	17.94
Nov.	1.04	1.17	0.29	1.24	3.74	-1.66	1.85	0	2.88	4.73	0.67	18.48	18.52
Dec.	1.29	0.93	0.51	2.75	5.48	-2.58	7.08	0	1.60	8.68	-0.63	18.42	18.38
Total	28.69	2.10	0.80	19.12	50.71	-1.61	18.02	16.56	17.37	51.95	0.36		
1974													
Jan.	0.86	0.92	0	1.15	2.93	-1.74	0.88	1.22	1.97	4.07	0.60	18.42	18.45
Feb.	1.13	0.56	0	0.92	2.61	-2.08	2.33	0.40	2.45	5.18	-0.49	18.13	18.05
Mar.	0.48	1.78	0.21	1.24	3.71	-2.41	1.86	0	4.24	6.10	0.02	18.14	17.96
Apr.	1.42	3.37	1.37	1.37	7.53	-1.10	4.01	0	4.36	8.37	0.26	17.94	17.94
May	3.11	1.41	0	1.16	5.68	-1.62	2.98	0	4.92	7.90	-0.60	17.91	17.71
June	18.64	0.55	0	3.17	22.36	4.95	6.77	7.37	3.08	17.22	0.19	18.26	17.91
July	10.86	0	0	4.20	15.06	1.21	2.67	8.40	2.85	13.92	-0.07	18.07	17.74
Aug.	10.61	0	0	3.65	14.26	2.28	3.65	5.15	3.32	12.12	-0.14	18.36	18.10
Sept.	6.14	0	0	3.10	9.24	0.89	2.42	2.71	3.14	8.27	0.07	18.24	18.12
Oct.	2.56	0.67	0	3.70	6.93	0.08	3.94	0.03	2.92	6.89	-0.04	17.85	17.76
Nov.	2.38	0.80	0	4.78	7.96	2.55	3.59	0	1.98	5.57	-0.16	17.78	17.71
Dec.	1.32	0.67	0	5.46	7.45	1.32	2.95	1.56	1.50	6.01	0.10	17.84	17.90
Total	59.51	10.73	1.58	33.90	105.72	4.33	38.07	26.84	36.73	101.64	-0.26	18.08	17.95
1975													
Jan.	0.14	0.77	0	4.08	4.99	-0.04	3.27	0	1.92	5.19	-0.17	17.94	17.94

NOTE:
Dry Land 862 acres
Ditches 12 acres

TABLE V

SEEPAGE INTO GROVE AREA

The evaluation of seepage inflow to the grove area is essential in determining the soil moisture variation and the storage coefficient and, therefore, will be discussed in detail herein. The primary factors (aside from the aquifer characteristics) which control the seepage are the surface water elevations in the grove's drainage system, in the reservoir, in C-25 Extension, and the groundwater table in the areas adjacent to the study area. Of these factors, the surface water elevations were gaged while the groundwater elevations in the areas adjacent to the subject grove were not. Since the neighboring areas are developed for citrus, it can be assumed that the pattern of groundwater regulation is similar to that being followed in the study area. The pattern, generally speaking, is to maintain a lower groundwater table in the wet season than that in the dry season.

Accordingly, the seepage-into-grove can be related to the pertinent factors in the following function:

$$S_e = f(H_R, H_C, H_W, C) \tag{16}$$

in which H_R is the water surface elevation in the reservoir, H_C the water surface elevation in C-25 Extension, H_W the water surface elevations in the grove drainage system, and C a parameter reflecting the seasonal variation of the seepage pattern due to groundwater table fluctuations.

The daily seepage inflow (negative for outflow) during the rain-free, pumpage-free and siphon-free periods can be computed from Equation 15 with assigned storage coefficients. The relationship between seepage flow so obtained and the independent variables cited in Equation 16 were studied with a multivariate linear correlation analysis.

Based on the principles of the Darcy Formula, seepage flow from the reservoir and C-25 Extension which enters the grove area can be assumed directly proportional to the surface elevation differentials between these water bodies and the grove's drainage system.

Let S_e be the seepage in inches per day, x_1 the differential head between reservoir and the drainage ditch, and x_2 the differential head between C-25 Extension and the drainage ditch. The resulting equation for seepage flow is then:

$$S_e = a + b_1 x_1 + b_2 x_2 \qquad (17)$$

The computed regression constants and regression coefficients for three intervals per year are listed in Table VI. The breakdown of three intervals is a result of the data analysis that reflects seasonal variation of the seepage flow.

The estimated seepage will have a standard error of about 0.014 to 0.04 inches per day. On the average about one out of three estimates will have errors greater than 0.027 and one out of 20 will have errors greater than 0.054. The magnitude of these errors are considered to be acceptable.

COEFFICIENTS FOR REGRESSION EQUATION

$$S_e = a + b_1 x_1 + b_2 x_2$$

Applicable Interval	Regression Constant a	Regression Coefficients b_1	b_2	Adjusted Correlation Coefficient $\bar{R}$	Standard Error Of Estimate $\bar{\sigma}$
7-1-73 to 12-15-73 & 6-2-74 to 11-30-74	-0.459	0.057	0.033	0.726	0.040
12-16-73 to 3-2-74 & 12-1-74 to 2-18-75	-1.168	0.148	0	0.916	0.036
3-3-74 to 6-1-74	-0.098	0.017	0.020	0.806	0.014

TABLE VI

IRRIGATION

Supplemental irrigation applications at the grove commence some time following the end of the rainy season when the citrus trees begin to show signs of wilting. Irrigation is normally performed following a cycle which consists of three stages of operation; namely: pumping, siphoning, and spray irrigation. The period of each 3-stage cycle is normally 14 days.

The first stage calls for pumping water from the grove's surface water system into the reservoir. This pumping terminates when the water level in the grove's surface water system reaches a pre-set elevation which produces a corresponding groundwater table elevation favorable to that which will stimulate citrus root system development. The second stage of the cycle starts two days before scheduled irrigation is to take place and consists of approximately 14 hours of siphoning from the reservoir, and occasionally from C-25 Extension, into the grove's surface water system. During the third stage, supplemental water is applied to the citrus trees for five consecutive days utilizing eight gun-type sprayers pumping from the grove's surface water system at a rate of 740 gallons per minute per sprayer. Under average conditions, each irrigation cycle applies 1 1/3 inches of water on each acre of the grove.

The irrigation histogram is plotted in Figure 5 for ease of comparison with water surface elevations in the drainage system. It can be seen from this figure that each of the irrigation cycles was reflected by a comparable change in the water surface elevation of the drainage system.

The monthly irrigation in inches during the period of study is tabulated in Table VII. Also presented in the same table are the irrigation withdrawals

IRRIGATION TO GROVE IN INCHES

Month	Irrigation	Irrigation Withdrawal From C-25	From Reservoir	Total
1973				
July	0	0	0	0
August	0	0	0	0
September	0	0	0	0
October	0	0	0	0
November	2.40	0.29	1.17	1.46
December	0.80	0.51	0.93	1.44
Total	3.20	0.80	2.10	2.90
1974				
January	2.40	0	0.92	0.92
February	1.60	0	0.56	0.56
March	2.67	0.21	1.78	1.99
April	2.13	1.37	3.37	4.74
May	1.87	0	1.41	1.41
June	0	0	0.55	0.55
July	0	0	0	0
August	0	0	0	0
September	0	0	0	0
October	0.27	0	0.67	0.67
November	1.07	0	0.80	0.80
December	0.53	0	0.67	0.67
Total	12.54	1.58	10.73	12.31
1975				
January	1.07	0	0.77	0.77

TABLE VII

from the reservoir and C-25 Extension. The irrigation season started at the end of October in 1973 and at mid-October in 1974. The 1974-75 irrigation season can be considered a drier than normal season in which about 13.87 inches of water was applied to the grove. It can be seen in the table that the irrigation withdrawals were 2.38 inches from C-25 Extension and 10.69 inches from the reservoir during the 1973-74 irrigation season.

The reservoir under this study serves two purposes as related to irrigation. First, it conserves the rainy season's excessive runoff and carries it over for irrigation season use. Second, it serves as a detention storage area for temporarily holding the water which is withdrawn from the grove's surface water system at the beginning of each irrigation cycle.

Observation of the reservoir operations reveals that temporary storage accounts for most of the irrigation water. As noted previously, this temporary storage water is withdrawn from the grove's surface water systems which in turn draws from adjacent surface water bodies and groundwater storage.

DISCUSSION

The reservoir-grove system as a whole is further illustrated by a flow diagram as shown in Figure 10. In addition, Table VIII presents an overall study area water budget for the 1973-74 water year (Oct. 1 - Sept. 30). Of the 61.65 inches of rainfall that fell during the year (which is 4.15 inches above the normal annual rainfall of 57.50 inches) 63% was lost to evaporation and evapotranspiration in the grove and reservoir, and 37% was net discharge to C-25.

