TECHNICAL PUBLICATION SJ 85-3

THE MEAN ANNUAL, 10-YEAR, 25-YEAR, AND 100-YEAR FLOOD PROFILES FOR THE UPPER ST. JOHNS RIVER UNDER THE EXISTING CONDITIONS

BY

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PALATKA, FLORIDA

MARCH 1985

PROJECT NUMBER 2030506

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ABSTRACT

The Upper St. Johns River Basin has been severely altered by man's activities since the turn of the century. These alterations have affected both flood flows and low flows, invalidating possible predictions based on past observations. For the current basin conditions, synthetic streamflow data have been generated for a period of 35 years (1949-1983) by a continuous hydrologic simulation model. Flood flow frequency analyses have been conducted on the annual peak flow data. The mean annual, 10-year, 25-year, and 100-year flood profiles have been computed for the St. Johns River at its head waters near Florida Turnpike to State Road 46. These flood profiles are useful to determine flood elevations for any location in the Upper St. Johns River Flood Valley under the existing conditions.

INTRODUCTION

About 2000 sq. mi. of drainage area stretching from State Road 46 south of Lake Harney to its headwaters near the Florida Turnpike is commonly known as the Upper St. Johns River Basin (Figure 1). The river is about 110 miles long in this portion of the basin.

The Upper St. Johns River Basin (USJRB) has been drastically impacted by man, especially south of Lake Washington, since about year 1900. Over 60 percent of floodplain has been ditched, diked and drained to expose the underlying fertile muckland for prime agricultural production. These developments caused, (i) a loss of river valley storage capacity due to marked reduction in the virgin marsh areas, (ii) increased runoff from the westerly slopes as a result of improved drainage, and (iii) serious restrictions in the water conveyance capacity of the original broad, gently sloping marsh floodway. In general, the floods have aggravated and low flows have dwindled.

The consequences of the Upper St. Johns Basin exploitation were severely felt when prolonged heavy rainfall in 1947, and again in 1953, caused generally disastrous flooding for quite long periods. In the wake of the 1947 flood, a preliminary (flood protection) plan was formulated by the U.S. Army Corps of Engineers which provided for diversion of flood waters to the Indian River. A Plan for Water Control in the Upper St. Johns Basin was finalized in 1962 and went into construction in the late 60's. However, construction was halted in 1972 to allow

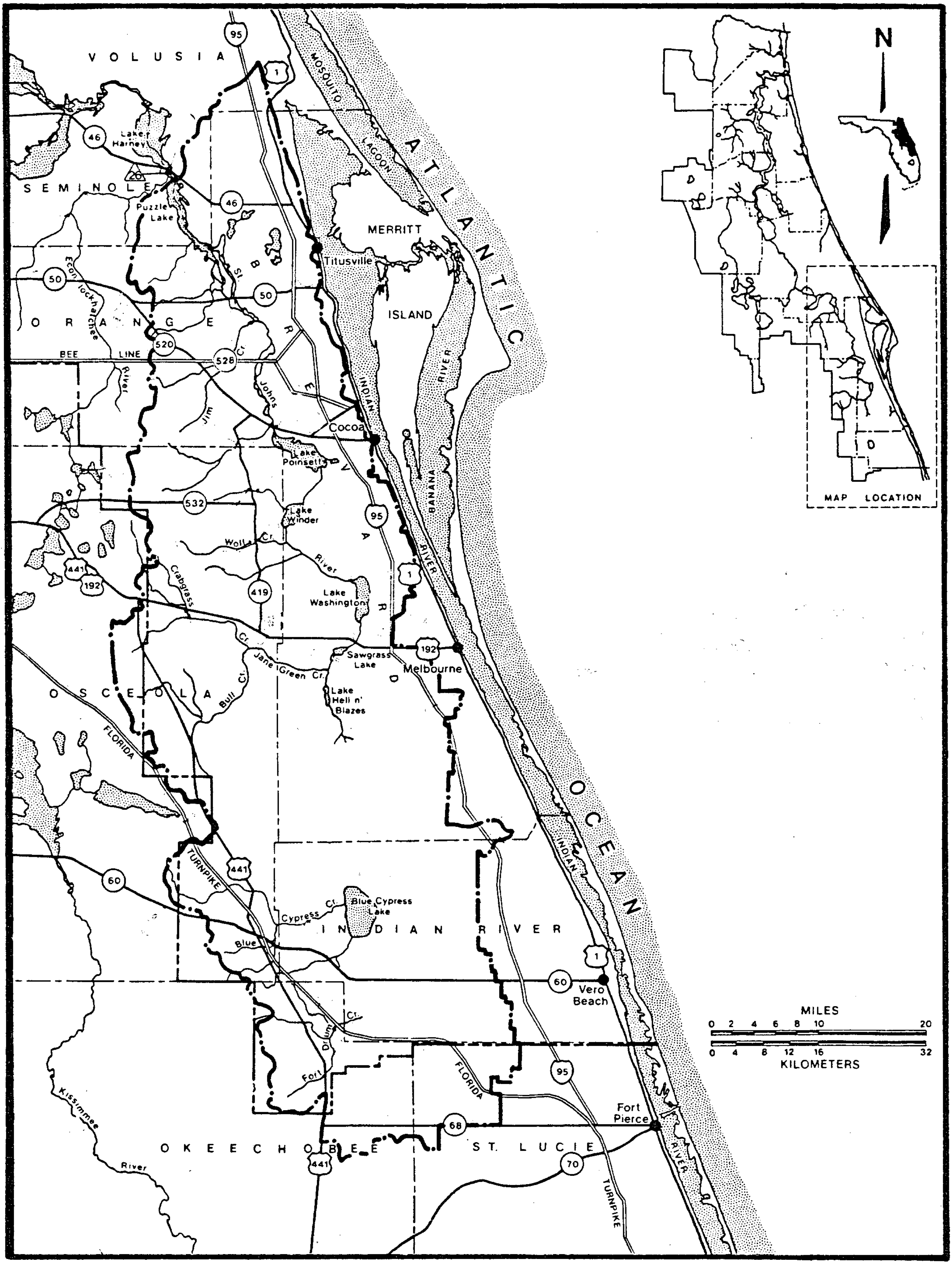


Figure 1. The Upper St. Johns River Basin

preparation of an Environmental Impact Statement (EIS). In January 1977, the local sponsorship of the project was transferred to the St. Johns River Water Management District. The District's involvement in the project was both to conduct independent studies and to provide technical support to the U.S. Corps of Engineers in their efforts to finalize the plan. As Phase I report (Ref. 3), the District published in 1979 rather detailed analyses of hydrologic, hydraulic, environmental, socioeconomic, and other pertinent data presently available for the basin.

As of this date, practically all the engineering, environmental and other pertinent studies for finalizing the new Comprehensive Water Management Plan for the USJRB have been completed. This Technical Publication presents the results of flood flow analysis for the USJRB under the existing conditions.

METHODOLOGY

For a given stream, flood profiles for different return periods are commonly determined by conducting statistical frequency analysis on observed or simulated annual peak flow data and then performing backwater computations on flood flows of desired frequencies. The study area has seven U.S. Geological Survey stream gaging stations on the main stem of the river (Table 1), but only three of which gage the discharge. The highest stages observed at these stations are presented in Table 1.

Major developments affecting streamflow characteristics took

Table 1. Surface Water Gaging Stations in the Upper St. Johns River

	Gaging Station	Date Established	Highest Stage Observed (ft. NGVD)
1.	St. Johns Headwaters Near Vero Beach (S.R. 60)*	Feb. 1942	27.78 (9-5-79)
2.	St. Johns Headwaters Near Kenansville*	Feb. 1942	26.37 (10-16-56)
3.	St. Johns River Near Melbourne (U.S. 192)**	Oct. 1939	20.88 (9-30-60)
4.	Lake Washington Near Eau Gallie*	July 1942	20.39 (10-1-60)
5.	St. Johns River Near Cocoa (S.R. 520)**	Oct. 1953	16.96 (10-11-53)
6.	St. Johns River Near Christmas (S.R. 50)**	Oct. 1933	12.43 (9-28-60)
7.	St. Johns River Above Lake Harney (S.R. 46)***	July 1941	10.62 (10-13-53)

* Stage Only.
** Stage and Discharge.
*** Stage. (Also Discharge Measurements for Some Periods.)

place in the Basin since the establishment of the foregoing gaging stations. Thus, in general, the historic data from these stations can not be regarded homogeneous for conducting flood flow frequency analysis. Instead, annual peak flows at twelve locations along the river were derived for the period of 1949-1983 by extensive watershed modeling. Frequency analyses were performed by Log Pearson Type III distribution. Finally, water surface profiles were computed for floods of different frequencies by the U.S. Army Corps of Engineers' HEC-2 step backwater program.

EXISTING BASIN CONDITIONS

Existing conditions consider all the changes, both physiographic and hydrologic, that have taken place in the USJRB during the past several decades. Physiographic changes include basin alterations by dikes and ditches. Hydrologic changes are alterations in drainage patterns such as discharge by pumpage, interbasin diversion, etc. The data for this purpose have been collected from several sources and updated, where necessary, by extensive field survey. A major portion of Ref. 3 was devoted to inventorying all available basin data, as of 1979. These data, with some updates, have been used in the final streamflow modeling.

For the purpose of water management planning, the entire Upper St. Johns River Basin has been divided into seven planning units and a special unit named C-25 Extension Basin (Figure 2). In general, major highways are the dividing lines for the planning units. Each planning unit has been further divided into sub-basins based on drainage divides. Hydrologically, each sub-basin forms a primary modeling unit for runoff calculations.

