Exhibit 1A

ACME IMPROVEMENT DISTRICT

MASTER WATER CONTROL PLAN

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Exhibit 1A

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ACME IMPROVEMENT DISTRICT MASTER WATER CONTROL PLAN

I. Background Data

A. Description and Location

The Acme Improvement District comprises more than 18,000 acres of land in central Palm Beach County, Florida bounded on the north by the C-51 Canal of the South Florida Water Management District (SFWMD) on the east by the north/south section line lying approximately one mile west of U. S. 441 (State Road 7), on the west by the L-40 Dike and Canal of the SFWMD and on the south by the extension of Homeland Road. (See location map, Figure 1) . Acme Improvement District contains the Wellington PUD, several other major Planned Unit Developments and a large agricultural area, all of which are rapidly approaching complete development.

The vegetation in the District is primarily Florida native pine palmetto flatwoods interspersed with remnant cypress strands and ponds and southwesterly into the edge of the historic Florida Everglades, muck soils and sawgrass, myrtle and willow communities are prevalent. Elevations range from in excess of 17' near the north and east property boundaries to approximately elevation 15' on the south and west property boundary.

Exhibit 1A

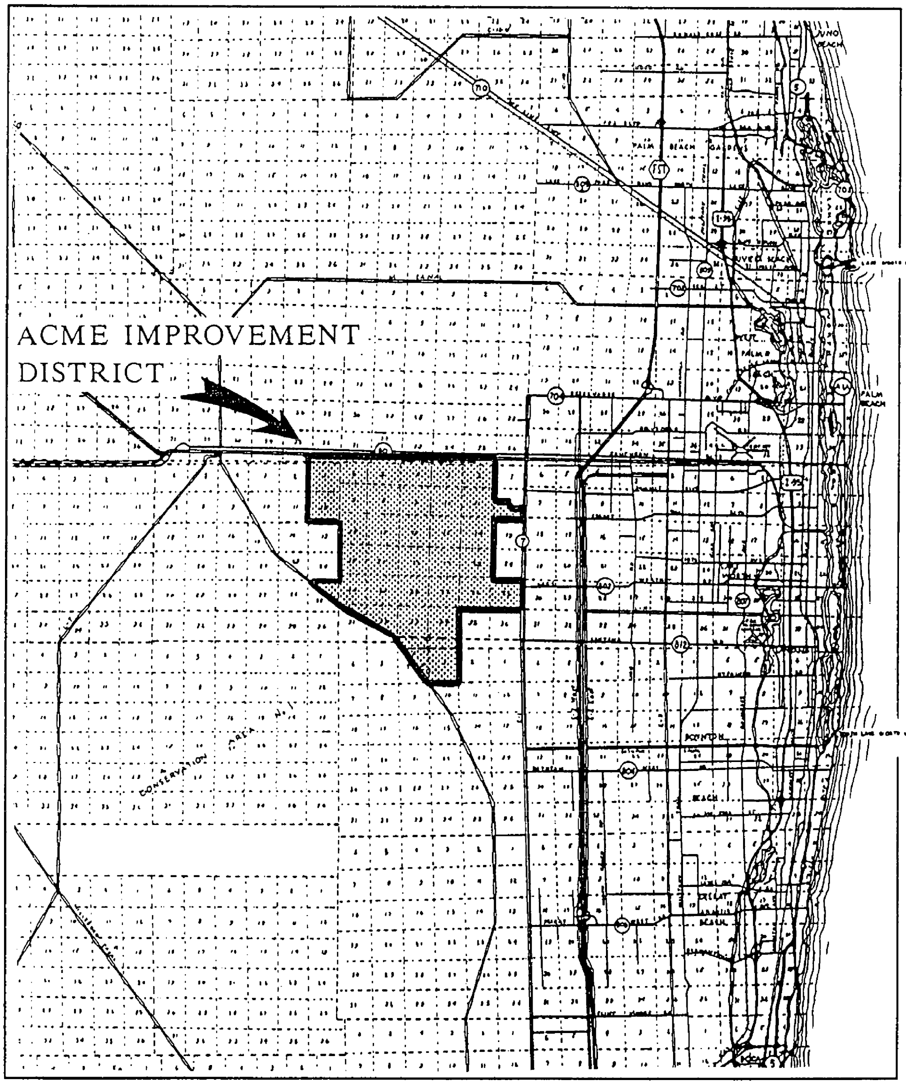

Figure 1. Location Map.

II. History of Acme Improvement District

The Acme Improvement District was created originally as the Acme Drainage District by an act of the legislature in June of 1953. At that time the area was 100% agricultural and under essentially a single ownership. Improvements were made following the creation of the District by the installation of levees, canals and pumping stations to "reclaim" the land for agricultural purposes. Several revisions to the District's enabling legislation were completed expanding the District's boundaries and adding to its powers. In about 1972, the owners of the lands in the District entered into a contract with a land development organization and created Wellington PUD, Unit of Development No. 1 and Unit of Development No. 2. This began the development of what is now known as the Wellington Community.

III. Water Control Permits

Acme Improvement District was developed prior to the permitting authority of the SFWMD, however, as plans for residential development in the District were advanced, surface water management permits were applied for and received for the District's system. Permits to operate existing facilities were issued on several occasions by SFWMD and were finally included in a comprehensive permit issued in February 1978. Minor modifications to this permit were made since that time to incorporate new Planned Unit Developments outside of the Wellington PUD. Presently all operating facilities of Acme Improvement District are permitted by SFWMD.

present time all operating facilities of Acme Improvement District are permitted by SFWMD.

IV. Description of Water Management System

The water management facilities of the Acme Improvement District consists of more than 100 miles of interconnecting lakes and canals with associated culverts for roadway crossings, water control structures for water level regulation, dikes around the perimeter of the District to prohibit outside water from flowing into the District's facilities and pumping stations for the purpose of removing surplus stormwater from system during heavy rainfall periods. Figure 2 is a map showing the location and partial description of the District's facilities. Table 1 gives a description of all culverts and water control structures owned and operated by the District. Table 2 is a summary of the pumping equipment located in each of the District's four major pumping stations. These stations are capable of removing an average of approximately 1.2 inches per day of runoff from the District with increases in that removal rate on a standby basis subject to permission of the SFWMD to operate the pumping. Three large lakes are included in the system which were excavated, primarily for fill material, which provide stormwater storage and detention as a part of the overall system.

V. Hydraulic Model of the Water Management System

In order to simulate the hydraulic response of the system to storm events UNET was applied to the District. It is a one-dimensional, unsteady flow model that simulates flow through a full network of open channels. It was selected based on its ability to handle highly interconnected canal networks (as has the District) and its ability to perform unsteady flow (ie. storm) routing with networks having numerous water control structures and storage areas (lakes).

Data requirements include channel and water control structure geometry (canal cross-sections, culvert size, location, and inverts, pump station capacity and location, and storage area size and location), lake geometry, runoff hydrographs (generated by a hydrologic model), and boundary conditions (initial lake elevations and pump operating rules). Output includes stage and discharge hydrographs at pumping stations and in canals and lakes and water surface profiles showing energy losses in canals and through culverts. The UNET schematic for the District is shown in Figure 3. It shows upstream and downstream boundaries, modeled canal reaches and connections, and lakes (storage areas). One hundred thirty-eight cross-sections of the District canals were surveyed in June 1993 (see Figure 4). A typical cross-section was comprised of seven to eight station-elevation data pairs extended from top-of bank to top-of bank or road crown if the canal was parallel to a roadway. Elevations and canal depths were measured relative to the water surface. Water surface elevations were read from the staff gages at pumping stations to tie the elevations to

NGVD. The cross-section information was coded into the model along with estimates of canal and overbank roughness. Culvert geometry (size, shape, length, and number) was also coded into the model. Information was taken from the culvert exhibit and from an updated inventory taken in December 1992. The surface areas of the three lakes in Basin A were estimated from aerial photographs and coded into the model.

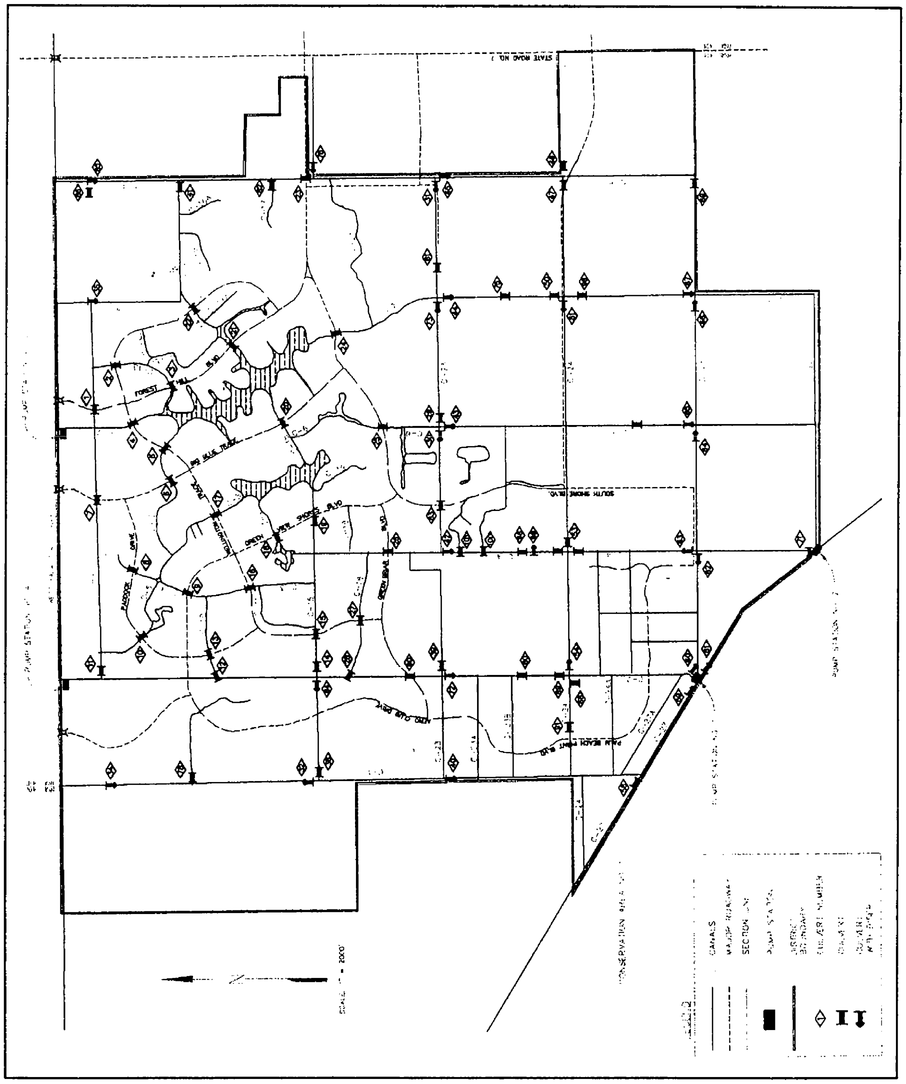

Figure 2. Facilities Map.