As noted in Table VIII, during the 1973-74 water year, the total inflow into the study area was 9.2 inches. Of this amount, 70% (from perimeter seepage) was discharged to C-25 Extension while the remaining 30% (2.58 inches) was actually used to help meet irrigation demands. The latter amount was taken during periods when withdrawals were competitive amoung users.

The results of this study pertaining to the specific objectives defined earlier are discussed in greater detail herein. Objective One will be discussed under sub-section "Irrigation Season Evaluation", whereas Objective Two will be discussed under "Rainy Season Evaluation".

1. Irrigation Season Evaluation

The supplemental water availability in the reservoir at the end of the month was presented numerically in Table IX and graphically in Figure 11. Curve A in Figure 11 shows that the reservoir under its 1973-74 irrigation season operation method, in which the reservoir storage was replenished by periodic pumpage from the grove's drainage system, was deficit in storage by 175 acre-feet to sustain the irrigation season demand. The same reservoir, if not replenished by such periodic pumpage, would deplete its storage completely in about a month following the start of the irrigation season.

WATER BUDGET IN INCHES OVER STUDY AREA OF 1034 ACRES

Month	Rainfall	Loss To Atmosphere	OUTFLOW Pumpage	OUTFLOW Overflow	OUTFLOW Total	INFLOW Siphoning	INFLOW Seepage	INFLOW Total	Net Gain Due to Exchange
<u>1973</u>									
Oct.	6.07	3.04	4.55	0.71	5.26	0	1.26	1.26	-4.00
Nov.	1.04	2.98	0	0	0	0.25	0	0.25	0.25
Dec.	1.29	1.67	0	0	0	0.43	0.51	0.94	0.94
<u>1974</u>									
Jan.	0.86	2.03	1.04	0	1.04	0	-0.65	-0.65	-1.69
Feb.	1.13	2.58	0.34	0	0.34	0	-0.52	-0.52	-0.86
Mar.	0.48	4.41	0	0	0	0.18	-0.15	0.03	0.03
Apr.	1.42	4.53	0	0	0	1.17	0.07	1.24	1.24
May	3.11	5.04	0	0	0	0	0.12	0.12	0.12
June	18.64	3.22	6.27	0.38	6.65	0	0.85	0.85	-5.80
July	10.86	2.85	7.14	2.55	9.69	0	2.21	2.21	-7.48
Aug.	10.61	3.32	4.38	4.00	8.38	0	1.96	1.96	-6.42
Sept.	6.14	3.17	2.30	1.55	3.85	0	1.51	1.51	-2.34
Total	61.65	38.84	26.02	9.19	35.21	2.03	7.17	9.20	-26.01

TABLE VIII

SUPPLEMENTAL WATER AVAILABILITY FROM HYPOTHETICAL RESERVOIRS IN ACRE-FEET

END OF MONTH OF	EXISTING CONDITION	RESERVOIR 1		RESERVOIR 2	
		ORIGINAL OPERATIONAL SCHEDULE	NO PUMPAGE INTO RESERVOIR IN IRRIGATION SEASON	ORIGINAL OPERATIONAL SCHEDULE	NO PUMPAGE INTO RESERVOIR IN IRRIGATION SEASON
OCT (1973)	212.8	643.2	643.2	915.2	915.2
NOV	140.8	526.4	462.4	736.0	668.8
DEC	424.0	737.6	227.2	921.6	374.4
JAN (1974)	265.6	536.0	27.2	672.0	112.0
FEB	256.0	502.4	0	582.4	0
MAR	118.4	321.6		310.4	
APR	14.4	201.6		115.2	
MAY	0	192.0		54.4	
JUNE	473.6	662.4		697.6	
JULY	432.0	779.2		912.0	
AUG	348.8	747.2		1,040.0	
SEPT	329.6	723.2		998.4	

Note: A total of 173 acre-feet of supplemental water has been siphoned from C-25 during this irrigation season

TABLE IX

The reservoir at its crest elevation of 27.5 feet msl (a maximum permissible elevation under existing conditions) would conserve a maximum of 480 acre-feet or 5.57 inches of runoff from the study area. It is equivalent to about 6.6 inches of irrigation for the grove area. This amount falls short of the 12.6-inch irrigation requirement itself, not to mention the losses by evaporation and seepage.

An agricultural reservoir of the size and type under study apparently cannot provide seasonal carry-over storage which alone would satisfy irrigation requirements. Perimeter seepage (from adjacent surface water bodies and groundwater storage) is the major source providing the supplemental water to meet the demand.

Before a conclusion can be reached of the value of such an agricultural reservoir in developing on-site irrigation water, it is essential to investigate the retention characteristics of such a reservoir as influenced by: (1) increasing the depth of the conservation pool; and, (2) increasing the surface area. The intended purpose was to see whether or not an optimized reservoir could be designed accordingly so that carry-over storage could be better developed to meet the irrigation demand.

Two hypothetically larger reservoirs were studied for this purpose by using gathered hydrologic data. The first hypothetical reservoir (hereinafter identified as Reservoir One) assumed that the bank of the reservoir was elevated so that the overflow weir crest could be increased by 1.6 feet from the existing maximum permissible elevation to 29.0 feet msl. The surface area of the reservoir remained unchanged at 160 acres. The second hypothetical reservoir (Reservoir Two) assumed that the surface area of the existing reservoir was increased by 100% to 320 acres (373 feet square) while the overflow weir crest was maintained at 27.5 feet (essentially unchanged from the existing maximum permissible elevation).

The hydrologic data and the irrigation drafts from the existing reservoir gathered in the 1973-74 water year, were routed through the two hypothetical reservoirs with two different irrigation operation methods. The first operation method assumed that pumpage to the reservoir from the grove was identical with that of the actual 1973-74 operation. The second operation method assumed that there was no pumpage to the reservoir during the irrigation season.

By following the first operation method, Reservoir One retained available irrigation water in excess of the demand by 19.0 acre-feet at the end of the 1974 irrigation season, as shown on Curve B in Figure 11. The corresponding stage fluctuations in the reservoir are shown in Figure 12. Reservoir Two (Curve D in the same figure) on the other hand, still did not develop a storage that alone met the irrigation demand and a total of 119 acre-feet had to be procured from the Project canal system. The advantage of Reservoir One over Reservoir Two can be determined from Figure 11 which shows that the depletion of storage is faster on Curve D than that on B. This is due to the greater combined seepage and evaporation losses as a result of an increased exposed area and an elongated perimeter in Reservoir Two.

The depletion of storage in the two hypothetical reservoirs under the second operation method, as shown on Curves C and E in Figure 11, clearly indicate that the increase of the storage capacity alone (to the degree under investigation) will not create any substantial improvement of the seasonal retention characteristics. The storages in either of the two hypothetical reservoirs were completely depleted by mid-February.

The examination of two hypothetical reservoirs has clearly established that no carry-over storage can be developed on-site with any reasonable enlargement of the reservoir. It has also established that increasing the depth is more effective than increasing the area in developing the supplemental irrigation water availability. Factors affecting reservoir capability indicate the optimum storage capacity is developed only after a careful analysis is made of the drainage constraints along with the proposed irrigation practices and reservoir operational methods to be employed. Other factors to be considered in designing an agricultural reservoir include seepage outflow from the reservoir, seepage inflow to the drainage system, groundwater storage, and potential evaporation.

2. Rainy Season Evaluation

The study area, when in its natural state, contributed runoff north to the St. Johns River Watershed. Under present existing conditions the excessive runoff is discharged south to the Project canal system by two pumping facilities. During the critical drainage periods, one pump (south pump) discharges directly into C-25 Extension, and another into the reservoir where any excess storage exits over the stop-log weir through the culvert into C-25 Extension.

The analysis based on the collected data indicates that the reservoir did not have a significant effect on reducing the peak flow rate into the primary receiving canal under the existing rainy season operation method. The method calls for filling as soon as possible after the beginning of the rainy season. This procedure eliminates storage capacity to the detriment of reducing peak discharge rates to the primary receiving canal system.

The summation of the daily mean discharge through the north and south pumps, and that through the culvert and south pump, are plotted in Figure 13. The blanked areas above S & C lines on top of the bars show the reduction of the peak flow rate due to the reservoir detention, while the darkened areas above N & S lines show the discharge increase due to culvert overflow. It can be seen from this figure that the reduction in maximum daily mean discharge due to detention and/or retention of excess runoff in the reservoir is small.

A discharge hydrograph of a typical storm is plotted in Figure 14 to illustrate the instantaneous peak discharge as influenced by the reservoir. This storm started with the reservoir filled to above the stop-log crest. (Probably a good representation of a likely occurrence due to present operational practices). The solid line in the figure shows the total discharge through the culvert and south pump, and the dashed lines the total discharge through the north and south pumps.