The entire river valley length in the seven planning units is divided into 12 river reaches, each planning unit having at least one reach (Figures 3 to 9). The valley receives, conveys and stores runoff from its tributary basins along its entire length. Over 100 river cross-sections were used to determine the storage and conveyance characteristics of the river valley. Figures 3-9 depict the general boundary conditions of the upper

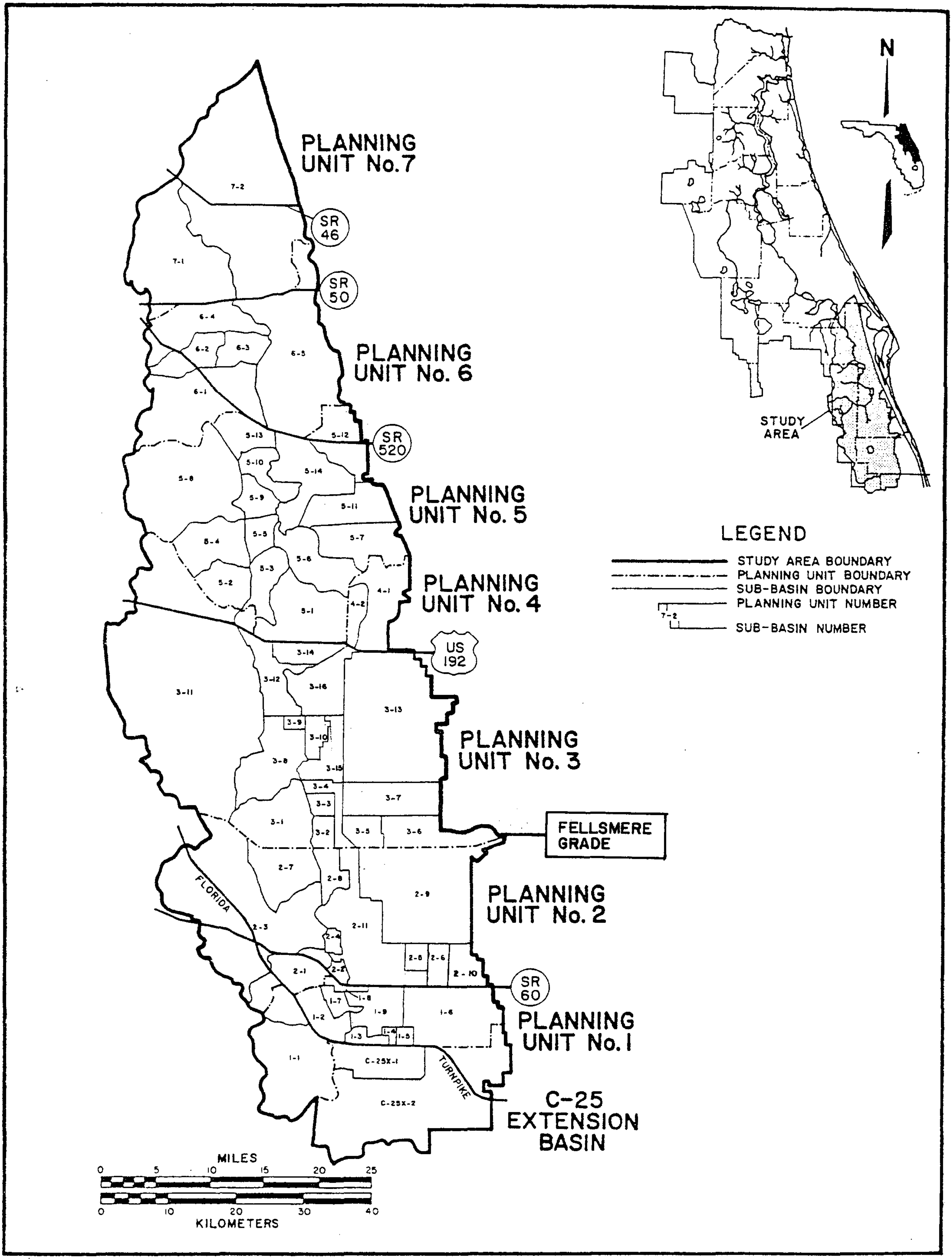


Figure 2. Planning Unit and Basin Sub-Division of the Upper St. Johns River Basin

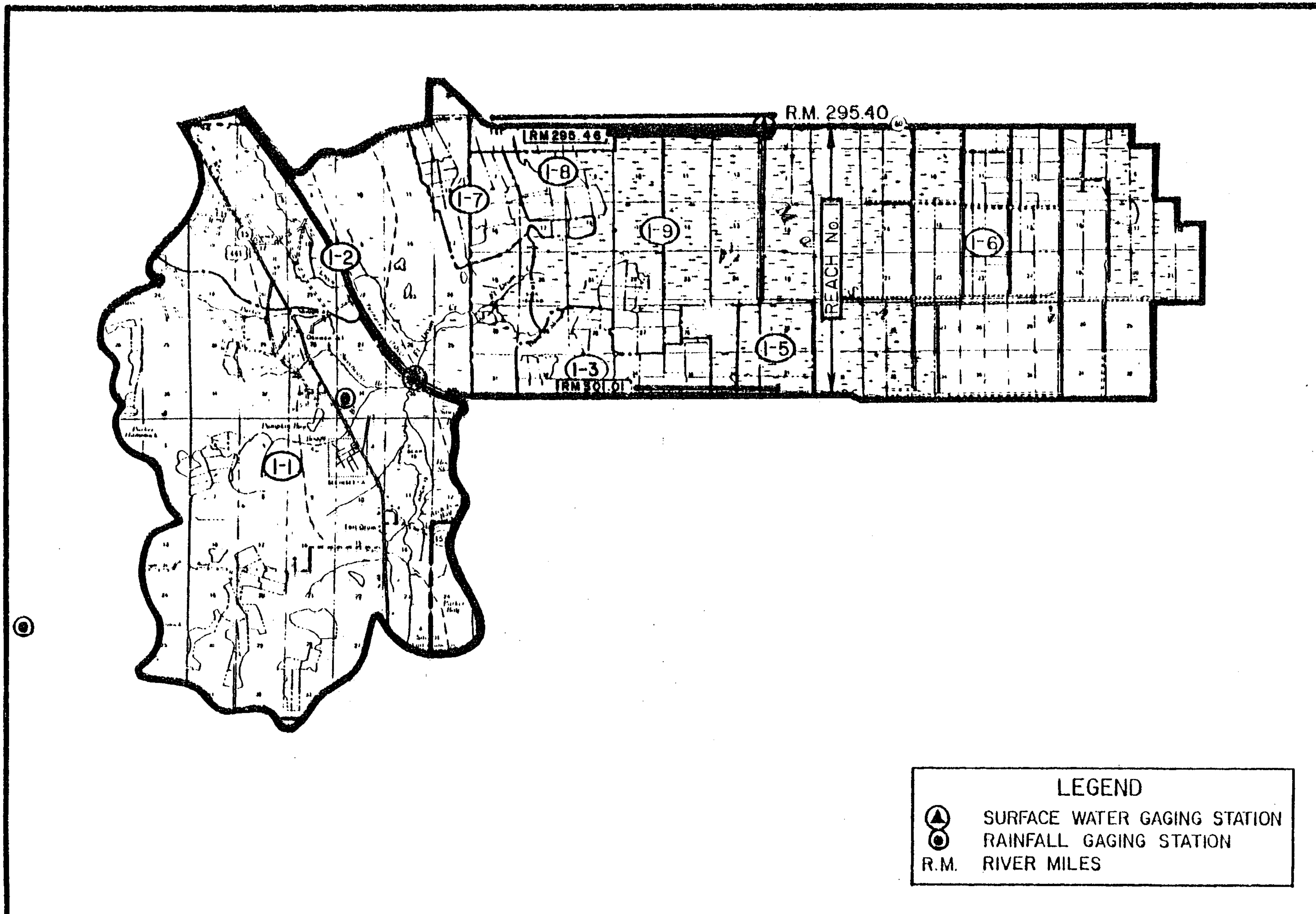


Figure 3. Planning Unit No. 1 Sub-basin, River Reach, and River Mile Designation.

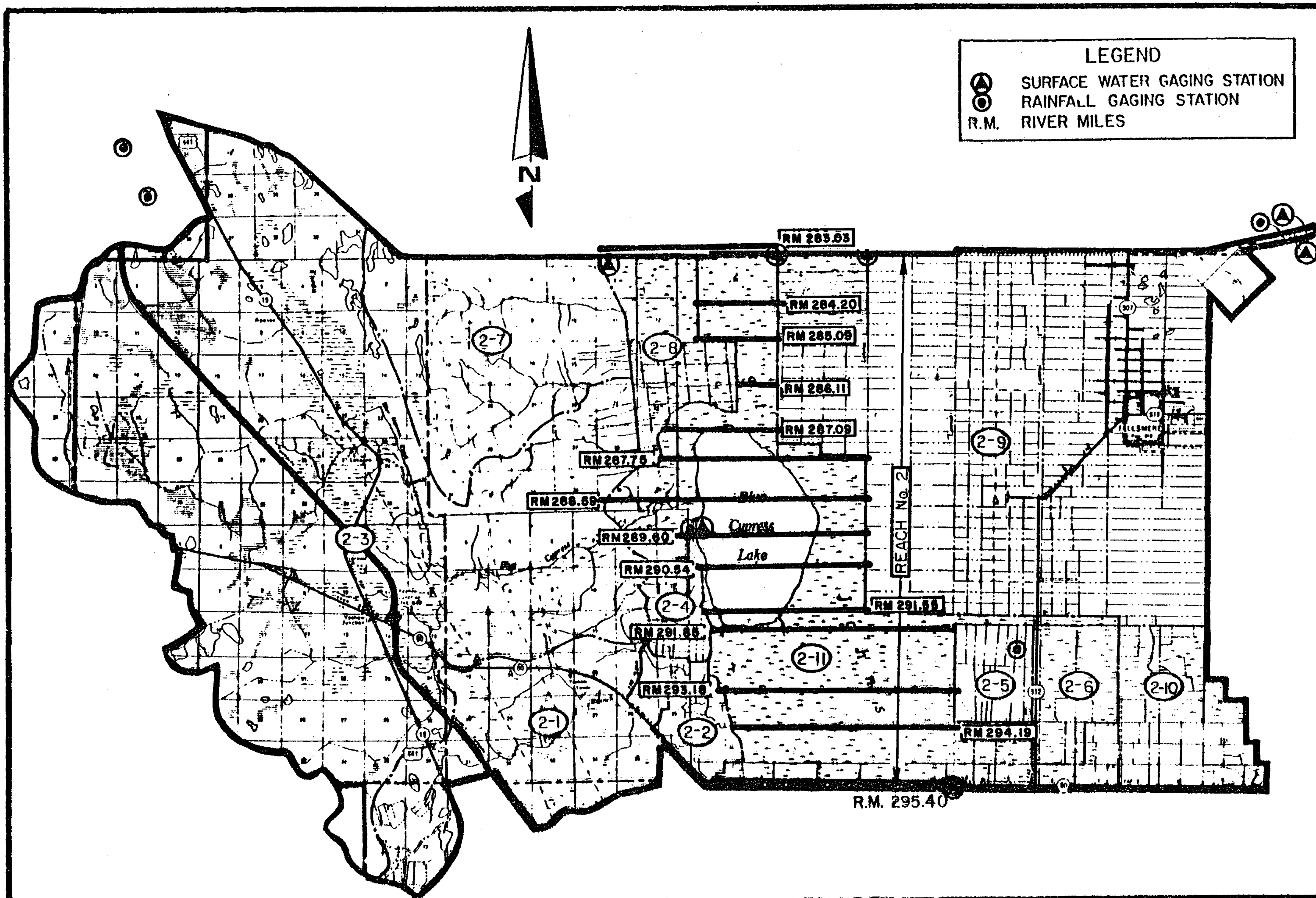


Figure 4. **Planning Unit No. 2 Sub-basin, River Reach, and River Mile Designations**

