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LAKE LOWERY BASIN

SURFACE WATER MANAGEMENT STUDY

By

Gary Bethune

Engineer

and

C. Charles Tai, Ph.D, P.E.

Project Manager

Division of Engineering

Department of Water Resources

St. Johns River Water Management District

P.O. Box 1429

Palatka, Florida 32078-1429

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ACKNOWLEDGEMENT

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INTRODUCTION

BACKGROUND

Lake Lowery is located near Haines City in north-central Polk County. It lies within the Palatlakaha River Basin, one of two watersheds forming the headwaters of the Oklawaha River, a tributary of the St. Johns River (Figure 1). The lake is situated in the southeast corner of the Green Swamp, which is an area of approximately 870 square miles consisting of wetlands interspersed with pine flatwoods, low sand ridges, and sinkhole lakes. The Lake Lowery study area is bounded by Interstate 4 to the north, U.S. Highways 17 and 92 to the south, U.S. Highway 27 to the east, and State Road 557 to the west. The total area is approximately 55 square miles, all of which is within Polk County.

During the later half of 1982, lake water levels rose to relatively high stages and remained there until late 1984 (Figure 2). A residential area located on the north shore experienced localized flooding, primarily resulting in septic system failures. In response to concerns expressed by residents of the flooded area and a request made by the Polk County Board of County Commissioners, a study was undertaken by the St. Johns River Water Management District (SJRWMD).

PURPOSE AND SCOPE

The purpose of this water management study is to develop and evaluate solutions to the problem of localized flooding. The correct identification and accurate assessment of the factors which interact with Lake Lowery's surface water system was the

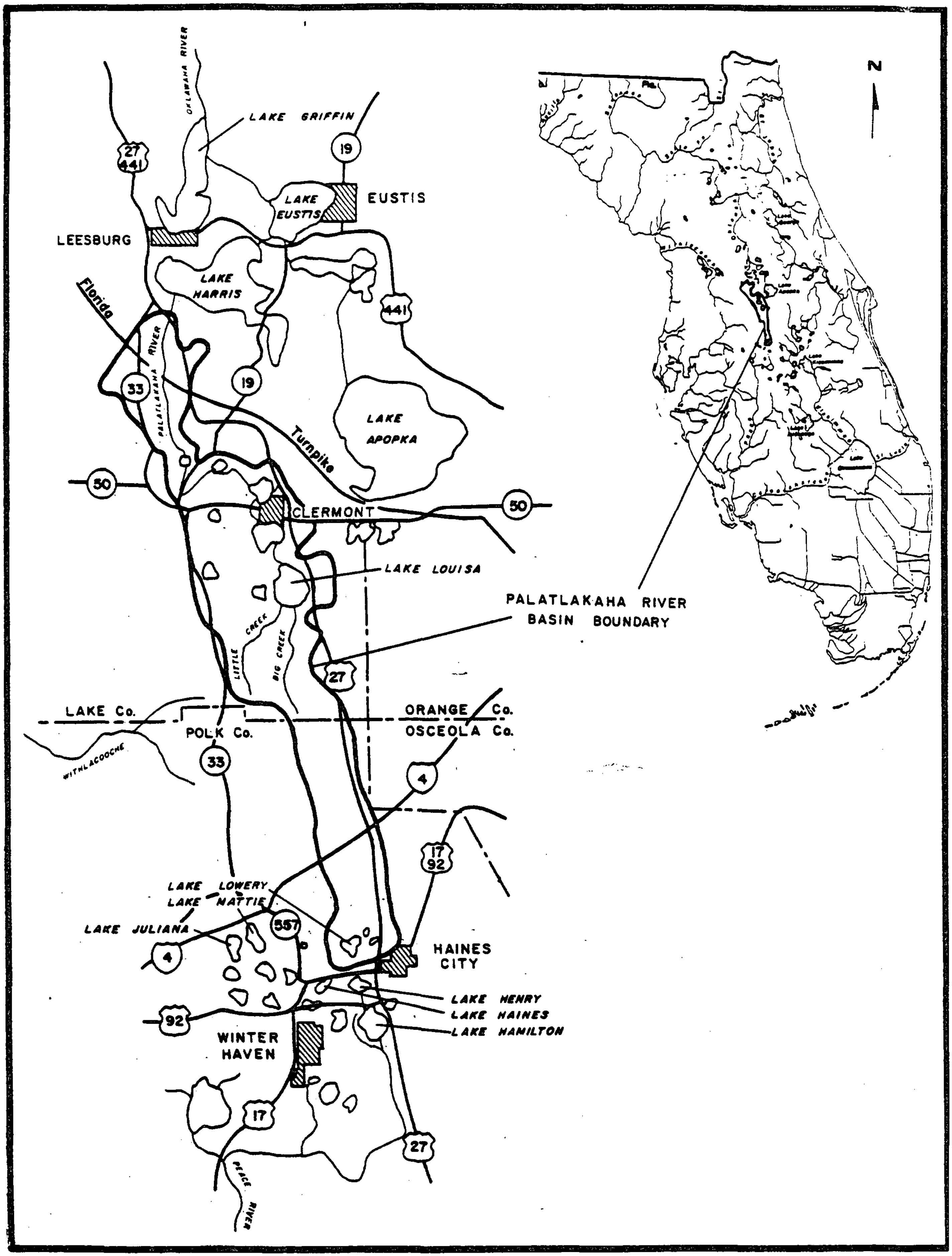


Figure 1. Location of the Study Area and the Palatlakaha River Basin.

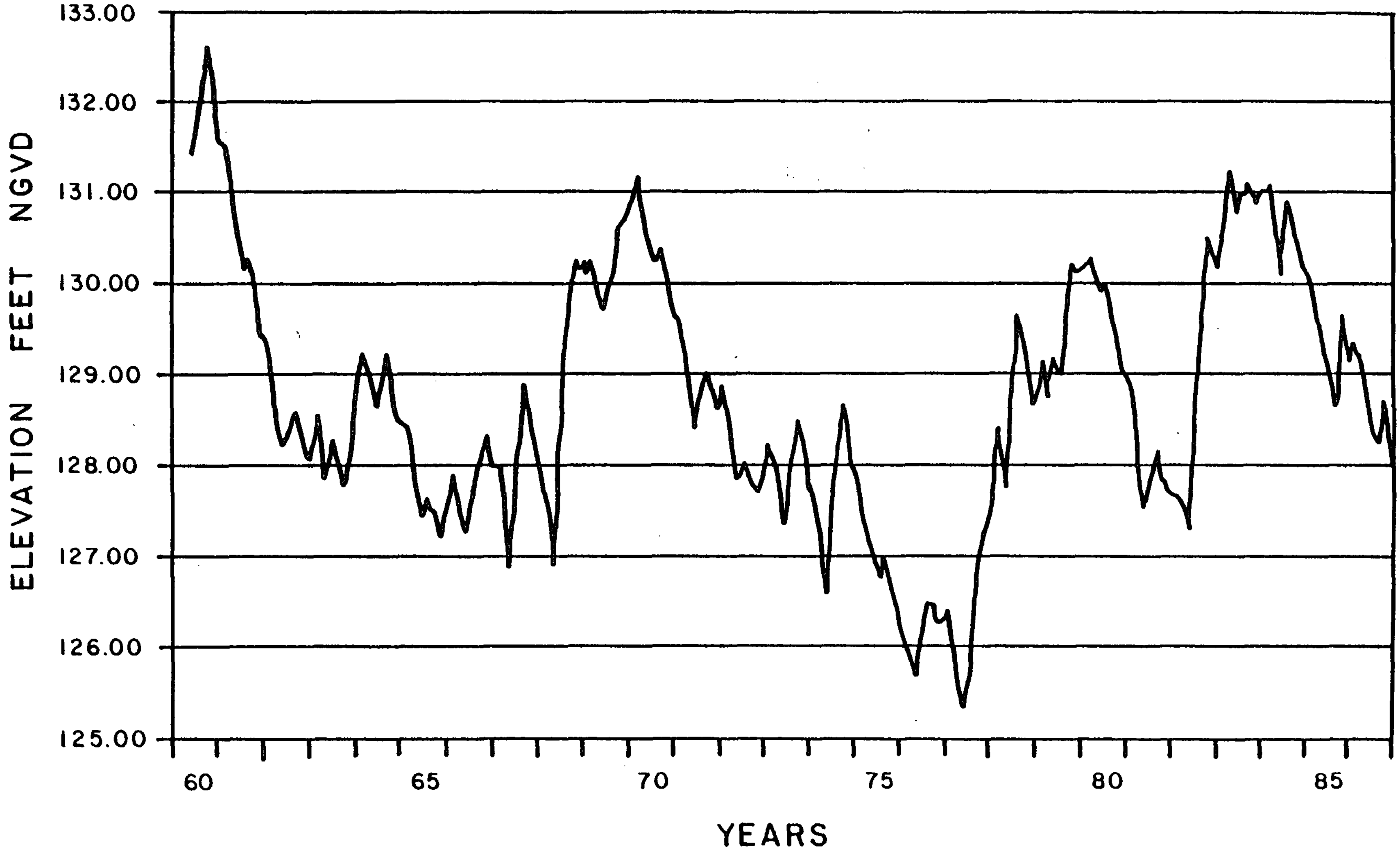


Figure 2. Lake Lowery Monthly Mean Stage (1960-1986)

initial step in the derivation of possible solutions. Socio-economic and hydrologic conditions of the study area were recognized as factors which must be thoroughly understood and quantified in order to develop alternative solutions. Each design configuration was analyzed for its hydrologic and socio-economic impact. The conclusions were summarized and the best water resources management alternative was identified.

SOCIO-ECONOMIC DESCRIPTION

POPULATION DISTRIBUTION

The area surrounding Lake Lowery is predominantly rural and much of the land remains in its natural state -- marshes and wet prairies. Interspersed among the wetlands are established citrus groves located on sandhill uplands (Figure 3). All of the land surrounding the lake is privately owned and some of the wetlands have been cleared for pasture. The entire region has experienced an increase in population recently with the development of new residential areas. Table 1 provides information on population growth for the area. A trend indicating that population growth in the rural areas is faster than in the urban areas can be inferred from Table 1, shown in Figure 4.

The residential development occurring on the north shore of the lake has expanded from a few isolated mobile homes in the 1960's to approximately 170 residences at present. About 115 (68 percent) are mobile homes. Most residences are owner occupied; however, some are rental and/or vacation homes. Out of 140 acres in the area suitable for development, based on existing patterns, about 80 acres have been subdivided into small lots which contain 145 (85 percent) of the residences. This area is predominantly lake and canal frontage. The remaining 60 acres are less developed, larger tracts. Assuming the underdeveloped areas progress toward a density similar to that already developed, the area when fully developed could contain 265 single family residences.

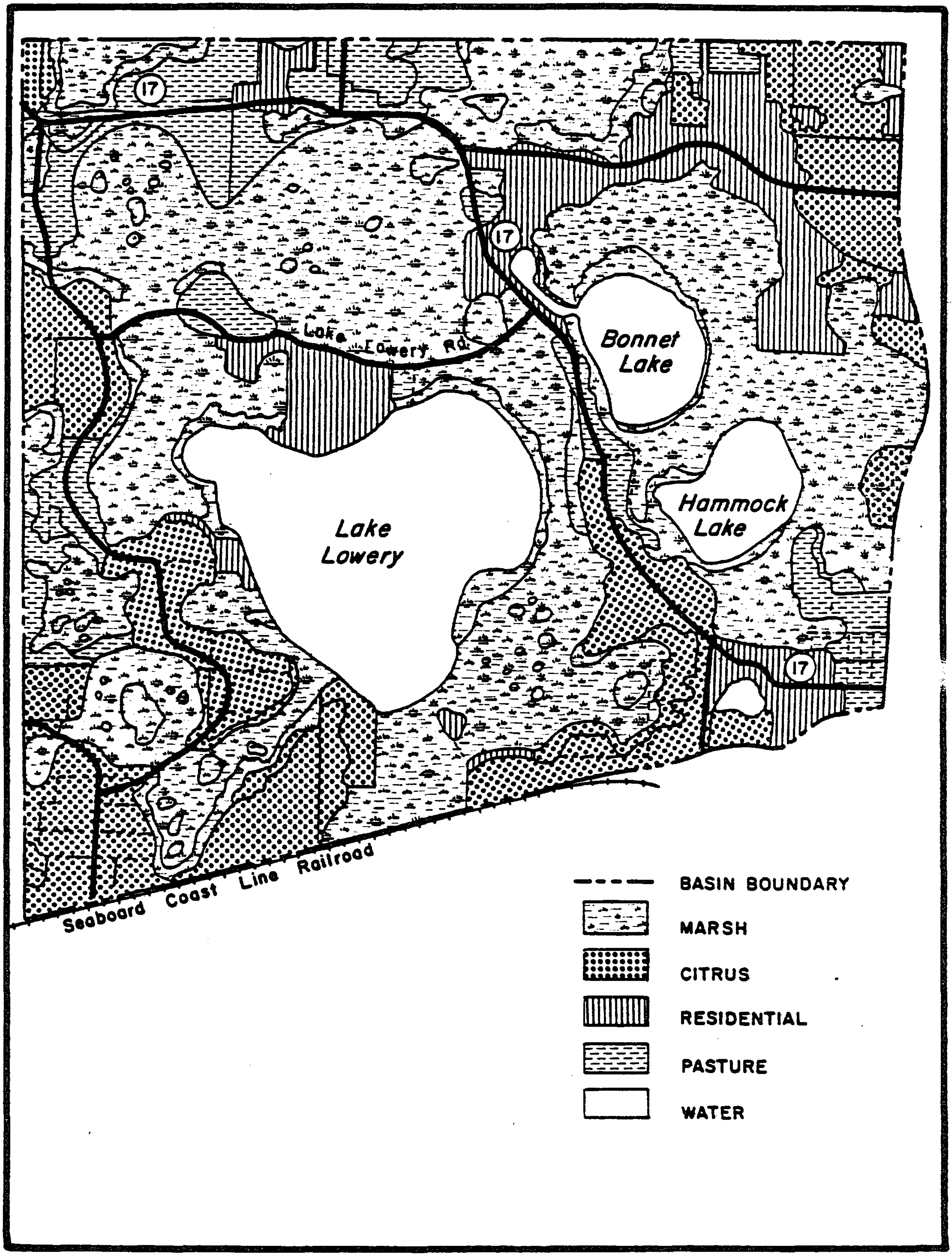


Figure 3. Lake Lowery Land Use Map

Table 1. Population Growth in the Study Area

Year	Polk County	Lake Alfred	Haines City	Unincorporated Areas
1960	195,139			
1970	228,515	2,847	8,956	114,894
1975	275,973	3,184	9,481	150,077
1980	321,632	3,134	10,799	190,071
1982	338,865	3,256	11,488	200,937
1990	401,700			
2000	455,697			

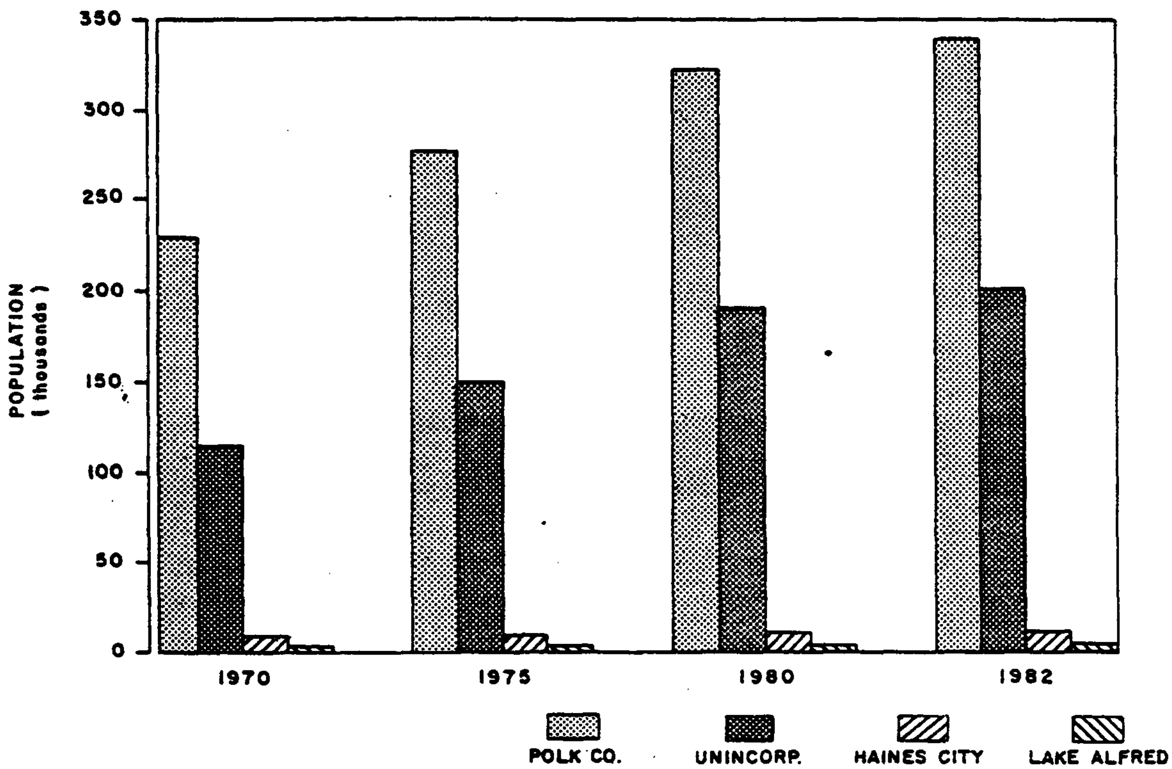


Figure 4. Polk County Population Trends

Access to the area is via a paved county road -- Lake Lowe Road (Figure 5). Portions of the road west of Lake Lowery a: estimated to have been built in the 1930's; the segment north c Lake Lowery linking it with Polk County Road 17 was built afte 1960. The county maintains Lake Lowery Road, while all othe roads are privately maintained.

East of Lake Lowery, around Bonnet and Hammock lakes (Figur 3), is a second large residential area. Approximately 150 to 20(residences are currently located there. Other population areas exist on the west and south side of the lake. These areas include about ten and five residences respectively.

WATER USES AND DISPOSAL

Residents in the area use individual wells tapping the Floridan aquifer for potable water supply. Sewage disposal is by septic tanks with drain fields.

Lake Lowery serves as a multiple use reservoir. There are currently two recreational facilities located on the lake -- a fish camp and a camping resort. The lake also provides irrigation water to adjacent citrus groves. Currently five consumptive use permits have been issued for withdrawal of surface water from the lake. The permitted total maximum daily withdrawal of water from the lake is 2.35 million gallons per day with a permitted annual average withdrawal of 83,500 gallons per day.

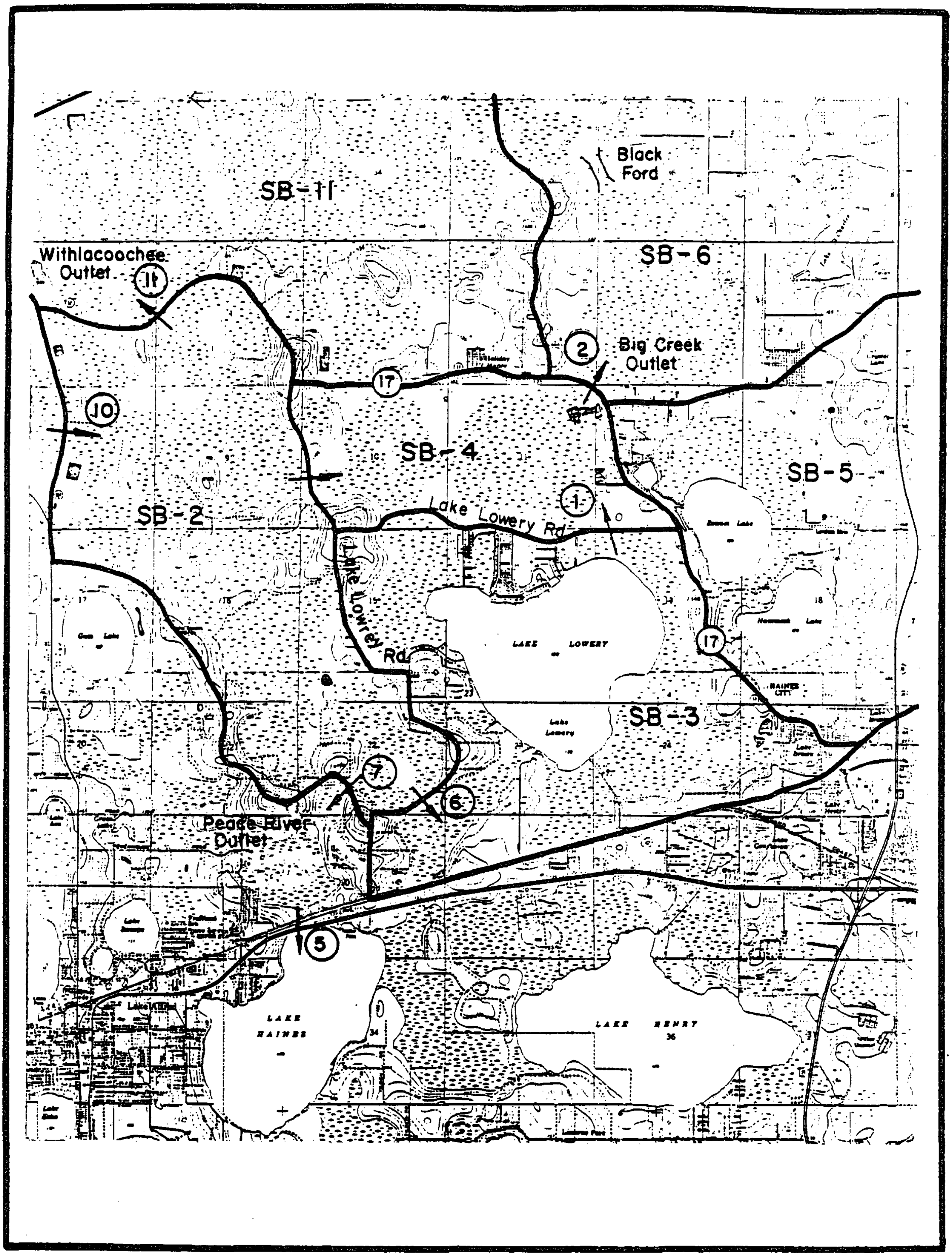


Figure 5. Lake Lowery Study Area.