Table 1. Culvert Schedule.

No.	Location		Trib. to Pump	Qty	Size (in.)	Type	Length (ft.)	Invert Elevation(s) (ft. NGVD)
	Canal	Street						
1	C-9	Forest Hill Blvd.	3	1	54	CMP	200	W-9.12; E-8.99
2	C-13	Wellington Trace	3	1	42	CMP	114	7.9
3	C-13	Forest Hill Blvd.	3	1	42	CMPA	150	W-8.80; E-8.20
4	C-7	Wellington Trace	3	2	72	CMP	124	NW-5.52; NE-5.57; SW-5.42; SE-5.72
5	C-12	Wellington Trace	3	1	48	CMP	108	8.89
6	C-12	Big Blue Trace		1	48	CMP	108	W-6.73; E-7.62
7	C-9	Big Blue Trace	3	1	36x22	CMPA	150	W-12.30; E-11.5
8	C-3	Paddock Drive	4	1	72	CMP	120	N-6.88; S-7.85
9	C-3	Greenview Shores	4	1	72	CMPA	120	7.0
10	C-5	Paddock Drive	4	2	60	CMP	122	NW-5.97; SW-6.83
11	C-9	Ousley Road	4	1	84	CMP	90	E-0.99; W-1.6
12	C-10	Ousley Road	4	1	72	CMP	90	5.1
13	C-10	Paddock Drive	4	1	72	CMP	126	6.4
14	C-15	Ousley Road	4	1	72	CMP	60	5.0
15	C-15	Wellington Trace	4	1	48	CMP	116	8.0
16	C-3	Wellington Trace	4	1	54	CMP	118	7.1
17	C-5	Wellington Trace	4	2	60	CMP	108	SE-6.34; SW-6.33
18	C-4	Greenview Shores	4	1	48	CMP	100	9.8
19	C-15	Greenview Shores	4	1	65x40	CMPA	114	W-8.28; E-7.85
20	C-15	Big Blue Trace		1	48	CMP	104	7.9
21	C-14	Forest Hill Blvd.	3	1	8x10	CONC	120	8
22	C-14	Wellington Trace	3	2	58x36	CMPA	120	8.3
23	C-8	Forest Hill Blvd.	3	2	54	CMPA	150	8.5
24	C-7	South Shore Blvd.	3	1	72	CMP	260	N-6.0; S-5.5
25	C-6	South Shore Blvd.	3	1	65x40	CMPA	90	7.9
26	C-4	Greenbriar Blvd.	4	1	48	CMP	104	7.9

No.	Location		Trib. to Pump	Qty	Size (in.)	Type	Length (ft.)	Invert Elevation(s) (ft. NGVD)
	Canal	Street						
27	C-18	Wellington Trace	4	1	72	CMP	120	5.9
28	C-18	Ousley Road	4	1	84	CMP	90	4.99
29	C-15	Gallaway Trail		1	72	RCP	110	5.5
30	C-23	C-6	4	1	72	RCP	115	W-5.2; E-5.7
31	C-23	Fairlane Farms Rd.	3	1	54	CMP	110	8.0
32	C-2A	Palm Bch Pt Blvd		1	30	CMP	120	8.1
33	C-11	Drafthorse		1	48	CMP	126	7.6
34	C-1	0.2 mi. S C-51		1	123x84	CMPA	40	9.5
35	C-14	0.2 mi. S C-51		2	60x36	CMPA	48	7.0
36	C-11	Drafthorse		1	48	CMP	126	7.7
37	C-23	Acme facility		1	48	CMP	58	W-8.8; E-8.3
38	C-15	C-1		1	36	CMP	20	10
39	C-23	C-2	4	1	54	CMP	95	7.0
40	C-1	C-23	1	1	72	CMP	45	5.8
41	C-28	C-8	3	1	48	CMPA	30	9.0
42	C-4	C-23	1	1	60	CMP	80	8.0
43	C-6	C-23	1	1	48	CMP	65	11.0
44	C-7	C-23	2	1	48	CMP	65	9.0
45	C-8	C-23	2	1	54	CMP	74	9.5
46	C-24	C-8	2	1	36	CMP	50	12
47	C-24	C-8	2	1	48	CMP	30	9.0
48	C-23	100' E of C-4		1	72	CMP	170	3.0
49	C-9	Squire Drive		1	96	RCP	120	5.0
50	C-7	C-24	2	1	48	CMP	40	8.1
51	C-24	C-7	2	1	48	CMP	40	8.6
52	C-24	South Shore Blvd.	1	1	48	CMP	65	8.0
53	C-24	C-4	1	1	48	CMP	40	7.5

No.	Location		Trib. to Pump	Qty	Size (in.)	Type	Length (ft.)	Invert Elevation(s) (ft. NGVD)
	Canal	Street						
54	C-24	C-2	1	1	48	CMP	40	8.0
55	C-9	Horseshoe Tr W		1	96	RCP	125	4.6
56	C-9	Horseshoe Trace E		1	60	RCP	110	6.2
57	C-9	Access to PS #3		1	48	CMP	50	8.0
58	C-27	C-1	1	1	36	CMP	30	9.0
59	C-27	C-2	1	1	72	CMP	30	7.0
60	C-25	C-2	1	1	60	CMP	60	8.4
61	C-27	C-25	1	1	48	CMP	40	8.0
62	C-25	C-4	1	1	72	CMP	52	7.5
63	C-4	C-25	1	1	72	CMP	53	6.4
64	C-25	C-6	2	1	72	CMP	40	8.0
65	C-6	C-25	2	1	72	CMP	70	N-6.0; S-6.5
66	C-25	C-7	2	1	60	CMP	40	9.5
67	C-7	C-25	2	1	72	CMP	70	6.5
68	C-2B	C-25		1	60	CMP	120	6.5
69	C-25	C-8	2	1	48	CMP	35	10.0
70	C-8	C-25	2	1	65x40	CMP	125	8.5
71	C-27	C-4	2	1	48	CMP	50	9.0
72	C-2	C-23		1	48	CMP	40	8.3
73	C-23	C-7		1	36	CMP	50	10
74	C-25	Homeland Road		1	85x54	CMP	150	7.5
75	C-7	0.5 mi. S of C-23		1	48	CMP	40	S-9.3; N-8.4
76	C-7	Polo Golf Course		1	60	CMP	40	4.5
78	C-27A	Sun Glades Trail		2	48	CMP	85	7.3
79	C-23	C-2		1	30	CMP	55	7.5
80	C-2	0.4 mi. S C-23		1	48	CMP	60	
81	C-23	0.2 mi. E C-7		1	60	CMP	60	8.0

No.	Location		Trib. to Pump	Qty	Size (in.)	Type	Length (ft.)	Invert Elevation(s) (ft. NGVD)
	Canal	Street						
82	C-17	C-8		1	48	CMP	51	10
83	C-23	South Shore Blvd		1	72	RCP	170	5.5
84	C-15	At Canal C-2		1	48	CMP	30	8.0
85	C-8	200' S Acme Rd.		1	84	CMP	70	5.0
86	C-2B	Palm Bch Pt Blvd		1	60	CMP	105	7.0
87	C-23B	C-2		1	48	CMP	100	8.0
88	C-24C	Palm Bch Pt Blvd		1	36	CMP	110	8.0
89	C-24D	Stables Way		1	36	CMP	96	9.0
90	C-2	Greenbriar		2	72	CMP	180	6.8
91	C-24	Palm Bch Pt Blvd		2	60	CMP	110	6.0
92	C-18	Appaloosa Trail E		1	72	CMP	106	6.0
93	C-18	Appaloosa Trail W		1	72	CMP	106	5.2
94	C-24B	Palm Bch Pt Blvd		1	36	CMP	96	8.0
95	C-25	E C-2B		1	60	CMP	60	7.9
96	C-2A	C-25		1	36	CMP	60	9.5
97	C-4	C-24		1	72	CMP	60	5.5
98	C-2	2.0MG Tank		2	60	CMP	40	6.0
99	C-2	Greenview Shores		2	84	RCP	140	5.5
100	C-23	Palm Bch Pt Blvd		1	48	CMP	110	7.0
101	C-23A	Palm Bch Pt Blvd		1	30	CMP	120	8.3
102	C-23B	Palm Bch Pt Blvd		1	30	CMP	110	8.0
104	C-24A	Palm Bch Pt Blvd		1	30	CMP	110	8.0
105	C-24C	C-2		1	54	CMP	93	6.5
106	C-24C	South Road		1	48	CMP	115	7.0
107	C-24B	South Road		1	48	CMP	115	7.0
108	C-24	South Road		1	54	CMP	45	5.5
109	C-25	400' E C-4		1	72	CMP	60	7.5

No.	Location		Trib. to Pump	Qty	Size (in.)	Type	Length (ft.)	Invert Elevation(s) (ft. NGVD)
	Canal	Street						
110	C-25	South Shore Blvd		1	72	CMP	47	6.5
111	C-6	Whitebirch		1	54	CMP	70	6.3
112	C-23C	C-4		1	48	CMP	72	7.0
113	C-23C	Quarter Horse Trail		1	36	RCP	118	7.5
114	C-23B	C-4		1	54	CMP	60	7.0
115	C-4	0.25 miles N C-24		1	72 x 54	CMP	60	6.0
116	C-23B	South Shore Blvd		1	72	RCP	167	6.2
117	C-23B	C-6		1	48	CMP	109	8.0
118	C-23A	500' E Santa Barbara Dr.		1	42	RCP	140	8.0
119	C-23A	Santa Barbara Dr		1	42	RCP	97	8.0
120	C-6	Indianmound Road		1	42	CMP	60	6.5
121	C-7	400' S C-24		1	48	CMP	40	8.3
122	C-7	W.W.T.P. Nursery		1	48	CMP	50	S-7.5; N-8.4
123	C-23	Horse Trail Xing		1	48	CMP	60	7.0
124	C-24	200' W C-8		1	54	CMP	30	9.5
125	C-15	Greenview Cv GC		1	54	CMP	75	7.5
126	C-15	Greenview Cv GC		1	54	CMP	80	6.5
127	C-6	Polo Club Road		1	60	CMP	114	5.7
128	C-7	0.2 miles N C-23		1	66	CMP	60	6.7
129	C-7	Polo Club Road		1	76 x 43	RCP	100	6.5
130	C-7	Muir Circle		1	60	CMP	115	6.0
131	C-21	Fairlane Farms Rd		1	54	CMP	110	8.0
132	C-21	50' W C-8		1	48	CMP	60	W-8.5; E-7.4
133	C-21	Birkdale Drive		1	72 x 44	CMP	110	6.0
135	C-12	Azure Drive		1	54	RCP	110	6.0
136	C-12	Exotica Lane		1	54	RCP	110	6.5

No.	Location		Trib. to Pump	Qty	Size (in.)	Type	Length (ft.)	Invert Elevation(s) (ft. NGVD)
	Canal	Street						
137	C-5	Azure Drive		1	72	RCP	116	4.5
138	C-5	Horseshoe Trace		1	84	RCP	118	4.0
139	C-17	Polo Golf Course		1	54	CMP	150	7.0
140	C-10	Turf Lane		1	72	CMP	114	5.7
141	C-18	Aero Club Dr.		1	72	RCP	140	6.0
142	C-1B	Cedar Bluff Pl.		1	72	RCP	110	6.0
143	C-23	S Shore Blvd.		1	72	RCP	80	5.5
144	C-24	Mida Farms		1	48	CMP	50	9.0
145		Ret. Area at C-4		1	36	CMP	70	9.0

Table 2. Pump Station Data.