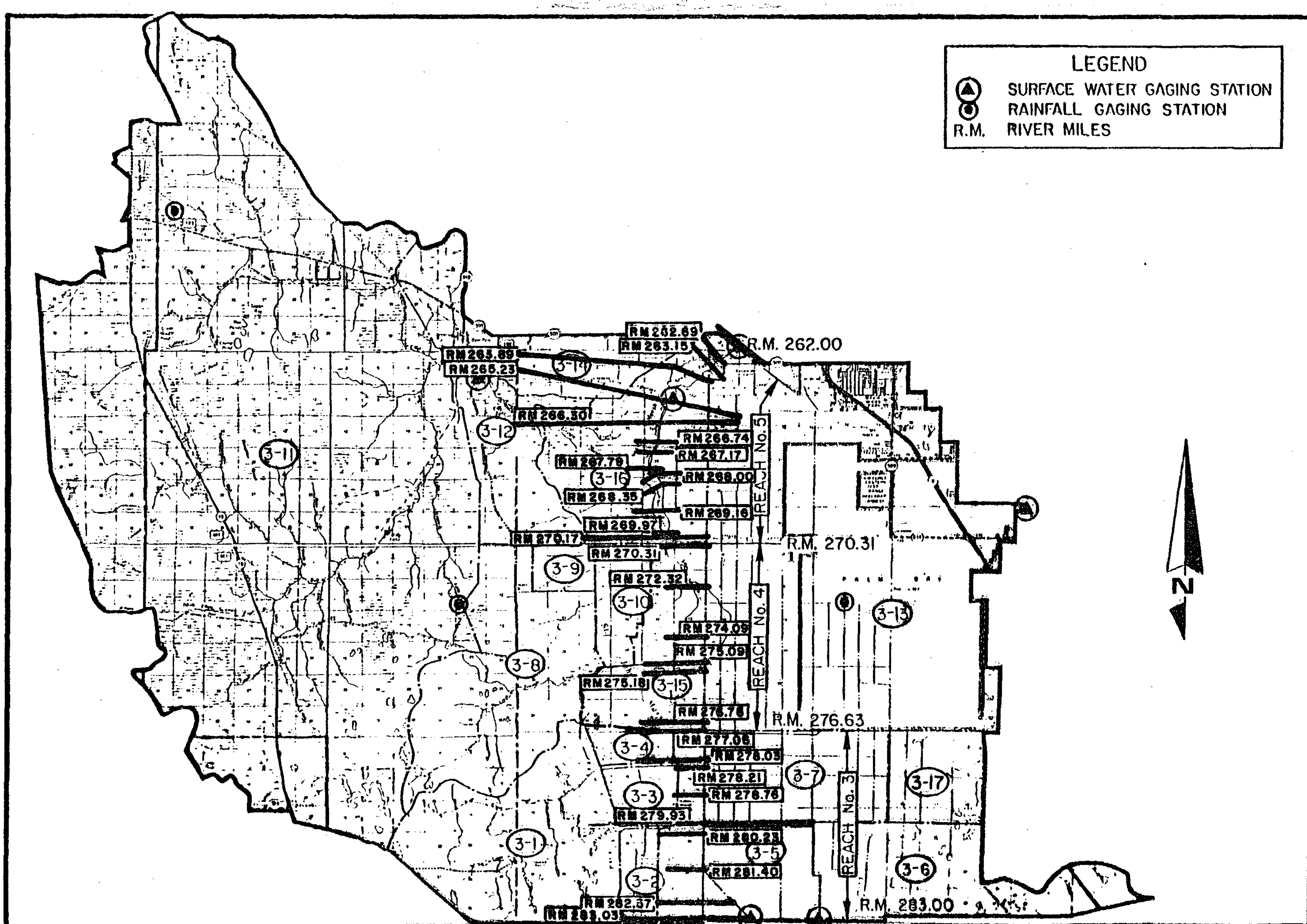


Figure 5. Planning Unit No. 3 Subbasin, River Reach and River Mile Designations.

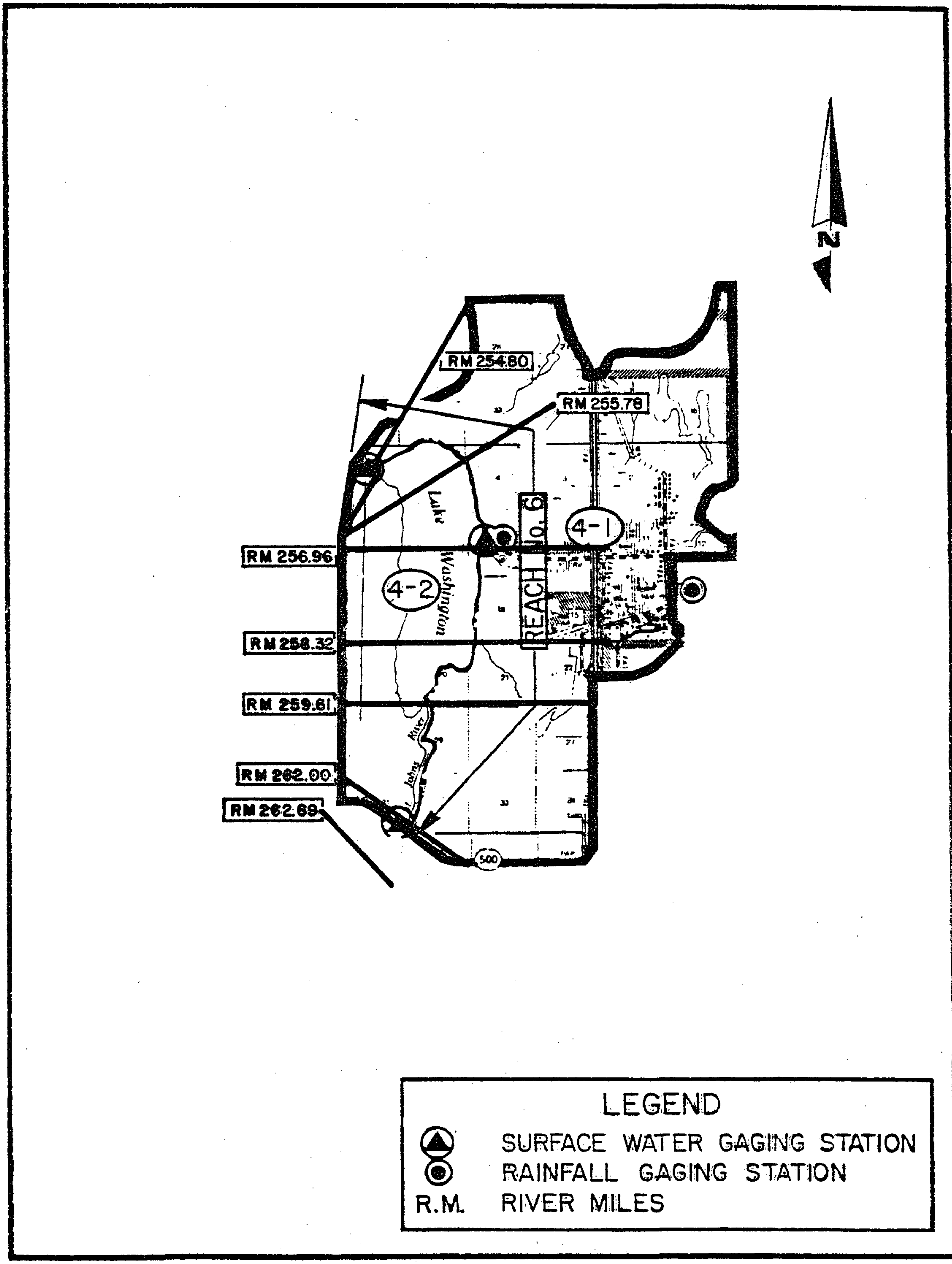


Figure 6. Planning Unit No. 4 Subbasin , River Reach, and River Mile Designations

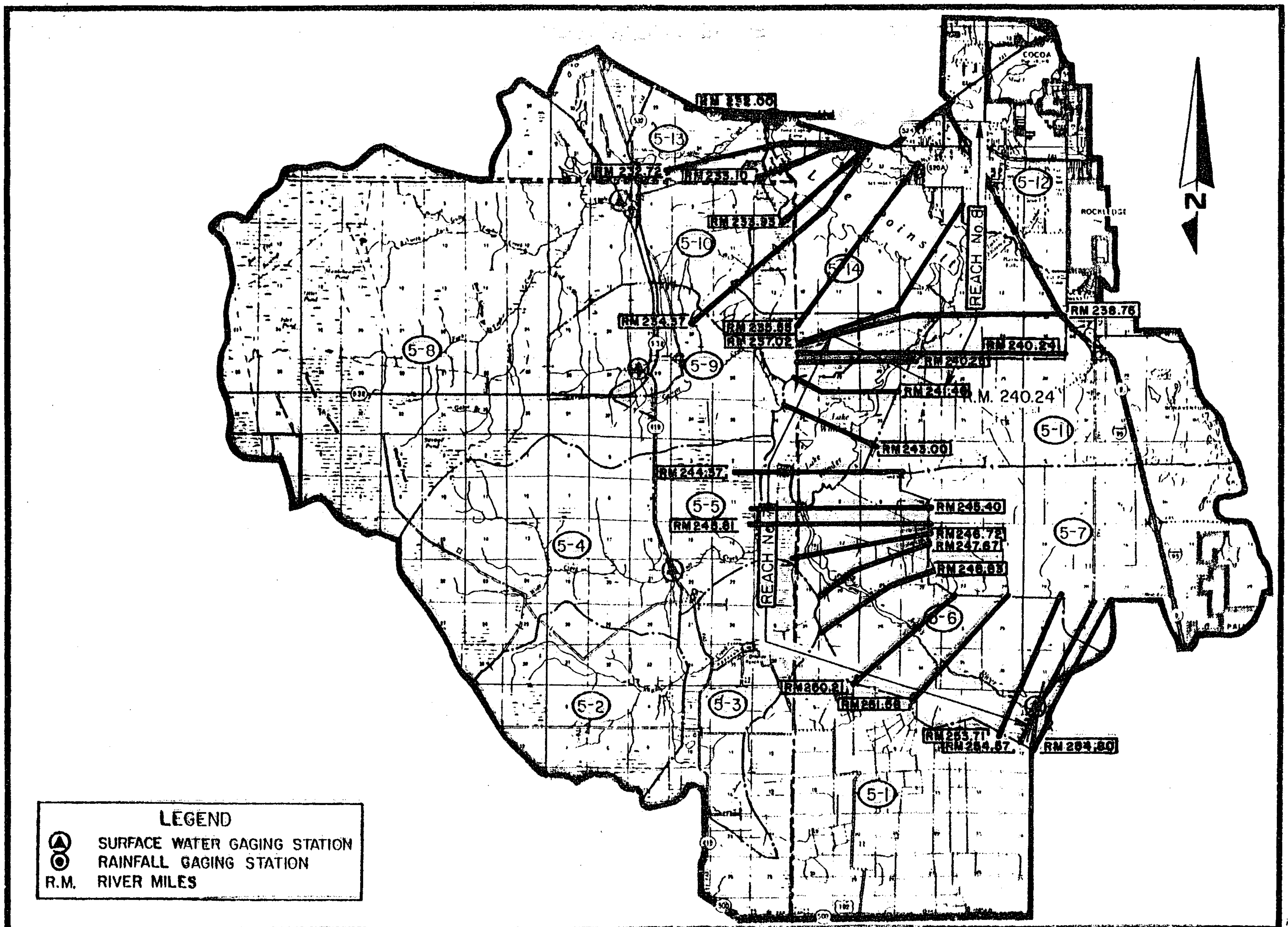


Figure 7. Planning Unit 5 Subbasin, River Reach, and River Mile Designation.