HYDROLOGIC DESCRIPTION

TEMPERATURE

The Lake Lowery area is located near latitude 28° 06' North and longitude 81° 40' West. The climate of the area is characterized by long, warm, humid summers and mild, dry winters. Summer temperatures are fairly uniform from year to year and show little daily variation. Winter temperatures vary considerably from day to day due to periodic cold fronts invading from the north.

PRECIPITATION

An average of 64 percent of the annual precipitation falls during the months May through September, while the remaining 36 percent is nearly evenly distributed throughout the rest of the year. A long term rainfall record near the Lake Lowery area was obtained from the National Oceanic and Atmospheric Administration's Lake Alfred station. Located three miles southwest of Lake Lowery, the Lake Alfred station has recorded daily precipitation since 1924 (Table 2). Monthly normal rainfall, Class "A" Pan evaporation, and mean temperature from the Lake Alfred station are presented in Table 3.

Significant differences in annual precipitation have been recorded at the Lake Alfred Station. A maximum annual rainfall of 76.57 inches was recorded in 1959 and a minimum annual rainfall of 35.12 inches was recorded in 1932. The 60 years of recorded rainfall from the Lake Alfred Station is presented

Table 2. Monthly and Annual Rainfall Data for Lake Alfred

RAINFALL IN INCHES - LAKE ALFRED EXP NOAA NUMBER: 4707

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YRLY
1925	7.62	1.20	1.92	0.72	6.95	9.25	13.30	10.89	0.82	1.69	1.87	5.33	61.56
1926	5.76	2.00	4.16	5.45	5.09	10.07	9.89	9.54	4.77	0.64	2.95	0.65	60.97
1927	0.00	2.86	2.92	0.54	1.22	6.74	6.75	10.78	2.81	1.88	0.78	0.89	38.17
1928	0.64	0.80	2.80	5.67	3.43	5.24	6.68	13.92	14.25	1.66	0.13	0.31	55.53
1929	1.54	2.13	0.88	1.21	8.65	7.98	7.06	3.42	12.37	3.53	3.09	1.96	53.82
1930	2.32	3.72	6.05	2.17E	3.69	8.61	2.17	3.38	9.58	1.65	2.29	4.47	50.10E
1931	2.47	1.03	5.02	7.67	2.46	3.66	4.77	6.92	5.03	0.90	0.06	1.80	41.79
1932	1.57	0.23	3.71	0.25	6.98	9.31	2.65	4.16	3.14	1.49	1.63	0.00	35.12
1933	1.59	3.11	1.83	3.85	4.65	6.82	6.47	6.70	17.25	0.97	1.08	0.20	54.52
1934	0.82	3.39	4.12	7.66	6.00	21.27	8.92	4.92	6.23	0.50	0.26	0.63	64.72
1935	0.77	1.16	0.18	1.75	3.96	5.89	4.78	5.32	11.65	0.50	0.73	3.65	40.34
1936	3.37	7.03	4.27	3.40	3.72	2.96	5.97	10.41	3.57	5.67	1.29	1.53	53.19
1937	0.10	6.83	2.86	2.90	4.33	8.18	10.63	4.19	2.18	6.17	3.05	1.10	53.32
1938	0.84	0.60	2.69	0.10	4.50	5.20	7.36	5.66	4.68	5.25	0.50	0.12	37.50
1939	1.24	0.39	2.05	5.93	8.77	14.03	6.42	12.16	5.26	1.12	0.26	1.28	58.91
1940	2.34	4.21	3.11	2.85	0.71	5.90	6.14	4.79	4.23	0.54	0.10	4.15	39.07
1941	4.36	4.32	3.11	6.82	0.74	8.13	10.66	4.10	4.18	2.05	3.54	5.18	57.19
1942	2.28	3.55	6.19	2.68	2.24	11.11	3.50	4.30	6.65	0.10	0.33	1.92	44.85
1943	0.90	1.42	4.03	1.89	10.75	11.21	11.13	5.58	3.63	1.52	1.14	0.44	53.64
1944	1.28	0.27	6.19	2.62	5.25	9.92	9.83	8.43	4.32	9.54	0.37	0.00	58.02
1945	4.14	0.00	0.37	0.99	0.34	18.84	11.41	6.40	8.40	4.01	0.72	4.60	60.22
1946	1.69	3.33	1.80	0.34	7.75	10.97	9.78	7.31	3.95	2.11	0.97	1.89	51.89
1947	0.43	4.16	5.34	4.17	5.39	6.88	6.54	3.74	13.10	1.66	4.61	1.56	57.58
1948	7.79	0.96	4.51	3.51	1.29	0.97	12.82	10.40	11.92	2.08	0.44	2.80	59.49
1949	0.30	0.90	2.79	1.16	3.45	5.20	7.13	14.59E	6.33	1.77	1.48	3.94	49.04E
1950	0.00	0.19	2.88	2.88	1.12	5.95	7.64	6.34	12.62	5.34	0.17	4.48	49.61
1951	0.24	2.42E	2.07	8.66	1.26	7.39	9.61	9.53	8.94	1.78	5.40	2.68	59.98E
1952	0.80	5.15	7.05	1.03	2.30	8.57	6.31	10.25	7.64	9.11	1.31	1.40	60.92
1953	2.36	3.19	2.27	4.37	1.12	7.35	6.92	10.16	10.33	4.41	4.85	5.13	62.46
1954	0.79	0.98	1.21	3.35	2.75	7.35	7.33	2.88	6.34	1.58	2.58	1.13	38.27
1955	2.42	2.08	2.73	3.23	3.60	3.81	4.92	2.77	4.50	2.40	1.67	1.53	35.66
1956	1.64	0.83	0.45	1.87	7.69	3.45	6.02	8.19	4.57	9.11	0.52	0.06	44.40
1957	2.15	4.64	5.04	8.09	11.27	3.74	3.87	10.44	4.68	0.77	1.05	2.25	57.99
1958	4.89	1.32	4.54	3.03	4.51	4.43	5.45	5.95	5.15	3.17	1.44	3.37	47.25
1959	4.55E	3.39	10.76	5.52	5.71	11.29	11.34	5.11	7.59	8.29	0.98	2.04	76.57E
1960	1.23	5.56	9.89	3.22	1.26	5.92	12.68	6.20	19.44	1.92	0.00	1.06	69.18
1961	1.82	2.55	3.25	1.84E	4.89	4.42	3.83	7.77	0.89	1.97	0.53E	1.06	35.62E
1962	1.95	0.90	3.19	1.50	6.12	4.79E	0.45E	8.85	5.84	0.89	2.29E	0.23	37.00E
1963	1.92	6.72	3.42	0.13	6.72	9.27	7.30	4.03	4.31	1.36	6.82	2.37	54.37
1964	5.24	4.22	3.80	1.69	4.04	3.15	8.31	5.75	7.84	1.83	2.29E	1.78	49.94E

Table 2. (Continued)

RAINFALL IN INCHES - LAKE ALFRED EXP NOAA NUMBER: 4707

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YRLY
1965	1.40	4.31	3.12	1.46	0.07	10.86	7.94	6.39	6.16	2.23	0.57	3.45	47.96
1966	6.12	4.02	1.89	1.61	4.47	9.99	7.71	8.53	5.42	1.53	0.10	1.27	52.66
1967	0.97	5.13	0.89	0.00	2.46	15.70	11.87	14.30	4.05	0.91	0.09	2.34	58.71
1968	0.35	2.45	1.44	0.57	4.49	17.06	8.18	5.57	6.14	4.60	2.97	0.44	54.26
1969	3.01	1.47	6.99	1.21	1.98	5.70	6.48	9.40	6.29	6.73	2.34	4.19	55.79
1970	3.01	3.25	7.65	0.78	2.66	3.85	9.21	3.39	6.48	1.84	0.90	0.34	43.36
1971	1.03	4.41	2.58	0.60	3.20	4.85	5.30	7.43	3.80	6.93	2.76	0.79	43.68
1972	0.83	5.83	1.84	0.70	4.99	7.81	4.94	9.15	0.55	5.45	4.60	2.92	49.61
1973	4.70	2.47	3.26	1.87	3.40	3.07	11.28	6.10	9.11	1.16	1.62	1.99	50.03
1974	0.25	1.32	2.28	1.23	5.27	11.43	5.67	4.82	8.95	0.32	0.20	1.59	43.33
1975	1.64	1.81	2.43	0.60	6.50	4.18	3.54	10.31	6.77	4.29	1.05	0.77	43.89
1976	0.31	0.54	1.98	3.17	10.34	6.29	4.57	8.39	7.91	1.64	1.75	1.64	48.53
1977	2.33	2.66	1.29	0.29	4.19	5.62	6.73	8.13	8.87	1.52	1.93	3.43E	46.99E
1978	2.87	4.63	2.34	0.53E	5.73E	10.56E	9.74E	3.87E	2.04	1.30	0.34	3.82	47.77E
1979	6.32	1.30	3.54	1.39	13.93	1.84	6.28	11.04	13.58	0.53	2.90	1.75	64.40
1980	3.16	2.03	2.33	2.58	7.30	8.04	5.60	2.82	2.78	0.91	4.44	0.76	42.75
1981	0.47	3.15	0.87	0.01	1.50	6.85	5.03	13.16	5.88	1.31	0.85	2.12	41.20
1982	1.72	2.99	4.78	3.25	6.87	10.12	8.49	5.83	6.02	4.94	0.60	0.68	56.29
1983	1.60	8.63	7.67	2.76	2.45	10.64	3.17	10.72	5.85	4.05	2.28	5.32	65.14
1984	1.45	4.15	1.67	2.68	3.59	3.31	9.54	4.17	7.14	0.44	1.49	0.27	39.90
MEAN	2.19	2.84	3.44	2.57	4.53	7.72	7.27	7.33	6.75	2.76	1.67	2.04	51.11
NORM	2.34	3.05	3.52	2.20	4.81	7.06	6.98	7.25	6.57	3.02	2.01	1.97	50.78

E - ESTIMATED VALUE

TABLE 3

NORMALS FOR RAINFALL, EVAPORATION AND TEMPERATURE AT LAKE ALFRED

MONTH	RAINFALL (INCHES	CLASS A PAN EVAPORATION* (INCHES)	TEMP (^{O} F)
JANUARY	2.34	3.41	59.6
FEBRUARY	3.05	4.13	60.8
MARCH	3.52	6.17	66.3
APRIL	2.20	7.49	71.3
MAY	4.81	8.38	76.4
JUNE	7.06	7.50	80.3
JULY	6.98	7.62	81.7
AUGUST	7.25	7.21	81.9
SEPTEMBER	6.57	6.34	80.2
OCTOBER	3.02	5.31	73.8
NOVEMBER	2.01	3.98	66.4
DECEMBER	1.97	3.17	60.8
ANNUAL	50.78	70.71	71.6

* Mean for period of Record (1965-1984)

in Figure 6. Figure 7 contains graphs of the five and ten year moving averages. For 60 years of rainfall record, the maximum 10 year moving average annual rainfall was 59.7 inches and the minimum was 47.2 inches.

A cumulative frequency histogram is shown in Figure 8. This frequency curve provides information concerning the occurrence of annual rainfall quantities during the 60 year period of record. For example, the annual rainfall exceeded 35.00 inches 100 percent of the time, while an annual rainfall totaling 55.00 inches was equaled or exceeded only 35 percent of the time.

Estimates of single-event rainfall amounts, given various duration storms, are shown in Figure 8. Rainfall depths are presented for return period variates of 10 and 100 years, and duration variates of 6 hours up to 10 days.

EVAPORATION

Estimations of lake evaporation was made using pan evaporation records from the Lake Alfred station during the period 1966 to 1984. The ratio of annual lake-to-pan evaporation (pan coefficient) was assumed to be 0.80. The maximum and minimum annual evaporation values were 69.0 and 53.4 inches, respectively. Monthly normals are presented in Table 3.

LAKE STAGES

Stage records are available for Lake Lowery from September 1960 to present. Records are also available for Lake Juliana from December 1961 to present. A plot of monthly averages for each lake is shown in Figure 10. A frequency analysis of flood stages at Lake Lowery using a Log Pearson III distribution

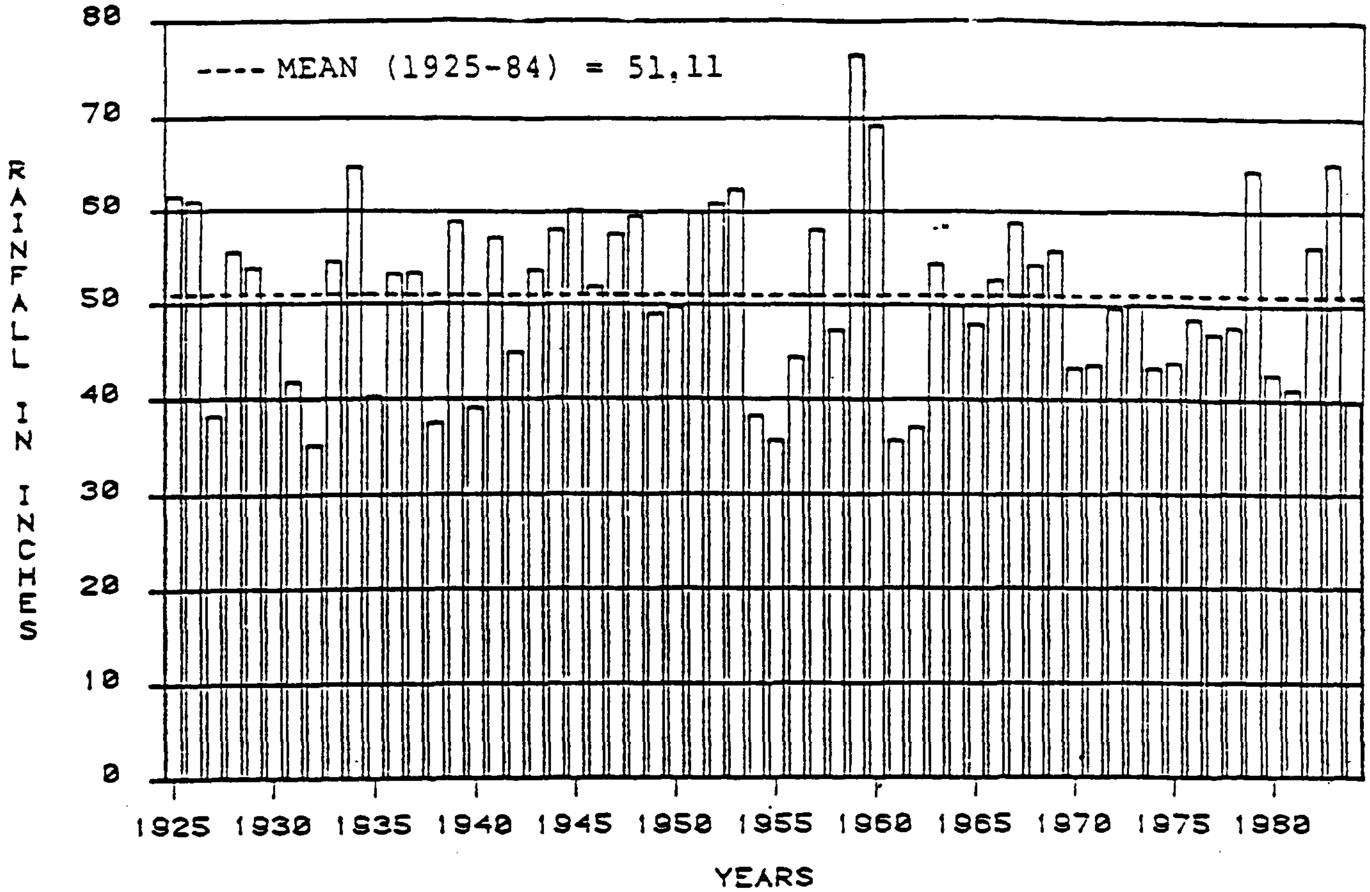


Figure 6. Annual Rainfall for Lake Alfred (1925-1984)

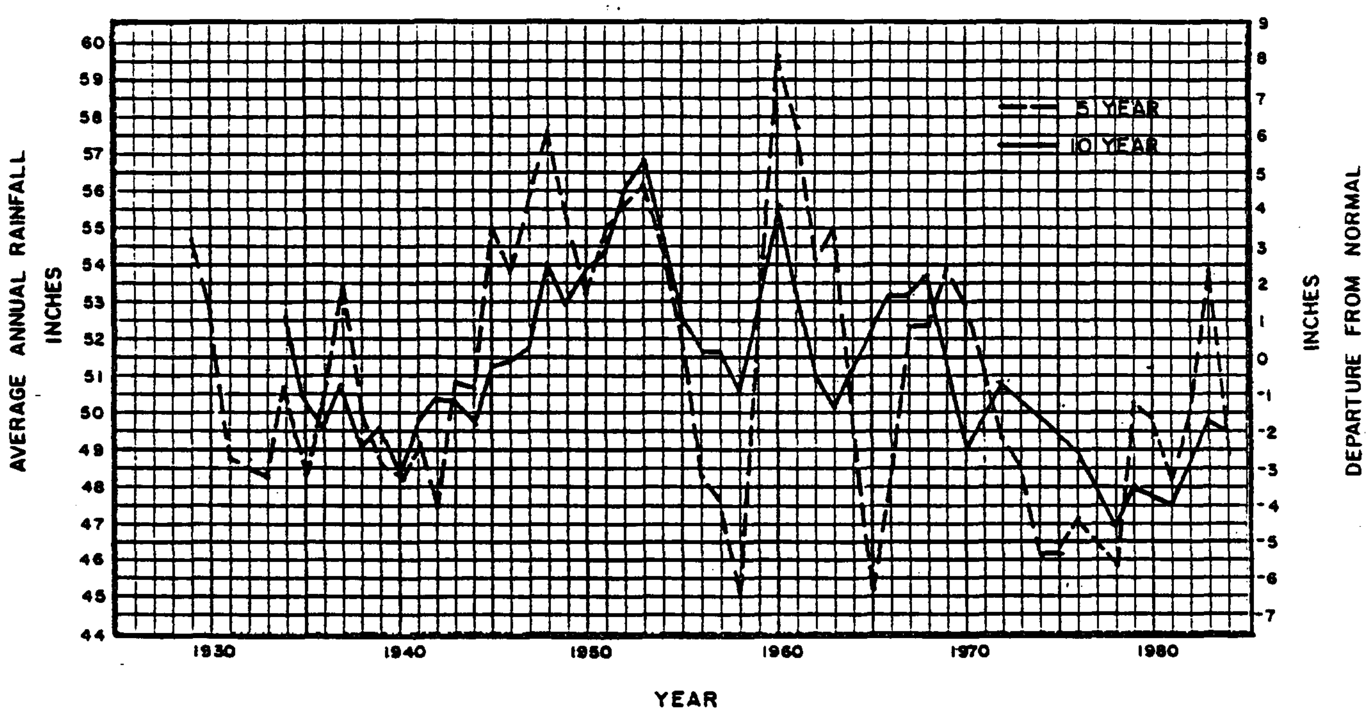


Figure 7. Five and Ten Year Moving Average Annual Rainfall.

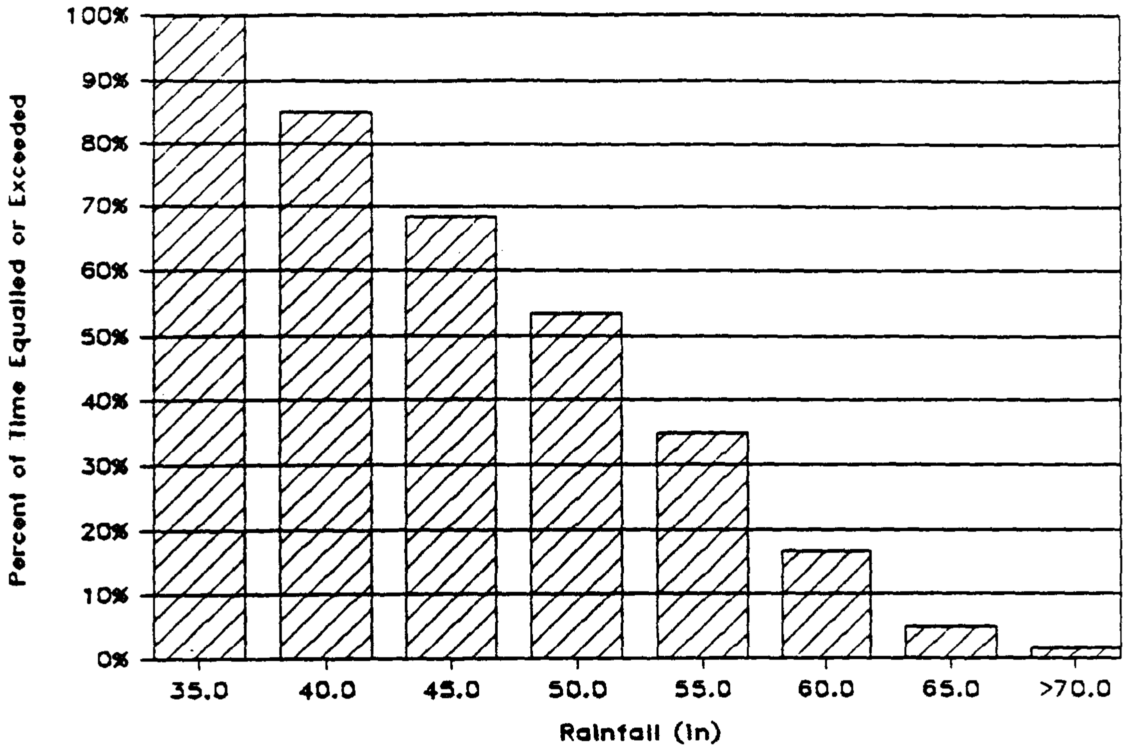


Figure 8. Cumulative Rainfall Frequency Histogram.