Again, it appears that the reduction in peak discharge due to reservoir retention and/or detention is small. Should a heavier rainfall with a longer duration occur, the reservoir will reach a stage where culvert outflow is equal to or greater than inflow into the reservoir. Under this condition, the peak discharge into the District's canal would be equal to or greater than that of the system without a reservoir. However, had the operational method been to reserve storage until near the end of the rainy season, the same storm would not necessitate release.

This analysis indicates that under present operational practices, the reservoir capacity should be larger to be of benefit in reducing flood peaks while at the same time meeting drainage requirements. To cite some results of present operational procedures: In 1973 the reservoir was filled by the latter part of June, with relatively dry conditions in April and May and only normal rainfall in June. In 1974 the reservoir was filled by mid-June due primarily to above normal rainfall that month. Rainfall in March, April and May was only 0.48 inches, 1.42 inches, and 3.11 inches, respectively.

The deficit in the size of the reservoir can be remedied for flood control purposes by gradually lowering the stage after each heavy rainfall, thereby providing some reservoir storage capacity for any up-coming storm event. Any reservoir storage deficit at the end of the rainy season could be supplied by storage from the primary canal system, but this procedure would not be viewed as practical by the management.

CONCLUSION

The basic hydrologic parameters pertaining to an agricultural area planted to citrus has been evaluated, and the methodology for evaluating such parameters has been indicated.

An operational procedure for irrigation of a citrus grove in an area such as studied has been shown. The efficiency of providing on-site storage for reuse of water has been clearly indicated.

The study has indicated the reservoir investigated has insufficient storage to provide ample water under normal conditions to meet the total seasonal irrigation requirements. This is due to combined seepage and evaporation losses.

The characteristics of two hypothetical reservoirs of larger capacities indicate that the increased size provides a slight seasonal excess which would vary with seasonal rainfall. The same analysis appears to indicate that increasing the reservoir depth is more effective than increasing the area. Also indicated is that even these enlarged reservoirs would go dry prior to the end of the irrigation season if the excess water after irrigation was not recycled into the reservoir.

The study further indicates that the size of the reservoir studied had little effect in reducing peak runoff from the area studied due to operational procedures of having insufficient capacity during such periods. The operational procedures followed were those intending to benefit the grower which was to retain a full storage should additional rainfall not be forthcoming. Although it would be of definite advantage to the receiving basin to operate to reduce peak inflows, the primary concern is grove management. However, reservoirs such as the one studied do provide storage under any type of operation to hold water otherwise being usually discharged to tidewater. They all serve to reduce withdrawals from the supply source during the dry season.

These two factors alone serve to indicate the advantage to this District of private reservoirs. An allied quality study in conjunction with this one further indicates this fact as nutrients in on-site water are reduced by storage retention.

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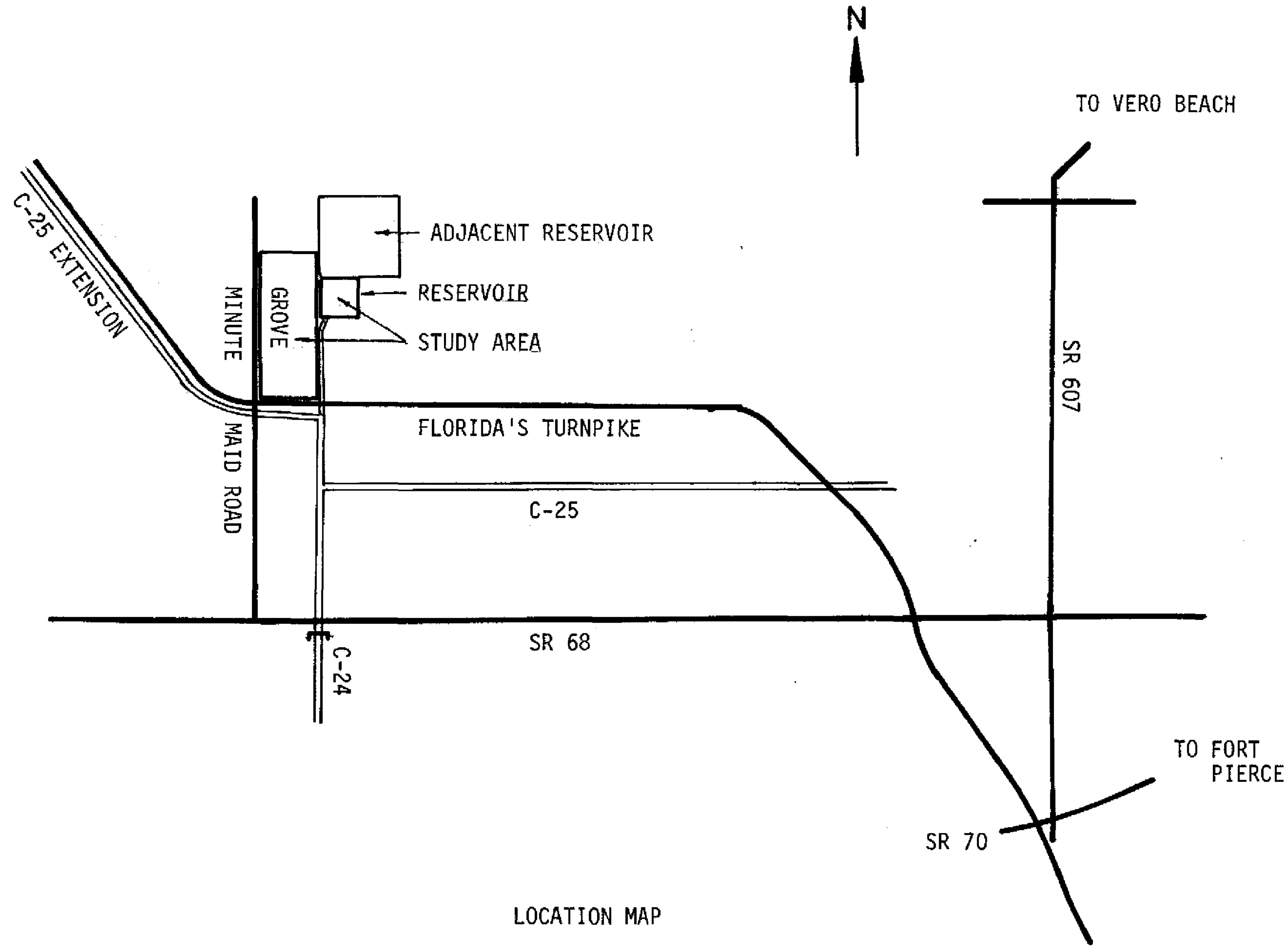
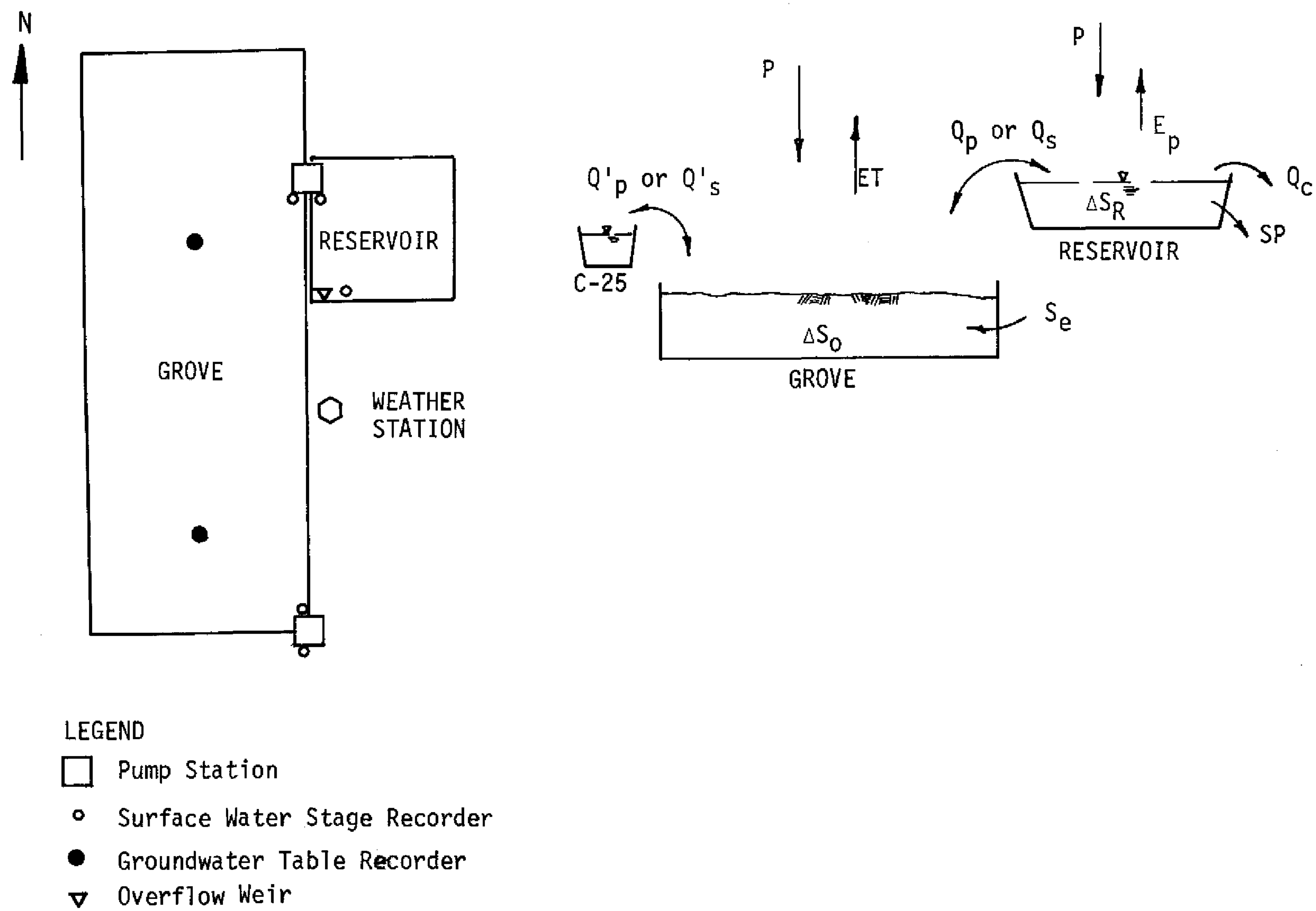