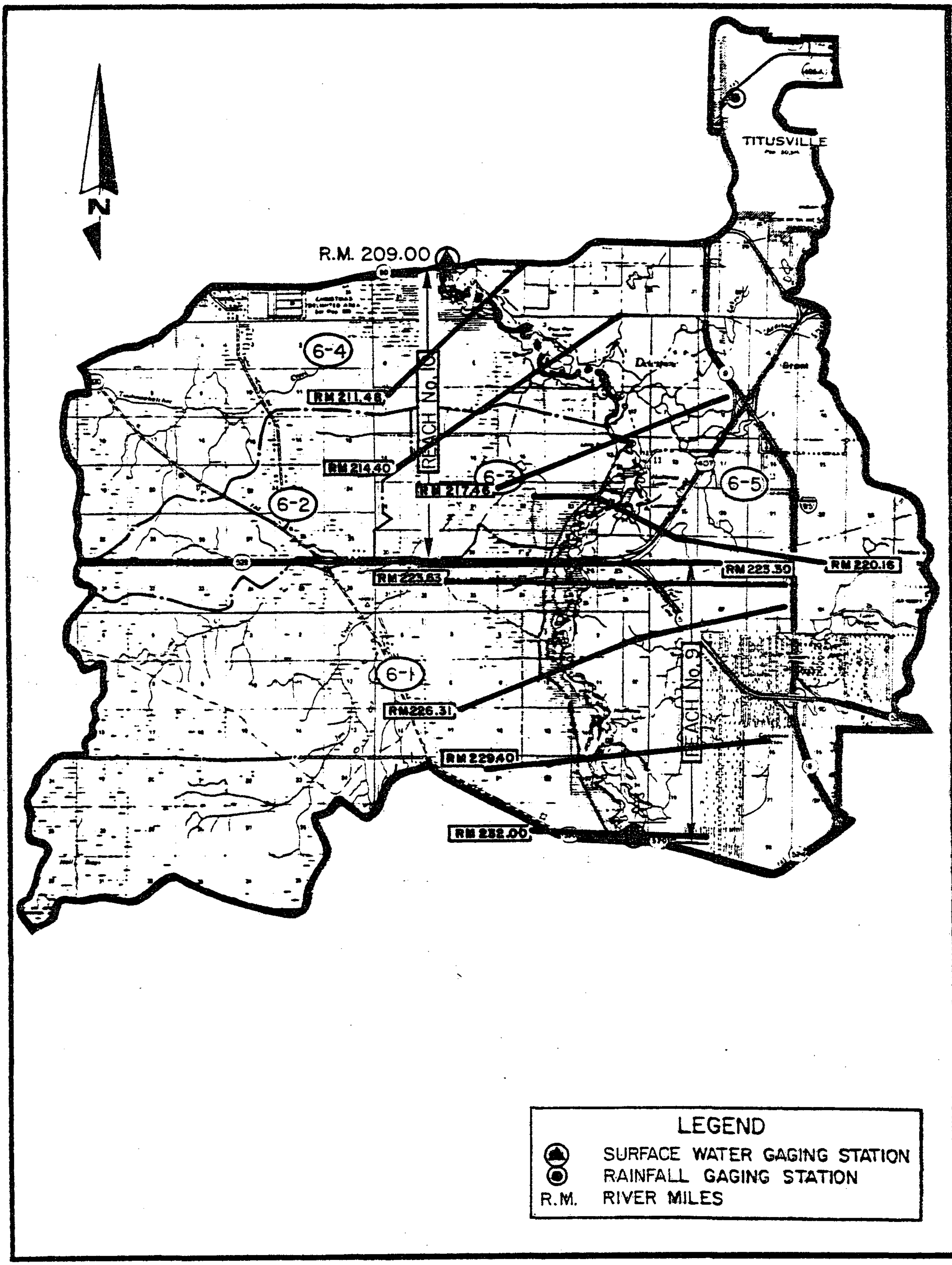


Figure 8. Planning Unit No. 6 Sub-basin, River Reach, and River Mile Designations.

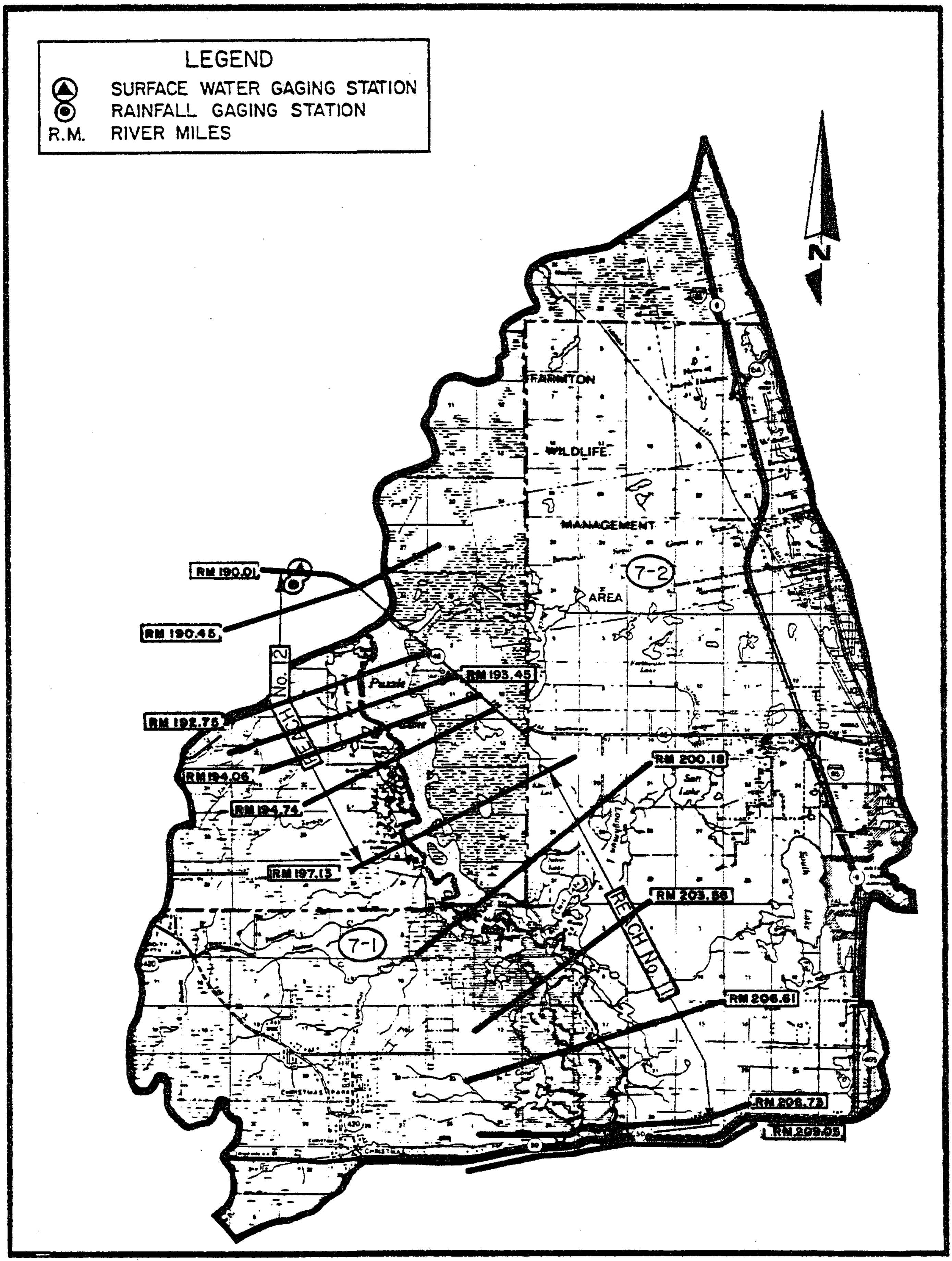


Figure 9. Planning Unit No. 7 Subbasin, River Reach, and River Mile Designations

St. Johns River and its watershed sub-basins under the existing conditions.

For each planning unit and its sub-basins, Part II of Ref. 3 presents complete details of physiography, hydrologic and hydraulic data, groundwater, land use, floodplain encroachment, water quality, ecology, and other information.

WATERSHED SIMULATION

A Continuous Streamflow Simulation Model has been developed to generate discharges and stage/storage data. Designated as the Upper St. Johns Hydrologic Model (USJHM), it consists of two main elements: a rainfall-runoff simulation routine (Figure 10) and a routing routine. The rainfall-runoff routine takes into account the basin evapotranspiration and continuously simulates soil moisture, surface retention and surface runoff by applying water balance method. Further details of the model may be found in References 2 and 3.

The model generates daily data for each sub-basin and for each of the 12 river reaches. Because of the flat topography of the river valley, each river reach is assumed to act as a reservoir receiving runoff from the adjacent sub-basin tributaries and the discharge from the upstream reach. Flows from an upstream reach discharge into the downstream reach based on a storage-discharge relationship (Puls Method). The stage-storage-discharge data for different reaches are developed by the HEC-2 program. Manning's roughness coefficients (used in HEC-2) for