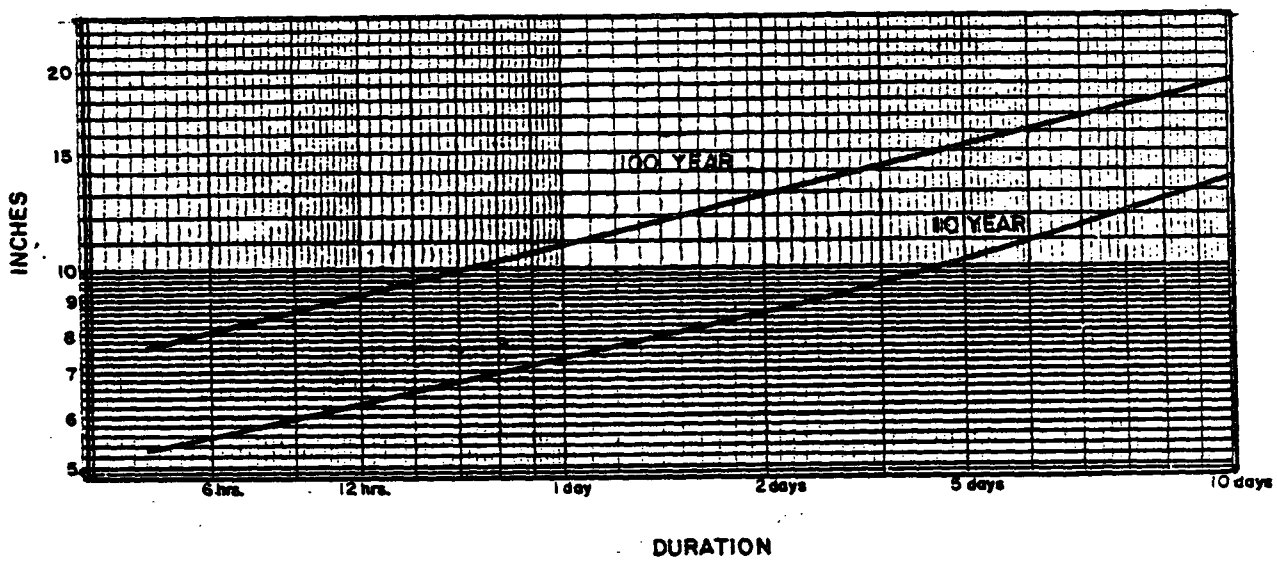


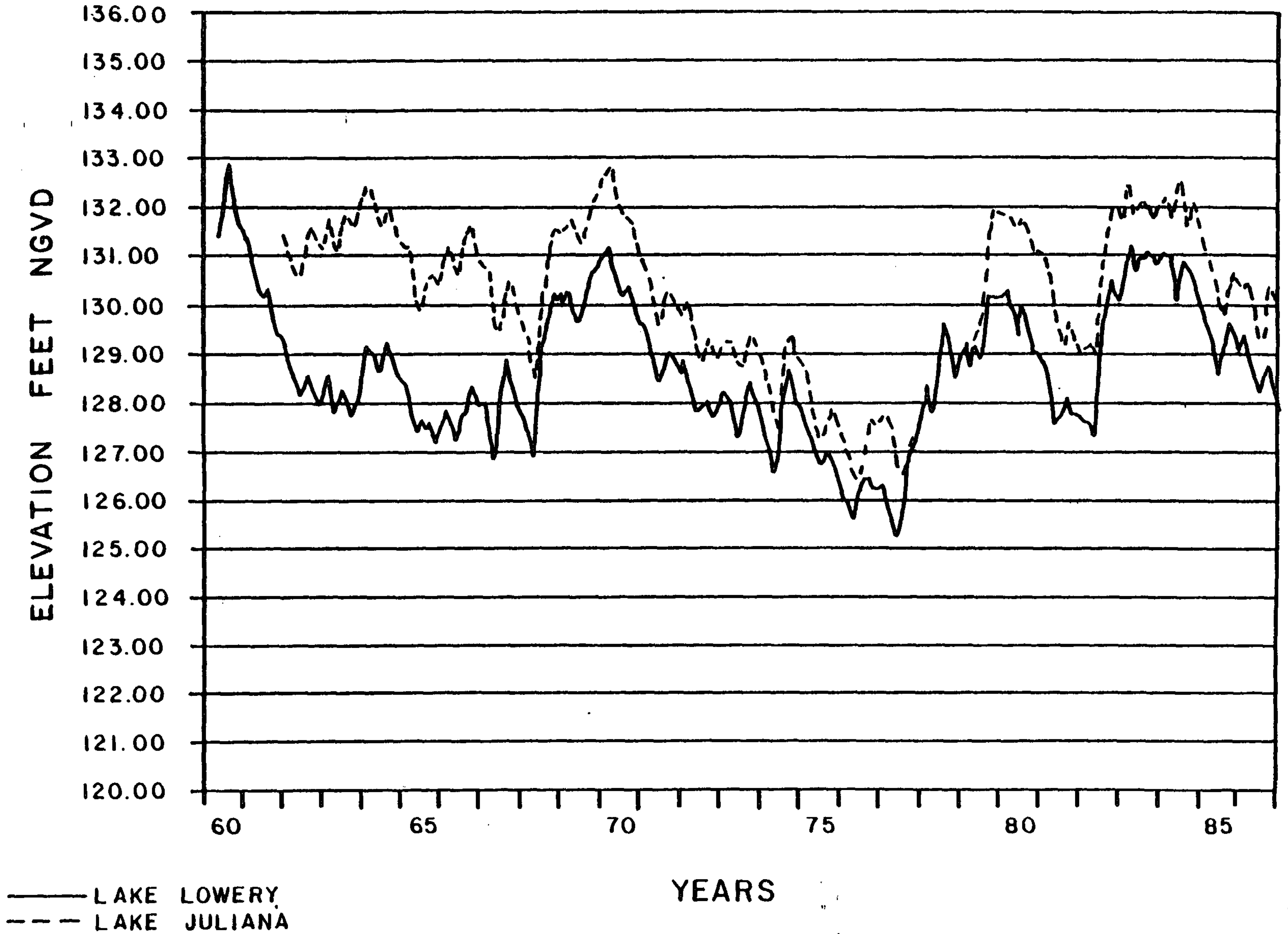
Figure 9. Rainfall Depth Verses Duration Graph for Lake Alfred.

resulted in the flood stage estimates listed in Table 4. The data is represented as exceedence stages -- stages which are equalled or exceeded continuously for the given period of time. Results of a similar analysis for low lake stages are given in Table 5.

The use of historical records to estimate the frequency of future flood stages is subject to the limitation that the recorded hydrologic conditions are representative of future conditions, including rainfall and drainage. It is believed that the drainage has not been significantly altered since 1960. However, as discussed earlier, the precipitation during the period from 1960 to 1984 has been abnormally low when compared to the long-term average.

SURFACE WATER SYSTEM

The surface water in the Greater Lake Lowery Basin (Lake Lowery, Grassy Lake, Lake Mattie, Lake Juliana and Lake Van) is contained in the lakes and surrounding wetlands. The flat topography in the area causes any increase in the height of the surface water to impact a large surface area. Outflow from the low-lying marshes occurs only after the water level in the marshes rises above the sand ridges scattered throughout the area. The direction of such flow depends on the intensity, duration, and areal distribution of the storm rainfall which produces the rise in water levels. However, the general movement of surface water is from west to east, i.e., from Lake Mattie toward Lake Lowery, then north into the Palatlakaha or Withlacoochee River basins.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YEARS
LAKE LOWERY
LAKE JULIANA

TABLE 4

LAKE LOWERY FLOOD STAGES

BY LOG PEARSON III ANALYSIS OF STAGE RECORDS

1960-1984

Frequency Years	Elevation Feet (NGVD) 1 Day	30 Day	60 Day	183 Day
2	129.38	129.19	129.03	128.60
5	130.71	130.49	130.31	129.75
10	131.51	131.26	131.07	130.53
25	132.47	132.18	131.96	131.46
50	133.16	132.82	132.58	132.13
100	133.81	133.43	133.17	132.79
200	134.45	134.03	133.75	133.43

TABLE 5

LAKE LOWERY LOW STAGES

BY LOG PEARSON III ANALYSIS OF STAGE RECORDS

1960-1984

Frequency Years	1 Day	30 Day	60 Day	183 Day
2	127.56	127.95	128.15	128.81
5	126.47	126.76	126.96	127.64
10	125.95	126.18	126.36	127.07
25	125.43	125.60	125.76	126.50
50	125.11	125.24	125.39	126.15
100	124.84	124.93	125.07	125.85
200	124.60	124.65	124.78	125.57

The watershed of the Greater Lake Lowery Basin and the adjoining areas has been divided into several sub-basins for surface water analysis (Figure 11). The boundaries were determined by topography, location of water control structures, and roadways. The area of each sub-basin is listed in Table 6.

GROUNDWATER SYSTEM

The groundwater system in the area consists of the Floridan aquifer and a surficial, water table, sand aquifer. The Floridan aquifer is an artesian aquifer replenished by rainfall in the aquifer's recharge areas. The area around Lake Lowery is one such recharge area. The general movement of water into the Floridan aquifer is from land surface downward into the surficial aquifer and finally into the Floridan aquifer.

The rate of recharge from the surficial aquifer to the Floridan aquifer is directly related to the difference in the elevation of the water table of the surficial aquifer and the potentiometric surface of the Floridan aquifer. The relationship between the potentiometric surface of the Floridan aquifer and the water table of the surficial aquifer can be observed by comparing the maps in Figures 12 and 13.

Increased diversion of flood waters away from this area could influence the rate of recharge. However, a significant change in this water table/potentiometric surface relationship would be required to create a significant change in the rate of recharge.

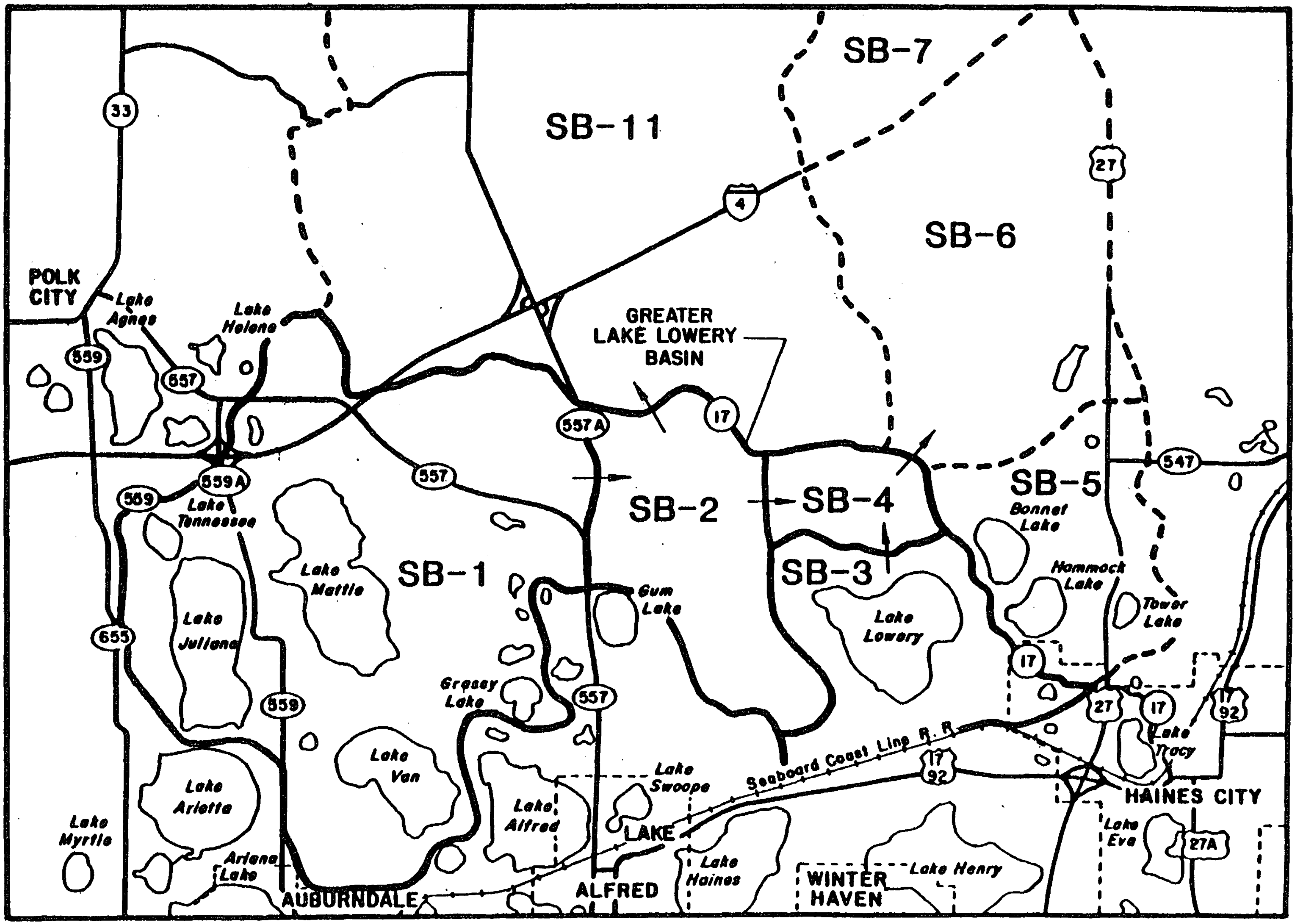
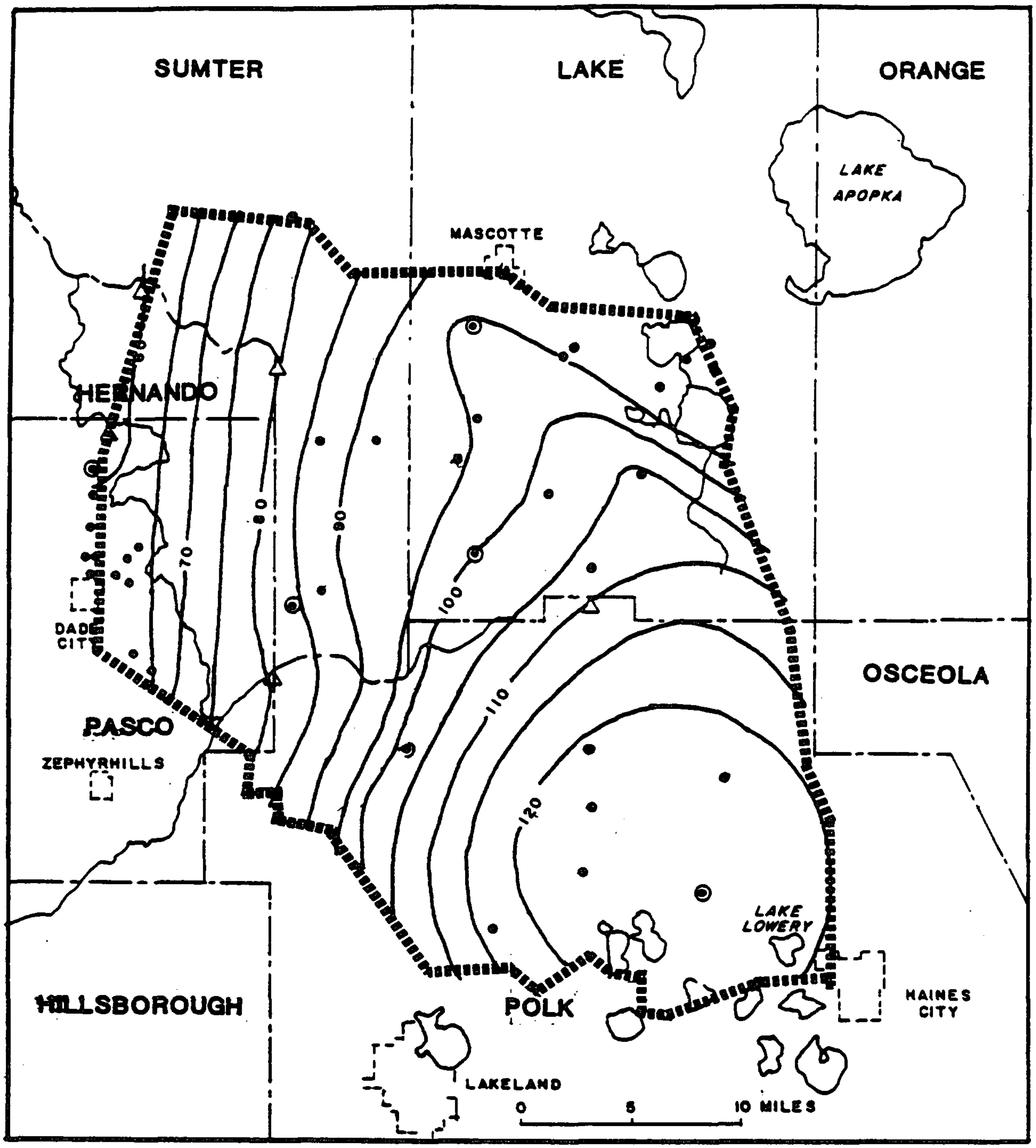


Figure 11. Watershed Sub-Basin for Greater Lake Lowery and Adjoining Basins.

TABLE 6

SUB-BASIN DRAINAGE AREAS

SB No.	D.A. (acres)	
1	14,000	Lake Mattie
2	3,560	Mattie-Lowery Marsh
3	3,230	Lake Lowery
4	1,355	Lowery Marsh
5	2,435	Bonnett Lake
6	6,120	Kuder Road
TOTAL	30,700	



EXPLANATION

BOUNDARY OF GREEN SWAMP AREA

—90— POTENTIOMETRIC CONTOUR—SHOWS ALTITUDE AT WHICH WATER LEVEL WOULD HAVE STOOD IN TIGHTLY CASED WELLS. CONTOUR INTERVAL 5 FEET. DATUM IS MEAN SEA LEVEL.

△ STREAM GAGING STATION

• OBSERVATION WELL

○ OBSERVATION WELL WITH RECORDER

Figure 12. Potentiometric Surface of the Floridan Aquifer (May 1977), (Grubb, Hayes; U.S.G.S. 1979).

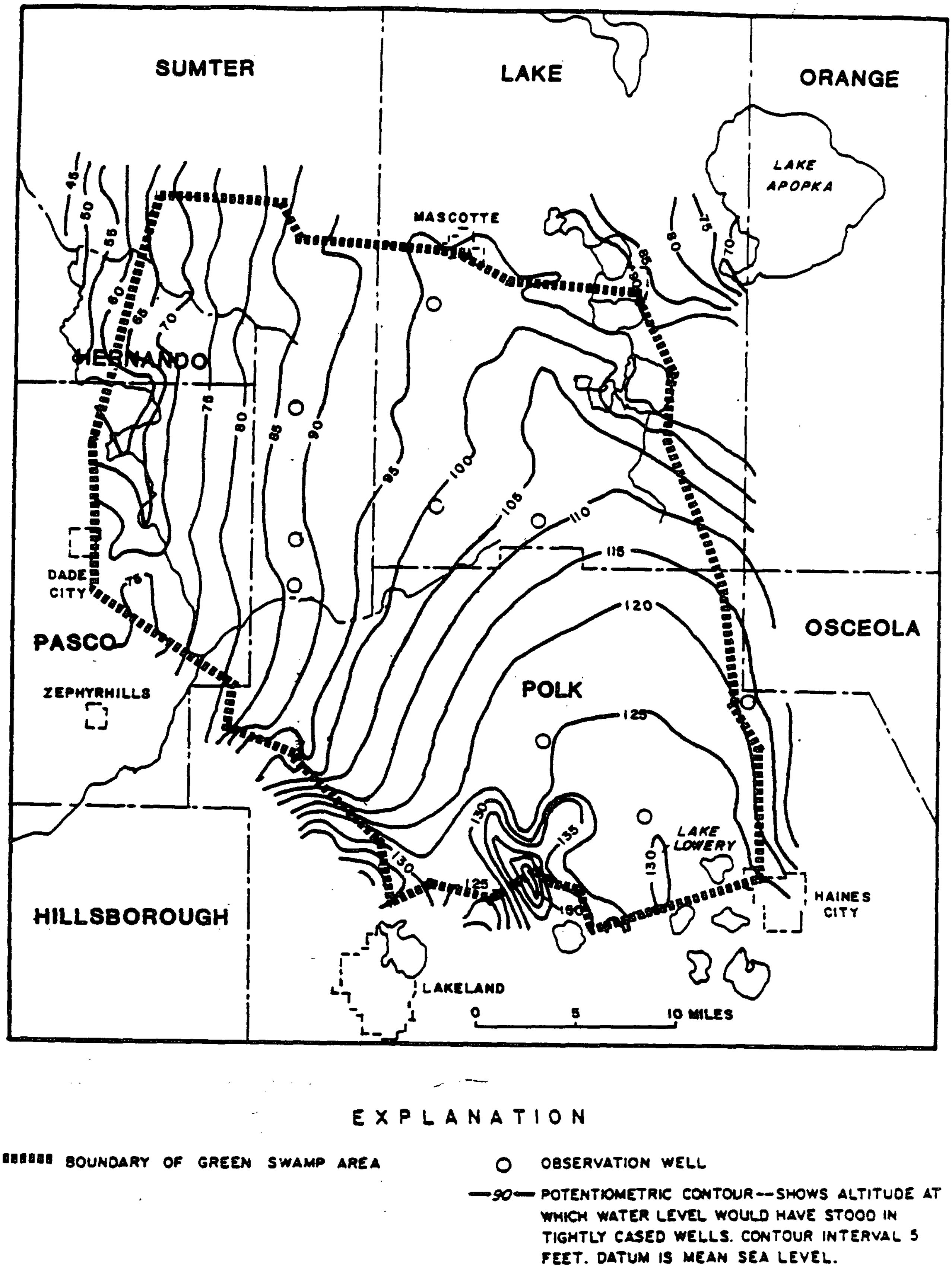


Figure 13. Water Table of the Surficial Aquifer (May 1977), (Grubb, Hayes; U.S.G.S. 1979).

HYDROLOGIC CONCERNS

Analysis of long-term rainfall records indicate the period preceding the flood occurrence of 1982-1984 was an atypical period of low annual precipitation (Figure 14). A statistical analysis of the 12 years preceding 1982 confirmed the abnormality of the period. The probability of having 11 out of 12 years of annual rainfall below the 60 year mean is 0.32 percent. This period of low rainfall may have given a false sense of security to those residents who considered building in low-lying areas. Following the below normal trend was a high water period during late 1982, 1983, and 1984 when the lake rose to flood stages due to unusually high rainfall.

The hydrologic analysis of the study area was complicated by the dampened and lengthened response time of the surface water drainage systems due to the large moisture holding capacity of the surrounding wetlands; the flat topography of the area which causes large surface areas to be impacted by small rises in the water level; and variable intensities, durations, and areal distributions of storm rainfall. All of these factors combine to produce complex, multivariate relationships which will require sophisticated mathematical techniques in order to describe the actual physical processes of the hydrologic cycle.