Pump Station	Basin	Capacity (gpm)	Discharge to	"On" elevation (ft. NGVD)	"Off" elevation (ft. NGVD)
1	B	100,000 75,000 (standby)	WCA-1 WCA-1	14.0	13.0
2	B	120,000 50,000 (irrigation)	WCA-1 ---	14.0	13.0
3	A	60,000 25,000 (irrigation)	C-51 ---	12.0 wet season	11.0 12.0
4	A	60,000 60,000 (standby) 25,000 (irrigation)	C-51 C-51 ---	12.0 wet season 13.0 dry season	11.0 12.0

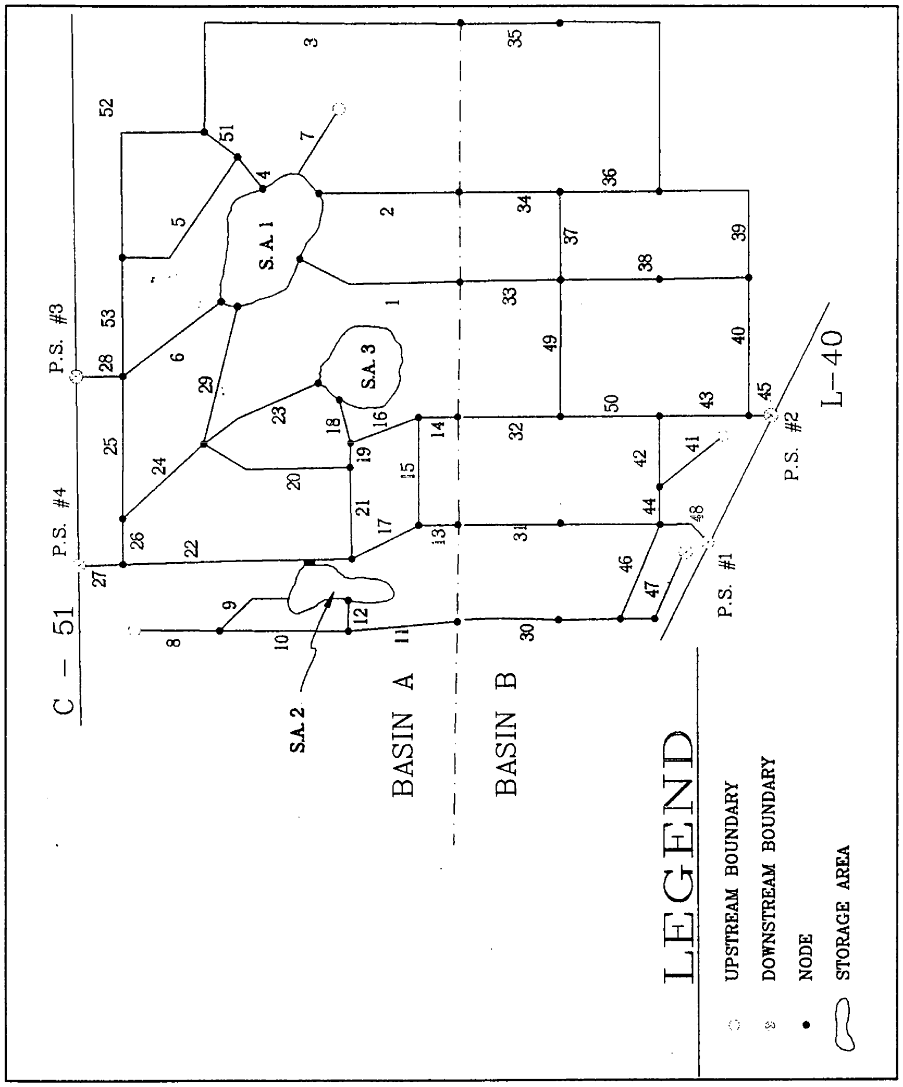

Figure 3. UNET Schematic.

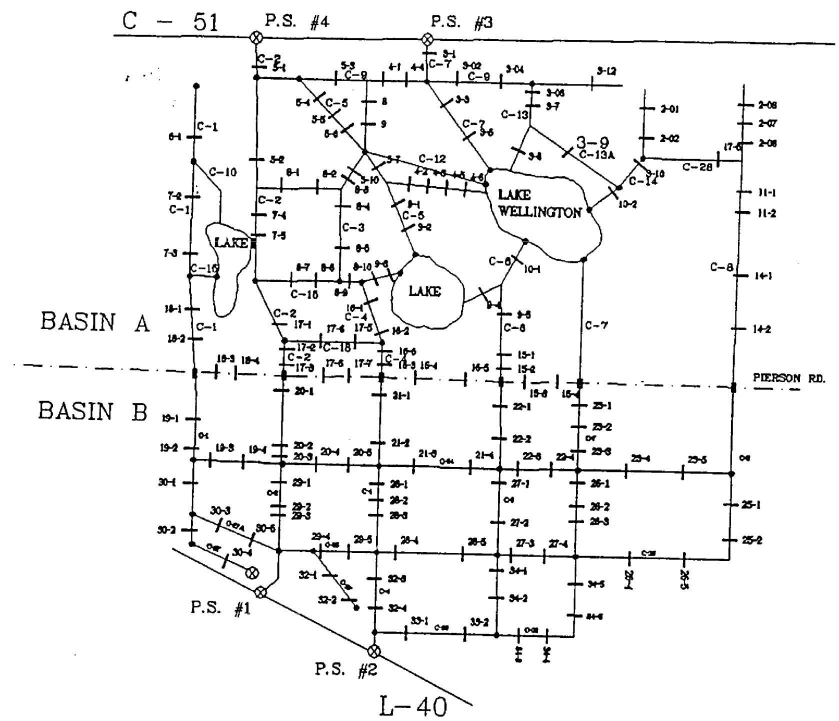

Figure 4. Location of Surveyed Cross-sections.

Runoff hydrographs for each of the subbasins within the District were generated using the HEC-1 computer model. A set of hydrographs was generated for three hypothetical storms: a storm having return period of 10 years and a duration of one day (10-year, 1-day storm), the 25-year, 3-day storm, and the 100-year, 3-day storm. The rainfall depths for these storms are shown in Table 3. The SFWMD rainfall distribution was used. Hydrologic properties (SCS curve number and kinematic wave routing parameters) were estimated for each subbasin from aerial photographs. In some instances (Wycliffe and Country Place), routed discharge hydrographs from surface water management permits were used. The hydrographs were input to UNET via DSS.

Table 3. Hypothetical Storm Rainfall Depths.

Hypothetical Storm (Return Period and Duration)	Rainfall Depth (Inches)
10-year, 1-day	8.0
25-year, 3-day	10.0
100-year, 3-day	12.5

Boundary conditions, initial conditions, and job control parameters for the simulations were assembled in the boundary conditions file. The file specifies computational time step, maximum number of iterations to solve the system of equations at each time step, upstream flow hydrographs, and instructs UNET where the inflow hydrographs are located. The file also describes how the pumps are to be operated, specifies initial stages in the lakes, and directs output from UNET to DSS. An operation schedule was assigned to each of the four major pumping stations (see Table 4). The pumping rate is a function of upstream stage only. During the rising limb of the hydrograph, the pumping rate increases incrementally at each "pump on" stage. Likewise during the falling limb of the hydrograph, the pumping rate decreases incrementally at each "pump off" stage. This gradual approach to reaching full pump capacity is necessary to avoid artificial stage oscillations induced by intermittent pumping at full capacity.

Table 4. Pump Operation Schedules.

Capacity (Percent)	Pumping Station #1		Pumping Station #2	
	On	Off	On	Off
0	11.0	11.0	11.0	11.0
20	11.2	11.1	11.2	11.1
40	11.4	11.3	11.4	11.3
60	11.6	11.5	11.6	11.5
80	11.8	11.7	11.8	11.7
100	12.0	11.9	12.0	11.9

Capacity (Percent)	Pumping Station #3		Pumping Station #4	
	On	Off	On	Off
0	11.0	11.0	11.0	11.0
25	11.3	11.1	11.3	11.1
50	11.6	11.5	11.6	11.5
75	11.9	11.8	11.9	11.8
100	12.1	12.0	12.1	12.0

In order to allow UNET to become numerically stable under steady-state (constant flow) conditions, the canal network was simplified by not including many secondary canals. The first task was to achieve near steady-state conditions without the model becoming numerically unstable. This is a tedious task which requires careful selection of computational time step and initial flow magnitudes and direction. Once steady-state was achieved, subsequent simulations of the hypothetical storm events was initiated by a "hot-start" of the model. A time step of 15 minutes was used. An initial stage of 11.0 NGVD was used.

Stage and discharge hydrographs at pumping stations 1-4 are shown in figures 5 through 8, respectively. Operations at each of the pumping stations are summarized in Table 5.

Table 5. Pump Operations During 10-Year, 1-Day Hypothetical Storm.

	Pump Station 1	Pump Station 2	Pump Station 3	Pump Station 4
Pumping at capacity	2 days	2 days	5+ days	5+ days
Peak stage (ft. NGVD)	14.6	14.7	17.4	15.9
Capacity (cfs)	330	380	130	130

Maximum water surface profiles of key canal reaches are shown in figures 9 through 15. The profiles indicate there are some canal reaches in which excessive head loss occurs due to inadequately sized culverts. Table 6 lists these culverts along with the head loss and associated discharge.