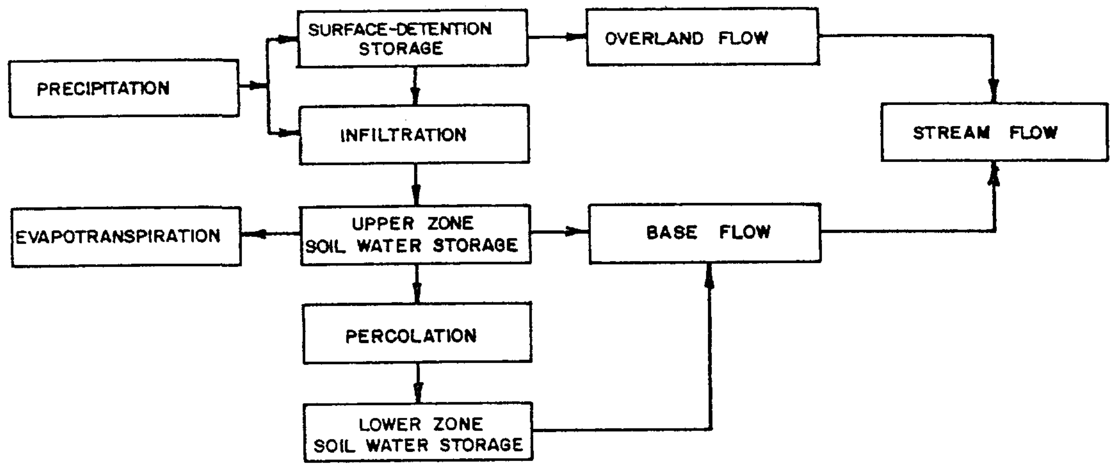


Figure 10. Flow Chart for USJHM Runoff Routine

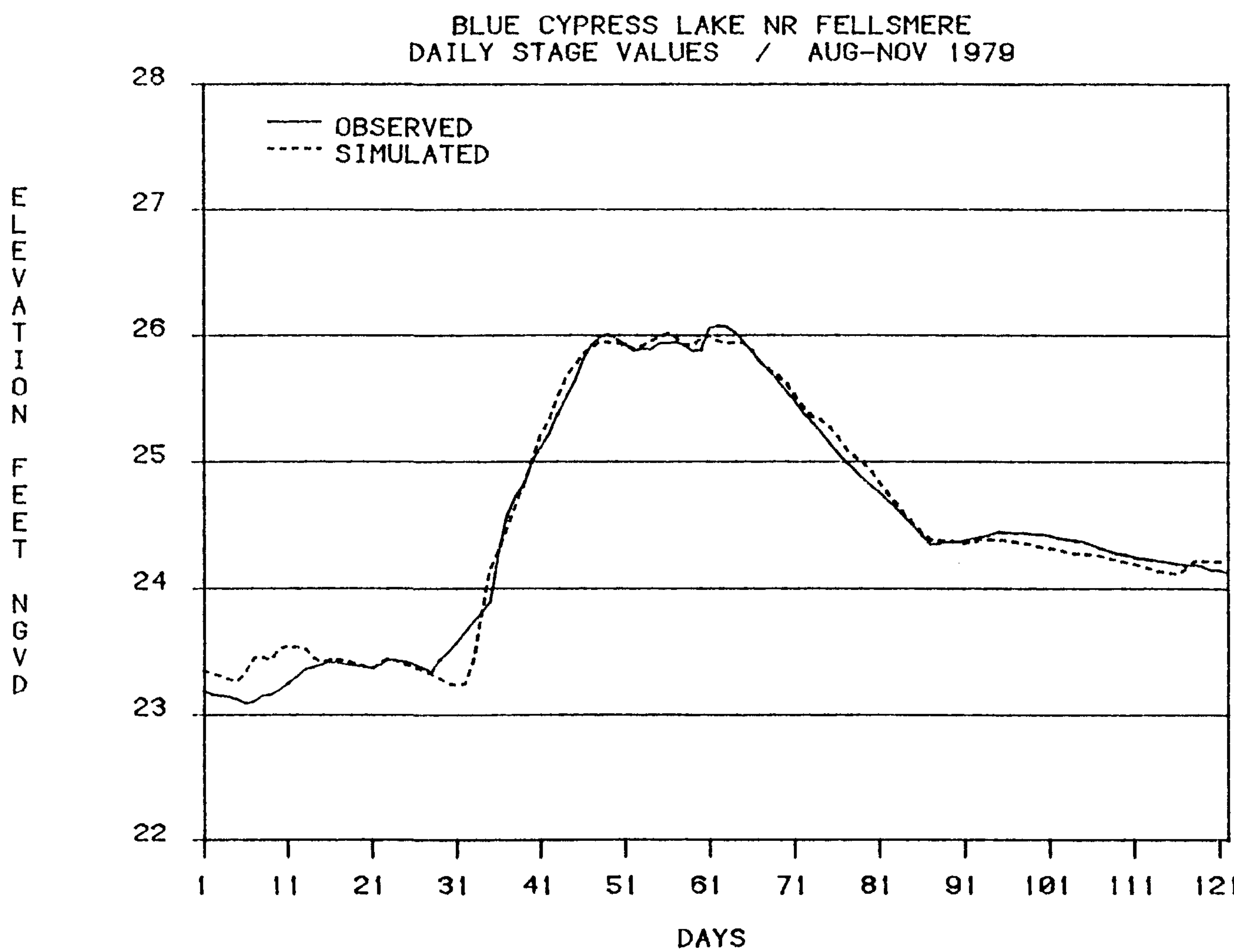


Figure 11. Observed and Simulated Stage Hydrographs for Blue Cypress Lake (Hurricane David)

the river channel and floodplain are calibrated against the flood elevations of major storms, especially Hurricane David of 1979.

The USJHM was calibrated for both runoff volumes and runoff hydrographs. Owing to the existence of Fellsmere Grade, the upper two reaches (Reach No. 1 and Reach No. 2, Figures 3 and 4) act primarily as impoundment areas. Thus, importance was given to stages and calibration was performed against stages. Figure 11 shows the observed and simulated stage hydrographs for Blue Cypress Lake for Hurricane David. For the same storm, Figure 12 shows discharge hydrographs for four other locations. Figures 11 and 12 show that, given adequate data support, the model has the ability of closely simulating the natural hydrologic events.

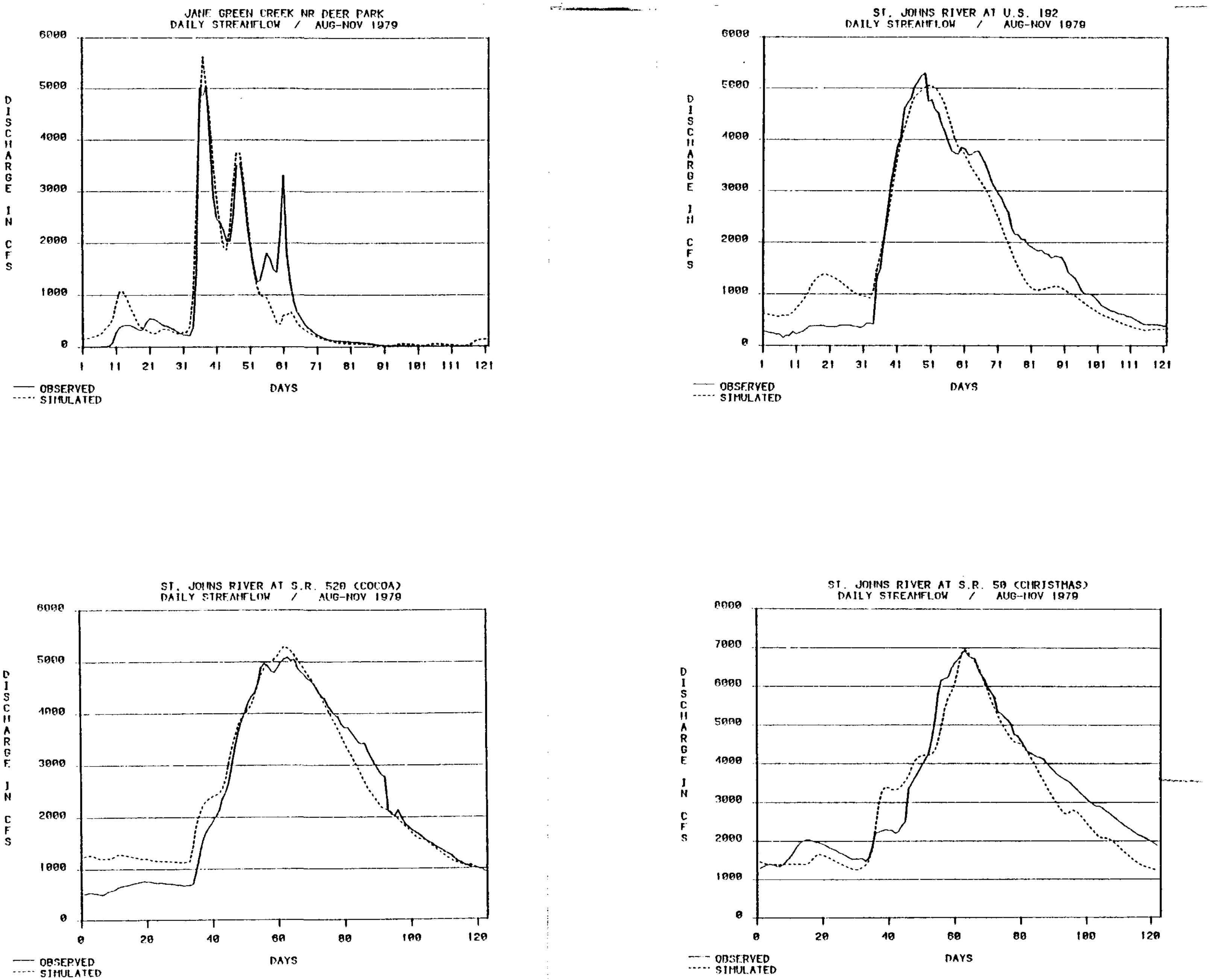


Figure 12. Observed and Simulated Hydrographs for Hurricane David (August-November 1979)

FLOOD ELEVATIONS

Flood elevations for different return periods have been determined based on simulated streamflows for the period of 1949-1983. These data represent the streamflows which might have occurred in the basin if the present conditions existed during 1949-1983. Thus, a homogeneity of data base is achieved by the simulation process. The period 1949-1983 was chosen for simulation because rainfall data were available from a reasonably adequate number of gauging stations during this time (eight stations during 1949-1977 and eighteen during 1978-1983). While it will be rather lengthy to list all the current basin conditions that have been simulated, some important features modeled are:

<u>Planning Unit (P.U.) No. 1 (Figure 3)</u>: St. Johns Water Control District (Sub-basin No. 1-6) was assumed to depend on its own surface water resources, a reservoir in the northwest corner of the sub-basin. No withdrawal was made from outside this reservoir. Excess flood waters are discharged into P.U. No. 2.

<u>Planning Unit No. 2 (Figure 4)</u>: Fellsmere Grade located just south of River Mile (R.M.) 283.03 impounds water in both P.U. No. 1 and P.U. No. 2. It has been assumed that 20,000 acres draw irrigation water from the impoundment. Rate of withdrawal depends upon the monthly evapotranspiration (ET) and effective rainfall (Re). Net irrigation requirement (NIR) for a given month is given by

$$NIR = ET - Re, \text{ in inches}$$

Effective rainfall is calculated by, (Ref. No. 5)

$$Re = (0.70917\ R_t^{\ 0.82416} - 0.11556)\ (10^{\ 0.02426\ ET})\ (f)$$

in which R_t = total monthly rainfall, and f = soil water storage factor which was assumed to be 0.93 in this study. Rainfall records at Fellsmere/Becker Groves were used to calculate R_t and ET values were obtained from the Institute of Food and Agricultural Sciences of the University of Florida, Gainesville. An irrigation efficiency of 75% was assumed and the average daily withdrawal was calculated by dividing monthly gross irrigation requirement by the number of days in the month.