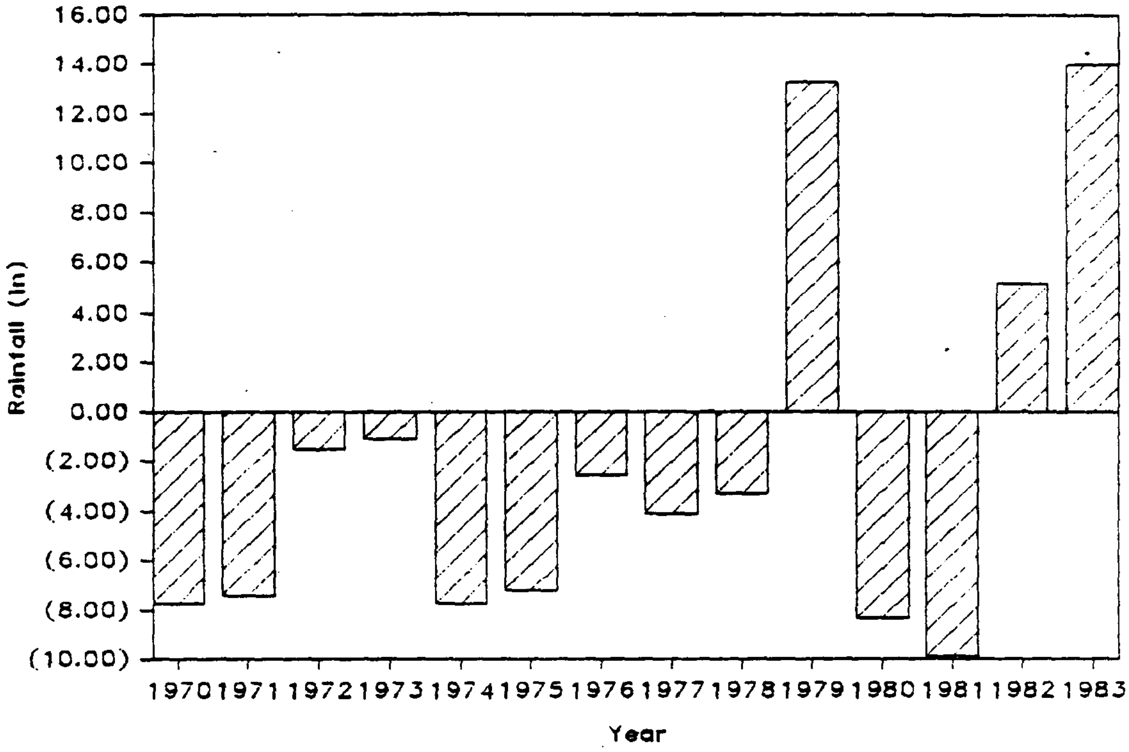


Figure 14. Annual Rainfall Departures from Normal for Lake Alfred (1970-1

STUDY AREA DESCRIPTION

TOPOGRAPHY AND SOIL DESCRIPTION

The Greater Lake Lowery Basin can be divided into two topographically unique sections. They are the flat marshes and wet prairies, and the upland sand hills. Each of these sections have widely differing soil characteristics. The two primary groups of soils which are present in each section are the Astatula-Tavares-Basinger group, and the Fresh Water Swamp group (Florida Department of Administration, 1975). The Astatula-Tavares-Basinger group is generally located in areas around the east, south, and west sides of the lake. The Fresh Water Swamp group is the predominate soil in the wetlands.

Runoff Characteristics

Soil and topographical features, which determine the surface runoff characteristics of the various sub-basins, differ considerably between the flat marshes and wet prairies, and the upland sand hills. However, since the wetlands comprise the most extensive surface area and are the watercourses for drainage leaving the Lake Lowery area, their runoff characteristics are of critical concern. Within the Fresh Water Swamp group the soil and topographical features are very uniform, i.e., the topography is generally flat; the surface culture has a large precipitation interception, detention, and storage potential due to the dense vegetal cover; the soil group has very low infiltration rates; and the watercourses present tortuous drainage patterns.

SURFACE WATER OUTLETS

Lake Lowery is situated at the headwaters of the Palatlakaha, Withlachoochee, and Peace rivers; therefore, surface water which drains from the Greater Lake Lowery Basin discharges into one or more of these river basins. Three major outlets -- Big Creek in the Palatlakaha River Basin, the Withlachoochee River Basin, and the Peace River Basin are indicated in Figure 15 along with the direction of surface water flow during periods of high water levels. A description of the size, construction material, and invert elevations of the water control structures situated within each basin are listed in Table 7. The locations of these structures are also shown in Figure 15.

Big Creek Outlet

Surface water exits the Lake Lowery area (Sub-Basin 3) via a culvert structure under Lake Lowery Road into Sub-Basin 4, then through another culvert structure under County Road 17 into Sub-Basin 6. Flow then continues through a natural constriction, locally referred to as Black Ford, and then into the Big Creek Basin. This waterway is heavily vegetated and averages 100 to 200 feet wide. A section of this waterway was surveyed in 1982 and the records indicate that the lowest point was 128.8 feet (NGVD). When the water level in Lake Lowery was 130.5 feet in 1984, the velocity of the surface water flowing through this area was scarcely detectable due to flow-resistance caused by heavy vegetation.

The flat terrain of the watershed enables relatively small

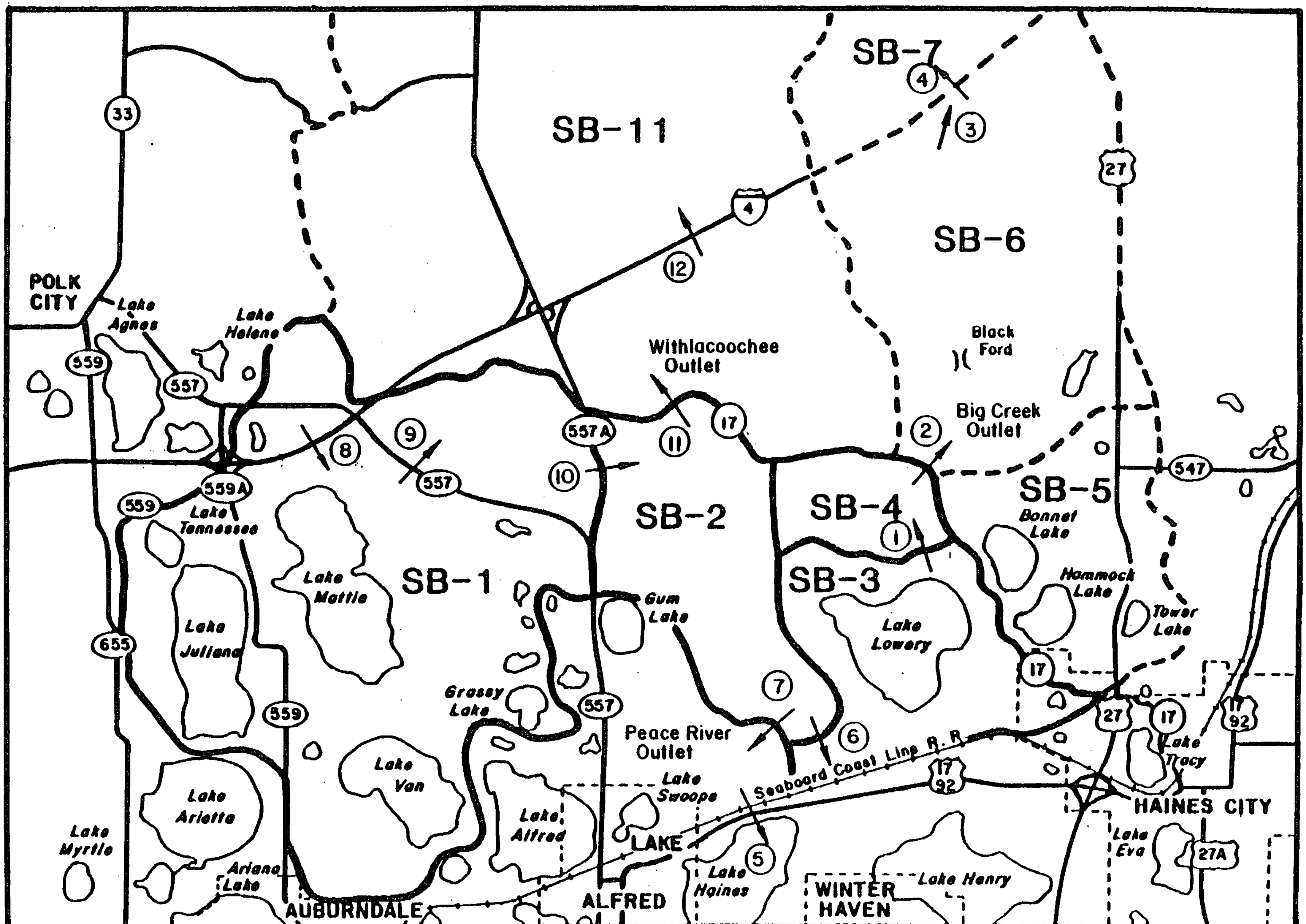


Figure 15. Sub-Basins, Surface-Water Outlets, and Location of Structure for Greater Lake Lowery Basin.

TABLE 7

DESCRIPTION OF STRUCTURES

STRUCTURE		DESCRIPTION	INVERT ELEVATION
1	Lake Lowery Rd.	48" CMP In 1984 replaced by 2-48" CMP with 48" flashboard riser	127.3±
2	County Road 17 (Sub Basin 4)	2-48" CP	127.5
3	Private Rd. South of I-4 (Sub Basin 6)	2-58" x 36" CMPA	126.6
4	I-4 (Sub Basin 6)	3-30" 4-24" 1-36" 3-48"	
5	Seaboard Coast Line RR	1-8' x 3' CBC	
6	Lake Lowery Rd. (South)	24" CMP	130.2±
7	Farm Rd (Sub Basin 2)	24" CP 2-18" plastic pipe added in 1984	128.3 128.4
8	I-4 (Sub Basin 1)	6' x 4' CBC	130.1
9	County Road 557	3-10' x 4' CBC	
10	County Road 557A	5-24" CP 5-30" CP	129.5±
11	County Road 17 (Sub Basin 2)	120- wide bridge 2-36" CP	130.2
12	I-4 (Sub Basin 11)	3-24" CP 2-30" CP 3-36" CP 1-42" CP 2-8' X 4' CBC	 128.5

water control structures to impact the hydrology of a large area; therefore, accurate hydraulic analysis requires very detailed topographic data and flow and water level measurements. Flow measurements made over a four month period (from January through April, 1984) of high water levels allowed reliable inferences to be made with respect to the hydraulic characteristics of the watershed.

In 1983, after high water levels were experienced, there was concern that the roads which traverse the channels and marshes in the Lake Lowery area were impeding flow. The District issued a consent order to the County authorizing the addition or replacement of culverts under these roads in the Big Creek Basin. However, upon completion of the work, no benefits were observed. No culverts were found at which there was a significant difference in the water level elevations across the structures.

Another impedence to flow was thought to have been caused by the construction of Interstate 4. The interstate highway was still under construction when severe flood conditions occurred during 1960. High water elevations were documented and some design modifications were made. Observations during the high water period of 1983-84 did not support this claim.

During high water periods, a portion of the surface and subsurface water flows from the Lake Mattie area (Sub-Basin 1), into Sub-Basin 2, and then may move eastward through a low point in the drainage basin divide between Sub-Basins 2 and 4. Prior to the extension of Lake Lowery Road, when the water level in the lake was lower than the water level in the marsh, water was free to spread out and move at an extremely slow velocity from the north marsh (Sub-Basin 4) into the area south of the current road embankment (Sub-Basin 3). This flow regime allowed water levels in the lake and marsh to quickly equilibrate. However, when the extension to Lake Lowery Road was constructed, all of the flow had to pass through structure 1. The culvert reduced the conveyance of water so that the marsh and the lake now take much longer to equilibrate. Normally, some flow travels from the marsh (Sub-Basin 4) through structure 1, into the lake. However, a storm's precipitation rate, rainfall intensity pattern, and areal distribution can change the direction of flow. Under unusual conditions the water level in the lake can rise to a higher level than the water level in the marsh, resulting in flow from the lake into the marsh. If a sufficient hydraulic gradient is developed, the water will then continue north through structure 2, into Sub-Basin 6. It is also possible, but highly unlikely, that flow from Lake Lowery (Sub-Basin 3) could move westward through structure 6 into Sub-Basin 2, and then through structures 7 and 5, and into Lake Haines.

During a high water level period (April 5, 1984) an estimate was made of the water surface profile. Figures 16 and 17 show the profile of a meandering watercourse discharging into Big Creek. The watercourse threads from Lake Lowery, northward toward Sand Mine Road near the Polk-Lake County line. Figure 18 shows the channel invert and the estimated water surface profile along a pathway from Lake Mattie eastward into the marsh north of Lake Lowery (Sub-Basin 4). The pathways of the two watercourses are shown on Figure 19.

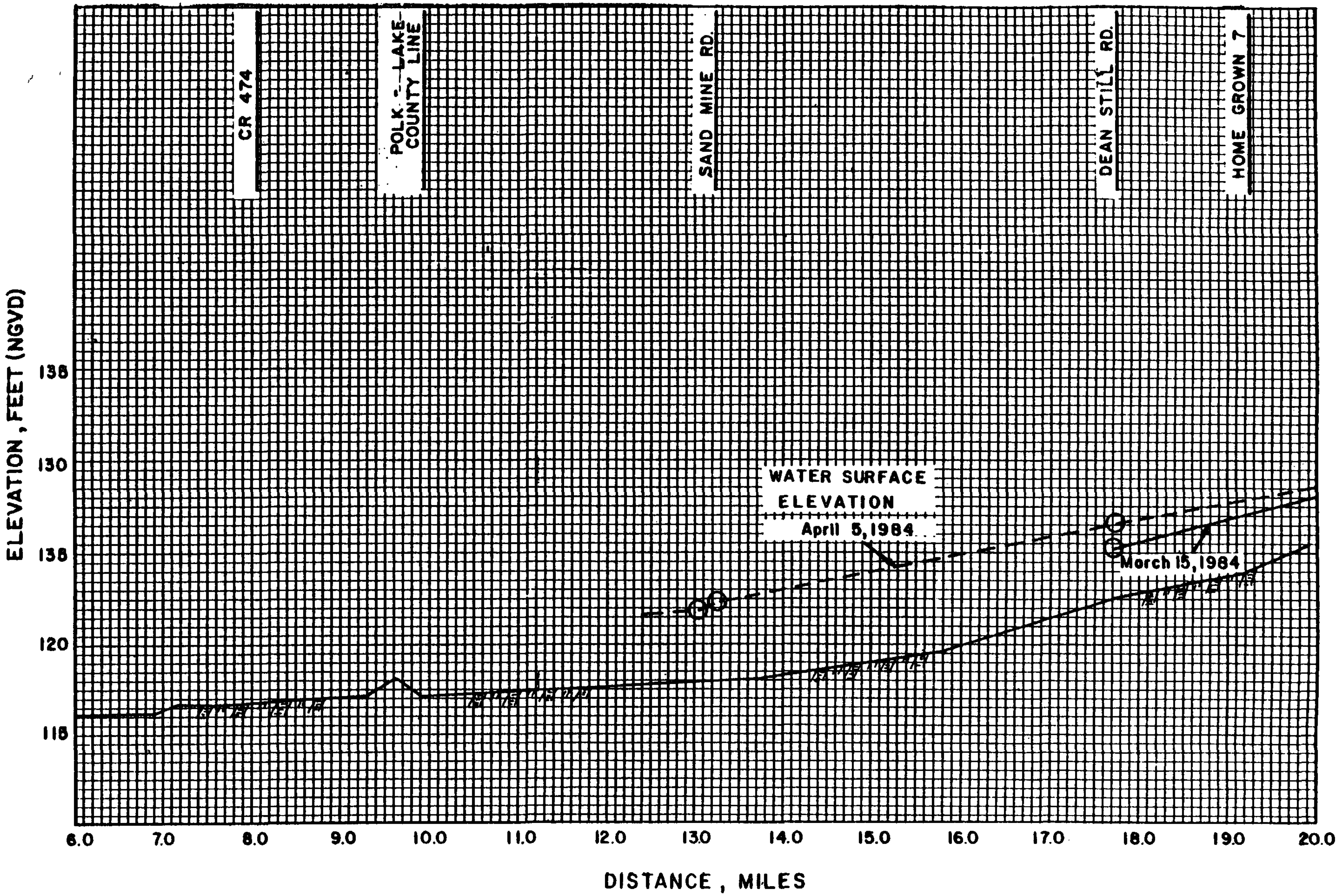


Figure 16. Watercourse Profile I from CR474 to Lake Lowery. (Part I)

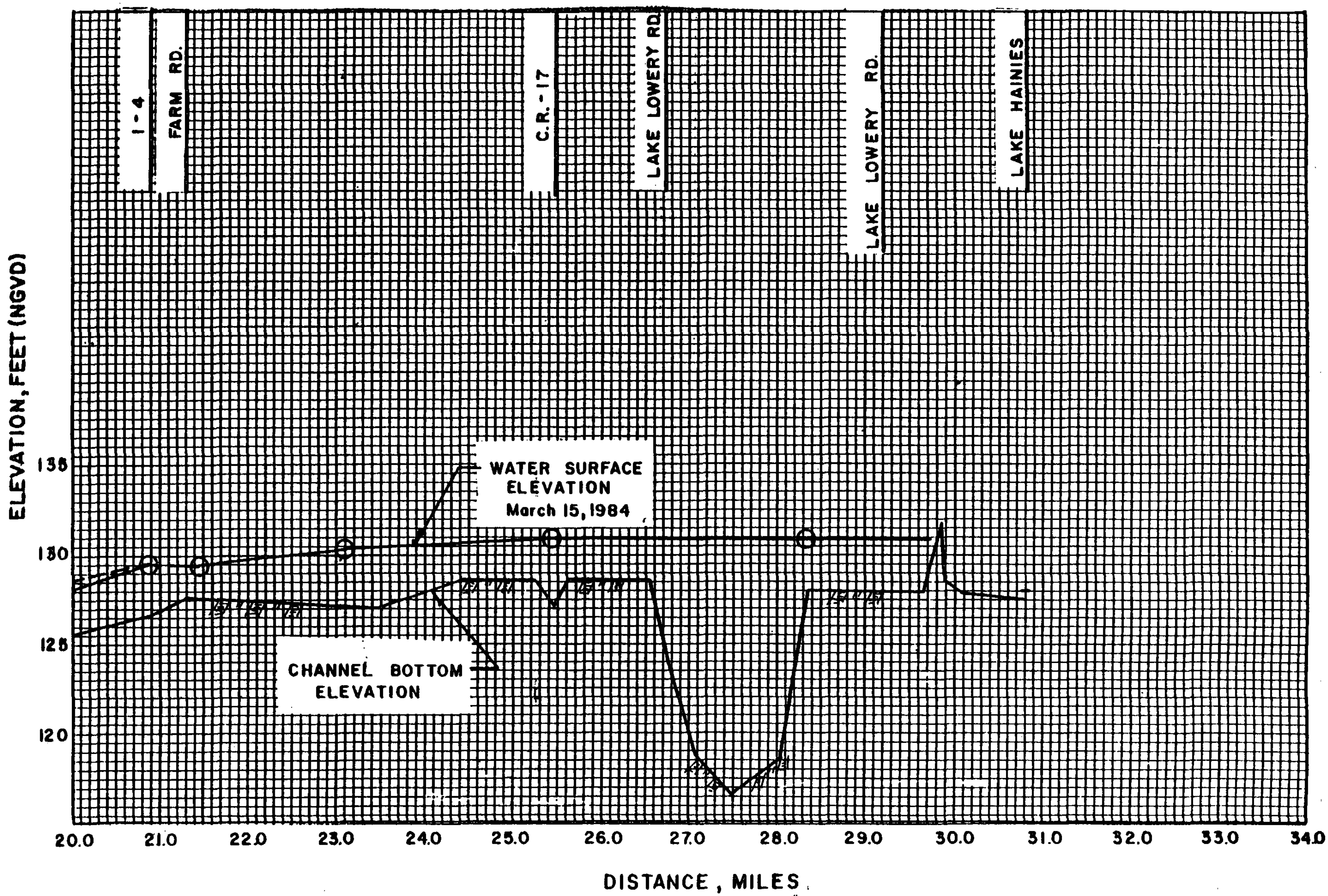


Figure 17. Watercourse Profile I from CR474 to Lake Lowery (Part 2)

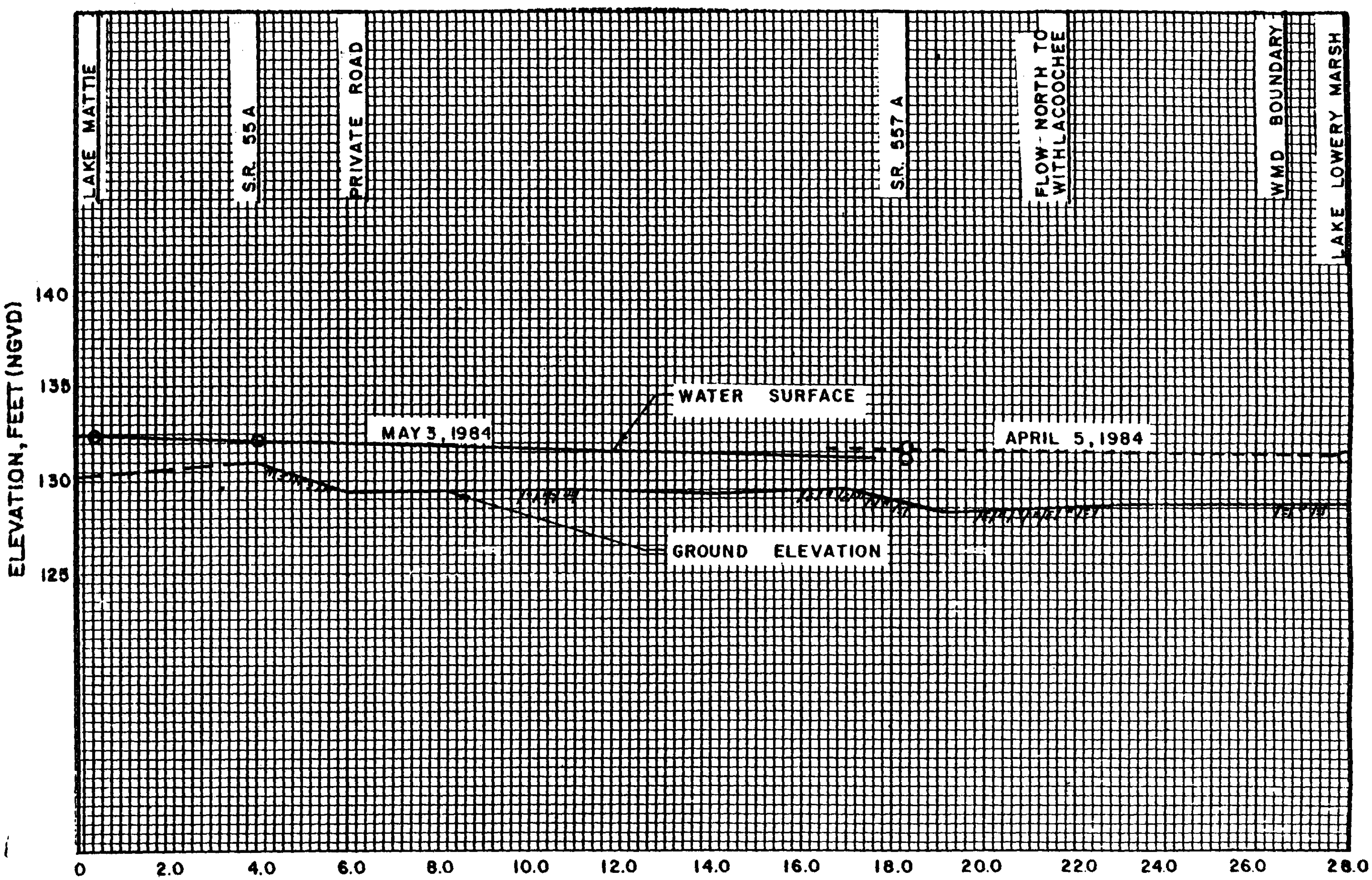


Figure 18. Watercourse Profile II from Lake Mattie to Lake Lowery Marsh.