Modeling results also indicate that high elevations occur in the northeast corner of the District as a result of a lack of connection between C-9 Canal of C-14 Canal in the northeast corner of Section 3. The connection of these two canals will result in a more direct flow path from the Section 11 portion of the District to Pump Station No. 3 and result in lower flood stages in that area. Also, an area of difficulty during severe rainfall events in the South Shore 2-A area near C-17 Canal in Section 11. The District has in the past used a temporary pump in this area on severe rainfall occurrences to lower flood stages. It is proposed that a permanent electric lift pump with an automatic control be installed to improve the response time and operating efficiency of this system.

The model also demonstrates that there is a significant transfer of flood water from Basin A to Basin B in severe rainfall events. The six culverts under Pierson Road in Canals C-1 through C-8 now contain stop-logs which must be manually removed to obtain full flow through each structure. It is recommended that vertical lift gates be placed in the existing risers enabling District personnel to quickly operate the structures and commence flow from Basin A to Basin B. It is also recommended that the proposed gates in two culverts in C-4 and C-6 be equipped with electric drive motors and remote operating systems to be able to open these gates from the command center in order to increase flood flows instantaneously.

Figure 16 shows the peak stages resulting from the hypothetical 10-year 1-day storm under existing conditions and with the proposed improvements.

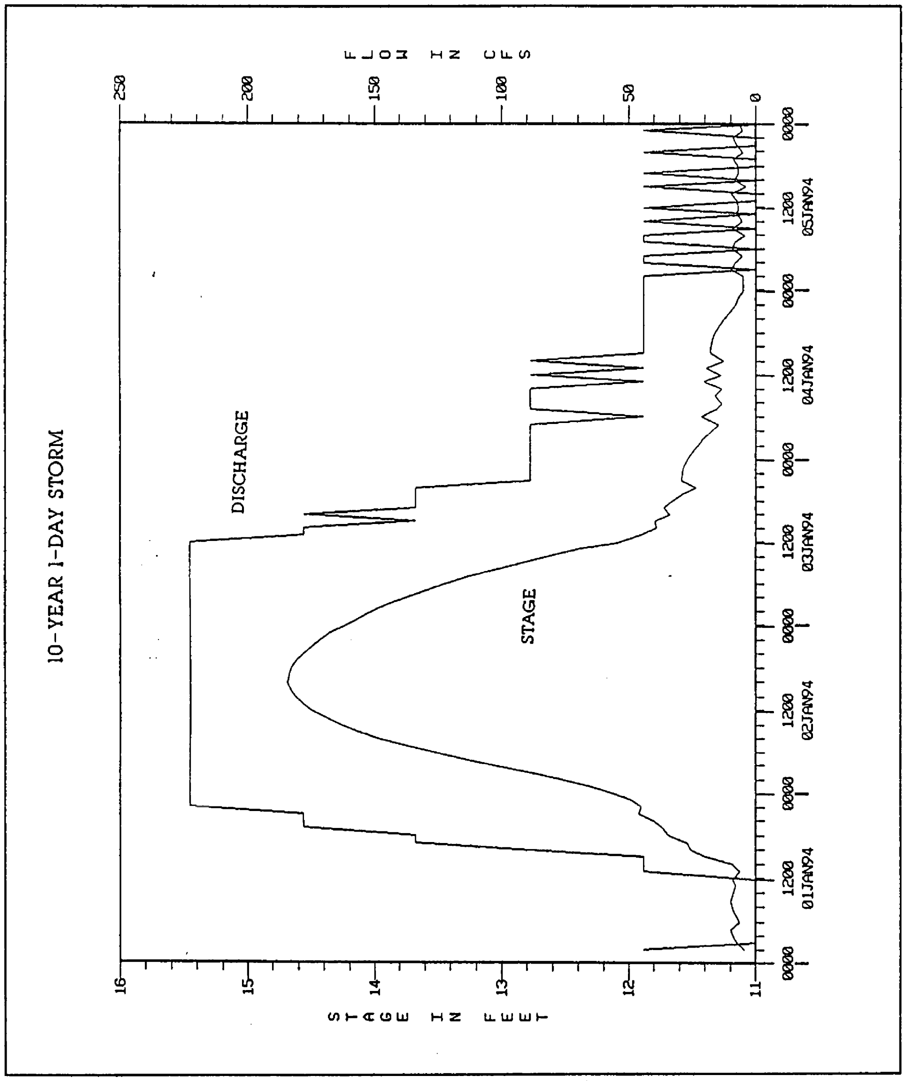

Figure 5. Stage and Discharge Hydrograph, Pumping Station 1.

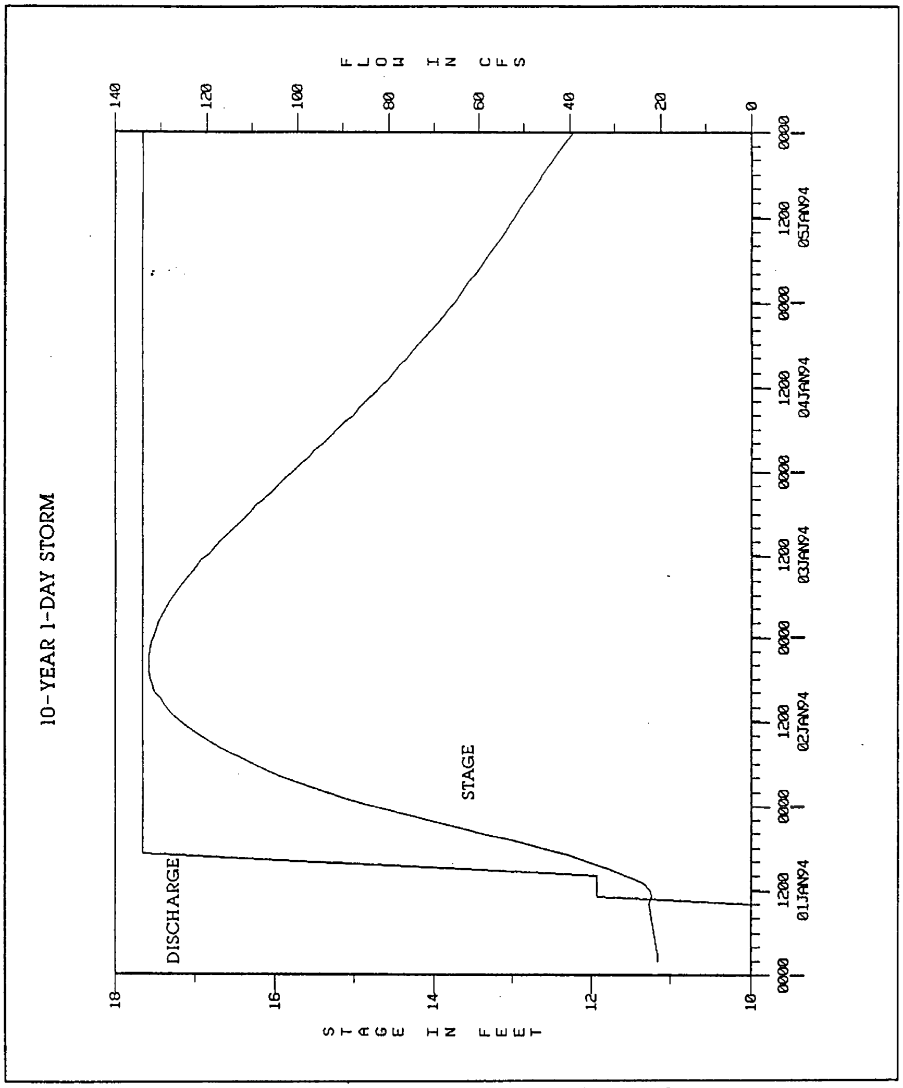

Figure 7. Stage and Discharge Hydrograph, Pumping Station 3.

Exhibit 1A

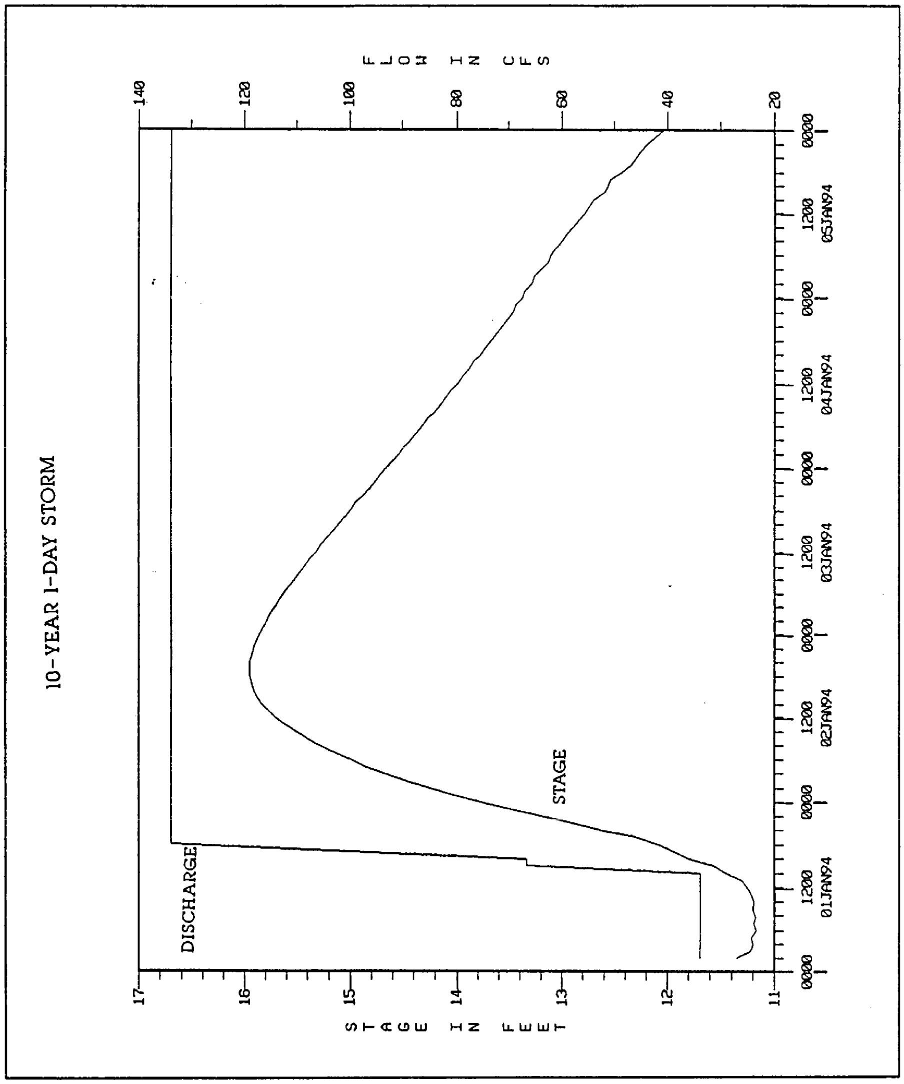

Figure 8. Stage and Discharge Hydrograph, Pumping Station 4.