LOCATION MAP

FIGURE 1

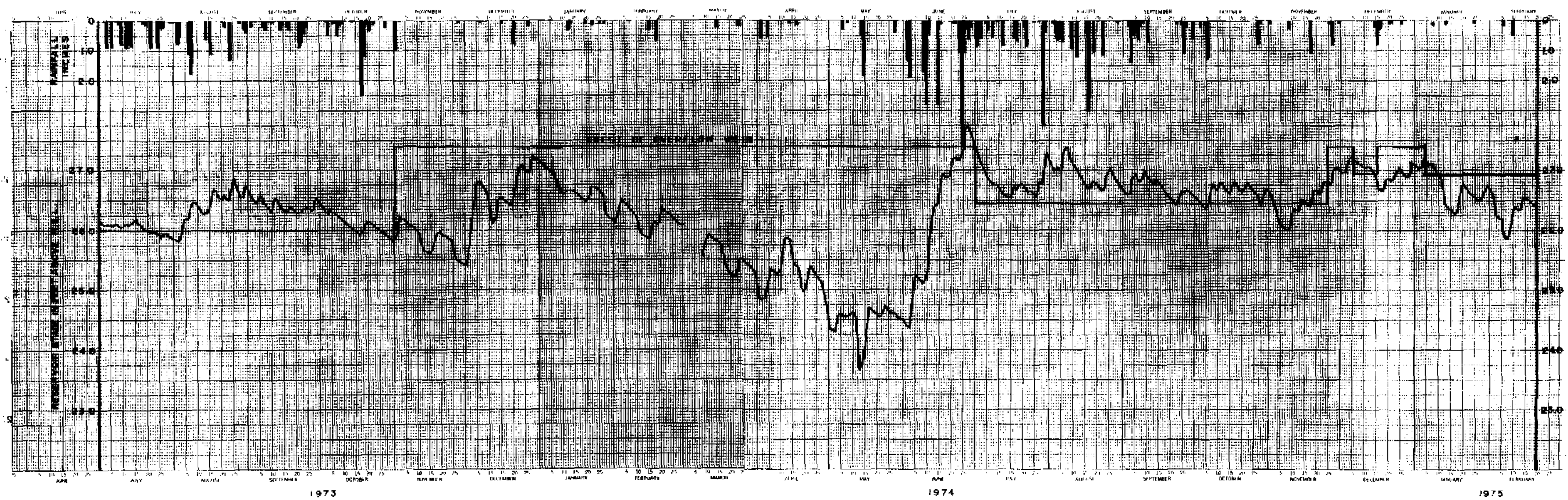
FIGURE 1



FLOW DIAGRAM

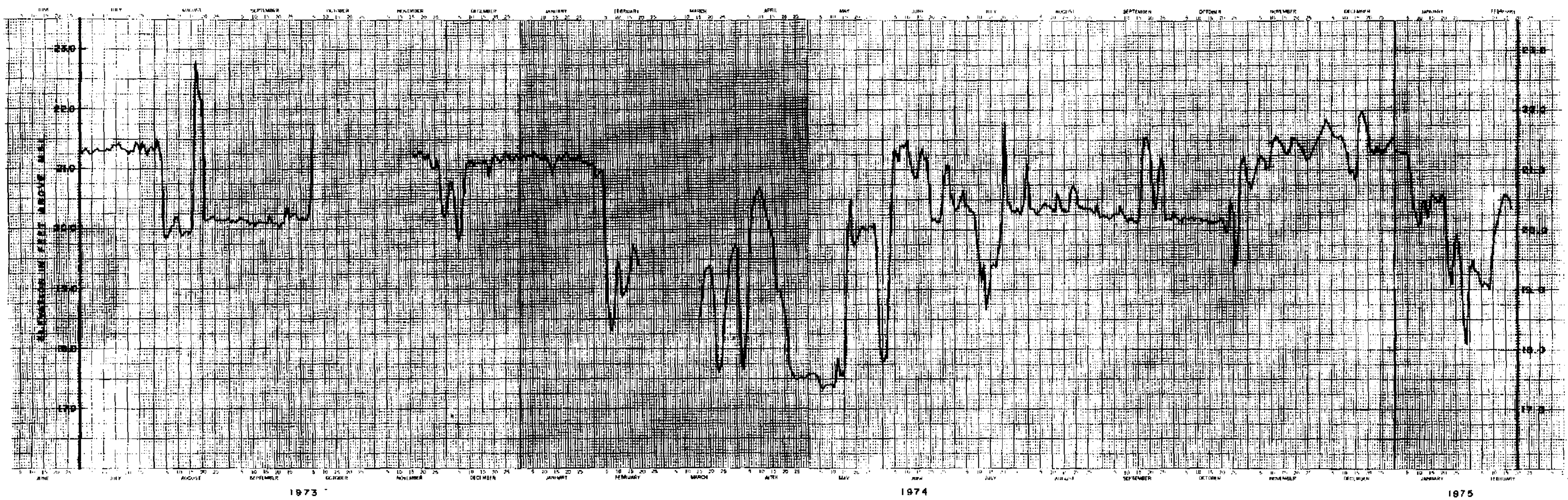
FIGURE 2

FIGURE 2



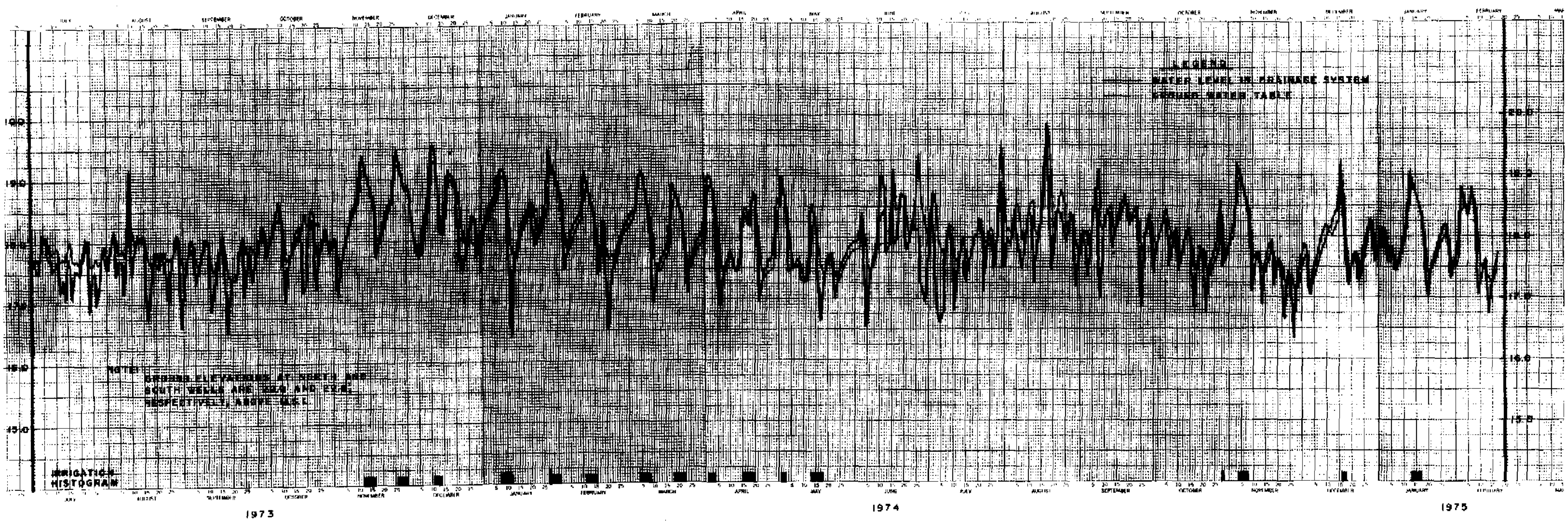
PRECIPITATION AND RESERVOIR DAILY MEAN STAGE