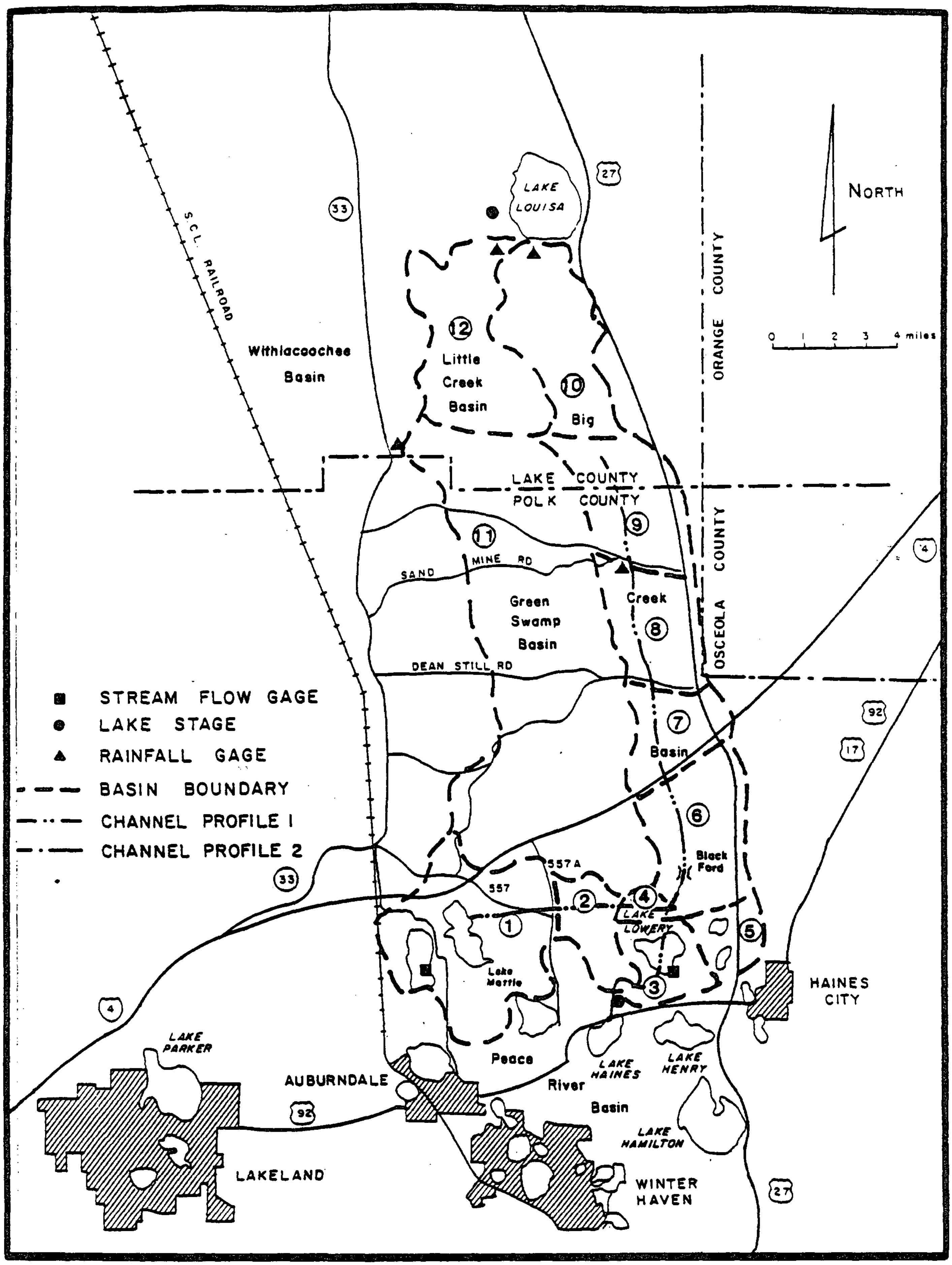


Figure 19. Modeled Sub-Basins and Watercourses Tracks.

Withlacoochee River Outlet

Surface water exiting the Lake Lowery area (Sub-Basin 3) into Sub-Basin 4, as discussed in the previous sections, can flow into Sub-Basin 2 and then through structure 11 under Polk County Road 17, into Sub-Basin 11. The majority of the flow at this outlet passes under a bridge while a small amount flows through two culverts located just west of the bridge. The bridge was a design component of the U.S. Army Corps of Engineers "Four River Basins Florida Project." Design plans for the bridge indicate a natural ground elevation of approximately 130 feet below the bridge deck (U.S. Army Corps of Engineers, Jacksonville District, 1960).

Peace River Outlet

Surface water exiting the Lake Lowery area (Sub-Basin 3) could flow west through structure 6 into Sub-Basin 2, then through a natural depression between two sand ridges (Figure 20). The controlling elevation for the waterway is 131.6 feet (NGVD). A private farm road has been constructed across a narrow point in the flow-path (Figure 21). The road was originally constructed with one 24 inch conduit to allow flow to pass under the roadway. In late 1984, two 18 inch diameter corrugated plastic pipes were placed parallel to the existing conduit in order to increase the discharge capacity. However, the ground elevation north of the road still controls water through this structure. A description of the failure of the farm road in 1960, (Pride, et al., 1966) when the water level in Lake Lowery reached 133.3 feet, indicates

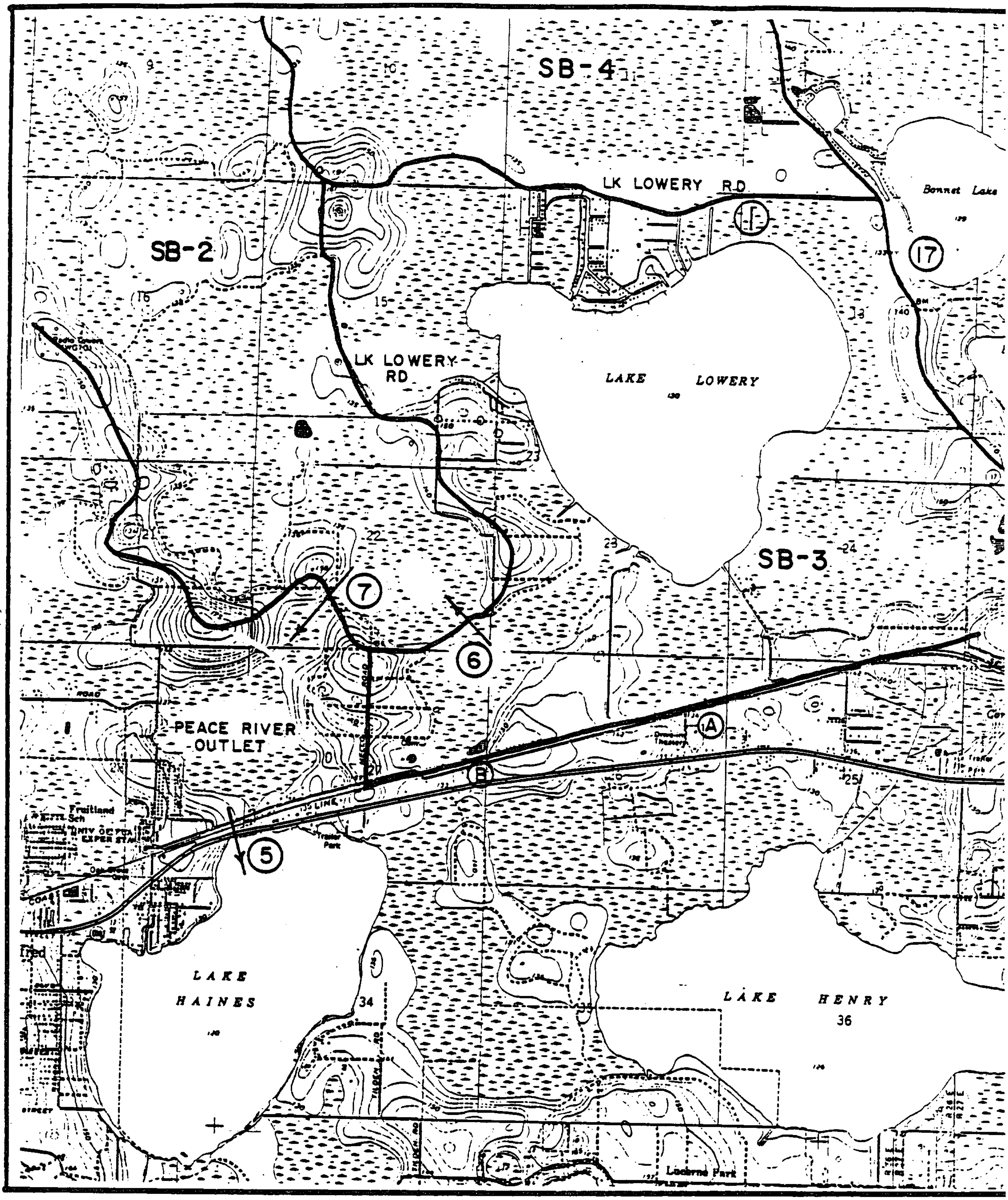


Figure 20. Peace River Outlet.

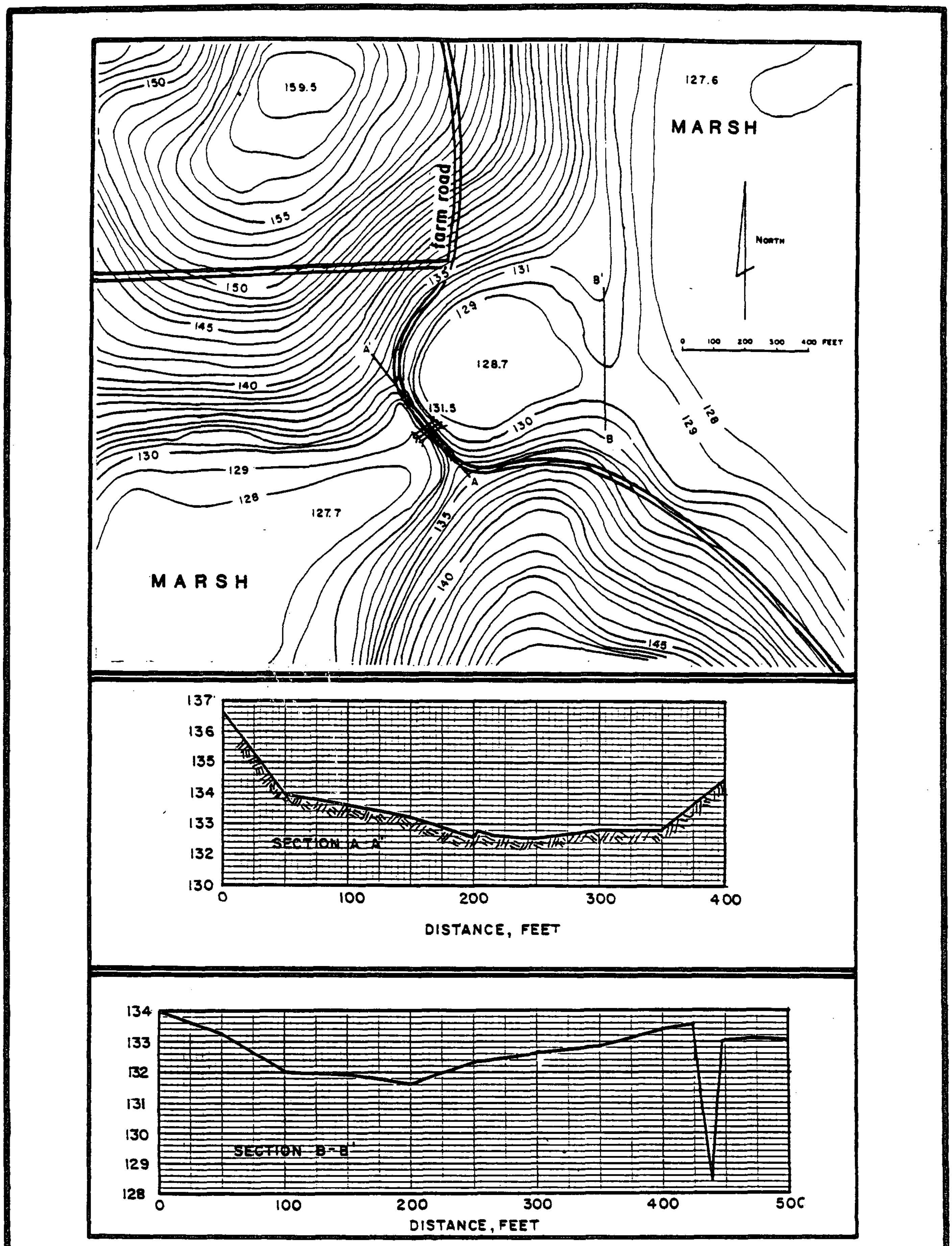


Figure 21. Cross Section View of Peace River Outlet at Structure 7.

that the road does indeed reduce the drainage capacity of this outlet.

The ground elevations in the surrounding marsh range from approximately 128 to 129 feet (NGVD). Therefore, a water depth of three to four feet in the marsh is required to initiate flow through this outlet. During the past 25 years the lake stage has exceeded 131.6 feet only about two percent of the time. Any water flowing through this outlet would ultimately enter a three foot by five foot concrete box culvert located under the railroad tracks, and finally into Lake Haines. The flow from the Gum Lake area also passes through this box culvert.

The possible blockage of natural outlets from Lake Lowery into the Peace River Basin has been a subject of concern. Two locations where fill-dirt was reportedly deposited across the low areas separating Lake Lowery and lakes Henry and Haines are indicated in Figure 20 as "A" and "B". These reports were investigated through the evaluation of historical aerial photographs and evaluation of soil borings by the U.S. Soil Conservation Service. These investigations yielded no evidence to indicate that fill material had ever been placed in these areas. The present topography at location A indicates the controlling elevation for overflow exiting Lake Lowery to be 132.9 feet (NGVD), provided there was no blockage at the Seaboard Coast Line Railroad track and the adjoining roadway locally known as the "Old Dixie Highway" which travel east/west along the south side of the lake (Figure 22).

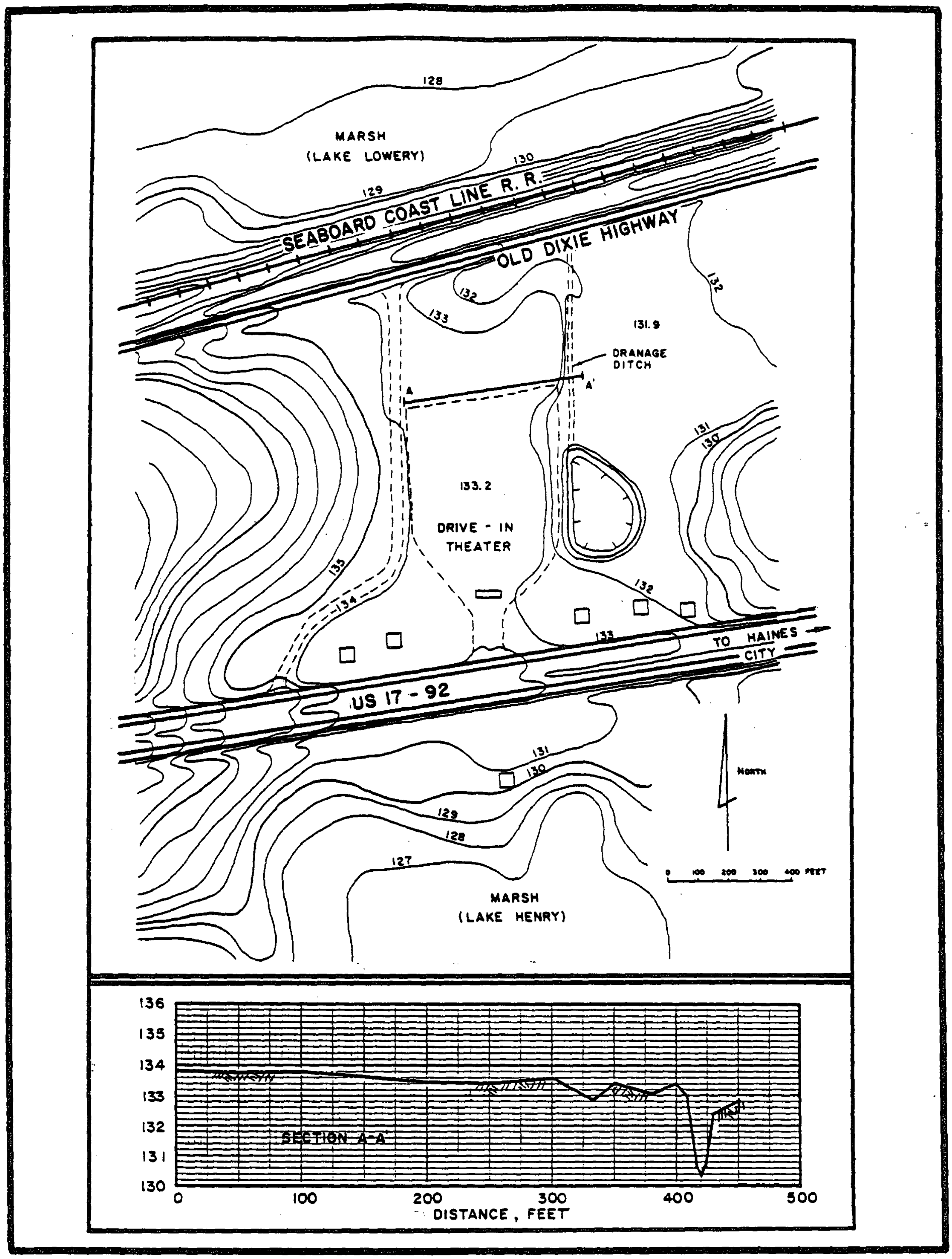


Figure 22. Cross-Section View of Blocked Location "A ".

Efforts were made to determine the drainage impacts of the Seaboard Coast Line Railroad track and the "Old Dixie Highway." Background information has been limited to a series of aerial photographs dating from 1941. Photographs taken around 1950 show the existence of a railroad trestle at location "A" (Figure 22) which could have allowed flow from the lake. However, shortly thereafter, the trestle was removed and the area was back-filled blocking the previous flow pattern. Presently, the water must overtop the railroad tracks at an elevation of 136.0 feet (NGVD) before southward flow would be possible.

During the course of this study the District received reports that a channel had existed between the "Old Dixie Highway" and U.S. Highway 17/92 connecting Lake Lowery to Lake Henry, and that the channel was filled in during the construction of a drive-in theater situated along U.S. Highway 17/92 (Figure 22). Aerial photography indicates a small sink-hole was located near the theater's east property boundary. This would have required back-filling. However, there is no evidence of a need to back-fill along the northern property line. The vegetation type and soil color indicate a dry upland soil along a saddle ridge which separates the marsh adjoining Lake Lowery and the sink. Another similar ridge separates the sink from Lake Henry to the south. Nevertheless, soil borings were made along the northern boundary of the theater to obtain information about the natural ground elevation from soil stratigraphy. With the assistance of a Soil Conservation Service soil scientist, the soil borings were found to indicate native soils along that boundary.

At location "B", similar results, which also indicate no evidence of back-filling, were found near outlet location "B", shown in Figure 23. Cross-section A-A' shown in the figure locate the site where the soil borings were taken. The natural topography indicates an approximate elevation of 132 feet (NGVD).

In conclusion, connections to the Peace River Basin continue to allow drainage to discharge into the Peace River Basin when Lake Lowery water levels exceed 131.6 feet (NGVD). Water levels in Lake Lowery would have to exceed 136.0 feet (NGVD) before the railroad tracks would be overtopped. However, the railroad would create unnaturally high flood levels only if the lake stage exceeded 132 feet (NGVD).