Exhibit 1A

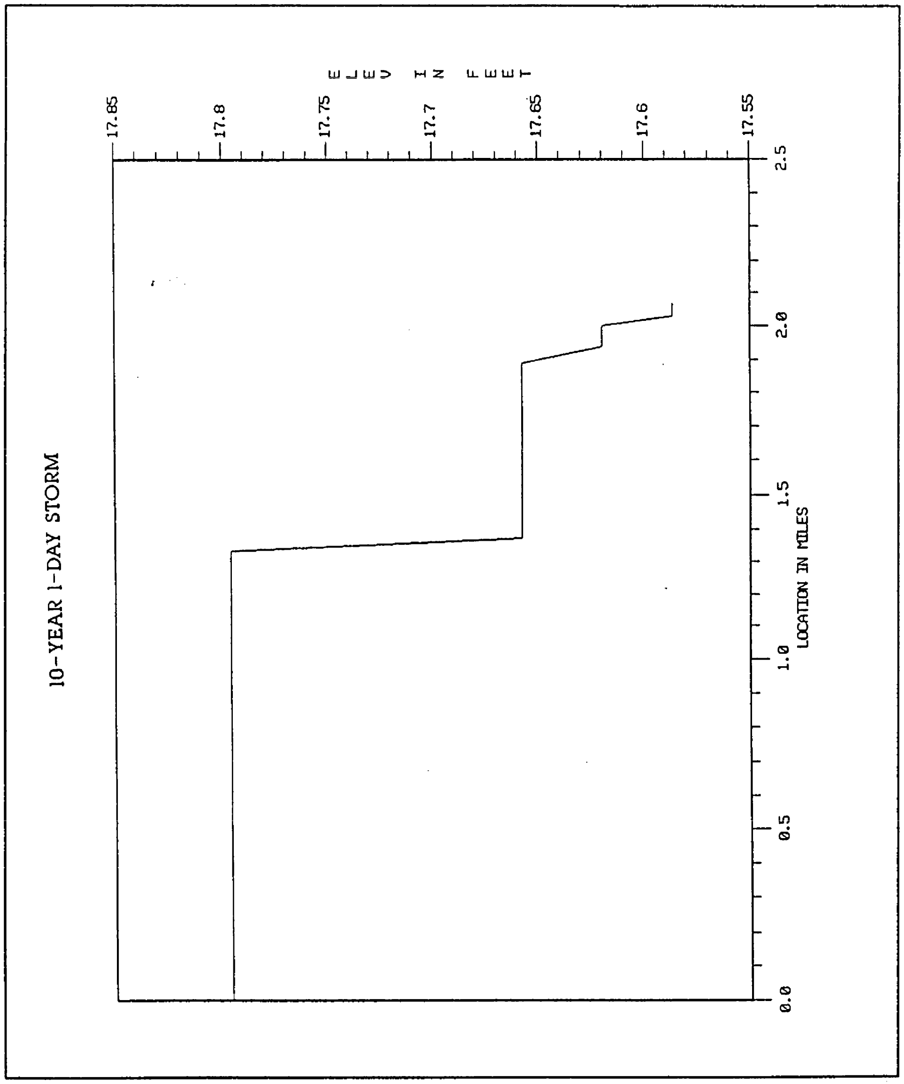

Figure 9. Water Surface Profile, C-9, C-13, & C-13A.

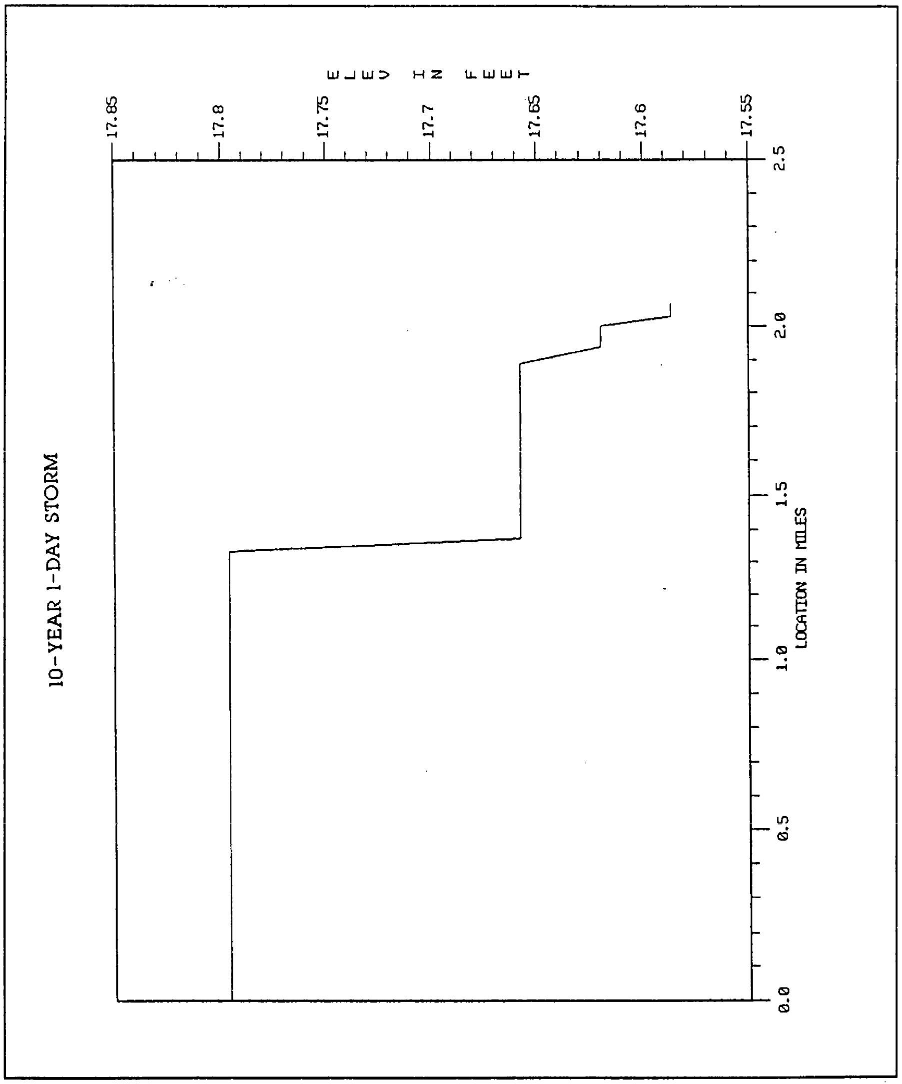

Figure 9. Water Surface Profile, C-9, C-13, & C-13A.

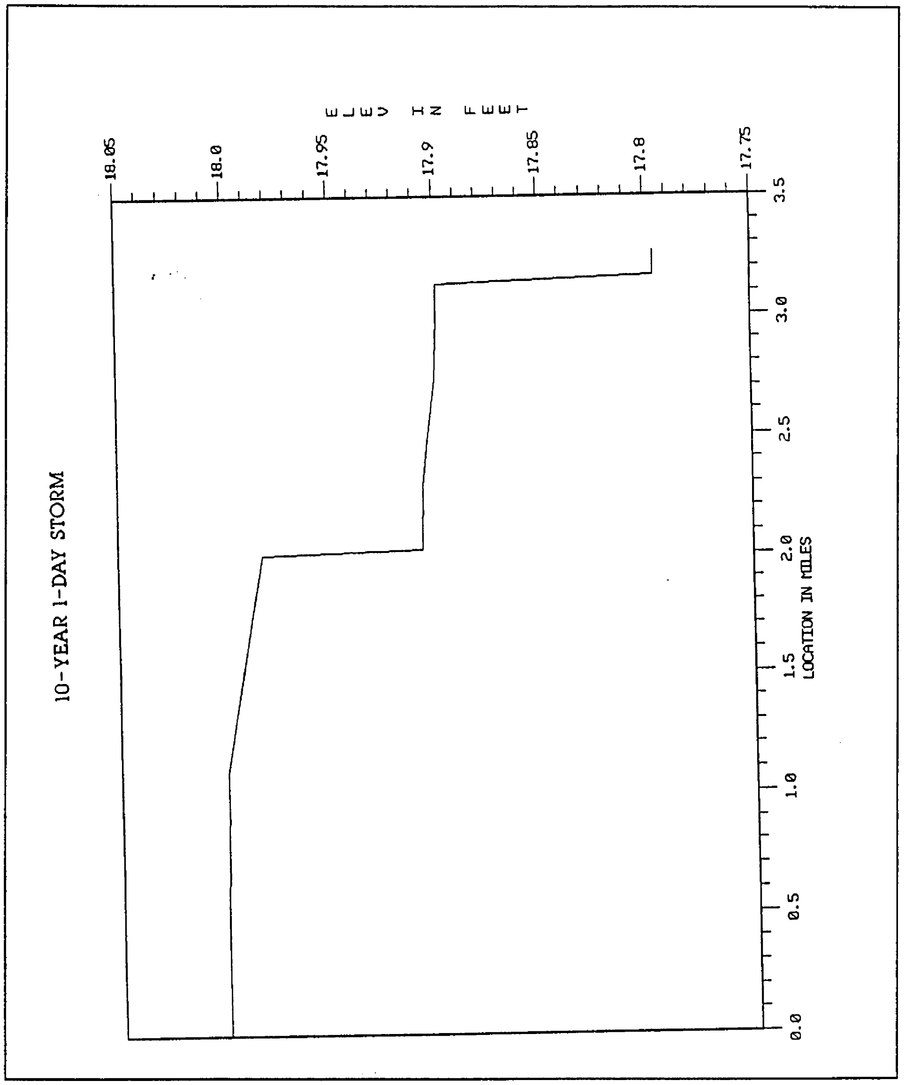

Figure 10. Water Surface Profile, C-28 & C-8.