FIGURE 3



DAILY MEAN STAGE AT SOUTH PUMP DOWNSTREAM IN C-25 EXTENSION

FIGURE 4



DAILY MEAN SURFACE WATER ELEVATION AND GROUND WATER FLUCTUATION IN GROVE AREA

FIGURE 5

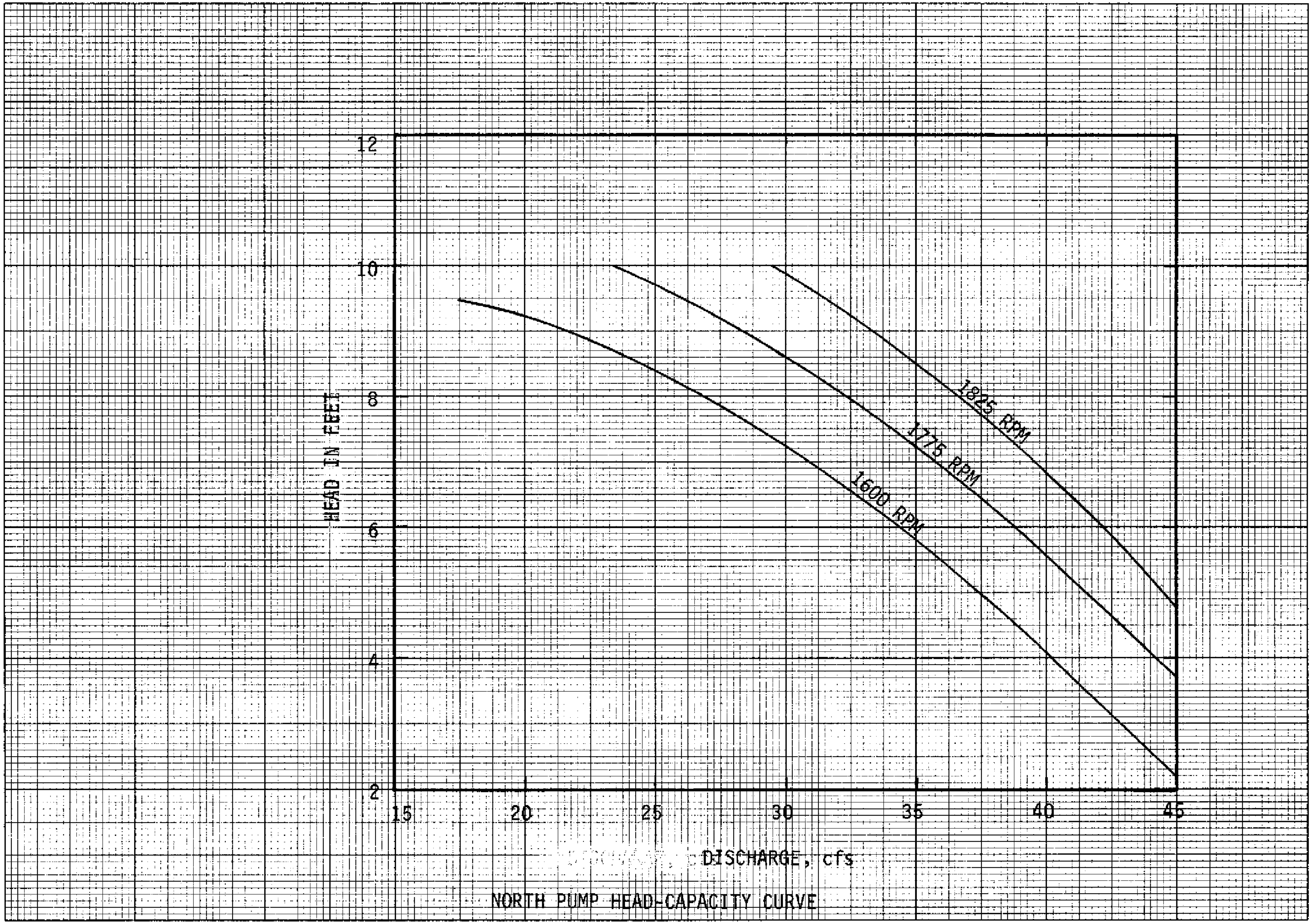
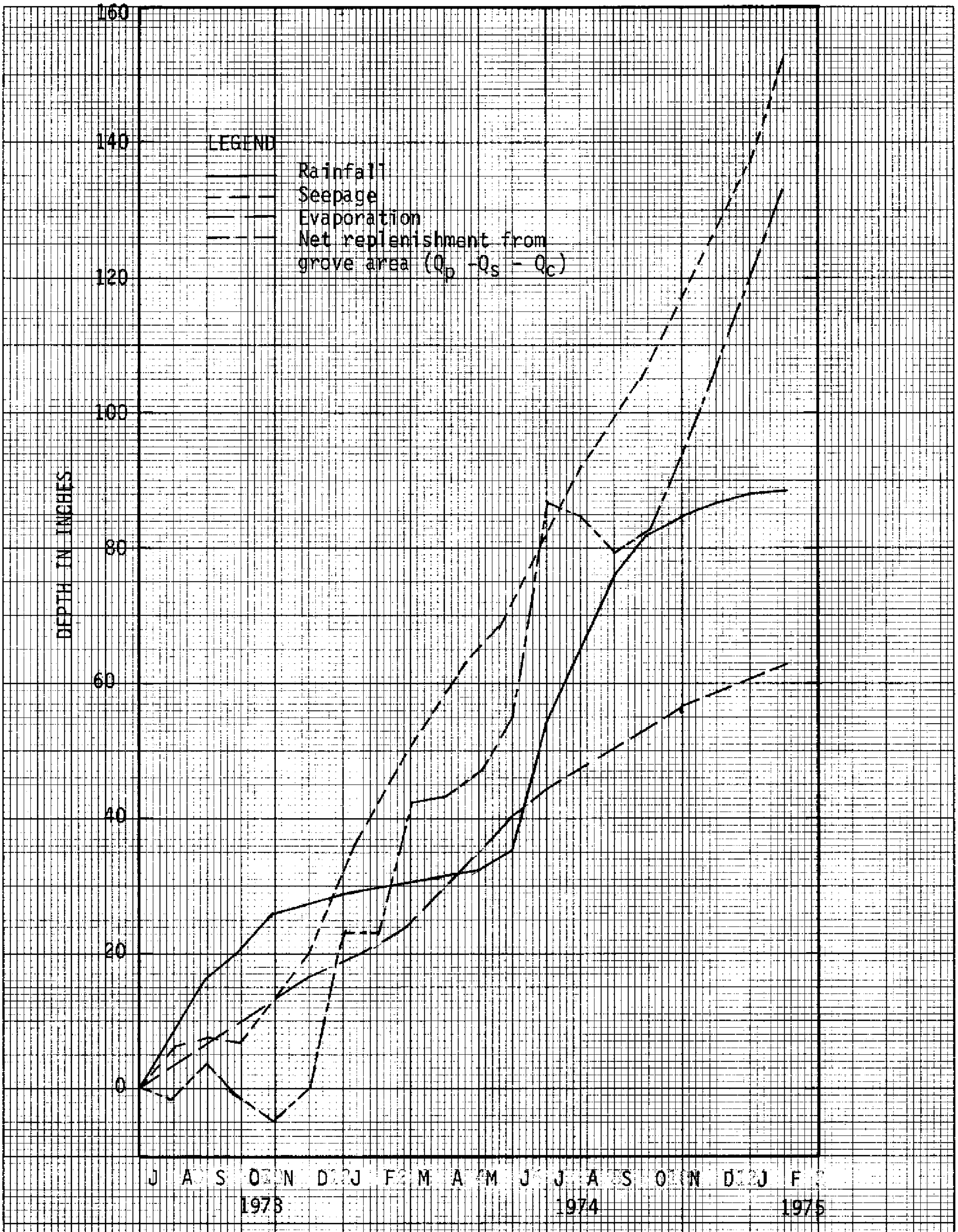