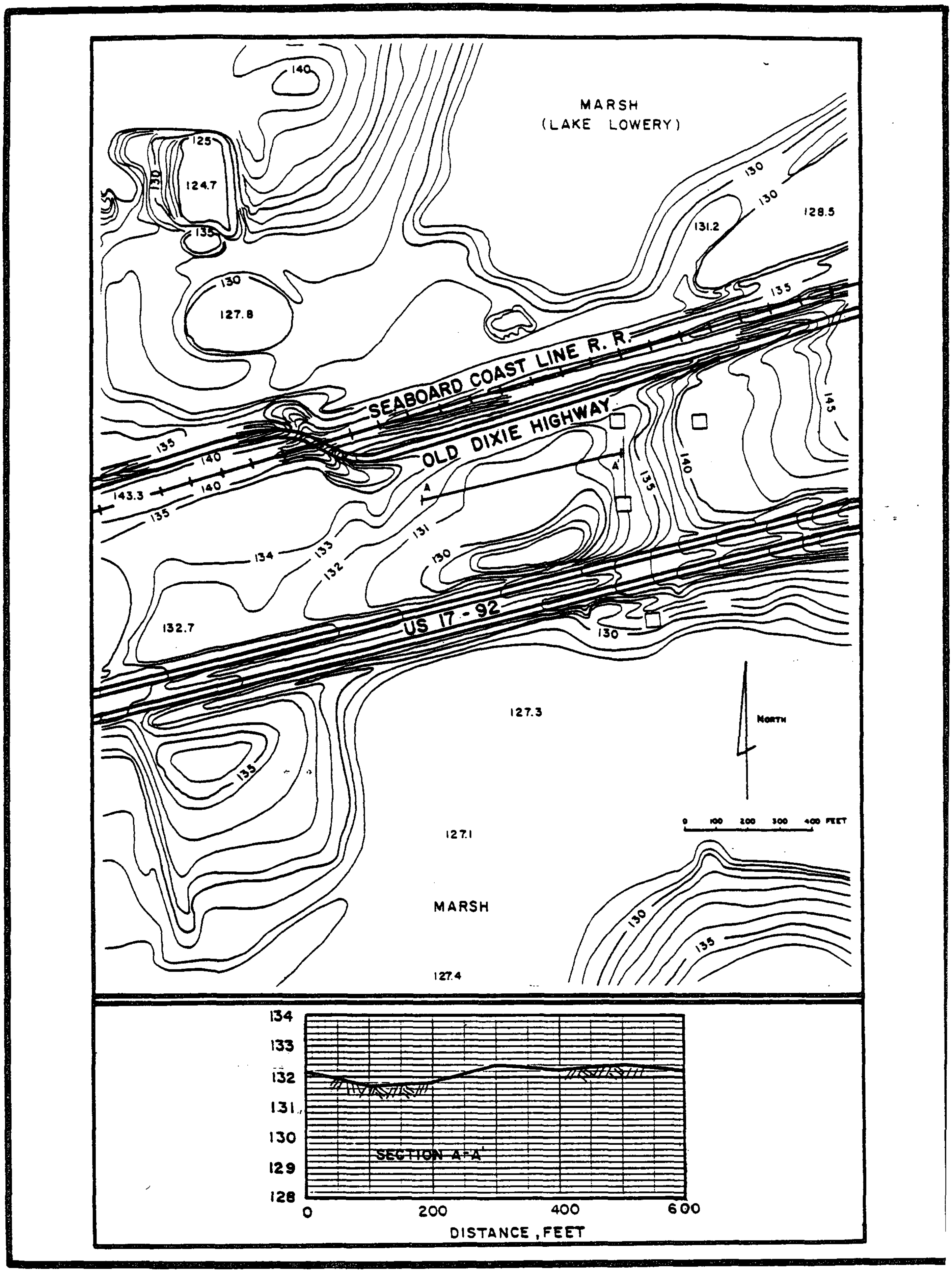


Figure 23. Cross-Section V‑ ‑ of Blocked Location "B" .

SOLUTION DEVELOPMENT

The development of alternative solutions to the problem of localized flooding should include consideration of the impacts of each solution on recreation, navigation, water quality, fish and wildlife, wetlands, floodplains and other environmentally sensitive lands, reasonable beneficial use and other factors relating to public health, safety and welfare pursuant to the provision of Chapter 17-40.07 Florida Administrative Code. Several conceptual alternatives were developed and evaluated in detail to provide an indication of the benefits and the potential adverse impacts for each design.

DESCRIPTION OF ALTERNATIVES

No Action Alternative

This alternative requires no changes to the drainage system; hence, no cost. Flood damages would continue to recur periodically.

Southward Diversion Alternatives

Three different designs were evaluated under this alternative.

1. Open Ditch - This design requires the construction of an open ditch to allow overflow from Lake Lowery to discharge into Lake Henry. Based on requests from Polk County officials this alternative was designed such that flow from Lake Lowery will begin at a lake stage elevation of 128.5 feet (NGVD). Because of the projected adverse impacts to the wetlands adjoining Lake Lowery

the construction of a water control structure at the south shore of Lake Lowery with a fixed crest elevation of 130.0 (NGVD) was included. The open ditch would use an existing 36 inch conduit under U.S. Highway 17/92.

2. Closed Conduit with No Northerly Connection - This design includes a drop-inlet connected to a closed conduit to discharge overflow from Lake Lowery into Lake Henry instead of an open ditch. Permanent blockage of structure 1 (Figure 20) would be necessary to prevent surface water inflow from the marsh north of Lake Lowery (Sub-Basin 4). By preventing the inflow of surface water from the marsh north of Lake Lowery the discharge into Lake Henry would be reduced.

3. Closed Conduit with Controlled Northerly Connection- This design includes a drop-inlet connected to a closed conduit to discharge overflow from Lake Lowery into Lake Henry combined with the construction of a gate at structure 1 (Figure 20) designed to regulate the flow of surface water from the marsh north of Lake Lowery (Sub-Basin 4) into the lake. By regulating flow through the gate at structure 1 the back-water profile in Sub-Basins 2 and 4 could be minimized during periods of high water levels. This control would also allow Lake Lowery to receive inflow from the same marsh (Sub-Basin 4) during low water level periods, because flow normally travels from that marsh in a southward direction into Lake Lowery.

Northward Diversion Alternatives

Two different designs were evaluated under this alternative.

1. Pumped Drainage - This design requires the construction of a pump station to lift water from Lake Lowery and discharge it into the marsh situated north of the lake. A pump with a 15 cubic foot per second (cfs) capacity was selected. The design requires southward flow through structure 1 (Figure 20) to be stopped during pumping. This configuration uses the Lake Lowery Road embankment as a levee and also requires the construction of a gate control at structure 1.

2. Channel Improvements - This solution requires large scale drainage improvements to be made to increase the gravity discharge capacity into the Big Creek Basin (Figure 19) from Lake Lowery by excavating drainage ditches and/or clearing away some of the existing vegetation. The most restrictive reach in the watercourse is a five mile section between Polk County Road 17 and Interstate 4. Approximately 3000 feet of the section through Black Ford is extremely restrictive. Drainage improvements to increase conveyance capacity may cause adverse impacts to the wetland environment and significantly increase the peak flow rates downstream. Nevertheless, clearing vegetation through a 20 foot wide waterway through this reach was investigated as a means of removing surface water more rapidly from the Lake Lowery area.

Flood Protection Levee Alternative

This alternative requires the construction of an earthen berm surrounding the flood-prone, high flood-damage-potential area. This solution provides protection to the residential areas up to a selected high water mark at a greater cost savings than protecting each residence on an individual basis.

High water levels in the lake are only a few feet above normal lake levels; therefore, the protective levee would be a reasonably low structure. Roadways which transit the area would need to be raised slightly and they too would serve as flood control berms. Sections of the levee would need to be constructed across some residential property boundaries. At most locations the berm would extend a maximum of 2 to 3 feet above the existing land surface elevations.

Presently, two canals provide navigable access to most of the residences. In order to continue to use the canals for waterways, a gate would have to be constructed. This gate would be closed when lake levels rise to flood stages. A boat lift may be needed to provide access to the lake during such periods. An interior drainage system and lift station would be required during times when the canal is closed. This system would remove water which seeps through the earthen berm and rainfall which falls within the perimeter of the diked properties.

Flood Proofing Alternatives

Three different designs were evaluated under this alternative.

1. Elevating Septic Tanks - This solution requires raising the septic tanks which are affected during flood periods. This alternative specifies all septic tanks be raised to a minimum elevation of 134.0 feet (NGVD). Following this specification, a total of 164 septic tanks would need to be raised and most of those would require the installation of a lift pump and dosing tank. It is assumed that subsequent permits issued by the County for the Lake Lowery area would specify a minimum septic tank elevation of 134.0 feet (NGVD).
2. Elevating Septic Tanks and Low-Lying Structures - This solution includes raising septic tanks as well as low-lying residences affected during high water periods. Structural and contents damages would be reduced by raising the first floor elevations of the affected houses. This alternative specifies that all residences with first floor elevations below 133.0 feet (NGVD) be raised to 134.0 feet (NGVD). Three of the residences are mobile homes which could be inexpensively elevated However, there are twelve permanent structures, some of which are on at-grade concrete slab foundations that would require raising. Elevating this type of structure would be difficult and expensive.

3. Regional Wastewater Treatment System - This solution requires the construction of a wastewater treatment system. Since a major portion of the damages are related to failures of the septic tank and drain fields, a wastewater collection and treatment system would be constructed to reduce or eliminate these problems. This alternative includes the construction of a collection system, lift station, wastewater treatment plant, and a slow-rate irrigation or overland flow system for effluent disposal.

<u>Relocation Alternative</u>

This solution would be accomplished through land purchases. Flood damages would be reduced by the purchase of the affected property by an appropriate public agency. This is a costly alternative. Twelve home-sites are at a significant risk of flood damage (i.e., first floor elevations below 134.0 feet NGVD); therefore, if they were removed from the list of flood-prone structures, the expected damage would be reduced.

HYDROLOGIC ANALYSIS

METHODOLOGY

For the purposes of this analysis a continuous hydrologic simulation model which maintains an accounting of all the water movement within the system over a continuous time period (i.e., a continuous model) was developed by the SJRWMD. This model produced simulated hydrologic data (e.g., runoff, storage, lake stage, etc.) which can be used to determine the frequencies of hydrologic events by statistical methods.

The Study area analyzed by the model is shown in Figure 19. The area delineated for analysis extended beyond the Greater Lake Lowery Basin in order to address impacts from drainage modifications on streamflow outside the Lake Lowery area and to use streamflow data from downstream gaging stations for calibrating hydrologic parameters. This resulted in sub-basins of various sizes. For example, the area north of Polk County Road 17 included 99 square miles of surface area, yet is modeled by a single sub-basin (Sub-Basin 11) with the outlet located at a gaging station where the main channel flows under State Road 33. Simulating this area was valuable for calibrating the model to yield accurate long-term average surface water runoff. On the other hand, the Big Creek Basin was simulated in much greater detail. The area north of Interstate 4 was segmented into five sub-basins (Sub-Basin 6, 7, 8, 9 and 10) of roughly equal size. The outlets for each sub-basin are Interstate 4, Dean Still Road, Sand Mine Road, State Road 474, and the United States Geological

Survey (USGS) streamflow gaging station near Lake Louisa, respectively.

The model was calibrated using the streamflow data from the Lake Louisa gaging station, because daily streamflow records at this location were available from 1959 to present. Emphasis was placed on simulating the effect of cyclonic precipitation to model continuous, area-wide rainfall, rather than the more localized convective precipitation. Flood and drought periods were simulated to obtain the diametric impacts of proposed drainage modifications. After calibrating the model for area-wide storms, the results were statistically compared to observed values. Figure 24 illustrates the observed stage-duration relationship compared to the simulated relationship for the period 1960 to 1984. Figure 25 shows the actual and simulated frequency of floods and droughts for the Lake Louisa streamflow gaging station during the period 1960 to 1984. The data presented in Table 8 summarizes the results generated by the simulation model.

SOUTHWARD DIVERSION ALTERNATIVES

Impacts to Lake Lowery

The first design (Open Ditch) is the least beneficial of the configurations which employ an overflow outlet to divert flow into Lake Henry. This design also produces more frequent and severe low lake level problems during dry periods (Figure 26). The second design (Close Conduit with No Northerly Connection) reduces flood levels significantly; however, it too severely lowers lake levels during dry periods (Figure 26). The third

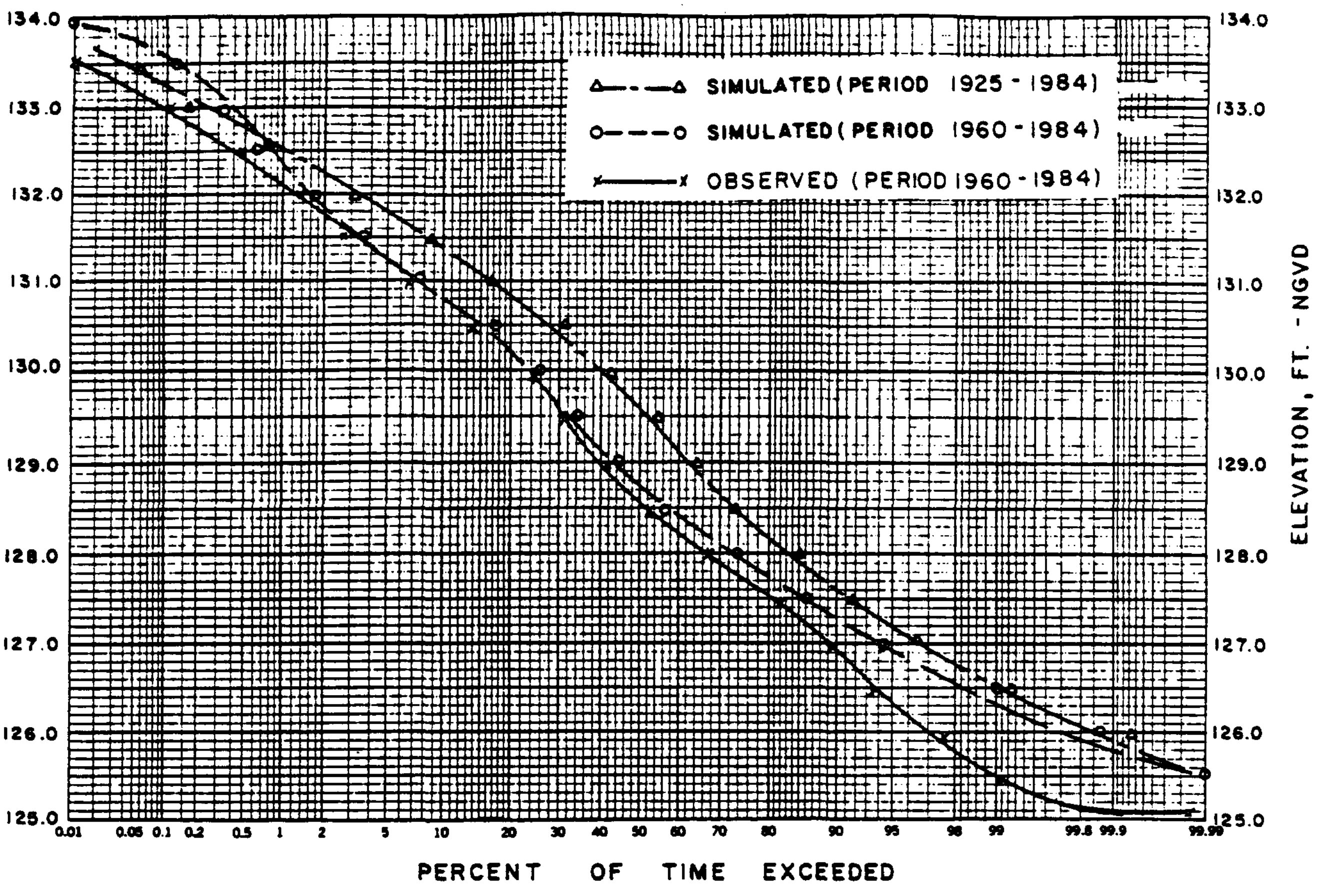


Figure 24. Observed and Simulated Stage Verses Duration Curves.

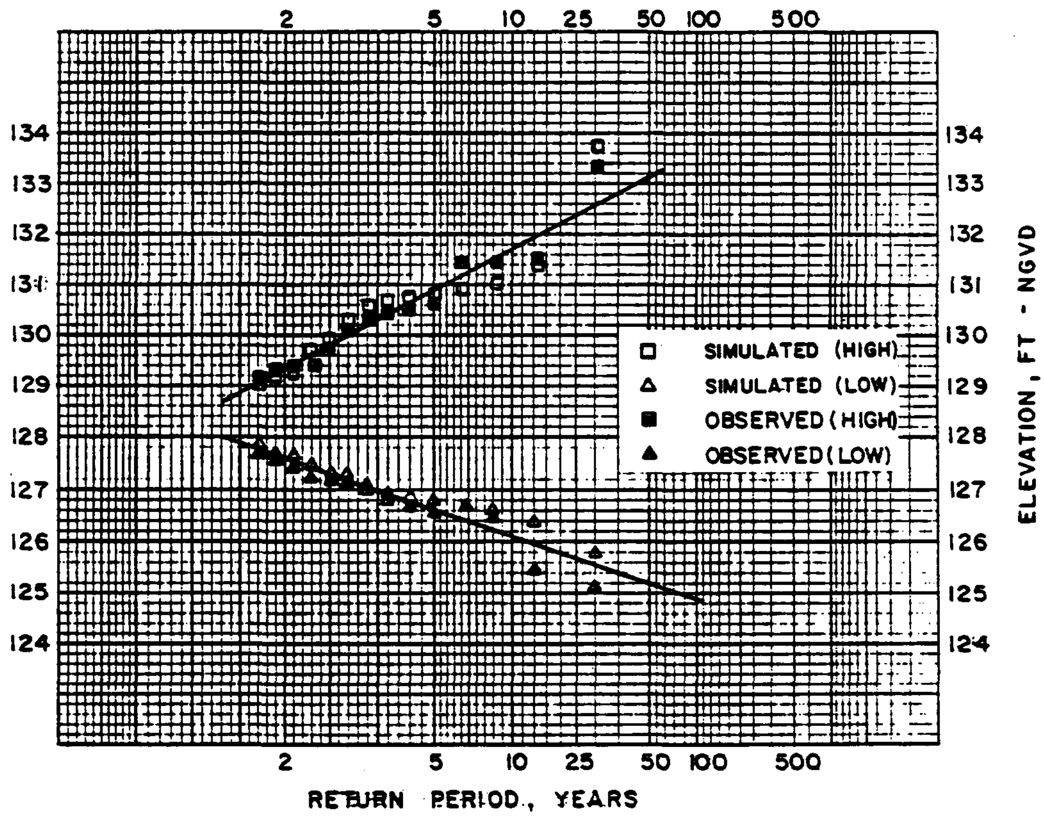


Figure 25. Observed and Simulated Stage Verses Return Period Curves.

TABLE 8

LAKE LOWERY STAGE-FREQUENCY FOR THE NO ACTION ALTERNATIVE (NA)
AND THE THREE SOUTHWARD DIVERSION ALTERNATIVES (SD1, SD2, AND SD3)

Lake Lowery High Stages (Feet-NGVD)

Frequency (Yrs)	1-Day Duration NA	SD 1	SD 2	SD 3	60-Day Duration NA	SD 1	SD 2	SD3
2	130.30	129.60	129.03	129.92	129.98	129.28	128.64	129.56
5	131.55	130.63	130.03	130.70	131.17	130.30	129.63	130.27
10	132.27	131.24	130.58	131.09	131.82	130.87	130.14	130.60
25	133.08	131.96	131.16	131.49	132.54	131.50	130.67	130.91
50	133.63	132.46	131.55	131.73	133.02	131.93	131.00	131.09
100	134.15	132.94	131.91	131.95	133.45	132.33	131.30	131.24
200	134.65	133.40	132.24	132.14	133.85	132.70	131.57	131.37

Lake Lowery Low Stages (Feet-NGVD)

Frequency (Yrs)	30-Day Duration NA	SD 1	SD 2	SD 3	183-Day Duration NA	SD 1	SD 2	SD 3
2	128.91	128.30	127.57	128.67	129.70	129.04	128.35	129.39
5	127.71	127.29	126.46	127.61	128.51	128.07	127.30	128.43
10	127.08	126.72	125.87	127.02	127.89	127.54	126.74	127.87
25	126.41	126.11	125.25	126.38	127.24	126.97	126.16	127.25
50	125.99	125.72	124.85	125.97	126.82	126.60	125.78	126.83
100	125.62	125.36	124.51	125.60	126.45	126.27	125.45	126.46
200	125.29	125.04	124.20	125.26	126.12	125.97	125.15	126.11

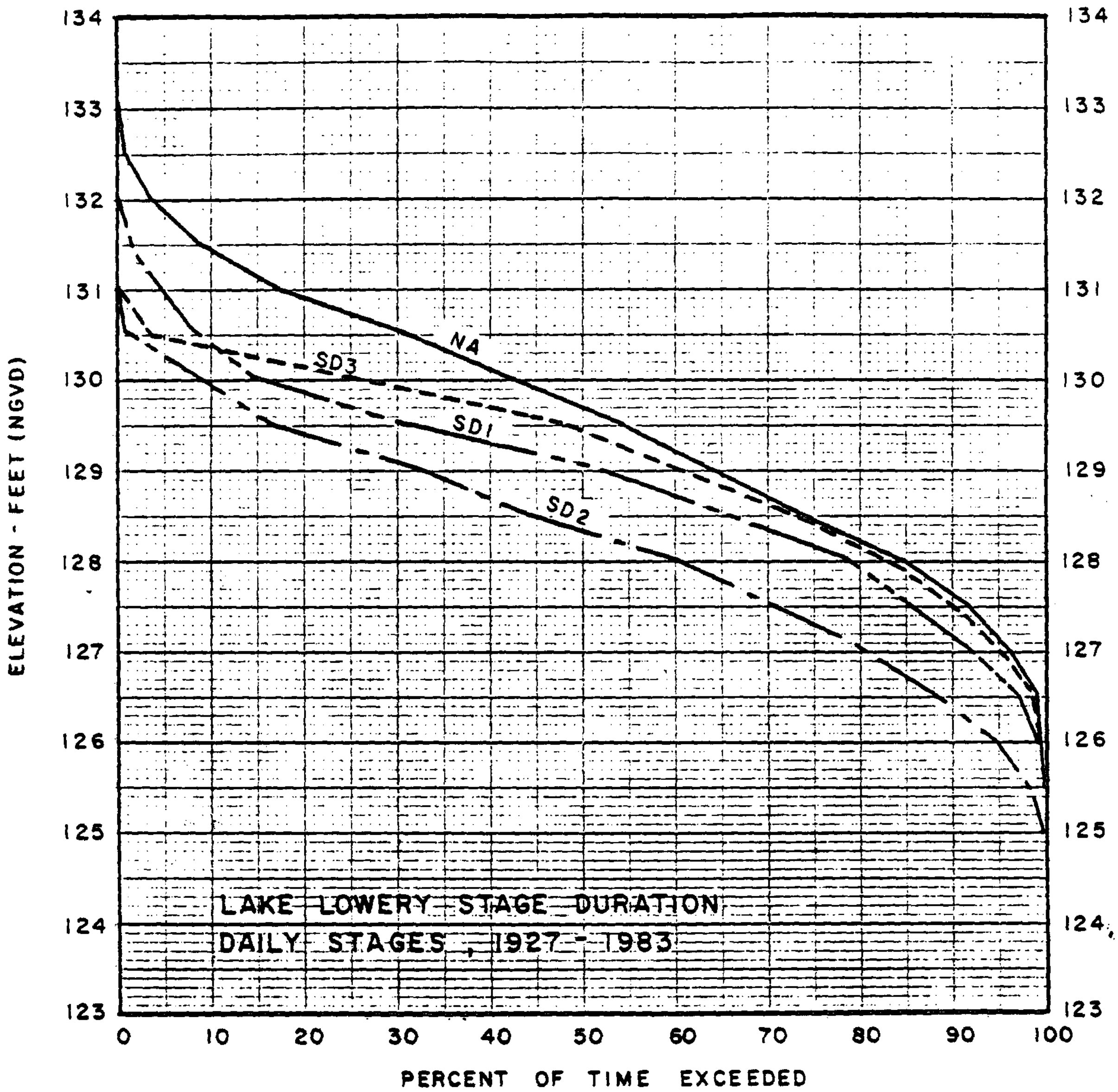


Figure 26. Lake Lowery Stage Verses Duration Curves for No Action Alternative and Southerly Diversion Alternatives.

design (Closed Conduit with Controlled Northerly Connection) reduces 10 year, 25 year and 100 year flood levels by 1.18, 1.59, and 2.25 feet respectively, while maintaining low lake levels to within 0.2 feet of existing conditions.