Discharge from P.U. No. 2 to P.U. No. 3 takes place at two locations: 1) through a gap on the western side of Fellsmere Grade (Figure 13), and 2) through an outlet structure known as S-1. However, flow through gap is very low compared to S-1, e.g., it has a value of 0 cfs at 21.5 ft. NGVD, 70 cfs at 23.5 ft. NGVD, and 130 cfs at 24.5 ft. NGVD. To realize maximum storage benefits, Structure S-1 is closed when the Blue Cypress Lake falls below a certain elevation: 23.5 ft. NGVD, during June through September and 24.5 ft NGVD during October through May. Sudden opening or closure of structure S-1 is avoided to prevent undue environmental damage downstream, especially the fish kills. For this purpose, the stage-discharge schedule shown in Table 2 went into effect recently.

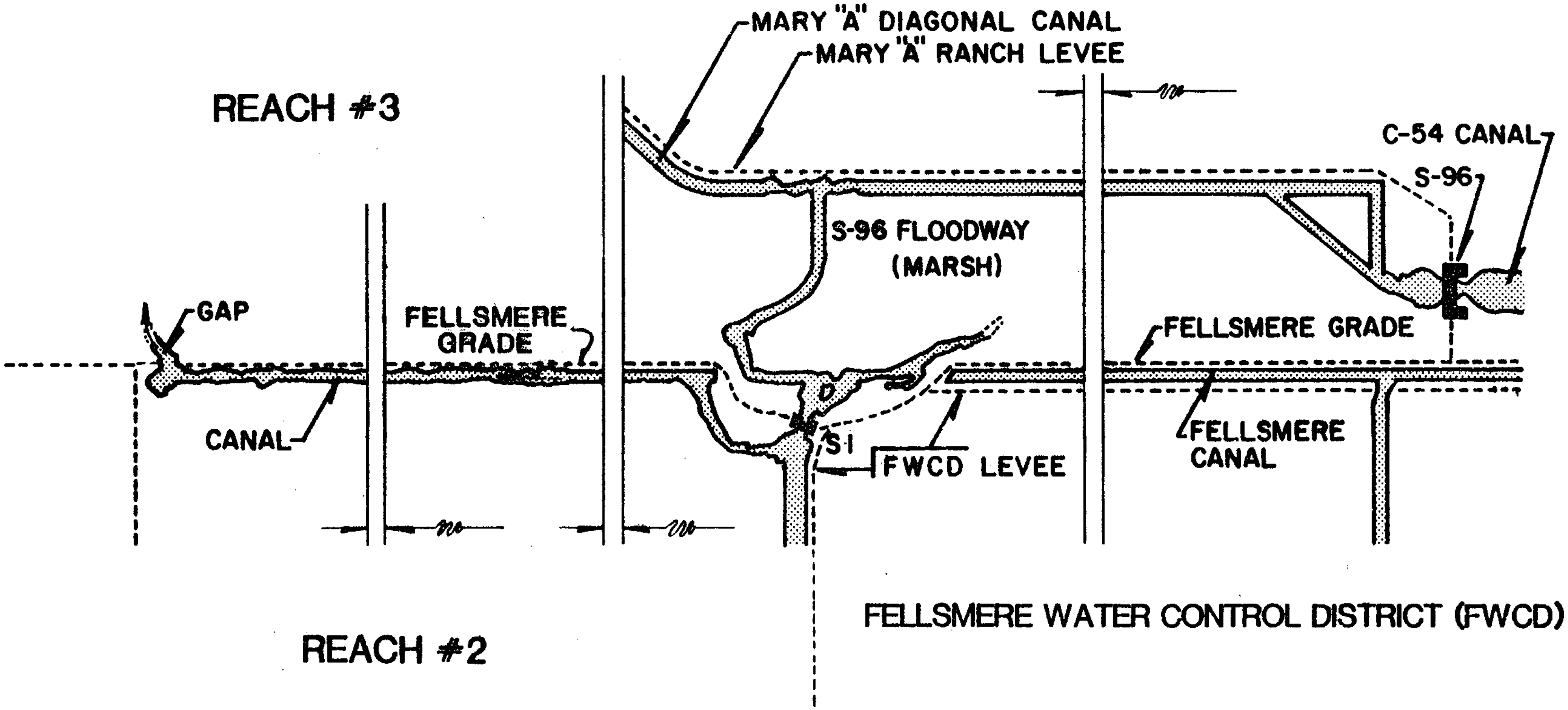


Figure 13. The Canal and Structure Configuration near Fellsmere Grade.

Table 2. Discharge Through Structure S-1, in cfs.

	Elevation in Blue Cypress Lake, ft. NGVD																
MONTH	23.0	23.1	23.2	23.3	23.4	23.5	23.6	23.7	23.8	23.9	24.0	24.1	24.2	24.3	24.4	24.5	>24.5
June 1 - Sept. 30	50	100	200	400	800	*	*	*	*	*	*	*	*	*	*	*	*
Oct. 1 - Oct. 10	0	0	0	0	50	100	200	400	800	*	*	*	*	*	*	*	*
Oct. 11 - Oct. 20	0	0	0	0	0	0	0	50	100	200	400	800	*	*	*	*	*
Oct. 21 - Oct. 31	0	0	0	0	0	0	0	0	0	50	100	200	400	800	*	*	*
Nov. 1 - April 30	0	0	0	0	0	0	0	0	0	0	50	100	200	400	800	*	*
May 1 - May 3	0	0	0	0	0	0	0	0	0	50	100	200	400	800	*	*	*
May 4 - May 6	0	0	0	0	0	0	0	0	50	100	200	400	800	*	*	*	*
May 7 - May 9	0	0	0	0	0	0	0	50	100	200	400	800	*	*	*	*	*
May 10 - May 12	0	0	0	0	0	0	50	100	200	400	800	*	*	*	*	*	*
May 13 - May 15	0	0	0	0	0	50	100	200	400	800	*	*	*	*	*	*	*
May 16 - May 18	0	0	0	0	50	100	200	400	800	*	*	*	*	*	*	*	*
May 19 - May 21	0	0	0	50	100	200	400	800	*	*	*	*	*	*	*	*	*
May 22 - May 25	0	0	50	100	200	400	800	*	*	*	*	*	*	*	*	*	*
May 25 - May 31	0	50	100	200	400	800	*	*	*	*	*	*	*	*	*	*	*

* Maximum discharge (Gate fully open.)

If the above table indicates that discharges must be reduced, then flow reductions shall adhere to the following constraints:

1) When flow through the structure is greater than 800 cfs, flow may be reduced abruptly to 800 cfs.
2) When flow is between 800 and 100 cfs, flow will be reduced gradually, at a maximum rate of 100 cfs every 72 hours.
3) When flow is less than or equal to 100 cfs and has been maintained for 72 hours, flow may be reduced to 50 cfs.
4) Gates may be closed after discharging at 50 cfs for 72 hours.

Sub-basin No. 2-9, which is mostly owned by the Fellsmere Joint Venture, diverts about 60% of its drainage to Indian River via Fellsmere Canal (Figure 13) during wet season.

Planning Unit No. 3 (Figure 5): To prevent downstream flooding, a portion of the discharge received from the P.U. No. 2 can be diverted to the Indian River through Canal C-54 by operating Structure S-96 (Figure 13). Elevation in the Blue Cypress Lake governs the operation of Structure S-96 as follows: The structure is opened when the lake elevation exceeds 25.2 ft. NGVD. The diverted discharges vary from 1500 cfs at EL 25.2 ft. NGVD to 2400 cfs at EL 28.0 NGVD. Above EL 28.0 NGVD the discharge remains constant at 2400 cfs.

Drainage from Sub-basins 3-6, 3-13, and 3-17 is completely diverted to the Indian River. Partial impoundment of Jane Green Creek flood discharge occurs behind the levee L-73 which extends from P.U. No. 3 through P.U. No. 5. L-73 was constructed as a part of an earlier project (construction of which was halted later on) to control flood waters from the western upland tributaries. The levee, however, has several gaps and, thus, is not fully effective.

Planning Unit No. 4 (Figure 6): A discharge of 27 cfs (17.45 mgd) is withdrawn from Lake Washington for municipal use.

Planning Unit No. 5 (Figure 7): The Cox Creek and Taylor Creek waters are fully impounded behind Levee L-73 and discharge takes place through a control structure. The Pennywash and Wolf creeks are uncontrolled.

Planning Unit Numbers 6 and 7 (Figures 8 and 9): These two basins do not have any modeling features requiring special attention.

The flood stages for P.U. No. 1 and P.U. No. 2 were determined by converting daily storage data into stage data and then conducting a frequency analysis on the annual peak stage data. This procedure was used because the river valley in these two planning units act primarily as an impoundment area. For P.U. No. 3 through P. U. No. 12 flood profiles were computed by the U. S. Army Corps of Engineers' HEC-2 water surface profiles program. Flood discharges for different return periods were determined by Log Pearson Type 3 method described in Ref. 1. Annual peak flows for this purpose have been picked up from the simulated data using June 1 to May 31 as the reference year instead of the regular water year (October to September). In this part of Florida wet season continues through October. Thus, the customary water year causes a break in the annual flooding season which sometimes results in picking up two values from the same flooding season as annual peak flows. Choosing June-May as reference year will prevent this possibility. Table 3 presents flood elevations for some important locations, and also the highest observed flood elevations and low chord elevations of different bridges.

Figures 3 through 9 show the location of some of the river cross-sections used in the HEC-2 program. These are designated by their river miles. The mean annual, 10-year, 25-year, and 100-year flood profiles are plotted on Figures 14 through 18. Some minor discrepancies may be found between the results presented in this report and other agency publications, e.g., U. S. Army Corps of Engineers, etc. Such discrepancies may be attributed primarily to varying methodologies selected in deriving various results.

Table 3. Flood Elevations in the Upper St. Johns River Under 1984 Basin Conditions (Feet NGVD).