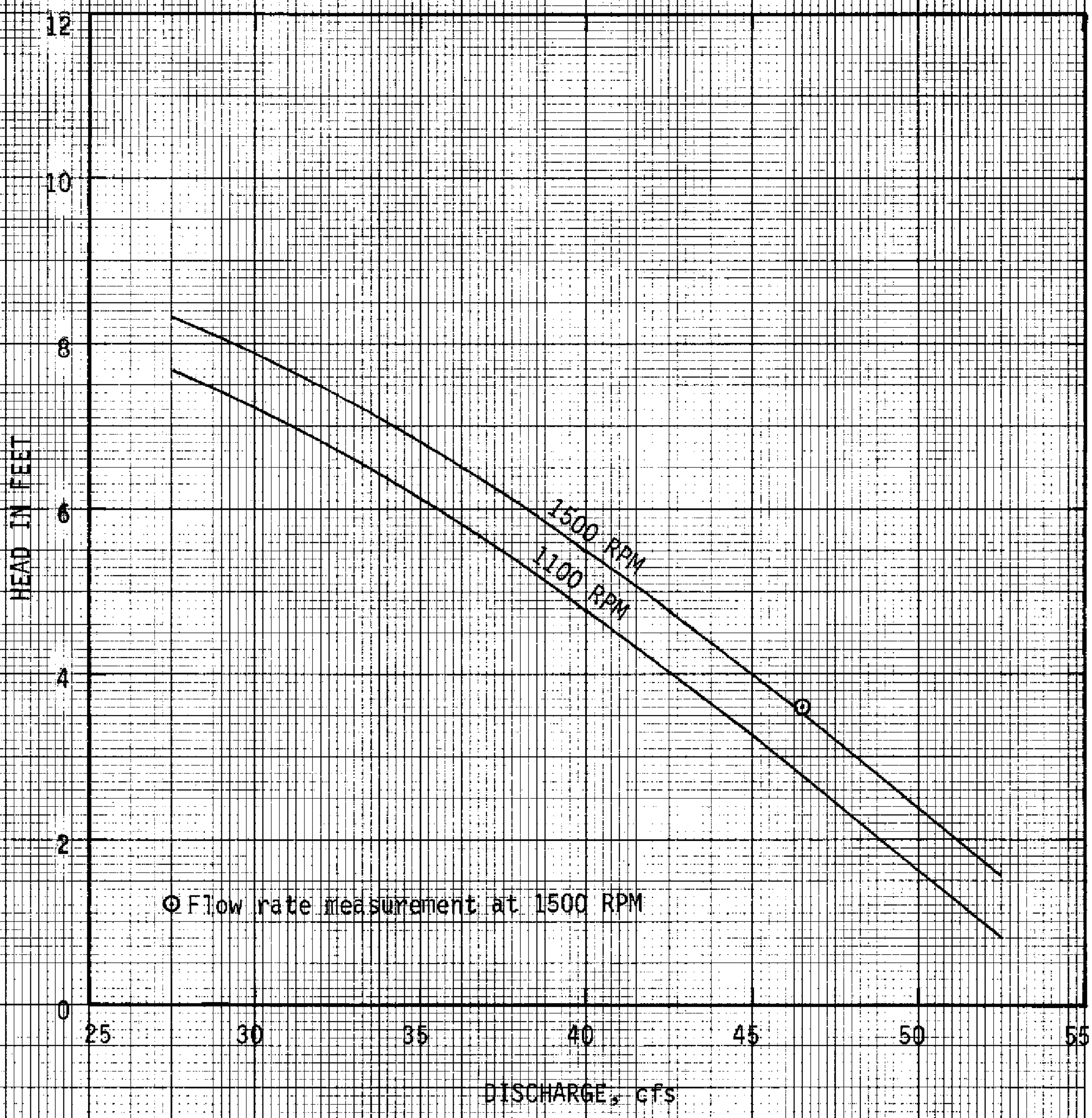
FIGURE 6

FIGURE 6



MASS CURVES FOR RESERVOIR SYSTEM

FIGURE 7



SOUTH PUMP HEAD-CAPACITY CURVE

FIGURE 8

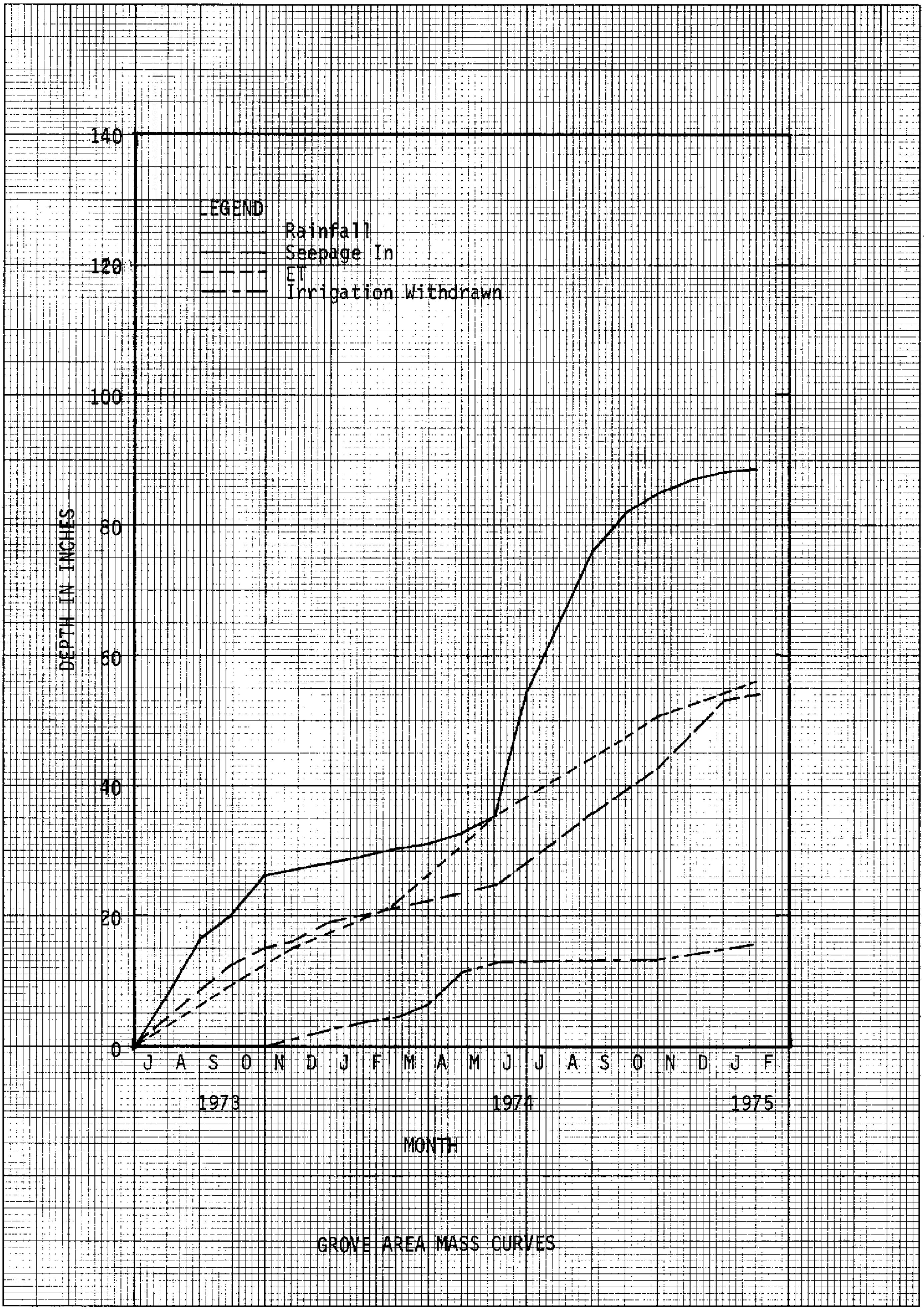
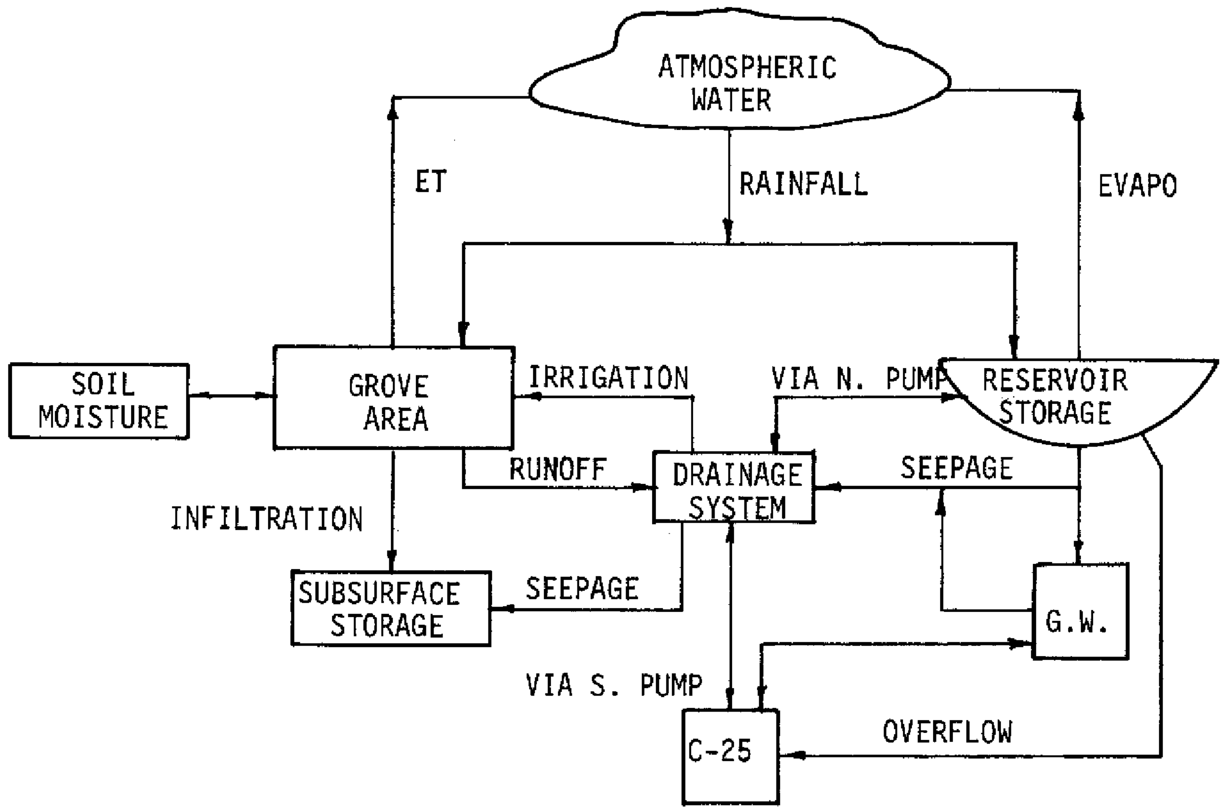