Impacts to Areas North of Lake Lowery

The three surface-water systems which would be impacted by drainage modifications in the Lake Lowery area are the Palatlakaha, Withlacoochee, and the Peace River basins. Their relationship is such that any increase in drainage through one basin would decrease the flow through one or both of the others. This is particularly important given recent concern over the loss of drainage area within the Palatlakaha River Basin to the Withlacoochee River Basin due to the excavation of agricultural drainage canals (East Central Florida Regional Planning Council, 1983; and Pride, et al., 1966). Therefore, it is necessary to quantify any proposed changes in the hydrologic performance of the total area. The statistical characteristics of the streamflow at three locations north of Lake Lowery Road were analyzed. These sites are at Polk County Road 17, Dean Still Road, and at the outlet of Big Creek into Lake Louisa.

Based on this analysis the first design (Open Channel) has the greatest effect on streamflow during both high and low water periods. Based on this first design, during high water periods flow drains from the marshes north of Lake Lowery (Sub-Basins 1-5) into Lake Lowery and finally into Lake Henry. This means that 24,580 acres of wetland would drain into the Peace River Basin rather than the Palatlakaha River Basin. During low water periods, drought conditions are intensified. Streamflow at Polk County Road 17 would be reduced from a mean discharge of 1.3 cfs to 0.4 cfs. The 2 year, 30 day mean flow rate would drop from 2.5 cfs to 0.7 cfs. The percent of time that flow would be less than 1.0 cfs increased from 45 to 80. Downstream, at Dean Still Road the impact would not be as severe. The mean flow rate would drop from 8.0 cfs to 7.3 cfs. A flow of less than 1.0 cfs would occur 22 percent of the time compared to 20 percent for the existing condition. Further downstream, at the outlet from Big Creek into Lake Louisa, there would be no change.

The second design (Closed Conduit with No Northerly Connection) results in a maximum increase in water levels of 0.1 foot in the marshes north and west of the lake during high water periods. Since the Lake Lowery storage area (Sub-Basin 3) would be closed to flow coming from the northern marshes, flow would be diverted into the Big Creek Basin and the Withlacoochee Basin.

The third design (Closed Conduit with Controlled Northerly Connection) results in an initial increase in the water level in the marsh north of the lake during high water periods, followed by a reduction in water levels once the gate at structure 1 (Figure 20) is opened and flow is allowed to drain into Lake Lowery. The period of inundation in the surrounding marshes is reduced by less than 10 percent.

<u>Impacts to Lake Henry</u>

Although discharge into Lake Haines, which is in the Peace River Basin, can occur under existing conditions, there is no flow from Lake Lowery directly to that basin. Hence, the diversion of flow from the various alternatives would increase flow.

The first design (Open Ditch) results in the largest flow rates and volumes, while the second and third designs (Closed Conduit with No Northerly Connection and Closed Conduit and Controlled Northerly Connection) substantially reduce the volume of water being diverted (Table 9), yet do not greatly reduce the one day maximum for stages less frequent than the 10 year high (Table 8).

NORTHWARD DIVERSION ALTERNATIVES

The first design (Pumped Drainage) was simulated but later eliminated from further analysis when the benefits were found to approximate the third design of the Southerly Diversion Alternatives, but at a much greater cost. The second design, (Channel Improvements) offers little benefit. The flat hydraulic gradient and the dense vegetation would require expensive maintenance to provide even a slight increase in conveyance of flood water from Lake Lowery. Therefore, this design was also eliminated from further analysis.

FLOOD PROTECTION LEVEE ALTERNATIVE

This design has little effect on water levels outside of the project area (approximately 150 acres). Water levels could be controlled within the diked system when the lake stage is above an acceptable level. This would eliminate damages up to a

TABLE 9

LAKE LOWERY DISCHARGE FREQUENCY
FOR THE SOUTHWARD DIVERSION ALTERNATIVES
(SD1, SD2, and SD3)

Frequency	Lake Lowery Diversion Flow, cfs								
	1-Day Average			30-day Average			120-day Average		
(Yrs)	SD1	SD2	SD3	SD1	SD2	SD3	SD1	SD2	SD3
2	14	0	2	12	0	1	9	0	1
5	23	3	17	22	0	12	20	0	6
10	27	15	23	26	10	17	25	5	11
25	31	24	26	30	19	23	29	10	15
50	33	27	27	33	24	26	31	13	17
100	35	29	29	34	28	29	32	16	19

selected high water stage for which the surrounding dikes are designed. It was assumed that the levee would be designed for a 100 year flood. The interior drainage system was assumed to be designed for a 10 year, 24 hour storm with a maximum stage of 131.5 feet (NGVD). These criteria and a regulated stage of 130.0 feet (NGVD) result in a required pump capacity of 15 cfs (6560 gallons per minute).

SUMMARY OF HYDROLOGIC ANALYSIS

A continuous hydrologic simulation model was used to generate 60 years of daily surface water levels and stream flow data for the study area. Drainage modifications as well as the existing condition were simulated. Surface water levels and streamflows from each design were statistically analyzed to determine their respective impacts to the lake's hydraulic system.

It was found through simulation analysis that achieving flood damage reduction objectives via diversions (Southward and Northward Diversion Alternatives) would require the isolation of Lake Lowery from the surrounding marshes during high water periods, and opening the lake to inflow from the marsh north of the lake during low water periods. The third design of the Southward Diversion Alternatives (Closed Conduit with Controlled Northerly Connection) was found to offer a substantial reduction of high water levels while minimizing low water levels in the lake and surrounding marshes.

None of the alternatives produced a detectable reduction in streamflow at the outlet from Big Creek into Lake Louisa.

SOCIO-ECONOMIC ANALYSIS

METHODOLOGY

The consideration of all social and economic impacts from the various solution configurations was a complex task. Some of the designs are difficult to quantify in monetary terms and can only be judged relatively to one another. Therefore, in order to quantify the relative degrees of flood damage, five categories have been established -- direct, indirect, secondary, intangible, and uncertainty damages.

Direct damages are those normally associated with flooding, they include damage to private and public property by inundation, erosion, and sediment deposition. Indirect damages include the value of lost business and services and the cost of emergency measures which are incurred during flood periods. Secondary damages occur when the economic loss extends beyond those persons experiencing the flooding, this includes lost services or goods to persons outside the flood area. Intangible damages include such factors as environmental quality, social well-being, and aesthetics. Such factors are not usually assigned an economic value, but considered in a overall assessment. Uncertainty damages exist when residents must live with an ever present potential for flood damage, not knowing when or how severe it may be. The uncertainty damage may be estimated by how much residents would be willing to pay, beyond the expected value of flood damages in order to avoid unplanned and possibly financially severe economic loss and social hardships.

The damages reported in the Lake Lowery area during the 1982-84 flood were a result of the following:

1. Inundation of the first floor of one residence.
2. Malfunctioning and/or failure of individual septic systems.
3. Inundation of private wells and pumps.
4. Inundation of private unpaved roads.
5. Inundation of lawns and landscapes.
6. Fear of severe flood damage if a large storm occurred while the water level remained high.
7. Inundated pasture and agricultural land.

The most common problem was the failure of septic tank drain fields due to high water table levels. The duration of high water table levels was the major factor causing septic tank failures. Most residents did not experience direct flood damage (e.g., physical damage to personal property); however, longer durations and higher flood stages than those which occurred in 1983 could cause considerable damage.

To quantify the relative merits of each alternative, the monetary cost incurred through flood damage was required. This would include the cost of property damage, failure of septic systems, and emergency measures. A detailed analyses of the residential area located on the north side of the lake was done using standard methods for estimating the dollar value of damage to property, structures, and contents, as a function of water level. Elevations of first floors and septic tanks for nearly all residences were surveyed. Standard relationships were then used to estimate the damage incurred at a given water level as a percent of the total structural value and contents value. Structure values were obtained from the county tax assessor's records (adjusted to reflect the true-market value) and contents values were estimated as a percent of the structural value. The depth versus percent damage curves used in the analysis are given in Table 10. These relationships were published by the Federal Flood Insurance Administration in 1970. It was assumed that the average value of structural damage by flood waters was 20 percent of the total value of the structure for first floor inundation. This ratio is lower than the 30 to 35 percent ratio used in other studies (Grigg, 1975). This can be justified by the lack of depth of the flood water and that the slow rate of rise in water table levels would allow a considerable amount of time to lift and place items above the anticipated high water mark.

The major factors in the indirect damage category applicable to the Lake Lowery area are the inconveniences and economic costs from the failure of septic tank and drain field systems. These were not physical damages and standard procedures are not available for guidance in estimating this cost as a dollar value. One option which could be used is to assume that if the system fails the resident will evacuate and then estimate the cost of evacuation (i.e., temporary room and board). The validity of this assumption is questionable since it has been determined that few of the residents who experienced septic system failures actually evacuated. The majority of those residents choose to accept the inconveniences and continue living at home. This implies that the cost of evacuation was greater than the inconvenience

TABLE 10

DEPTH-PERCENT DAMAGE RELATIONSHIPS
(Federal Insurance Administration, 1970)

Stage (Depth)	1 Story Without Basement Structure (%)	1 Story Without Basement Contents (%)	1-1/2 and 2 Story without Basement Structure (%)	1-1/2 and 2 Story without Basement Contents (%)	Split Level without bsmt Struct (%)	Split Level without bsmt Conts (%)	Mobile Homes Struct (%)	Mobile Homes Conts (%)
-3	0	0	0	0	0	0	0	0
-2	0	0	0	0	0	0	0	0
-1	0	0	0	0	0	0	0	0
0	7	10	5	7	3	2	8	0
1	10	17	9	9	9	19	45	20
2	14	23	13	17	13	32	64	50
3	26	29	18	22	25	41	74	60
4	28	35	20	28	27	47	79	70
5	29	40	22	33	28	51	80	73
6	41	45	24	39	33	53	81	76
7	43	50	26	44	34	55	82	79
8	44	55	31	50	41	56	82	82
9	45	60	36	55	43	62	82	85
10	46	60	38	58	45	69	82	85

of remaining home. In any case, the best means of assessing a dollar value for indirect damage appears to be by assigning a cost per day and multiplying that value by the number of days the system malfunctioned.

The principal failure mechanism of the septic system was hydraulic. Figure 27 shows two schematics which illustrate the relationship of the water table and the septic system. The top figure represents a standard septic tank and soil absorption system (ST-SAS) and the lower schematic shows a mound system. For purposes of analysis it was assumed that the water table level was the same as the lake level throughout the residential area.

When the water table rises to the level of the drain field pipe the waste treatment efficiency of the soil absorption system begins to decline. Recommended standards call for a minimum of two feet (measured vertically) between the water table and the drain field pipe, although an ST-SAS will continue to function hydraulically at lesser differences in drain-field-pipe/water table elevations. Hydraulic failure begins to occur during periods of high volume loading; for example, during washing machine or dishwasher use. At higher water table levels the severity of the problem increases until a total hydraulic failure occurs and effluent begins backing-up inside the building. This threshold elevation is dependent on the percolation rate of the drain field. The critical elevation is represented by ELCRIT in Figure 27.

To estimate costs for these conditions an assumption was made that user inconvenience will begin when the water table

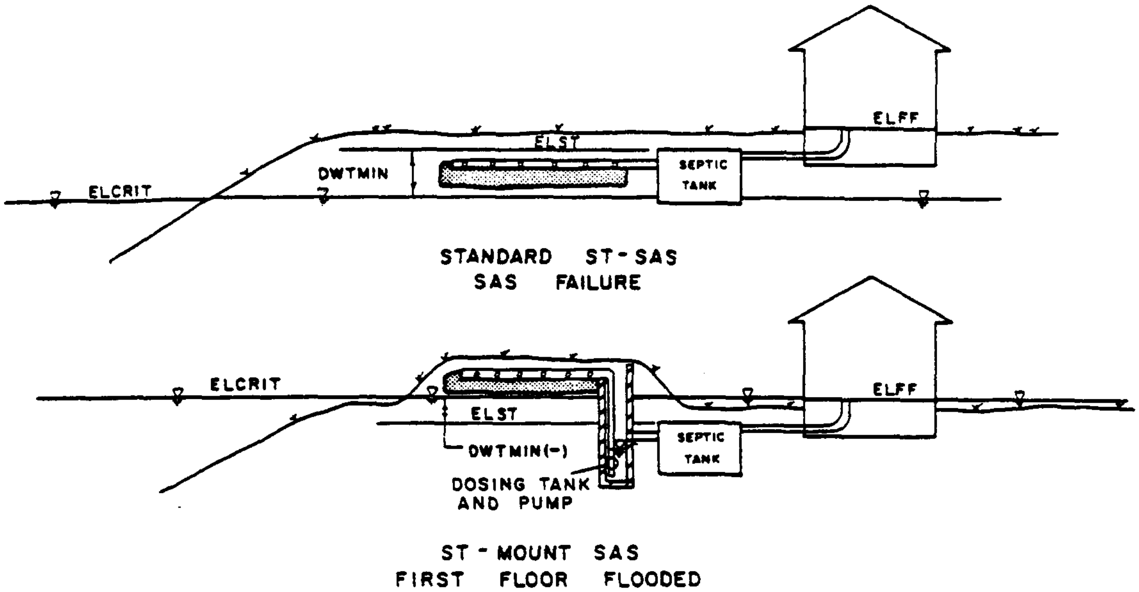


Figure 27. Schematic of Typical Septic Tank System.

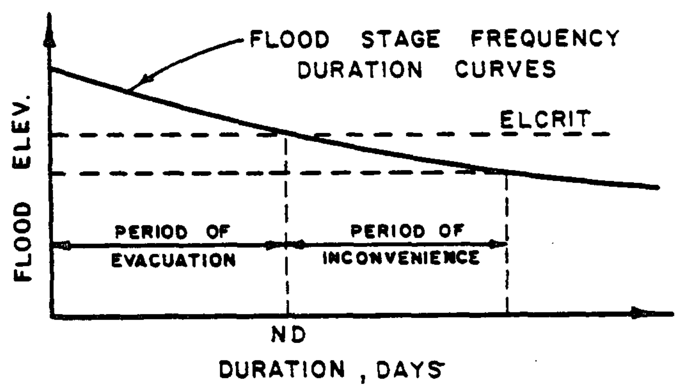


Figure 28. Evacuation and Inconvenience Period Graph.

To estimate costs for these conditions an assumption was made that user inconvenience will begin when the water table elevation is within 0.5 feet of the top of the septic tank (ELST). Total failure was assumed to occur when the water table elevation reached the top of the septic tank. Figure 28 indicates how duration times were determined from stage-frequency-duration data. The damage per day when the water table elevation reached the top of the septic tank was judged to be five dollars. The damage per day at impending failure (i.e., water table elevation is 0.5 foot below ELST) was judged to be zero dollars. The cost function for intermediate points between the two levels was assumed to be proportional to their respective differences in elevation, as shown in Figure 29. For higher water table elevations than ELCRIT it was assumed that evacuation would begin; therefore, no inconvenience damages would be incurred during this period. Evacuation would also begin when water levels reached first floor elevation (ELFF). The point of evacuation for each residence was determined from the stage-duration data for the lake. A cost of $30.00 per day was given for up to 14 days of evacuation time and $10.00 per day was given for time periods which exceeded 14 days.

EVALUATION OF ALTERNATIVES

Flood Damage Reduction

Using the lake stage data developed by simulating each design, the average annual flood damage was determined. For alternatives requiring no drainage modifications the existing condition flood stages were used and the appropriate changes in

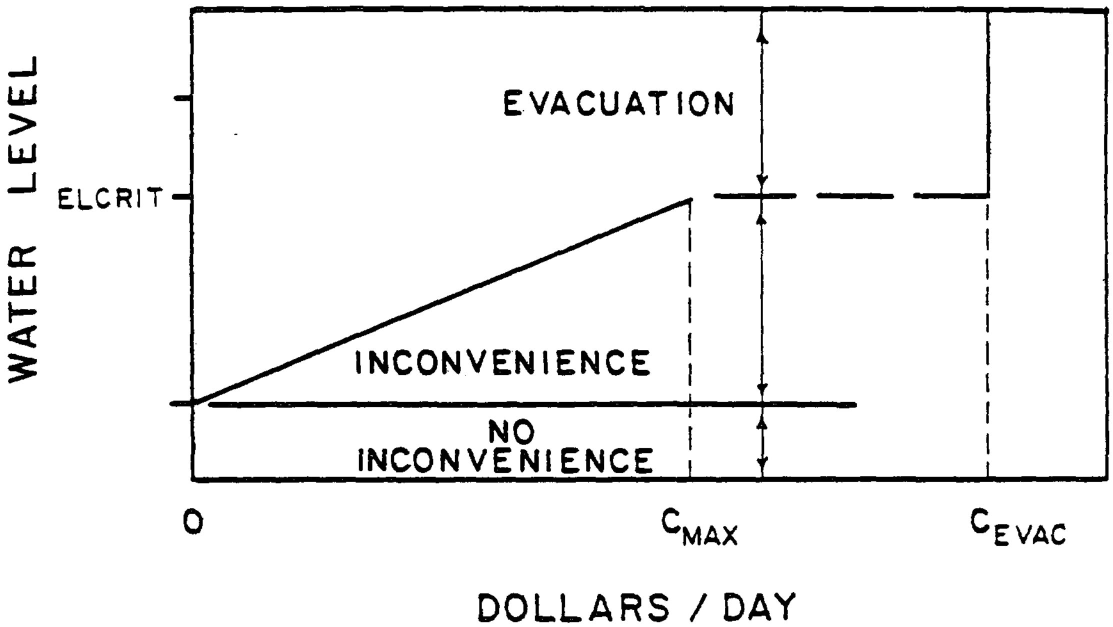


Figure 29. Inconvenience Cost Function Graph.

alternatives requiring no drainage modifications the existing condition flood stages were used and the appropriate changes in structural and/or septic system elevations were assumed. The damages were calculated for the 172 residences on the north side of the lake. It may be noted that the drainage modifications would also reduce flood damages at other lake-front sites. The damage estimates for each design are summarized in Table 11. Detailed information on each design analysis are included in Appendix A.

Low Lake Level Conditions

The second design of the Southward Diversion Alternatives (Closed Conduit with no Northerly Connection) produces a significantly lower lake stage -- a 1.21 foot drop in the 10 year, 30 day low stage. The first design of the Southward Diversion Alternatives (Open Channel) results in a 0.36 foot drop, but also lowers water levels throughout the marshes north and west of the lake. The third design of the Southward Diversion Alternatives (Closed Conduit with Controlled Northerly Connection) results in only a 0.06 foot drop. The results of the low lake level calculations by the simulation model are listed in Table 8.

Environmental Impacts

The changes which could result from modifications to the hydraulic regime could alter vegetation and wildlife habitats. The impacts would be greatest in the surrounding marshes and the littoral zone where plants and animals adapted to periodic, shallow inundation would be subjected to more frequent and longer

TABLE 11

FLOOD DAMAGE ESTIMATES FOR THE LAKE LOWERY RESIDENTIAL AREA

	Average Annual Flood Damage (Dollars)			
Alternative	Structure & Content	Evac.	Inconv.	Total
No Action	16,000	33,000	3,000	52,000
Southward Diversion Design 1	4,000	10,000	1,000	15,000
Southward Diversion Design 2	1,000	3,000	1,000	5,000
Southward Diversion Design 3	1,000	5,000	1,000	7,000
Flood Proofing Design 1	16,000	3,000	- 0 -	19,000
Flood Proofing Design 2	10,000	2,000	- 0 -	12,000
Flood Proofing Design 3	16,000	3,000	- 0 -	19,000
Flood Protection Levee	6,000	4,000	1,000	11,000

duration drought conditions. This could be beneficial in some cases since the extreme high water stages are reduced to a lower level, thereby reducing flood damage to vegetation not adapted to flooding. A number of trees bordering the eastern shore of Lake Lowery died during the 1982-84 high water period apparently as a result of the extended duration of root-zone saturation. These natural damages are regarded as relatively minor when compared to the impacts created by lowering normal water levels and reducing fluctuation ranges.

A constraint to design configurations was that no significant loss of wetlands would be accepted in order to achieve flood damage mitigation. Of the drainage modifications analyzed, the first and second designs of the Southward Diversion Alternatives (Open Ditch and Closed Circuit with No Northerly Connection) result in excessive detrimental impacts while the impact from the third design of the Southward Diversion Alternatives (Closed Conduit with Controlled Northerly Connection) is much less severe.

Recreational Impacts

Significantly lower water levels in Lake Lowery would be detrimental to boating and fishing activities. Of the alternatives investigated only the Southward Diversion Alternatives result in lowered lake levels. Of these, the third design of the Southward Diversion Alternatives (Closed Conduit with Controlled Northerly Connection) results in the least lowering of lake levels. This design produces stages lower than those under existing conditions by 0.24, 0.10, and 0.06 feet for the 2, 5, and 10 year frequency drought events, respectively.

PROJECTED COSTS AND BENEFITS

Project construction, operation, and maintenance costs have been estimated for the remaining, viable design configurations. These are rough estimates not based on detailed plans and specifications. For comparing the projected annual costs relative to each other, all construction costs are calculated at an 8 percent annual interest rate, compounded monthly, and amortized over a 40 year expected life span.