Reach No.	Location	River Mi.	Mean Annual	Return Period 5-Yr.	10-Yr.	25-Yr.	50-Yr.	100-Yr.	Observed Max. El.	Low Chord of the Bridge
1	State Rd. 60	295.42	25.5	26.1	26.6	27.2	27.6	28.0	27.78	27.50
2	Blue Cypress Lake	287.75	24.8	25.4	26.0	26.7	27.2	27.8	26.7	
3	Below Fellsmere Grade	283.03	23.7	24.1	24.3	24.4	24.5	24.6		
3	Downstream End of Reach No. 3	277.06	19.1	19.5	19.9	20.4	20.8	21.1		
4	Downstream End of Reach No. 4	270.31	17.9	18.6	19.2	19.9	20.4	20.9		
5	State Rd. 500/ U.S. 192	262.00	17.4	18.2	18.8	19.5	20.0	20.5	20.88	24.70
6	Lk. Washington Weir	254.59	17.0	17.8	18.4	19.1	19.5	19.9	20.39	
7	Below Lake Winder	240.25	15.8	16.6	17.2	17.8	18.1	18.5		
8	State Road 520	232.03	15.4	16.3	16.8	17.3	17.6	17.9	16.96	18.05
9	State Road 528	223.30	13.0	13.9	14.3	14.9	15.2	15.4		30.75
10	State Road 50	209.03	9.7	10.6	11.0	11.6	12.0	12.4	12.43	21.50
11	Downstream End of Reach No. 5	197.13	7.6	8.7	9.4	10.3	10.7	11.2		
12	State Road 46	190.01	6.3	7.6	8.5	9.4	9.6	10.4	10.62	28.50

Figure 14. MEAN ANNUAL, 10 YR, 25 YR, AND 100 YR FLOOD PROFILES FOR THE UPPER ST JOHNS RIVER (R.M. 190 TO R.M. 225)
(UNDER 1984 BASIN CONDITIONS)

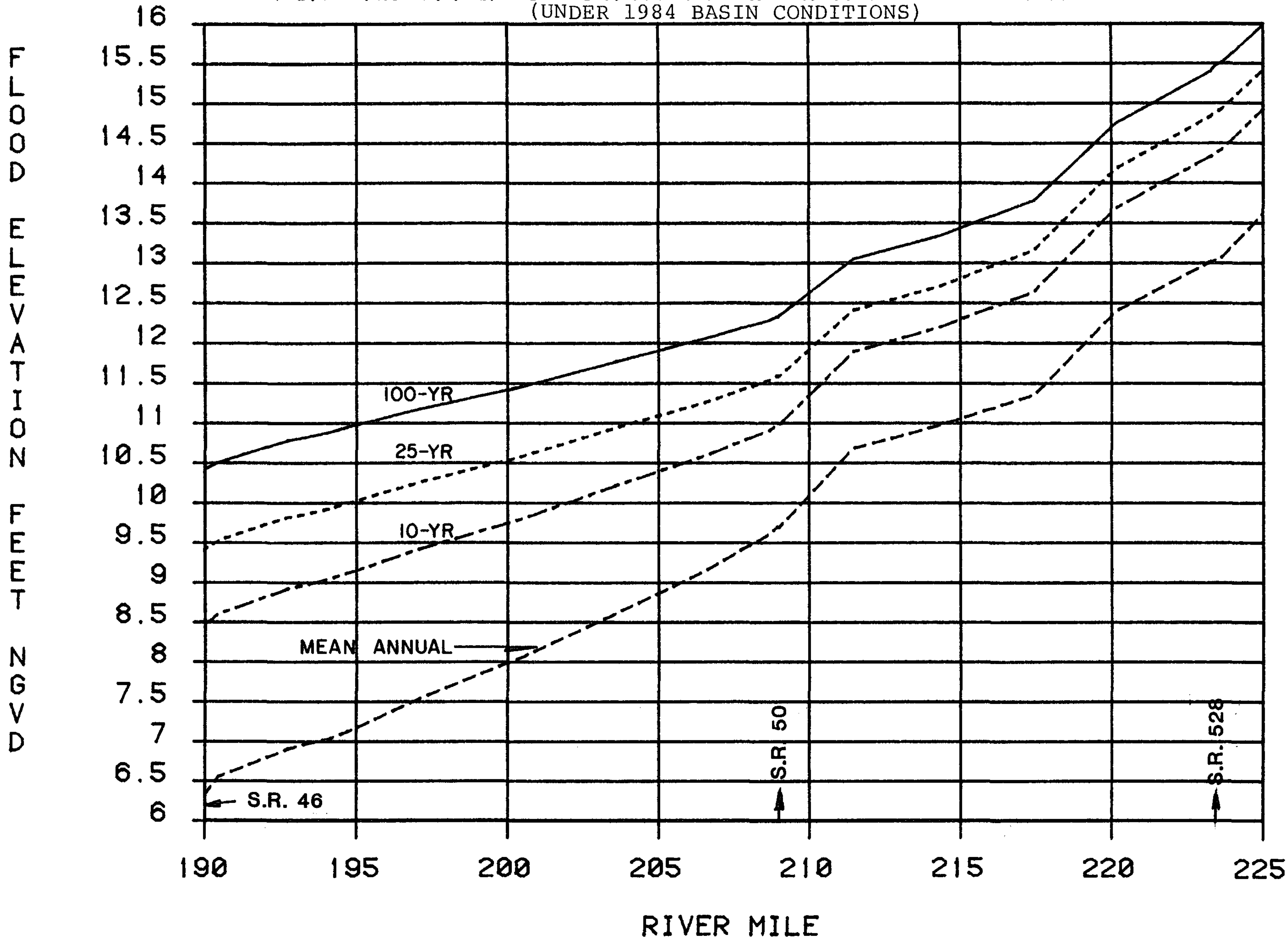


Figure 15. MEAN ANNUAL, 10 YR, 25 YR, AND 100 YR FLOOD PROFILES FOR THE UPPER ST JOHNS RIVER (R.M. 220 TO R.M. 245)
(UNDER 1984 BASIN CONDITIONS)

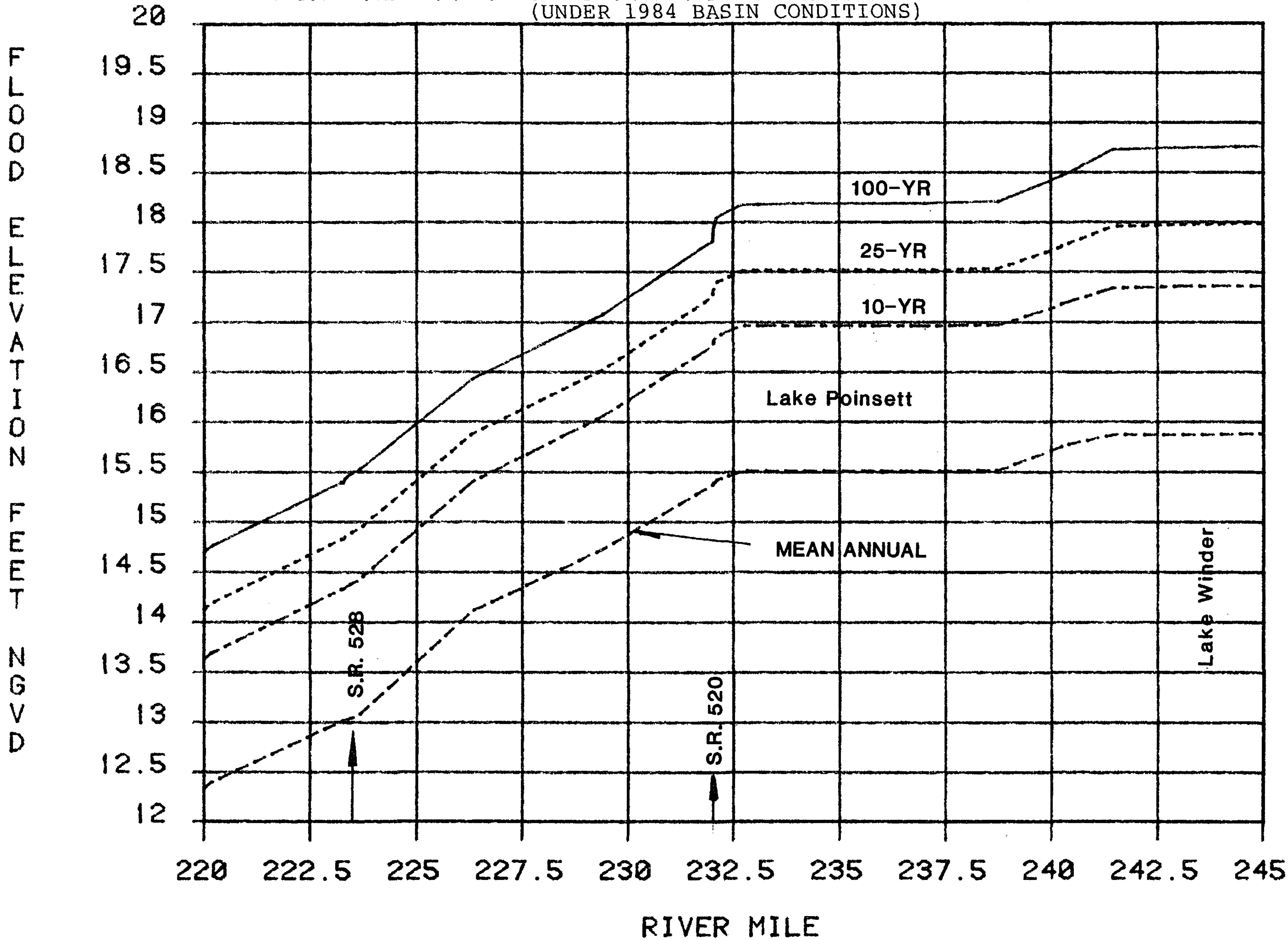


Figure 16. MEAN ANNUAL, 10 YR, 25 YR, AND 100 YR FLOOD PROFILES FOR THE UPPER ST JOHNS RIVER (R.M. 240 TO R.M. 265)
(UNDER 1984 BASIN CONDITIONS)