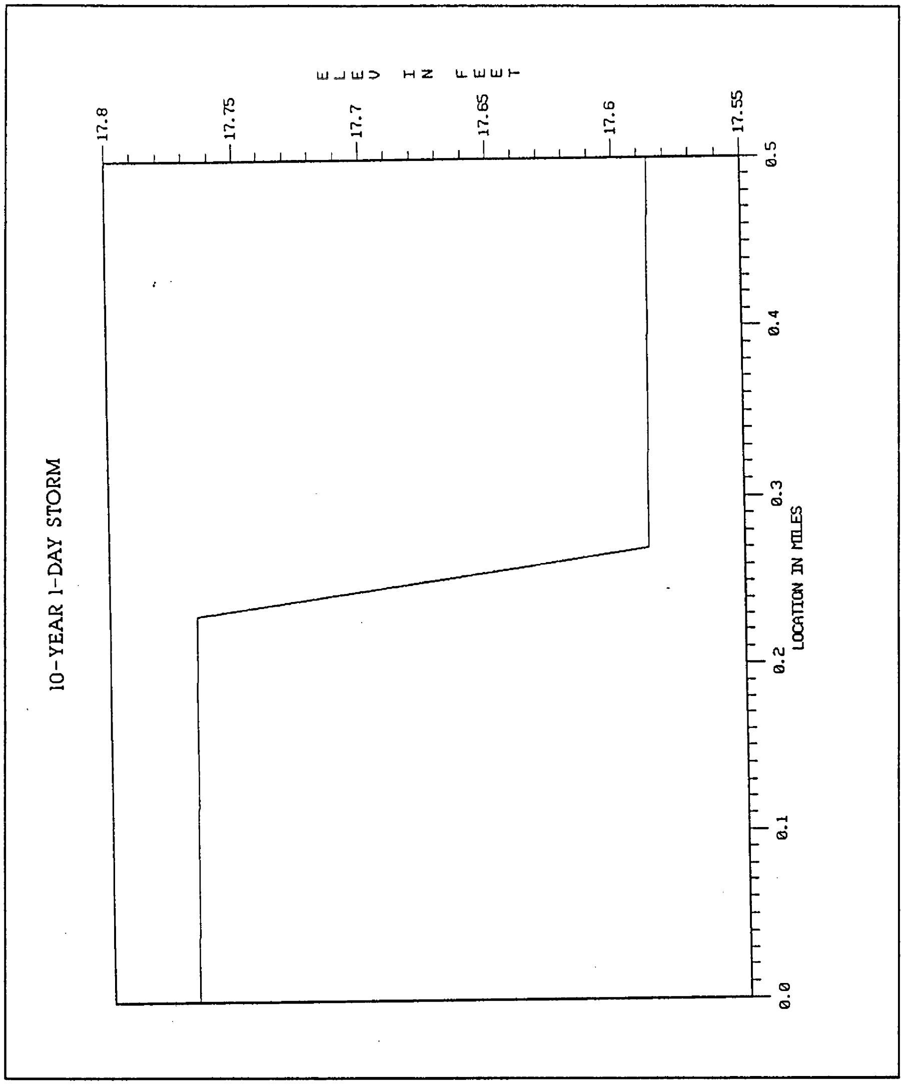

Figure 11. Water Surface Profile C-7 Basin A

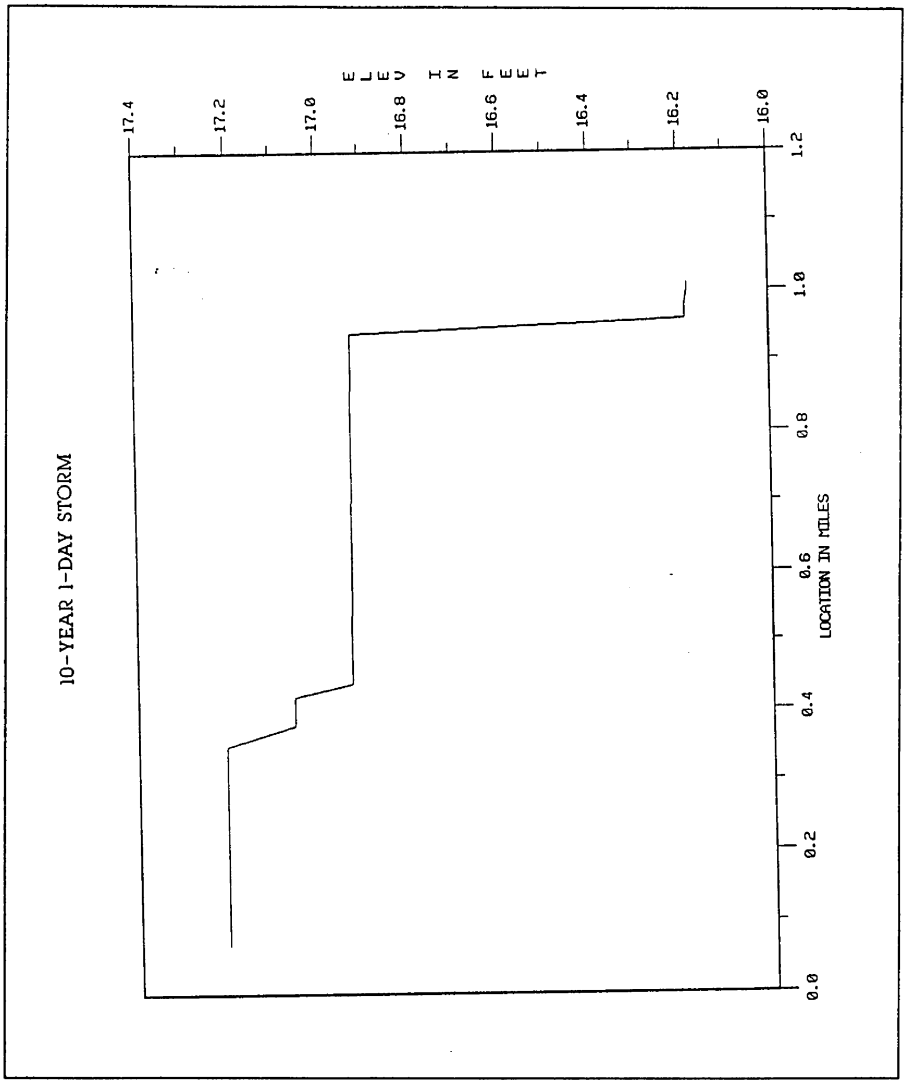

Figure 12. Water Surface Profile C-7 Between C-23 & C-24.

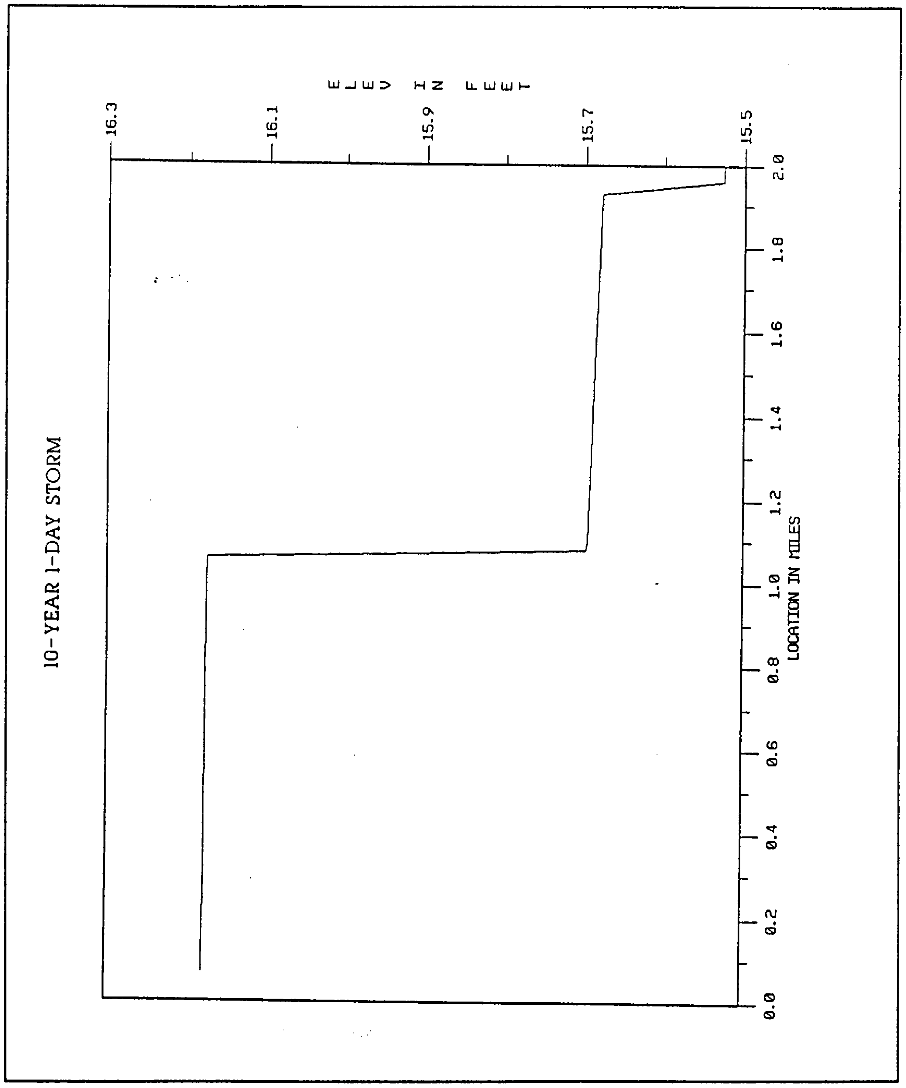

Figure 13. Water Surface Profile C-7 Between C24 & C-25.