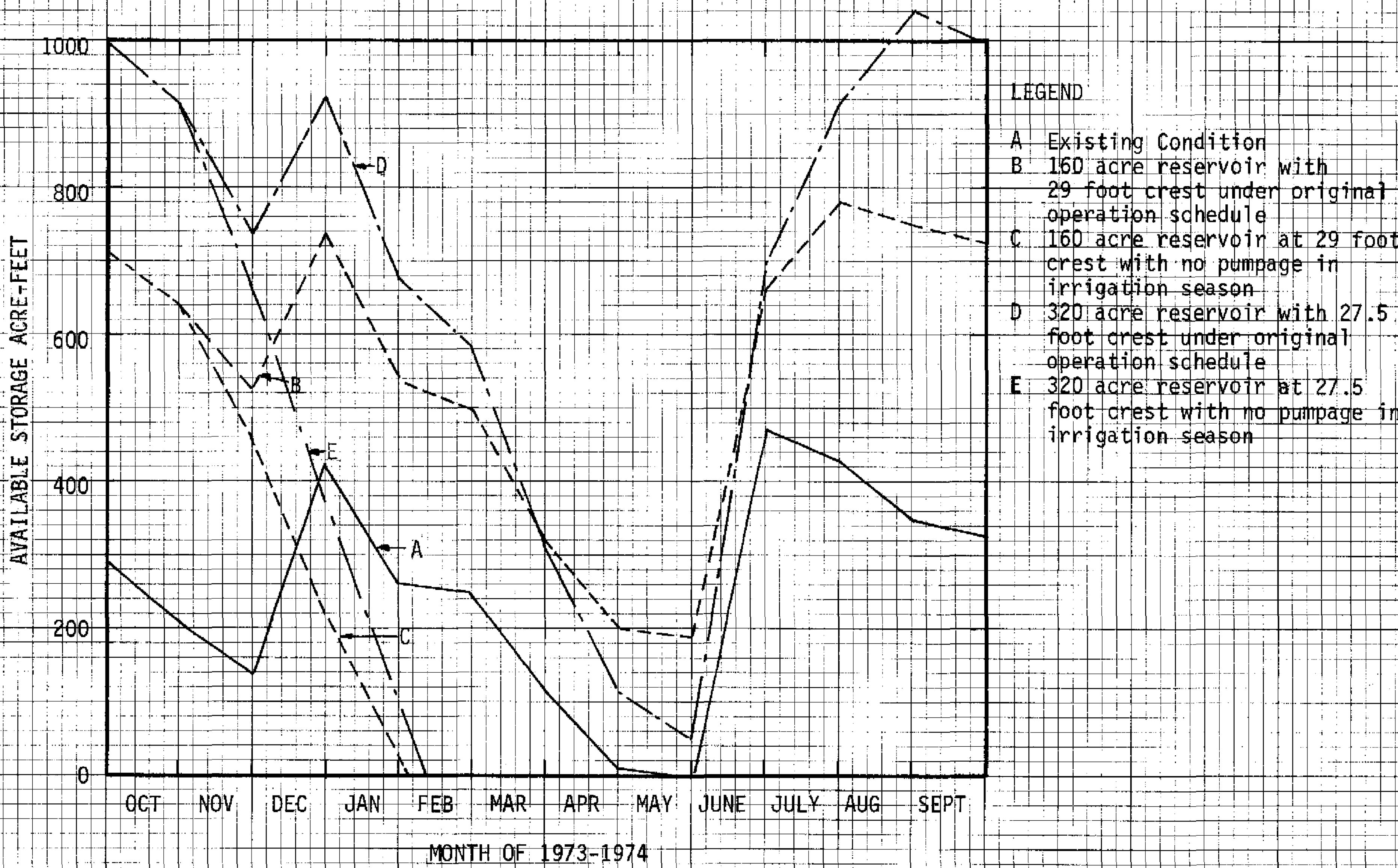

LEGEND
Rainfall
Seepage In
ET
Irrigation Withdrawn
DEPTH IN INCHES
0
20
40
60
80
100
120
140
J A S O N D J F M A M J J A S O N D J F
1973
1974
1975
MONTH
GROVE AREA MASS CURVES