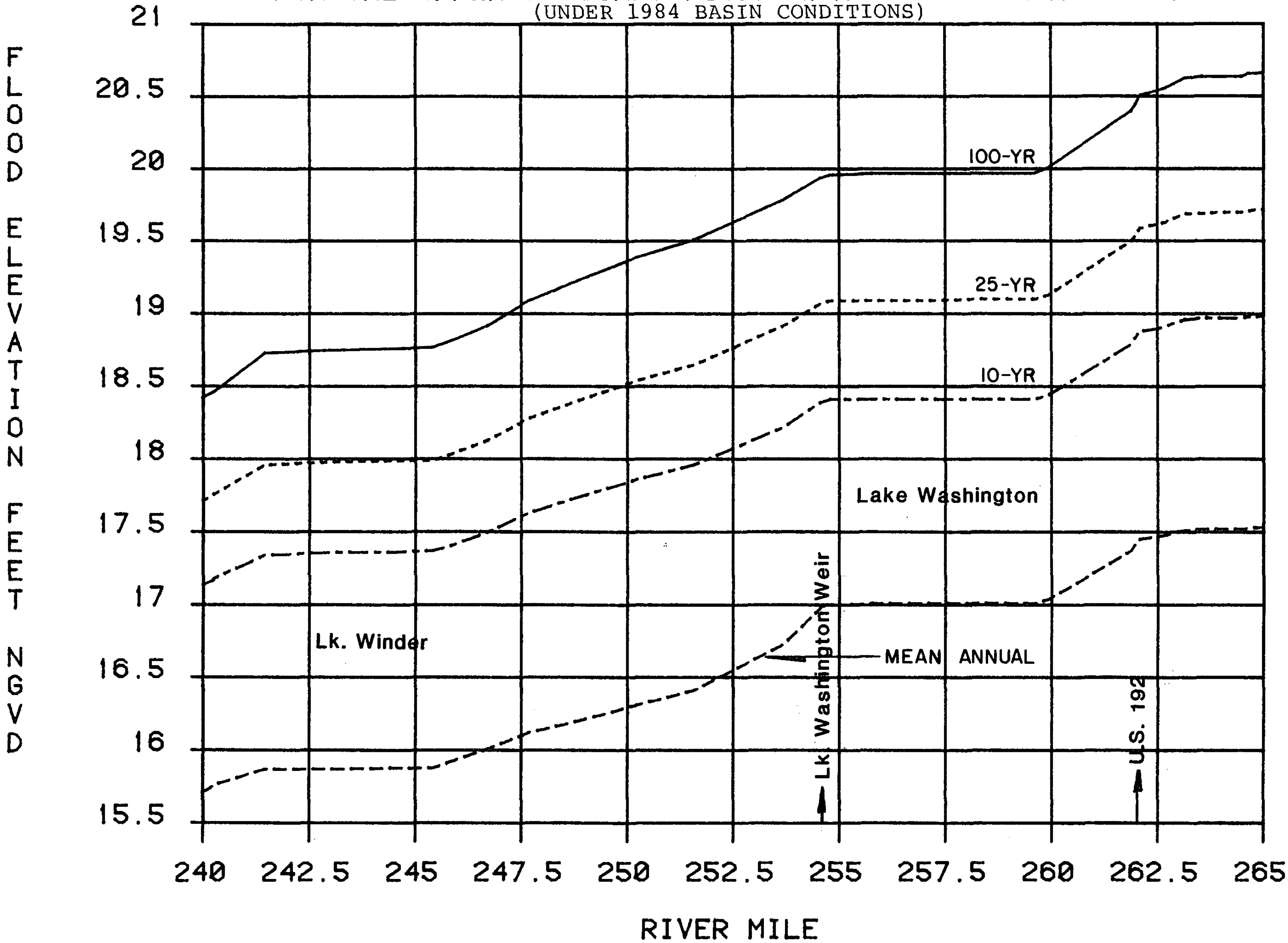


Figure 17. MEAN ANNUAL, 10 YR, 25 YR, AND 100 YR FLOOD PROFILES FOR THE UPPER ST JOHNS RIVER (R.M. 260 TO R.M. 280)
(UNDER 1984 BASIN CONDITIONS)

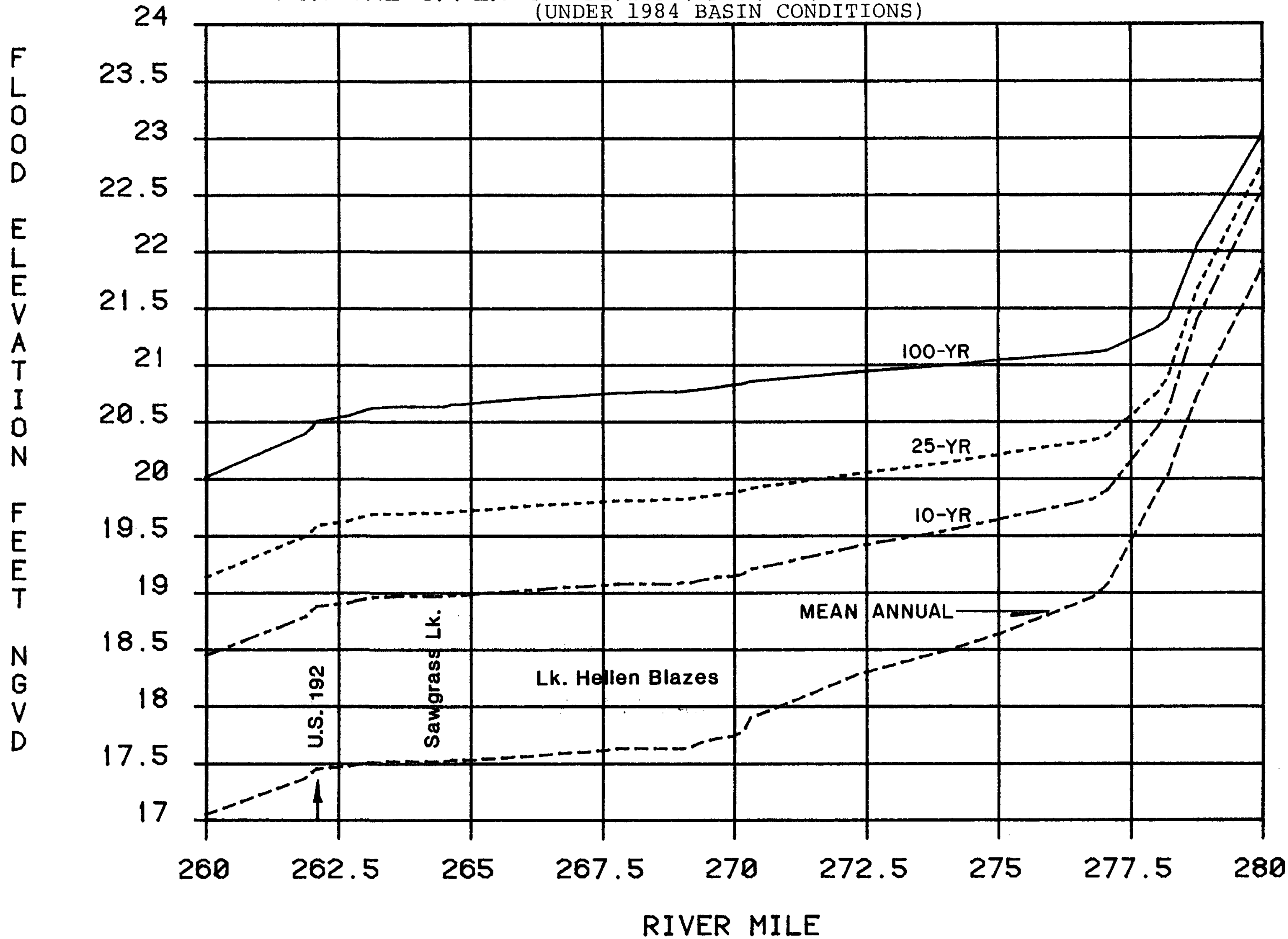
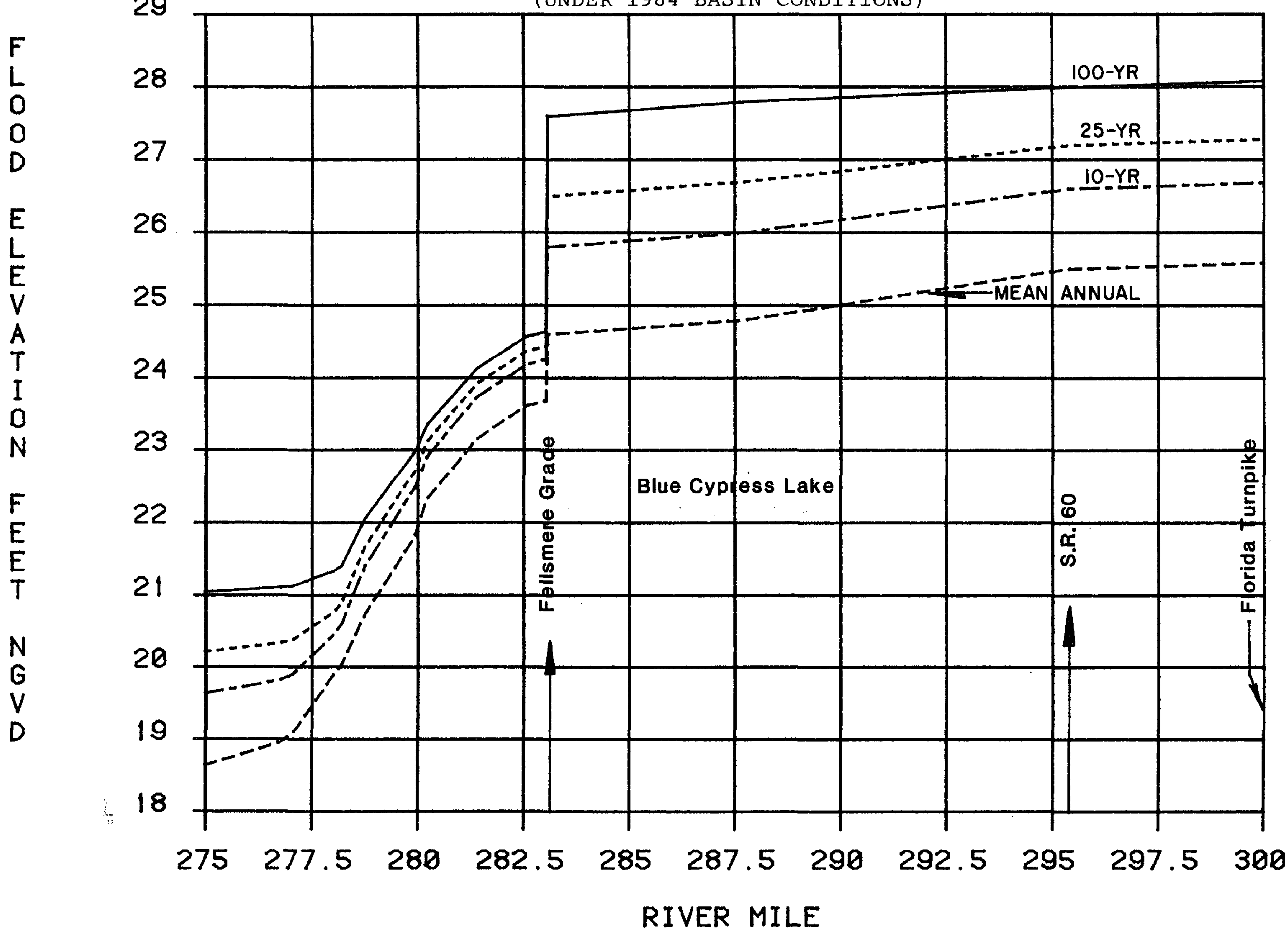


Figure 18. MEAN ANNUAL, 10 YR, 25 YR, AND 100 YR FLOOD PROFILES FOR THE UPPER ST JOHNS RIVER (R.M. 275 TO R.M. 300)
(UNDER 1984 BASIN CONDITIONS)



SUMMARY

The Upper St. Johns River Basin has been drastically impacted by man's activities since the turn of the century. This process has been continuous and affected both high flows and low flows in the basin. Long-term records of stage and/or discharge are available at seven locations along the river. Prediction of future maximum events based on this data, however, will not be satisfactory because the hydrologic parameters of the basin were not stationary during the record period.

Synthetic daily streamflows have been generated for a period of 35 years (1949-1983) by a continuous hydrologic simulation model. The model took into consideration all current conditions in detail. These include the current land use, basin boundaries, river cross-sections, basin alterations by dikes and ditches, discharge schedules at different drainage structures, interbasin diversion, pumpage, irrigation and municipal water use, etc. Annual peak flow (or stage) data have been compiled for 12 locations between State Road 46 and Florida Turnpike. Frequency analyses have been conducted to determine mean annual, 10-year, 25-year, and 100-year flood flows (or stage) at these 12 locations. Finally, flood profiles have been computed (for the preceding four frequencies) for the St. Johns River at State Road 46 to its headwaters near Florida Turnpike by a step backwater computer program (HEC-2). These flood profiles are useful to determine flood elevations for any location in the Upper St. Johns River flood valley under the existing conditions.

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