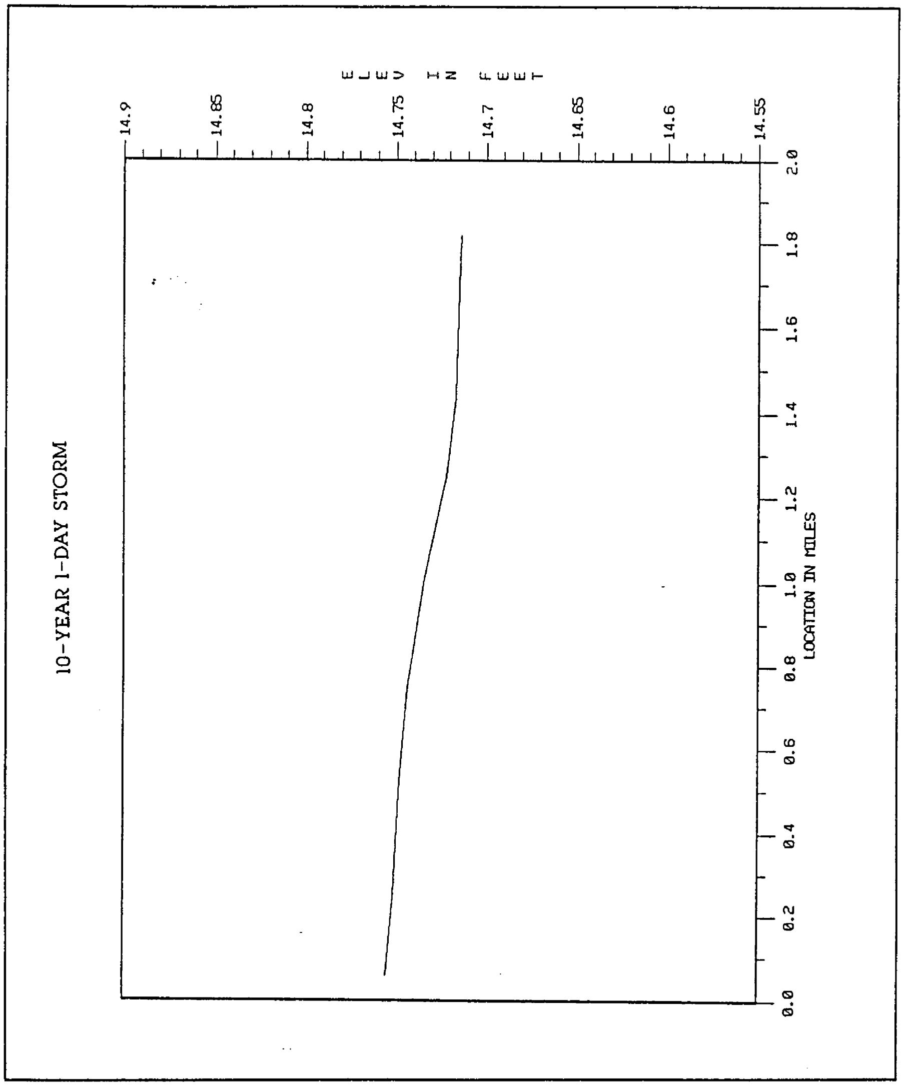

Figure 14. Water Surface Profile C-2 Basin B.

Exhibit 1A

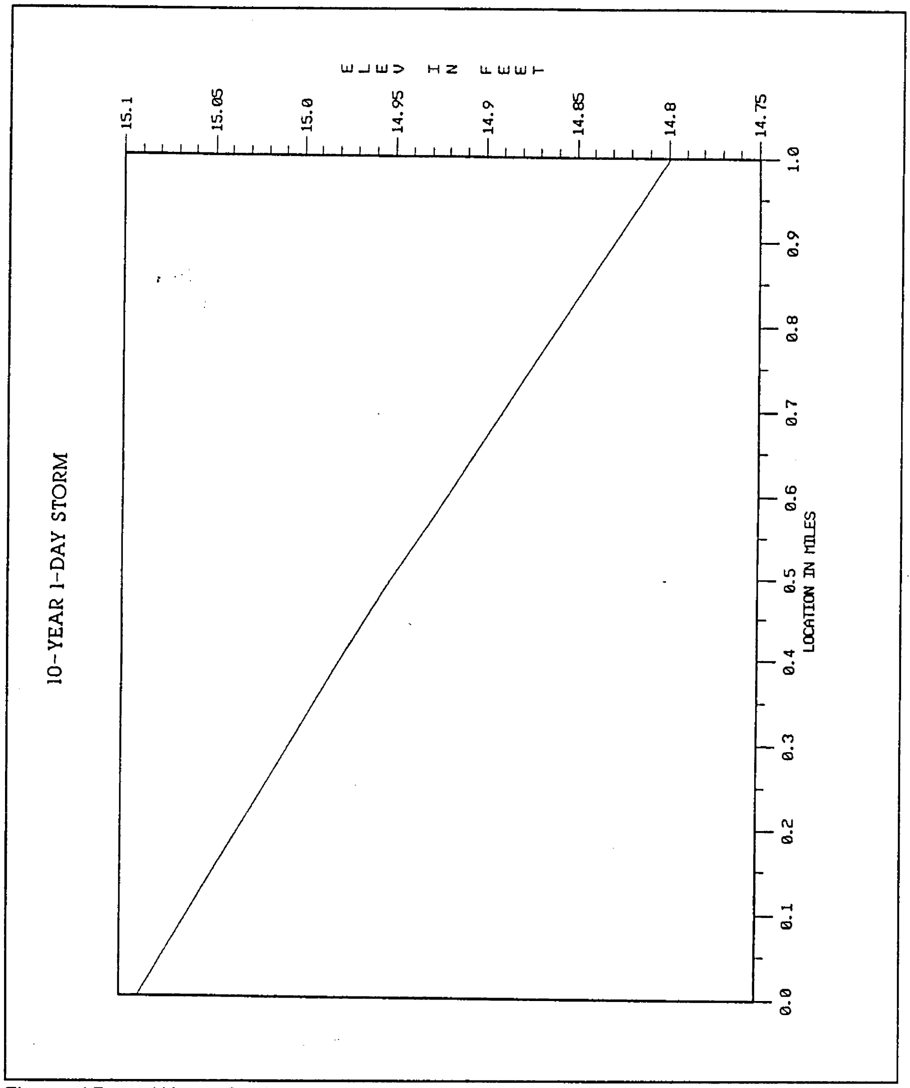

Figure 15. Water Surface Profile C-26 Between C-4 & C-6.

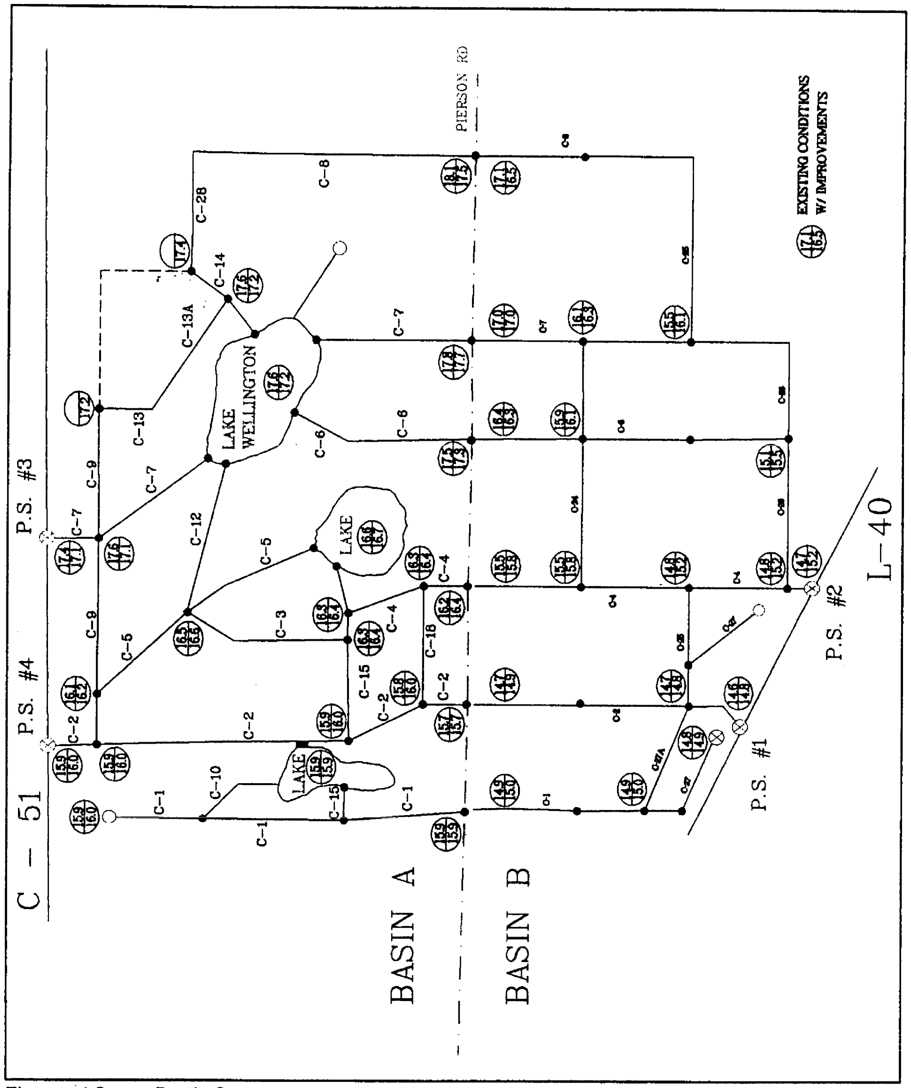

Figure 16. Peak Stages from the 10-Year, 1-Day Hypothetical Storm.

Table 6. Culverts with Excessive Head Loss.

Culvert Number	Location	Description	Head Loss (ft)	Discharge (cfs)
121	C-7 400' S. of C-24	48" CMP; 40'	0.5	50
50	C-7 at C-24	48" CMP; 40'	0.7	60
41	C-28 at C-8	48" CMPA (2); 30'	0.3	130

VI. Modeling of 25- and 100-Year Events

Simulation of the more extreme events (the three-day, 25-year and three-day, 100-year hypothetical storms) by UNET revealed that stages throughout Acme Improvement District exceeded top-of-bank elevations of the canals. The typical surveyed cross-section used in the model extended only to a local high point such as an adjacent road crown and thus did not fully capture the entire floodplain or overbank storage and conveyance capacity. Simulated stages are likely to be higher than actual since UNET restricts overbank storage and conveyance by extending vertically each end of the cross-section. A hydrologic routing model was developed for each basin in order to better estimate the peak stages resulting from the hypothetical storms. The modeling approach taken here assumes that there is significant overbank flow and that each basin acts as a large storage area (ie. the entire basin can be represented by one stage). The three necessary components for the hydrologic model are inflows, outflows, and a relationship between stage and basin storage. The inflows generated by HEC-1 were also used in these simulations; total present pumping capacity was used for each basin. Flows through the Pierson Road culverts as predicted by UNET were included as additional outflow for Basin A and inflow to Basin B. A mathematical relationship between land surface elevation and storage volume was developed for each basin using 1-foot topographic information shown in Figure 17. Table 9 reports the peak stage and number of days pumping at capacity by basin for the 25-year and 100-year hypothetical storm events.

Table 7. Summary of 25- and 100-Year Simulations.