Southward Diversion Alternatives

1. Open Ditch - It is estimated that the construction cost of the outlet structure and conveyance channel would be about $40,000.00. This does not include the cost of modifications to the U.S. Highway 92 culvert. Operation and maintenance costs were estimated at five percent of the construction cost or an average of $2,000.00 per year. The total annual cost is $5,340.00 per year.

2. Closed Conduit with No Northerly Connection - The estimated cost of this design is the same as the first design (Open Ditch), with the exception of a negligible cost of permanently blocking the culvert under Lake Lowery Road.

3. Closed Conduit with Controlled Northerly Connection - The cost of this alternative includes $5,340.00 per year for the diversion structure plus the cost of a gated structure under Lake Lowery Road. Protection of the gates from vandalism and tampering would require a fenced enclosure. The cost of the fencing is estimated at about $5,000.00. It is assumed that the existing

flashboard riser would be used in addition to a culvert gate. The cost for regulating water levels by operating the gate would depend on how frequently the gate must be operated and by whom it is operated. The total annual cost is estimated to be $6,000.00.

Flood Protection Levee Alternative

This is a complex plan which makes cost estimation difficult. A thorough design is required to develop a reliable cost estimate. However, the following is an itemized list of the components and associated estimated costs of construction of this alternative.

Construct a 6,000 foot perimeter berm at $50.00 per linear foot	$300,000.00
Placement of bulkheads and stoplogs	50,000.00
Construct bridges	50,000.00
Alter interior canals and install drainage system	100,000.00
Install 6560 gpm pump station	30,000.00
Construct boat lift	20,000.00
	$550,000.00
Engineering, legal, and administration costs	113,000.00
	$663,000.00

Including maintenance cost of $5,000.00 per year, the total annual cost is $60,320.00.

Flood Proofing Alternatives

1. Elevating Septic Tanks - The cost of this alternative includes raising 164 septic tanks to an elevation of 134.0 feet (NGVD) and the increased operation and maintenance cost for pumping. Installation of a single mound septic tank system was estimated to be $3,500.00, with an annual operation and maintenance cost of $25.00 per year. The total annual cost is $52,000.00.

2. Elevating Septic Tanks and Low-Lying Structures - Costs include those incurred in the first design (Elevating Septic Tanks) plus the cost of raising 12 houses having an estimated average value of $35,000.00 each, along with three mobile homes. Assuming the cost of raising the houses is 70 percent of their value, and the cost of raising the mobile homes are $1,000.00 each for plumbing and electrical modifications, the total annual cost is $76,770.00.

3. Regional Wastewater Treatment System - A community wastewater treatment system with a spray or overland flow field for effluent disposal would require approximately five acres. It was assumed that the plant design capacity would be for a future density of 250 residences rather than the 172 currently located in the area. The costs are approximately $300,000.00 for the treatment plant, $25,000.00 for land purchases, $170,000.00 for the collection system, for a total cost of $495,000.00. Including $25,000.00 per year for maintenance and operation, the total annual cost is $66,300.00.

Relocation Alternative

Assuming a $50,000.00 average value for 50 permanent residences and $20,000.00 for mobile homes, lots and relocation costs, the total property value is $4,780,000.00 or a total annual cost of approximately $400,000.00. This design could be varied by purchasing only the flood-prone residences which suffer the most damage; however, this does not appear to be favorable and further evaluation was not continued.

SUMMARY OF SOCIO-ECONOMIC ANALYSIS

The projected costs and benefits for the various alternatives are summarized in Table 12 with values presented as average annual costs. Designs 1, 2, and 3 of the Southward Diversion Alternatives involve discharging flood waters to downstream areas for which a monetary estimate is not available. Inconvenience costs are difficult to assess. Flood damage costs are significantly influenced by evacuation and inconvenience costs. The relationship of the existing condition (No Action Alternative) average annual damage estimates to the unit costs for inconvenience is as follows: $41,000.00, $53,000.00, $72,000.00, and $88,000.00 per year for inconvenience costs per residence of $5.00, $10.00, $15.00, and $20.00 per day, respectively. The first design of the Flood Proofing Alternatives (Elevating Septic Tanks) and the Flood Protection Levee Alternative become feasible at unit costs of about $15.00 and $18.00 per day, respectively, while the second and third designs of the Flood Proofing Alternatives (Elevating Septic Tanks and Low-Lying Structures and Regional Wastewater Treatment System) become feasible at costs of about $28.00 and $24.00 per day respectively.

TABLE 12

SUMMARY OF FLOOD DAMAGE MITIGATION ECONOMICS

Alt.	Avg. Annual Costs: Project	Avg. Annual Costs: Downstream Flood Damages	Avg. Annual Benefits: Flood Damage Mitigation	Avg. Annual: Net Benefit
No Action	0	0	0	0
Southward Diversion Design 1	5,340	Not Avail	37,000	+31,660
Southward Diversion Design 2	5,340	Not Avail	47,000	+41,660
Southward Diversion Design 3	6,000	Not Avail	45,000	+39,000
Flood Proofing 1	52,000	0	33,000	-19,000
Flood Proofing 2	76,770	0	40,000	-36,770
Flood Proofing 3	66,300		33,000	-33,300
Flood Protection Levee	60,320	Yes	41,000	-19,320

SUMMARY AND CONCLUSIONS

The purpose of this study was to develop the best water management plan to solve the problem of localized flooding at Lake Lowery. The primary objective was to design an economically feasible solution which will minimize flood damage to the residences of the area without causing unacceptable impacts to wetlands or other environmentally sensitive lands, fish and wildlife, private property and the public, streamflows and/or lake levels, and recreation, navigation, and water quality.

As part of the study the socio-economic, hydrologic, and site-specific characteristics of the study area were qualified and quantified. Eleven design configurations were developed as possible solutions. Each design was then evaluated by means of computer simulation of its hydraulic characteristics and by estimating its socio-economic and environmental impacts.

The three Southward Diversion Alternatives best meet the objectives of the study. In particular, the third design (Closed Conduit with Controlled Northerly Connection) has the best combination of desireable factors. The average annual benefit cost ratio for the third design is (7.5:1). Downstream flood damages to the Peace River Basin were not included in the cost analysis. The average annual benefit/cost ratio may be reduced if these potential damages to the Peace River Basin are to be entirely prevented.

The Northward Diversion Alternatives proved to be inefficient and costly. Drainage modifications to enhance northward flow were determined to be unrealistic given the flat topography, environmental impacts, and the increased potential for downstream flooding.

The Flood Proofing Alternative -- Design 1, and the Flood Protection Levee Alternative proved to be effective flood control designs. However, the costs for implementing these projects are greater than the flood mitigation benefits which would be realized from them. Not until there is a fifty and eighty percent increase (from $10.00 to $15.00 and $10.00 to $18.00 per residence per day) in evacuation costs incurred by the 172 residences located on the north side of Lake Lowery, would the Flood Protection Levee Alternative and the Flood Proofing Alternative -- Design 1, be economically feasible.

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APPENDIX A

NO ACTION ALTERNATIVE

EVAC. COST = $ 30./DAY THRU 14 DAYS AND $10./DAY AFTER
INCONVEN. BEGINS AT DEPTH TO W.T. = 0.50 FT, EVAC AT DEPTH TO W.T. = 0.0 FT
MAX INCONVENIENCE COST $5./DAY
VCVS = 20.0%, PCVAL= 118.00 %, RMVS = 1.0%, RENTAL EVAC. AFTER 7 DAYS

Default House Values
1 SNH = $25000. 2 SNB = $35000. SPL = $30000. DWMH = $12000. SWMH = $6000.

TR	REACH 1		HYDROLOGIC DATA			(STAGE)		
YRS	1- DY	7- DY	14- DY	30-DY	60-DY	120-DY	183-DY	273-DY
2.	130.30	130.24	130.24	130.11	129.96	129.75	129.54	
5.	131.55	131.48	131.42	131.32	131.17	130.92	130.71	
10.	132.27	132.18	132.12	132.00	131.82	131.55	131.33	
25.	133.08	132.97	132.90	132.76	132.54	132.24	132.00	
50.	133.63	133.51	133.44	133.24	133.02	132.70	132.43	
100.	134.15	134.02	133.94	133.75	133.45	133.11	132.82	
200.	134.65	134.50	134.41	134.19	133.85	133.49	133.18	
500.	135.27	135.00	135.00	134.75	134.34	133.95	133.61	
PMF	135.27	135.00	135.00	134.75	134.34	133.95	133.61	

REACH 1 RESIDENTIAL DAMAGES ($1000)

DAMAGE	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	PMF	AAC
STRL	0.	6.	22.	60.	142.	244.	401.	653.	653.	13.
CONT	0.	1.	5.	15.	33.	51.	77.	117.	117.	3.
EVAC	2.	41.	121.	211.	292.	323.	339.	350.	350.	33.
INCON	1.	9.	16.	14.	8.	4.	3.	1.	1.	3.
TOTAL	4.	56.	164.	300.	475.	622.	819.	1121.	1121.	
AAC	1.	5.	10.	14.	8.	5.	4.	3.	2.	

SOUTHWARD DIVERSION ALTERNATIVE -- DESIGN 1

POLK COUNTY PROPOSAL, DIVERSION AT 130.00 FT.

EVAC. COST = $ 30./DAY THRU 14 DAYS AND $10./DAY AFTER
INCONVEN. BEGINS AT DEPTH TO W.T. = 0.50 FT, EVAC AT DEPTH TO W.T. = 0.0 FT
MAX INCONVENIENCE COST $5./DAY
VCVS = 20.0%, PCVAL= 118.00 %, RMVS = 1.0%, RENTAL EVAC. AFTER 7 DAYS

Default House Values
1 SNH = $25000. 2 SNB = $35000. SPL = $30000. DWMH = $12000. SWMH = $6000.

TR	REACH 1		HYDROLOGIC DATA			(STAGE)		
YRS	1- DY	7- DY	14- DY	30-DY	60-DY	120-DY	183-DY	273-DY
2.	129.60	129.55	129.49	129.40	129.28	129.08	129.80	
5.	130.63	130.57	130.51	130.43	130.30	130.04	129.83	
10.	131.24	131.17	131.11	131.03	130.87	130.55	130.30	
25.	131.96	131.87	131.81	131.73	131.50	131.10	130.79	
50.	132.46	132.35	132.31	132.20	131.93	131.46	131.10	
100.	132.94	132.81	132.74	132.66	132.33	131.78	131.37	
200.	132.40	133.26	133.23	133.09	132.70	132.07	131.60	
500.	133.99	133.93	133.81	133.64	133.16	132.42	131.88	
PMF	134.43	134.25	134.24	134.05	133.50	132.67	132.07	

REACH 1 RESIDENTIAL DAMAGES ($1000)

DAMAGE	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	PMF	AAC
STRL	0.	1.	3.	9.	30.	64.	112.	206.	326.	3.
CONT	0.	0.	0.	2.	7.	16.	27.	45.	65.	1.
EVAC	0.	6.	22.	55.	129.	185.	229.	273.	297.	10.
INCON	0.	2.	5.	10.	14.	14.	12.	9.	6.	1.
TOTAL	0.	9.	31.	76.	181.	278.	380.	533.	694.	
AAC	0.	1.	2.	3.	3.	2.	2.	1.	1.	

FLOOD PROTECTION LEVEE ALTERNATIVE

PERIMETER DIKE AND INTERNAL DRAINAGE SYSTEM DESIGNED FOR 100-YEAR FLOOD

EVAC. COST = $ 30./DAY THRU 14 DAYS AND $10./DAY AFTER
INCONVEN. BEGINS AT DEPTH TO W.T. = 0.50 FT, EVAC AT DEPTH TO W.T. = 0.0 FT
MAX INCONVENIENCE COST $5./DAY
VCVS = 20.0%, PCVAL= 118.00 %, RMVS = 1.0%, RENTAL EVAC. AFTER 7 DAYS

Default House Values
1 SNH = $25000. 2 SNB = $35000. SPL = $30000. DWMH = $12000. SWMH = $6000.

TR	REACH 1		HYDROLOGIC DATA			(STAGE)		
YRS	1- DY	7- DY	14- DY	30-DY	60-DY	120-DY	183-DY	273-DY
2.	130.00	130.00	130.00	130.00	130.00	130.00	130.00	
5.	130.00	130.00	130.00	130.00	130.00	130.00	130.00	
10.	130.00	130.00	130.00	130.00	130.00	130.00	130.00	
25.	130.00	130.00	130.00	130.00	130.00	130.00	130.00	
50.	130.00	130.00	130.00	130.00	130.00	130.00	130.00	
100.	130.00	130.00	130.00	130.00	130.00	130.00	130.00	
200.	134.65	134.50	134.41	134.19	133.85	133.49	133.18	
500.	135.27	135.00	135.00	134.75	134.34	133.95	133.61	
PMF	135.27	135.00	135.00	134.75	134.34	133.95	133.61	

REACH 1 RESIDENTIAL DAMAGES ($1000)

DAMAGE	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	PMF	AAC
STRL	0.	0.	0.	0.	0.	0.	401.	653.	653.	4.
CONT	0.	0.	0.	0.	0.	0.	77.	117.	117.	1.
EVAC	2.	2.	2.	2.	2.	2.	339.	350.	350.	4.
INCON	0.	0.	0.	0.	0.	0.	3.	1.	1.	0.
TOTAL	2.	2.	2.	2.	2.	2.	819.	1121.	1121.	
AAC	1.	1.	0.	0.	0.	0.	2.	3.	2.	

SOUTHWARD DIVERSION ALTERNATIVE -- DESIGN 2

DIVERSION AT 130.00 FT, LAKE LOWERY RD CULVERTS PERMANENTLY CLOSED

EVAC. COST = $ 30./DAY THRU 14 DAYS AND $10./DAY AFTER
INCONVEN. BEGINS AT DEPTH TO W.T. = 0.50 FT, EVAC AT DEPTH TO W.T. = 0.0 FT
MAX INCONVENIENCE COST $5./DAY
VCVS = 20.0%, PCVAL= 118.00 %, RMVS = 1.0%, RENTAL EVAC. AFTER 7 DAYS

Default House Values
1 SNH = $25000. 2 SNB = $35000. SPL = $30000. DWMH = $12000. SWMH = $6000.

TR	REACH 1		HYDROLOGIC DATA			(STAGE)		
YRS	1- DY	7- DY	14- DY	30-DY	60-DY	120-DY	183-DY	273-DY
2.	129.03	128.97	128.91	128.79	128.64	128.38	128.19	
5.	130.03	129.95	129.89	129.78	129.63	129.40	129.23	
10.	130.58	130.49	130.42	130.30	130.14	129.93	129.77	
25.	131.16	131.07	131.00	130.86	130.67	130.49	130.34	
50.	131.55	131.46	131.37	131.22	131.00	130.85	130.69	
100.	131.91	131.81	131.73	131.54	131.30	130.17	130.01	
200.	132.24	132.13	132.05	131.84	131.57	131.45	131.30	
500.	132.65	132.52	132.44	132.20	131.88	131.80	131.63	
PMF	132.94	132.80	132.71	132.45	132.10	132.04	131.86	

REACH 1 RESIDENTIAL DAMAGES ($1000)

DAMAGE	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	PMF	AAC
STRL	0.	0.	0.	2.	6.	11.	21.	41.	64.	1.
CONT	0.	0.	0.	0.	2.	3.	5.	10.	16.	0.
EVAC	0.	0.	4.	13.	35.	48.	104.	157.	200.	3.
INCON	0.	0.	2.	3.	8.	7.	14.	17.	15.	1.
TOTAL	0.	1.	7.	18.	50.	67.	144.	225.	295.	
AAC	0.	0.	0.	1.	1.	1.	1.	1.	1.	

SOUTHWARD DIVERSION ALTERNATIVE -- DESIGN 3

DIVERSION AT 130.0 FT, LAKE LOWERY RD CULVERTS GATED

EVAC. COST = $ 30./DAY THRU 14 DAYS AND $10./DAY AFTER
INCONVEN. BEGINS AT DEPTH TO W.T. = 0.50 FT, EVAC AT DEPTH TO W.T. = 0.0 FT
MAX INCONVENIENCE COST $5./DAY
VCVS = 20.0%, PCVAL= 118.00 %, RMVS = 1.0%, RENTAL EVAC. AFTER 7 DAYS

Default House Values
1 SNH = $25000.   2 SNB = $35000.   SPL = $30000.   DWMH = $12000.   SWMH = $6000.

TR	REACH 1		HYDROLOGIC DATA			(STAGE)		
YRS	1- DY	7- DY	14- DY	30-DY	60-DY	120-DY	183-DY	273-DY
2.	129.92	129.85	129.79	129.69	129.56	129.38	129.19	
5.	130.70	130.61	130.53	130.41	130.27	130.10	129.96	
10.	131.09	130.98	130.90	130.76	130.60	130.43	130.32	
25.	131.49	131.37	131.27	131.09	130.91	130.74	130.65	
50.	131.73	131.63	131.49	131.29	131.09	130.93	130.85	
100.	131.95	131.81	131.69	131.46	131.24	131.08	131.01	
200.	132.14	131.99	131.86	131.61	131.37	131.20	131.14	
500.	132.36	132.20	132.06	131.78	131.51	131.34	131.29	
PMF	132.51	132.35	132.19	131.89	131.60	131.42	131.38	

REACH 1   RESIDENTIAL DAMAGES ($1000)

DAMAGE	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	PMF	AAC
STRL	0.	1.	2.	4.	8.	11.	17.	26.	33.	1.
CONT	0.	0.	0.	1.	1.	2.	4.	6.	8.	0.
EVAC	0.	7.	15.	25.	43.	61.	79.	99.	113.	5.
INCON	0.	2.	3.	6.	9.	11.	12.	14.	14.	1.
TOTAL	0.	10.	20.	36.	62.	86.	112.	145.	169.	
AAC	0.	1.	1.	2.	1.	1.	0.	0.	0.	

FLOOD PROOFING ALTERNATIVE -- DESIGN 1

RAISE ST-SAS OR INSTALL MOUND SYSTEMS

EVAC. COST = $ 30./DAY THRU 14 DAYS AND $10./DAY AFTER
INCONVEN. BEGINS AT DEPTH TO W.T. = 0.50 FT, EVAC AT DEPTH TO W.T. = 0.0 FT
MAX INCONVENIENCE COST $5./DAY
VCVS = 20.0%, PCVAL= 118.00 %, RMVS = 1.0%, RENTAL EVAC. AFTER 7 DAYS

Default House Values
1 SNH = $25000. 2 SNB = $35000. SPL = $30000. DWMH = $12000. SWMH = $6000.

HYDROLOGIC DATA (STAGE)

REACH 1

TR YRS	1- DY	7- DY	14- DY	30-DY	60-DY	120-DY	183-DY	273-DY
2.	130.30	130.24	130.20	130.11	129.98	129.75	129.54	
5.	131.55	131.48	131.42	131.32	131.17	130.92	130.71	
10.	132.27	132.18	132.12	132.00	131.82	131.55	131.33	
25.	133.08	132.97	132.90	132.76	132.54	132.24	132.00	
50.	133.63	133.51	133.44	133.24	133.02	132.70	132.43	
100.	134.15	135.02	133.94	133.75	133.45	133.11	132.82	
200.	134.65	134.50	134.41	134.19	133.85	133.49	133.18	
500.	135.27	135.00	135.00	134.75	134.34	133.95	133.61	
PMF	135.27	135.00	135.00	134.75	134.34	133.95	133.61	

REACH 1 RESIDENTIAL DAMAGES ($1000)

DAMAGE	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	PMF	AAC
STRL	0.	6.	22.	60.	142.	244.	401.	653.	653.	13.
CONT	0.	1.	5.	15.	33.	51.	77.	117.	117.	3.
EVAC	0.	2.	6.	12.	36.	66.	107.	185.	350.	3.
INCON	0.	0.	0.	0.	0.	0.	0.	4.	4.	0.
TOTAL	0.	8.	33.	87.	211.	361.	585.	959.	959.	
AAC	0.	1.	2.	4.	3.	3.	2.	2.	2.	

FLOOD PROOFING ALTERNATIVE -- DESIGN 3

RAISE ALL SEPTIC TANKS TO ELEV 134.0 FT, AND ALL RESIDENCE
WITH FIRST FLOOD ELEV BELOW 133.00 TO ELEV 134.0

EVAC. COST = $ 30./DAY THRU 14 DAYS AND $10./DAY AFTER
INCONVEN. BEGINS AT DEPTH TO W.T. = 0.50 FT, EVAC AT DEPTH TO W.T. = 0.0 FT
MAX INCONVENIENCE COST $5./DAY
VCVS = 20.0%, PCVAL= 118.00 %, RMVS = 1.0%, RENTAL EVAC. AFTER 7 DAYS

Default House Values
1 SNH = $25000.   2 SNB = $35000.   SPL = $30000.   DWMH = $12000.   SWMH = $6000.

TR	REACH 1		HYDROLOGIC DATA			(STAGE)		
YRS	1- DY	7- DY	14- DY	30-DY	60-DY	120-DY	183-DY	273-DY
2.	130.30	130.24	130.21	130.11	129.98	129.75	129.54	
5.	131.55	131.48	131.42	131.32	131.17	130.92	130.71	
10.	132.27	132.18	132.12	132.00	131.82	131.55	131.33	
25.	133.08	132.97	132.90	132.76	132.54	132.24	132.00	
50.	133.63	133.51	133.44	133.24	133.02	132.70	132.43	
100.	134.15	135.02	133.94	133.75	133.45	133.11	133.82	
200.	134.65	134.58	134.41	~~134.19~~	133.85	133.49	133.18	
500.	135.27	135.00	135.00	134.75	135.34	133.95	133.61	
PMF	135.27	135.00	135.00	134.75	134.34	133.95	133.61	

REACH 1   RESIDENTIAL DAMAGES ($1000)

DAMAGE	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	PMF	AAC
STRL	0.	0.	3.	27.	110.	216.	372.	619.	619.	8.
CONT	0.	0.	1.	7.	25.	43.	69.	110.	110.	2.
EVAC	0.	0.	0.	0.	11.	66.	150.	258.	258.	2.
INCON	0.	0.	0.	0.	1.	13.	19.	25.	25.	0.
TOTAL	0.	0.	4.	34.	146.	339.	610.	1011.	1011.	
AAC	0.	0.	0.	1.	2.	2.	2.	2.	2.	