FIGURE 9



GROVE-RESERVOIR SYSTEM FLOW DIAGRAM

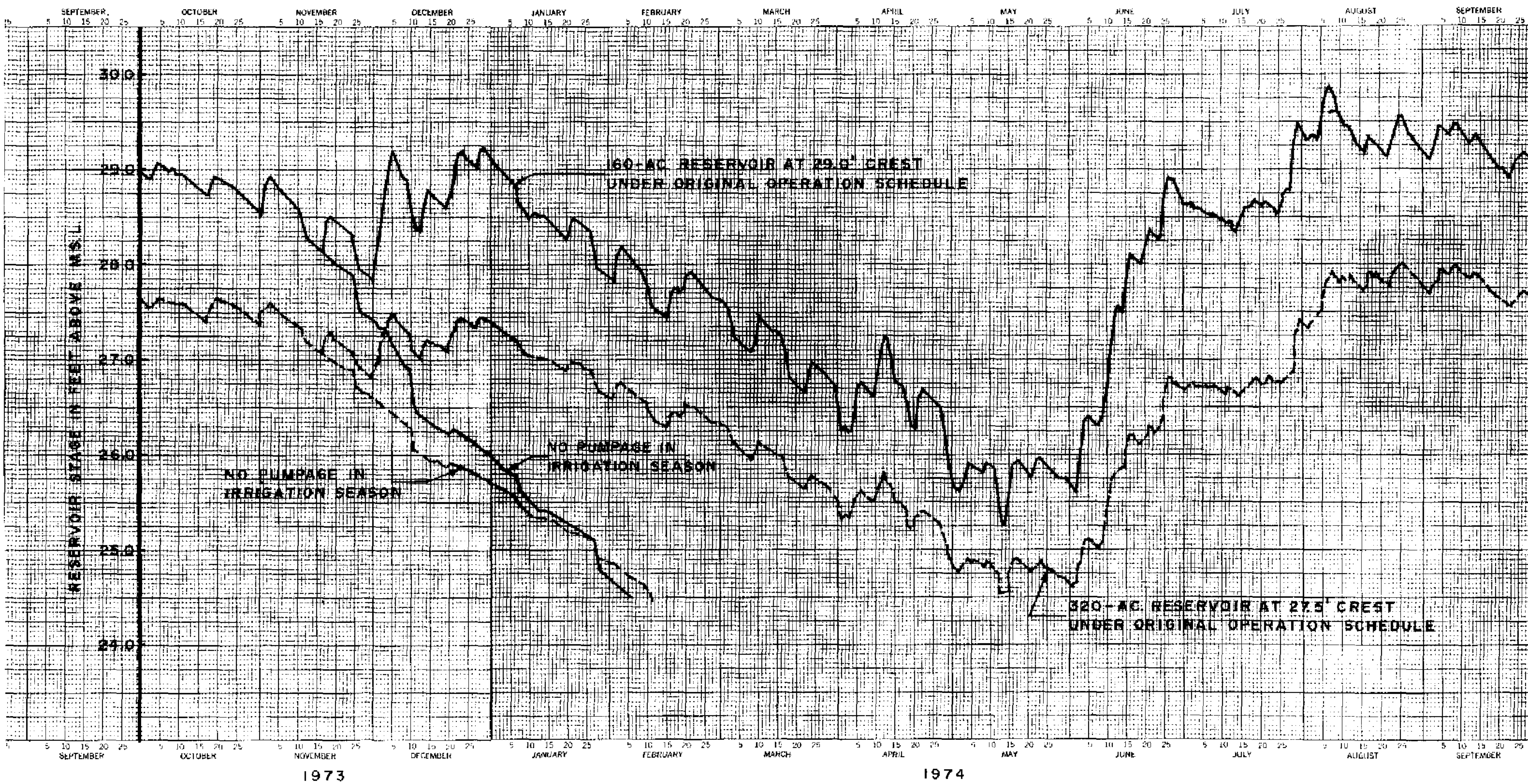
FIGURE 10



SUPPLEMENTAL WATER AVAILABILITY FROM EXISTING RESERVOIR AND TWO HYPOTHETICAL RESERVOIRS

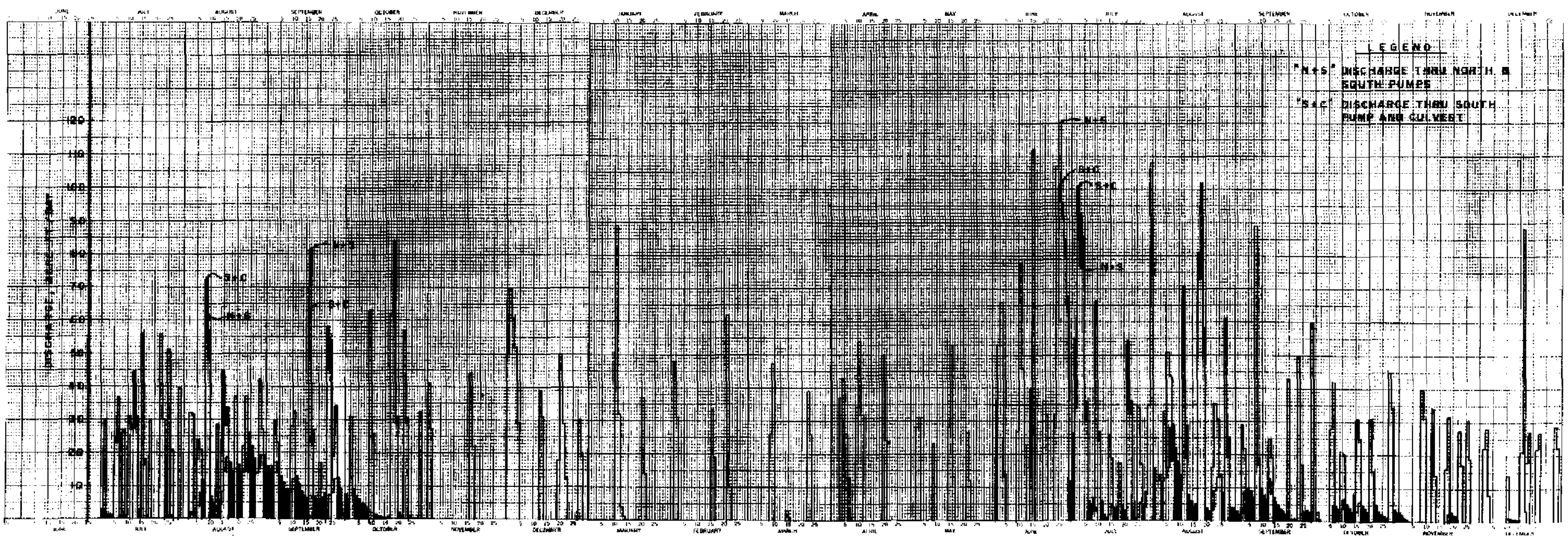
FIGURE 11

FIGURE 11



RESERVOIR STAGES UNDER VARIOUS ASSUMED CONDITION

FIGURE 12



DAILY MEAN DISCHARGE TO C-25

FIGURE 13

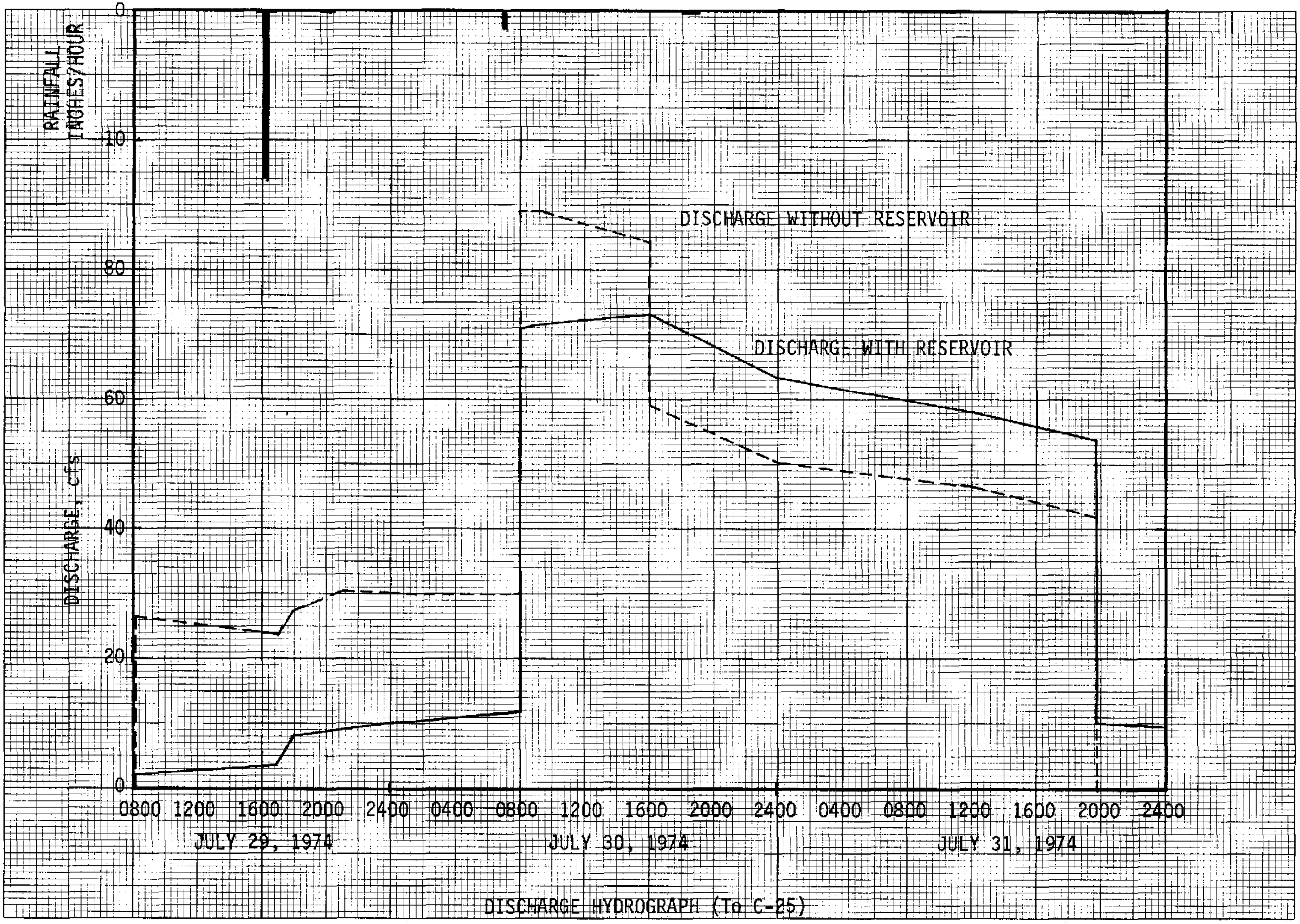


FIGURE 14

FIGURE 14