	Basin A		Basin B	
	25-Year	100-Year	25-Year	100-Year
Peak stage, ft NGVD	17.1	17.4	15.8	16.1
Pumping at capacity, days	9	12	2¼	3½

VII. Pumping Station Improvements

A. Background

The four pumping stations now serving the District have been in operation since the mid-1950s and much of the pumping equipment has been in place for that length of time. Additional pumping equipment added through the years was generally reconditioned pumps rather than new equipment and the power unit serving those pumps are generally in need of replacement. The pump stations are all diesel driven, belt drive systems with line shaft driven impellers on the pumps. This type of equipment requires

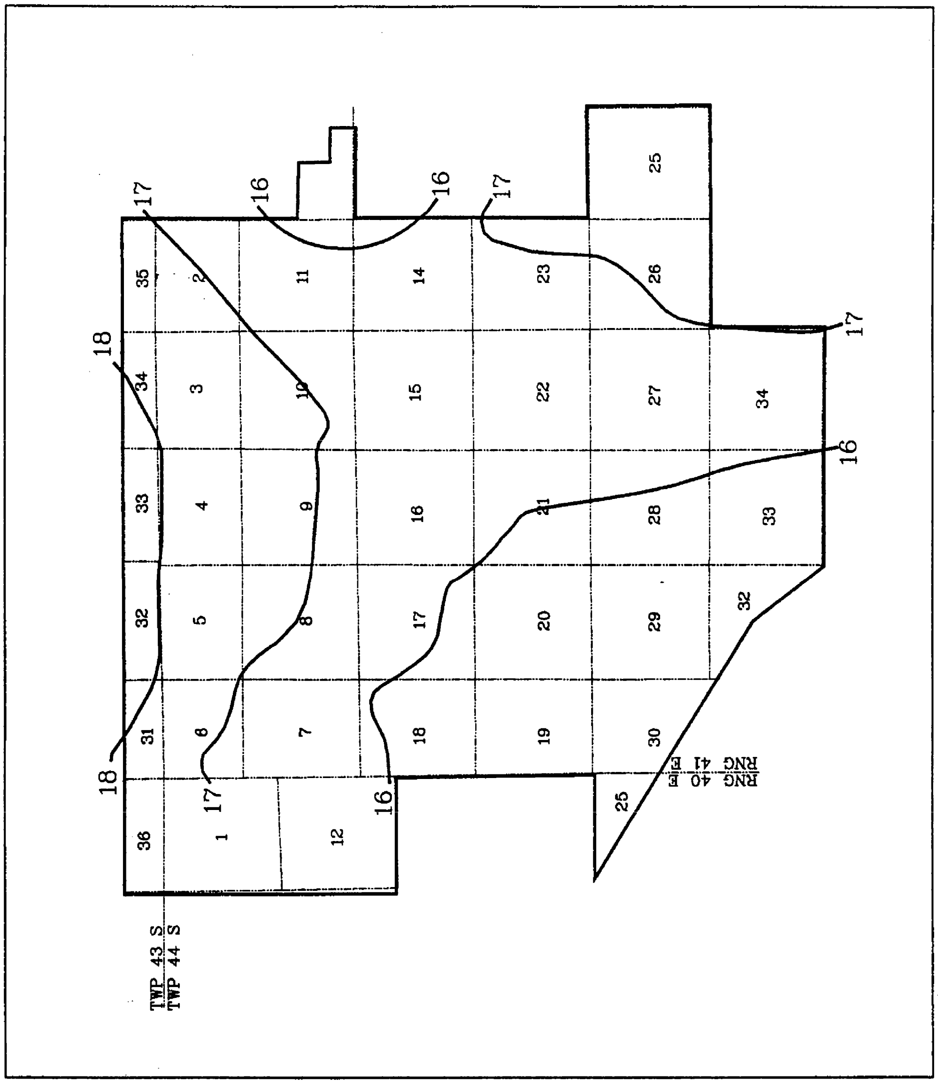

Figure 17. Acme Improvement District One Foot Contours.

high annual maintenance and high fuel cost for the diesel power units. Consideration has been given in the preparation of this report to alternatives for replacement of the existing power units and pumping system.

Consideration was also given in the model studies to alternative pumping capacity and locations and it was concluded that the four existing pumping station structures should be utilized and the equipment and buildings replaced. Standby capacity of Pumping Station No. 3 should be added to insure available pumping capacity. It is also recommended that the smaller pumps at Pumping Station No. 1, 3 and 4 be equipped as two-way pumps in order to be utilized both as irrigation and as a standby, low capacity, duty pump at each of the stations.

B. Alternatives Considered for Pumping Station Improvement

1. Replacement of Pumps with Line Shaft Pumps

Existing stations at the District are equipped with line shaft pumps. This type of pump has a vertical shaft driven impeller to which a gear or pulley is attached external to the pump for the purpose of attaching the power source to the pump shaft.

2. Gear Head Drives Versus Belt Drives

This is an option available to apply the power from the driving motor to the line shaft and would consist of either a gear head enclosed in a lubricated medium and would transfer the power by means of the gear to the shaft. The alternative would be grooved pulleys attached to the power head and reinforced rubber belts from a driving pulley at either the electric or diesel engine applying the power to the pump shaft.

3. Electric Versus Diesel Motor Power Sources

Electric motors could be utilized to drive the pumps either through a line shaft with a gear head or belt driven shaft. Electric motors could also be used in the submersible mode where the entire motor is enclosed in a waterproof container and submerged with a direct coupling to the impeller in the pump bowl. The use of electric motors has become extremely popular in high duty pumping systems due to the lower energy costs and lower maintenance costs. Approximately 98%-99% of the time electric power is dependable for this purpose and a backup diesel generator would be supplied in the event of power loss at a critical pumping period. Diesel power as is supplied now in all of the District stations is theoretically available 100% of the time, but historically has had down time due to mechanical failure of the engine or the driving units.

4. Remote Sensing and Automatic Controls

At the present time the District has no automatic pumping equipment at any of the stations. In order to save manpower, provide a better record of pumping and water level controls, it is recommended that remote sensing stations be located at key points in the system and that the remote sensing be tied by radio to a central control system at a command center in one of the permanent District buildings. The water levels indicated by the remote sensor would be fed to a central computer which would be used to start and stop the driving units at each of the four pumping stations. A program would be devised that meets the requirements of the water management system and the permitted operation. The remote sensing system would provide a permanent recorded record of water levels and pump operations available for downloading onto hard copy information for review by District staff and the SFWMD.

C. Alternative Pumping Station Costs

Based on the previous discussion, the following cost estimates for pump station improvements are a comparison of two basic alternatives. The first alternative consists of utilizing existing foundation structures and adding new line shaft, diesel drive pumps, similar to the existing facilities, but replacing the equipment and buildings and adding a remote sensing system for water level recording only. It may be possible to add remote starting for the diesel engines and contingency cost allowances are included for that option. The second alternative is for the use of electric submersible motors with emergency diesel generator for the main capacity of the station. Again, the existing foundations would be modified to accommodate the new equipment. The remote sensing under this alternative would be used to control the pump starting and stopping.

PUMP STATION NO. 1

Present capacity - 2-50,000 gpm, 1-50,000 gpm (standby), 1-25,000 gpm (standby), two-way pump.

Line shaft, diesel drive alternative - $527,000

Electric submersible motors with emergency diesel generator - $532,000

PUMP STATION NO. 2

Present capacity - 2-60,000 gpm, 1-50,000 gpm two-way pump.

Line shaft pump, diesel drive - $387,000

Electric submersible motors with emergency diesel generator - $452,000

PUMP STATION NO. 3

Present capacity - 1-60,000 gpm, 1-25,000 electric two-way.

Proposed - add 1-60,000 gpm (standby).

Line shaft diesel pumps - $377,000

Electric submersible motors with emergency diesel generator - $365,000

PUMP STATION NO. 4

Present capacity - 1-60,000 gpm, 1-60,000 gpm (standby), 1-25,000 gpm electric two-way.

Line shaft, diesel drive - $377,000

Electric submersible motors with emergency diesel generator - $365,000

Pump Station Totals -

Line shaft - diesel drive - $1,688,000

Electric submersible motors - $1,649,000

VIII. Estimated Cost of Improvements

Based on the results of the model analysis and the necessity for replacing pumping equipment, the following are the estimated costs of the various elements of the surface water management system:

A. Connection of C-9 to C-14: Excavation of new channel section and installation of 40' of 60" CMP pipe. Lump Sum $65,000.00

B. Culvert Improvements: The following culverts require replacement due to excessive head loss under design flow conditions:

Table 8. Proposed Culvert Improvements.

Culvert Number	Length, feet	Estimated Size	Cost
50	40	72" CMP	$ 15,000
121	40	72" CMP	15,000
69	40	72" CMP	15,000
Total			*$ 45,000*

C. Vertical Lift Gates on Pierson Road Culverts: The estimated cost of providing vertical lift gates at the Pierson Road culverts is $156,000.

D. Canal C-17 Internal Pump: The estimated cost of installing an electric operated lift pump with a float operated on-off switch is $30,000.

Table 9. Total Estimated Construction Costs.

Connection of C-9 to C-14	$ 65,000
Culvert Replacement	45,000
Vertical Lift Gates, Pierson Road	156,000
C-17 Canal Lift Pump	30,000
Pumping Stations (max)	1,649,000
Total	1,945,000
Contingencies, 15%	292,000
Engineering, permitting, project management, 15%	292,000
Total	*$ 2,529,000*

IX. Recommended Maintenance Practices and Programs

A. Canals

District operating personnel currently maintain rights-of-way and roadways necessary for the District's water management system. No changes would be necessary to the canal maintenance program under the revised water management plan since the present programs appear to be adequate and no major change to these facilities are contemplated by the plan.

B. Pumping Stations

With the recommended pumping station improvements, District operating staff would continue to operate the new equipment, perform routine maintenance and be on standby during emergency pumping situations to assure smooth operation of the system. Man-hours now utilized for maintaining diesel pumping units and repairing pumps would be reduced to a minimum with a new system and automatic operation using remote sensing will eliminate manual operation hours now incurred by District operating personnel.

X. Recommendations for Implementation and Financing of Improvements

The recommended improvements outlined in the master water management report are not considered to be costly yet probably would be a burden on a single year assessment to the taxpayers of the District. Therefore, the District could do one of the following:

1) Program the improvements over a three year period dividing the necessary funds in three equal assessments for the overall District. Design and permitting of the facilities could occur in the first year with the system improvements and perhaps one pump station that same year. The remaining three pump stations could be programmed in years two and three.

2) Bond financing in sufficient quantity to produce the required construction funds with the entire system being constructed in approximately 18 months.

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